INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CIB - W18

MEETING TWENTY - SEVEN

SYDNEY

AUSTRALIA

JULY 1994

Lehrstuhl für Ingenieurholzbau und Baukonstruktionen
Universität Karlsruhe
Germany
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MINUTES

1. CHAIRMAN'S INTRODUCTION

The Chairman **H J Blass** thanked **R H Leicester** for extending the invitation to CIB W18 to meet in Sydney. He also expressed appreciation for the comprehensive arrangements which had been made by **K Crews** and his team at the University of Technology, Sydney. He said that the good attendance at the meeting was pleasing, and it was especially rewarding to note that a number of delegates from the Southern Hemisphere had taken advantage of the venue to attend. He said it is hoped that this would encourage as many as possible to remain closely involved with CIB W18.

2. COOPERATION WITH OTHER ORGANISATIONS

a) CIB-W18B

R H Leicester reported that CIB-W18B activity had been limited, due to the usual difficulties over travel costs. However, there were groups of timber engineers in both West- and East- Africa, for example, who were interested in promulgating the use of timber in tropical structures. A timber conference is being planned in West Africa in 1995, and it may be possible for CIB-W18B to meet there. A short meeting of W18B would be held during PTEC 94, in Gold Coast, Queensland. The possibility of the group reconvening under another parent organisation, which might facilitate attendance, was still being considered.

b) ISO TC165

Members of ISO TC165 met in Quebec City, Canada on the 7th and 8th of April 1994. The meeting was attended by seventeen delegates, and there were guests from seven countries. The meeting heard reports from task groups on the performance requirements for glued laminated structures, and on characteristic values for sawn timber, these groups having met on the previous day. The

Committee also considered adoption with and without modification of a number of CEN documents.

Specifically, the Committee endorsed the recommendation of the task group on glued laminated structures to modify the delamination and glue line shear tests currently proposed (prEN 391 and prEN 392); the Committee requested that these modified EN documents and a document on the strength of dowels (prEN 409) should be sent for voting as ISO/DIS standards; it recommended that four CEN documents on mechanical joints should be sent for parallel voting (EN TC 124.112, 124.113, 124.114 & 124.115) and requested the addition of six new work items to its programme of work. It was reported that the new work items for the programme of work deal with test methods for joints under reverse cyclic loading, determination of characteristic strength properties of fasteners, strength grading and testing of softwood and hardwood poles, determination of characteristic values for poles, and the visual strength grading of sawn hardwood timber. The Committee received with thanks an informal invitation from the French National Advisory Committee to hold the next meeting of ISO TC165 in Paris, at the CTBA. This meeting has been scheduled tentatively for the Autumn of 1995.

c) **RILEM**

A Ceccotti reported on the timber activities of RILEM as follows:

In comparison with some fifty committees dealing with steel, concrete and masonry subjects, there were relatively few RILEM groups on timber. RILEM Committees were organised by groups under broad subject areas. Each of the groups, of which there are seven, have a RILEM TC Counsellor; an up-to-date list was provided to CIB W18 Secretariat. The three RILEM TC's dealing with timber were described thus: TC 133 -TF Fracture of timber, led by **A Ranta-Maunus** was actively engaged in producing a State of the Art report on the subject, which it hopes to have complete by 1995. TC 149 - HTS, coordinated by **W Rug** and dealing with Diagnosis and Repair of Historic Load-Bearing Timber Structures has only recently been set-up, and does not yet have a report. A similar situation exists with TC 155 - TCW Test Methods for Creep Measures of Wooden Materials, under **C Le Govic. A Ceccotti** urged CIB W18 delegates to consider making fuller use of the RILEM framework for activity in the timber field. In answer to a question from **R H Leicester**, it was confirmed that there is no restriction on the meeting venues of RILEM TC's.

d) **CEN**

H J Larsen gave a report on activities of the CEN TC's concerning European timber codes and standards. CIB W18 has close links with two CEN groups, namely TC 250 and TC 124. The former was concerned with Eurocodes. The position with these is as follows:

ENV 1995 - 1 - 1 Eurocode 5 Part 1 General Rules is in the final stages of publication, together with the respective National Application Documents in each of the EU member countries. It is now available in English, French and German. Part 1 - 2, dealing with fire, is also at an advanced stage, and may be available by the end of the year. Work has recently been started on Part 2, Timber Bridges. It is hoped that a draft for discussion will be available within approximately one year. TC 124 had a large work programme, much of which is now coming to fruition. Thirty four standards have reached the final stages of drafting/voting, and several more are actually published by national standards organisations. Only two standards are awaited. One covering test methods to obtain seismic data has been progressing well, after starting later than the bulk of the programme. The other, dealing with glued-in bolts and rods, has been unable to progress due to lack of convenership. A recent solution may have been found, if an application under the EC Framework programme is successful. It was noted that there are also certain TC 112 activities which are of interest to CIB W18 members, particularly those dealing with structural panel products.

e) **IABSE**

B Edlund had submitted a report, being unable to attend this meeting of CIB W18 in person. The report was presented by **S Thelandersson.** A Colloquium on Expert Systems in building had taken place in Beijing in May 1993, but there had only been one contribution covering timber applications. At the Colloquium on Structural Serviceability of Buildings in Göteborg, Sweden, June 1993, there had been three contributions on vibrations of wood-based floors and one on creep effects in timber structures.

At the IABSE Symposium in Rome in September 1993, on Structural Preservation of the Architectural Heritage, there had only been a couple of posters relating to timber subjects. These were interesting presentations on timber in Chinese temples.

There had, however, been an improvement in the number of timber papers being published in the SEI Journal. The special issue SEI 2/93 contained a focus on timber, with eleven articles, including a number by CIB W18 members as individuals. Each of the issues, which followed this special edition, have also carried a timber article. It is hoped that CIB W18 delegates will continue to support this action. Issue SEI 3/94, which will appear in August, will focus on shell and spatial structures.

Mentioning future events, the Symposium on "Places of Assembly and Long-Span Building Structures", scheduled to take place in Birmingham in September 1994, only gives a small mention to timber. Another Colloquium on "Knowledge-Based Systems in Civil Engineering" is to take place in Bergamo from 15th-17th March 1995. In August 1995 there will be a Symposium on "Extending the Lifespan of Structures", to take place in San Francisco.

For the IABSE's 15th Congress in Copenhagen, 15th-20th June 1996, the theme is to be "Structural Engineering in Consideration of Economy, Environment and Energy". Amongst the planned workshops, it is intended to include structural connections and "Structural Eurocodes - First Experiences". **B Edlund** requests especially that CIB W18 delegates give thought as to how to suggest coverage of EC5 in this respect, letting him know their views through the CIB W18 Chairman. He comments that engineers concerned with timber are missing opportunities to show structural engineers, contractors and building authorities worldwide that timber is a realistic alternative, especially for long spans, and it is important not to miss such future opportunities.

In 1997, a Colloquium focusing on timber structures is planned. The venue is likely to be Göteborg, Sweden. **S Thelandersson** sought the views of CIB W18 delegates regarding a suitable month. He suggested May or June 1997. He also said that the following five topics have been proposed:

1. General aspects of timber structures
2. Multi-storey dwellings in timber
3. Timber bridges
4. Aspects such as vibrations, deflections and fire protection
5. Durability and maintenance of timber structures

There was some discussion of the profusion of general timber conferences, such as those planned for New Orleans 1996 and Helsinki 1998. However, it was reiterated that IABSE is an important general forum for structural and civil engineers, rather than one specialising in timber. **G Johanssen** not only spoke in favour of this broader dissemination, but also said that he wished that timber researchers would try to publish more widely.

f) **IUFRO S5.02**

Although a number of the delegates had also been present at the IUFRO S5.02 meeting, which had taken place earlier in the week in Sydney, there was a brief report on the interesting series of papers and discussions which had taken place. A series of overheads had been collated, covering the current activities of the organisations which were represented, in the structural timber field. Copies are held by **P Hoffmeyer** and the CIB W18 Secretariat, and small numbers may be provided on request.

IUFRO S5.02 is to be represented at the IUFRO World Congress in Tampere, Finland, 6th-12th August 1995. The next ordinary meeting will be held in Copenhagen or Lund in 1996.

g) **CIB-W85 STRUCTURAL SERVICEABILITY**

The joint CIB/IABSE Colloquium on Structural Serviceability of Buildings, held in Göteborg, Sweden, in June 1993, was reported in last year's CIB W18 Minutes. The Colloquium Report is now published in the series IABSE Reports Vol. 69, 1993. The next meeting of W85 is scheduled for 24th September in Sydney, Australia.

h) **COST**

An additional agenda item was a report by **F Rouger** on activities in the European Programme COST, which is aimed at exchanges of ideas and personnel amongst European Research Organisations and Universities. COST is organised in six groups, three covering each of steel, concrete and timber and three dealing with numerical methods, seismic design and databases. These groupings are intended to operate in a cross-linked manner. Timber research items being examined include reinforced joints, truss plate modelling, dowelled connections and fracture mechanics. The programme involves eighteen countries and sixty laboratories. A forthcoming meeting on the mechanics of wood-based panel products is being organised by **J Dinwoodie** of BRE, UK. In general, it is possible for COST 508 "Wood Mechanics" meetings to include invited papers and participants from beyond the European Union.

i) **OTHER ORGANIZATIONS**

Under this agenda item, **R H Leicester** raised the point that many meetings which take place beyond Europe and which deal with structural timber topics are not normally considered by CIB W18. He suggested that even if CIB W18 does not formally seek to report and minute these, an annual list could be collated and circulated. This was discussed, and in principle agreed. Since many such meetings take place in North America, **T Williamson** was asked, and consented, to attempt to compile such a list.

3. SERVICEABILITY CONSIDERATIONS IN STRUCTURAL TIMBER DESIGN

27-20-1 Codification of Serviceability Criteria - R H Leicester:

This paper presented a statistical model for the codification of serviceability criteria, for both design codes and performance standards. The model which was proposed is based on optimising cost functions. A key consideration is the complex nature of human response to structural unserviceability. This is highly variable, and is affected by many non-structural parameters. General matters related to codification were also discussed in the paper. It was shown that there is a strong case that codes and standards should specify multiple serviceability limits, rather than following the present custom of recommending single values.

In the discussions which followed, **H J Larsen** commented on the frequency curves shown in Figure 1 of the paper, to illustrate unserviceability parameters. He said that these were a neat way of expressing the problem, however he doubted whether the author should have shown them without indicating that they are general trend sketches, since they infer a degree of scientific precision that is not present in the case of unserviceability problems with existing buildings. The author agreed, saying that the curves and the related cost models were a rational attempt to address such uncertainties. **S Thelandersson** said that the multiple serviceability limit suggestion had its merits, since although structural designers should not be encouraged to believe that there are hard and fast rules for serviceability, they do nevertheless require some guidance. There were already several serviceability concepts in EC5, including those relating to permanent damage, and those concerned with general user dissatisfaction.

27-20-2 on the Experimental Determination of Factor K_{def} and Slip Modulus K_{ser} from Short- and Long-term Tests on a Timber/Concrete (TCC) Beam - S Capretti and A Cecotti:

This paper described tests of three years duration on TCC beams using the k_{def} factor of EC5 as a means of analysis. The experimental deformation-time curves combined with moisture and shrinkage/swelling measurements, showed a strong influence of hygro-mechanical effects, due to the differential shrinkage between timber and concrete. Results from monitoring on a real structure incorporating TCC beams were also reported.

L R J Whale asked the author for a comparison between the design predicted final deflections and the monitored deflections. The author clarified this. **A Leijten** commented on the lack of information for the period immediately following construction. There was a gap between this and the time at which the monitoring was installed. The author felt that although installation immediately after completion would have been desirable, nevertheless the monitoring showed that the serviceability performance had been satisfactory. **J Ehlbeck** and **H J Larsen** became involved in a discussion with the author over the comparison between EC5 k_{def} values and those reported in the paper. It was agreed that the analysis given in the paper for TCC beams was important and useful for this form of construction, but that it did not necessarily have any bearing upon the appropriateness or otherwise of k_{def} values for the purpose intended by the code.

27-20-3 Serviceability Limit States, "A proposal for updating Eurocode 5 with respect to Eurocode 1 - P Racher and F Rouger:

This paper proposed a means of modifying EC5 with respect to load combination factors relating to the serviceability limit state, in order to obtain closer compatibility with other Eurocodes. It compared the application of EC5 principles and the alternative methods in relation to a composite I-beam example. During the discussions, **H J Larsen** and **S Thelandersson** mentioned several reasons why an EC5 approach to serviceability more closely related to those used in other Eurocodes would hypothetically be desirable, if the timber code could be redrafted whilst retaining economy of design. It was felt, however, that a number of the detailed points raised in this paper require further study before changes could be argued.

27-20-4 Influence of the Mechano-Sorptive Effect on the Creep Behaviour of Timber - T Toratti, F Rouger and P Morlier:

This paper was a successor to work published in the 1990 CIB W18 proceedings dealing with full-scale tests to provide a basis for k_{mod} and k_{def} factors for EC5. The earlier paper showed how data corresponding to Service Classes 1 and 2 could be fitted by means of a power law, whilst the present paper addressed a similar subject following long-term tests under Service Class 3 conditions.

Full-scale creep and duration of load experiments had been conducted on solid softwood timber and on glulam. Eight hundred days of data had been collected, including relative humidity and temperature information. Finite difference methods had been used to model transient moisture content distributions, and successive appropriation methods to represent creep behaviour. The authors found that shrinkage strains were greater in the experimental data than suggested by their model. They also expressed doubt over the suitability of the model for low quality solid timber. In response to a question from **E Karacabeyli, F Rouger** gave further details on how the short-term strength of the tested members was established.

4. TIMBER BRIDGES

27-12-1 State of the Art Report Glulam Timber Bridge Design in the US - M A Ritter and T G Williamson:

The State of the Art Report emphasised recent developments in highway bridge applications in the USA, which have included new glulam timber species, particularly native hardwoods, and new designs using stress-laminated glulam decks. The use of T-beam and box-beam sections has become well established. Timber guard rail designs have been proven by means of crash testing programmes. Emerging technologies include combinations of timber composites together with synthetic composites. Currently about three hundred timber road bridges are being built each year in the USA.

There was considerable interest in this timber road bridge report. Questions included that asked by **H J Blass** concerning the frequency of re-tightening of cross-deck tensioning rods. **T Williamson** said that this was done routinely a few weeks after a bridge erection had been completed. It normally involved compensating for about a ten percent stress relaxation. Thereafter, the rods were checked during routine maintenance. However, when glulam or microlam decks were used, little stress relaxation was normally observed. **S Thelandersson** asked about preservation methods. The author explained that pentachlorophenol or creosote treatment applied under pressure were normal in the USA and that suitable environmental and operator health precautions at treaters' works were obligatory. **C J Mettem** asked about stress grading procedures for hardwood laminations and it was explained that these were usually based on output controlled machine grading principles.

27-12-2 A Review of the Design of Timber Footbridges in the UK - C J Mettem and J P Marcroft:

The paper explained that although there had been a well established history of timber bridge use in the UK, these had not been well supported by national standards in more modern times. In particular, the absence of a timber bridge section in BS5400 was a disincentive to designers. Recently, however, two positive factors have emerged. The DoE was supporting a project relating to timber bridges on a national basis, and a Part 2 dealing with timber bridges had been commenced for EC5. Work had been started on categories of timber bridges, structural forms, dimensional standards, load requirements and a survey made of existing bridges with, generally, encouraging results.

There were no questions of fact in the discussion period relating to this paper. However, **E Aasheim** made mention of similar work which had been started by a Nordic timber bridge group. Subsequently, it was agreed that information exchange arrangements would be attempted, and that **T Williamson** or a US colleague would circulate all CIB delegates interested in the subject.

27-07-1 Glulam Arch Bridge and Design of Moment Resisting Joints - K Komatsu and S Usuki:

A two-hinged glulam arch bridge capable of carrying a twenty tonne vehicle and spanning 23m was completed in Japan in 1994. Some structural members were laminated from Japanese Sugi timber. The arch members were protected by copper sheeting cover plates. In addition to providing a general description of the design, the paper discussed the design of moment-resisting joints which were incorporated in the arch members. Tests were also conducted on a half scale model of these joints.

In answer to the question from **S Thelandersson, K Komatsu** explained that the two-pinned arch form had been chosen so that the arches could be more easily sub-divided for transport to site. Some details of the stress grading machine which was used for the Sugi laminations were provided, in answer to a question from **A R Fewell.**

5. MECHANICAL TIMBER JOINTS

27-07-2 Characteristic Load-Carrying Capacity of Joints with Dowel type Fasteners in Regard to the System Properties - H Werner:

The characteristic load-carrying capacities of various dowelled joints with a single fastener in lateral shear were calculated and compared with test results. A calculation model and a computer programme were used. This was intended to take account of non-linear embedding stress-strain behaviour; the actual yield movement of the metallic fasteners, and "chain effects" plus splitting effects in the timber. However, the theoretical basis for these two latter variables was not presented within the paper. The paper suggested that if the present theoretical basis of EC5 ENV is to be retained in future editions of the code, then a factor k_{sys} may be required for joints, which will vary according to the nature of the members being fastened (timber - timber, timber - steel) as well as the nature of the fastener surface.

Having received an advanced draft of paper 27-07-2, **E Gehri** had written to the meeting organisers with a supplementary paper commenting on this work. This had not been received in sufficient time for listing on the agenda, although a copy was made available to delegates. **E Gehri** pointed out that if simulation studies are made with yield moment values for dowels based on actual measured properties, then there should be an adjustment to certain expressions in the yield moment equations of the code, in order to provide a valid comparison. He also pointed out that Eurocode 5 should provide better guidance for situations where there are a number of dowels in line with load direction.

H J Larsen reiterated the comment of **E Gehri** regarding the yield strength comparisons, and misleading conclusions concerning k_{sys}. **H Petersson** questioned how the splitting effect had been dealt with. He referred to a paper presented at Meeting Twenty-Five, which would provide a fracture mechanics basis for the analysis of splitting effects in dowelled joints.

27-07-3 Steel Failure Design in Truss Plate Joints - T Poutanen:

This paper proposed a design method for that part of the entire truss plated joint design process which involves the check against the possibility of steel failure along the joint lines of the connected truss members. It was explained that each joint line is treated by simulation, using a series of linear springs. The internal loads borne by the joint are distributed along the joint line under the assumption that the line has a stiffness related to the measured test strength of the plate. The author regarded the proposals for this aspect of truss plate joint design as being superior to the recommendations of ENV EC5. The reasons for this were discussed in the paper, which also explained the necessary general assumptions regarding truss plate design, under which the method was proposed to operate.

L R J Whale welcomed the fact that more thought was being given to the subject, since he felt that this is a difficult and not fully resolved aspect of truss design. He asked the author how he would consider distributing the forces of the joint along the joint lines under the plate, if the resultant was at a varied angle to several lines within the same plate. The author explained that an iterative process is necessary in such a case, but that this is performed by computer program, and does not normally require an unacceptable number of iterations.

There was some discussion of the author's rejection of the EC5 principle of permitting full plasticity to have occurred along the joint line. **H J Larsen** contended that an estimate on the safe side would be obtained from an analysis permitting allowance for plastic strength, based on the assumption that equilibrium had been regained before yield. He also asked whether the author's alternative method had been evaluated by test. The author disagreed with **H J Larsen** with regard to the safe assumption which the questioner suggested. He said that he might agree, if the joint had adequate deformation capacity, but under some circumstances such truss plate joint lines could exhibit brittle failure. **G Boughton** agreed with this, saying that tests which he had performed on some Australian truss plate designs had shown brittle failure. The author pointed out that the truss plates only represent roughly ten percent of the cost of a truss, yet most of these components fail under test at such joints. He felt, therefore, that more research efforts should be concentrated on the design of truss connections.

6. OPEN FORUM

27-06-1 Development of the "Critical Bearing" Design Clause in CSA 086.1 - C Lum and E Karacabeyli:

This paper described a programme of work which had been undertaken to rectify inconsistencies in design procedures for compression perpendicular grain, when following the recommendations of CSA 086.1 Short-term loading tests, and tests of two months duration had been undertaken on a selection of materials of interest to Canadian designers. The test results were compared with finite element analysis of situations relevant in design, and proposed changes to the code were developed. These included new rules for cases where compression perpendicular to the grain is checked near positions of support in flexural members.

S Thelandersson asked whether alternative failure criteria had been considered in developing the proposed new rules. For example, exceeding compression perpendicular to the grain limits could lead either to failing serviceability limit states, or failing ultimate limit states. **C Lum** said that both conditions had been looked at, and it was recognised that in the case of a beam supporting a column, for example, excessive defection due to compression perpendicular to grain could lead to instability. In answer to a question from **K Komatsu**, it was stated that the tests had used material with annual rings orientated at random within the cross-section. **H Petersson** commented that if microscopical studies were to reveal that localised failures occurred at an early stage close to the support plates, then three-dimensional finite element analysis would be necessary to model this properly.

27-10-1 Determination of Shear Modulus - R Görlacher and J Kürth:

In prEN 338 and prEN 1194, the ratio of $E_{0,mean}$ to $G_{0,mean}$ is assumed to be constant, for all strength classes, with a value of 16. Using a test method stipulated in prEN 408, together with other tests involving longitudinal and torsional vibrations, this assumption was checked, for samples taken from industrial laminations selected from German glulam factories. It was found that in fact the ratio tends to increase, for material of a higher modulus of elasticity, with values in the order of $E_0/G_0 = 21$, for strength class C40.

There was some discussion of the simplifying assumptions which are required in the theories for this type of work, if precise wood orthotropicity cannot be assumed, either in test or in practical design. It was not felt that the differences which had been noted in the ratio were such that a single value would lead to excessively incorrect designs. Hence, there was no incentive to propose changes to the prEN's.

27-17-1 Statistical Control of Timber Strength - R H Leicester and H D Breitinger:

Various forms of quality control checks can be considered, in order to ensure that a particular type of structural timber continues to be produced in accordance with expectations. These range from the most expensive - a complete in-grade evaluation of all significant structural properties for all grades and sizes - down to random or daily checks on small samples. This paper described the evaluation of several alternative strategies, using Monte Carlo simulation techniques and examining cost/risk benefits to both producers and consumers. Discussion of the paper led to comments on the need to develop a better understanding of how the strength of a species or species group from any particular wide-ranging geographic region might change over periods of time. This links with the continuing need to keep practical sampling procedures under review.

27-06-2 Size Effects in Timber - F Rouger and A R Fewell:

Size effects were again examined, based upon the results of tests on French-grown softwood which had been selected by means of a forest-related sampling scheme (not explained within the paper). It was stated that it seemed from the data that basic assumptions of Weibull theory (brittle failure, statistical homogeneity) were not fulfilled. Size effects seemed to be significantly influenced by aspects such as sawing patterns, log diameter and visual grading method. **H J Larsen** commented that whilst the test results spoke for themselves, he doubted the validity of the interpretation of Weibull theory given in the paper. For simple tensile chain "weak link" theory to be applicable, it does not matter whether failure is brittle or ductile, he said. **T Poutanen** commented that in order to pursue such possible explanations of size effect in relation to the material investigated, it would be helpful if the researchers could examine test results from specimens loaded axially, in tension, and short-column compression, as opposed to analyses based purely on bending tests.

27-06-3 Comparison of Full-size Sugi Structural Performance in Bending of Round Timber, Two Surfaces Sawn Timber and Square Sawn Timber - T Nakai.

This paper described experimental and analytical work on structural members cut from logs taken from 50-60 year old Sugi trees, which had been grown under plantation conditions in Japan. The result of bending tests were reported, and the effects of cross-sectional shape on bending performance were discussed. There were no questions on the information provided by the author in his clear presentation.

7. ANY OTHER BUSINESS

There was none.

8. VENUE AND PROGRAMME FOR NEXT MEETINGS

The Chairman announced that the 28th meeting will be held in Copenhagen, from 18th-21st April 1995. **H-J Larsen** of the Danish Building Research Institute will be the host. The meeting will commence at 13:30 hrs, with a reception taking place the same evening.

The proposal for 1996 is still to hold a meeting in Bordeaux, France at the end of August or early September, as reported last year.

9. CLOSE

R H Leicester expressed appreciation for the good attendance from Europe and North America, at this CIB meeting in Australia. **H J Blass** thanked everyone for the active interest in the papers, as evidenced by the lively discussions. He also thanked **K Crews** and his team for the well organized meeting, at the University of Technology, Sydney, and for the interesting tour of the laboratories and other related visits.

10. List of CIB-W18 Papers, Sydney, Australia 1994

LIST OF CIB-W18 PAPERS, Sydney, Australia 1994

27-6-1	Development of the "Critical Bearing": Design Clause in CSA-086.1 - C Lum and E Karacabeyli
27-6-2	Size Effects in Timber: Novelty Never Ends - F Rouger and T Fewell
27-6-3	Comparison of Full-Size Sugi (Cryptomeria japonica D.Don) Structural Performance in Bending of Round Timber, Two Surfaces Sawn Timber and Square Sawn Timber - T Nakai, H Nagao and T Tanaka
27-7-1	Glulam Arch Bridge and Design of it's Moment-Resisting Joints - K Komatsu and S Usuku
27-7-2	Characteristic Load - Carrying Capacity of Joints with Dowel - type Fasteners in Regard to the System Properties - H Werner
27-7-3	Steel Failure Design in Truss Plate Joints - T Poutanen
27-10-1	Determination of Shear Modulus - R Görlacher and J Kürth
27-12-1	State of the Art Report: Glulam Timber Bridge Design in the U.S. - M A Ritter and T G Williamson
27-12-2	Common Design Practice for Timber Bridges in the United Kingdom - C J Mettem, J P Marcroft and G Davis
27-12-3	Influence of Weak Zones on Stress Distribution in Glulam Beams - E Serrano and H J Larsen
27-17-1	Statistical Control of Timber Strength - R H Leicester and H O Breitinger
27-20-1	Codification of Serviceability Criteria - R H Leicester
27-20-2	On the Experimental Determination of Factor k_{def} and Slip Modulus k_{ser} from Short- and Long-Term Tests on a Timber-Concrete Composite (TCC) Beam - S Capretti and A Ceccotti
27-20-3	Serviceability Limit States: A Proposal for Updating Eurocode 5 with Respect to Eurocode 1 - P Racher and F Rouger
27-20-4	Creep Behavior of Timber under External Conditions - C Le Govic, F Rouger, T Toratti and P Morlier

11. Current List of CIB-W18(A) Papers

CURRENT LIST OF CIB-W18(A) PAPERS

Technical papers presented to CIB-W18(A) are identified by a code CIB-W18(A)/a-b-c, where:

a denotes the meeting at which the paper was presented. Meetings are classified in chronological order:

1 Princes Risborough, England; March 1973
2 Copenhagen, Denmark; October 1973
3 Delft, Netherlands; June 1974
4 Paris, France; February 1975
5 Karlsruhe, Federal Republic of Germany; October 1975
6 Aalborg, Denmark; June 1976
7 Stockholm, Sweden; February/March 1977
8 Brussels, Belgium; October 1977
9 Perth, Scotland; June 1978
10 Vancouver, Canada; August 1978
11 Vienna, Austria; March 1979
12 Bordeaux, France; October 1979
13 Otaniemi, Finland; June 1980
14 Warsaw, Poland; May 1981
15 Karlsruhe, Federal Republic of Germany; June 1982
16 Lillehammer, Norway; May/June 1983
17 Rapperswil, Switzerland; May 1984
18 Beit Oren, Israel; June 1985
19 Florence, Italy; September 1986
20 Dublin, Ireland; September 1987
21 Parksville, Canada; September 1988
22 Berlin, German Democratic Republic; September 1989
23 Lisbon, Portugal; September 1990
24 Oxford, United Kingdom; September 1991
25 Åhus, Sweden; August 1992
26 Athens, USA; August 1993
27 Sydney, Australia; July 1994

b denotes the subject:

1 Limit State Design
2 Timber Columns
3 Symbols
4 Plywood
5 Stress Grading
6 Stresses for Solid Timber
7 Timber Joints and Fasteners
8 Load Sharing
9 Duration of Load
10 Timber Beams
11 Environmental Conditions
12 Laminated Members
13 Particle and Fibre Building Boards
14 Trussed Rafters
15 Structural Stability
16 Fire
17 Statistics and Data Analysis
18 Glued Joints
19 Fracture Mechanics
20 Serviceability
100 CIB Timber Code
101 Loading Codes
102 Structural Design Codes
103 International Standards Organisation
104 Joint Committee on Structural Safety
105 CIB Programme, Policy and Meetings
106 International Union of Forestry Research Organisations

c is simply a number given to the papers in the order in which they appear:

Example: CIB-W18/4-102-5 refers to paper 5 on subject 102 presented at the fourth meeting of W18.

Listed below, by subjects, are all papers that have to date been presented to W18. When appropriate some papers are listed under more than one subject heading.

LIMIT STATE DESIGN

1-1-1	Limit State Design - H J Larsen
1-1-2	The Use of Partial Safety Factors in the New Norwegian Design Code for Timber Structures - O Brynildsen
1-1-3	Swedish Code Revision Concerning Timber Structures - B Noren
1-1-4	Working Stresses Report to British Standards Institution Committee BLCP/17/2
6-1-1	On the Application of the Uncertainty Theoretical Methods for the Definition of the Fundamental Concepts of Structural Safety - K Skov and O Ditlevsen
11-1-1	Safety Design of Timber Structures - H J Larsen
18-1-1	Notes on the Development of a UK Limit States Design Code for Timber - A R Fewell and C B Pierce
18-1-2	Eurocode 5, Timber Structures - H J Larsen
19-1-1	Duration of Load Effects and Reliability Based Design (Single Member) - R O Foschi and Z C Yao
21-102-1	Research Activities Towards a New GDR Timber Design Code Based on Limit States Design - W Rug and M Badstube
22-1-1	Reliability-Theoretical Investigation into Timber Components Proposal for a Supplement of the Design Concept - M Badstube, W Rug and R Plessow
23-1-1	Some Remarks about the Safety of Timber Structures - J Kuipers
23-1-2	Reliability of Wood Structural Elements: A Probabilistic Method to Eurocode 5 Calibration - F Rouger, N Lheritier, P Racher and M Fogli

TIMBER COLUMNS

2-2-1	The Design of Solid Timber Columns - H J Larsen
3-2-1	The Design of Built-Up Timber Columns - H J Larsen
4-2-1	Tests with Centrally Loaded Timber Columns - H J Larsen and S S Pedersen
4-2-2	Lateral-Torsional Buckling of Eccentrically Loaded Timber Columns - B Johansson
5-9-1	Strength of a Wood Column in Combined Compression and Bending with Respect to Creep - B Källsner and B Norén
5-100-1	Design of Solid Timber Columns (First Draft) - H J Larsen
6-100-1	Comments on Document 5-100-1, Design of Solid Timber Columns - H J Larsen and E Theilgaard
6-2-1	Lattice Columns - H J Larsen
6-2-2	A Mathematical Basis for Design Aids for Timber Columns - H J Burgess
6-2-3	Comparison of Larsen and Perry Formulas for Solid Timber Columns - H J Burgess
7-2-1	Lateral Bracing of Timber Struts - J A Simon
8-15-1	Laterally Loaded Timber Columns: Tests and Theory - H J Larsen
17-2-1	Model for Timber Strength under Axial Load and Moment - T Poutanen
18-2-1	Column Design Methods for Timber Engineering - A H Buchanan, K C Johns, B Madsen
19-2-1	Creep Buckling Strength of Timber Beams and Columns - R H Leicester
19-12-2	Strength Model for Glulam Columns - H J Blaß

SYMBOLS

PLYWOOD

STRESS GRADING

26-5-1	Structural Properties of French Grown Timber According to Various Grading Methods - F Rouger, C De Lafond and A El Quadrani

STRESSES FOR SOLID TIMBER

4-6-1	Derivation of Grade Stresses for Timber in the UK - W T Curry
5-6-1	Standard Methods of Test for Determining some Physical and Mechanical Properties of Timber in Structural Sizes - W T Curry
5-6-2	The Description of Timber Strength Data - J R Tory
5-6-3	Stresses for EC1 and EC2 Stress Grades - J R Tory
6-6-1	Standard Methods of Test for the Determination of some Physical and Mechanical Properties of Timber in Structural Sizes (third draft) - W T Curry
7-6-1	Strength and Long-term Behaviour of Lumber and Glued Laminated Timber under Torsion Loads - K Möhler
9-6-1	Classification of Structural Timber - H J Larsen
9-6-2	Code Rules for Tension Perpendicular to Grain - H J Larsen
9-6-3	Tension at an Angle to the Grain - K Möhler
9-6-4	Consideration of Combined Stresses for Lumber and Glued Laminated Timber - K Möhler
11-6-1	Evaluation of Lumber Properties in the United States - W L Galligan and J H Haskell
11-6-2	Stresses Perpendicular to Grain - K Möhler
11-6-3	Consideration of Combined Stresses for Lumber and Glued Laminated Timber (addition to Paper CIB-W18/9-6-4) - K Möhler
12-6-1	Strength Classifications for Timber Engineering Codes - R H Leicester and W G Keating

TIMBER JOINTS AND FASTENERS

5-7-1	Influence of Loading Procedure on Strength and Slip-Behaviour in Testing Timber Joints - K Möhler
5-7-2	Recommendations for Testing Methods for Joints with Mechanical Fasteners and Connectors in Load-Bearing Timber Structures - RILEM 3 TT Committee
5-7-3	CIB-Recommendations for the Evaluation of Results of Tests on Joints with Mechanical Fasteners and Connectors used in Load-Bearing Timber Structures - J Kuipers
6-7-1	Recommendations for Testing Methods for Joints with Mechanical Fasteners and Connectors in Load-Bearing Timber Structures (seventh draft) - RILEM 3 TT Committee
6-7-2	Proposal for Testing Integral Nail Plates as Timber Joints - K Möhler
6-7-3	Rules for Evaluation of Values of Strength and Deformation from Test Results - Mechanical Timber Joints - M Johansen, J Kuipers, B Norén
6-7-4	Comments to Rules for Testing Timber Joints and Derivation of Characteristic Values for Rigidity and Strength - B Norén
7-7-1	Testing of Integral Nail Plates as Timber Joints - K Möhler
7-7-2	Long Duration Tests on Timber Joints - J Kuipers
7-7-3	Tests with Mechanically Jointed Beams with a Varying Spacing of Fasteners - K Möhler
7-100-1	CIB-Timber Code Chapter 5.3 Mechanical Fasteners;CIB-Timber Standard 06 and 07 - H J Larsen
9-7-1	Design of Truss Plate Joints - F J Keenan
9-7-2	Staples - K Möhler
11-7-1	A Draft Proposal for International Standard: ISO Document ISO/TC 165N 38E
12-7-1	Load-Carrying Capacity and Deformation Characteristics of Nailed Joints - J Ehlbeck

12-7-2	Design of Bolted Joints - H J Larsen
12-7-3	Design of Joints with Nail Plates - B Norén
13-7-1	Polish Standard BN-80/7159-04: Parts 00-01-02-03-04-05. "Structures from Wood and Wood-based Materials. Methods of Test and Strength Criteria for Joints with Mechanical Fasteners"
13-7-2	Investigation of the Effect of Number of Nails in a Joint on its Load Carrying Ability - W Nozynski
13-7-3	International Acceptance of Manufacture, Marking and Control of Finger-jointed Structural Timber - B Norén
13-7-4	Design of Joints with Nail Plates - Calculation of Slip - B Norén
13-7-5	Design of Joints with Nail Plates - The Heel Joint - B Källsner
13-7-6	Nail Deflection Data for Design - H J Burgess
13-7-7	Test on Bolted Joints - P Vermeyden
13-7-8	Comments to paper CIB-W18/12-7-3 "Design of Joints with Nail Plates" - B Norén
13-7-9	Strength of Finger Joints - H J Larsen
13-100-4	CIB Structural Timber Design Code. Proposal for Section 6.1.5 Nail Plates - N I Bovim
14-7-1	Design of Joints with Nail Plates (second edition) - B Norén
14-7-2	Method of Testing Nails in Wood (second draft, August 1980) - B Norén
14-7-3	Load-Slip Relationship of Nailed Joints - J Ehlbeck and H J Larsen
14-7-4	Wood Failure in Joints with Nail Plates - B Norén
14-7-5	The Effect of Support Eccentricity on the Design of W- and WW-Trussed with Nail Plate Connectors - B Källsner

22-7-11 Joints on Glued-in Steel Bars Present Relatively New and Progressive Solution in Terms of Timber Structure Design - G N Zubarev, F A Boitemirov and V M Golovina

22-7-12 The Development of Design Codes for Timber Structures made of Compositive Bars with Plate Joints based on Cyclindrical Nails - Y V Piskunov

22-7-13 Designing of Glued Wood Structures Joints on Glued-in Bars - S B Turkovsky

23-7-1 Proposal for a Design Code for Nail Plates - E Aasheim and K H Solli

23-7-2 Load Distribution in Nailed Joints - H J Blass

24-7-1 Theoretical and Experimental Tension and Shear Capacity of Nail Plate Connections - B Källsner and J Kangas

24-7-2 Testing Method and Determination of Basic Working Loads for Timber Joints with Mechanical Fasteners - Y Hirashima and F Kamiya

24-7-3 Anchorage Capacity of Nail Plate - J Kangas

25-7-2 Softwood and Hardwood Embedding Strength for Dowel type Fasteners - J Ehlbeck and H Werner

25-7-4 A Guide for Application of Quality Indexes for Driven Fasteners Used in Connections in Wood Structures - E G Stern

25-7-5 35 Years of Experience with Certain Types of Connectors and Connector Plates Used for the Assembly of Wood Structures and their Components- E G Stern

25-7-6 Characteristic Strength of Split-ring and Shear-plate Connections - H J Blass, J Ehlbeck and M Schlager

25-7-7 Characteristic Strength of Tooth-plate Connector Joints - H J Blass, J Ehlbeck and M Schlager

25-7-8 Extending Yield Theory to Screw Connections - T E McLain

25-7-9 Determination of k_{def} for Nailed Joints - J W G van de Kuilen

LOAD SHARING

DURATION OF LOAD

24-9-1 Long Term Bending Creep of Wood - T Toratti

24-9-2 Collection of Creep Data of Timber - A Ranta-Maunus

24-9-3 Deformation Modification Factors for Calculating Built-up Wood-Based Structures - I R Kliger

25-9-2 DVM Analysis of Wood. Lifetime, Residual Strength and Quality - L F Nielsen

26-9-1 Long Term Deformations in Wood Based Panels under Natural Climate Conditions. A Comparative Study - S Thelandersson, J Nordh, T Nordh and S Sandahl

TIMBER BEAMS

4-10-1 The Design of Simple Beams - H J Burgess

4-10-2 Calculation of Timber Beams Subjected to Bending and Normal Force - H J Larsen

5-10-1 The Design of Timber Beams - H J Larsen

9-10-1 The Distribution of Shear Stresses in Timber Beams - F J Keenan

9-10-2 Beams Notched at the Ends - K Möhler

11-10-1 Tapered Timber Beams - H Riberholt

13-6-2 Consideration of Size Effects in Longitudinal Shear Strength for Uncracked Beams - R O Foschi and J D Barrett

13-6-3 Consideration of Shear Strength on End-Cracked Beams - J D Barrett and R O Foschi

18-10-1 Submission to the CIB-W18 Committee on the Design of Ply Web Beams by Consideration of the Type of Stress in the Flanges - J A Baird

18-10-2 Longitudinal Shear Design of Glued Laminated Beams - R O Foschi

19-10-1 Possible Code Approaches to Lateral Buckling in Beams - H J Burgess

19-2-1	Creep Buckling Strength of Timber Beams and Columns - R H Leicester
20-2-1	Lateral Buckling Theory for Rectangular Section Deep Beam-Columns - H J Burgess
20-10-1	Draft Clause for CIB Code for Beams with Initial Imperfections - H J Burgess
20-10-2	Space Joists in Irish Timber - W J Robinson
20-10-3	Composite Structure of Timber Joists and Concrete Slab - T Poutanen
21-10-1	A Study of Strength of Notched Beams - P J Gustafsson
22-10-1	Design of Endnotched Beams - H J Larsen and P J Gustafsson
22-10-2	Dimensions of Wooden Flexural Members under Constant Loads - A Pozgai
22-10-3	Thin-Walled Wood-Based Flanges in Composite Beams - J König
22-10-4	The Calculation of Wooden Bars with flexible Joints in Accordance with the Polish Standart Code and Strict Theoretical Methods - Z Mielczarek
23-10-1	Tension Perpendicular to the Grain at Notches and Joints - T A C M van der Put
23-10-2	Dimensioning of Beams with Cracks, Notches and Holes. An Application of Fracture Mechanics - K Riipola
23-10-3	Size Factors for the Bending and Tension Strength of Structural Timber - J D Barret and A R Fewell
23-12-1	Bending Strength of Glulam Beams, a Design Proposal - J Ehlbeck and F Colling
23-12-3	Glulam Beams, Bending Strength in Relation to the Bending Strength of the Finger Joints - H Riberholt
24-10-1	Shear Strength of Continuous Beams - R H Leicester and F G Young

ENVIRONMENTAL CONDITIONS

LAMINATED MEMBERS

9-12-1 Experiments to Provide for Elevated Forces at the Supports of Wooden Beams with Particular Regard to Shearing Stresses and Long-Term Loadings - F Wassipaul and R Lackner

9-12-2 Two Laminated Timber Arch Railway Bridges Built in Perth in 1849 - L G Booth

9-6-4 Consideration of Combined Stresses for Lumber and Glued Laminated Timber - K Möhler

11-6-3 Consideration of Combined Stresses for Lumber and Glued Laminated Timber (addition to Paper CIB-W18/9-6-4) - K Möhler

12-12-1 Glulam Standard Part 2: Glued Timber Structures; Rating (3rd draft)

12-12-2 Glulam Standard Part 3: Glued Timber Structures; Performance (3 rd draft)

13-12-1 Glulam Standard Part 3: Glued Timber Structures; Performance (4th draft)

14-12-1 Proposals for CEI-Bois/CIB-W18 Glulam Standards - H J Larsen

14-12-2 Guidelines for the Manufacturing of Glued Load-Bearing Timber Structures - Stevin Laboratory

14-12-3 Double Tapered Curved Glulam Beams - H Riberholt

14-12-4 Comment on CIB-W18/14-12-3 - E Gehri

18-12-1 Report on European Glulam Control and Production Standard - H Riberholt

18-10-2 Longitudinal Shear Design of Glued Laminated Beams - R O Foschi

19-12-1 Strength of Glued Laminated Timber - J Ehlbeck and F Colling

19-12-2 Strength Model for Glulam Columns - H J Blaß

19-12-3 Influence of Volume and Stress Distribution on the Shear Strength and Tensile Strength Perpendicular to Grain - F Colling

19-12-4 Time-Dependent Behaviour of Glued-Laminated Beams - F Zaupa

24-12-2 Influence of Perpendicular-to-Grain Stressed Volume on the Load-Carrying Capacity of Curved and Tapered Glulam Beams - J Ehlbeck and J Kürth

25-12-1 Determination of Characteristic Bending Values of Glued Laminated Timber. EN-Approach and Reality - E Gehri

26-12-1 Norwegian Bending Tests with Glued Laminated Beams-Comparative Calculations with the "Karlsruhe Calculation Model" - E Aasheim, K Solli, F Colling, R H Falk, J Ehlbeck and R Görlacher

26-12-2 Simulation Analysis of Norwegian Spruce Glued-Laminated Timber - R Hernandez and R H Falk

26-12-3 Investigation of Laminating Effects in Glued-Laminated Timber - F Colling and R H Falk

26-12-4 Comparing Design Results for Glulam Beams According to Eurocode 5 and to the French Working Stress Design Code (CB71) - F Rouger

27-12-1 State of the Art Report: Glulam Timber Bridge Design in the U.S. - M A Ritter and T G Williamson

27-12-2 Common Design Practice for Timber Bridges in the United Kingdom - C J Mettem, J P Marcroft and G Davis

27-12-3 Influence of Weak Zones on Stress Distribution in Glulam Beams - E Serrano and H J Larsen

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7-13-1 Fibre Building Boards for CIB Timber Code (First Draft) - O Brynildsen

9-13-1 Determination of the Bearing Strength and the Load-Deformation Characteristics of Particleboard - K Möhler, T Budianto and J Ehlbeck

9-13-2 The Structural Use of Tempered Hardboard - W W L Chan

11-13-1 Tests on Laminated Beams from Hardboard under Short- and Longterm Load - W Nozynski

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10-14-1	Design of Roof Bracing - The State of the Art in South Africa - P A V Bryant and J A Simon
11-14-1	Design of Metal Plate Connected Wood Trusses - A R Egerup
12-14-1	A Simple Design Method for Standard Trusses - A R Egerup
13-14-1	Truss Design Method for CIB Timber Code - A R Egerup
13-14-2	Trussed Rafters, Static Models - H Riberholt
13-14-3	Comparison of 3 Truss Models Designed by Different Assumptions for Slip and E-Modulus - K Möhler
14-14-1	Wood Trussed Rafter Design - T Feldborg and M Johansen
14-14-2	Truss-Plate Modelling in the Analysis of Trusses - R O Foschi
14-14-3	Cantilevered Timber Trusses - A R Egerup
14-7-5	The Effect of Support Eccentricity on the Design of W- and WW-Trusses with Nail Plate Connectors - B Källsner
15-14-1	Guidelines for Static Models of Trussed Rafters - H Riberholt
15-14-2	The Influence of Various Factors on the Accuracy of the Structural Analysis of Timber Roof Trusses - F R P Pienaar
15-14-3	Bracing Calculations for Trussed Rafter Roofs - H J Burgess
15-14-4	The Design of Continuous Members in Timber Trussed Rafters with Punched Metal Connector Plates - P O Reece
15-14-5	A Rafter Design Method Matching U.K. Test Results for Trussed Rafters - H J Burgess
16-14-1	Full-Scale Tests on Timber Fink Trusses Made from Irish Grown Sitka Spruce - V Picardo
17-14-1	Data from Full Scale Tests on Prefabricated Trussed Rafters - V Picardo

17-14-2 Simplified Static Analysis and Dimensioning of Trussed Rafters - H Riberholt

17-14-3 Simplified Calculation Method for W-Trusses - B Källsner

18-14-1 Simplified Calculation Method for W-Trusses (Part 2) - B Källsner

18-14-2 Model for Trussed Rafter Design - T Poutanen

19-14-1 Annex on Simplified Design of W-Trusses - H J Larsen

19-14-2 Simplified Static Analysis and Dimensioning of Trussed Rafters - Part 2 - H Riberholt

19-14-3 Joint Eccentricity in Trussed Rafters - T Poutanen

20-14-1 Some Notes about Testing Nail Plates Subjected to Moment Load - T Poutanen

20-14-2 Moment Distribution in Trussed Rafters - T Poutanen

20-14-3 Practical Design Methods for Trussed Rafters - A R Egerup

22-14-1 Guidelines for Design of Timber Trussed Rafters - H Riberholt

23-14-1 Analyses of Timber Trussed Rafters of the W-Type - H Riberholt

23-14-2 Proposal for Eurocode 5 Text on Timber Trussed Rafters - H Riberholt

24-14-1 Capacity of Support Areas Reinforced with Nail Plates in Trussed Rafters - A Kevarinmäki

25-14-1 Moment Anchorage Capacity of Nail Plates in Shear Tests - A Kevarinmaki and J. Kangas

25-14-2 Design Values of Anchorage Strength of Nail Plate Joints by 2-curve Method and Interpolation - J Kangas and A Kevarinmaki

26-14-1 Test of Nail Plates Subjected to Moment - E Aasheim

26-14-2 Moment Anchorage Capacity of Nail Plates - A Kevarinmäki and J Kangas

STRUCTURAL STABILITY

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12-16-1	British Standard BS 5268 the Structural Use of Timber: Part 4 Fire Resistance of Timber Structures
13-100-2	CIB Structural Timber Design Code. Chapter 9. Performance in Fire
19-16-1	Simulation of Fire in Tests of Axially Loaded Wood Wall Studs - J König
24-16-1	Modelling the Effective Cross Section of Timber Frame Members Exposed to Fire - J König
25-16-1	The Effect of Density on Charring and Loss of Bending Strength in Fire - J König
25-16-2	Tests on Glued-Laminated Beams in Bending Exposed to Natural Fires - F Bolonius Olesen and J König
26-16-1	Structural Fire Design According to Eurocode 5, Part 1.2 - J König

STATISTICS AND DATA ANALYSIS

13-17-1	On Testing Whether a Prescribed Exclusion Limit is Attained - W G Warren
16-17-1	Notes on Sampling and Strength Prediction of Stress Graded Structural Timber - P Glos
16-17-2	Sampling to Predict by Testing the Capacity of Joints, Components and Structures - B Norén
16-17-3	Discussion of Sampling and Analysis Procedures - P W Post
17-17-1	Sampling of Wood for Joint Tests on the Basis of Density - I Smith, L R J Whale
17-17-2	Sampling Strategy for Physical and Mechanical Properties of Irish Grown Sitka Spruce - V Picardo
18-17-1	Sampling of Timber in Structural Sizes - P Glos

FRACTURE MECHANICS

23-10-1 Tension Perpendicular to the Grain at Notches and Joints - T A C M van der Put

23-10-2 Dimensioning of Beams with Cracks, Notches and Holes. An Application of Fracture Mechanics - K Riipola

23-19-1 Determination of the Fracture Energie of Wood for Tension Perpendicular to the Grain - W Rug, M Badstube and W Schöne

23-19-2 The Fracture Energy of Wood in Tension Perpendicular to the Grain. Results from a Joint Testing Project - H J Larsen and P J Gustafsson

23-19-3 Application of Fracture Mechanics to Timber Structures - A Ranta-Maunus

24-19-1 The Fracture Energy of Wood in Tension Perpendicular to the Grain - H J Larsen and P J Gustafsson

GLUED JOINTS

20-18-1 Wood Materials under Combined Mechanical and Hygral Loading - A Martensson and S Thelandersson

20-18-2 Analysis of Generalized Volkersen - Joints in Terms of Linear Fracture Mechanics - P J Gustafsson

20-18-3 The Complete Stress-Slip Curve of Wood-Adhesives in Pure Shear - H Wernersson and P J Gustafsson

22-18-1 Perspective Adhesives and Protective Coatings for Wood Structures - A S Freidin

SERVICEABILITY

27-20-1 Codification of Serviceability Criteria - R H Leicester

27-20-2 On the Experimental Determination of Factor k_{def} and Slip Modulus k_{ser} from Short- and Long-Term Tests on a Timber-Concrete Composite (TCC) Beam - S Capretti and A Ceccotti

27-20-3 Serviceability Limit States: A Proposal for Updating Eurocode 5 with Respect to Eurocode 1 - P Racher and F Rouger

27-20-4 Creep Behavior of Timber under External Conditions - C Le Govic, F Rouger, T Toratti and P Morlier

CIB TIMBER CODE

2-100-1 A Framework for the Production of an International Code of Practice for the Structural Use of Timber - W T Curry

5-100-1 Design of Solid Timber Columns (First Draft) - H J Larsen

5-100-2 A Draft Outline of a Code for Timber Structures - L G Booth

6-100-1 Comments on Document 5-100-1; Design of Solid Timber Columns - H J Larsen and E Theilgaard

6-100-2 CIB Timber Code: CIB Timber Standards - H J Larsen and E Theilgaard

7-100-1 CIB Timber Code Chapter 5.3 Mechanical Fasteners; CIB Timber Standard 06 and 07 - H J Larsen

8-100-1 CIB Timber Code - List of Contents (Second Draft) - H J Larsen

9-100-1 The CIB Timber Code (Second Draft)

11-100-1 CIB Structural Timber Design Code (Third Draft)

11-100-2 Comments Received on the CIB Code
a U Saarelainen
b Y M Ivanov
c R H Leicester
d W Nozynski
e W R A Meyer
f P Beckmann; R Marsh
g W R A Meyer
h W R A Meyer

11-100-3 CIB Structural Timber Design Code; Chapter 3 - H J Larsen

12-100-1	Comment on the CIB Code - Sous-Commission Glulam
12-100-2	Comment on the CIB Code - R H Leicester
12-100-3	CIB Structural Timber Design Code (Fourth Draft)
13-100-1	Agreed Changes to CIB Structural Timber Design Code
13-100-2	CIB Structural Timber Design Code. Chapter 9: Performance in Fire
13-100-3a	Comments on CIB Structural Timber Design Code
13-100-3b	Comments on CIB Structural Timber Design Code - W R A Meyer
13-100-3c	Comments on CIB Structural Timber Design Code - British Standards Institution
13-100-4	CIB Structural Timber Design Code. Proposal for Section 6.1.5 Nail Plates - N I Bovim
14-103-2	Comments on the CIB Structural Timber Design Code - R H Leicester
15-103-1	Resolutions of TC 165-meeting in Athens 1981-10-12/13
21-100-1	CIB Structural Timber Design Code. Proposed Changes of Sections on Lateral Instability, Columns and Nails - H J Larsen
22-100-1	Proposal for Including an Updated Design Method for Bearing Stresses in CIB W18 - Structural Timber Design Code - B Madsen
22-100-2	Proposal for Including Size Effects in CIB W18A Timber Design Code - B Madsen
22-100-3	CIB Structural Timber Design Code - Proposed Changes of Section on Thin-Flanged Beams - J König
22-100-4	Modification Factor for "Aggressive Media" - a Proposal for a Supplement to the CIB Model Code - K Erler and W Rug
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WORKING COMMISSION W18 - TIMBER STRUCTURES

DEVELOPMENT OF THE "CRITICAL BEARING" : DESIGN CLAUSE IN CSA-086.1

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JULY 1994

ABSTRACT

There are currently inconsistencies in the compression perpendicular-to-grain (C-perp) design procedures in the Canadian code for Engineering Design In Wood, CSA-O86.1-M89. For example, a single specified design value is given for a species group irrespective of the loading condition, or the lumber grade. On the other hand, different values are assigned to Douglas-fir lumber and Douglas-fir glulam (glued-laminated timber). These inconsistencies not only make the design code confusing to designers, but they also hinder the introduction of appropriate C-perp strength values for products such as machine stress rated lumber. A research program focused on resolving these inconsistencies in the C-perp design procedures for solid sawn lumber and glulam has recently been completed. The work consisted of short term and 2-month constant compression perpendicular-to-grain load tests on two commercial species groups. Finite element analyses of typical C-perp stress conditions were also performed to study the stress distributions for various C-perp applications. This work resulted in proposed changes to the compression perpendicular-to-grain design clause for the next revision of CSA-O86.1. One of the significant changes proposed is the addition of a separate check for compression perpendicular-to-grain stresses produced by loads applied near beam supports. In this paper, the work done to develop this particular clause is presented.

1. INTRODUCTION

1.1 Establishing Characteristic C-perp Values

In the 1989 edition of the Canadian engineering design in wood code, commonly referred to as CSA-O86.1 (CSA, 1989), the ASTM D245-88 (ASTM, 1992) method of establishing compression perpendicular-to-grain strength from the computed stress at the 1-mm deformation level was adopted. ASTM D245-88, in turn, relies on the procedures outlined in ASTM D143-83 for tests on small clear wood samples and ASTM D2555-88 for establishing clear wood strength values (ASTM, 1992). In the 1989 edition of CSA-O86.1, C-perp design values were established by converting the working stress design values to values compatible with the limit states design code.

Adopting ASTM D245-88 resulted in substantial increases in the C-perp design values for the D.fir-L, S-P-F and Hem-Fir species groups. The reason for this increase is because in the past, the C-perp design values were based on measured stress at the proportional limit rather than the stress at a 1-mm deformation level which is well over the proportional limit. This change resulted in increases in the order of 40 to 60 percent from the 1984 code, CAN3/CSA-O86-M84 (CSA, 1984).

These changes, however, were only made to the C-perp design values and design equations for sawn lumber. Consequently, the 1989 edition showed differences in the C-perp design approach and specified strengths between glulam and sawn lumber. These discrepancies were summarized by Lum (1992).

Following approval of the 1989 edition of CSA-O86, the Technical Committee recommended that work on rationalizing the design procedure for C-perp be carried out. Therefore, this study focuses on either resolving or explaining the differences in the C-perp design procedures for glulam and sawn lumber.

While the 1989 code was under development, a study on the C-perp strength of machine stress rated (MSR) lumber had just been completed. Using the ASTM approach, it was demonstrated that higher C-perp values could be justified for MSR lumber (Lum, 1990). However, it was recommended that, given that the C-perp design values for all sawn lumber were to increase substantially, these additional increases for MSR lumber not be implemented until the C-perp design procedure had been reviewed and, if necessary.

1.2 Design Procedures for C-perp

One important requirement of any revised C-perp design procedure is that the procedure be simple to apply. While this is true for nearly all code design procedures, it is particularly important for C-perp because of the large number of places in a structure where C-perp occurs. The current C-perp design procedure allows the designer to assume a uniform stress distribution between the bearing plate and the wood member at the supports of a beam. This approach ignores stress concentrations at the edge of a bearing plate and the non-uniformity of bearing stresses due to rotation of the end of a beam.

Madsen *et al* (1982) proposed a different approach. His proposal distinguished between the C-perp resistance due to the stress concentrations at the ends of the bearing plates and those C-perp stresses that were produced under the bearing plate but away from the ends of the plate. His proposal also involved a new C-perp test specimen and a new criterium for establishing the characteristic C-perp strength. In his proposed test procedure, the bearing plate would cover the full length of the test specimen thus eliminating the contribution from the shear forces that appear at the edges of the bearing plate in a standard ASTM C-perp test. This modification to the test method is necessary because the Madsen's proposed design procedure attempts to quantify the actual amount of C-perp deformation.

The current Canadian C-perp design procedure does not allow the designer to calculate what C-perp deformations may occur for a particular detail. In the U.S., there is a provision whereby the allowable C-perp stress is reduced to approximately 72 percent of the basic value if the designer feels that the C-perp stress is for a "critical" application (NFPA, 1986). The design specification is not clear as to when this applies. Nevertheless, the intent appears to be to use this provision where the C-perp deformations may be critical without actually stating how to calculate the C-perp deformation.

There are two possible reasons for this omission. First, most engineering design codes do not specify serviceability limits for C-perp; that is, limits that do not affect safety are left to the discretion of the engineer. Secondly, it is very difficult to compute the C-perp deformation with a reasonable level of accuracy over the range of possible C-perp stress conditions.

This study was developed considering the following:

- C-perp design values for sawn lumber had just been increased based on the ASTM D245-88 approach
- C-perp design procedures for lumber and glulam had to be rationalized with little or no changes to the recently introduced design values
- A previous study had shown that higher C-perp values for MSR lumber were justified
- There were doubts whether the simple procedures currently used would continue to be safe for new engineered applications
- C-perp design procedures had to be kept simple

All these requirements pointed towards an incremental change to the existing C-perp design procedures rather than a major change to the design philosophy and test procedures as suggested by Madsen *et al* (1982).

2. MATERIALS AND METHODS

2.1 Test Material Sampling and Preparation

Short term and 2-month constant load test samples were prepared from nominal 38-mm by 140-mm pieces of lumber from the S-P (white spruce and lodgepole pine) and D.fir-L (Douglas-fir and larch) commercial species groups that were originally sampled for a test program on Canadian lamstock (Karacabeyli, 1991).

The use of dimension lumber sizes meant that square specimens measuring 38-mm by 38-mm had to be used and that tests to relate the C-perp test results using a 38-mm deep specimen to the standard ASTM 51-mm deep specimen had to be performed. This was done with samples of amabilis fir where it was possible to prepare groups of 38-mm deep by 38-mm wide specimens that were matched to 51-mm deep by 51-mm wide specimens. It was arbitrarily decided that the 38-mm specimens would be tested with 38-mm long bearing plates while the 51-mm deep specimens would be tested with 50-mm long bearing plates. All specimens were 152-mm long. The testing showed that the average C-perp stress at a 1-mm deformation in a 51-mm deep specimen could be obtained at a 0.75-mm deformation in a 38-mm deep specimen. For the purposes of the C-perp rationalization study, a 0.75-mm deformation limit therefore was chosen to obtain values comparable to the ASTM D143 approach.

A stratified sampling plan was followed where specimens were selected based on their clear wood density. The intent was to develop a procedure that could be applied to all grades, species, and lumber densities. Once the 38-mm by 140-mm samples were selected, matched C-perp test specimens were prepared from each sample. The cutting pattern for each 38-mm by 140-mm lumber sample was presented by Lum (1992). The cutting pattern yielded specimens for the following tests: short term control tests, 2-month constant load high humidity tests, 2-month constant load normal humidity tests, and tests to quantify the variation in C-perp stiffness across the wide face. Each short term test group consisted of 30 replicates. Constant load test groups contained either 28 or 30 replicates. Other minor tests to confirm items such as the effect of growth ring orientation and to develop a factor for bearing on the wide faces are discussed elsewhere (Lum 1993, 1994).

2.2 Short Term Test Procedures

The test procedure was similar to that outlined in ASTM D143 except for the specimen size, as discussed above, and the growth ring orientation. The growth ring orientation was maintained between end matched specimens. Otherwise, no attention was given to growth ring orientation and the presence of pith.

2.3 Constant Load Test Procedures

Test frames originally built to test lumber in compression parallel-to-grain (Samson and Olson, 1987) were modified for the constant C-perp load tests. The modification allowed the C-perp specimens to be tested in pairs as shown in Figure 1. A spherical washer under the bottom bearing plate and a half-round section above the top bearing plate allowed rotation of the bearing plates as the specimens deformed.

Steel bearing plates were used that covered the full length of the specimens and extended beyond the ends of the specimens. This permitted the vertical deformations of the specimens to be periodically measured with calipers while the specimens were under load. The top two bearing plates were 13-mm thick, while the bottom bearing plate was 19-mm thick. A bearing area of 38 mm by 152 mm was assumed for all bearing stress calculations.

The constant loads were set at a fraction of the maximum design loads given in CSA-O86.1-M89. Design stresses, converting back to a working stress basis, for C-perp are 2.9 MPa for S-P-F and 4.3 MPa for D.fir-L. In this paper, these design stresses will be referred to as the allowable working stress for the S-P and D.fir-L samples, respectively. These stresses do not have adjustments for duration of load, and were consistent with the short term tests results performed in this study.

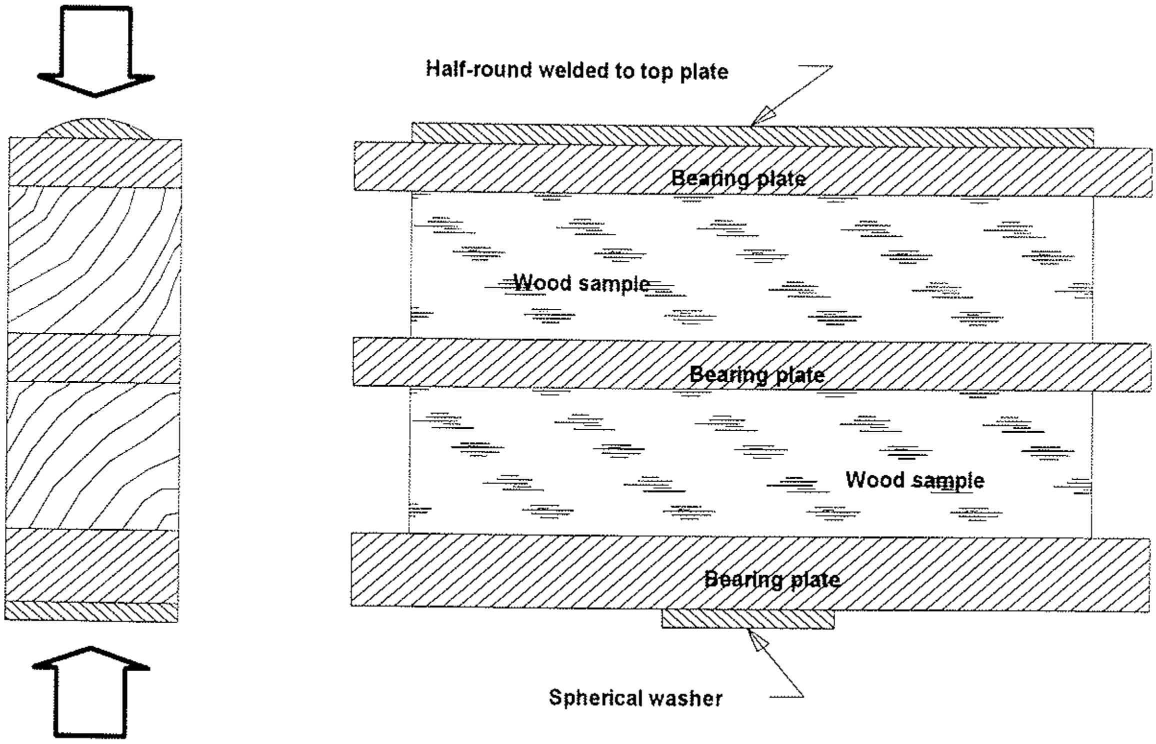

Figure 1. Test set-up for constant load tests

Constant load tests were performed with matched groups at two environmental conditions: 20°C/65% RH and 12°C/85% RH. Both conditions are considered "dry service". All specimens selected for testing were first conditioned for several months at the constant environmental conditions.

For Phase I of the constant load tests, a load required to induce an average C-perp stress equal to 60% of the allowable working stress was applied. In Phase II, the constant load tests at the two environmental conditions were repeated with another set of end matched specimens. This time, the constant load levels were set at 40% of the allowable working stress. For both Phases I and II, the constant loads were maintained for at least 2 months. During this period, the average vertical deformation in each specimen was recorded.

2.4 Finite Element Stress Analysis

Before assessing and revising the design procedures on C-perp, it was necessary to obtain information on the true C-perp stress distribution. Unfortunately, the true C-perp stress distribution within a wood member, even for a simple case of a simply supported beam, cannot be determined analytically. The presence of stress concentrations at the edges of the bearing plate, and the orthotropic nature and non-linear stress-strain behavior of the wood all contribute to making assessing the stress distribution difficult if not impossible.

One alternative is to estimate the stress distribution numerically using, for example, a finite element model. In this study, a finite element program based on two-dimensional quadratic iso-parametric elements was used. The elements are capable of modeling a non-linear orthotropic material that is assumed to follow an exponential stress-strain curve. Only small deformations can be considered as the elements are not formulated to account for geometric non-linearity.

The two dimensional elements assume a uniform and isotropic material through the thickness of the element. Therefore, the model does not represent the actual growth ring pattern or the presence of other growth characteristics. Input parameters for the finite element analysis are summarized in previous reports by Lum (1993, 1992). A sensitivity analysis showed that the stress patterns were not very sensitive to the material parameters used. Also, a similar stress pattern would be obtained with a 89-mm deep beam provided the bearing plate lengths and positions are scaled down at the same proportion as the depth.

A common beam application was then selected for analysis by the finite element method. For practical reasons, the analysis was limited to a rectangular beam. Figure 2 shows the dimensions of the beam analyzed and the five bearing locations considered. Steel bearing plates 89 mm long and 6.3 mm thick were used in all of these cases. Load was applied to the top plate such that the average bearing stress computed by a designer would be 3.4 MPa (500 psi). All bearing plates were allowed to rotate with the beam; that is, there is always full contact between the bearing plate and the beam.

3. RESULTS AND DISCUSSIONS

3.1 Constant Load Tests

Preliminary plots of C-perp deformation over time for Phase I were presented by Lum (1993). Figure 3 to Figure 6 show the mean trend line of the C-perp deformation over time from Phase I of the study. Figure 7 to Figure 10 present the mean trend lines for data collected from Phase II. Constant load levels were 60% of the working stress design value in Phase I and 40% of the working stress design value in Phase II. Deformations within one standard deviation (SD) of the mean trend line are also shown. As expected, creep at the high humidity conditions (12°C/85% RH) is significantly higher than at standard conditions (12°C/65% RH).

Both species groups crept excessive amounts at the high humidity levels; in some cases, the deformation was visible to the eye. When the deformations were high (greater than 1 mm over the 38 mm depth), the specimens tended to move laterally producing a rolling-shear type failure.

Under the 20°C/65% RH conditions, the spruce/pine group performed well deforming on average less than 0.75 mm over a two month period (Figure 6). This amount of deformation would be comparable to the deformation caused by shrinkage and swelling from climatic variations. The Douglas-fir/larch group, however, crept over 100% more than the spruce/pine sample (Figure 5). This suggests some inconsistency in the assignment of the C-perp design values for Douglas-fir/larch compared with the spruce/pine group.

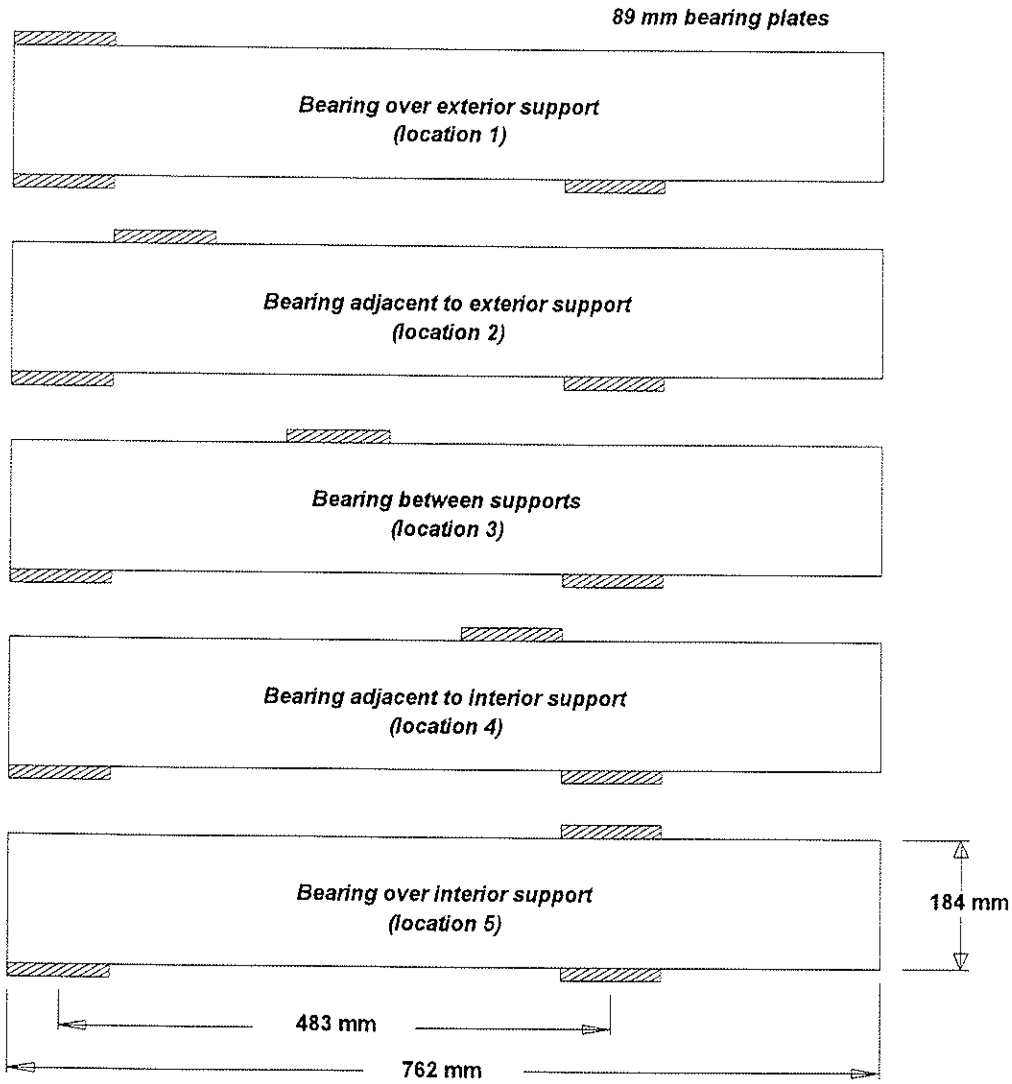

Figure 2. Bearing locations and beam configuration analysed

Reducing the constant stress level from 60% to 40% of the working stress values resulted in reasonable creep performance from the Douglas-fir/larch test group. At the end of the two month period, the average total deformation under the high humidity conditions was about 2% of the specimen depth (Figure 7 and Figure 8).

Under the normal environmental conditions and constant stress levels corresponding to 40% of the working stress design values, the average total deformation of the two species groups was about 1% of the total specimen depth (**Figure** 9 and Figure 10). These deformations are insignificant compared with the deformations due to shrinkage and swelling. But even at this low level, the amount of deformation in the Douglas-fir/larch sample was on average twice that of the spruce/pine sample.

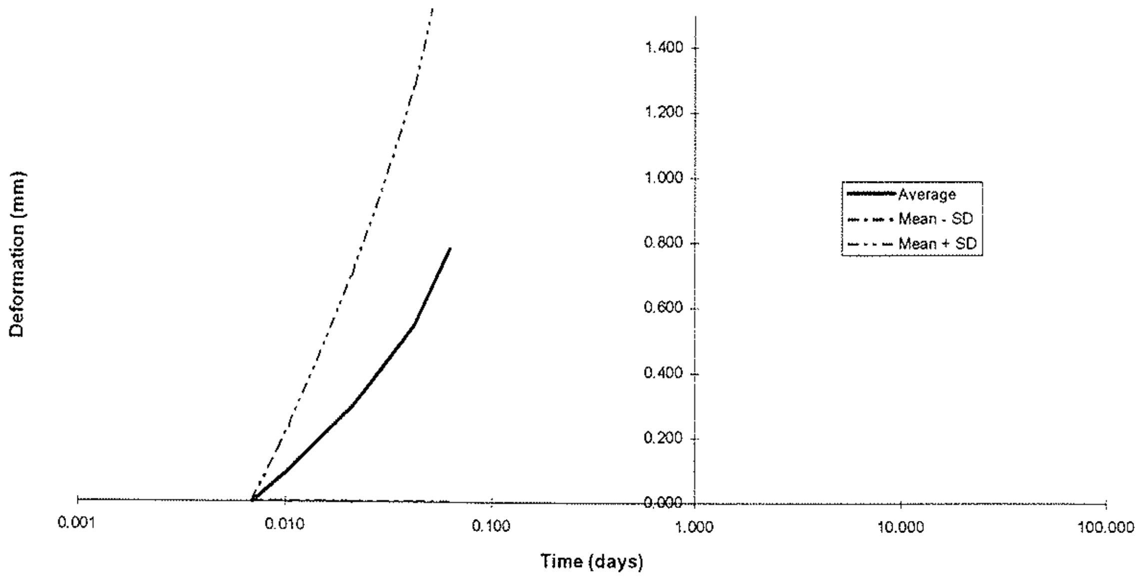

Figure 3. Douglas-fir/Larch at 12°C/85%RH under constant 2.6 MPa stress (28 samples)

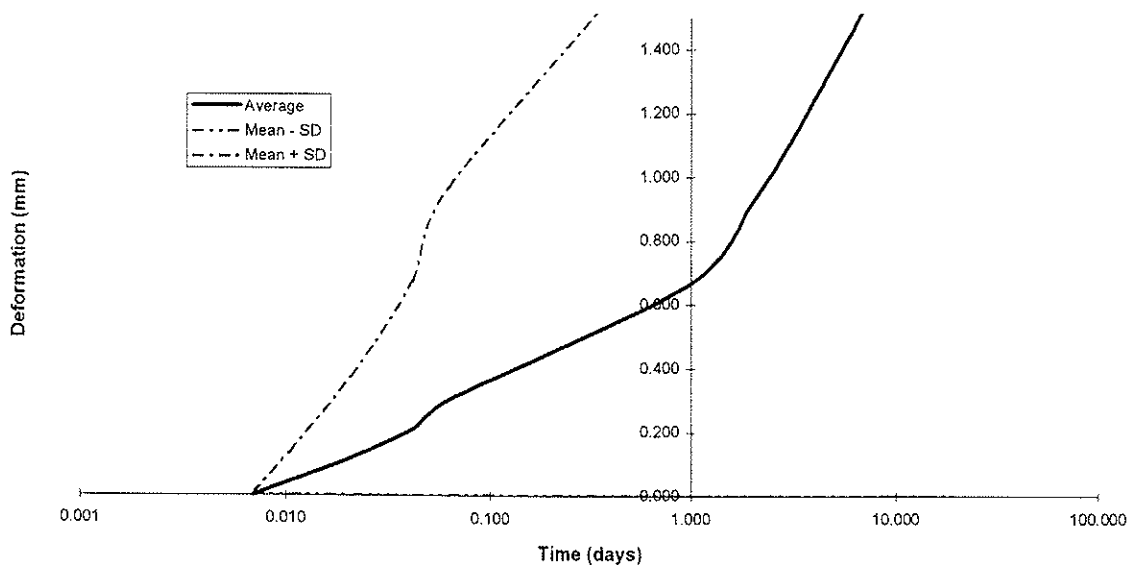

Figure 4. Spruce/Pine at 12°C/85%RH under constant 1.8 MPa stress (30 samples)

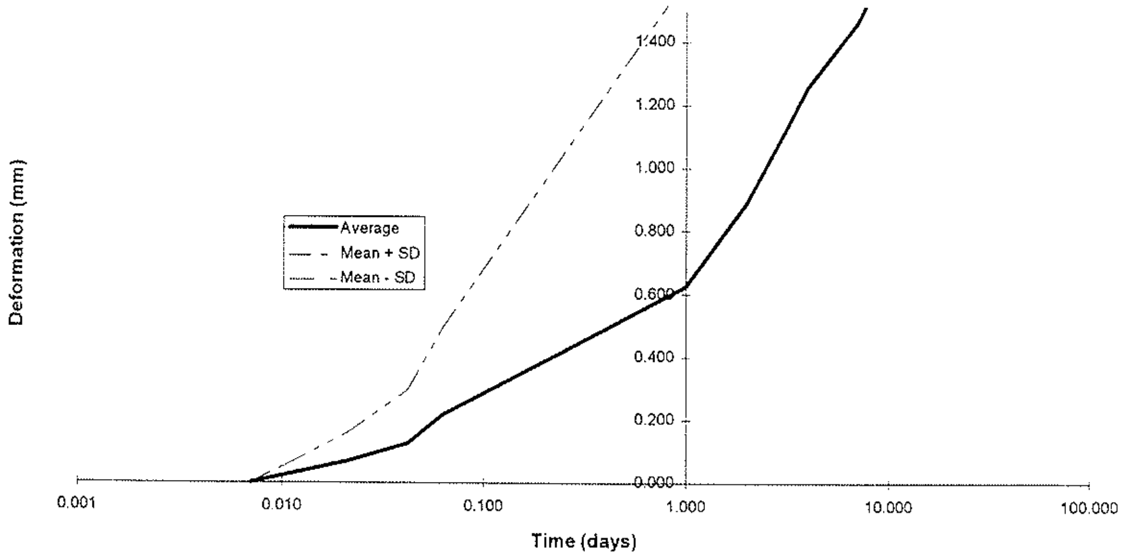

Figure 5. Douglas-fir/Larch at 20°C/65%RH under constant 2.6 MPa stress (28 samples)

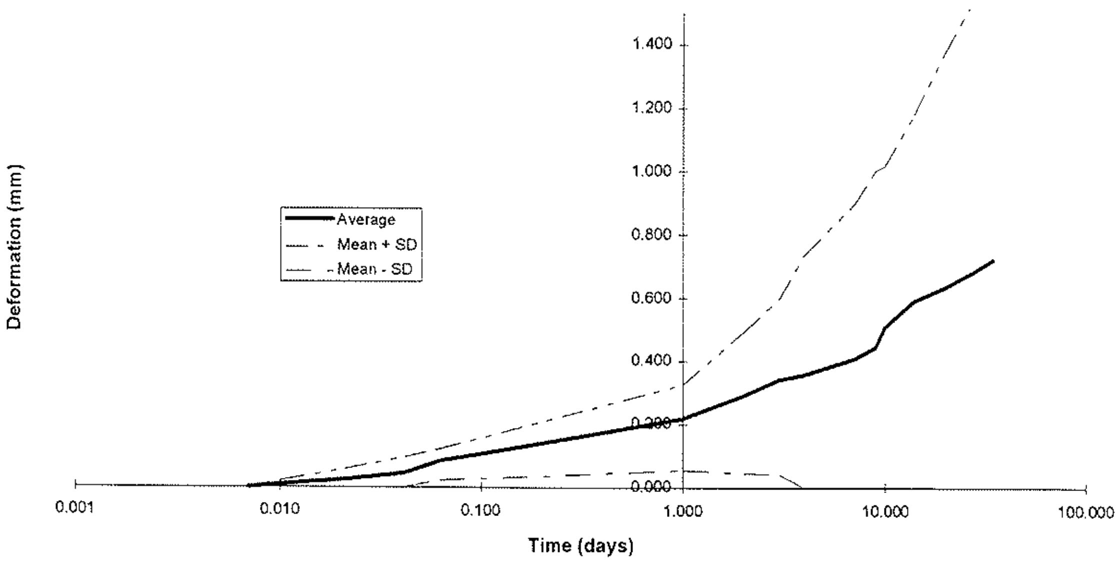

Figure 6. Spruce/Pine at 20°C/65%RH under constant 1.8 MPa stress (30 samples)

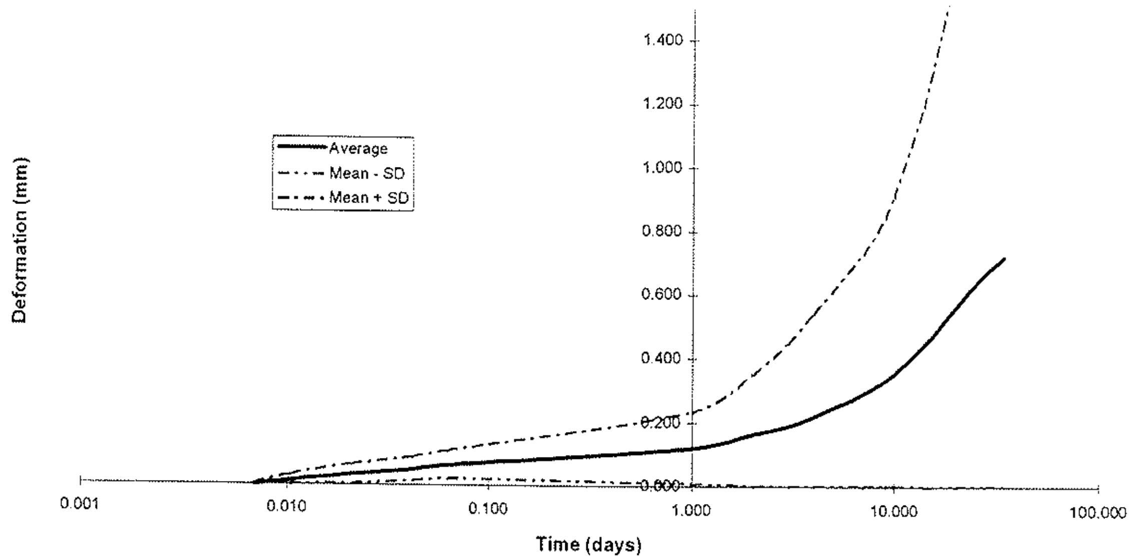

Figure 7. Douglas-fir/Larch at 12°C/85% RH under constant 1.7 MPa stress (30 samples)

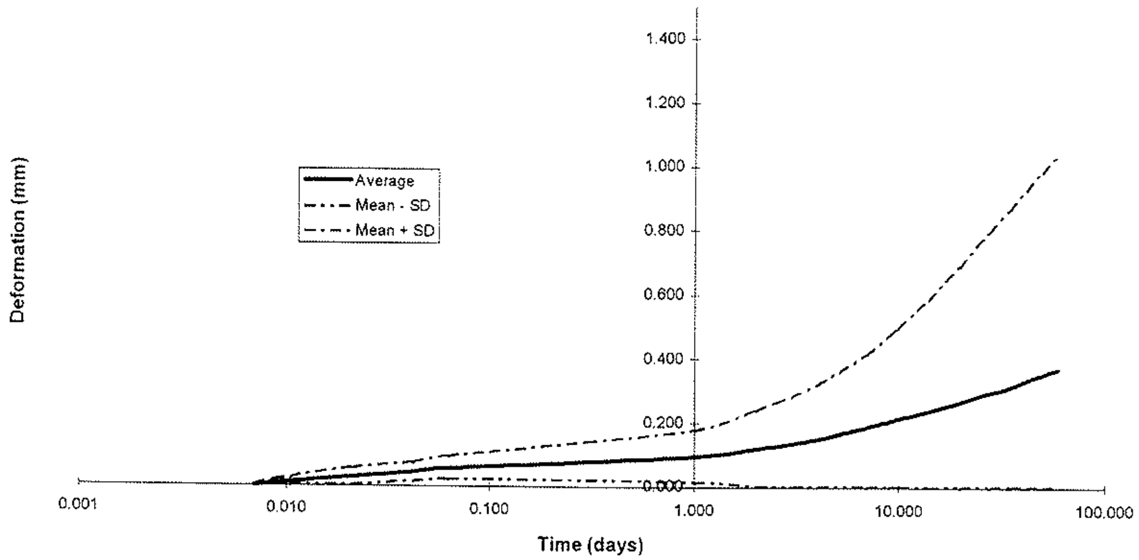

Figure 8. Spruce-Pine at 12°C/85% RH under constant 1.2 MPa stress (30 samples)

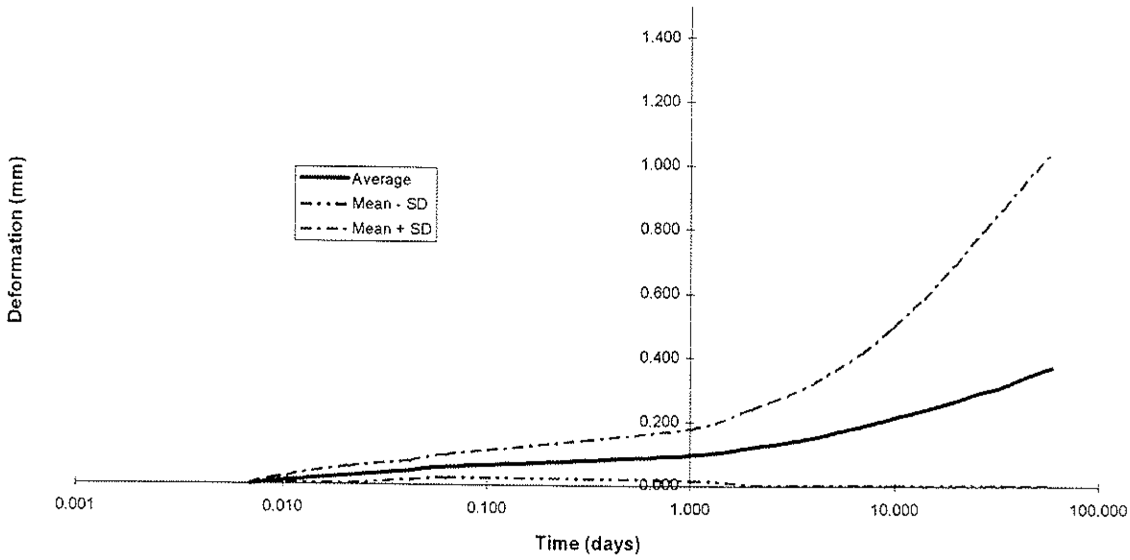

Figure 9. Douglas-fir/Larch at 20°C/65% RH under constant 1.7 MPa stress (30 samples)

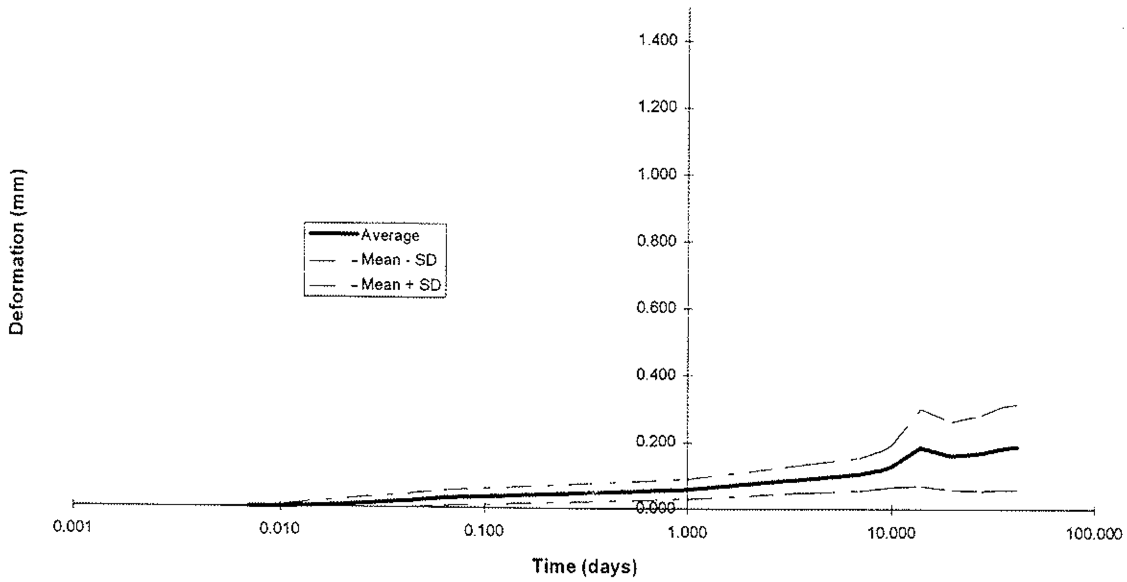

Figure 10. Spruce-Pine at 20°C/65% RH under constant 1.2 MPa stress (30 samples)

The results also showed that when using these 38-mm deep specimens, tertiary or accelerated creep deformation tends to occur when a 0.75 mm deformation is reached. Over a two month constant load period, the data suggests that limiting the applied C-perp stress to about 50 to 60% of the working stress value would, on average, prevent the deformations from reaching that level. The reduction to 50% of the working stress value would be more appropriate for Douglas-fir/larch while the reduction to 60% would be sufficient for spruce/pine. C-perp specified strength values used with these reductions would be appropriate for applications where large C-perp deformations are undesirable.

3.2 Finite Element Stress Analysis

For the beam configurations shown in Figure 2, plots of the C-perp stress distributions along the length and at various depths in the beam as determined by the finite element analysis are shown in Figure 11 to Figure 15. These plots show the levels of C-perp stresses in a 184-mm deep beam when a single concentrated load is applied to a bearing plate on the top edge and is resisted by bottom bearing plates at the left and right supports. C-perp stresses are plotted as positive values; negative values imply tension perpendicular-to-grain stresses. As noted above, these C-perp stress distributions are the results of a load applied to the top bearing such that the average C-perp stress computed by a designer (who assumes a uniform C-perp stress distribution) would be 3.4 MPa (500 psi).

When the applied load is directly over the end of the beam, Figure 11 shows that the C-perp stresses in the beam may be significantly higher than those assumed by the designer. These high stresses are due primarily to the uneven C-perp resistance provided by the beam. At the interior edge of the top bearing plate, there is a stress concentration which tends to produce a high C-perp resistance. At the beam end, there is no stress concentration so the additional C-perp resistance can only be achieved after the beam undergoes a large C-perp deformation.

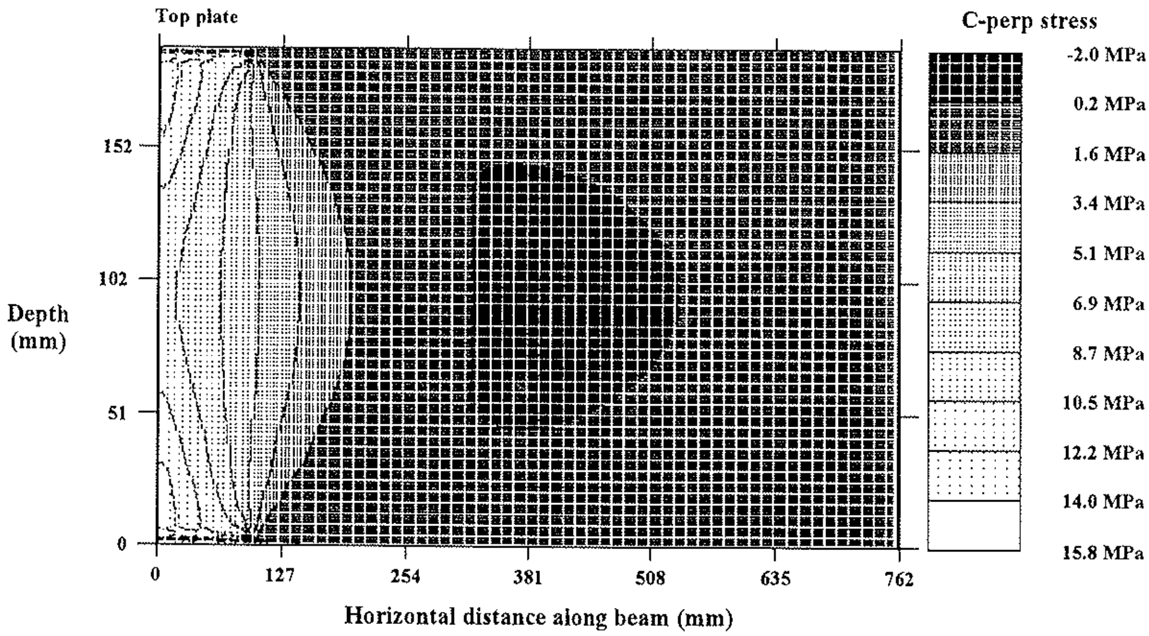

Figure 11. C-perp stress distribution for bearing over end support (Figure 2 - location 1)

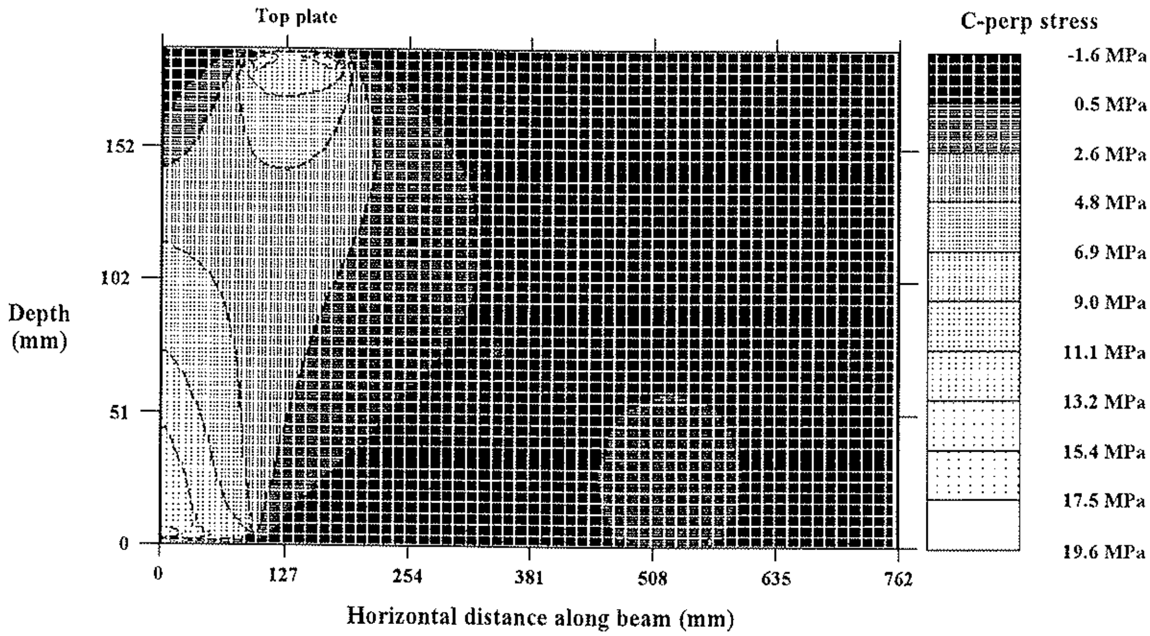

Figure 12. C-perp stress distribution for bearing adjacent to end support (Figure 2 - location 2)

As shown in Figure 12, moving the bearing plate away from the end of the beam influences the bearing stresses in two ways. First, there are now two edges of the bearing plate that are producing a stress concentration. Therefore, large deformations do not have to occur in order to produce a balanced C-perp resistance under the bearing plate. Secondly, some of load applied to the top bearing plate is transferred to the far bearing plate through shear in the beam. As shown next in Figure 13, when the bearing plate is located at mid-span, the load is converted to shear in the beam, and the C-perp stresses within the beam are dramatically reduced. This is a typical situation where the C-perp stress is localized and only occurs at the point of contact between the beam and the bearing plate. In these cases, no reduction in the specified C-perp strength would be necessary.

Thus the volume of wood subjected to the C-perp stress is determined by whether the loading configuration provides an opportunity for the applied load to be converted to a shear force in the beam. Figure 13 gives an indication of the volume of the beam subjected to high C-perp stresses when the applied loads have an opportunity to be converted to shear forces within the beam. In this case, the C-perp stresses that are greater than or equal to the level assumed by the designer occur only around the bearing plate and extend about one-quarter to one-fifth of the beam depth into the beam.

Figure 14 gives a C-perp stress distribution that is similar to Figure 11. Compared to Figure 13, there is a larger volume of the beam that is subjected to a C-perp stress that is equal to or greater than the C-perp stress computed by the designer using the simple design equation. In Figure 15, the bearing plate is directly over an interior support and there is a large volume of the beam that is subjected to the C-perp stress. But unlike the configuration shown in Figure 11, the C-perp stresses for this beam configuration are not much higher than those assumed by the designer.

The C-perp stresses shown in Figure 11 may approach those shown in Figure 15 provided the top and bottom bearing plates are restrained against rotation. Doing so would result in the bearing plate relying more on the stress concentration at the end of the bearing plate for C-perp resistance. An example of this would be bearing plates at the end of short or intermediate length columns bearing on a beam as shown in Figure 11. In this case, the bearing plate would be held essentially horizontal as it is loaded.

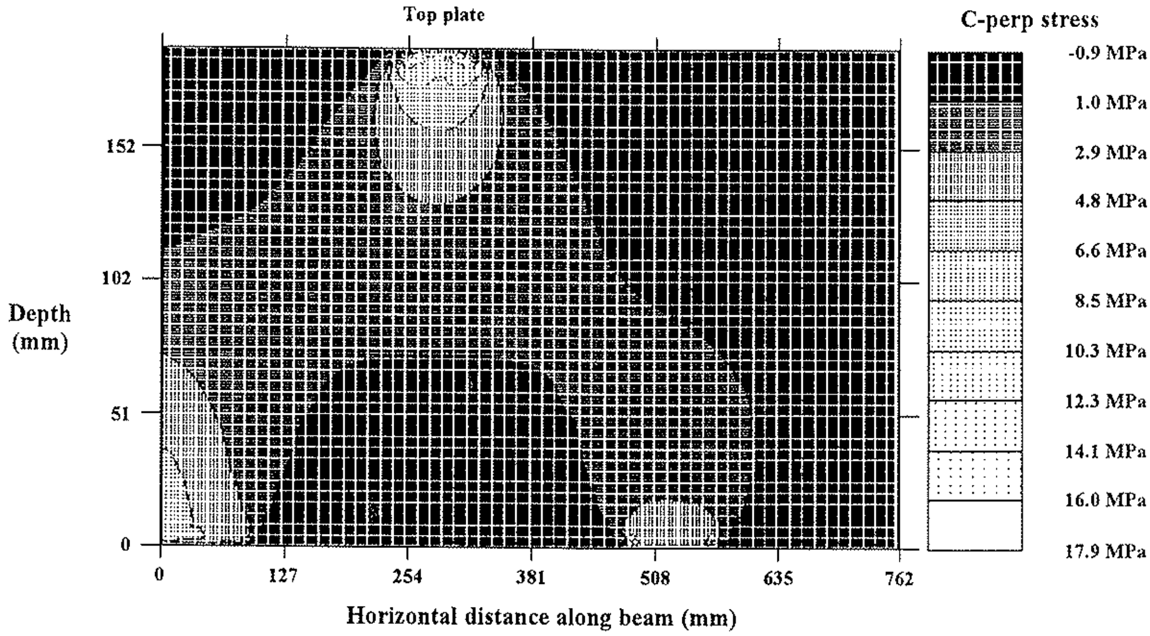

Figure 13. C-perp stress distribution for mid-span loading (Figure 2 - location 3)

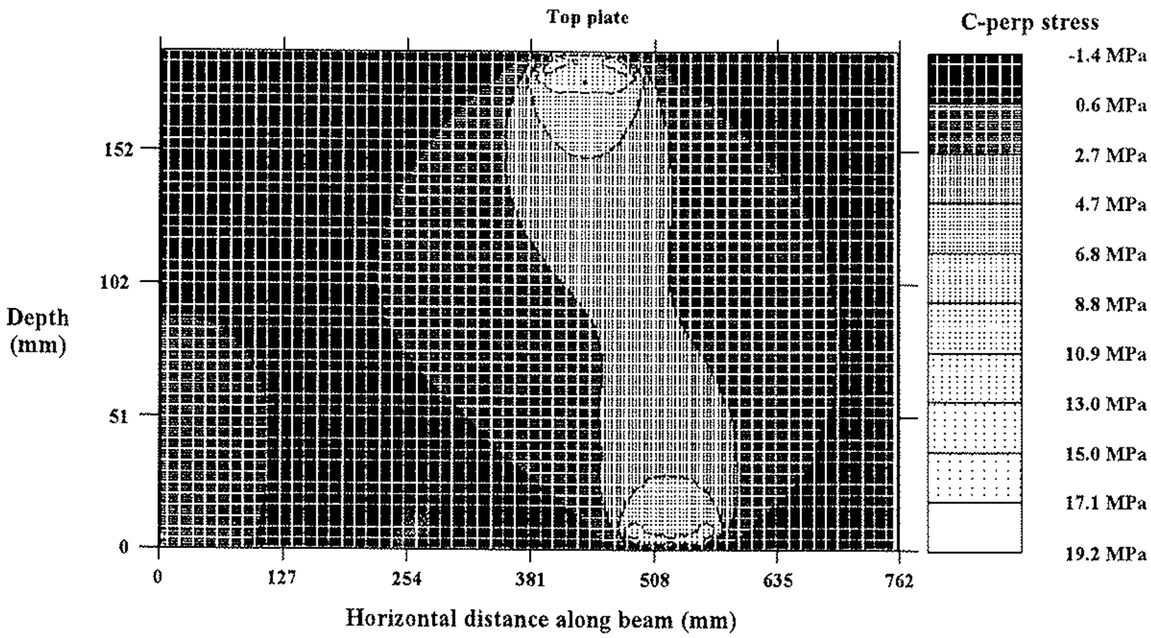

Figure 14. C-perp stress distribution for bearing adjacent to interior support (Figure 2 - location 6)

To summarize, the plots show that although C-perp stresses are high, only a small volume is subject to these high C-perp stresses at the support of a simply supported beam. These stresses may be considered as "contact type" C-perp stresses; that is, C-perp stresses that only occur at the point of contact. This situation would also be valid for beams under uniformly distributed loads. In the case of long beams, the loads applied near the supports will be insignificant compared with the total load. However, for cases where most of the load is applied near a support, the volume of the member subjected to the C-perp stresses at or above that computed by the design is greater than in the case of a simply supported beam with concentrated loads located away from the supports. In effect, the cases shown in Figure 11 and Figure 15 are using the beam as a column to transfer the loads into the support. This could potentially lead to stability problems. These would be considered "critical type applications" where a reduction in the specified C-perp strength would be warranted.

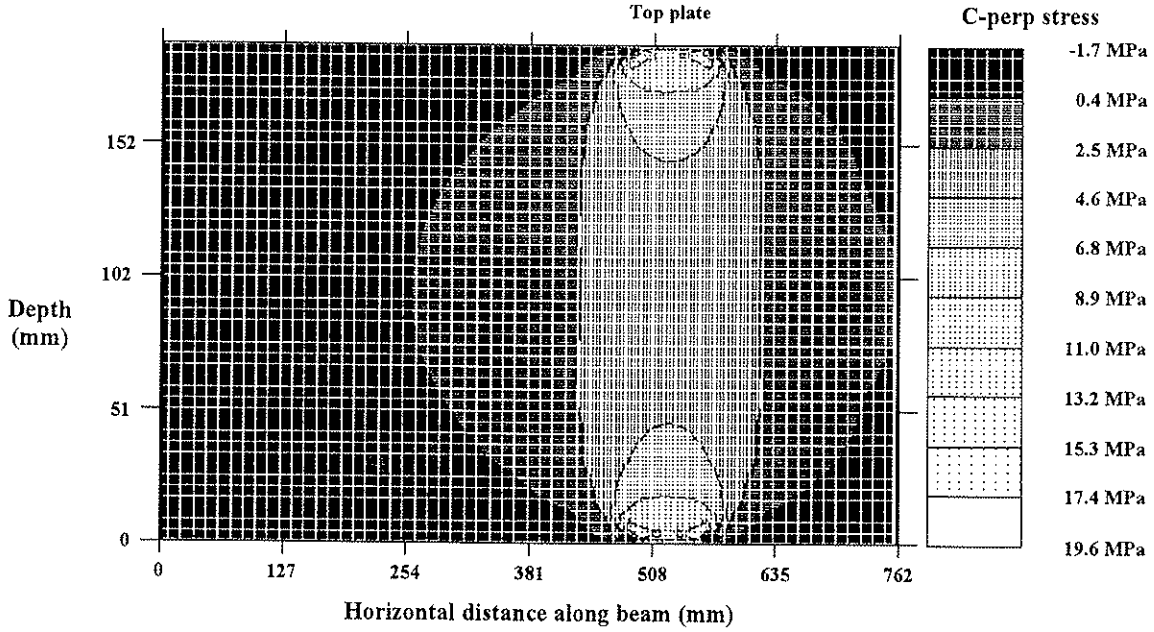

Figure 15. C-perp stress distribution for bearing over interior support (Figure 2 - location 5)

4. CODE IMPLEMENTATION

The findings of this study were considered in the code change proposal for the 1994 edition of CSA-O86.1. The proposed change consists of three major revisions: an increase in C-perp capacity for loading on the wide face; a check on "critical" C-perp loading as opposed to "contact" type loading; and revisions to the characteristic strength values for C-perp design. As part of the review of characteristic C-perp design values, a 10% reduction in the C-perp design value for the Douglas-fir/larch species group was recommended. This was judged to be sufficient to account for the discrepancy in the constant load performance between the Douglas-fir/larch and spruce-pine species group tested in this study.

The code proposal submitted to the CSA Technical Committee On Engineering Design in Wood is included in Appendix I. Highlights of the code change proposal that came about as a result of this study are:

- a C-perp design value increase factor of 15% for C-perp loading on the wide face

- retention of the current design code equation for "contact" type C-perp loading
- addition of the "critical" C-perp check for those cases where load is applied near the support
- use of a 2/3 reduction factor for "critical" loading
- reintroduction of a duration of load factor for C-perp to account for permanent type loading

Some example applications of the proposed design procedure for C-perp are included in Appendix II. These examples cover some of the common C-perp applications found in engineering wood construction. Emphasis has been placed on glued-laminated timber and metal plate connected wood truss applications.

The proposed code change on C-perp design has been submitted and has been voted on by members of the Canadian technical committee on engineering design in wood. However, ballot results and comments have yet to be released. If accepted, this proposal would result in a more rational design procedure for C-perp design in CSA-O86.1-M94. It will also allow the introduction of higher C-perp design values for products such as MSR lumber. Furthermore, the refinement to the design procedures has been done incrementally and without making the procedure more complicated than the current procedures.

5. REFERENCES

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APPENDIX I

Code Change Proposal for CAN/CSA-O86.1-M94
(Sawn Lumber Section Only)

5. Sawn Lumber

5.5. Strength and Resistance

5.5.7. Compressive Resistance Perpendicular to Grain

Factored bearing forces shall not exceed the factored compressive resistance perpendicular to grain in accordance with the provisions of clauses 5.5.7.1 and 5.5.7.2.

5.5.7.1. Effect of All Applied Loads

The factored compressive resistance perpendicular to grain under the effect of all factored applied loads shall be taken as Q_r in the following formula:

$$Q_r = \emptyset F_{cp} A_b K_B K_{Zcp}$$

where

$\emptyset$	=	0.8
F_{cp}	=	$f_{cp} (K_D K_{Scp} K_T)$
f_{cp}	=	specified strength in compression perpendicular to grain (Tables 5.3.1 to Table 5.3.2), MPa
A_b	=	bearing area, mm^2
K_B	=	length of bearing factor (Clause 5.5.7.5)
K_{Zcp}	=	size factor for bearing (Clause 5.5.7.4)

5.5.7.2. Effect of Loads Applied Near a Support

The factored compressive resistance perpendicular to grain against the effect of only those loads applied within a distance from the centre of a support equal to the depth of the member shall be taken as Q'_r in the following formula:

$$Q'_r = \frac{2}{3} \emptyset F_{cp} A'_b K_{Zcp} K_B$$

where

$\emptyset$	=	0.8
F_{cp}	=	$f_{cp} (K_D K_{Scp} K_T)$
A'_b	=	average bearing area (see Clause 5.5.7.3), mm^2

5.5.7.3.

Where unequal bearing areas are used on opposite surfaces of a member, the average bearing area shall not exceed the following:

$$A'_b = b\left(\frac{l_{b1}+l_{b2}}{2}\right), \text{ but} \leq 1.5\ b\ l_{b1}$$

where

$l_{b1} =$ lesser bearing length, mm

$l_{b2} =$ larger bearing length, mm

$b =$ average bearing width (perpendicular to grain), mm

5.5.7.4. Size Factor for Bearing, K_{Zcp}

When the width of the member (dimension perpendicular to the direction of the load) is greater than the depth of the member (dimension parallel to the direction of the load), the specified strength in compression perpendicular to grain may be multiplied a size factor for bearing, K_{Zcp}, in accordance with Table 5.5.7.4.

Table 5.5.7.4

Bearing Width Factor, K_{Zcp}

Ratio of member width to member depth*	Modification factor
1.0 or less	1.00
2.0 or more	1.15

* Interpolation applies for intermediate ratios.

5.5.7.5. Length of Bearing Factor, K_B

When lengths of bearing or diameters of washers are less than 150 mm, specified strengths in compression perpendicular to grain may be multiplied by a length of bearing factor, K_B, in accordance with Table 5.5.7.5 provided that:

(a) any part of the bearing area is more than 75 mm from the end of the member; and (b) bearing areas do not occur in positions of high bending stresses.

Table 5.5.7.5

Length of Bearing Factor, K_B

Bearing length measured parallel to grain, or washer diameter (mm)	Modification factor, K_B
12.5 and less	1.75
25.0	1.38
38.0	1.25
50.0	1.19
75.0	1.13
100.0	1.10
150.0 or more	1.00

APPENDIX II

Applications Using Proposed Provisions for C-Perp Design

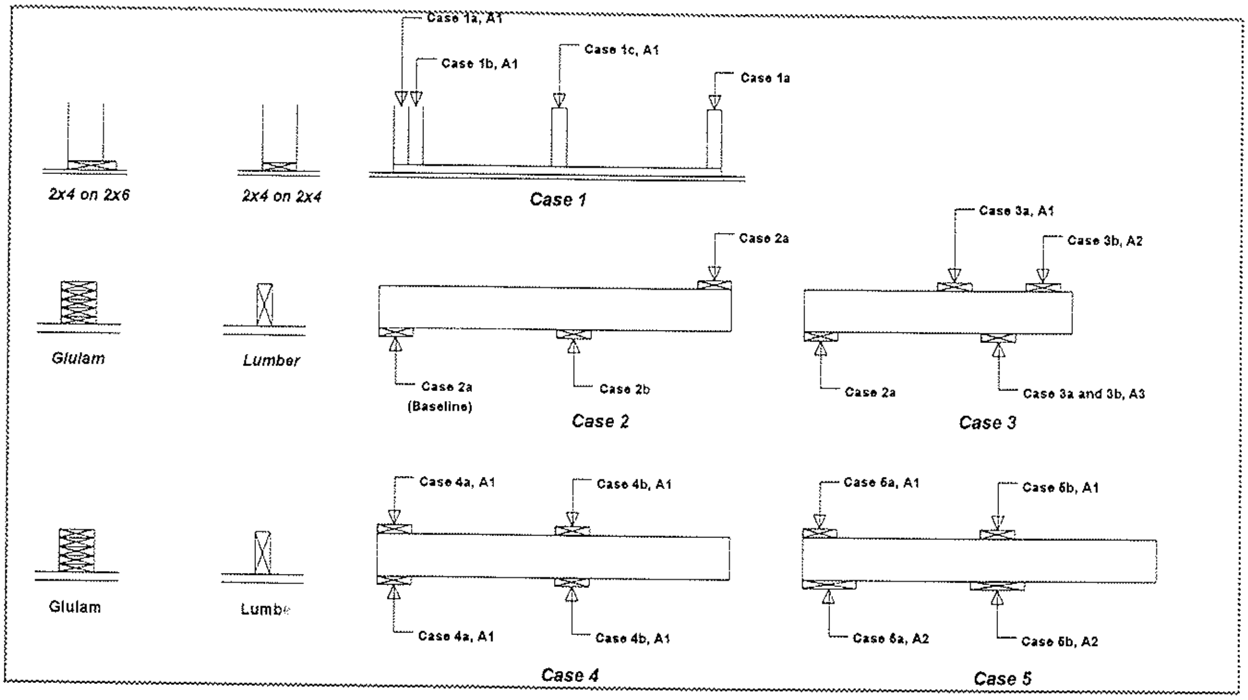

Figure 1. Beam Applications

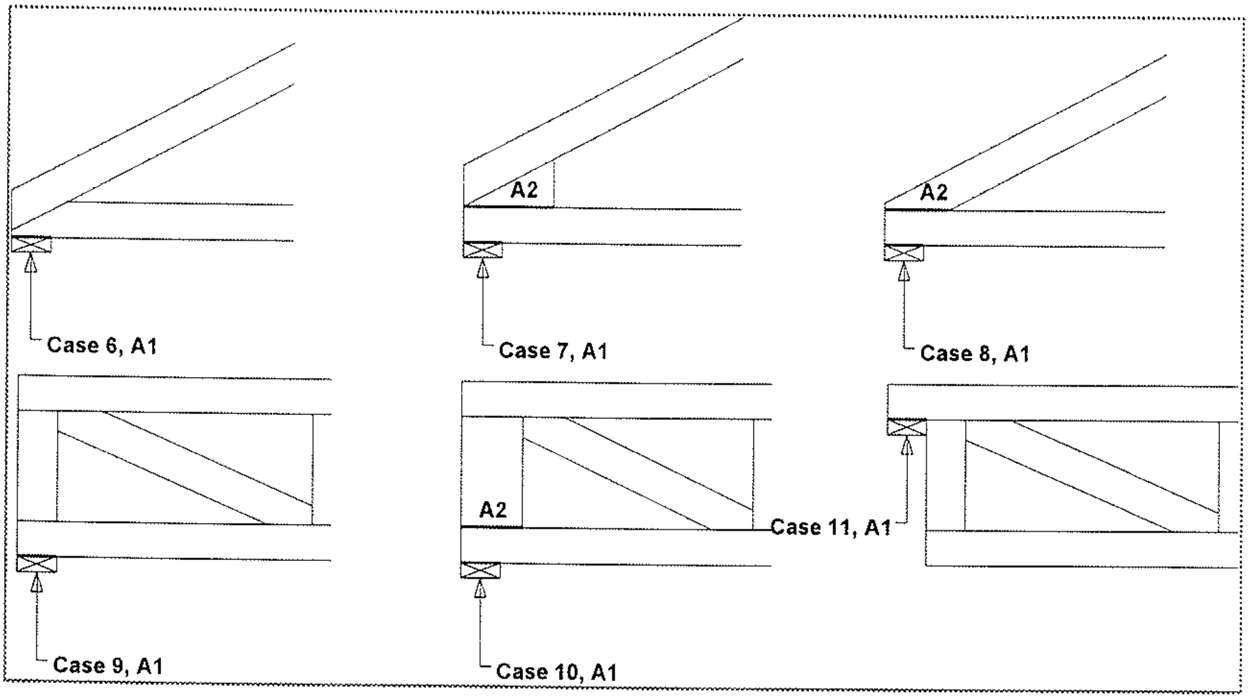

Figure 2. Truss Applications (lumber on edge)

Description	A'_b	Sawn Lumber K_B [1]	Sawn Lumber K_{Zcp}	Sawn Lumber Q_r Ratio [2]	Sawn Lumber Q'_r Ratio [2]	Glulam K_{Zcp}	Glulam Q_r Ratio [2]	Glulam Q'_r Ratio [2]
Case 1a - Stud on bottom wall plate at wall opening	$1.5A_1$	1.0	1.15	1.15	1.15	1.15	1.15	1.15
Case 1b - Stud adjacent to stud (1a) on bottom wall plate	$1.5A_1$	1.0	1.15	1.15	1.15	1.15	1.15	1.15
Case 1c - Stud on bottom wall plate away from wall opening	$1.5A_1$	1.25	1.15	1.15	1.43	1.15	1.15	1.15
Case 2a - Bearing at an exterior support	A_1	1.0	1.0	Baseline	NA	1.15	1.15	NA
Case 2b - Bearing at an interior support	A_1	1.25	1.0	1.25	NA	1.15	1.15	NA
Case 3a - Multiple loads applied near an interior support (checking A_1)	$\frac{A_1 + A_3}{2}$	1.25	1.0	1.25	0.83	1.15	1.15	0.77
Case 3b - Multiple loads applied near an interior support (checking A_2)	$\frac{A_2 + A_3}{2}$	1.0	1.0	1.0	0.67	1.15	1.15	0.77
Case 4a - Bearing over an exterior support, $A_1=A_2$	A_1	1.0	1.0	1.0	0.67	1.15	1.15	0.77
Case 4b - Bearing over an interior support, $A_1=A_2$	A_1	1.25	1.0	1.25	0.84	1.15	1.15	0.77
Case 5a - Bearing over an exterior support, $A_1 \neq A_2$	$\frac{A_1 + A_2}{2}$	1.0	1.0	1.0	0.67	1.15	1.15	0.77
Case 5b - Bearing over an interior support, $A_1 \neq A_2$	$\frac{A_1 + A_2}{2}$	1.25	1.0	1.25	0.84	1.15	1.15	0.77
Case 6 - Pitched chord truss detail - bottom chord scarf cut	A_1	1.0	1.0	1.0	NA	1.15	1.15	NA
Case 7 - Pitched chord truss detail [3] - raised heel $A_1 \neq A_2$	$\frac{A_1 + A_2}{2}$	1.0	1.0	1.0	0.67	1.15	1.15	0.77
Case 8 - Pitched chord truss detail [3] - top chord scarf cut $A_1 \neq A_2$	$\frac{A_1 + A_2}{2}$	1.0	1.0	1.0	0.67	1.15	1.15	0.77
Case 9 - Bottom chord supported flat truss [3] - $A_1=A_2$	A_1	1.0	1.0	1.0	0.67	1.15	1.15	0.77
Case 10 - Bottom chord supported flat truss [3] - $A_1 \neq A_2$	$\frac{A_1 + A_2}{2}$	1.0	1.0	1.0	0.67	1.15	1.15	0.77
Case 11- Top chord supported flat truss [3]	A_1	1.0	1.0	1.0	NA	1.15	1.15	NA

NOTES:

[1] For C-perp away from the ends of the member, K_B generally equal to 1.25 for bearing between dimension lumber, and 1.0 for bearing between glulam.

[2] RATIO is capacity relative to "Baseline" case (i.e. Bearing at exterior support for sawn lumber). NA - not applicable.

[3] When checking Q'_r for truss applications, generally only the loads from the top chord need to be considered.

CIB-W18/27-6-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

SIZE EFFECTS IN TIMBER : NOVELTY NEVER ENDS

by

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JULY 1994

Introduction.

Size effects in timber have been widely investigated by different authors. These authors apparently find results which are conflicting, and the end result is a compromise among them which has been adopted for code purposes (Eurocode 5). Recently, a wide number of species has been investigated in France, using a forest based sampling. The results were analysed and provided additional information to the size effect problem. The purpose of this paper is to discuss the apparent discrepancy of the size effect.

Basic Theory.

The weakest link theory has been developed by Pierce (1926), Tucker (1927) and Weibull (1939) who studied brittle materials, including concrete. This theory says that "when subjected to tension, a chain is as strong as its weakest link". To explain this theory, consider a reference volume subjected to tension. The probability of failure P_f of this volume is defined by :

$$P_f = F(\sigma) = Probability\,(Strength \leq \sigma) \tag{1}$$

where F is the cumulative distribution of the strength.
Now consider a series assembly of N reference volumes. This system survives if each of the members survives, i.e. :

$$\begin{aligned} P_S &= P_S(1)\ P_S(2)\ldots P_S(N) \\ &= [1-P_f(1)]\ [1-P_f(2)]\ldots[1-P_f(N)] \end{aligned} \tag{2}$$

where P_S is the probability of survival of the system and $P_S(i)$ is the probability of survival of an individual element i. From equation (2) and assuming that reference volumes have the same probability of failure and that the events of failure are independent in all reference volumes, the probability of failure of the system can be evaluated:

$$P_f = 1-P_S = 1-[1-F(\sigma)]^N = 1-e^{N\,Log(1-F(\sigma))} \simeq 1-e^{-N\,F(\sigma)} \tag{3}$$

Now, assume that the lower tail of F has been fitted by a power model, i.e.

$$F(\sigma) = a(\sigma-\sigma_0)^k \tag{4}$$

The probability of failure is then expressed by :

$$P_f(\sigma) = 1-e^{-Na(\sigma-\sigma_0)^k} = 1-e^{-V(\frac{\sigma-\sigma_0}{m})^k} \tag{5}$$

This model is known as the 3 parameter Weibull model. It is also well known as the 2 parameter Weibull model when $\sigma_0 = 0$. The theory can be used to explain the size effect in tension. Consider a volume V_1 which has a given probability of failure $P_f(\sigma_1)$ at level σ_1 and a volume V_2 which has a given probability of failure $P_f(\sigma_2)$ at level σ_2. If the characteristic strengths of these two volumes are compared, the following is obtained :

$$P_f(\sigma_1) = P_f(\sigma_2) \Rightarrow V_1\left(\frac{\sigma_1}{m}\right)^k = V_2\left(\frac{\sigma_2}{m}\right)^k \Rightarrow \frac{\sigma_2}{\sigma_1} = \left(\frac{V_1}{V_2}\right)^{1/k} \tag{6}$$

This equation is the basic explanation of size effect. The two important assumptions of this theory are remembered below :

(1) The theory is valid for a brittle material.

(2) It assumes statistical independance between events of failure, i.e. material has to be statistically homogeneous.

Research results.
A vast amount of data has been published to explain size effect for structural size timber. These results are sometimes conflicting (Barrett and Lam, 1992; Madsen, 1992). The theory has been succesfully applied to tension parallel and perpendicular to grain (Barrett,1974; Colling,1986), and to shear (Foschi and Barrett,1976; Foschi,1985; Colling,1986). But in the case of compression, and particularly in bending which is a mixed mode of failure between tension and compression, the use of this theory is debatable. When tests are conducted for constant span to depth ratios in bending, the size effect is a combination of a depth effect and a length effect (Barrett and Fewell, 1990). These effects cannot be identified separately.

In Table 1, different factors for bending size effects are recorded :

- a length factor S_L (for beams tested at constant depths) which is calculated from:

$$\frac{\sigma_2}{\sigma_1} = \left(\frac{L_1}{L_2}\right)^{1/k_L} = \left(\frac{L_1}{L_2}\right)^{S_L} \quad (7)$$

- a depth factor S_h (for beams tested at constant spans) which is calculated from:

$$\frac{\sigma_2}{\sigma_1} = \left(\frac{h_1}{h_2}\right)^{1/k_h} = \left(\frac{h_1}{h_2}\right)^{S_h} \quad (8)$$

- a "size factor" S_R (for beams tested at constant span to depth ratio, i.e. $L_i = k\,h_i$), which, according to the combination of equations (11) and (12), is calculated from:

$$\frac{\sigma_2}{\sigma_1} = \left(\frac{L_1}{L_2}\right)^{S_L}\left(\frac{h_1}{h_2}\right)^{S_h} = \left(\frac{h_1}{h_2}\right)^{S_L}\left(\frac{h_1}{h_2}\right)^{S_h} = \left(\frac{h_1}{h_2}\right)^{S_h+S_L} = \left(\frac{h_1}{h_2}\right)^{S_R} \quad (9)$$

Author	S_L	S_h	S_R
Barrett and Fewell 1990	0,17	0,23	0,40
Madsen 1992	0,20	0,0	0,20
Ehlbeck and Colling 1990	0,15	0,15	0,30

Table 1 : ***Size factors for bending.***

In order to qualify French grown species, a wide research program has been carried out in France on several species (Rouger & al., 1993). A forest based sampling lead to an important database (more than 10000 full size tests). As an example, the size effect for Spruce and Fir has been found to the following :

S_R	Raw material	Visually graded material	Material graded according to MOE
CF18	0,1	0,01	0,23
CF22		-0,1	
CF30		-0,5	0,20

Table 2 : *Size effect in bending for French Grown Spruce & Fir*

The corresponding sample sizes and cross-sections are given below :

Grading Method	Grades	Sizes of the specimens			
		40x100x2000	50x150x3000	65x200x4000	TOTAL
Raw Material		1175	1065	160	2400
Visual Grading	CF18	16	248	49	313
	CF22	669	452	59	1180
	CF30	180	63	4	247
Grading based on MOE	CF18	406	261	61	728
	CF30	657	761	94	1512

Table 3 : ***Sample sizes of the French data***

The size effect has been shown by many researchers to be very variable, mainly because it is extremely difficult to match the different cross-section size test samples. This difficulty reflects the different production procedures (material, selection, sawing pattern and grading) used in sawmills so that what is considered to be matched samples in one country may not be in another.

Despite the variability of the size effect, it has been shown to be present from timber supplied from many countries around the world, and structural codes have to deal with it. Furthermore, when a strength class system is used, a common size effect has to be assumed for all timber, resulting in a compromise. Table 2 shows the difference in the size effect for visual and machine grading for the French timber. Data from other sources (Barrett and Fewell, 1990) for many samples and different species of visually graded timber, show a very different size effect. It has been said in the past that the size effect for visual grading is the important one, because grading machines can be adjusted to give a corresponding effect. This is true only to a limited extent because if the machine settings (MOE) have to be adjusted to obtain a common size effect, then the different sizes will have different MOE's which may become critical and lead to inefficient grading.

Discussion about the Weibull concept.

(1) The theory is valid for a brittle material.

For 1160 specimens, the failure modes in bending have been recorded. In order to simplify the analysis, three failure modes have been defined :

Mode A : brittle failure　　Mode B : bi-linear curve　　Mode C : ductile failure.

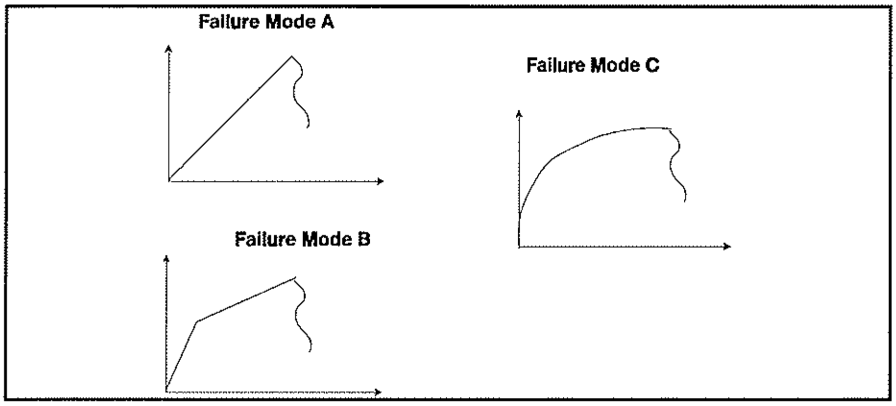


Figure 1 : *Modes of failure.*

The proportions of these failure modes for the raw material and for the visually graded material are reported in the following figure.

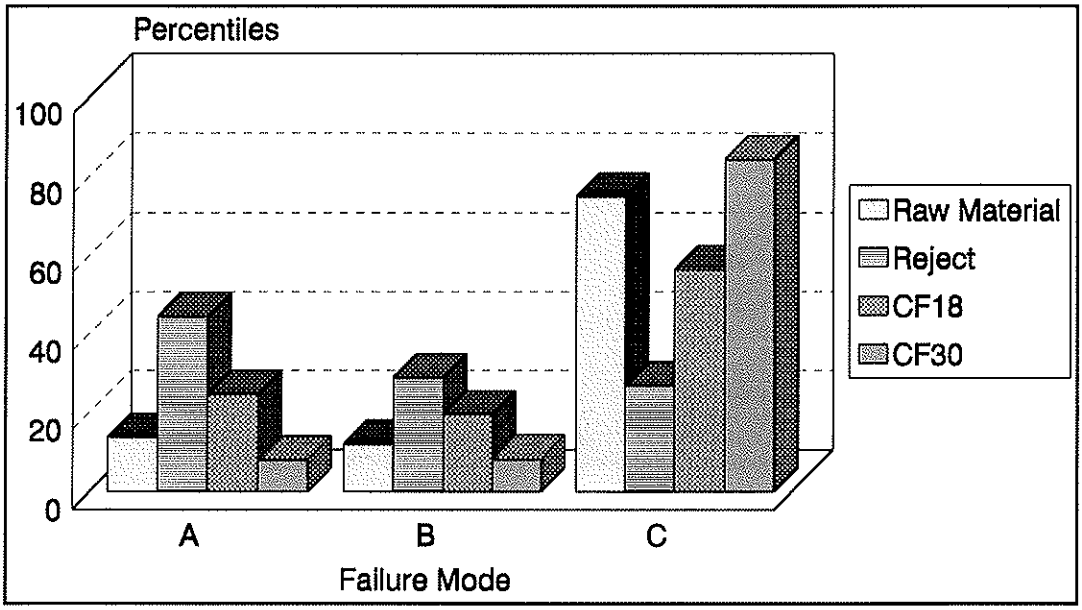


Figure 2 : *Modes of failure for Spruce & Fir.*

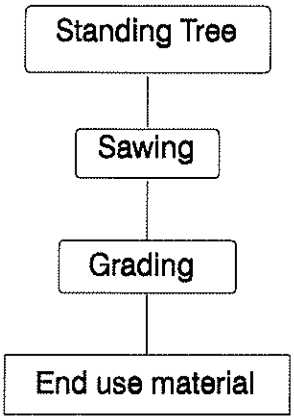
It can be observed that failure mode A only represents 10% for the raw material and a maximum of 40% for the visually graded material. In any case, the basic theory of Weibull is questionnable.

Another interesting point is that brittle failure mode tends to disappear as the quality of the material increases, and that ductile failure mode tends to increase as the quality increases. It seems necessary to formulate new theories for ductile materials.

(2) Weibull theory assumes statistical independance between events of failure, i.e. defects have to be randomly located.

This assumption might be true for Spruce, Fir or Douglas Fir but certainly not for Pines (Maritime Pine, Scots Pine) in which knots are clustered. For visually graded material, the probability of finding bigger knots increases with the size, and therefore, there is a statistical dependance between the volume and the events of failure.

Effects of sampling
From the tree to the end material used in construction, the following scheme is followed :



When investigating size effects in timber, the authors have sampled the material in different ways :

(a) ingrade tested specimens,
(b) ungraded specimens, but sampled in the sawmills
(c) ungraded specimens, randomly sampled in the forest.

In case (a), the effect of visual grading has been already explained, and the use of Weibull theory is debatable.
In case (b), this bias dos not exist, but the sawing pattern might have an influence on the apparent size effect (as explained below).
In case (c), the size effect can be investigated regardless of any bias.

The sawing pattern has been analysed on the French Database, in three ways :

- influence of the distance from the pith on the strength,
- influence of the distance from the butt on the strength,
- influence of the log diameter on the strength.

The distance from the pith has been recorded for 1263 full size specimens. This distance is marked by a target which is painted on each log before sawing. The precision of this measurement is +- 2,5 cm. A linear regression between this distance (d) and the strength has been formulated :

$$f_{m,k} = a.d + b \qquad (10)$$

with

d : distance from the pith (in mm)
a : slope = $1.66\ 10^{-2}$
b : intercept = 23

The correlation coefficient is : $\rho = 0.77$

This influence is illustrated in Figure 3.

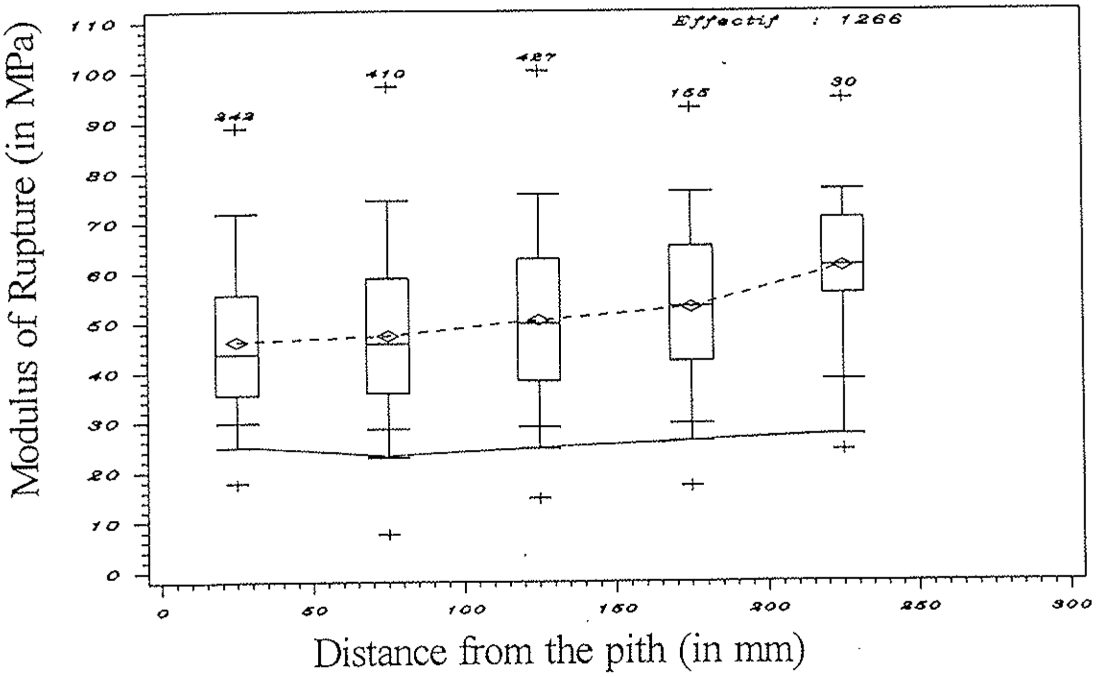


Figure 3 : ***Influence of the distance from the pith on the strength***

If we assume that depth=150 mm has been cut at d=25 mm and depth=100 mm has been cut at d=250 mm, it leads to an apparent size effect :

$$S_R = \frac{\ln \frac{23.4}{27.2}}{\ln \frac{100}{150}} = 0.371 \qquad (11)$$

The distance from the butt has been recorded for 1251 specimens. A linear regression between this distance (h) and the strength has been formulated :

$$f_{m,k} = a.h + b \qquad (12)$$

with

h : distance from the butt (in m)
a : slope = -0.43
b : intercept = 28.5

The correlation coefficient is : $\rho = -0.93$

This correlation is illustrated in Figure 4.

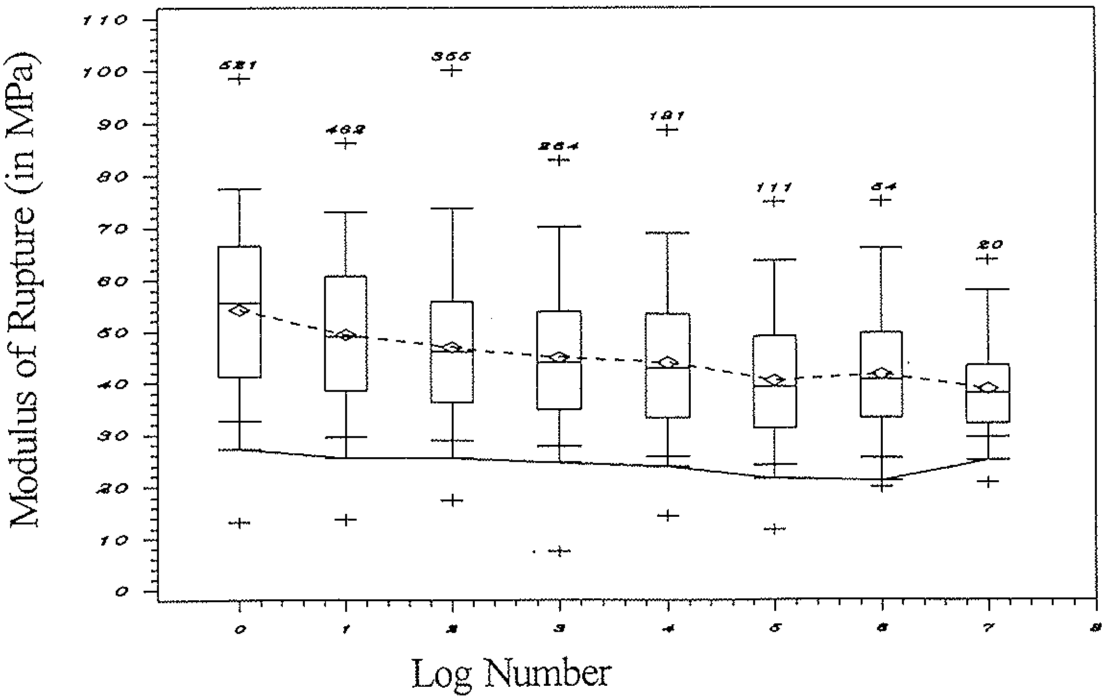


Figure 4 : ***Influence of the distance from the butt on the strength***
(The distance from the butt is equal to 3,30 m x Log Number)

If we assume that depth=150 mm has been cut at h=1m and depth=100 mm has been cut at h=20 m, it leads to an apparent size effect :

$$S_R = \frac{\ln \frac{28}{20}}{\ln \frac{100}{150}} = -0.83 \qquad (13)$$

The log diameter has been recorded for 1700 specimens. Linear regression has not been possible because diameters were classified into three classes. But it is obvious in Figure 5 that the log diameter has a negative effect on the strength.

If we assume that depth=150 mm has been cut in a log of diameter 65 cm and depth=100 mm has been cut in a log of diameter 25 cm, it leads to an apparent size effect :

$$S_R = \frac{\ln \frac{22}{30}}{\ln \frac{100}{150}} = 0.76 \qquad (14)$$

Although Figure 4 indicates that strength decreases as log number increases, the influence of log number on the size effect is probably insignificant because, in production, the sawing of small and large cross-section pieces is not restricted to logs from a particular distance from the butt. In addition, in some sawmills butt logs are reserved for furniture and other special products.

However, the distance from the pith and the log diameter greatly influence the location of the boards. Small cross-section pieces tend to be sawn from small trees and far from the pith, whereas large cross-section pieces tend to be sawn from large trees and close to the pith. As a consequence, size effect would be significantly influenced by the sawing pattern.

The French test data insure a uniform sampling within the tree, and do not correspond to normal production sawing patterns. Therefore, this data helps to isolate the sawing pattern influence, resulting in a size effect much lower.

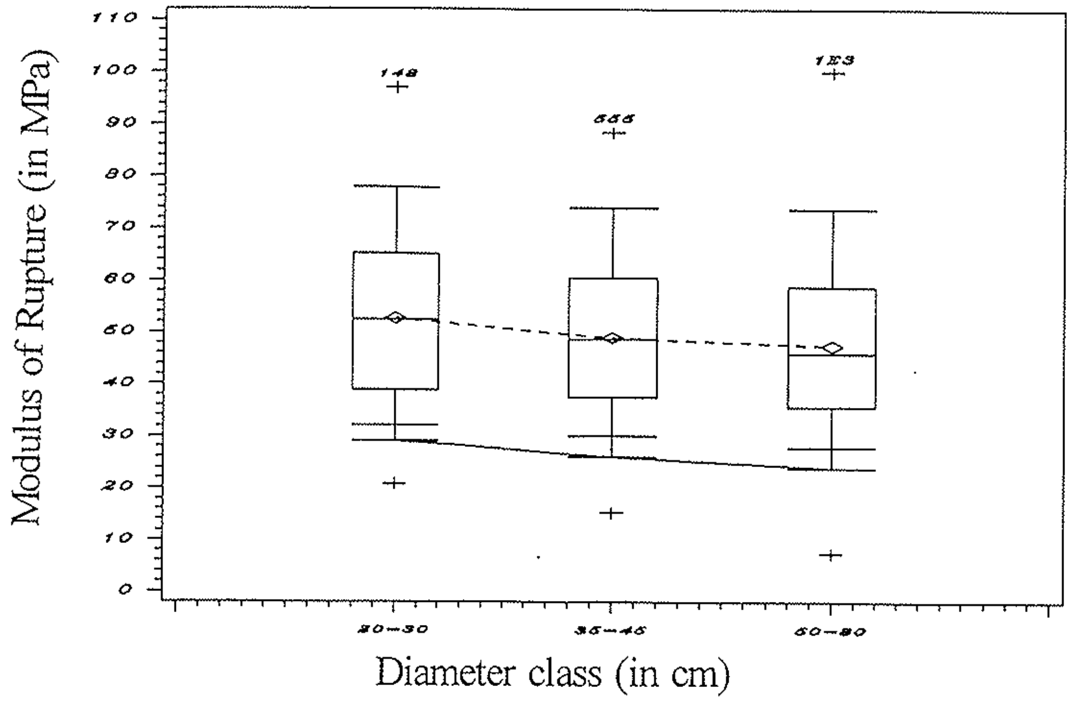


Figure 5 : *Influence of the log diameter on the strength.*

Conclusions :

In this paper, we demonstrated that the basic assumptions of the Weibull theory (brittle failure, statistical homogeneity) were not verified. We also demonstrated that the so-called "size effect" could be significantly influenced by the sawing pattern and by the visual grading method. For this reason, further research needs to be achieved to isolate and quantify the different things that affect the size effect. A theory that undertakes the ductile behavior and the statistical heterogeneity of timber material has to be formulated. Further testing programs also need to keep a precise track of the sawing pattern, the log selection and the grading practice. These elements will help to provide a compromise in design codes.

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CIB-W18/27-6-3

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

COMPARISON OF FULL-SIZE SUGI (CRYPTOMERIA JAPONICA D.DON) STRUCTURAL PERFORMANCE IN BENDING OF ROUND TIMBER, TWO SURFACES SAWN TIMBER AND SQUARE SAWN TIMBER

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MEETING TWENTY - SEVEN

SYDNEY

AUSTRALIA

JULY 1994

Paper to be presented at the 27th CIB W18A Meeting, Sydney, July,1994

Comparison of Full-Size Sugi(*Cryptomeria japonica* D.Don) Structural Performance in Bending of Round Timber, Two Surfaces Sawn Timber and Square Sawn Timber.

by

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1. Introduction

More than 1 million ha man-made forest has been developing in Japan, of which about 45% in planted area and 60% of volume in stock are covered by sugi (*Cryptomeria japonica* D.Don). Due to the various economical and social reasons, ordinary sugi could not find reasonable market. Foresters have a tendency not to cut their trees when the market is not so good. Recently it is pointed out that medium size sugi log, which is regarded as from 22 cm to 28 cm at the butt end diameter, has relatively low in market price. To utilize this size of sugi as a construction material not only for building, but also for public works, is one of the target for Japanese forester.

To use the relatively large diameter effectively, it was decided to investigate the structural performance of round timber and also two surfaces sawn timber (T.S.S.T.), which is called in Japanese as Taiko (Drum in English), although both cross sections have been using for long time in traditional, non-engineered timber construction, there are few technical data available. It was expected that round timber may has relatively higher strength because there is few slope of grain when compared with sawn timber.

The main purposes of this experimental work were set as below;

1) To know the effect of moisture content on bending performance in round timber.

2) To compare the bending performance among round timber, two surfaces sawn timber in edgewise and in flatwise and square sawn timber.

3) To get fundamental information for non-destructive evaluation method on bending performance in round timber.

2. Test procedure

2.1 Material

From Tokushima prefecture in Shikoku island, 102 medium size sugi logs were prepared for specimens. The tree ages were from about 50 to 60 year. The most common four meter length logs with 22 cm butt end diameter were sorted from the 2nd, 3rd and 4th of cutting position in trunk.

In green condition, the fundamental vibration frequency and density were measured to calculate modulus of elasticity Efr(tf/cm^2). Using these Efr values, three groups were sorted, namely green round timber group, air-dry round timber group and air-dry two surfaces sawn timber group.

2.2 Bending test method

In Fig.1, an outline of bending test methods were shown. In the case of round timber and two surfaces sawn timber(T.S.S.T.) at edgewise bending test, the total span was taken as 3900 mm. The width of T.S.S.T. was taken 120 mm. The concentrated loading was applied, the ratio of span by mid-span diameter were from 16 to 17. After finishing T.S.S.T. edgewise bending test, two surfaces sawn timber in flatwise and square sawn timber with 120 mm by 120 mm in cross section were cut from the non-destructive section in the length of 1800 mm. Then span by depth ratio was taken as 14, concentrated load was applied.

3 Results and discussion

3.1 Results of sorting

Each group had 34 specimens with the average Efr as 92 (tf/cm^2) and 12% in C.V. The number of annual rings at the butt end was 40 to 41. The degree of taper, which is expressed the deference in diameter along one meter length in unit in cm/m, was about 1.07.

3.2 Results of bending test

3.3.1. Moisture content effect on MOE and MOR in round timber

Moisture content of round timber in green condition was about 135%. Whereas round timber in dry condition was 20.8%, T.S.S.T. in edgewise, flatwise and square sawn timber were 12% in moisture content..

In Fig.2, the normalized rank of MOR of round timber in green condition and dry condition were shown. It is noted that MOR in both green and dry conditions were almost the same less than 50%ile, which is quite resemble to the results in flat square sawn timber's case.(Nakai et al. 1992)
Assuming the fiber saturation point of sugi as 28%, the rate of change per 1% moisture content were obtained as 0.78% and 0.71% respectively for MOE and MOR.

3.3.2. Effect of cross section shape on bending performance

Using the moisture content effect factors obtained in this test, adjusted data to 15% moisture content were plotted in Fig.3. Also Table 1 showed the over-all results obtained in this test. From which it is noted that the average modulus of elasticity in Round Timber : T.S.S.T. edgewise : T.S.S.T. flatwise : Square S.T. = 1.14 : 1.20 : 1.03 : 1.00. was obtained. It is interesting that the same relationship were obtained (NAKAI ,1990) in the case of about 20 year old sugi thinning tree, which showed 1.23 : 1.18 : 1.05 : 1.00 respectively, although the absolute value of MOE were lower than that obtained in this case. The modulus of elasticity in 50 to 60 year old tree was 1.4 to 1.6 times of that obtained in 20 year old thinning tree.

Regarding the 5th percentile value of MOR, the ratio of 1.32 : 1.21 : 1.13 : 1.00 were obtained for Round Timber, T.S.S.T. edgewise, T.S.S.T. flatwise and Square S.T. respectively. From this results, it is clear that for round timber and two surfaces sawn timber in edgewise could be allocated more higher characteristic values than square sawn timber.

3.3.3. Non-destructive evaluation method on bending performance in round timber.

In Fig. 4, the relationship between Efr and MOR of round timber both in dry condition was shown. It is obvious that Efr could be used as a sorting parameter for strength of round timber.

4 Conclusion

Conducting full-size bending test in round timber , two surfaces sawn timber and square sawn timber, the following results were obtained.

1) The rate of change per 1% moisture content change in round timber were obtained as 0.78% and 0.71% respectively for MOE and MOR.
2) The average modulus of elasticity in Round Timber : T.S.S.T. edgewise : T.S.S.T. flatwise : Square S.T. = 1.14 : 1.20 : 1.03 : 1.00. was obtained.
In the case of the 5th percentile value of MOR, the ratio of 1.32 : 1.21 : 1.13 : 1.00 were obtained for Round Timber, T.S.S.T. edgewise, T.S.S.T. flatwise and Square S.T. respectively.
3) Efr could be used as a sorting parameter for strength of round timber.

5 References

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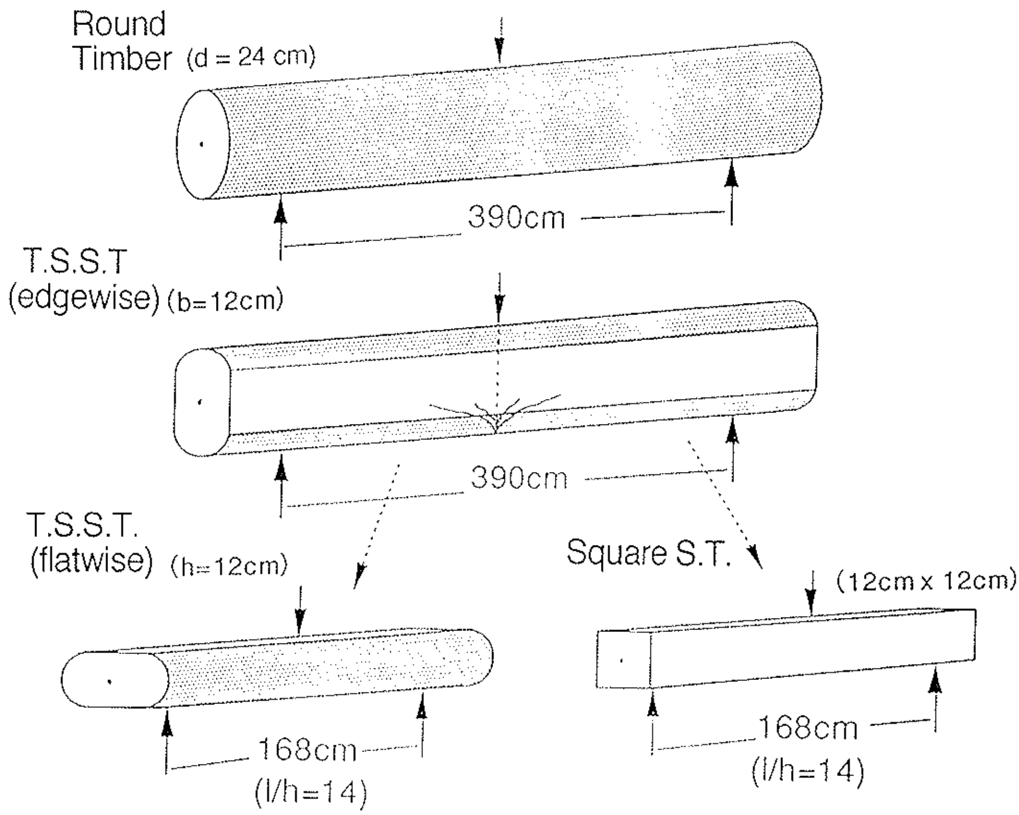

Fig. 1. Outline of bending test

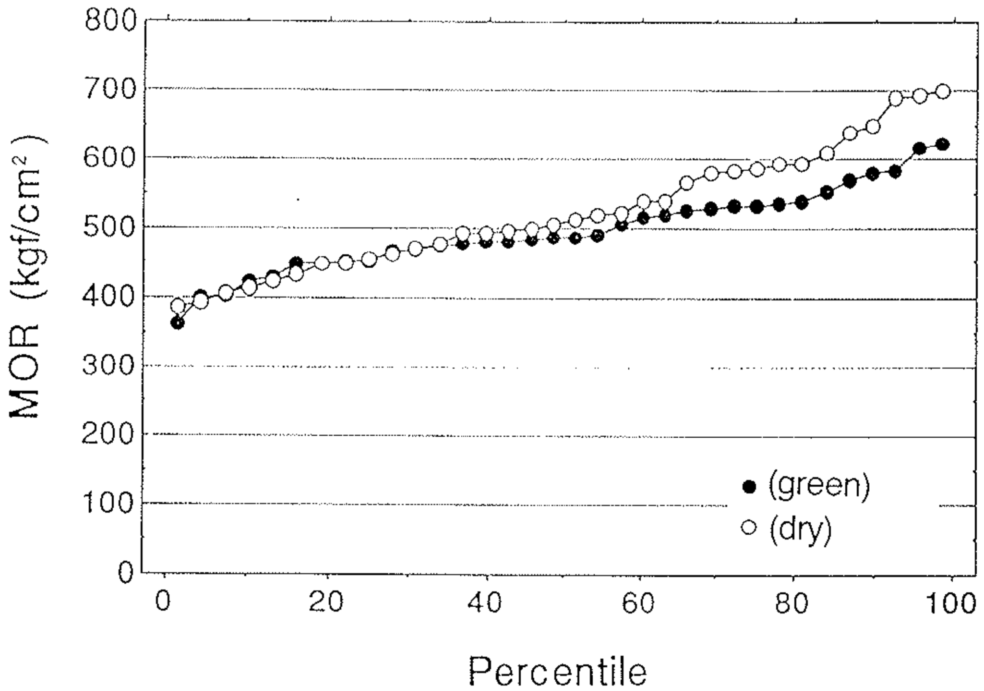

Fig. 2. Normalized MOR rank of round timber in green and dry conditions

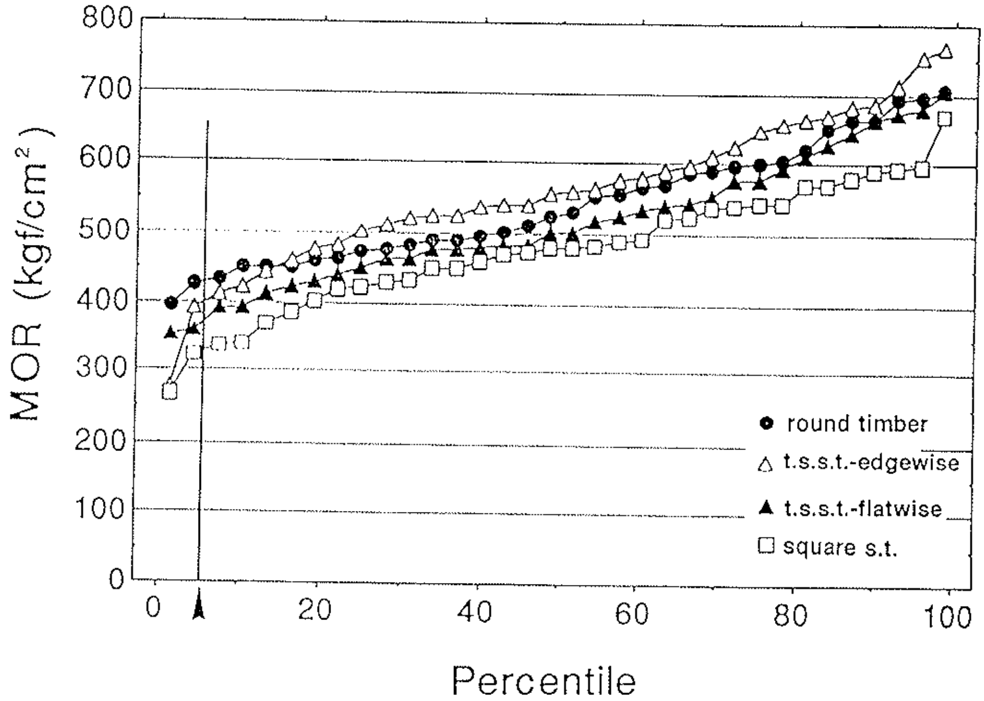

Fig. 3. Normalized MOR rank after adjusted to 15% M.C.

Table 1. Effect of cross section shape on bending performance

		Cross Section			
		Round Timber	T.S.S.T. (edgewise)	T.S.S.T. (flatwise)	Square S.T.
MOE(tf/cm2)					
	mean	100	106	91	88
	(ratio)	(1.14)	(1.20)	(1.03)	(1.00)
MOR (kgf/cm2)					
	mean	543	562	517	478
	5th%ile	429	396	367	326
	(ratio)	(1.32)	(1.21)	(1.13)	(1.00)

adjusted to 15% M.C.

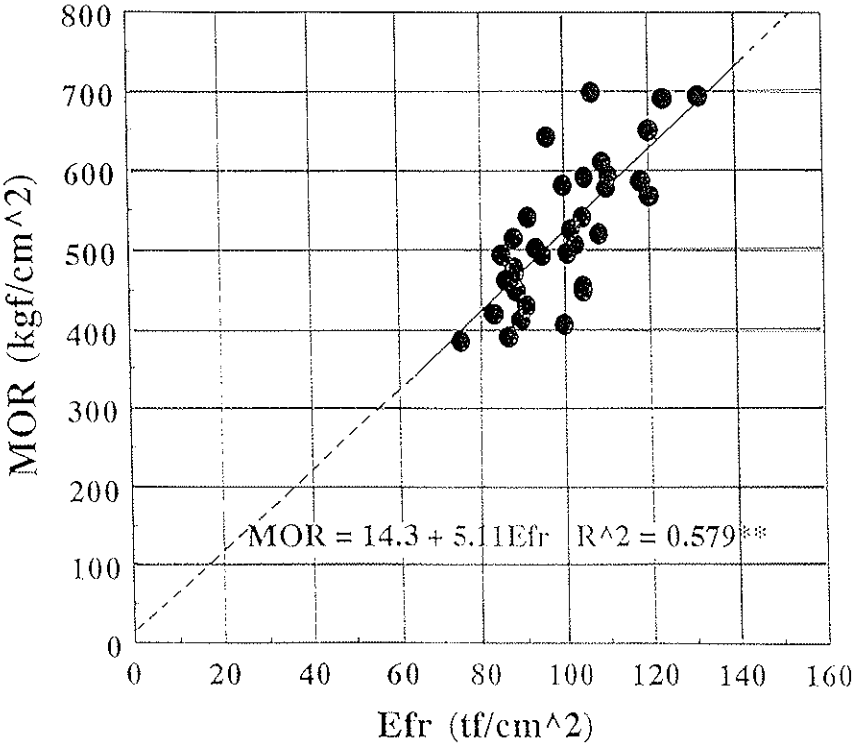

Fig. 4. Relationship between Efr and MOR(dry) in round timber

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

GLULAM ARCH BRIDGE AND DESIGN OF IT'S MOMENT-RESISTING JOINTS

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MEETING TWENTY - SEVEN

SYDNEY

AUSTRALIA

JULY 1994

Paper to be presented at The 27th Meeting of CIB-W18, Sydney, Australia, 1994.

Glulam Arch Bridge and Design of It's Moment-Resisting Joints

by

Kohei Komatsu[*1] and Seizo Usuki[*2]

ABSTRACT

A two-hinged glulam arch bridge having a clear span length of 23m and width of 5.0m has just completed at Hirota-village, Ehime prefecture, Shikoku island, Japan in May 1994. In this report, some characteristic points of this glulam bridge are first outlined. Then, a newly proposed design procedure for calculating a glulam moment-resisting joint using drift-pins with steel insert-plates is explained. Finally, edge-wise bending properties tested using 1/2 reduced model specimens are showed and joint stiffness, strength and load factor etc. are discussed.

1. INTRODUCTION

In recent Japan, about 250 timber bridges have been constructed since 1987. Most timber bridges, however, are pedestrian bridges and only a several bridges were designed for heavy traffic roads. Among these rare cases, one of the most dramatic timber bridges will be "You-kura" bridge completed in 1992 at vicinity of new Hiroshima air-port. This three spans diagonal tension bridge, having a total span length of 145m and glulam deck truss, was designed for bicycle road racing, but up to 14tons vehicle load is permitted for inspection or/and maintenance works.

The first timber bridge in Japan which can sustain 20tons vehicle load is "Kami-no-mori (meaning the Forest of Lord)" bridge just completed at Hirota village, Ehime prefecture , Shikoku island, Japan in May 1994. This glulam two-hinge arch bridge was designed by the members of timber bridge committee organized in Japan Housing and Wood Technology Center. Professor Usuki, who took part of structural design of this arch bridge, is to present about the structural design of this bridge in '94 PTEC (Usuki et al 1994). The first named author of this report is to present about construction outline and the edge-wise bending performance of moment-resisting joints used in the on-site arch-rib connections of "Kami-no-mori" bridge.

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2. OUTLINE OF ARCH BRIDGE

Figure 1 shows a general view of "Kami-no-mori" bridge (Usuki et al 1994). The span length is 23.0m and the bridge length is 26.36m. Almost all structural members, except for the pre-stressed laminated timber (PSLT) deck member, are glulam made of Hirota-village grown Sugi (Japanese cedar, *Cryptomeria japonica* D.Don). For PSLT deck member, Southern Pine sawn lumber imported from USA was used.

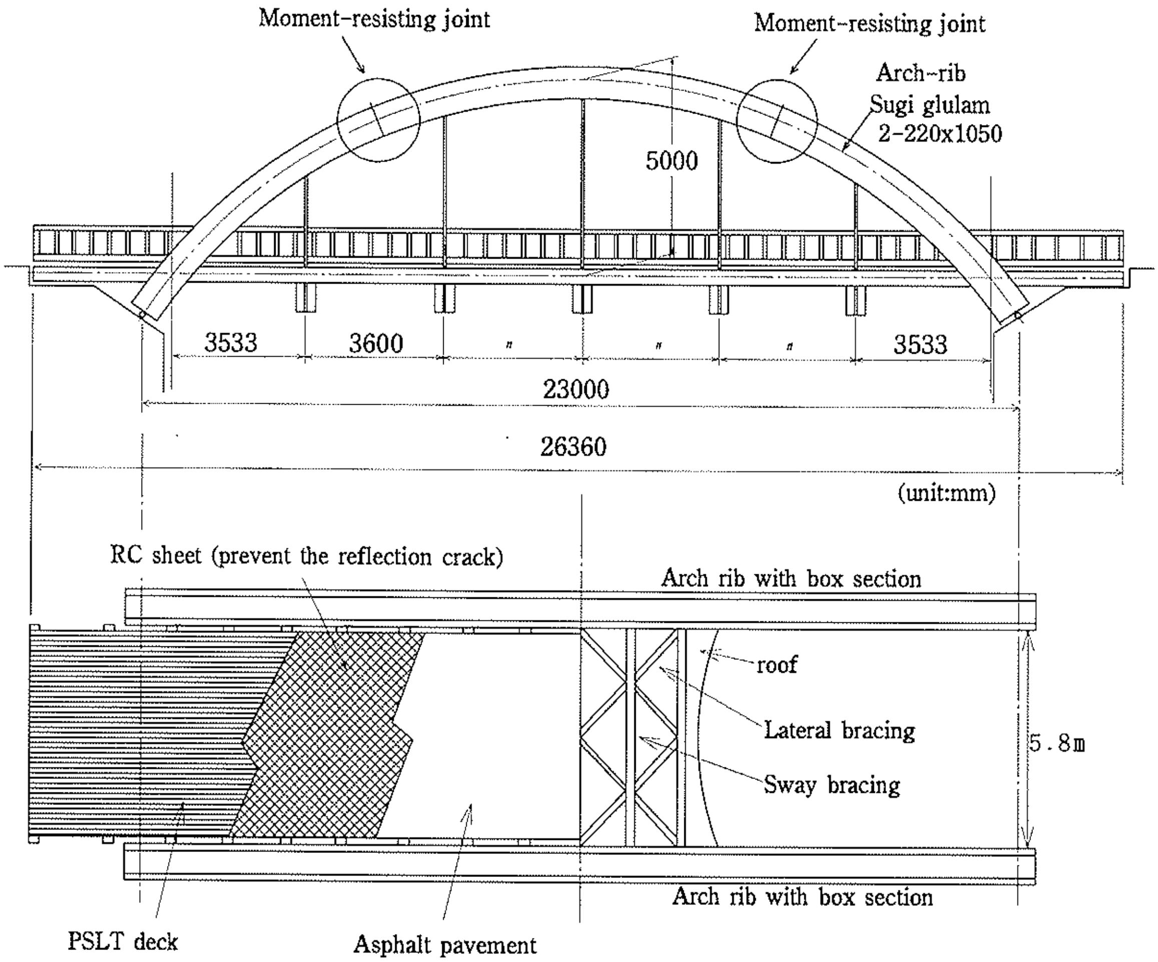


Fig.1 General view of 2-hinged glulam arch "Kami-no-mori" bridge.

One of the most characteristic point of this bridge is that main glulam members were preliminary pressure treated by a water-borne preservative A*1 before the lamination gluing.

This water-borne preservative A*1 is relatively new wood preservative developed by a Japanese company. From a preliminary experiment, it was confirmed that this preservative affect scarcely strength and durability of lamination gluing subjected to using resorcinol resin adhesive (Nakamura 1993). By this chemical treatment and additional structural innovation to be mentioned in the latter section, durability of this bridge is expected to become more than 30 years.

*1 : Xyence Corporation patent: Wood Preservative Composition which comprise quaternary ammonium compounds and polyalkylenglycols.

Figure 2 shows a cross sectional view of the bridge. Arch-ribs are composed of double glulam vertical beams of 220mm wide and 1050mm deep and double glulam horizontal members of 660mm wide and 160mm deep. These vertical and horizontal members were preliminary glued together in a factory by employing resorcinol resin adhesive and ϕ 24mm lagscrews per 500mm pitch so as to make a box section for ensuring higher lateral stability.

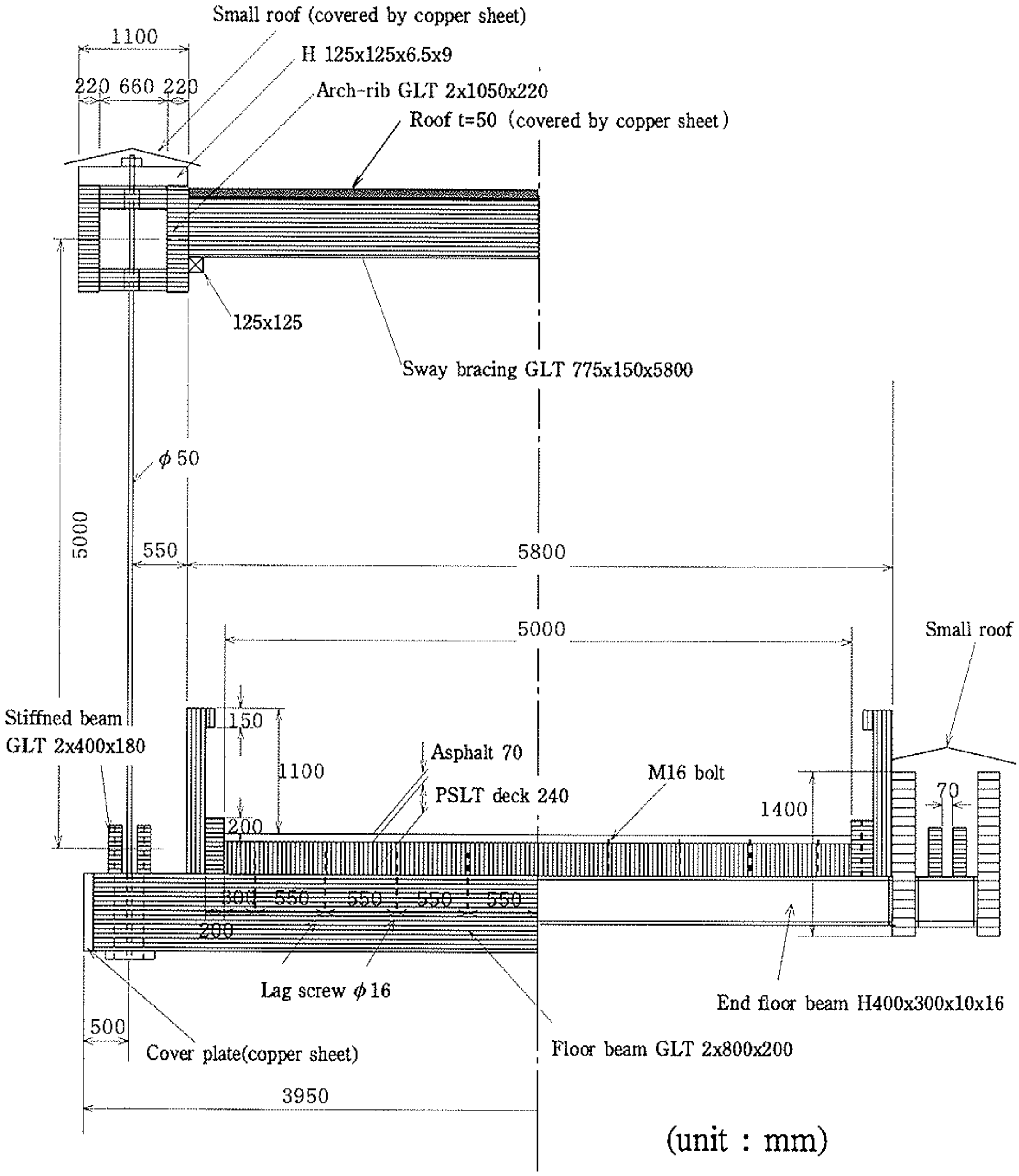


Fig.2 Cross section.

The second characteristic point of this bridge is that the arch-ribs are covered by a small roof along the longitudinal direction for preventing the arch-rib surfaces from rain fall and direct sunshine.

Further more, sway bracings and lateral bracings are also covered by a relatively large roof for preventing bare steel connections and glulam members from rain fall and direct sunshine.

The copper sheet was selected as a cover material for the roof and caps on the glulam end-grain , because copper has been recognized traditionally as an excellent cover material in Japan.

3. DESIGN OF MOMENT-RESISTING JOINT

3.1 Design condition

According to the structural analysis (Usuki et al 1994), the design loads for the arch-rib moment-resisting joints were;

Moment	:	M = 33.1 tfm
Axial force	:	N = 31.5 tf
Shear force	:	Q = 4.1 tf

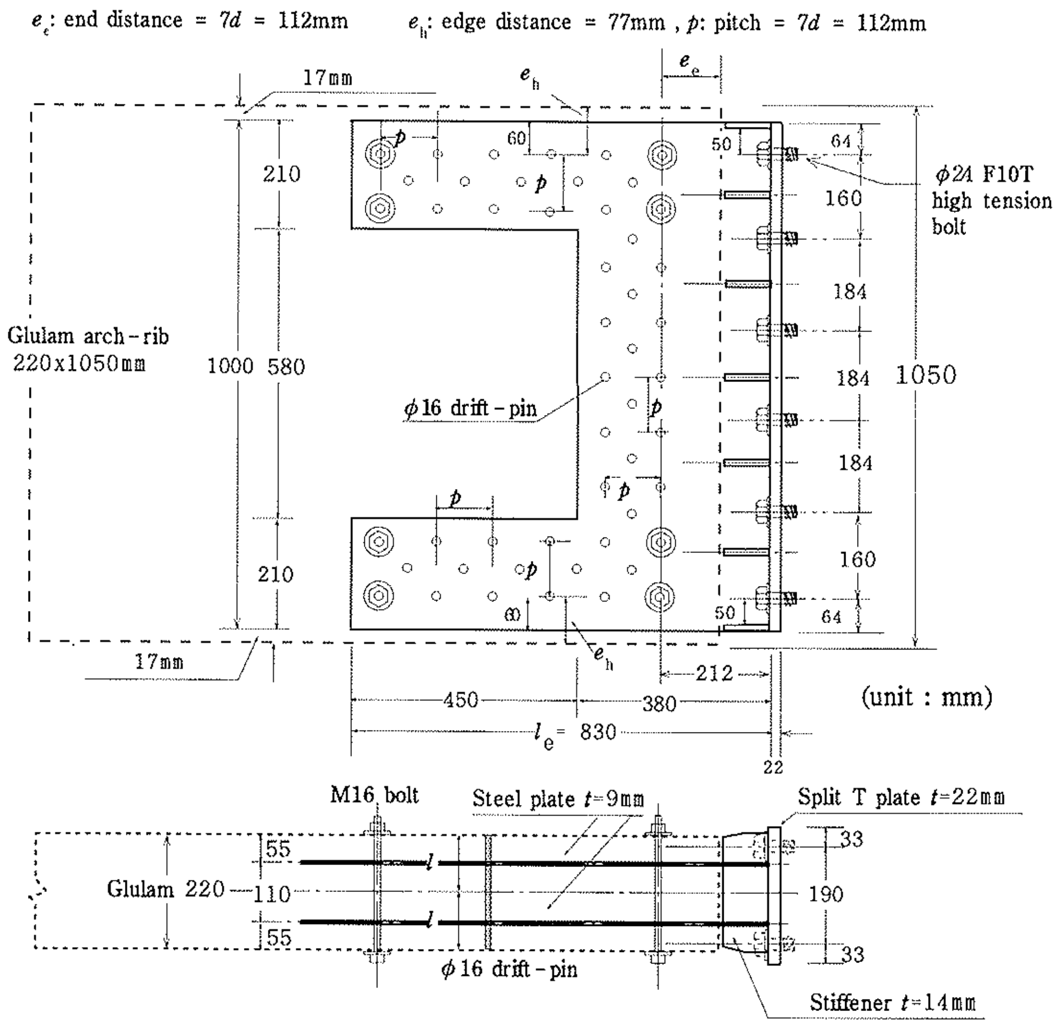


Fig.3 Details of moment-resisting joint using Π -shaped insert steel plates with ϕ 16 mm drift-pins (including 8-M16 bolts for using on-site connection).

By considering following assumptions, and by employing a design method proposed by the first-named author (Komatsu 1994) , the arrangement of drift-pins within the Π -shpaed steel plate was determined as shown in Fig.3.

3.2 Assumptions

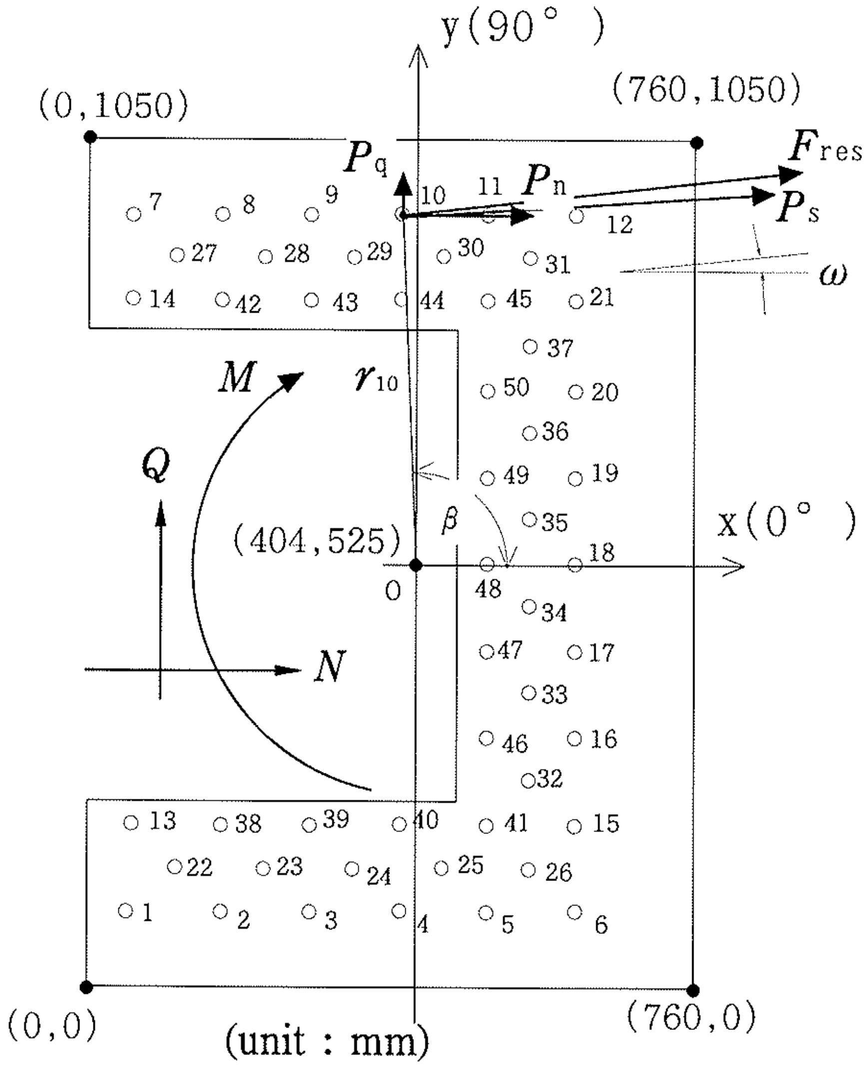
1)Consider only embedment of drift-pin into glulam and rigid-body deformation of steel plate for estimating relative deformations between glulam and steel insert plates.

2)Slip direction of individual drift-pin due to moment is tangential direction measuring from the rotation center.

3)Forces acting on each drift-pin depend on the slip modulus of its's force direction.

4)Any quantities between 0 and 90 degree can be evaluated by using Hankinson's formula.

3.3 Check of allowable strength of drift-pin



(unit : mm)

Fig.4 Forces acting on the most critical No.10 drift-pin in Π -shaped steel plate.

According to a draft design guidelines for timber bridge (Timber bridge committee 1994), allowable strength of drift-pined or/and bolted timber joints are to be derived as follows;

$$P_0 = P_{y0}/3 \quad \text{(pararell to the grain)}$$

$$P_{90} = P_{y90}/3 \quad \text{(perpendicular to the grain)}$$

where,

P_{y0} and P_{y90} are the yield loads which are to be estimated by the Eyropean Yield Theory (for example; Larsen 1973). And 1/ 3 is a sort of safety coefficient.

For the case of drift-pin joint composed of d =16mm, l =110mm, t=9mm and glulam having a nominal density of TD=375 kg/m^3, allowable strengths of drift-pined joint are derived as follows by employing some regression equations between mechanical parameters (Komatsu 1994);

$$P_0 = 424 \text{ kgf} \quad \text{(per shear plane)}$$

$$P_{90} = 202 \text{ kgf} \quad \text{(per shear plane)}$$

Theoretically determined joint rigidities of Π -shaped steel plate joint are (Komatsu 1994);

Rotational rigidity: $R_J = n_s \Sigma K_{\phi i} \cdot r_i^2 = 56500$ tfm/rad.

Shear rigidity: $S_J = n_s \cdot n_i \cdot K_{90} = 1758$ tf/cm

Axial rigidity: $D_J = n_s \cdot n_i \cdot K_0 = 5132$ tf/cm

where,

$K_{\phi i} = K_0 \cdot K_{90} / (K_0 \sin^2 \phi_i + K_{90} \cos^2 \phi_i)$: Slip modulus defined in ϕ_i

$\phi_i = \pi/2 - \tan^{-1}(y_i/x_i)$: Angle of tangential direction

x_i, y_i : coordinate of i-th drift-pin

K_0 : slip modulus of drift-pined joint parallel to the grain (per shear plane) = 25665 kgf/cm

K_{90} : slip modulus of drift-pined joint perpendicular to the grain (per shear plane) = 8790 kgf/cm

n_s : number of shear plane = 4

n_i : number of drift-pin = 50

According to the assumptions shown in clause 3.2, the most critical drift-pin in Π -shaped steel plate joint is not No.7 nor No.12 but No.10 as shown in Fig.4. Thus, strength check is only shown for No.10 pin as follows:

Force due to moment M is;

$$P_s = K_{\phi 10} \cdot r_{10} \cdot M/R_J = 336 \text{ kgf} \quad \text{(per shear plane)}$$

Force due to shear Q is;

$$P_q = K_{90} \cdot Q/S_J = 10 \text{ kgf (per shear plane)}$$

Force due to axial force N is;

$$P_n = K_0 \cdot N/D_J = 79 \text{ kgf (per shear plane)}$$

Resultant force is;

$$F_{res} = \{F_x^2 + F_y^2\}^{0.5}$$
$$= \{(P_s \sin\beta + P_n)^2 + (P_s \cos\beta + P_q)^2\}^{0.5}$$
$$= 415 \text{ kgf (per shear plane)}$$

The allowable strength of drift-pin for the direction of F_{res} is;

$$P_\omega = P_0 \cdot P_{90} / (P_0 \sin^2\omega + P_{90}\cos^2\omega) = 422 \text{ kgf}$$
$$\omega = \tan^{-1}(F_x/F_y) = 0.053 \text{ rad.}$$
$$F_{res} = 415 \text{ kgf} < P_\omega = 422 \text{ kgf} \qquad \text{o.k}$$

Thus, the allowable strength of the most critical No.10 drift-pin was ensured.

3.4 On-site connectin of moment-resisting joints

From past bitter experiences, we decided that structural components having complicated drift-pin joints should be completed at a glulam factory where precise fabrication jobs could be expected. Thus, pre-fabricated box section arch-ribs of about 10m long were transported to the construction site.

At the construction site, moment-resisting joints were quickly completed by connecting the sprit-T plates with each other uisng several ϕ 24mm High-Tension-Bolts (HTB) as shown in Fig.3 or Fig.5.

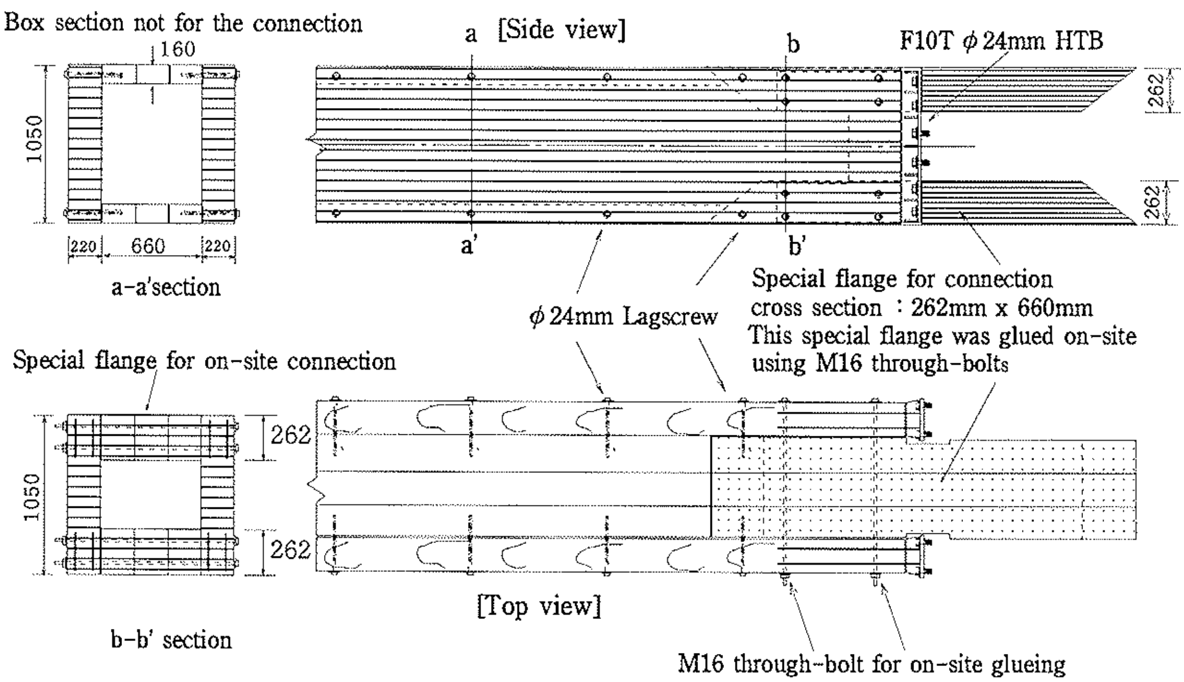


Fig.5 Arrangement of pre-fabricated box section arch-rib and a pair of special flanges

It was, however, a little difficult problem to connect the box sections rigidly at every surfaces on-site. Figure 5 shows an arrangement of box section arch-rib and a pair of special flanges by which top and bottom openings at the arch-rib connections could be rigidly jointed.

The process for jointing box section arch-ribs with a pair of special flanges is as follows;

1) Connect the sprit-T plates with each other using F10T- ϕ 24mm High-Tension-Bolts.
2) Spread resorcinol resin adhesive both on side and scarf surfaces of special flanges and the corresponding arch-rib surfaces then give pressure using M16 through bolts and ϕ 24 mm lagscrews.
3) Hold pressure at least for one day with keeping temperature at higer than 25℃.

4. EVALUATION TEST BY 1/2 REDUCED MODEL

4.1 Edge-wise bending test

In order to evaluate the design procedure for the moment-resisting joint having a Π -shaped steel plates, two types of 1/2 reduced models were fabricated using the same materials as the real arch-rib. One of them is a simple joint specimen composed of straight glulam beams having a Π -shaped steel joint at each end. Another type of specimen is a box section specimen, and by using this type of specimen, lateral flexural properties were mainly evaluated (Usuki et al 1994). In this CIB-report, however, results of edge-wise bending test is only reported.

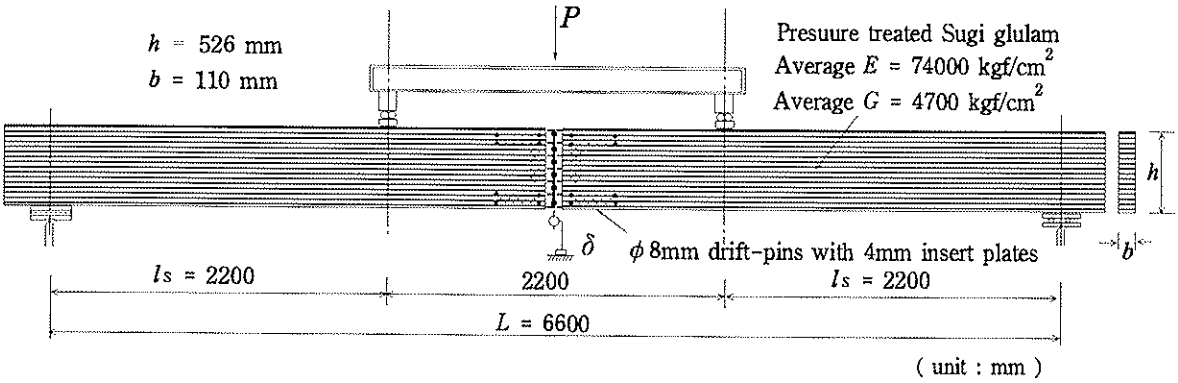


Fig.6 Four points edge-wise bending test set-up for 1/2 reduced model.

Figure 6 shows a set-up for the edge-wise four points bending test. Span length (L) was 6.6m and a constant shear force length (l_s) was 2.2m. Two sets of test specimens were made of Hirota-village grown Sugi glulam beams whose laminae were preliminary pressure treated by the water-borne wood preservative just the same as the real arch-rib glulams.

The 1/2 reduced moment-resisting joints were composed of drift-pins of d=8mm, l=55mm and t=4mm thick Π -shaped double insert-steel plates.

Bending tests were done using an oil-jack type testing machine having 20tons capacity for bending, and one-way, three cycles static loading was applied.

4.2 Results and Discussion

Figure 7 shows load(P) – midspan dflection(δ) relationships of edge-wise bending test specimens. In this figure, P_d = 2000kgf is an equivalent design load by which the most outer fiber stress σ_{bx} of the 1/2 reduced model specimen becomes equal to that due to the actual design moment M=33.1tfm.

It can be seen from Fig.7 that P– δ relationships are almost linear up to the maximum load P_{max} and that there are no yield points in this type of drift-pin joint having double insert steel plates. This is because the ratio of drift-pin length l to diameter of drift-pin d was smaller than 8 (l / d <8) so that the joint failed in a brittle mode before yielding of drift-pins occured.

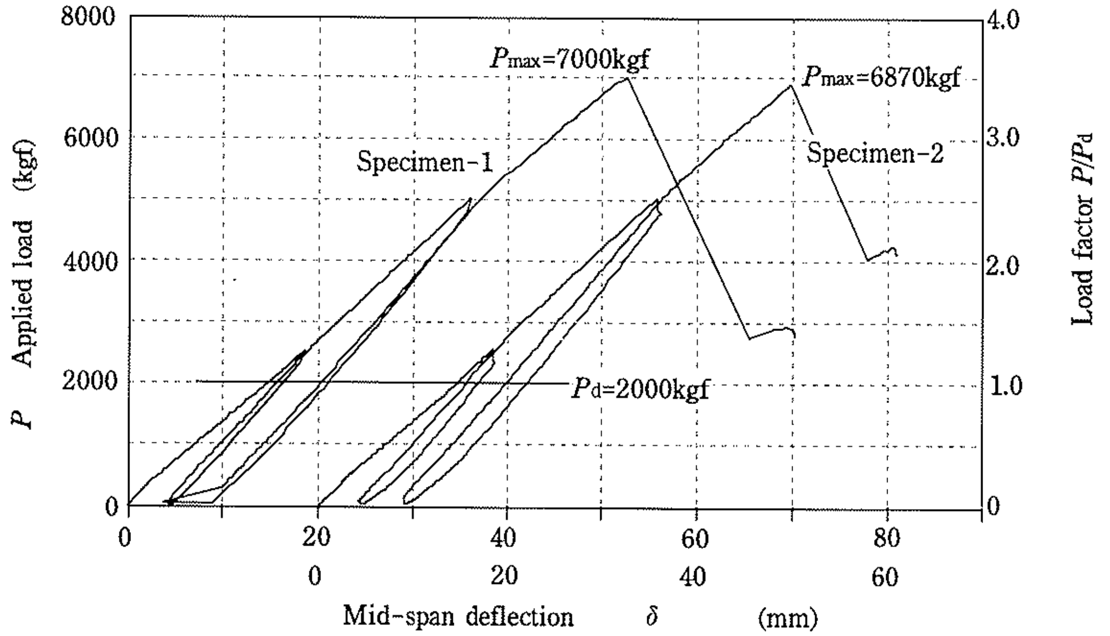


Fig.7 Load(P)–midspan dflection(δ) relationship

Table 1 Results of edge-wise bending tests

	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	(kgf/cm^2)		(kgf)			(ratio)		
Specimen-1	62900	162	7000	7158	0.85	0.47	3.50	1.02
Specimen-2	61420	159	6820	7158	0.83	0.46	3.41	1.05

(1)Apparent *MOE* of joint specimen : $E' = L^3(3\beta - 4\beta^3) / 48I\{\delta / P - \kappa\, l_s / GA\}$, $\beta = l_s / L$

(2)Apparent *MOR* of joint specimen : $\sigma_{max} = P_{max} \cdot l_s / 2Z$, $Z = bh^2 / 6$, $I = bh^3 / 12$, $\kappa = 1.2$

(3)Observed maximum load : P_{max} G = 4700 kgf/cm^2, $A = bh$

(4)Predicted yield load P_{cal} : based on the method shown in clause 3.2 to 3.3 (Komatsu 1994)

(5)Stiffness efficiency = (1) / average *MOE* of glulams (= 74000 kgf/cm^2)

(6)Strength efficiency = (2) / material strength value for Sugi first class glulam (= 345 kgf/cm^2)

(7)Load factor = (3) / equivalent design load P_d (= 2000 kgf)

(8)Prediction / observation = (4) / (3)

Table 1 shows a summary of the experiment. The stiffness (efficiency was 0.83 to 0.84) was satisfactory. While, strength (efficiency was smaller than 0.5) was thought to be "so-so" although the load factor was acceptable level (higher than 3.0).

The predicted yield load P_{cal} was slightly higher than those of observed maximum loads P_{max}, but this discrepancy is very small, thus the design procedure showed in clause 3.2 to 3.3 seemed to be reasonable.

5. CONCLUSION

A two-hinged glulam arch bridge named as "Kami-no-mori" bridge was completed at Hirota village, Ehime prefecture, Shikoku island, in May 1994. This bridge is the first one which was designed by only Japanese bridge technology researchers cooperating with forest products researchers. There are several characteristic points in this bridge. Those are as follows;

1) Most structural members are glulam made of Hirota-village grown Sugi (Japanese ceder).
2) Laminae were preliminary pressure treated by a water-borne wood preservative before lamination glueing.
3) Arch-ribs and lateral bracing were covered by roof for preventing rain fall and direct sunshine.
4) First timber bridge in Japan which was permitted to sustain 20 tons vehicle load.

Through the project for designing and constructing this bridge, a draft design and construction guideline for timber bridge was also proposed.

We think that timber bridge had just a new start also in Japan.

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CIB-W18/27-7-2

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CHARACTERISTIC LOAD - CARRYING CAPACITY OF JOINTS WITH DOWEL - TYPE FASTENERS IN REGARD TO THE SYSTEM PROPERTIES

by

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MEETING TWENTY - SEVEN

SYDNEY

AUSTRALIA

JULY 1994

Characteristic Load-carrying Capacity of Joints with Dowel-type Fasteners in Regard to the System Properties

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1 Introduction

The equations used in ENV 1995-1-1 (EUROCODE 5) for determining the load-carrying capacity of joints with one individual dowel-type fastener are based on the "Johansen-theory". In deriving these equations the following assumptions are made:

- both the fastener (yield moment) and the timber members (embedding strength) are ideal rigid-plastic materials
- the so-called "chain effect" is disregarded
- a splitting failure in the member is excluded.

Starting from the principle of the "Johansen-theory" both a calculation model and a computer programme were developed in order to consider these neglected effects.

The characteristic load-carrying capacity should be determined by the joint geometry and the characteristic material properties, i.e. the fastener yield moment and the embedding strengths of the timber or wood-based members. By now, the variability of the material properties has not been taken into account. Using timber of the same strength class the designer supposed to calculate the embedding strength from the characteristic density of this strength class. In reality, the two or three members in a timber joint have different densities. With a computer programme taking all these influences into account simulation calculation can be made.

It is the objective of this paper to provide simplified "system and correction factors" for modifying the strict "Johansen-theory" in regard to the material properties as explicitly defined in EUROCODE 5 (1993). In the final draft of EUROCODE 5 a constant system factor of 1,1 was added to those equations describing failure modes with fastener yield points. The effect of multiple fasteners on the connection strength is described by BLAß 1994.

2 A comparison between material properties given in EUROCODE 5 and test values

Embedding strength

Simulation calculations showed that the embedding strengths of timber have a larger variation than the corresponding densities. An analysis of 12000 simulated data leads to a reduction of about 6% compared to EC5. For bolts and dowels in softwood the following expressions were found:

$$f_{h,\alpha,k} = \frac{f_{h,0,k}}{k_{90} \cdot \sin^2 \alpha + \cos^2 \alpha} \qquad (1)$$

$$f_{h,0,k} = 0{,}077 \cdot \rho_k \cdot (1 - 0{,}01 \cdot d) \qquad (2)$$

$$k_{90} = 1{,}35 + 0{,}015 \cdot d \qquad (3)$$

with $f_{h,\alpha,k}$, $f_{h,0,k}$ in N/mm^2, ρ_k in kg/m^3 and d in mm

Yield moment

Following a Swiss proposal (GEHRI 1992) the yield moment for round steel bolts and dowels should be determined as follows:

$$M_{y,k} = 0{,}8 \cdot f_{u,k} \cdot \frac{d^3}{6} \qquad (4)$$

$f_{u,k}$ characteristic tensile strength of the fastener

This proposal was accepted in the final draft of EUROCODE 5. In order to check this equation the yield moments of dowels and metric threaded rods made of different steel qualities were tested in line with EN 409. The bending tests (n = 31) with dowels made of cold formed steel (St 37 K according to DIN 1652) ended up with the following results:

mean tensile strength:	$f_{u,mean}$	$= 583\ N/mm^2$
mean yield strength:	$f_{y,mean}$	$= 610\ N/mm^2$
characteristic tensile strength:	$f_{u,k}$	$= 466\ N/mm^2$
characteristic yield strength:	$f_{y,k}$	$= 490\ N/mm^2$

The characteristic strength values were determined with a coefficient of variation of 13% in line with DIN 1652.

The guaranteed minimum tensile strength of this material is $f_{u,min}$ = 440 N/mm² according to DIN 1652. As a result of these tests, a ratio of

$$\frac{f_{y,mean}}{f_{u,mean}} = \frac{f_{y,k}}{f_{u,k}} = 1{,}05$$

was found. This apparent contradictions is attributed to the different test-methods used for the determination of f_u and f_y.

Also tests with common smooth steel wire nails (WERNER, SIEBERT 1991), dowels made of free cutting steel and metric threaded rods (Property class 8.8) showed that the yield strength tested in bending is higher than the tensile strength. It can be concluded that equ. (4) considerably underestimates the effective yield moment of dowels made of commercial steel.

3 Calculation model

A computer model for calculating the ultimate loads of joints with dowel-type fasteners was presented in WERNER 1993 and is used to determine characteristic values of the load-carrying capacity. The following influencing properties are included in this calculation model:

- thicknesses of the timber members and the diameter of the fasteners
- embedding stress-strain behaviour in and perpendicular to grain direction of the timber members
- yield moment of the fastener
- withdrawal resistance and gliding coefficient of friction between fastener and wood
- fracture toughness for describing the splitting effect
- end and edge distances of the fasteners
- width of longitudinally crushed timber beneath a fastener

With the computer program "XJOINT" different dowelled joints were calculated and compared with test results. The load-carrying capacity and the load-deformation behaviour of double-shear softwood and hardwood joints with dowels and metric threaded rods up to 30 mm were investigated. The test results are published in EHLBECK and WERNER 1989 and 1992. All specimens were loaded in grain direction. The joint configuration was chosen with the minimum spacings

(a_1 = 5d and $a_{3,t}$ = 6d) allowable according to the German timber design code DIN 1052.

In **Table 1 and 2** the mean values of the measured maximum load F_u per fastener and the slip at F_u (v_{split}) are compared with the calculated load-carrying capacities R_u and the appertaining slip max v. **Fig. 1** shows the ratio F_u/R_u over the slenderness $\lambda = t_2/d$ (t_2 is the thickness of the centre member). A good agreement between the test results and the calculated values can be stated.

Table 1: Comparison between the measured maximum load F_u per fastener and the calculated load-carrying capacity R_u

Mean values of the tests loaded in tension with glued laminated timber and cold formed dowels

test serie	diameter d (mm)	slender-ness $\lambda = \frac{t_2}{d}$	maximum load F_u (kN)	cal. load-carrying capacity R_u (kN)	slip at F_u v_{split} (mm)	slip at R_u max v (mm)	ratio $\frac{F_u}{R_u}$
U 1/6	8	6	8,09	7,59	3,3	3,5	1,06
V 1/6	8	6	8,92	8,65	4,1	3,5	1,03
U 1/8	8	8	9,30	8,93	2,6	3,0	1,04
V 1/8	8	8	9,13	9,16	3,8	3,0	1,00
U 2/4	16	4	27,0	26,9	2,6	2,5	1,00
N/U 2/6	16	6	27,8	28,0	3,1	3,5	0,99
U 2/6	16	6	31,2	28,7	3,6	4,0	1,09
H/U 2/6	16	6	32,5	33,8	2,4	3,5	0,96
U 2/8	16	8	33,1	30,8	4,5	3,5	1,07
V 2/6	16	6	31,2	30,4	4,7	3,0	1,02
V 2/8	16	8	32,4	34,1	4,4	3,0	0,95
U 3/6	24	6	57,2	63,4	4,0	4,0	0,90
U 4/6	30	6	87,8	98,6	4,2	4,5	0,89

V = staggered arrangement
U = non-staggered arrangement of the dowels
H = high density
N = low density

Table 2: Comparison between the measured maximum load F_u per fastener and the calculated load-carrying capacity R_u

Mean values of the tests loaded in tension with hardwood and cold formed dowels

test serie	diameter d (mm)	slenderness $\lambda = \frac{t_2}{d}$	maximum load F_u (kN)	cal. load-carrying capacity R_u (kN)	slip at F_u v_{split} (mm)	slip at R_u max v (mm)	ratio $\frac{F_u}{R_u}$
A1-Z 8/2	8	2	6,12	6,30	1,7	1,0	0,97
A1-Z 8/4	8	4	10,8	10,8	6,7	3,5	1,00
A1-Z 8/8	8	8	19,5	19,1	14,0	11,0	1,02
A2-Z 8/2	8	2	6,86	6,95	3,2	1,0	0,99
A2-Z 8/4	8	4	11,3	11,1	5,5	2,5	1,02
A2-Z 8/6	8	6	13,8	13,1	10,2	3,0	1,05
A2-Z 8/8	8	8	16,0	13,8	7,9	3,5	1,16
A2-Z 16/2	16	2	16,7	20,7	1,0	1,0	0,81
A2-Z 16/4	16	4	35,6	37,4	3,3	3,5	0,95
B1-Z 8/2	8	2	6,68	6,55	2,0	0,5	1,02
B1-Z 8/4	8	4	9,91	11,2	1,1	2,0	0,89
B1-Z 8/8	8	8	15,4	15,2	5,2	3,5	1,01
B2-Z 8/2	8	2	7,15	8,33	1,4	1,0	0,86
B2-Z 16/2	16	2	24,4	25,5	4,8	1,0	0,96
B2-Z 16/4	16	4	43,9	40,9	1,1	3,0	1,07
C-Z 8/2	8	2	10,7	11,5	1,8	1,5	0,93
C-Z 8/4	8	4	15,1	15,0	2,9	3,0	1,01
AS/C-Z 8/4	8	4	15,4	17,6	2,3	4,0	0,88
AS/C-Z 8/6	8	6	19,2	20,3	6,1	5,0	0,95
AS/C-Z 8/8	8	8	19,9	19,9	6,4	5,0	1,00

A1 = Europ. beech　　A2 = Europ. oak　　B1 = Merbau (*Intsia*)
B2 = Afzelia (*Afzelia*)　　C = Bongossi (*Lophira alata*)
AS = free cutting steel　　Z = Test specimen loaded in tension

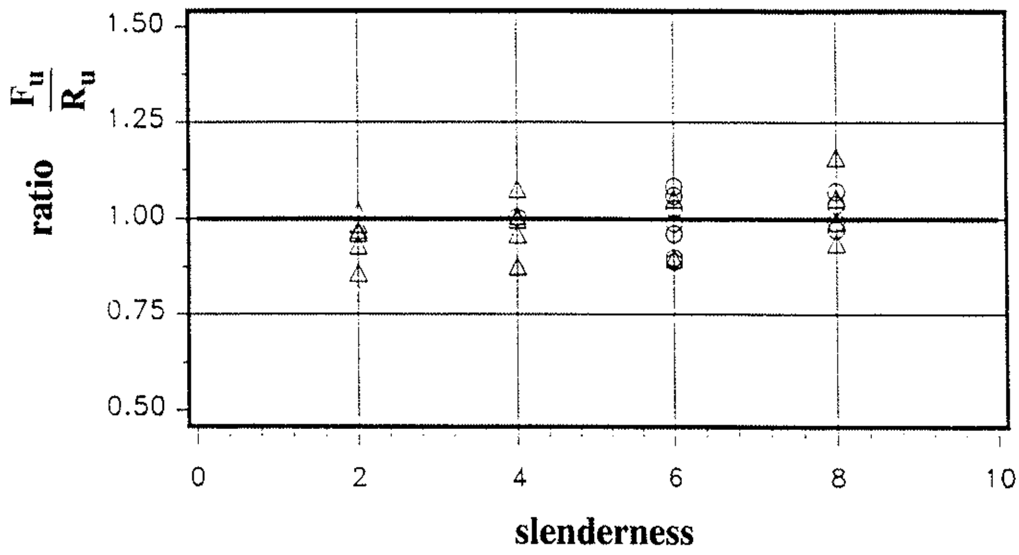


Figure 1: Ratio F_u/R_u over slenderness λ

With the calculation model also the load-carrying capacity R_u of steel-to-timber joints can be determined.

4 Stochastic investigations

The stochastic model of joints with dowel-type fasteners includes the statistic distribution functions of the influencing properties and their autocorrelation, i.e. embedding stress-strain relationship in and perpendicular to grain direction, yield moment of the fastener, thicknesses of the members, withdrawal resistance of the fastener and fracture toughness. The diameter of the fastener is regarded as a deterministic variable. The stress-strain relationship from 539 embedment tests was fitted by an approximation function with a nonlinear least squares minimization technique. The function and the Pearson's correlation coefficients of strength and stiffness properties are given in WERNER 1993. For simulation the mutual correlation of the parameters of the approximation function a transformation of the multivariate normal distribution was used. This procedure was described by TAYLER, BENDER 1988 and also used by BLAß 1991 for simulating correlated timber properties. For every joint configuration 600 replicas were simulated and the load-carrying capacity of each individual joint was determined.

5 Load-carrying capacity of different kinds of joints

Timber-to-timber joints with dowels

The ultimate loads of dowelled joints with softwood loaded in grain direction as shown in **Fig. 2** under normal climate conditions were calculated. The following variations were investigated:

Fastener:	cold formed smooth round dowels (St 37 K)
Fastener diameter:	d = 8; 16; 24 and 30 mm
Slenderness:	$\lambda = t_2/d = 2; 4; 6; 8$ and 10
ratio t_1/t_2:	$t_1/t_2 = 0{,}25; 0{,}50; 0{,}75$ and 1,00

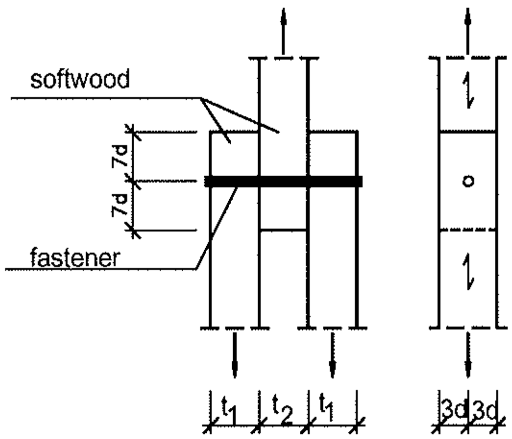


Figure 2: Dowelled timber-to-timber joint

By way of example, the evaluation of the calculation for d = 8 mm and t_1/t_2 = 0,75 is shown in **Fig. 3**. In this diagram the load-carrying capacity curves of the mean values and the nonparametric values of the 5th percentile are presented. Under the diagram the shares expressed as percentage of failure modes for different slendernesses are given. The influence of the ratio t_1/t_2 on the load-carrying capacity is illustrated in **Fig. 4** for the slenderness t_2/d = 6 and d = 16 mm.

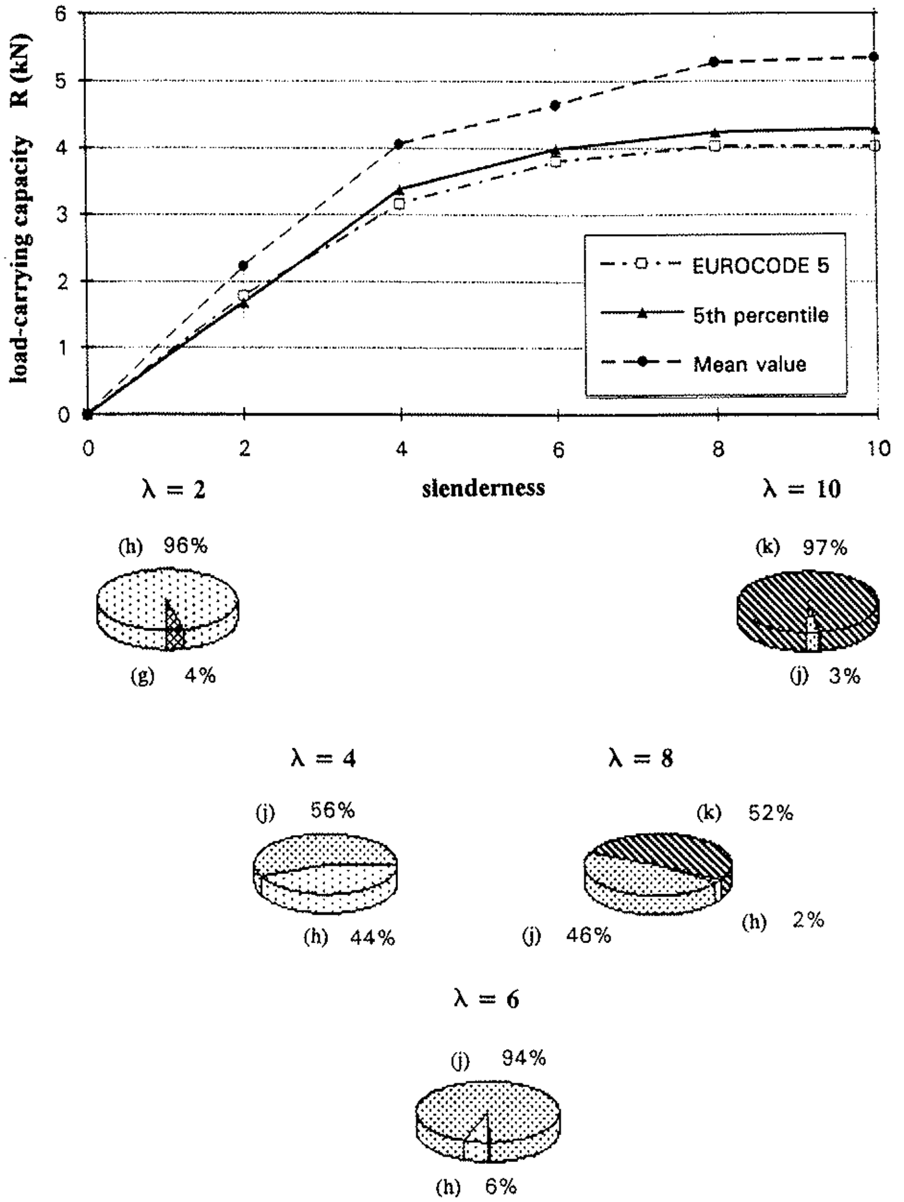
The comparison between the simulation results and the design procedure (see **Fig. 3 and 4**) as proposed in EUROCODE 5 (1993) with

$$f_{h,k} = 0{,}082 \cdot \rho_k \cdot (1 - 0{,}01 \cdot d) \qquad (\mathrm{N/mm^2}) \qquad (5)$$

$$M_{y,k} = 0{,}8 \cdot f_{u,k} \cdot d^3/6 \qquad (\mathrm{Nmm}) \qquad (6)$$

$$\rho_k = 370\ \mathrm{kg/m^3} \qquad f_{u,k} = 440\ \mathrm{N/mm^2} \qquad \beta = 1$$

gives a good agreement. For high slendernesses the 5th percentile values of the simulation results are higher, without exception. Under the assumptions made in EC5, saying that the embedding strength and the yield moment are calculated with eqs. (5) and (6), the system and correction factor $k_{sys} = 1{,}1$ can be added to the equations describing failure modes with fastener yield points. But it is better to use M_y-values from bending tests and $k_{sys} = 1{,}0$ to describe the behaviour of these joints.



(The letters corresspond to the relevant failure modes as defined and illustrated in EUROCODE 5)

Figure 3: Load-carrying capacity per shear plane per fastener and the shares of failure modes expressed as percentage over slendernesses (Dowel St 37 K; $d = 8$ mm; $t_1/t_2 = 0{,}75$)

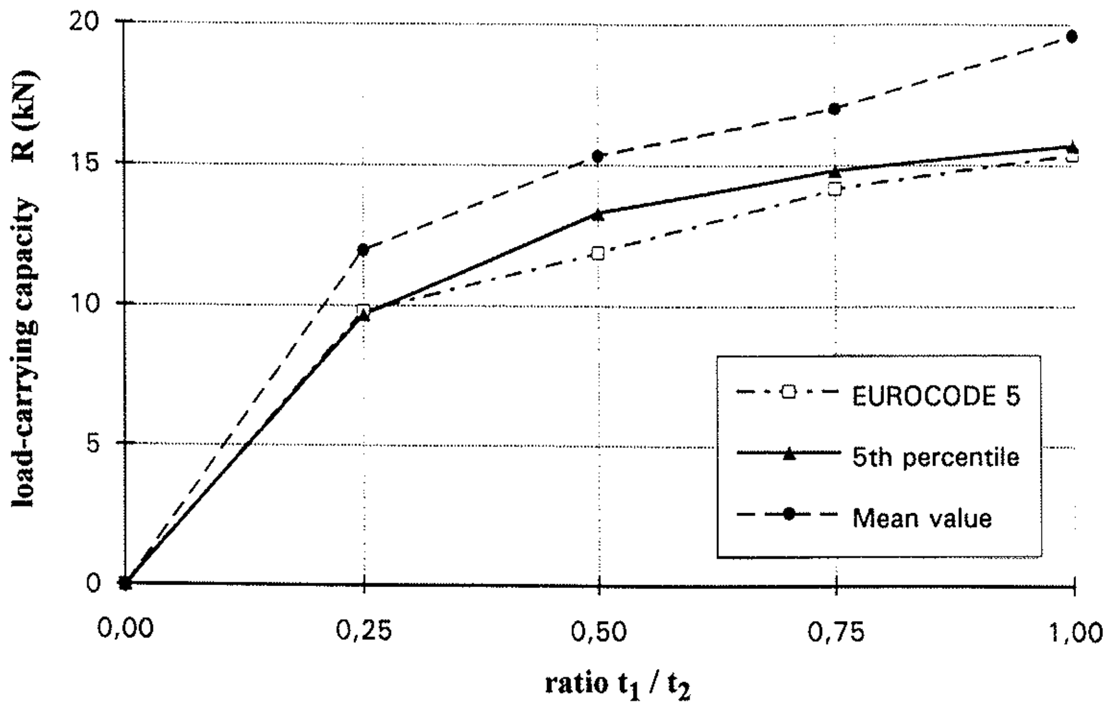


Figure 4: Load-carrying capacity per shear plane per fastener over ratio t_1/t_2 (Dowel St 37 K; $d = 16$ mm; $t_2/d = 6$)

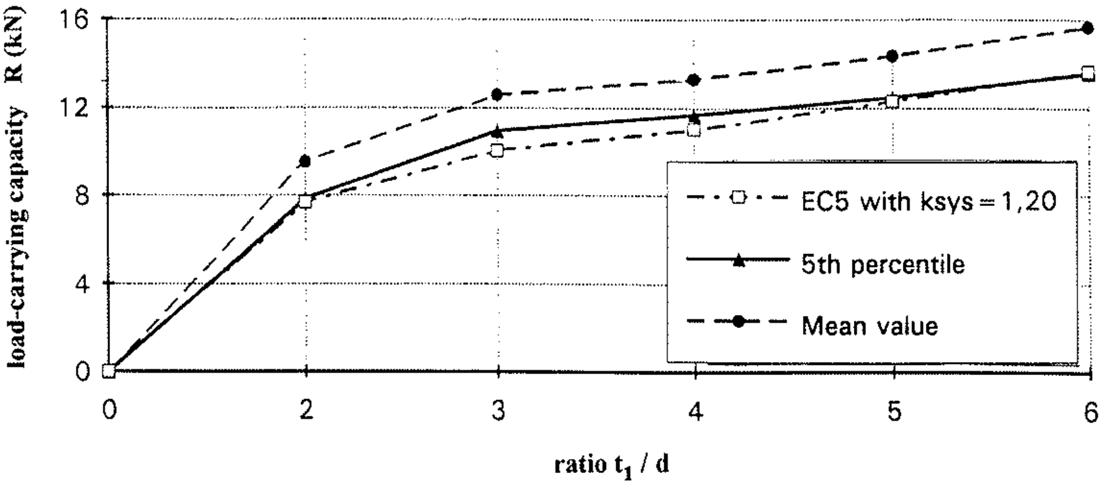
Steel-to-timber joints with the centre member of steel and dowels

Steel-to-timber joints with the centre member of steel is one of the most economic connections. With the calculation model the load-carrying capacities of joints with different dowel diameters were calculated. The results are given in **Fig. 5**.

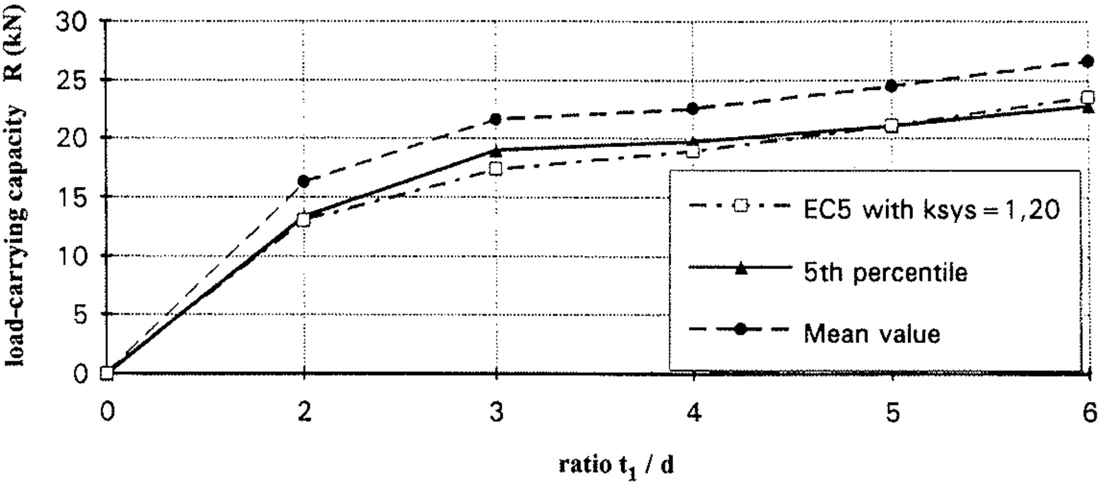
Under the assumption that the embedding strength and the yield moment are calculated with eqs. (5) and (6), a system and correction factor

$$k_{sys} = 1{,}20$$

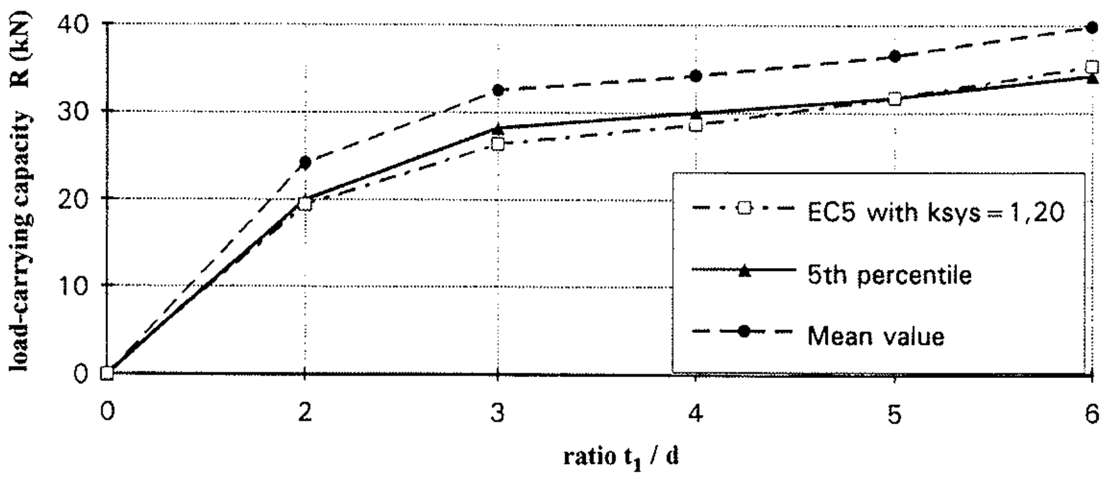
gives a good agreement. If using the M_y-values from bending tests a system factor $k_{sys} = 1{,}05$ can be added to the relevant equations.



(a) Dowel **d = 12 mm**



(b) Dowel **d = 16 mm**



(c) Dowel **d = 20 mm**

Figure 5: Load-carrying capacity per shear plane per fastener over ratio t_1/d of steel-to-timber joints (t_1 = thickness of side member) (Dowel St 37 K; Centre member of steel)

Timber-to-timber joints with metric threaded rods

Timber-to-timber joints with metric threaded rods (Property class 8.8) with the same joint configuration as shown in **Fig. 2** were simulated. A comparison with the load-carrying capacity of dowelled joints with approximately the same yield moments mark out the influence of the "chain effect" (see **Fig. 6**).

Under the presumption -as proposed in EUROCODE 5- that the embedding strength and the yield moment were calculated with the following equations

$f_{h,k} = 0{,}082 \cdot \rho_k \cdot (1 - 0{,}01 \cdot d)$ (N/mm^2)

$M_{y,k} = 0{,}8 \cdot f_{u,k} \cdot d_s^3/6$ (Nmm)

$\rho_k = 370$ kg/m^3 $f_{u,k} = 800$ N/mm^2 $\beta = 1$

d: nominal diameter

d_s: diameter calculated from the tensile stress area

a system and correction factor

$$k_{sys} = 1{,}55 \cdot d^{-0,1}$$

can be added to those equations describing failure modes with fastener yield points. If using the M_y-values from tests a factor $k_{sys} = 1{,}40 \cdot d^{-0,1}$ gives comparible results.

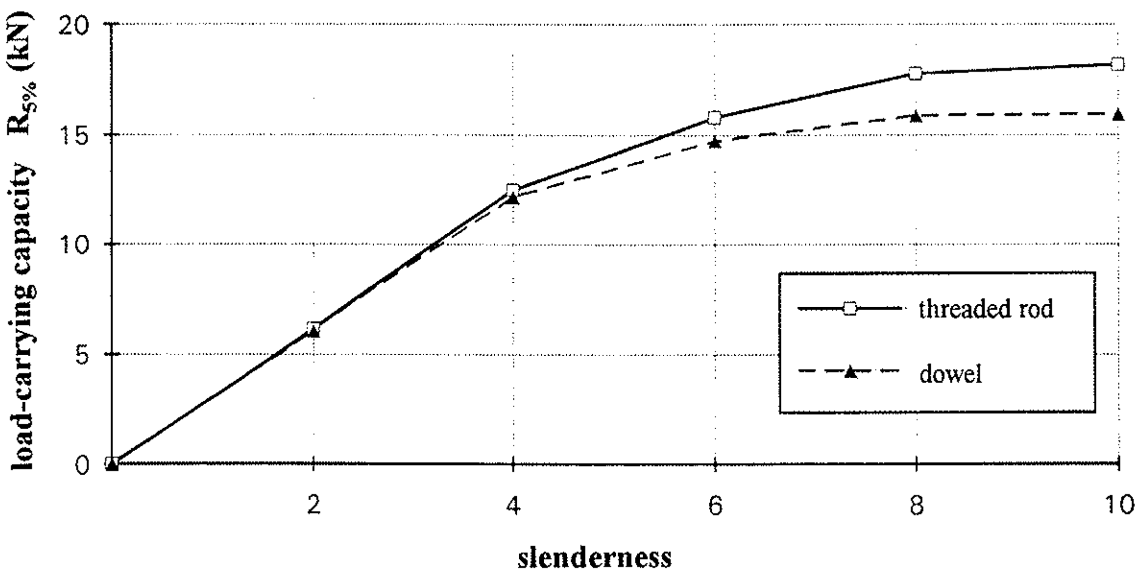


Figure 6: Comparison between the 5th percentile of the load-carrying capacity of joints with metric threaded rods (Property class 8.8) and dowels (St 37 K) (Nominal diameter $d = 16$ mm; $t_1/t_2 = 0{,}75$)

6 Conclusions

The characteristic load-carrying capacities of different kinds of joints with one dowel-type fasteners were calculated. A calculation model and a computer programme were used, which take into account the non-linear embedding stress-strain behaviour, the yield moment according to EN 409, the so-called "chain effect" and the splitting effect. The results from the Monte Carlo simulation were compared with the "Johansen-theory" in regard to the material properties as defined in EUROCODE 5 (1993).

It was found, that in EUROCODE 5 for bolted and dowelled joints the relationship for the embedding strength over-estimates the effective strength properties by about 6 %. The relationship for the yield moment, however, underestimate the real bending resistance of the fastener.

Presuming that the embedding strengths and the yield moments are calculated as proposed in EUROCODE 5, a system and correction factor k_{sys} can be added to the equations describing failure modes with fastener yield points. This factor is not a constant but is influenced by the kind of joint. A factor $k_{sys} = 1{,}1$ (as inserted in the final draft of EUROCODE 5) is reasonable for timber-to-timber joints with commercial smooth round dowels and bolts; but k_{sys} may be increased for steel-to-timber joints with centre members of steel and for fasteners such as metric threaded rods. The load-carrying capacity of joints is, however, described more realistic by using the M_y-values determined from bending tests in line with EN 409 and smaller system factors, respectively.

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

STEEL FAILURE DESIGN IN TRUSS PLATE JOINTS

by

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MEETING TWENTY - SEVEN

SYDNEY

AUSTRALIA

JULY 1994

Steel failure design in truss plate joints[1]

by

T. Poutanen[2]

Abstract

In the design of a truss plate (connector plate, nail plate) joint there are three major tasks: the analysis of joint loads, anchorage failure and steel failure (failure along the joint line). This paper deals with joint line failure and proposes a new design method.

The basic idea in the proposed design method is to assume that the joint line has an analogy with a large number of linear springs. These springs have very high stiffness and they are distributed along the joint line according to connector strength. This analogy and the model are clear. The model is applicable to all kind of joints, even to the most complicated ones as such. This makes programming easy, as one design algorithm can handle all cases.

Some discussion has been conducted concerning the plasticity and steel yielding of the connector. One conclusion is that considering the moment also requires considering the rotation stiffness i.e. a check needs to be done to determine that the rotation and the moment match each other. There exist some relevant data and design methods relating to the rotation between a plate and timber. However, neither relevant data nor theory exist on the rotation which is caused by the steel yielding in the joint line. The proposed design method is based on the elastic state. Therefore data on steel yielding and plasticity are not needed. The other conclusion is that applying the theory of plasticity is quite complicated and diffuse.

The method has been programmed and it has been introduced into practical truss design.

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[1]Prepared for presentation in CIB-W18-meeting in Sydney, Australia, 7-8 July 1994

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Introduction

The truss plate (connector plate, nail plate) joint has to resist force loads as well as moment loads. Without a moment resistance the connector plate joint is an unstable mechanism. Moment resistance is quite a complicated matter. Due to this fact, these joints are often designed approximately by ignoring moment loads and -resistance.

The connector plate joint has two failure types[3], the anchorage (lateral resistance, grip) between a plate and timber and the failure along the joint line (steel failure). In figure 1 the two failure types have been illustrated and the internal loads: F_a and M_a act in the plate-timber interface inducing shear stresses between the connector and timber. These shear stresses are at their greatest at the outermost corners of the contact area in the connector-timber interface. F_s and M_s act in the plate (steel) in the joint line inducing tension, compression and/or shear stresses in the connector. The connector plate joint is often analysed and designed by considering only the force loads i.e. F_a, F_s. However, the moments M_a, M_s should be considered in design, too. Making an anchorage design for a connector plate joint which is loaded by force and moment (F_a and M_a) is not a clear matter. Even more unclear is the steel failure design when the connector is loaded by force and moment (F_s and M_s). This paper focuses on this problem and suggests a new design method.

Displacement

The joint displacement affects the stress distribution. Therefore the joint displacement must be known and it must be taken into account in order to make the correct analysis and design. There occur both translational δ and rotational displacement Φ in the joint. The designer must know whether the joint displacement affects the stress distribution i.e. the designer must have a grasp of the translational and the rotational displacements of the joint. The translational displacement is fairly well known. It is caused (mainly) by the joint force F and its function is simple. The rotational displacement is not so well known and its function is complicated. It is caused mainly by moment M.

In the connector plate joint there are three factors which essentially effect the joint rotation:

1 Rotation Φ_a in the plate-timber interface induced by moment M_a.
2 Rotation Φ_s which occurs in the joint line due to steel yielding induced by moment M_s and
3 contact between adjacent timber members in the joint.

These factors are different by their basic nature and they influence the joint rotation separately. There are joints where only one of these things is dominant e.g.: In a diagonal joint with a small connector, Φ_a is usually fully dominant. If the connector is large and the load is high, Φ_s becomes dominant. In a compressed joint with contact (typically a top chord splice joint), the contact dominates the joint rotation.

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3 The third failure type is timber splitting, which has minor importance.

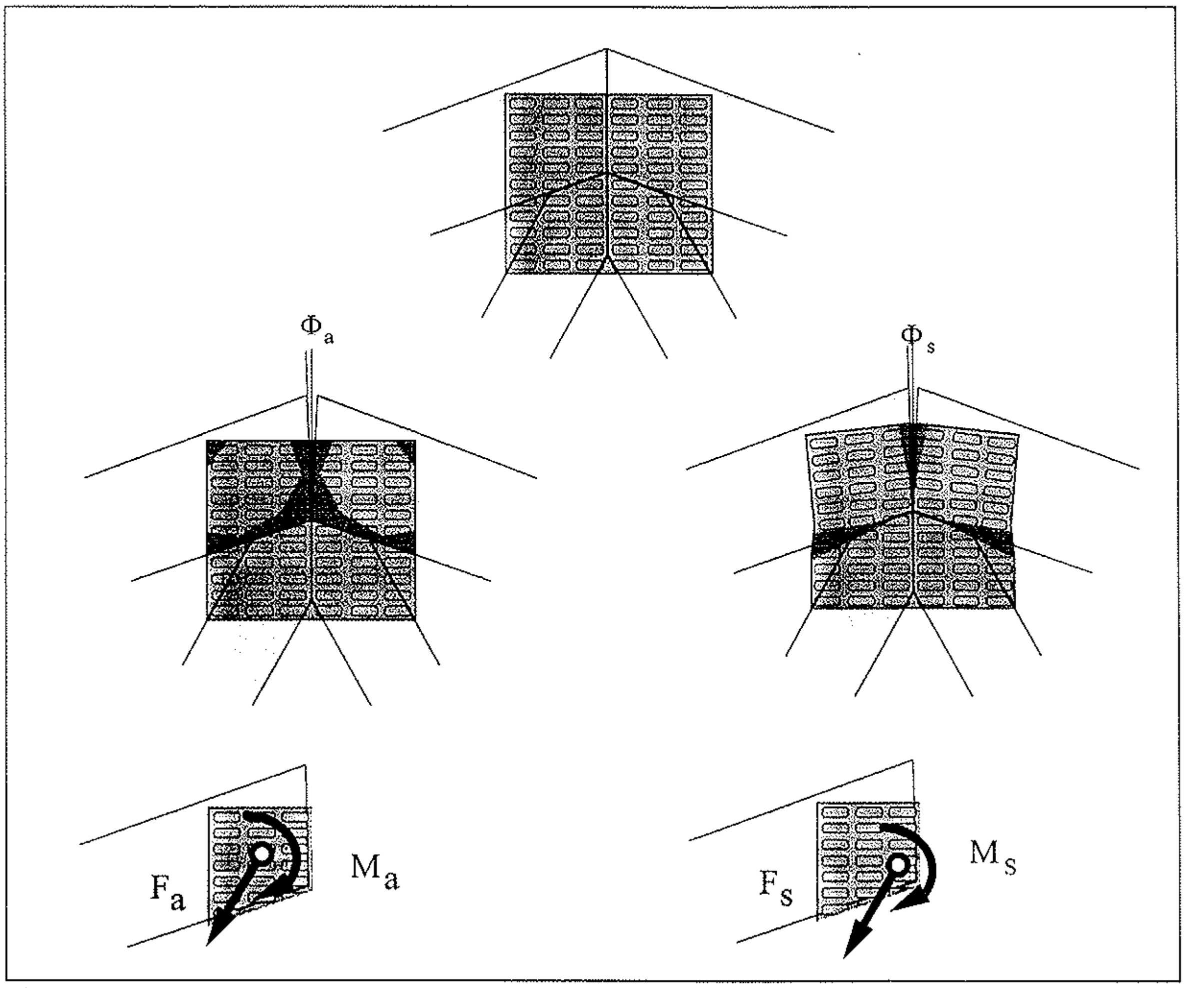


Fig 1
A joint (apex joint) at the unloaded state at the top, in the loaded state in the middle, anchorage rotation Φ_a between the connector and timber demonstrated on the left, joint line rotation Φ_s (caused by steel yielding) demonstrated on the right. Areas where the major displacements take place are shown by dark colour.
The plate joint loads (resultant composed of force and moment) are demonstrated at the bottom: anchorage force F_a and anchorage moment M_a on the left and joint line force F_s and joint line moment M_s on the right.

There is a misunderstanding that Φ_a is the major (or even the only) reason for the joint rotation. However, the rotation Φ_s is important, too, if the steel yields. The moment resistance is actual especially in joints with high loads (high force and moment) i.e. the moment joints are typically big and the connectors within them big, too. In a joint with a big connector, Φ_a is small. If the steel yields, Φ_s is in most cases the major factor causing the rotation. If the steel does not yield, a big joint is typically very stiff and the joint rotation is small, usually so small that the joint rotation may be ignored and it is a good approximation to assume the joint as completely stiff.

Rotation tests

Until recent years research has not paid enough attention to the different nature of effects 1-3. A lot of tests have been done without separating these factors and therefore the results have been diffused and of minor relevance.

Recent research e.g. Aasheim [1], Kevarinmäki and Kangas [2] has focused attention into Φ_a i.e. the rotation between a plate and timber. In both studies, small connectors (a connector overlaps timber appr. 50 mm) were selected to avoid a steel yielding in the joint line i.e. Φ_S=0.

The author does not know a single study which would give relevant data concerning the interaction between Φ_S and M_S. This interaction, however, should be known in order to make a consistent analysis and design. In this method the design is based on the elastic state and there is no steel yielding i.e. $\Phi_S = 0$.

Eurocode 5

Eurocode 5 design code [3] is the only code which includes a recommendation on steel failure design under the moment load. This code has the following characteristics:

1 The design state is plastic i.e. the joint line has a stress distribution which corresponds to the full steel yielding.

2 There are several approximations e.g.

- 2a In defining the sum stresses induced by force and moment the scalar sum is used instead of the more correct vector sum.
- 2b It is assumed that the moment induces stresses with equal compression and tension stress. This assumption may in some cases be very conservative. It would be more correct to assume stresses are distributed according to strength.
- 2c It is assumed that the moment induces stresses perpendicular to the joint line only. This assumption is conservative because in some cases it would be favourable to transfer moment loads via shear capacity.

3 No specific recommendation is given on how stresses are distributed along the joint line, there are many alternatives especially if the joint line consists of several lines (polygon). The designer has to choose between several alternatives, some of them leading to a more economic design outcome than the others. In principle, the design task includes an optimisation task to the fact of how loads are distributed along the joint line.

This new design model has been developed during the process of making a new truss design program. There were several reasons why an improvement to the Eurocode 5 recommendation was sought:

- The Eurocode 5 recommendation was considered as unclear and diffuse. Complicated joints consisting of several failure lines caused problems e.g. it was unclear whether the joint should be stressed first by force and then by moment or vice versa and in which order the several failure lines are stressed. This code also offers the possibility of "tricks" e.g. the moment is transferred over the joint via eccentric force (i.e. shear resistance and normal resistance are utilised fully against forces in the part of the joint where a force and a moment make the sum action). This means that shear capacity is utilised (indirectly) against moment loads. Eurocode 5 offers such a large number of possibilities, it was difficult to find a clear and definite design algorithm.
- In the new program all joints are analysed and designed as semi-rigid. Moment is taken into account in all the joints. The aim was to handle moments with a clear and definite algorithm without unnecessary approximations. The Eurocode 5 recommendation contains approximations and unclear statements especially in moment design.
- Eurocode 5 advises that stresses induced by the moment are distributed perpendicular to the joint line. This principle does not necessarily lead to an optimum design.

- The principle of full plasticity was experienced as questionable mainly for two reasons. Joint behaviour is very different in the plastic state from the behaviour in the elastic state. Forces are distributed differently and e.g. eccentricities are different, the internal load eccentricity is not the same as the strength eccentricity, which causes imbalance in equilibrium (if not taken into account). If plasticity were to be utilised, two models, elastic and plastic, would be needed in a consistent analysis. Secondly, joint stiffness data is needed in the analysis. There is no stiffness data for a plastic connector plate joint (especially data for the case where steel yielding is the only reason for deformation). One could search for quidance in this matter from a steel code e.g. from the new European steel code Eurocode 3 [5]. In this code, in paragraph 6.1.3 "Resistance of connection" the second clause reads as follows: "Linear-elastic analysis shall generally be used in the design of the connection. Alternatively non-linear analysis of the connection may be employed provided that it takes account of the load deformation characteristics of all the components of the connection." The steel code supports elastic design.

The author's opinion is that the new design model presented here is more consistent and also more simple than the Eurocode 5 recommendation for steel failure design in a connector plate joint. The design model presented here is in all cases more strict than the Eurocode 5 model, because the design is based on the elastic state. In this respect the results obtained from this design model are safe compared to the Eurocode 5 design. This does not mean that the proposed method would necessarily lead to a more expensive design outcome. The Eurocode 5 design has an economic outcome if the designer can find a suitable stress distribution - if not, the result may be quite uneconomic.

Assumptions

This new design method is based on a standard two-dimensional frame program and general assumptions of statics and material physics plus some specific assumptions:

Specific assumptions

There are the following specific assumptions:

1 The connector may fail only in the joint line i.e. along the periphery of the timber member.
2 The connector (steel) has no strength and stiffness outside the periphery of the timber.
3 Only the supported timber periphery, i.e. the periphery which touches another timber, is effective.
4 The connector does not yield.
5 The connector is completely solid with no deformation.
6 The stress distribution along the joint line is defined by assuming the joint line consists of a large number of linear springs with very high stiffness distributed along the joint line according to connector strength.
7 The failure state in the joint line is reached as stress in one point of the joint line reaches the critical stress i.e. characteristic strength (capacity) in the elastic state.

There are some other more obvious assumptions, too:

- The contact of timber members under the connector does not affect the steel failure i.e. a joint with contact and a joint without contact are designed by the same algorithm and the contact is taken into account in a separate procedure. The contact can be taken into account in the analysis or by doing the analysis without contact and correcting the results afterwards. However, it is better to consider the contact in the analysis, which is more correct and more simple, too.
- There is relevant test data (test report, approval document) to define the characteristic strength of the connector in any direction of stress and any direction of a joint line (with reference to the connector direction).

Discussion about the assumptions

Assumption 1: Connector and truss tests support this assumption, connectors do not cut off or buckle on the top of timber.

Assumption 2: This assumption corresponds to the general design praxis in Europe, and is conservative and safe. The American praxis is that a small part of the connector outside the timber periphery is considered as effective, too.

Assumption 3: This is a practical consequence from assumption 2.

Assumption 4: A stressed connector always has elastic deformation. It has plastic deformation, too, if the stress gets high enough. This assumption means there is no plastic deformation in the design state i.e. the design state is the ultimate elastic state. This assumption does not mean that the connector should not have any yielding capacity. Actually, this design model is consistent only if the connector has yielding capacity. Yielding brings some favourable, as well as some unfavourable effects, and gives over several consequences which makes the problem more complicated.

Assumption 5: The elastic deformation of the connector is small compared to deformations caused by other reasons e.g. deformation in the connector-timber interface, deformation which occurs in timber and the deformation caused by steel yielding. The elastic deformation which occurs in the connector is so small it may be assumed to be nil.

Assumption 6: Assumption 5 states that the connector is completely solid and has no deformation. However, in the derivation of the equations of this model one has to assume there is some (very small, fictitious) deformation. The numerical value of stiffness in one location of the joint line has meaning only as a relative value with reference to stiffness in another location.

This assumption, however, leads to some practical problems because strength depends on the stress direction which is unknown (before the joint design) and an approximation would be useful. There are several alternatives e.g.:

a It is assumed that the stiffness is uniform. This assumption works well if the joint line consists of one single line and there is not any moment acting in the joint. In this case this assumption is fully (theoretically) correct.

b It is assumed that the stiffness is distributed along the joint line according to the stiffness which is defined by force direction. If there is not any moment in the joint this assumption is quite correct.

c Some stiffness distribution is assumed, stress distribution is calculated, stiffness distribution is corrected and the calculation is continued until a stabile result is obtained in an iterative process.

The force direction is known in the beginning, therefore assumption b is better than a. Assumption b is also useful in the first cycle of the iterative process. Depending upon the accuracy desired methods a, b or c should be used. According to experience the iteration convergence is good. Depending upon the number of iteration cycles one may talk about first, second, third e.t.c. order joint design theory. The second iteration cycle (two calculations) is somewhat stable i.e. the second order theory is accurate enough.

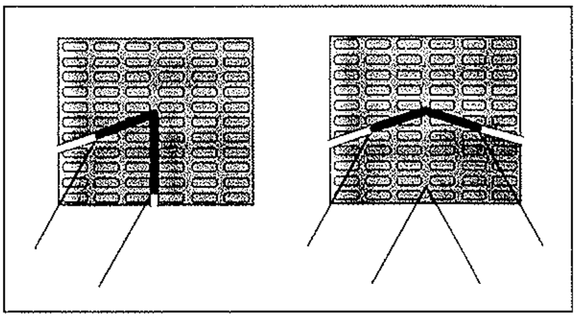
No tests are made on joint line stiffness and one may ask whether it is justified to assume that stiffness is distributed according to strength. Firstly, it is logical that the joint line which is stronger, is stiffer, too. Secondly, it is a common principle in structural design that if the structure has yielding capacity and if the stresses are distributed over the structure in any way and if the strength design is made based on this stress distribution, the design is safe. Connector plates have yielding capacity, so this assumption is safe.

Assumption 7: This assumption corresponds to the general elastic design condition.

Equations

The design equations for steel strength are applied after analysis of the truss. In this phase all internal loads, normal force F, shear force Q and moment M are known in all the beam elements.

Fig 2
An elementary joint which includes one timber member, one connector and one joint line (failure line) across the connector dividing the connector into two parts. The whole design is based on this elementary joint i.e. all the joints are made of this elementary joint. A timber member may also be a "super" member where two or more timber members are considered as one solid timber element.



Elementary joint

The idea of this design model is to check that each timber member is able to transfer the internal loads from timber to connector without failing. Most of the failure lines are linked to one timber member and one connector. There are some failure lines, too, which are linked to one connector and two (or even more) timber members. In this case the several timber members are considered as one solid "super" timber member which is in complete analogy to the case of one timber member and one connector. So, in all cases steel failure analysis can be made by studying an elementary case which includes one timber member (one single member or "super" member) and one connector which form one failure line (just one line or polygon) across this connector. If the joint has several connectors close to each other all these connectors are studied separately. So, during the study there is one connector divided into two halves by the failure line. The whole design procedure can be made only by studying one connector half. Due to the equilibrium conditions, the joint line stresses are the same whichever connector half is used as a basis. The result is the same in both cases.

Stiffness

It is assumed that the spring stiffness of the joint line is K and it is a linear function of strength f, therefore K = k f. According to assumption 6 this stiffness is defined by the equation:

$$\sigma_s = K\,\delta \qquad (1)$$

where

σ_s spring force acting in the joint line along one unit length (e.g. mm),
δ spring displacement in the joint line.

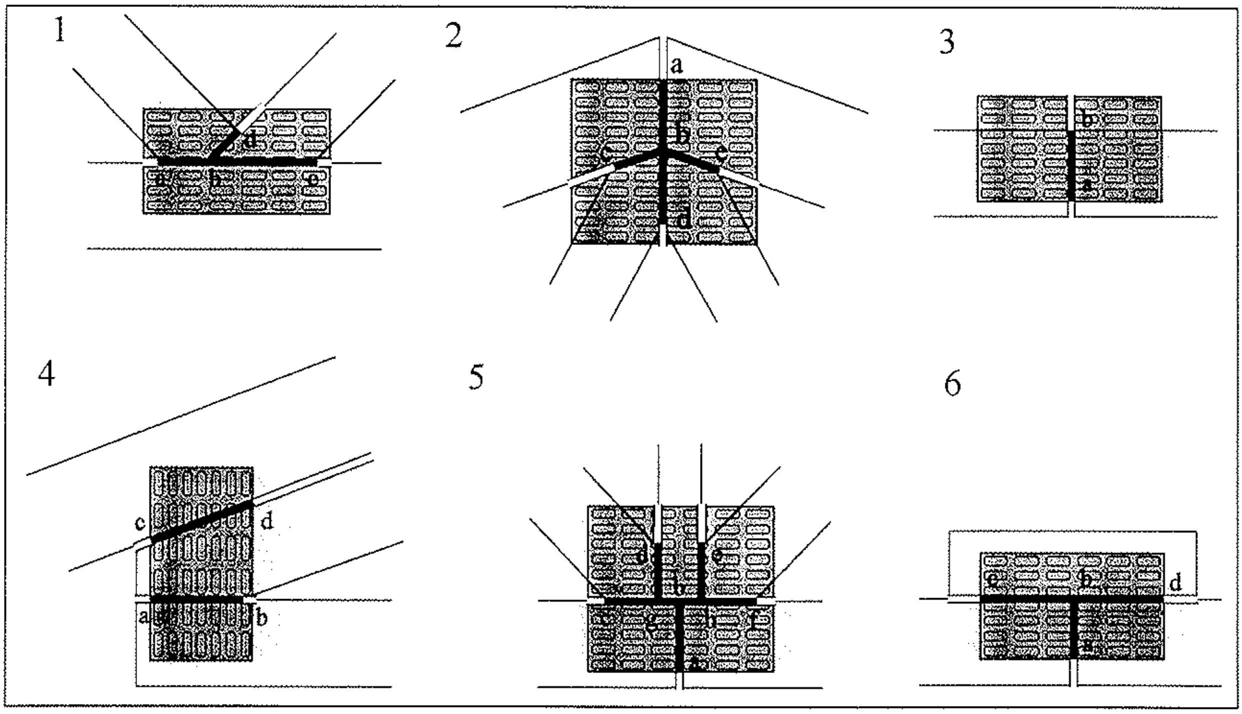


Fig 3
A figure demonstrating the effective joint line of some joints. In these joints there are several elementary joints and several joint lines, the joint lines are:

1 a-b-d, a-b-c, d-b-c,
2 c-b-a, c-b-d, a-b-e, d-b-e, c-b-e, a-b-d,
3 a-b,
4 a-b, c-d, a-b+c-d (note the discontinuous joint line),
5 c-g-d, c-h-e, c-f, c-b-a, d-g-h-e, d-g-f, d-g-b-a, e-h-f, e-h-b-a, a-b-f (this joint has several elementary joint lines containing three lines, there are 10 elementary joints altogether),
6 c-b-d, c-b-a, a-b-d.

Because the stiffness has some meaning only as a relative value i.e. the actual stiffness value has no meaning, it is practical in the derivation of the equations to replace the stiffness K by the strength f, i.e. K = f. This stiffness is denoted in the following by f_k (= K), which has the unit of stiffness (e.g. N/mm^2) but its numerical value is the strength value (which may have any unit e.g. N/mm).

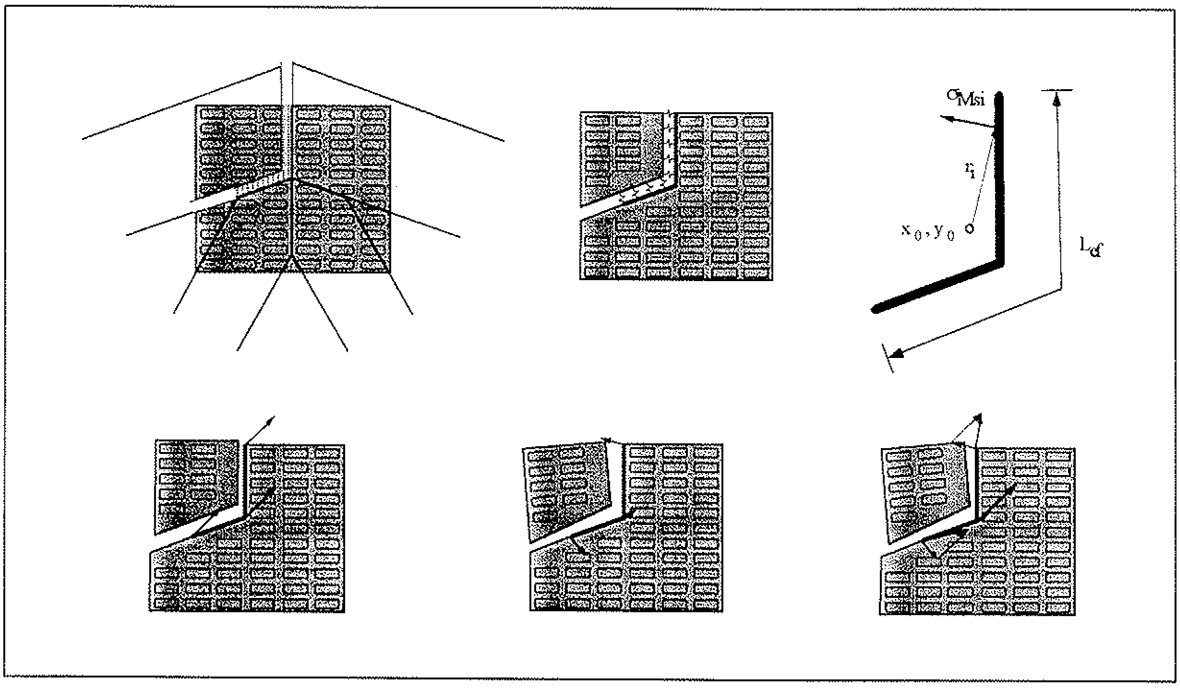


Fig 4
The principle of the model is demonstrated in an apex joint at the top left, a structural model is in the top center, the principle of moment calculation at the top right. Displacement and stresses induced by force is at the bottom left, and those induced by moment in the centre. Total displacement and stresses is at the bottom right.

Effective joint line

The failure line is the timber periphery line which stays under the connector and where the periphery touches another timber i.e. those parts of the periphery line where the timber periphery is not supported (i.e. the periphery does not touch another timber) are considered as inactive. In some cases the connector approval prescribes a fixed reduction to be made in the connector ends. If the current connector is of the type where a reduction must be made, the connector length is reduced. There may be reductions made due to a plate mispositioning. However, these reductions are not applied and it is assumed that the small mispositioning of the connector does not affect the steel strength. It is also assumed that the timber periphery is the actual and the nominal timber periphery e.g. not the periphery of the effective area A_{ef}. The effective joint line (failure line) is denoted as l_{ef} and its actual length as L_{ef}.

Internal loads

The internal load which is used as the basis of design is the load in the connector in such a location where the force load does not cause rotation in the joint line. This location is the elastic centroid of the joint (failure) line x_0, y_0 defined as:

$$x_0 = \frac{1}{L_{kef}} \int x \, f_k \, d(l_{ef}) \tag{2}$$

$$y_0 = \frac{1}{L_{kef}} \int y \, f_k \, d(l_{ef}) \tag{3}$$

In these equations it is denoted $L_{kef} = \int f_k \, d(l_{ef})$, which may be called "stiffness weighted effective joint line length".

The internal load of the joint line is from now on determined in this location (x_0, y_0). In the truss analysis the internal load is usually given in frame nodes only. This means that the frame analysis results must be worked further until one can do the joint design. The joint line load is defined by force components in the direction of the x- and y-coordinates: F_x, F_y and moment M.

Stresses induced by force

The force vector $\overline{F} = (F_x, F_y, 0)$ which is located in the centroid of the effective joint line causes, according to assumption 6, a stress vector $\overline{\sigma}_F = (\sigma_{Fx}, \sigma_{Fy}, 0)$:

$$\overline{\sigma}_F = \frac{1}{L_{kef}} f_k \overline{F} \tag{4}$$

and in the scalar form:

$$\sigma_{Fx} = \frac{1}{L_{kef}} f_k F_x \tag{5}$$

$$\sigma_{Fy} = \frac{1}{L_{kef}} f_k F_y \tag{6}$$

Stresses induced by moment

As the moment M acts in the joint line L_{ef} of the connector there occurs a rotation displacement Φ around the centroid x_0 ,y_0. The displacement of the joint line between the two parts of the connector in location i is:

$$\delta_i = r_i \Phi \tag{7}$$

where

δ_i displacement between the two parts of the connector in location i, this is equal to the spring displacement in location i,

r_i distance between the location i and the centroid.

From (1) and (7) we can write that the spring stress σ_{Ms} induced by the rotation Φ (and the moment) in location i is:

$$\sigma_{Msi} = K \, \Phi \, r_i \tag{8}$$

The moment M acting in the centroid of the joint line is the sum of the elementary moments, $dM = r_i \, \sigma_{Msi} \, d(l_{ef})$, acting on the whole joint line L_{ef}. When K is replaced by f_k, we can write:

$$M = \int r \, f_k \, \Phi \, r \, d(l_{ef}) \tag{9}$$

$$M = \Phi \int r^2 \; f_k \, d(l_{ef}) \tag{10}$$

$$M = \Phi \, I_p \tag{11}$$

where $I_p = \int r^2 \; f_k \, d(l_{ef})$ is the (stiffness weighted) polar moment of the inertia of the joint line L_{ef} with reference to its centroid x_0 ,y_0.

From (8) and (11) we can write

$$\sigma_{Msi} = \frac{1}{I_p} f_k \, r_i \, M \tag{12}$$

where σ_{Msi} is the joint line stress in location i induced by moment M. This stress acts perpendicular to the lever arm r_i. This equation may also be written in vector form:

$$\overline{\sigma}_{Msi} = - \frac{1}{I_p} f_k \, \overline{r}_i * \overline{M} \tag{13}$$

where

$\overline{\sigma}_{Msi}$ stress vector (σ_{Msxi}, σ_{Msyi}, 0) induced by moment $\overline{M}$ in location i
$\overline{r}_i$ liver arm vector (r_{xi}, r_{yi}, 0) between the centroid of the joint line and location i
$\overline{M}$ moment (0, 0, M_z) acting in the centroid of the joint line.

Equation 13 is in a scalar form:

$$\sigma_{Msxi} = -\frac{1}{I_p} f_k r_{yi} M_z \qquad (14)$$

$$\sigma_{Msyi} = \frac{1}{I_p} f_k r_{xi} M_z \qquad (15)$$

Joint stresses induced by force and moment

The total stress is the vector sum of the stresses induced by force and moment i.e.:

$$\overline{\sigma}_{si} = \overline{\sigma}_{Fsi} + \overline{\sigma}_{Msi} \qquad (16)$$

By using a scalar form:

$$\sigma_{sxi} = \frac{1}{L_{kef}} f_k F_x - \frac{1}{I_p} f_k r_{yi} M_z \qquad (17)$$

$$\sigma_{syi} = \frac{1}{L_{kef}} f_k F_y + \frac{1}{I_p} f_k r_{xi} M_z \qquad (18)$$

The total shear stress acts from the plate to the timber in the direction

$$\varphi_{\sigma si} = \mathrm{ARCTAN}(\sigma_{sxi}, \sigma_{syi}) \qquad (19)$$

Equations 16-19 define the steel stresses in the joint line. These stresses must not exceed the critical stresses i.e. the characteristic strength capacity in any location of the joint line.

Design

In application of this model it is not essential what kind of design equations are used and in which form the strength data is presented.

Strength data

There are two alternative ways to express the connector plate strength:

Method 1: The connector tension strength $f_t(\alpha)$ is given in angles $\alpha = 0\text{-}90°$, the connector compression strength $f_c(\alpha)$ is given in angles $\alpha = 0\text{-}90°$ and the shear strength $f_v(\alpha)$ is given in angles $\alpha = 0\text{-}180°$. These strength values are obtained from tests where the connector is tested under tension, compression and shear with several angles α. In this alternative the design equations are written in the direction of the joint line and perpendicular to it.

Method 2: The connector strength is given only in the main directions $\alpha = 0°$ and $\alpha = 90°$. Tests are made to determine the six strength values:

- $f_{t,0}$ tension capacity, $\alpha = 0°$,
- $f_{t,90}$ tension capacity, $\alpha = 90°$,
- $f_{c,0}$ compression capacity, $\alpha = 0°$,
- $f_{c,90}$ compression capacity, $\alpha = 90°$,
- $f_{v,0}$ shear capacity, $\alpha = 0°$,
- $f_{v,90}$ shear capacity, $\alpha = 90°$,

In this case the design equations are written in the direction of the connector and perpendicular to it. As there exists test data only in the connector's main directions, the strength values in other directions are calculated with special rules from these test results. These calculation rules can not be as exact as the test values (method 1), therefore this method is more approximate than method 1. Method 2 is new and has been introduced in the new Eurocode 5 design code.

Design condition

Traditionally the connectors have been tested according to method 1 and the design conditions have been such that the tension and compression stress and the shear stress must not exceed the strength capacity:

$$\sigma_{sp,\alpha} \leq f_{p,\alpha} \qquad (20)$$

In this equation $f_{p,\alpha}$ applies to compression strength $f_{c,\alpha}$ or tension strength $f_{t,\alpha}$ depending upon whether the joint is under compression or tension, and

$$\sigma_{sv,\alpha} \leq f_{v,\alpha} \qquad (21)$$

which applies to the shear stress. The assumption in applying these equations is that the shear, the tension and the compression stresses do not affect each other. This assumption most probably is unsafe and therefore the following design condition (22) is recommended to be used instead of (20) and (21).

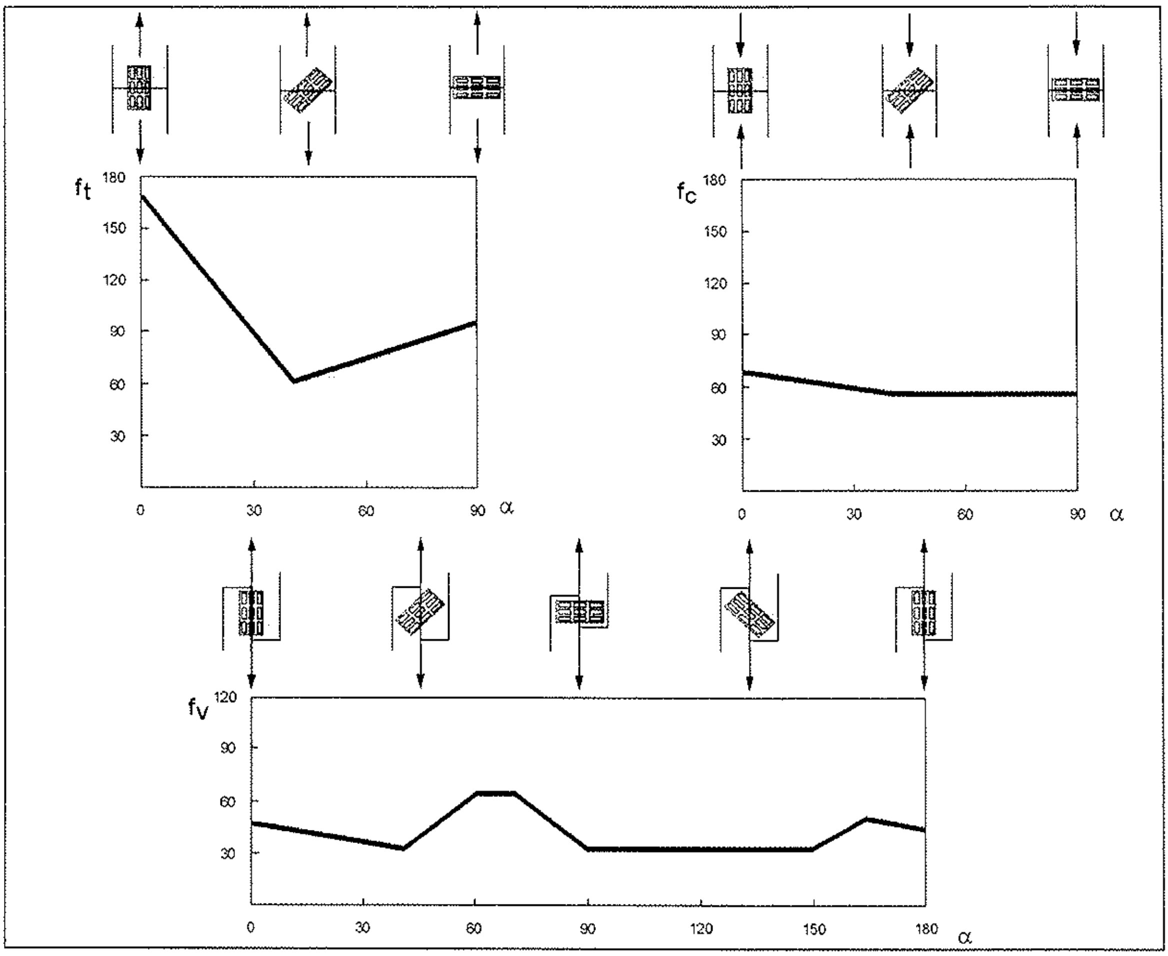


Fig 5
This figure shows how the connector capacity data (N/mm) of a typical connector is presented in method 1. Connector tension capacity $f_{t,0\text{-}90°}$, compression capacity $f_{c,0\text{-}90°}$ and shear capacity $f_{v,0\text{-}180°}$ are given. In method 2 there exit only the capacity values in angles $\alpha = 0°$ and $\alpha = 90°$.

$$\left(\frac{\sigma_{sv,\alpha}}{f_{v,\alpha}}\right)^2 + \left(\frac{\sigma_{sp,\alpha}}{f_{p,\alpha}}\right)^2 \leq 1 \tag{22}$$

If method 2 is used the design condition is:

$$\left(\frac{\sigma_{sx}}{f_x}\right)^2 + \left(\frac{\sigma_{sy}}{f_y}\right)^2 \leq 1 \tag{23}$$

where

f_x connector strength in x-direction,
σ_{sx} connector stress in x-direction (main direction),
f_y connector strength in y-direction,
σ_{sy} connector stress in y-direction.

In this design model both the equations (22) and (23) can be used.

Conclusion

1 In the proposed design model joints are divided into elementary joints which consist of one timber member (one single member or one super member) and one connector. A design pattern dealing with this elementary joint can handle all cases, which makes programming easy.
2 The model is consistent, especially when two important factors are taken into account:
   - The design state is the ultimate elastic state in the analysis as well as in the joint design. The joint design matches the analysis.
   - Equilibrium conditions are fulfilled, and eccentricities especially load eccentricity and strength eccentricity are taken into account.
3 The model is general, is applicable to all joints (simple as well as complicated) and should work well in all cases.
4 The essential point in this model is the distribution of internal loads along the joint line, accordingly, the model can be defined by the following statement: "In plate strength design the internal loads of the joint may be distributed along the joint line assuming the joint line has linear stiffness corresponding to connector strength".

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CIB-W18/27-10-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

DETERMINATION OF SHEAR MODULUS

by

R Görlacher

J Kürth

University of Karlsruhe

Germany

MEETING TWENTY - SEVEN

SYDNEY

AUSTRALIA

JULY 1994

Determination of shear modulus

by

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1 Introduction

In prEN 338 "Structural timber - Strength classes" and prEN 1194 "Timber structures - Glued laminated timber - Strength classes and determination of characteristic values" the ratio of the mean modulus of elasticity $E_{0,mean}$ to the mean shear modulus G_{mean} is assumed to be constant for all strength classes with:

$$\frac{E_{0,mean}}{G_{mean}} = 16 \qquad (1)$$

This assumption presumes that material properties such as density or knots-area ratio (KAR) have no or equal influence on both the E- and the G- value; otherwise, the E/G ratio can not be constant for all strength classes.

Test methods for determining the modulus of elasticity and the shear modulus are specified in prEN 408. The test specimen shall normally have a minimum span of 18 times the depth of the cross section. The values of E and G then represent average values over the whole test specimen length.

In a former research project on glulam beams the modulus of elasticity of board sections of 15 cm length was determined in order to obtain data about the variation of E along the board length. A special test procedure based on vibration time measurements was used.

2 Test method

Pieces of steel are glued to the wood specimen on both ends of the 15-cm-long board section (**Fig. 1**). With that kind of test specimen longitudinal vibrations can be forced easily by striking in longitudinal direction. Vibration times were measured with a special equipment (GRINDO SONIC) and the modulus of elasticity E_0 was calculated following the method proposed by Görlacher (1990).

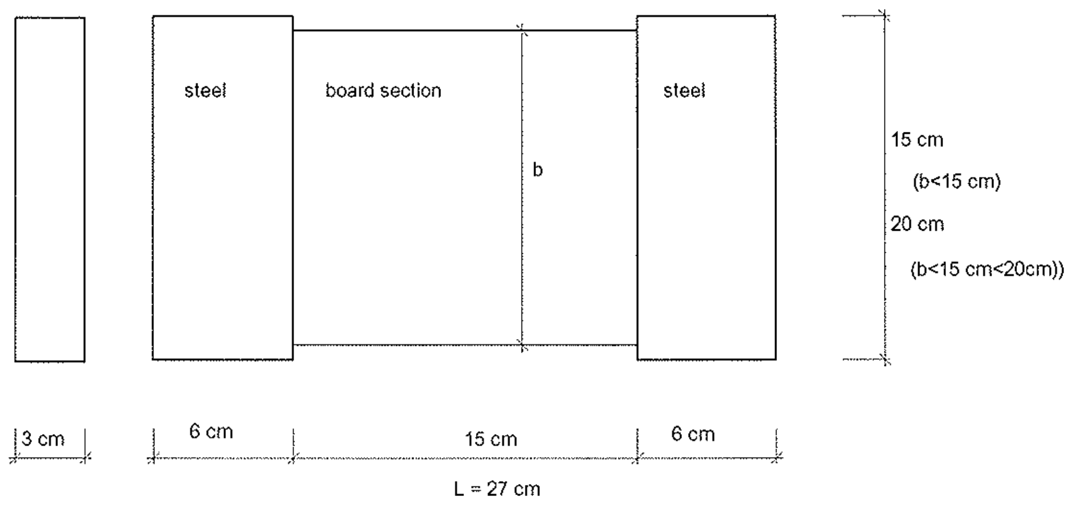

Fig 1 Test specimen

In addition to longitudinal vibrations also torsional vibrations were induced to the test specimens. The vibration time was measured, and the shear modulus was calculated as described in the following section.

3 Shear modulus from vibration tests

For wood as an anisotropic material the theory of torsion normally requires two shear moduli, radial and tangential to the orientation of the growth rings, but they can be replaced by a combined single shear modulus (G_{tor}).

This torsional shear modulus G_{tor} can be calculated for rods with circular cross section from the torsional vibration time by:

$$G_{tor} = (\frac{2\pi l}{T})^2 \cdot \rho \qquad (2)$$

with:

l = length of the specimen; T = torsional vibration time; ρ = density

For test specimens with steel plates at both ends, as used in this investigation, a shape factor R is needed (see eq. 3) taking into account the rectangular cross section of the wood and the steel plates and the different material properties of steel and wood. This leads to the following equation:

$$G = G_{tor} = \left(\frac{2\pi l}{T}\right)^2 \cdot \rho \cdot R \qquad (3)$$

where

l = length of whole specimen (including the steel plates); T = torsional vibration time; ρ = density (steel); R = shape factor

The shape factors for different test specimens with different height and width can be determined by a dynamic analysis using the method of finite elements (see **Fig. 2**).

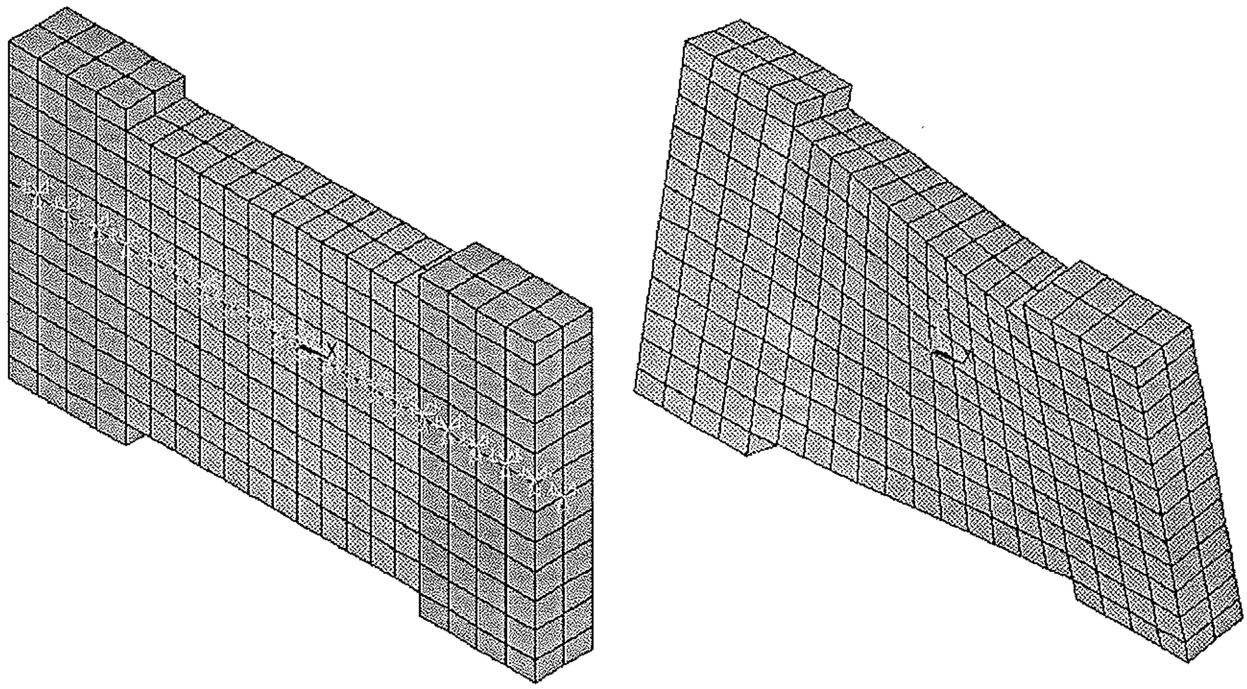

Fig. 2 Test specimen modelled with FE-methods in torsional vibration

4 Tests

The test specimens were taken from 48 boards of different origins, taken from German glulam factories. The dimensions of the boards were:

length: ca. 4,50 m
height: between 10 and 19 cm
width: between 30 and 35 mm

with a moisture content of ca. 12 %. Each board was cut into sections of 15 cm length (25 - 30 sections/board). In total, 1188 specimens were tested.

From each section the modulus of elasticity, the shear modulus, the density of the clear wood (u = 12 %) and the knot area ratio were determined.

5 Test results

The results of the tests are shown in **Fig. 3**, where the shear modulus is drawn in relation to the density of wood (moisture content 12 %). With increasing density G is increasing. A linear regression analysis leads to eq. (4):

$$G = 1,78 \cdot \rho - 146 \qquad r = 0,66 \qquad (4)$$

with ρ in kg/m^3 and G in N/mm^2.

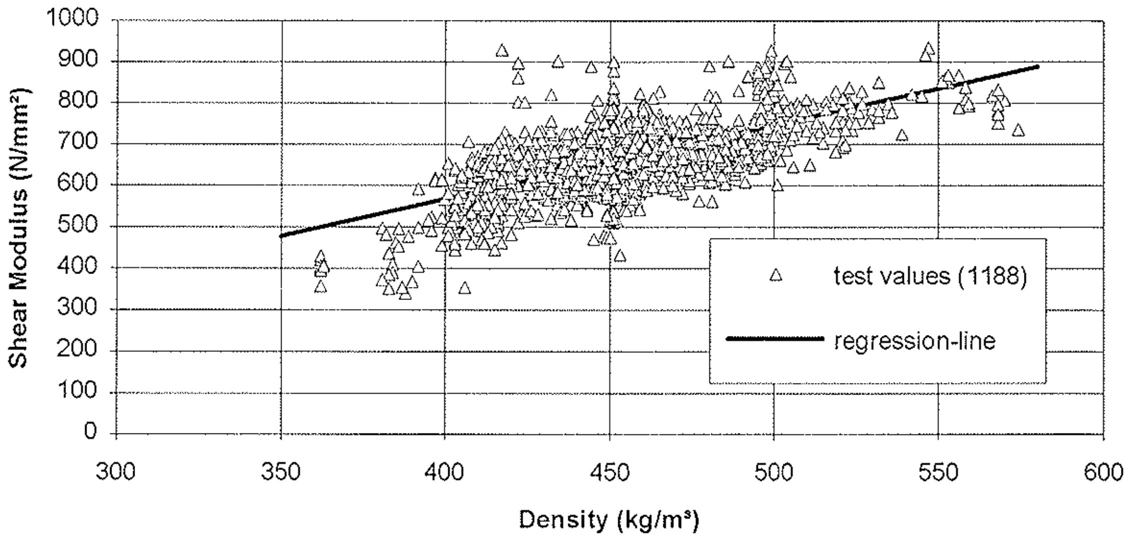

Fig. 3 Shear modulus in relation to the wood density (moisture content 12 %)

In **Fig. 4** the ratio E_0/G of modulus of elasticity to shear modulus is shown in relation to the modulus of elasticity. A linear regression analysis results in eq.(5):

$$\frac{E_0}{G} = 0,00112 \cdot E_0 + 5,43 \qquad r = 0,66 \qquad (5)$$

with E_0 in N/mm^2.

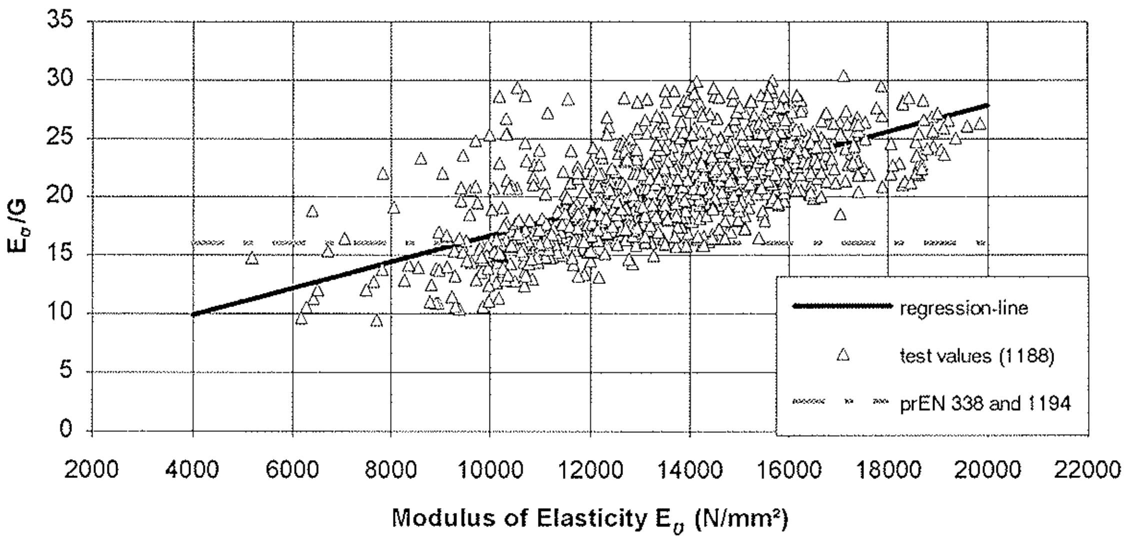

Fig. 4 Ratio of E_0/G in relation to the modulus of elasticity E_0.

6 Conclusions

Tests with boards lead to the finding that the ratio E_0/G depends on the modulus of elasticity. With increasing modulus of elasticity the ratio E_0/G is also increasing. Since in prEN 338 the modulus of elasticity is assumed to be different in each strength class, the E_0/G value should also be different in each strength class. A proposal is given in **Table 1.**

It should be mentioned that the tests were carried out only on test specimens with dimensions of cross sections of boards with thicknesses up to 35 *mm*. Further investigations are desirable with other cross-sectional dimensions. In any case, the proposal could be realistic for the strength classes of glued laminated timber, as these beams are made of boards.

Table 1 Comparison of test results with assumptions in prEN 338

According prEN 338				Test results	
strength class	$E_{0,mean}$ N/mm^2	E_0/G	G_{mean} N/mm^2	E_0/G -	G N/mm^2
C18	9000	16	560	16	580
C22	10000	16	630	17	600
C24	11000	16	680	18	620
C30	12000	16	750	19	640
C35	13000	16	810	20	650
C40	14000	16	880	21	660

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CIB-W18/27-12-1

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

STATE OF THE ART REPORT : GLULAM TIMBER BRIDGE DESIGN IN THE U.S.

by

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MEETING TWENTY - SEVEN

SYDNEY

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JULY 1994

ABSTRACT

Structural glued laminated timber has been successfully used as a highway bridge material in the United States for approximately 50 years. From the mid 1940's to the mid 1960's, virtually all of these bridges were longitudinal girder or arch type glulam superstructures with a nail-laminated wood deck or some form of composite concrete deck. The next evolution of these bridges occurred between the early 1970's and the late 1980s, when the large majority of these bridges were constructed using longitudinal glulam girders and transverse glulam decks or longitudinal glulam deck superstructures manufactured from conventional softwood lumber species. Recently, highway bridge applications employing glued laminated timber have been expanded to include alternative wood species and new designs utilizing the concept of stress-laminating. Additionally, current research using composite plastic materials in conjunction with glulam may lead to future innovations in timber highway bridges.

INTRODUCTION

Structural glued laminated timber (glulam) is an engineered, stress-rated product of a timber-laminating plant. It consists of selected and prepared lumber laminations that are bonded together on their wide faces with structural adhesives. Glulam has been successfully used as a structural building material in Europe since the 1890s. In the United States, it has been used in buildings since approximately 1935. The introduction of wet use adhesives in the mid 1940's allowed the uses of glulam to be expanded to include exposed applications such as highway and railway bridges, transmission facilities and other structures.

Glulam is a versatile stress-rated wood material that provides several distinct advantages for bridge construction. Because it is a manufactured product, glulam can be produced in a wide range of shapes and sizes to fit virtually any end use requirements. Most glulam used in bridges involves straight members, but curved members have also been used successfully in a number of applications. For example, the Keystone Wye bridge was built in 1968 in South Dakota as a unique tri-level interchange using both a straight girder glulam bridge and a long span glulam arch structure. The upper bridge structure has an overall length of 88 meters with an arch span of 49 meters. This high visibility bridge structure has performed well for almost 25 years with only minimal maintenance being required such as re-staining the glulam members to preserve the aesthetic appearance of the structure.

Recent installations of glulam arch highway bridges have been constructed in Colorado and Michigan and other states with several of these modern structures winning awards in a national Timber Bridge Awards program. This national awards programs acknowledges outstanding achievements in timber bridge design in four categories, these being, long span vehicular, short span vehicular, light vehicular/pedestrian and rehabilitation of an existing timber bridge.

Another advantage of glulam as compared to sawn lumber is related to the laminating process which randomly disperses the strength-reducing characteristics (src), such as knots, throughout the member. This random dispersal of src's, results in reduced material properties variability and increased strength characteristics. Glulam also provides better dimensional stability because it is manufactured from dry lumber.

For horizontally laminated bending members, glulam is manufactured using selective lamination placement so that higher quality material can be positioned in the top and bottom of the beam, where bending stress is greatest, and lower quality material can be placed in the inner layers of the beam, where bending stress is lowest. This practice helps to extend the available lumber resource and improves the economy of the final glulam product.

While the majority of the glulam bridges built in the United States have been conventional girder-deck or longitudinal deck superstructures (Ritter, 1990), there has been considerable research activity since the late 1980's to extend the use of glulam for bridge applications into several innovative areas. This paper will briefly describe the evolution of modern glulam bridge design in the U.S. including the development of vertically laminated deck systems, the use of alternative species for glulam manufacture, the introduction of technology for glulam stress-laminated decks, T and box sections and the development of crash tested timber guardrail systems.

GLULAM DECK PANEL TECHNOLOGY

During the late 1960's, research engineers at the USDA Forest Products Laboratory, in cooperation with Forest Service regional bridge engineers and the glulam industry undertook a research program to develop a glulam deck panel to replace the traditional nail-laminated deck system. The concept was to use a vertically laminated glulam member spanning transversely across the longitudinal bridge girders. The length of the deck panel was equal to the overall width of the bridge with the thickness of the panel being dependent on the grade and species of laminating lumber and the spacing of the deck panels. In order to facilitate handling of these deck panels at the manufacturing facility, during transportation and on the jobsite, an arbitrary decision was made to fabricate these deck panels in widths of approximately 122 cm.

In order to achieve plate action for this deck system along the longitudinal direction of the bridge and to minimize the differential deflection at the joints beyond the individual panels, several alternative load transfer mechanisms between panel interfaces were evaluated. The most efficient was the use of a steel dowel inserted in holes pre-bored at the mid-depth of each panel face. While hundreds of bridges were successfully constructed using this dowel system, problems were encountered in the field when attempting to pull the individual panels together.

To provide the required load transfer between panel edges but not require the close construction tolerances associated with the steel dowel system and it's associated pre-bored deck panel holes, the Weyerhaeuser Company developed a cast aluminum bracket. This bracket was positioned in grooves pre-routed in the side of the glulam girders prior to pressure treating and was attached to the deck panels with a single through bolt as shown by Figure 1. Since the bracket is manufactured from aluminum, this eliminates concerns of corrosion. The use of this deck bracket essentially replaced the steel dowel and became the state of the art for this system.

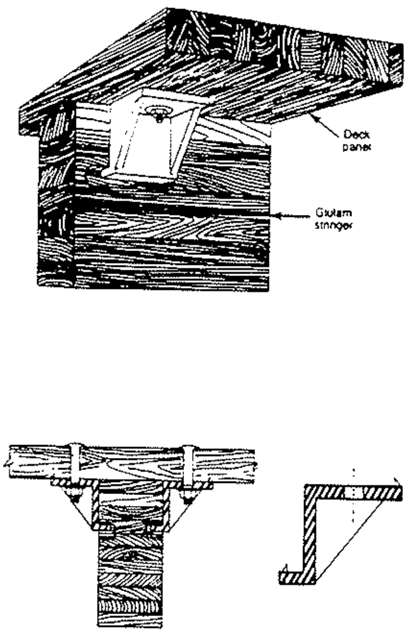

Figure 1 Cast Aluminum Deck Bracket Installation

A natural evolution of the transverse glulam deck panel was to use these members as vertically laminated beams spanning longitudinally between supports. However, the load distribution provisions in the American Association of State Highway and Transportation Officials (AASHTO) Specifications for Highway Bridges were not favorable to the use of these longitudinal deck panels. The glulam industry sponsored an extensive test program of this system which was conducted at Iowa State University. The results of this study led to more favorable and realistic distribution factors for this type of deck system which were adopted in the AASHTO standards.

As with the transverse deck panels, the longitudinal deck panels are also manufactured in widths of 122 cms. This created a necessity to develop a mechanism for transferring loads transversely between these longitudinal panels. Thus, in addition to developing new load distribution factors for the longitudinal glulam deck panel system, the Iowa State research also led to design provisions for stiffener beams which are beams (glulam or other materials such as steel W or I sections) positioned transversely beneath the longitudinal deck panels at approximately 2.45 meters on center. These stiffener beams can be attached to the deck panels with a variety of mechanical fastening devices as shown by Figure 2. One of the most successful has been the use of the same cast aluminum deck bracket used to attach transverse deck panels to longitudinal stringers as shown in the top detail of Figure 2.

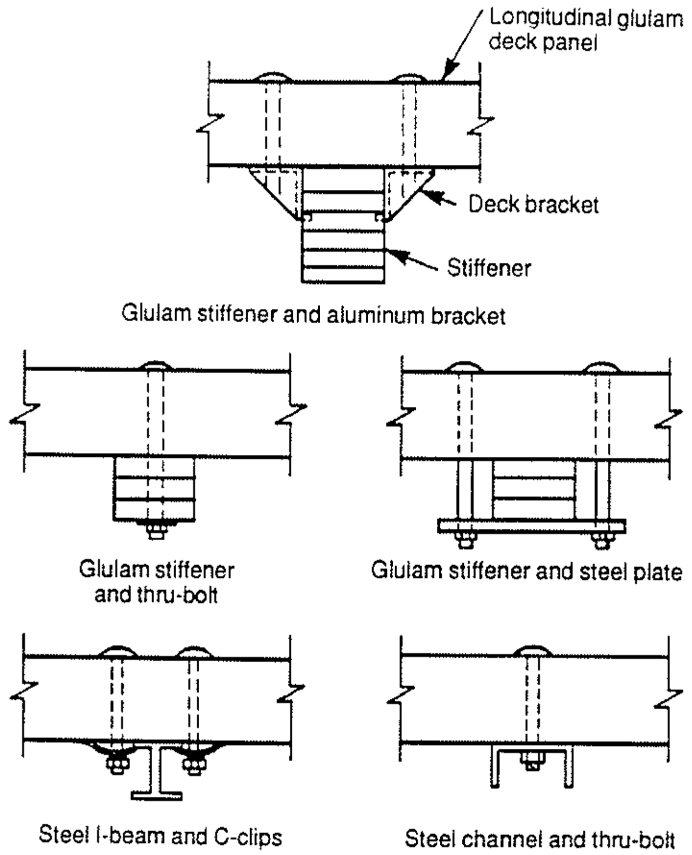

Figure 2 Attachment of Stiffener Beams to Longitudinal Glulam Decks

STRESS-LAMINATED GLULAM DECKS

Due to the general lack of availability of laminating lumber in sizes greater than 2x12's or 2x14's the use of the vertically laminated longitudinal deck systems was limited to spans of approximately 10 meters. Thus, an alternative system was sought which would permit the construction of longitudinal deck systems (those without girders) for spans greater than 10 meters. It was conceived that one such solution would be to apply the concept of stress-laminating to a series of longitudinal glulam beams placed side by side.

Stress-laminating has been an evolving technology in both Canada and the U.S. for the past 5-10 years and has achieved considerable success in highway bridge construction. The idea of using stress-laminating techniques for the rehabilitation of existing timber bridges and for the construction of new timber bridges was first introduced in Canada. The first use of this emerging technology in the U.S. was in the late 1980's. Since that time, over 150 bridges have been constructed in the U.S. using sawn lumber laminations, and a guide specification for the design of this type of timber bridge has been published by the American Association of State Highway and Transportation Officials (AASHTO, 1991).

Stress-laminated decks are typically constructed by placing sawn lumber laminations (either 2x, 3x or 4x material) on edge and stressing the laminations together on the wide face with high-strength steel bars threaded through the laminations (Ritter, 1990). The compression stress existing between the laminations serves to transfer load between the laminations by friction, causing the deck to act as a large orthotropic wood plate.

In 1989, the concept of stress-laminating decks was expanded to use glulam beams as the deck laminations, rather than sawn lumber as shown by Figure 3. Using this approach, glulam beams of variable width, which are continuous between supports, are stressed together to form the bridge deck. The first known example of this type of construction was the Teal River bridge constructed in Wisconsin (Wacker and Ritter, 1992). Since construction of this bridge, several other structures have been built, including a second bridge in Wisconsin and one in West Virginia.

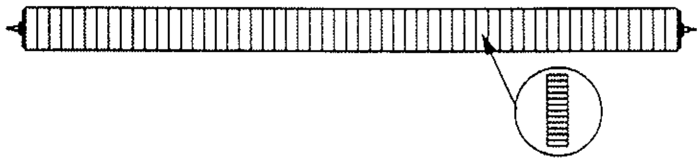

Figure 3 Stress-laminated Deck Using Glulam Beams

Bridges using glulam in stress-laminated deck applications have demonstrated excellent performance. Because horizontally laminated glulam beams allows for deeper sections, longer bridge spans are possible. Additionally, the glulam beams can be manufactured to be continuous over the bridge length and butt joints, which can reduce the bridge strength and serviceability of sawn lumber stress-laminated decks, are not required. These continuous long length beams can also be used to span across intermediate supports resulting in very high stiffness multiple span bridges, further reducing bridge deck deflections.

One of the most noteworthy advantages of glulam use has been the force retention in the pre-stressing bars. Because the glulam members are dry when installed (moisture content of 16% or less), the beams typically absorb moisture slowly and the deck swells slightly as it moves toward an equilibrium in-service moisture content. As a result, this minimal swelling offsets force loss in the pre-stressing rods due to stress relaxation in the wood and the net loss in bar force is minimal. Extensive monitoring of these bridges by the U.S. Forest Service has verified this performance characteristic.

STRESS LAMINATED T-BEAM AND BOX-BEAM SECTIONS

In the mid 1970's, an extensive test program was conducted at Colorado State University to determine the degree of T-beam action which could be expected in a longitudinal girder and transverse glulam deck system. Full size double T-sections spanning 12 meters were tested under simulated AASHTO truck loading. These test sections used conventional 122 cm wide transverse deck panels with steel dowels used to provide load transfer between adjacent panels. The deck panels were attached to the stringers using steel lag screws. While there was approximately a 10-15% decrease in stringer deflection, the degree of T-beam action was limited by the effectiveness of the mechanical connections and the associated slip which occurred during loading. Due to the potential varaibility in the degree of fastener slip which might be expected to occur on in-service bridges, it was decided not to pursue a revision to the AASHTO Bridge Specifications to provide for the T-beam action which invariably occurs to some degree in these bridges.

However, the advent of stress-laminating offered new opportunities for achieve more reliable composite T-beam action between the deck and stringers without being dependent on the mechanical fasteners between the deck and stringers. The clear span of glulam bridges is typically controlled by design considerations related to the depth of the superstructure and by economical limitations on the bridge depth. Creating T-section, Bulb T and other composite configurations allowed designers to overcome these limitations, thus permitting much greater span capabilities for glulam bridges.

Two types of experimental composite bridges that have been successfully used in the U.S. are the T-section and box-beam bridges as shown schematically in Figure 4. T-beam bridges can be constructed using vertical glulam web members with flanges constructed of sawn lumber or glulam deck panels. The composite action between the flange and the web is developed through friction by stress-laminating the section with stressing bars through the flanges and webs. The box section is similar to the T-section, but with flanges and stressing bars added to the bottom of the section to create a higher overall section modulus and moment of inertia.

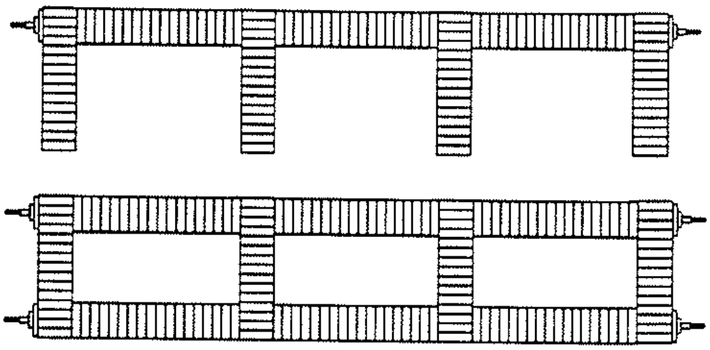

Figure 4 Stress-laminated T-beam Box-beam Bridge Schematics

The concept of stress-laminated T-section and box-beam bridges has been well received and more than 30 bridges have been built over the past 3 years in the U.S. The longest span structure to date is a 27 meter span stress-laminated T-beam bridge, which was built in Arkansas in 1993. Most research work regarding these glulam superstructure configurations was completed at West Virginia University, in cooperation with the Federal Highway Administration (FHWA) and the U.S. Forest Products Laboratory, and used a modular construction approach (Barger, et al., 1993; Davalos, et al., 1993).

In addition to continued research on stress-laminated T-section and box-beam bridges using glulam webs and sawn lumber flanges, research is underway at the University of Wisconsin, in cooperation with the U.S Forest Products Laboratory and the U.S. Federal Highway Administration, to develop systems constructed completely from glulam panels (Oliva and Rammer, 1993). It is estimated that glulam bridges built using this technology will be able to clear span over 30 meters with structural sections less than 106 cm in depth.

Research and field evaluation are continuing on the structural performance of these systems. Draft specifications for the design and construction of stress-laminated T-section and box-beam bridges are currently being developed for submission to AASHTO.

ALTERNATIVE SPECIES

Glulam can be manufactured from virtually any softwood or hardwood species provided the end product meets necessary strength and stiffness requirements. In actuality, most of the glulam manufactured in the U.S. during the past 60 years has utilized either Douglas Fir-Larch or Southern Pine lumber. However, with continuing changes in the availability of worldwide wood resources, and with increased emphasis on using underutilized local wood species, there has been a growing interest in the U.S. towards developing new glulam layups utilizing both hardwood and softwood species. Over the past 4 years, most of the work on alternative species for glulam has centered on the utilization of hardwood lumber, but several secondary softwood species have also been evaluated.

Recent glulam research completed at the Pennsylvania State University, West Virginia University, and the U.S. Forest Products Laboratory has been directed at developing glulam layups using red maple, red oak and yellow poplar (Manbeck, et al., 1993; Shaffer, et al., 1991; Moody, et al., 1993). Although an industry glulam standard for the use of hardwood species has been available for many years, the standard neither uses currently available structural hardwood lumber grades nor provides for efficient use of various grades throughout the beam cross-section as is done with softwoods. Recent full scale tests of glulam beams manufactured using red maple, red oak and yellow poplar indicate that bending design values comparable to those achieved with the traditional Douglas Fir-Larch and Southern Pine softwoods can be attained for these hardwood species.

In addition to developing specifications for glulam produced from hardwood species, efforts to develop high strength and cost efficient glulam layup combinations utilizing secondary softwood species have also been successful. A project in Wisconsin using a combination of red pine and Southern Pine to manufacture glulam beams resulted in the construction of a stress-laminated deck bridge in 1989 (Wacker and Ritter, 1992). These beams used Southern pine for the outer tension and compression zones with the red pine being used for the core of the beams. The resultant beams had similar bending strength and stiffness characteristics as beams manufactured form all Southern pine laminations. This further led to the design and construction of a stress-laminated bridge using glulam manufactured exclusively from red pine lumber. Other projects using secondary softwoods, such as Eastern hemlock, Ponderosa pine and cottonwood, are planned for the future.

CRASH TESTED RAIL SYSTEMS

One ongoing concern expressed by bridge designers in the U.S. has been related to the need for cost efficient crash-worthy timber bridge guardrail systems. AASHTO and the Federal Highway Administration have a program underway which will require all highway bridges guardrail systems to be fully crash tested. Several levels of guardrail performance are being considered in this program ranging from resisting the impact of passenger vehicles to that of large over the road commercial trucks.

Both the U.S. Forest Products Laboratory and the Federal Highway Administration have completed full scale crash test programs to evaluate the performance of various timber bridge guardrail systems on both longitudinal glulam deck and longitudinal glulam stringer and transverse deck bridge configurations.

These crash tests have been conducted using a variety of test vehicles ranging from passenger cars to pick-up trucks to larger commercial trucks. Rail systems tested have included (a) single glulam rail with wood posts, (b) single steel rail with wood posts, (3) glulam rail, wood wheelguard and wood posts and (d) other combinations of guardrail system components. To date, all of the guardrail systems tested in these two research studies have met the crash test requirements established by AASHTO and the Federal Highway Administration. Reports describing these various guardrail crash tests are expected to be available in late 1994. The availability of fully crash tested guardrail systems will provide a major impetus to the further use of glulam highway bridge systems in the U.S.

EMERGING TECHNOLOGIES

In virtually all instances, the bending strength of glulam is controlled by the tensile strength of the lumber or the end joints on the tension side of the beam. The potential for increasing the bending strength of glulam by reinforcing the outer tension zone has been evaluated by many investigators during the past 30 years using a variety of materials. Recent developments in fiber-reinforced plastic (FRP) suggest that this high-performance material offers the possibility of being bonded to the wood laminations under factory conditions thus providing this tension reinforcement.

Forming a composite beam by using a relatively small amount of FRP to reinforce the outer tensile zone offers the potential for significantly increasing the bending strength of glulam beams. However, the use of this reinforcement material may have limited effect on increasing overall stiffness when used in the relatively small percentages required to achieve the increased tensile performance.

Recent work has been completed using various types of fibers in FRP products to reinforce glulam (Tingley, 1990). At a poster session at the 1993 Forest Products Society Meeting in Clearwater, Florida, Tingley and other researchers from Oregon State University reported highly favorable results by reinforcing the tension zone of glulam using FRP with high-strength fibers. Cooperative research is underway between West Virginia University and the U.S. Forest Products Laboratory to investigate similar uses of FRP bonded to either the tension side only or to both the tension and compression sides of beams.

These research efforts could soon lead to the construction of experimental bridges using composite glulam and FRP beams. Reinforced beams have the greatest opportunity of showing economic advantages in applications where either (a) bending strength controls the design, (b) it is critical to minimize beam depth, or (c) the beams are part of a composite structure where the added strength provides substantial benefits.

CONCLUSIONS

Beginning in the late 1960's, extensive research was undertaken in the U.S. to advance the technology for using glulam in highway bridge construction. This research, which has been ongoing since that time, has resulted in many innovative technologies that have been successfully incorporated in numerous glulam highway bridge applications throughout the U.S. Continuing research will undoubtedly expand on existing technologies and lead to new technologies which will create additional opportunities for the use of glulam and other wood products in highway bridge construction.

It is further hoped that much of the glulam bridge technology developed in the U.S. over the past 25 years may have application in other countries where the use of timber in bridge construction is a design option. For example, although not located in the U.S., one of the most striking examples of the innovative use of glulam in highway bridge construction is the recently completed cable-stayed glulam bridge constructed near the airport in Hiroshima, Japan. This two lane wide bridge has a total length of 145 meters with a center clear span between support towers of 84 meters. This bridge uses a glulam truss configuration for the suspended superstructure. Although constructed in Japan, the glulam components for this unusual timber bridge were all manufactured, pre-fabricated for all connections and pressure preservatively treated at manufacturing facilities in the U.S.

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

COMMON DESIGN PRACTICE FOR TIMBER BRIDGES IN THE UNITED KINGDOM

by

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MEETING TWENTY - SEVEN

SYDNEY

AUSTRALIA

JULY 1994

COMMON DESIGN PRACTICE FOR TIMBER BRIDGES IN THE UNITED KINGDOM

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TRADA Technology Limited

INTRODUCTION

There is a well established history and a continued use of timber for bridges in the United Kingdom. In a recently initiated survey (1), a number of interesting designs were encountered, and the majority of these were found to be in good condition. TRADA and more recently TRADA Technology Ltd. (TTL) have a long record of encouraging, fostering, and advising on the use of timber in bridges. The often-mentioned advantages of timber, namely good appearance, low production energy, and weight saving, plus the good durability that can be achieved with correct design, are especially apparent in this high-profile application. TRADA also has long-standing and substantial experience in the use of timber for pre-fabricated modular bridges in developing countries (2). The first of a series of standard designs was prototyped in Kenya some twenty years ago. Modular wooden road bridges are still being produced in accordance with well-tried design manuals and drawings, using local timbers and labour, to the great advantage of rural communities in more than two dozen countries in all of the tropical continents.

Unfortunately, however, the official bridge design scene, viewed from the position of the average civil and structural consulting engineer in the United Kingdom, does nothing to encourage the use of timber for bridges. There is no British Standard dealing specifically with the design of timber bridges. The BS5400 series (3) only covers steel, concrete, and steel-concrete composite bridges. This absence of a British Standard is thought to have inhibited the specification of timber as the structural medium for many footbridges, as well as having resulted in a number of designs whose performance has not been entirely satisfactory. Although other authorities such as Department of Transport (DoT) have their own standards, recognising timber in footbridges to a small degree, the absence of a main code of practice is a deterent.

TTL were extremely impressed by the manner in which the US National Timber Bridge Initiative was launched, and by its subsequent success (4). Their programme involves many demonstration timber bridges, together with research and technology transfer. Starting from a relatively small financial basis, it was difficult to see how anything comparable could possibly be started in the United Kingdom.

Recently however, two positive factors have emerged. Firstly, thanks to extremely understanding support from the Department of the Environment (DoE) Construction Sponsorship Directorate, work has been started which is partly sponsored by government and which is supported by industry through the Collaborative Research Programme. During 1993, this was preceded by a feasibility study which was arranged through an extra-mural contract with the Building Research Establishment (BRE). This led to two preliminary study reports (1) (5). The second positive step is that active work has now been started on drafting Eurocode 5: Design of Timber Structures Part 2: Bridges. TTL is providing the engineer who is the UK representative on the drafting team. It is to be hoped that through these concerted efforts, the enthusiastic use of timber in significant bridge structures which is to be seen in other parts of Europe will cross-fertilize in Britain.

CATEGORIES OF TIMBER BRIDGES

Use categories

Timber bridges in the UK can be considered in three broad categories of use as follows:

1. Highway bridges
2. Footbridges
3. Footbridges with occasional vehicular access (eg farm and parkland bridges)

Category one is very rare in the UK. The present market size for categories two and three is modest, probably in the order of four million pounds sterling per annum. However it is felt that there are growing opportunities for bridge and associated timber suppliers and engineers, especially in the light of the positive factors mentioned in the introduction.

Localities

These are described by means of the following four categories:

1. Over roads.

Many footbridges are used to provide safe pedestrian crossings, and timber is permitted, as well as steel and concrete. However the Department of Transport points out the following in its Standard BD 29/87 (6):

"A footbridge is the least suitable form of crossing for disabled people and should only be provided when other forms of crossing - eg a crossing at grade or a subway are deemed to be unsuitable"

Timber has only a small share of this market. Furthermore its share is probably even smaller, as a result of some unfortunate instances of glulam bridges delaminating during the 1980's. These were manufactured by firms which were not members of the Glued Laminated Timber Association (GLTA), but they achieved notoriety for the industry as a whole.

2. Over railways.

Timber again has only a very small share of this market. There are a few instances, but railway structural engineers have a cautious approach and safety standards are very high.

3. Over rivers, canals and other water features.

This is a most important market, with timber footbridges having a sizeable portion of the total.

4. Associated with the leisure industry.

For example theme parks, zoos, golf courses etc. This is generally another expanding market, especially for timber.

Structural forms

The following five categories of structural form have been devised to describe the majority of footbridge types. The structural forms refer to the principal members making up the bridge:

1. Beams
2. Arches
3. Girder trusses
4. Bascule bridges
5. Suspension and cable stayed bridges

These five categories of bridge based upon the form of the principal members led to the summary shown in Table 1. This also indicates the usual static system for the principal members, which is related in turn to the structural analysis that will be required in the design. Also shown is the form, or shape alternatives for the principal members, and an indication of the materials which are commonly used for each type of bridge.

Table 1: Structural forms of timber bridges

Structural form of principal members	Beams	Arches	Girder trusses	Bascules	Suspension and Cable Stayed
Usual static system	Single simply supported span Multiple simply supported spans Cantilever side spans supporting suspended central span	3-pinned 2-pinned Multiple span	Single simply supported span Multiple simply supported span	Two-leaf cantilever Single-leaf cantilever	Cables/links Single or twin tower with side spans
Form of principal members	Straight, lightly curved or pre-cambered	Circular or parabolic (wide range of radii)	Parallel or near-parallel chorded (often Warren or Pratt trusses)	Main (lower) beams straight or single-tapered Balance (higher) beams straight or double-tapered	Deck beams straight or tapered Towers, parallel masts or A-frames
Common materials	Sawn timber Timber poles Glulam Mechlam	Glulam Mechlam	Sawn timber Glulam	Beams - Sawn timber Glulam Mechlam Portals - Sawn timber Glulam	Beams and towers - Sawn timber Glulam Mechlam

There are usually several options in choosing the elevation of the deck. For most forms of bridge shown in Table 1, these are fundamentally a low-level, mid-level or high-level deck. The choice of deck level has considerable influence on the design of the structure. It also relates to planning considerations and functional aspects. The former include for example headroom for vehicles or vessels beneath the bridge. The latter include the measure of protection provided by the deck to the remainder of the structure.

Possible elevations of the deck are interpreted in relation to the principal structural forms in Table 2. This table also incorporates some notes on variations on the basic forms. It mentions for example roofed bridges. Although these are not common in the UK, they are not unknown.

Table 2: Forms of timber bridge, deck elevations and variations

Principal Form	Beams	Arches	Girder Trusses	Bascules	Suspension and Cable Stayed
Possible elevations of deck	Over beams Between beams At lower edges/chords	Over crown of arch(es) From springing point to springing point Intermediate level	Over girders Between girders Between lower chords	Deck itself, as for beam bridges Overall deck elevation from base to base of portals	Deck itself, as for beam bridges Commonest is overall deck elevation from base to base of towers but towers may straddle
Additions and variations	Can be roofed Beams as part of parapet	A-frame arch Mansard arch with stairs Tied arch, tangent at deck or other level Skewed plan form	Trusses as part of parapet Parallel or pitched top chords Roof	Generally keeps to classical form of Dutch drawbridge Vessel passage also achievable by swing-bridge (another cantilever form)	Single cable, central in plan, from A-framed tower(s), or similar principle using stays

MATERIALS

Principal structural members

In relation to the previous categories of principal structural forms shown in Table 1, the materials which are commonly used are as follows:

1. Sawn timber
2. Timber poles
3. Glulam
4. Mechlam

Sawn timber may be used for all of the forms in Table 1 which do not involve significant curvature of the members and for which adequate lengths can be obtained to meet the span. Sawn timbers can range from small sections of softwood or hardwood for the simplest of short-span beam bridges, through larger sections, more usually hardwood, to very long lengths of specialist hardwoods for the biggest members such as masts for cable stayed bridges and pilings. The types of hardwood used for the intermediate applications include temperate species such as oak, often British grown, and established tropical hardwoods available for structures in the UK. BS5268: Part 2 lists twelve of these, typical examples being Iroko (West Africa) SC5; Keruing (South East Asia) SC7 and Ekki (West Africa) SC8. For the largest lengths and cross-sections, Greenheart (South America) SC9 and Basralocus (not listed in BS5256) are used.

Where smaller sawn cross-sections and lengths are required, there are better opportunities in bridges than in the building market generally for specially valued British grown softwoods. These include Scots Pine (SS grade = SC4), which has good preservative retention, and Larch (three British grown species, SS grade = SC4), which has a degree of natural durability.

Poles are on occasion used as the main beams of small span footbirdges. They are usually of softwood, with a preference for species with a degree of natural durability such as Douglas fir. Debarked, straight poles are preferred. A limit on taper such as 10mm per metre is recommended, and ad hoc grading rules and stresses are derived by organisations such as TTL.

Glued laminated timber (glulam) bridge members are manufactured to BS4169 (7). Both softwood and hardwood laminations are used, the latter to a far greater extent than in glulam beams for buildings. The British timber code and its related standard have a system of grading laminations and performing design calculations which is peculiar to the UK, but which has stood the test of time (8). The LB laminating grade is widely used as a basis, and design starts from the BS4978 (9) SS grade stress in the case of softwoods, and the BS5756 (10) HS grade stress in the case of hardwoods. This situation is currently under review in relevant BS code committees who are anxious to achieve European standard harmonization in relation to BS5268 and EC5. Indecisions over agreement concerning prEN 1194, European glulam strength classes, are proving an embarrassment.

Glulam bridge beams are possibly more common in one particular laminated hardwood, namely Iroko, than in any of the softwoods. This timber has found favour from its combination of good durability, the ability to be bent and glued, and its good joinery properties. Substitutes are being considered. Iroko is under pressure through perceived sustainability issues, and may even be coming into genuine shortage from some forests. Alternatives might include Dahoma (*Piptadeniastrum africanum*).

Where laminated softwoods are specified, European redwood rather than whitewood is preferred, due to its greater amenability to pressure preservative treatment. Douglas fir was hitherto more widely used.

Unlike the case in Germany, laminations for external structures are not restricted in Britain to a maximum of 33mm. The normally permitted maximum of 45mm for straight laminations is quite common in both softwoods and hardwoods. Permitted adhesive types are of course selected from the most rigorous Exterior/high hazard exposure category, and this normally indicates a phenol/resorcinol formaldehyde type. Provided the adhesive specification and manufacturing procedures are correct, including quality control tests in relation to the finger joints, there seems to be no reason to believe that 45mm thick laminations, including those from selected hardwoods, are unsuitable.

Mechanically laminated members have been dubbed "mechlam" as an abbreviation in this paper, although it should be clarified that this is not a universally recognised term. Recently encountered has been an interesting example of a mechanically laminated Greenheart bridge which was built in Britain in 1926 and which having remained in good condition has just been refurbished. However the modern manufacturing process which was developed in Germany and used quite extensively there and in the Netherlands, has become quite familiar in the UK. Numerous examples of bridges containing members of this type are to be found, ranging from simple short-span beam bridges to the more ambitious types such as arches and cable stayed structures. Formerly, the timber used was almost exclusively Ekki, or Azobe as it is known in Continental Europe. Recently, experiments and a few actual applications have occurred using oak. The design of mechlam involves some special considerations, which are discussed subsequently.

Decking

The commonest form of deck uses spaced sawn planks. These are usually laid transverse, but are sometimes placed longitudinally. The deck planks can be softwood or hardwood, with certain hardwoods preferred for maximum wear and durability. Softwood decking planks are specified as either GS or SS grade to BS4978. Suitable preservative treatment may be considered. Hardwood decking planks are usually from a naturally durable species, such as Iroko, Jarrah or Ekki, and are specified as HS grade to BS5756. Profiled hardwood decking planks provide a good solution where foot grip is important, and at higher gradients these are used in conjunction with kickplates which are nailed down to the deck.

It is generally felt that the gaps between decking should not be less than 5mm in order that dirt and debris can pass through the deck. This also allows air to circulate around the planks, thereby avoiding damp pockets where fungal decay can start. Larger gaps are sometimes used, and in remote country areas, deliberate gaps of up to 25mm have been specified. For certain bridges over roads and railways gaps in the walkway are not permitted, due to concern over vandals dropping objects onto vehicles or persons below. This has led some designers to use glulam beams which can provide the spanning medium for the bridge, as well as the deck. Such laminated decks are abutted together, to provide the walkway. An alternative solution has been to use plywood decking with additional non-slip surfaces, but this does not seem to have had a good record. Wear has been rapid and it has become evident that plywood decking requires special attention to drainage details.

Handrails/parapets

The primary function of the handrails and parapets is of course the protection of bridge users. Occasionally, in very remote areas such as forests and moorland trails, bridges are built with no parapet, or with only one handrail. A pair are however the norm. Various configurations are used, with the choice primarily depending on the following:

1. The type of footbridge user (for example - pedestrians only, or cyclists and pedestrians).

2. The nature of the site and locality, for example whether it is a rural or urban location, and whether it passes over a main road or a stream.

The first item dictates the height and strength requirements for the parapet, as discussed in more detail below. The second affects the degree of openness that is permitted for the handrail. Common solutions for bridges in rural locations use cantilevered handrails. A better solution is to arrange for the vertical post to be triangulated by projecting out the decking members in the vicinity of the post and adding a diagonal raking member. This solution is often adopted for short-span bridges where the main beam is of limited depth.

For suburban bridges, it is not uncommon for mesh infill to be fixed to the handrail, to prevent children climbing through the gaps. The mesh infill is typically 50mm grid galvanised or plastic coated galvanised chain link mesh. In urban areas, an altogether lesser degree of openness is often required. For bridges over roads and railways, concern over objects being dropped onto the highway or traffic has resulted in a number of bridges where the decking is located near the centre or towards the base of the main beams, which then provide the lower half of the parapet. However, this has other design implications, which are discussed elsewhere.

Both softwoods and hardwoods are used for parapets, with the latter, in a suitable species, preferred for durability and smoothness to touch. Softwood members will again be GS or SS grade to BS4978 and may have suitable preservative treatment. Hardwoods are usually from a naturally durable species, such as Iroko or Opepe, and are HS grade to BS5756.

Fixings

Fixings are normally of steel, and possible specifications include all of the mechanical fixings and connectors listed in BS5268: Part 2. EC5 will facilitate the design of steel dowels, which currently cannot be calculated using BS5268: Part 2. Steel fixings may be stainless, but more commonly, protection from corrosion is provided by means of galvanised surfaces. Where splice joints or similar require the use of steel plates, these are usually specified with a thickness of not less than 6mm, following steel bridge design practice.

DESIGN FOR DURABILITY

Detailing

Bridges are a particularly exacting application, and ensuring that the timber members have adequate durability is a vital consideration. Before considering this item from the perspective of material selection, it is important to note that much can be achieved in terms of increased durability by means of improved detailing. Indeed the converse is also unfortunately true, in that if poor detailing is provided, then even the most durable or well-preserved timber will eventually suffer from decay and insect attack. As mentioned in the introduction, survey work has been initiated as part of the new timber bridge initiative, and it is anticipated that this will need to be ongoing for some time.

Table 3 identifies six susceptible parts of a timber bridge in general. Most of these points apply to all types of bridge, irrespective of the precise form of the structure. The table then exemplifies poor detailing aspects and gives better alternatives. At this stage, the items in the table are regarded as pointers for guidance, and as suggestions for closer attention, rather than definitive solutions. It is anticipated that it will be necessary to pay considerable attention to detailing, and that these aspects will require discussion by timber experts and bridge manufacturers, in conjunction with the analysis of the survey results.

Table 3: Susceptible parts of the timber bridge structure with suggested detailing weaknesses and improvements

Part of the structure	Examples of poor detailing	Examples of better detailing
End grain of members in general eg: beams	Exposed end grain, leading to fissures, unattractive and ultimately a seat of decay	Protected end grain eg: by attaching other timber members having side grain, or by ventilated capping/sealing
Upper edges of exposed members eg: beams and handrails	Flat upper edges where water lies and which trap dirt, especially when weathered/ fissured	Chamfered and sloped upper edges which freely drain Edges protected by ventilated capping
Joinery details eg: handrails, parapet to beam connections	Details which trap moisture in mortises, fixing holes, recesses etc.	Freely draining, ventilated, flush details Raise parapet above splash level with separate drained kerb
Decking and its attachments	Deck which is tight jointed or with a sealed surface but which merely traps moisture Attachments to beams which form traps	Deck which freely drains, laterally and longitudinally, even when worne Drip mouldings beneath deckboards DPC between deck and beams
Member intersection points, column bases, especially with steelwork	Intersection points can easily form moisture/dirt entrapment regions Column bases, especially with steelwork very susceptible	Not easily avoided, but detail for maximum ventilation and drainage eg: by drilling/ arranging gaps
Bearing points, supports, bankseats etc.	Poorly ventilated, susceptible to silting up, dirt and debris entrapment	As well raised from surroundings, eg: by masonry and supporting steel, as possible

Natural durability

Even when the detailing is as good as possible, for an exacting, fully exposed application of timber such as a bridge, it is advisable to consider the use of a timber which falls into a natural durability category which is at least as good as "moderately durable", Table 4.

Such classifications are well-established in Britain for all of the better-known construction timbers, both softwoods and hardwoods, including all of those listed in BS5268: Part 2. Table 5 shows the natural durability classifications of the twelve tropical hardwoods listed in the code, together with European redwood and Douglas fir, for comparison. These classifications are based principally but not exclusively upon traditional ground contact stake tests. It is recognised that natural durability is a very variable property even within a single species. It should be noted that the ratings relate to UK conditions, which do not include a termite hazard, but which represent a high risk from fungal attack. Users of information of this type are reminded that the ratings only apply to the heart wood.

Recently, exposure trials have begun using prEN 330 "L-joint" type specimens, both to assess natural (untreated) durability, and to evaluate various forms of treatment. This is a collaborative BRE/TTL work programme, and an attempt is being made to include in the trials species likely to be in future commercial supply, including for example plantation teak from various sources. In due course, the information from this project will be of value to bridge designers, especially when they consider the joinery items such as parapets and handrails.

Table 4: Natural durability categories

Durability Category	Approximate life in ground contact, 50mm x 50mm section (years)
Very durable	More than 25
Durable	15 - 25
Moderately durable	10 - 15
Non-durable	5 - 10
Perishable	Less than 5

Preservation

In the modern philosophy of designing for durability, the use of chemicals to treat the timber, normally through pressure application, is regarded as the third line of defence, following good detailing and species selection. Increasingly, chemical treatment processes are under scrutiny from several angles. These range from vague and general concerns over the actual nature of the substances involved, to more specific anxieties to ensure that health and safety regulations are compiled with in respect of the treatment processes. This is an evolving public situation. However at any given point in time, it should be realized that only substances having government legislative approval are utilized by companies of the type and size likely to be involved in making timber bridges. Formal authorization procedures are in place to ensure that operations comply with legislation relating to aspects such as employee health and safety, material control and plant waste disposal.

The water-borne preservatives of the CCA type are still permitted in the UK, and are something used to treat softwood glulam bridge beams, usually in conjunction with European redwood as the timber. Both pre-treatment of individual laminations and treatment of the entire member after complete manufacture and machining are known, with each process having advantages and disadvantages. Pressure cylinders of up to 25m length are available. CCB and other alternatives are also known, and are slowly being introduced.

The efficiency of creosote as far as sheer timber protection is concerned is still recognised, and its specification is not out of the question, particularly for rural bridges or for bridge members which are inaccessible to touch.

Boron compounds clearly have a great appeal in relation to their mammalian non-toxicity. Full penetration by diffusion is not always practical for the types of timbers required for bridges, but boron in conjunction with vacuum pressure processes is becoming available. The use of boron glass rods as a prophylactic or as remedial protection in critical areas of the bridge, Table 3, is also considered.

Table 5: Natural durability classifications of the twelve tropical hardwoods listed in BS5268: Part 2, and of Douglas fir and European redwood

Timber species	Origin	Natural durability
Balau dense *Shorea* spp	SE Asia	Durable
Ekki *Lophira alata*	W Africa	Very durable
Greenheart *Ocotea rodiaei*	Guyana	Very durable
Iroko *Milicia excelsa*	W Africa	Very durable
Jarrah *Eucalyptus marginata*	Australia	Very durable
Kapur *Dryobalanops* spp	SE Asia	Very durable
Karri *Eucalyptus disersicolor*	Australia	Durable
Kempas *Koompasia malaccensis*	SE Asia	Durable
Keruing *Dipterocarpus* spp	SE Asia	Moderately durable
Merbau *Intsia* spp	SE Asia	Durable
Opepe *Nauclea diderrichii*	W Africa	Very durable
Teak *Tectona grandis*	SE Asia	Very durable
Douglas fir *Pseudotsuga menziesii*	N America	Moderately durable
Scots pine/European redwood *Pinus sylvestris*	Europe	Non-durable

Finishes

The benefits of effective and well maintained finishes have been very apparent in the survey work which has already been performed and which continues. Modern microporous water repellant finishes offer a considerable measure of protection to exterior timber structures such as bridges. The prevention of weathering of the timber surface itself has an important role in this respect. A high specification of finish and a good maintenance programme for the same would always be advocated in addition to the correct detailing and choice of durable species mentioned above.

DESIGN PRACTICE

General practice for design of bridges in the UK

Bridges constructed from steel or concrete, whether highway or footbridges, are generally designed using the BS5400 series of Standards. This series comprises ten parts, as follows:

Part 1 - General statement giving design objectives and definitions
Part 2 - Specification for loads
Part 3-5 - Codes of practice for design of steel, concrete and composite bridges
Part 6-8 - Specifications for materials and workmanship for steel, concrete and composite bridges
Part 9 - Specification for bridge bearings
Part 10 - Code of practice for fatigue.

The partial factor design process for a bridge will primarily involve only two of the above parts. Firstly the loads, and the partial safety factors for the loads, are obtained from Part 2. Secondly the design (including partial safety factors for materials) is undertaken in accordance with Part 3,4 or 5 depending on which construction material is being used.

In the case of road bridges or bridges over roads, the Department of Transport (DoT) has produced a number of Department Standards which occasionally override the requirements of the BS5400 Series. One such Department Standard BD29/87 gives directions to engineers on how to design timber footbridges.

Outline procedure for the design of timber footbridges over roads or in urban areas

The procedure may be considered in three stages, as follows:

1. Establish the general arrangements for the bridge, taking note of the requirements for layout and minimum dimensions given in DoT Departmental Standard BD29/87.

2. Evaluate the loads acting on the bridge, using the unfactored loads of BS5400: Part 2, unless these are made more onerous by a DoT Department Standard.

3. Design the members of the bridge in accordance with BS5268: Part 2 which is a permissible stress code, used principally for timber members in buildings. BS5268 does however contain sufficient basic materials properties, fastener design information and member design procedures for simpler types of timber bridge, as explained above in the section dealing with materials.

Outline procedure for design of timber footbridges in suburban or rural areas

Many engineers consider the BS5400/DoT provisions for minimum dimensions and loading too severe for lightly trafficked footbridges in suburban and rural areas. For such footbridges, typical alternative procedures are exemplified as follows:

1. Establish the general arrangements for the bridge taking note of the minimum dimensional recommendations given in publications such as "Footbridges in the Countryside, Design and Construction"(11).

2. Evaluate the loads acting on the bridge using the unfactored loads of BS5400: Part 2, or consider making them less onerous on the basic of recommendations given in publications such as the above.

3. Design the bridge members in accordance with BS5268: Part 2.

Examples of how the "Countryside Commission for Scotland" publication recommends less onerous minimum dimensions and loadings are given in the two tables below:

Table 6: Recommended deck widths (mm) for alternative bridge locations

Source of data	DoT Standard BD29/87	"Countryside Commission for Scotland" publication	
Location of bridge	**Urban area**	**Accessible rural area (two-way traffic)**	**Inaccessible rural area (one-way traffic)**
Pedestrians only	1800	1200	900
People with disabilities	2000	1700	1200

Table 7: Recommended imposed loadings for alternative bridge locations

Source of data	BS5400: Part 2	"Countryside Commission for Scotland" publication
Location of bridge	**Urban Area**	**Rural Area**
Vertical imposed uniformly distributed load on bridge deck (kN/m^2)	5.0	2.3 - 3.2
Horizontal load per metre (kN/m) perpendicular to handrail	1.4	0.74 - 1.4

Department of Transport criteria for layout and dimensions of footbridges

1. Layout of Footbridge

The DoT Standard BD29/87 stipulates several criteria relating to the layout of footbridges, some of which are quite fundamental to the bridge design. These may be summarised as follows:

Access: Access to footbridges located adjacent to carriageways should be sited so that pedestrians walking down the access face on-coming traffic. Plain ramped access is preferred to stairs as it is more satisfactory for people in wheelchairs and pedestrians pushing prams. However wherever possible both forms of access should be provided.

Layout: The main span of a footbridge should, wherever possible, be at right angles to the road carriageway.

Supports: Where a footbridge crosses a dual carriageway, preference should be given to spanning both carriageways in a single span, to avoid the need for a support in the central reserve. Supports which may be subject to collisions by errant vehicles shall be designed to resist collision loading.

2. Dimensions for footbridges

Width of bridges for pedestrians only:

The dimensions of the clear widths of the main span, ramps and stairs of a footbridge whould be derived on the basis of the information in Table 8 to meet the peak pedestrian traffic.

Table 8: Recommended clear widths (mm) for alternative bridge gradients

Gradient	Clear width (mm)
< 1/20	300mm per 20 persons per minute
Steps or > 1/20	300mm per 14 persons per minute

Minimum widths of 1800mm for general purposes, or 2000mm for bridges enhanced for use by disabled persons are also stipulated. Where bridges are to be designed for the combined use of pedestrians and cyclists, further width requirements apply. These range from a 1200mm wide footpath separated by a white line from a 1500mm wide cycle track, to a 1950mm wide lane for each, separated by railings. The DoT criteria for heights of parapets vary according to the types and combinations of bridge user. They are summarised in Table 9.

Table 9: Height requirements for bridge parapets

Type of footbridge	Parapet height (m)
Pedestrians only	1.15
Pedestrians only, where bridge is in area of high prevailing winds or with headroom under bridge greater than 10m	1.15 - 1.30
Pedestrians and cyclists	1.4
Pedestrians and horseriders	1.8

Stairway requirements may be summarised as follows:

Maximum number of stairs in a flight is 20
Riser dimension < 150mm
Tread dimension > 300mm.

It is a preference that ramps should not be steeper than 1 in 20. However if limitations of space dictate then steeper ramps may be used, up to a maximum gradient of 1 in 12. For ramps with gradients greater than 1 in 20, landings must be provided in order that the rise of any ramp section does not exceed 3.5m

Evaluation of loads using BS5400: Part 2

As mentioned above, timber footbridges are designed using the unfactored nominal loads of BS5400: Part 2. Interestingly, the use of unfactored nominal loads in structural design is not only limited to timber, with the design of foundations and that for aluminium structures also being based on unfactored loads. Where adequate statistical distributions are available, the nominal loads in BS5400: Part 2 are those appertaining to a return period of 120 years. The following types of nominal load relating to footbridges are considered by BS5400: Part 2

1. Permanent loads

Dead-weight of structural elements
Superimposed dead - road surfacing, etc.

2. Live loads from pedestrian traffic

Nominal vertical live load
Nominal load on pedestrian parapets

3. Wind loads

Transverse
Longitudinal
Vertical

4. Loads from temperature effects

5. Erection loads

BS5400: Part 2 suggests that in most cases snow loads can be ignored. This is logical since the full pedestrian design load is improbable under heavy snow falls of the duration likely to be experienced in the UK.

The maximum wind gust speed is evaluated by applying gust factors to mean hourly wind speeds extracted from a map of isotachs. The magnitude of the gust factor depends on the height of the bridge, and the horizontal wind loaded length. For footbridges only, BS5400: Part 2 allows the following reductions in wind load:

1. The mean hourly wind speed is reduced by 0.94, which is a conversion factor to obtain 50-year return period values from 120-year return period values.

2. A reduction in the gust factor when the bridge is located in urban areas or a rural environment with many windbreaks.

The use of BS5268: Part 2 for timber bridge design

1. Service conditions for bridges

To use BS5268: Part 2 for the design of bridge members, the designer has to decide upon the service conditions for the bridge. This mainly involves deciding the exposure and duration clauses that are appropriate for the member concerned. Experience has shown that designers usually err on the conservative side by choosing to design members using wet exposure stresses. With BS5268, the threshold moisture content between wet and dry exposure conditions is 18%. Hence this is often unnecessarily conservative. For a vertical imposed uniformly distributed loading which represents a crowded bridge, experience has shown that designers usually select medium-term duration. This is the duration class used with BS5268 for snow loading in the UK. Horizontal loads on handrails or parapets are usually designated as short-term. This is the same load duration class as that arising from the case of a man standing on a roof member.

2. Deflection limits

The limited guidance given in BS5268: Part 2 regarding deflection limit is of little relevance to the design of bridge members. Enquiries indicate that a static deflection limit of Span/200 under imposed load only is often used for beam members. The "Countryside Commission for Scotland" publication recommends a tighter limit of Span/240 under total loading. Lightly precambered glulam bridge beams are often designed using a deflection limit for live load only, which is permitted in principle by BS5268.

3. Mechanically laminated beams

As mentioned in the above section dealing with materials, the use of mechanically laminated bridge beams is increasing significantly. The absence of recommendations in BS5268: Part 2 relating to the design of dowels results in the unsatisfactory situation of two codes being used for the design of these beams. The member stresses are often checked in accordance with BS5268: Part 2, whilst the dowel loads are checked to Continental codes such as DIN 1052. This situation will change not only with the introduction of EC5, but also with a forthcoming revision of BS5268.

Eurocode 5

As explained above, it has proven possible to design acceptable timber footbridges using BS5268: Part 2 recommendations, supplemented by additional guidance from elsewhere. The eventual publication of EC5, Part 2 will be a great step forward and will be welcomed by everybody involved. Meanwhile, EC5, Part 1 is shortly to be available. Thus even at this stage, the Eurocodes will bring advantages to the more sophisticated aspects of timber engineering such as bridges.

EC5 introduces limit states design to timber for the first time in the UK. It is therefore a more radical change for timber than the introduction of Eurocodes for other major structural materials. EC5 contains the essential rules for design, but unlike the British Code, BS5268, it does not include material properties, tables of fastener loads and other such other design information. An immediately obvious change is that wherever possible, EC5 uses equations rather than tables. Also much of the nomenclature and terminology is considerable different.

As in all instances of limit states design codes, EC5 will require clear thinking about the distinction between ultimate and serviceability limit states. The latter are, of course associated with deflections, deformations and vibration. EC5 treats these matters in a considerably more sophisticated manner than their coverage in BS5268. As has already been pointed out, BS5268 gives no adequate guidance related to deflection limits for bridges. Furthermore it is likely that the Working Group dealing with EC5, Part 2 will require to give considerable thought to the serviceability topics.

EC5 offers a number of advantages over BS5268. It provides the opportunity to design with a wide selection of materials and components. The use of characteristic values for materials, based directly on test results, means that new materials and components, which have achieved suitable technical approval can more easily be assimilated, thus facilitating development and innovation. More guidance is given on the design of built-up components than in BS5268, and EC5 provides a unified design and safety basis for laterally loaded dowel-type joints (nails, staples, screws, bolts and steel dowels). The ENV contains no information on the design of joints using connectors such as toothed plates, shear plates and split rings, but a procedure is being developed through other sponsored research programmes. Interim guidance on the design of such joints is contained in the UK NAD.

PRIORITY WORK AREAS

Since this paper has set out the state of the art of timber bridge design in the UK, with wider references to Europe in general, it is not appropriate to give conclusions. Instead, it is possible to identify the priority work areas in which it is felt that effort should be concentrated. This is given firstly from a UK point of view, and secondly in even more general terms, from a European perspective.

Design and construction of timber footbridges

The above is the title of the TTL and UK industry project, which is part-sponsored by the DoE. The projected programme of work for this may be summarized as follows:

1. Review serviceability criteria for footbridges in all materials.

2. Continue bridge survey work, paying special attention to serviceability performance and durability aspects.

3. Carry out design studies leading to the construction of prototypes which are to be used for serviceability assessments and durability monitoring.

4. Produce nationally applicable design guidance.

Within item 3 above, the initial design studies have already led to the conclusion that there are four areas where supplementary design guidance is required. These are as follows:

1. The design and detailing of bracing systems to ensure member stability and to resist horizontal wind loads

2. Methods to ensure satisfactory vibrational performance of bridges under human footfalls

3. Simple methods to ensure that the excitation of bridges by wind is avoided.

4. In the case of mechanically laminated beams, a simplified method of evaluating deflections is required.

Eurocode 5, Part 2

The Project Team responsible for the drafting has only met once, at the time of writing, but it is interesting to note that several of the priority areas coincide with those felt to be necessary in the UK. The following have been suggested:

1. The dynamic behaviour of bridges under pedestrian and wind loadings.

2. The fatigue behaviour of connectors.

3. Timber-concrete composite behaviour.

4. Glued-in steel bars.

5. Durability requirements for timber birdges, including those for the connectors.

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(3) Steel, concrete and composite bridges, BS5400 series, parts as follows:

Part 1	- General statement giving design objectives and definitions
Part 2	- Specification for loads
Part 3-5	- Codes of practice for design of steel, concrete and composite bridges
Part 6-8	- Specification for materials and workmanship for steel, concrete and composite bridges
Part 9	- Specification for bridge bearings
Part 10	- Code of practice for fatigue

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

INFLUENCE OF WEAK ZONES ON STRESS DISTRIBUTION IN GLULAM BEAMS

by

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MEETING TWENTY - SEVEN

SYDNEY

AUSTRALIA

JULY 1994

Influence of weak zones on stress distribution in glulam beams

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1 Background

When analyzing a beam with non-homogenous cross-section, a commonly made assumption is that plane sections normal to the beam axis remain plane and normal when the beam is loaded. This assumption according to traditional beam-theory leads to the well known fact of piecewise linear stressdistribution over the crossection of a glulam beam of lamellas with different modulus of elasticity. A zone with lower stiffness would, according to these assumptions, be subjected to stresses corresponding to the reduction of stiffness. It is often claimed that a "weak" zone (i.e a knot or fingerjoint) with lower stiffness and strength than adjacent material would be subjected to stresses of less magnitude and therefore not have such severe effect on global beam strength. The present analysis shows that a low-stiffness zone not necessarily is relaxed in the above described way.

2 Present analysis

2.1 General remarks

The loadcase analysed is that of a glulam beam subjected to pure bending. The linear elastic analysis is performed with plane, 4-noded, membrane finite elements. At boundaries where bending moment is applied, plane sections of the beam are assumed to remain plane during deformation. The beam height is 315 mm (7 lamellas, each 45 mm thick) its length 600 mm and its width 100 mm. In the outer tension lamella a zone with lower stiffness than surrounding material is assumed. The weak zone is of 45 mm height and its length is varied in the analyses. The finite element mesh used in the analyses is shown in figure 1. In the weak zone all stiffness parameters are reduced with the same percentage. The surrounding material properties are:

E_x=12 GPa
E_y=0.4 GPa
G_{xy}=0.6 GPa
ν_{xy}=0.53

and thus an orthotropic material is assumed.

2.2 Calculations

Two different types of analyses are performed. In the first series of analysis the length of the weak zone is varied from being of the same length as the beam (600 mm) down to the length of two finite elements in the fine meshed area (7.5 mm), se figure 1. In these cases the weak zone is assumed to have a reduction of stiffness of 25 percent. The second series of analysis is made to investigate the influence of different magnitude of the stiffness reduction. The weak zone of length 30 mm is given a stiffness reduction of 25, 50, 75 and 100 percent respectively. In both types of analysis the height of the low stiffness zone is the same as the lamination thickness, i.e 45 mm.

2.3 Results

The results of the finite element analyses are given in figures 2–5. **Figure 2** shows the influence of the length of the weak zone on the stress distribution. As expected when the weak zone is as long as the beam, the stress distribution is indeed piecewise linear. As can be seen in the figure a reduction of the extension of the weak zone results in a redistribution of axial stresses. In the limit, as the length of the weak zone approaches zero, the stress distribution approaches the linear one expected in a homogenous cross section. In **figure 3** the stress distribution in the midsection of the beam is shown for a length of the weak zone of 30 mm. As can be seen the reduction of stresses in the weak zone is very local. In **figure 4** the influence on the axial tension *force* in the weak zone is given. As can be seen in the case of a 30 mm long weak zone the axial force is only reduced about 3 percent when stiffness is reduced 25 percent. In **figure 5** the influence of stiffness reduction on the stress distribution in the midsection is shown. The four curves represent a reduction of stiffness of 25, 50, 75 and 100 percent respectively. As expected for a 100 percent reduction of stiffness the stresses in the weak zone are equal to zero, as the weak zone now represents a hole or a notch.

2.4 Conclusions

The analyses suggests that the simple assumption that a local and proportional reduction of stiffness and strength has minor influence on beam strength is *not* valid for small zones, i.e knots and fingerjoints. The stress reduction in a small zone is far from proportional to the stiffness reduction and therefore the stress is closer to the strength of the material in a small weak zone, than one would expect by intuition.

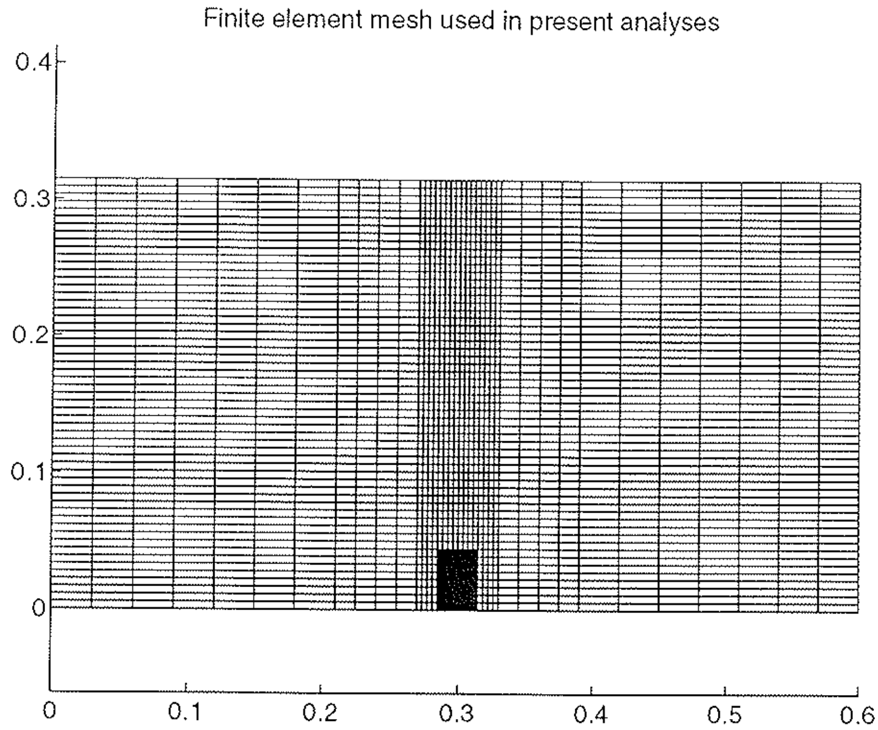

Figure 1: Finite element mesh used in present analysis. Dark area is the weak zone, here with length 30 mm.

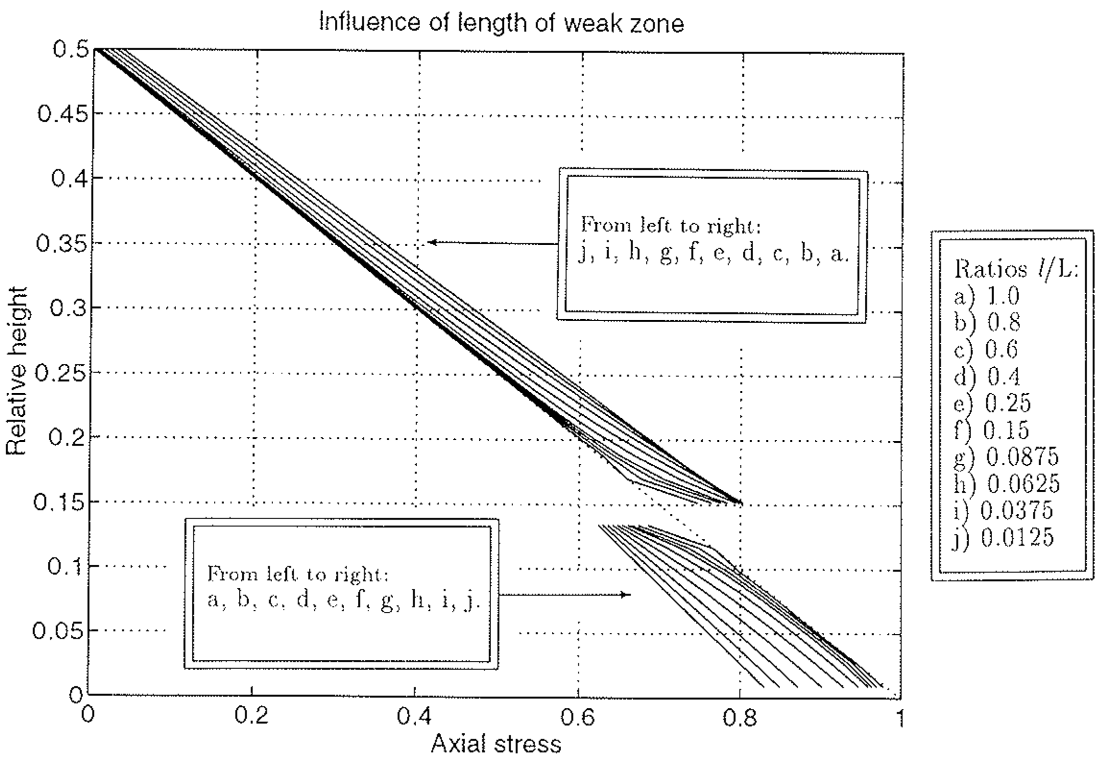

Figure 2: Influence of the length of the weak zone on the stress distribution in midsection. Stiffness reduction 25 percent. Curves *a-j* represent ratios (l/L): 1.0, 0.8, 0.6, 0.4, 0.25, 0.15, 0.0875 , 0.0625, 0.0375, 0.0125 respectively.

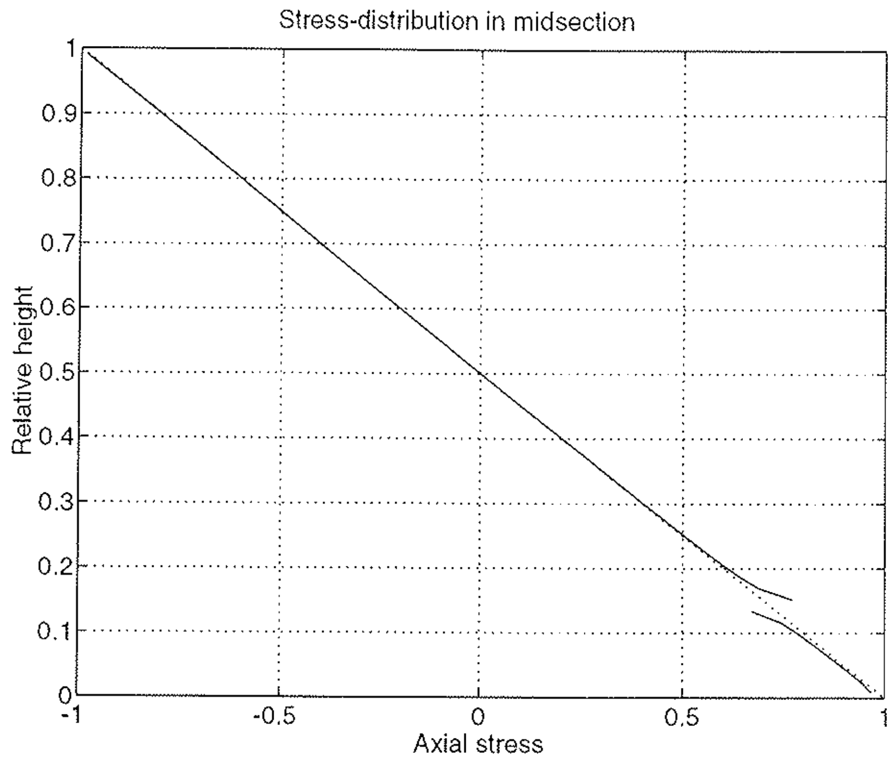

Figure 3: Stress distribution in the midsection of the beam for a length of the weak zone of 30 mm. Stiffness reduction 25 percent.

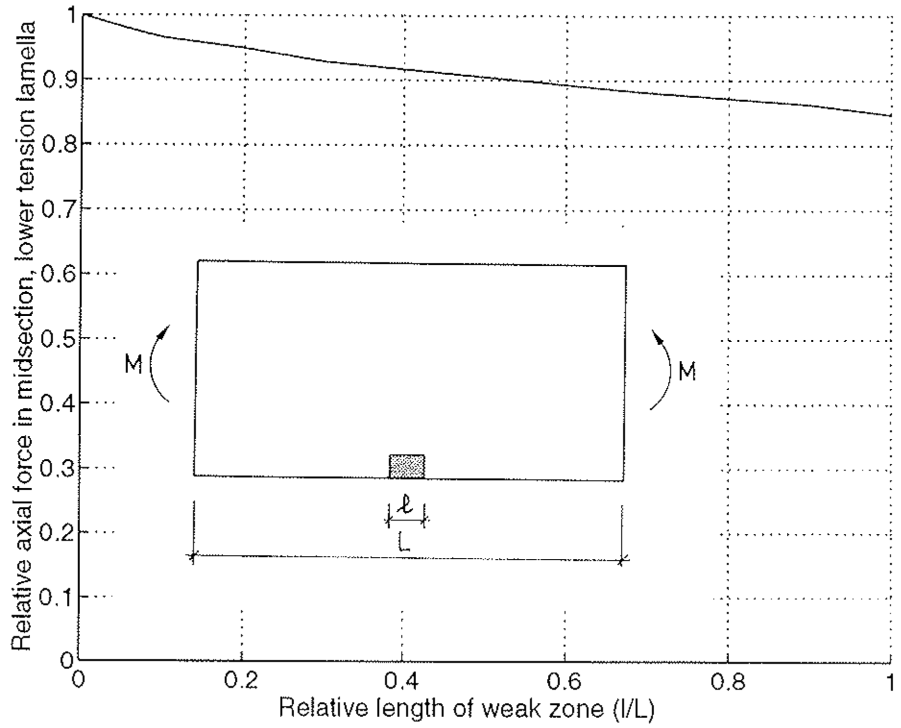

Figure 4: Influence of the length of the weak zone on the axial tension force in the lower tension lamella. Stiffness reduction 25 percent.

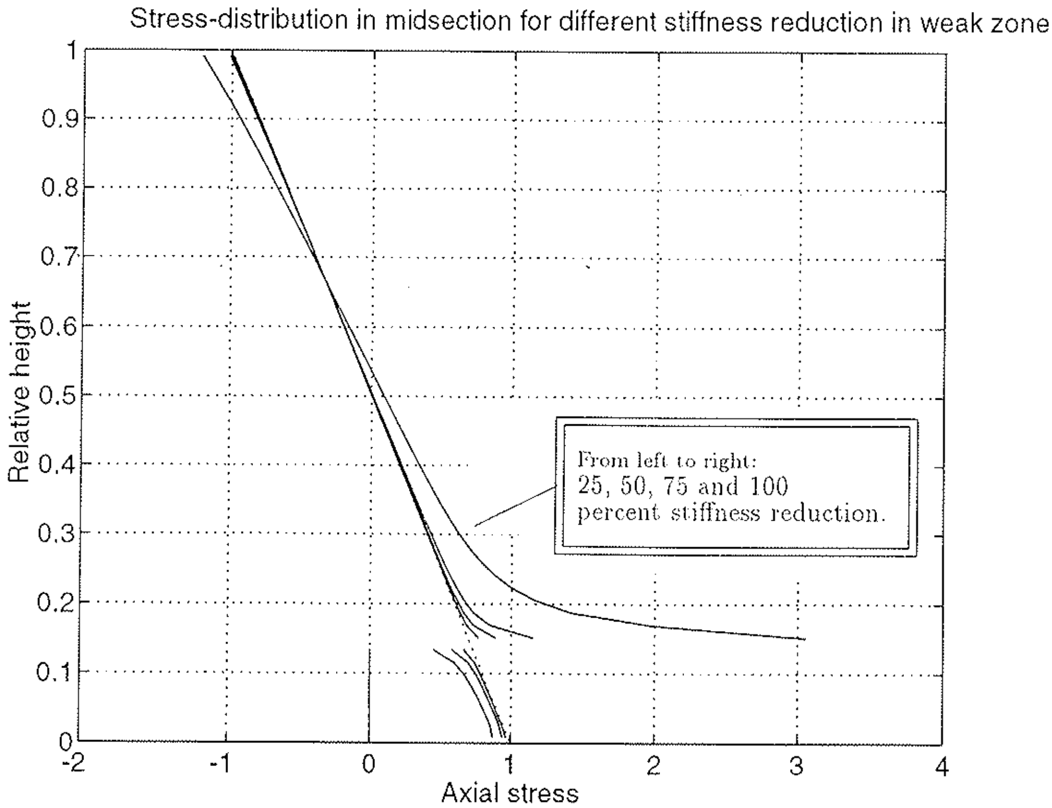

Figure 5: Influence of the magnitude of stiffness reduction on the stress distribution in the midsection. Four solid curves represent 25, 50, 75 and 100 percent stiffness reduction respectively.

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

STATISTICAL CONTROL OF TIMBER STRENGTH

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JULY 1994

STATISTICAL CONTROL OF TIMBER STRENGTH

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1. INTRODUCTION

In comparison with most other structural materials, the strength of stress-graded timber is highly variable. Because of this, timber strength is defined in statistical terms. Typically, it is defined as the five-percentile value of a stress-graded population.

In order to ensure reasonable confidence in obtaining the specified strength of this highly variable material, frequent strength measurements are made as part of quality control systems. In drafting Australian Standards, the following types of quality control measurements are under consideration:

- *in-grade evaluation:* a complete evaluation of all significant structural properties for all grades and sizes;
- *annual check:* an in-grade evaluation of a limited set of properties to ensure that there has not been a drift in the quality of the output of a mill;
- *daily check:* a special structural test on a small sample to ensure that the stress-grading process is in control;
- *random check:* a limited set of in-grade tests by a customer who may suspect that the quality of timber is below the quality claimed by the producer.

The relative frequencies of these four types of measurements are illustrated in Figure 1. In the above, the terminology 'annual check' and 'daily check' are used as descriptors to provide a rough guide as to the frequency requirements of this type of check. The actual frequency of checks of any type should be based on sampling at a rate per m^3 of material processed as specified by a quality control scheme.

The criteria for quality control tests should be simple. The use of complex rules only serves to obscure the impact of the rules; furthermore, the implied accuracy is not justified because of the lack of adequate knowledge of the characteristics of the tails of strength distributions.

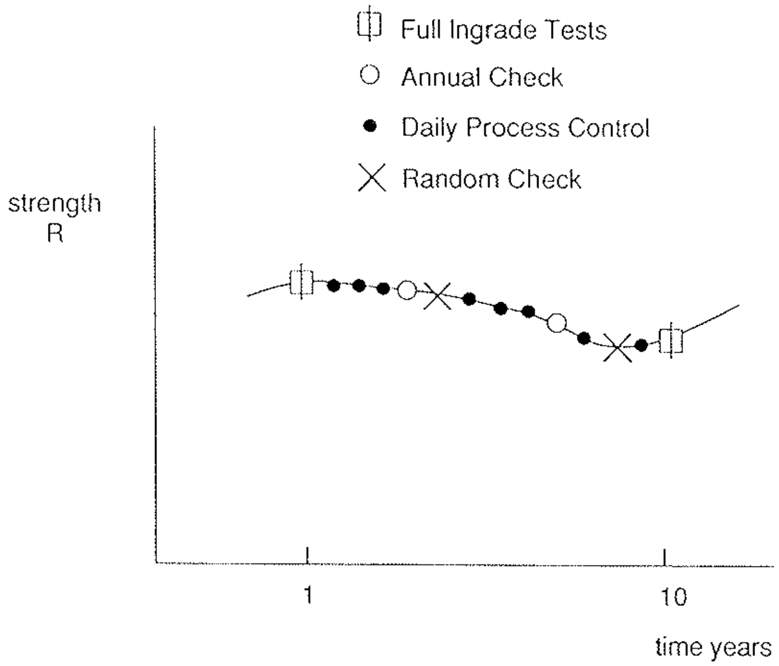

Figure 1. Schematic illustration of frequency of quality control checks.

A more effective method of assessment of a quality control criterion is to test it with Monte Carlo simulation; the statistical distributions used in these simulations should cover the range of distributions found in structural timber. For practical purposes it is reasonable to use a range of distributions bracketed by a Weibull and log normal distribution, and a coefficient of variation between 0.1 and 0.5 as shown in Figure 2.

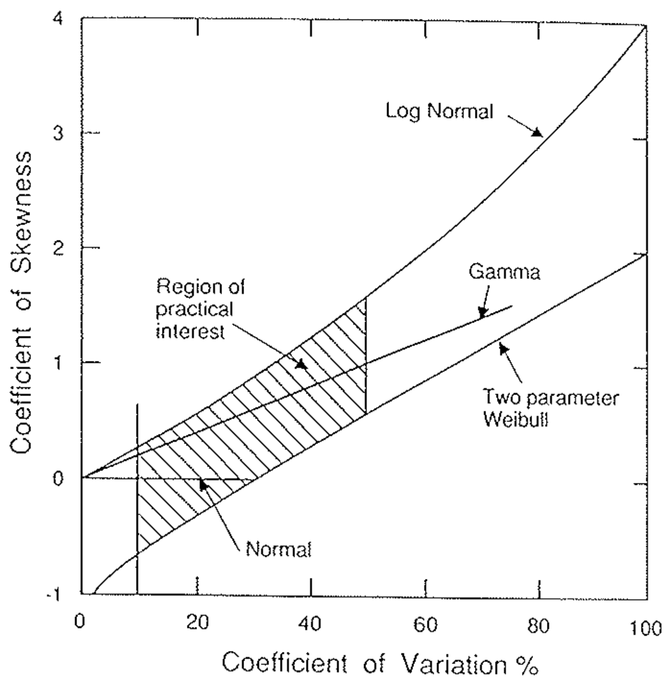

Figure 2. Range of practical distributions for structural timber.

The target level of quality may be considered to be an arbitrary quantity on which a producer and customer agree. Hence, for all quality control criteria, an assessment should be made of the value to both the producer and customer.

2. IN-GRADE EVALUATION

2.1 Quality Criterion

In order to maintain a reasonable level of reliability in specified design values, the five-percentile value of strength should be estimated with (at least) a 75% confidence, i.e. there should be only a 25% chance that the procedures used will provide an overestimate of the 5-percentile value (Leicester 1986).

In the Australian In-grade Standard AS/NZ: 4063 (1992), the required level of confidence is obtained by choosing a characteristic value R_k derived from testing a sample N as follows,

$$R_k = R_{sample,0.05}\left[1 - \left(2.7\, V_{sample}/\sqrt{N}\right)\right] \qquad (1)$$

where V_{sample} and $R_{sample,0.05}$ denote the coefficient of variation and the five-percentile value of the test sample respectively.

2.2 Value to Producer

For the producer, the value of his timber is given by the quantity ϕ_{ig} defined by

$$\phi_{ig} = R_{k,mean}/R_{parent,0.05} \qquad (2)$$

where $R_{k,mean}$ denotes the mean value of R_k and $R_{parent,0.05}$ denotes the five-percentile value of the parent population.

If measurements of R_k were to be made with an extremely large sample, then a value of $\phi_{ig} = 1.0$ would be obtained. Hence, any value of ϕ_{ig} less than 1.0 represents an average loss of potential benefit to producers. The effect of sample size on ϕ_{ig} is shown in Figure 3.

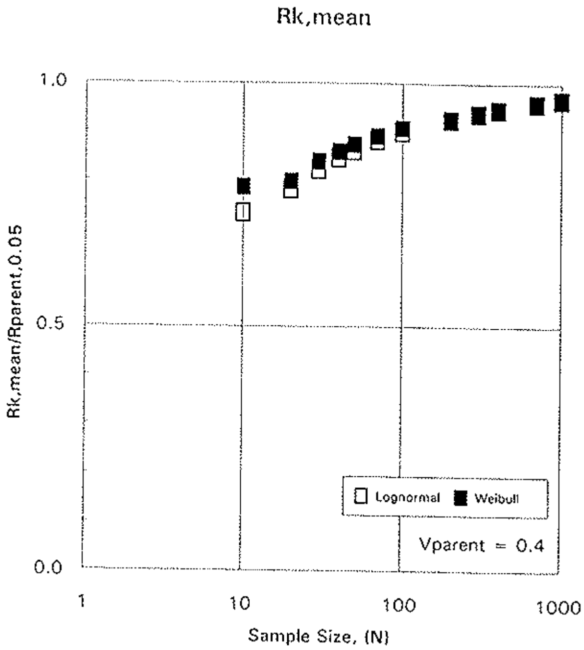

Figure 3. Effect of sample size on the value of in-grade evaluation to producers.

2.3 Value to Customer

Because the customer requires a 75% confidence level in the strength of his timber, the value of the quality criterion given by equation (1) can be stated as a quantity λ_{ig} defined as follows

$$\lambda_{ig} = R_{k,0.75}/R_{parent,0.05} \tag{3}$$

where $R_{k,0.75}$ denotes the 75-percentile value of R_k. Obviously any value of λ_{ig} less than the value of 1.0 represents a bonus in reliability to the customer. The effect of sample size N on the value of λ_{ig} is shown in Figure 4.

2.4 Frequency of Check

There is insufficient experience and data for specifying the desirable frequency of a full in-grade evaluation of every commercial size, grade and property. This frequency should be related to production rates. For a detailed in-grade evaluation of a specific species would be about once for the production of 10–50 million m^3; and for a specific mill it would be about once every 1–5 million m^3.

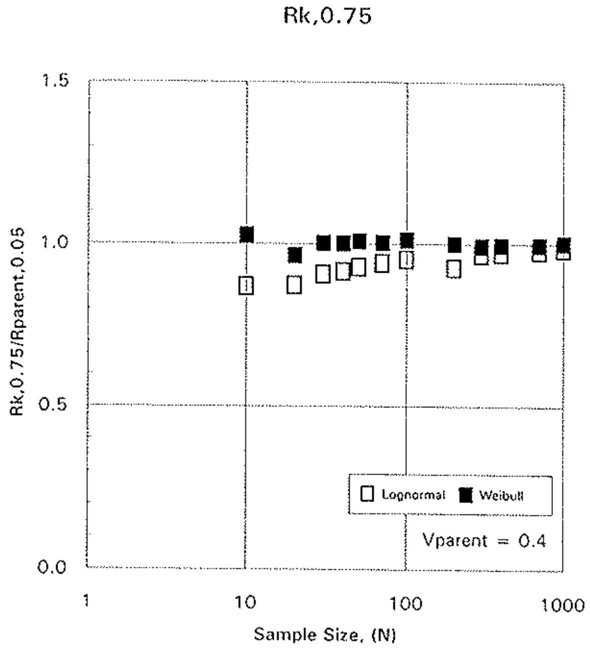

Figure 4. Effect of sample size on the value of in-grade evaluation to the customer.

3. ANNUAL CHECK FOR SINGLE MILL

3.1 Quality Criterion

The quality criterion is based on measuring the five-percentile value $R_{sample,0.05}$ of a sample of size N given by

$$N = 1000\ V_{parent}^2 \tag{4}$$

where V_{parent} denotes the coefficient of variation of the parent population. The quality criterion is

$$R_{sample,0.05} > 0.90\ R_{parent,0.05} \tag{5}$$

If the sample fails the test, then a second sample may be tested. If both samples fail, then the parent population is deemed to be inadequate.

3.2 Value to Producer

As indicated in Table 1, there is a chance of up to 4% that the quality test will fail if the timber is just adequate.

TABLE 1.
ANNUAL CHECK ON A SINGLE MILL

Description of parent population		Producer risk	Customer risk
Distribution type	Coefficient of variation	Probability of failing the test if the material is just adequate	There is a 10% chance of passing the test if the relative strength is reduced to the following value
lognormal	0.2	0.002	0.83
	0.4	0.002	0.83
Weibull	0.2	0.028	0.78
	0.4	0.039	0.77

In reality, the risk to the producer is somewhat less than this. As indicated in Figure 2, the producer is usually operating at less than 100% efficiency; this means that the target 5-percentile is usually less than $R_{parent,0.05}$. For example, if R_k turns out to be 5% less than $R_{parent,0.05}$, then it is found that the risk to the producers reduces from 4% to 1%.

3.3 Value to Customer

From Table 1 it is seen that there is a 10% risk to the customer that if the timber passes the quality criterion, it could be up to 23% under strength.

3.4 Frequency of Check

Recommended sampling rates for test pieces range from a minimum of 1 in 50 000 to a rate of 1 in 5 000 where tight control is required.

4. ANNUAL CHECK FOR A SET OF TEN MILLS

4.1 Quality Criterion

It will be assumed that the 10 mills sell to a common market and are pooling their quality control data. Hence, the sample size for the quality check will now be taken to be given by

$$N = 10\,000\ V_{parent}^2 \qquad (6)$$

and the check criterion for each sample will be

$$R_{sample,0.05} > 0.97\, R_{parent,0.05} \quad (7)$$

Again, the failure of two consecutive samples comprises a failure of the quality check.

4.2 Value to Producer

As can be seen from Table 2, the risk to the producer is about the same as that of the single mill.

TABLE 2.
ANNUAL CHECK FOR A SET OF 10 MILLS

Description of parent population		Producer risk	Customer risk
Distribution type	Coefficient of variation	Probability of failing the test if the material is just adequate	There is a 10% chance of passing the test if the relative strength is reduced to the following value
log normal	0.2	0.005	0.95
	0.4	0.005	0.95
Weibull	0.2	0.050	0.93
	0.4	0.050	0.92

4.3 Value to Customer

From Table 2 it is seen that there is a 10% risk to the customer that if the timber passes the quality criterion, it could be up to 8% under strength. Hence, pooling the data from mills provides more reliability to the customer.

5. DAILY CHECK

5.1 Quality Criterion

The most commonly used process control check is the cumulative sum (CUSUM) procedure, first publicised by Warren (1978) and used in many machine stress-grading standards such as the Canadian NLGA Standard SPS 2 (1987).

There are two commonly used CUSUM charting procedures; one termed the 'attribute chart' procedure is based on measuring d_i, the number of times a required attribute is missing; specifically the attribute will be taken to be the attainment of a strength in excess of a proof load; the other procedure is termed a 'variables chart procedure' and is based on measuring M_i, a mean value; specifically this will be taken to be the mean value of strength. The usual sample size used for these checks is $N = 5$.

The charting procedure is based on the use of three control parameters K, Y and Z, where $Y < Z$; values of these parameters are given in Tables A1 and A2 of Appendix A. For the i-th sample, a SUM is computed as follows,

$$SUM_i = CUSUM_{i-1} + (d_i - K) \qquad (8)$$

for an attributes chart and

$$SUM_i = CUSUM_{i-1} + (K - M_i) \qquad (9)$$

for a variables chart. Here $CUSUM_{i-1}$ denotes the value of CUSUM after the previous sample was tested. The value of SUM_i is combined with the previous CUSUM value, i.e. $CUSUM_{i-1}$, to obtain a new CUSUM value, i.e. $CUSUM_i$, as indicated in Table 3. If the value of $CUSUM_i$ is greater than Y, then the quality control process is deemed to be 'out-of-control'; otherwise it is deemed to be 'in control'.

In general, a single 'out-of-control' signal means that some check must be made on the stress grading process; if no processing errors are detected, then 'intensive' sampling of six more samples are taken. If the process does not return to the 'in-control' mode during 'intensive sampling', then production is halted. This procedure is illustrated in Figure 5, together with the definition of a 'clear run' (the number of samples between out-of-control signals) and a 'production run' (the number of samples between production halts).

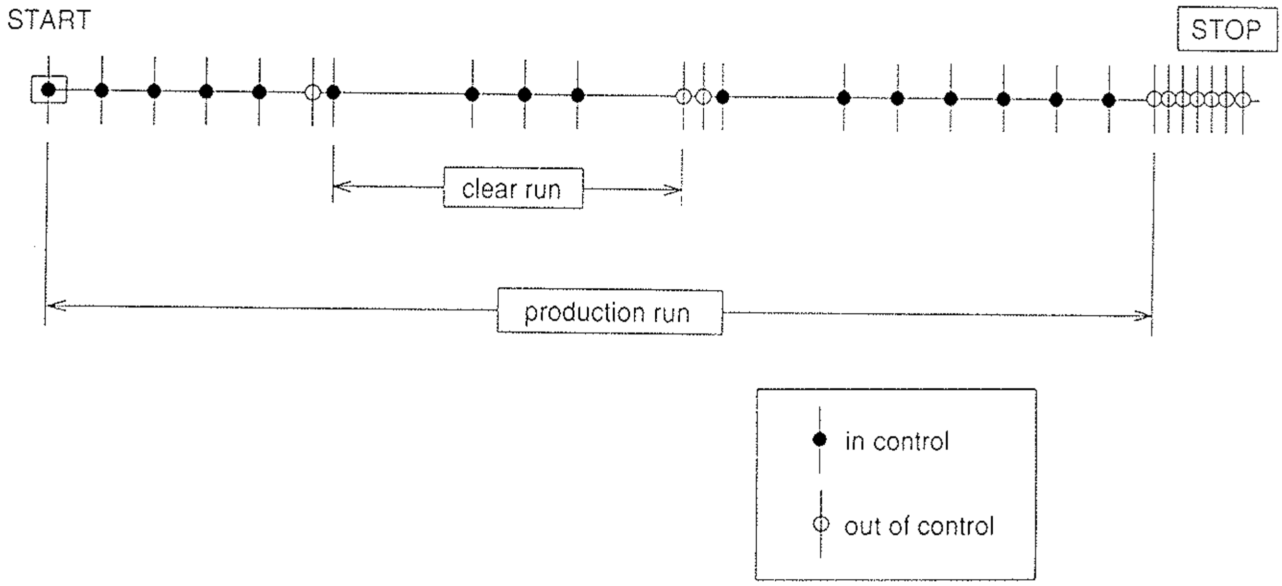

Figure 5. Nomenclature for CUSUM control procedure.

5.2 Value to Producer and Customer

To illustrate the impact of the charting procedures on the producer and customer, two sets of Monte Carlo simulations were undertaken. It was assumed that timber strength follows a Weibull distribution. The results are given in Tables A4 and A5 of the Appendix and an extract for the case N = 5 is shown in Table 3.

In the first set, an attributes chart procedure, it was assumed that the strength of the timber is a Weibull distribution with a coefficient of variation of 45%. The proof load used was the 3-percentile value, i.e. 10.28 MPa. In the first run of data, the simulated parent population was chosen correctly; in the second run of data the strength of the parent population was reduced by 20%, i.e. the mean value was 32 MPa, but the proof load was maintained at 10.28 MPa.

From Table 3 it is seen that if the timber is just adequate, i.e. it has a mean value of 40 MPa, then a halt in production will be rare, i.e. once every 511 samples, or once every 511 working days if one sample is taken per working day. This is obviously quite satisfactory from the producer's view point. In fact, the time between production halts will be rather longer than this theoretical value because of underestimates of the characteristic value as mentioned in Section 2 of this paper. However, for the customer the situation is not so satisfactory. From Table 3 it is seen that if the strength drops by 20%, there will be a production halt on average only once every 99 samples, i.e. once every 99 working days if one sample is taken per working day.

In the second set of data a variables chart procedure was used to control the mean value of strength. Again, a Weibull distribution was used for the strength distribution but now it was assumed that the coefficient of variation was only 20%. From Table 3 it is seen that this now produces a very satisfactory control; there is a long production run if the timber is satisfactory and a very short production run of only 2.5 samples (on average) if the strength drops by 20%.

To make use of the above information, it is proposed that the attributes chart not be used for control purposes. Rather, a sorting procedure be used to select out the lower tail of the population and the variables chart procedure be used to control the mean value of this tail. Ideally, this mean value should lie somewhere near the 5-percentile value of the parent population. A procedure for doing this has been suggested in a previous paper (Leicester and Young 1991).

TABLE 3.
EXAMPLE FROM CUSUM CHARTS PROCEDURES (N = 5)

Type of chart	Mean production run	
	Mean strength = 40 MPa	Mean strength = 32 MPa
Attributes chart, coefficient of variation = 0.45, proof load = 10.28 MPa (= 3-percentile value if mean strength is 40 MPa)	511	99
Variables chart, coeff. of variation = 0.2	331	2.5

5.3 Frequency Check

Recommended sampling rates for timber range from a minimum rate of 1 in 5000 for every size/grade combination to a rate of 1 in 1000 for tight control.

6. RANDOM CHECK ON DESIGN PROPERTIES

This refers to the situation where a group of customers, represented for example by a trade or standards association, wishes to check whether a new producer, such as for example, an overseas producer, has claimed the correct structural design properties. For this case, the annual check procedures discussed earlier in Section 3 would be suitable.

7. RANDOM CHECK ON A BATCH OF TIMBER

7.1 Quality Criterion

For this case, there is a limited supply of timber and hence, only a small sample is available for testing. The primary consideration here is that the producer not be penalised because of the small sample size used for the assessment. If the customer wishes to incur less risk then it is his choice and cost to test a larger sample.

The proposed quality criterion is

$$R_{sample,0.05}\,\{1 + (5V_{sample}/\sqrt{N}\,\} > R_{parent,0.05} \qquad (10)$$

An alternative or additional criterion is

$$R_{sample,mean}\,\{1 + (2V_{sample}/\sqrt{N}\,\} > R_{parent,mean} \qquad (11)$$

where $R_{parent,mean}$ denotes the mean value of the parent population.

7.2 Value to the Producer

With the above criteria there is a risk to the producer that if his material is just adequate then there is a 5% chance that his material will fail the test. However, for the reasons mentioned in Section 2, the producer is likely to have some conservatism in his original selection of the 5-percentile value and hence, on average, his risk will be considerably less than 5%.

7.3 Value to the Customer

There is considerable risk to the customer that he may accept batches of timber that are considerably under strength. For example, if the test sample size is N = 30, and the timber has a coefficient of variation of 40%, there is a 10% chance that timber which just passes the test is under strength by a factor of 0.6 if the test is based on the criterion stated in equation (10) and by a factor of 0.8 if the test is based on the criterion stated in equation (11).

Note that equation (11) examines the mean strength of the population and not the important 5-percentile strength. However, for small sample sizes the use of this criterion may constitute less of a risk than the use of equation (10).

8. CONCLUDING COMMENT

One feature exposed by all the proposed checks is the difficulty in the use of small test samples to provide a minimum risk to both producer and consumer. In practice, it will be necessary to negotiate as to which party is to be disadvantaged least by the choice of parameters used in the various criteria used for quality checks.

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APPENDIX A
TABLES FOR CUSUM CHART PROCEDURES

TABLE A1.
RECOMMENDED CONTROL CONSTANTS K, Y, Z
FOR ATTRIBUTES CUSUM CHART

Sample size	Control parameters		
	K	Y	Z
N = 5	1	1	6
N = 10	1	2	6
N = 20	1	4	7
N = 40	2	8	11
N = 60	4	8	15

TABLE A2.
CONTROL CONSTANTS K, Y, Z FOR VARIABLES CUSUM CHART

COV	Normalised constants* for N = 5			Normalised constants* for N = 10			Normalised constants* for N = 20		
	K	Y	Z	K	Y	Z	K	Y	Z
0.05									
0.10	0.9625	0.094	0.232						
0.15	0.9625	0.199	0.363	0.9625	0.105	0.263	0.9625	0.053	0.196
0.20	0.9625	0.334	0.513	0.9625	0.181	0.344	0.9625	0.094	0.232
0.25	0.9625	0.475	0.672	0.9625	0.264	0.435	0.9625	0.144	0.304
0.30	0.9625	0.644	0.865	0.9625	0.365	0.547	0.9625	0.201	0.363
0.35				0.9625	0.470	0.669	0.9625	0.261	0.430
0.40				0.9625	0.592	0.805	0.9625	0.335	0.514
0.45				0.9625	0.712	0.940	0.9625	0.406	0.592
0.50							0.9625	0.483	0.679

* All constants must be multiplied by the mean value of the variable.

TABLE A3.
RULES FOR COMPUTING CUSUM

Previous CUSUM*	New $CUSUM_i$				
	$SUM_i \leq 0$	$0 < SUM_i < Y$	$SUM_i = Y$	$Y < SUM_i < Z$	$SUM_i \geq Z$
$CUSUM_{i-1} = 0$	0	SUM_i	Z	Z	Z
$0 < CUSUM_{i-1} < Y$	0	SUM_i	Z	Z	Z
$Y < CUSUM_{i-1} < Z$	0	0	0	SUM_i	Z
$CUSUM_{i-1} = Z$	0	0	0	SUM_i	Z

$CUSUM_i \leq Y$, process is in-control
$CUSUM_i > Y$, process is out-of-control

* The condition CUSUM = Y cannot occur

TABLE A4.
EXAMPLE FROM ATTRIBUTES CHART PROCEDURE

Sample size	Average clear run		Average production run	
	Mean strength = 40 MPa	Mean strength = 32 MPa	Mean strength = 40 MPa	Mean strength = 32 MPa
N = 5	120	44	511	99
N = 10	260	51	1620	97
N = 20	293	22	894	32

Coefficient of variation of strength = 0.45

Proof load = 10.28 MPa; this is the 3-percentile value of timber with mean strength of 40 MPa

TABLE A5.
EXAMPLE FROM VARIABLES CHART PROCEDURE

Sample size	Average clear run		Average production run	
	Mean strength = 40 MPa	Mean strength = 32 MPa	Mean strength = 40 MPa	Mean strength = 32 MPa
N = 5	100	1.5	331	2.5
N = 10	95	1.0	405	1.6
N = 20	107	0.0	756	1.0

Coefficient of variation of strength = 0.2

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CODIFICATION OF SERVICEABILITY CRITERIA

by

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MEETING TWENTY - SEVEN

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JULY 1994

CODIFICATION OF SERVICEABILITY CRITERIA

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SUMMARY

This paper presents a statistical model for the codification of serviceability criteria for both design codes and performance standards. This model is based on optimising cost functions. A key consideration is the complex nature of human response to structural unserviceability; it is highly variable and is affected by many non-structural parameters. General matters related to codification are also discussed.

1. INTRODUCTION

Because of the increased use of higher strength and lighter weight materials for construction, serviceability considerations have become a controlling consideration in a large proportion of structural designs. However, control of serviceability tends to be an 'art' practiced by experienced engineers; no formal framework exists for discussions on this matter. In the following, a statistical model is described and proposed as a basis for developing a formal framework for unserviceability. It is discussed in the context of building deformation, material cracking, floor vibration and building sway.

2. THE CHARACTERISTICS OF UNSERVICEABILITY

2.1 General

In this paper, the term unserviceability will be taken to refer to all structural behaviour, excluding collapse, that renders a building or construction unfit for its intended use. The lack of fitness may relate to human responses (aesthetic, psychological or physiological) or it may relate to matters that hinder the satisfactory operation of equipment. In contrast to ultimate limits states, serviceability is defined in subjective terms and a failure thereof may, at least in theory, be remedied.

With respect to human responses, there are two complex aspects which cannot be ignored. The first is the large variety of human reactions that may result from each type of unserviceable structural behaviour, and secondly, the fact that many reactions are the result of a combination of structural and non-structural effects. Examples of these are given in the following.

The aesthetic impact of building deviations is known to be considerably influenced by non-structural parameters such as the incident angle of surface lighting, the surface colour and texture and also whether there are any visual references (such as free standing cupboards) to assess the magnitude of deviations [1,2].

Similarly, the impression of a crack in a material is known to be influenced by non-structural parameters such as mode of lighting, surface texture, the occurrence of dirt within cracks and the viewing distance [3]. Apart from aesthetics, there appears to be a strong psychological element in the human response to the observation of cracks. For example, in their survey on human response, Padilla and Robles used attitude scales with questions containing the phrases 'give a bad impression', 'annoy me', 'proof that bad materials were used', 'feeling of danger' and 'fear that the apartment will collapse' [4].

Human response to building sway ranges all the way from 'feeling refreshed', to annoyance, to nausea and acrophobia; also the effects of structural accelerations interact with both visual and sound stimuli to produce these responses [5, 6, 7]. Similarly, it has been found that the acceptance of a floor system (with respect to vibration characteristics) is strongly influenced by sound stimuli [8, 9].

3. STATISTICAL MODELS

3.1 Rationale

The proposal is to use a simple statistical model to integrate the influences of the most important parameters associated with serviceability design. These parameters are those related to uncertainty and cost. The use of formal models, as an alternative to expert opinion, has several advantages. It presents a format which facilitates code committee discussion, facilitates the incorporation of data and exposes deficiencies in information. Furthermore, the use of a formal model avoids the bias that is known to occur in the heuristic or intuitive processing of statistical data [10].

3.2 Model for Design Codes

It will be assumed that a design code is a code that is optimised from the viewpoint of an owner/builder. The statistical model used for this purpose is illustrated schematically in Figure 1. It is stated in terms of an unserviceability parameter such as crack width or floor flexibility. The scenario assumed is that the unserviceability parameter has a value T for a particular building; should this parameter exceed the tolerance level or complaint threshold of a particular tenant, denoted by U, then an effective additional cost of C_F will be incurred. This cost may be taken to include not only direct costs, such as remedial structural costs, but also indirect costs that may arise, for example, from bad publicity or the loss of tenants. The aim is to choose an optimum value of $\overline{T}$ so as to minimise the total costs involved.

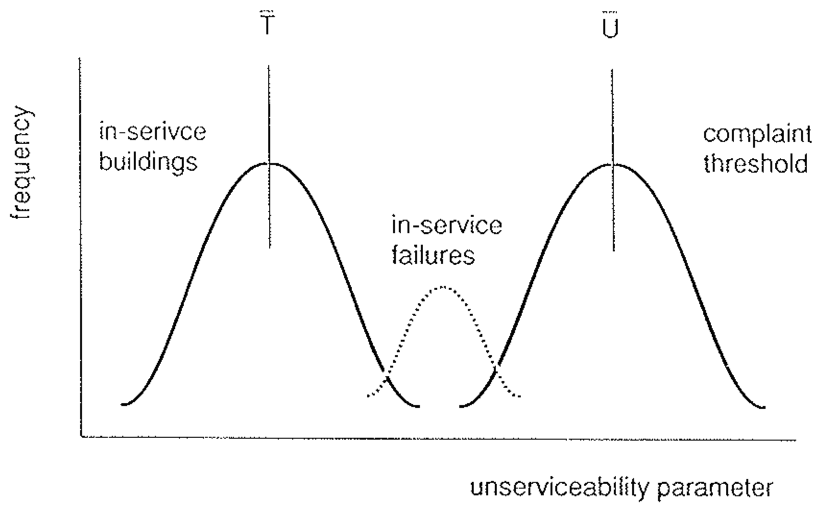


Figure 1. Statistical model for a design code.

For the building owner, the cost C associated with a design is given by

$$C = C_S + C_F\, p_F \tag{1}$$

where CS denotes the cost of a structure and $p_F = \Pr(U < T)$ is the probability of exceeding the complaint threshold.

If it is assumed that

$$C_S = A\overline{T}^{-m} \tag{2}$$

and

$$p_F = B(\overline{U}/\overline{T})^{-n} \tag{3}$$

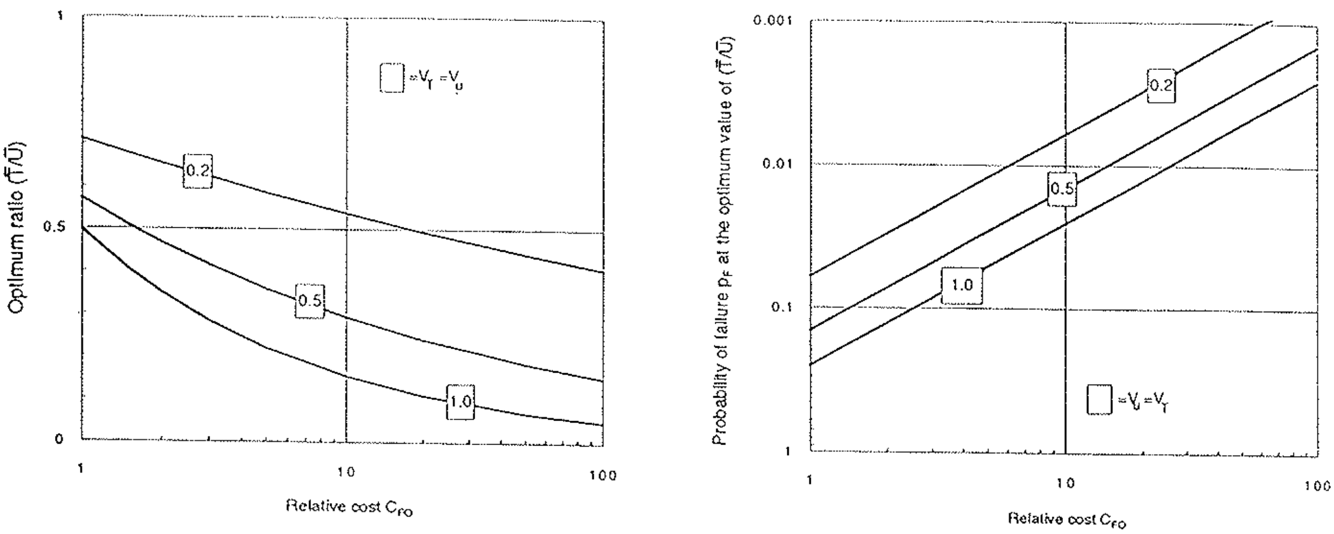
where A, B, m and n are constants, then optimisation of equation (1) with respect to choice of $\overline{T}$ leads to

$$(\overline{U}/\overline{T}) = [n\, B\, C_{FO}/m]^{1/n} \tag{4}$$

$$p_F = m/[n\, C_{FO}] \tag{5}$$

where $C_{FO} = C_F/C_{SO}$ in which C_{SO} denotes the cost of the optimum structure.

Appendix A gives a method for estimating the parameters B and n for use in equations (4) and (5); these parameters are stated in terms of V_T and V_U, the coefficients of variation of T and U respectively. Some typical optimised values of $\overline{T}/\overline{U}$ and p_F based on these assumptions are shown in Figure 2.



(i) ratio ($\overline{T}/\overline{U}$)

(ii) Probability of failure

Figure 2. Optimum values for design codes (m = 0.5).

3.3 Model for Performance Standards

The statistical model for this case is illustrated schematically in Figure 3. Here an in-service value L of the unserviceability parameter is specified as a legal limit. If this limit is exceeded, the builder must pay a remedial cost C_F. If the limit is not exceeded, but the unserviceability parameter exceeds the complaint threshold of a tenant, then the owner will pay for the costs of remedial action.

With these assumptions, the cost to the building owner is

$$C = C_S + C_F\, p_{F1} \tag{6}$$

where $C_S = A_1 L^{-m}$, A_1 is a constant and $p_{F1} \leq \Pr(U < L)$.

The cost to the builder is

$$C = C_S + C_F\, p_{F2} \tag{7}$$

where $C_S = A_2 L^{-m}$, A_2 is a constant and $p_{F2} = \Pr(T > L)$.

It is now assumed that first the building owner selects the legal limit L using the conservative assumption that $p_{F1} = \Pr(U < L)$ so as to minimise his costs, and then the builder selects the target in-service value of $\overline{T}$ so as to minimise his costs. Then the optimisation of equations (6) and (7) leads to equations identical to those for the optimisation of equation (1), except that the coefficients of variation $V_T = 0$ and $V_U = 0$ are to be used in the optimisation of equations (6) and (7) respectively. Some optimum solutions for these cases are shown in Figure 4.

The assumption used for p_{F1} is quite conservative. Possibly a more realistic approach would be to ignore equation (6) and to derive $\overline{U}$ from equation (4) after the optimum value of $\overline{T}$ has been obtained from the optimisation of equation (7).

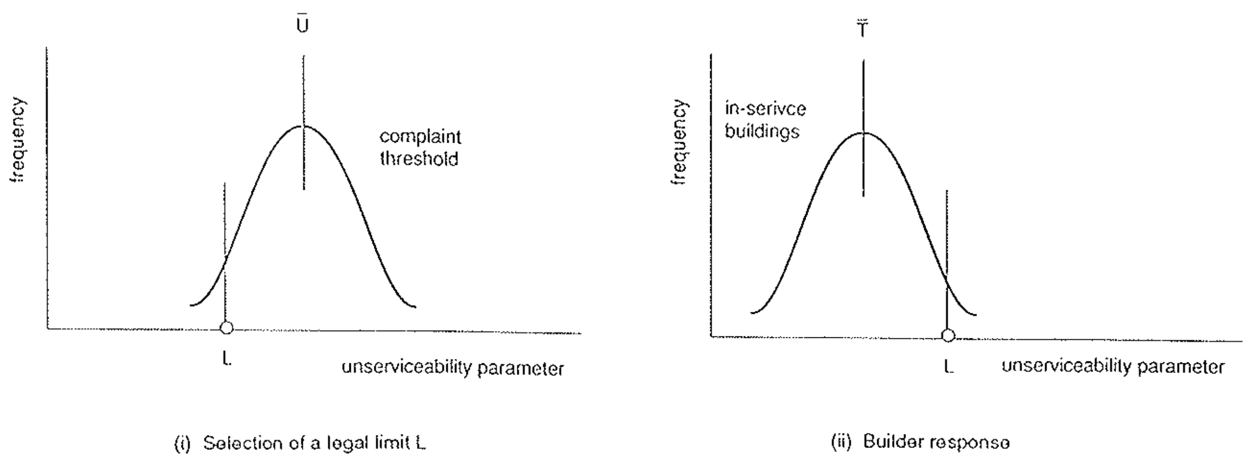
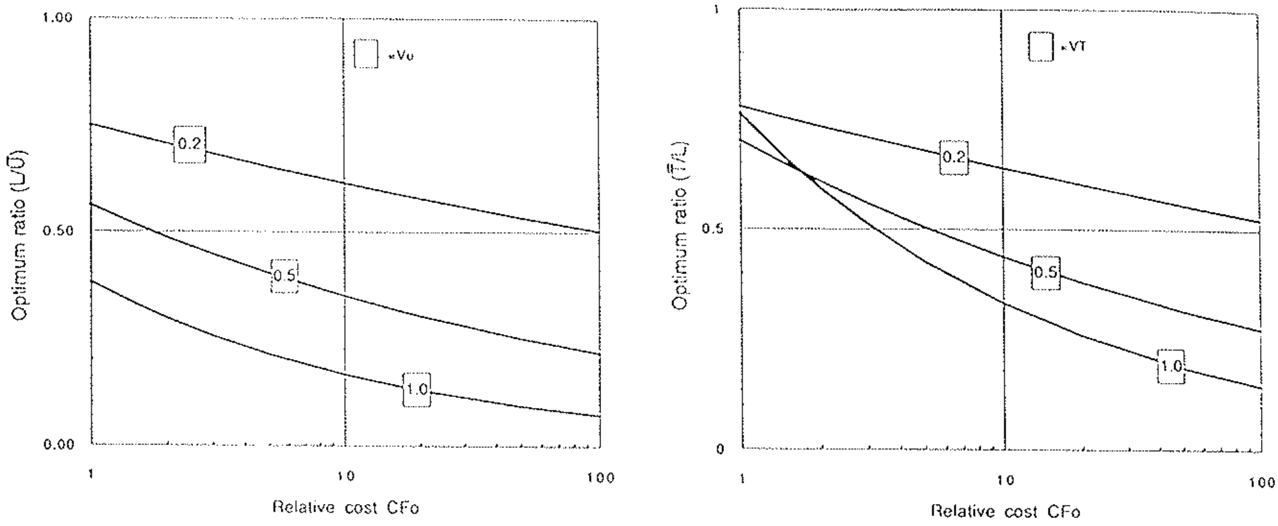


Figure 3. Statistical model for a performance standard.



(i) Selection of a legal limit L

(ii) Builder response

Figure 4. Optimum values for performance standards (m = 0.5).

4. CODE PARAMETERS

4.1 Unserviceability Parameters

The choice of an unserviceability parameter is a critical one in the use of a statistical model, since all uncertainties and structural properties must be stated in terms of the unserviceability parameter.

For building deformations the choices of unserviceability parameters have included deviations from straight lines, distorted right angles and slopes of floors [11, 2]. For cracks in a structural element the crack width is the normal parameter, although crack length is sometimes considered [3]. For building sway the unserviceability parameter is usually linear acceleration [12].

For floor vibrations a great variety of parameters have been used and include complex functions of displacement, velocity, acceleration, frequency and damping [13, 14]. Even for the narrow topic of a wooden floor, a great variety of choices have been made; Ohlsson uses the peak velocity arising from 1 N-s impulse loading [15], Chui and Smith use the peak acceleration due to a heel-drop loading [16], and Onysko and Russell both use the deflection due to a static point load [17, 9].

4.2 Human Response

Data on human response is not easy to obtain, but where it is available it shows a high variability. Figure 5 indicates a coefficient of variation of about 50% on the perception threshold for horizontal vibrations [19]; and Figure 6 shows a coefficient of variation of 30% on the rejection threshold for the flexibility of wooden floors [9].

It is important to note that these estimates of variability were all obtained under controlled laboratory conditions. As mentioned earlier, the in-service variability would be considerably greater due to the influence of non-structural parameters.

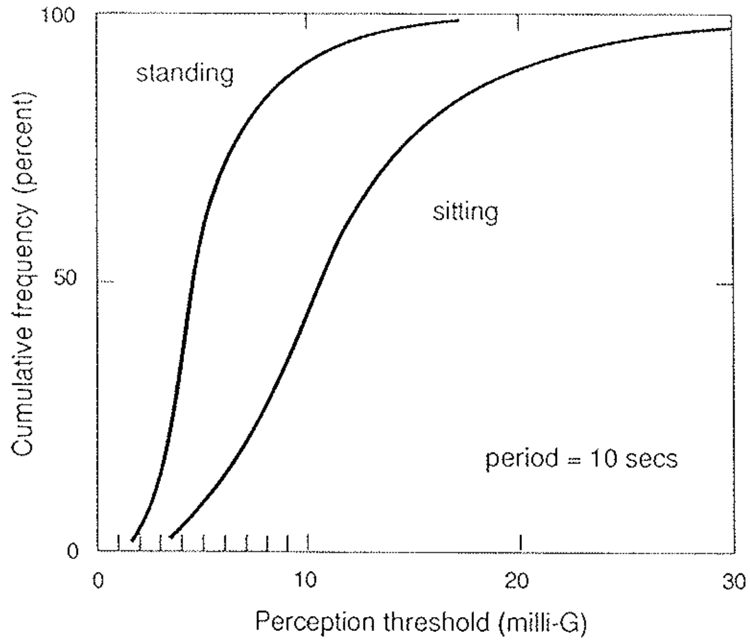


Figure 5. Perception threshold for horizontal vibrations, after Chen and Robertson [19].

4.3 Structural Response

A measure of the variability of structural response can be obtained from experimental data that compares measured and predicted structural properties. The author knows of no examples related to timber construction. However, some examples for other materials are a coefficient of variation of 35% on the long-term deflection of reinforced concrete beams [20], 30–100% on the prediction of horizontal response of tall buildings [22, 23] and 30% on the vertical accelerations of floors due to the heel impact test [24].

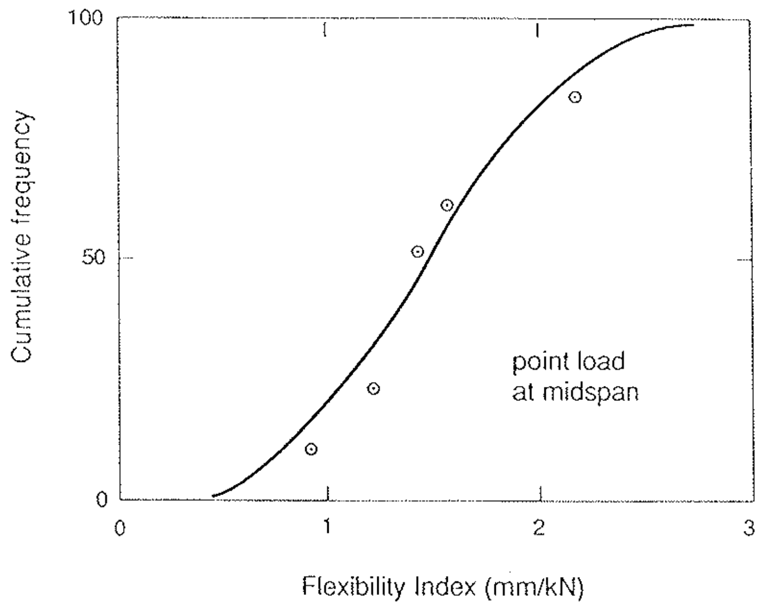


Figure 6. Rejection threshold for wooden floors [9].

4.4 Cost Parameters

Cost parameters have been examined in an earlier paper and are found to be in the range m = 0.2 – 1.0 and C_{FO} = 1 – 20 [25].

5. EXAMPLE

As an example, consider the design of a floor against vibration based on the data given in Figure 6. If it is assumed that the uncertainty of response and structural design are given by $V_U = V_T = 0.3$ and that the costs associated with the occurrence of an unsatisfactory floor is equal to the cost of the floor joist material, then from Figure 2(a) the mean target design flexibility $\bar{T}$ is given by $\bar{T}/\bar{U} = 0.7$. Hence, if the mean acceptable flexibility from Figure 6 is $\bar{U} = 1.3$ mm/kN; then the design target for the floor flexibility is $\bar{T} = 0.7 \times 1.3 = 0.9$ mm/kN.

6. CODIFICATION

In drafting a design code it is important that the intent of serviceability criteria is clear; ideally a total scenario, including remedial actions, should be stated. For example, difficulties frequently arise in the application of design codes because it is not clear as to whether deformation limits are intended for aesthetic or damage control purposes.

Similarly, for performance standards it is important that performance criteria be stated in terms of parameters that are easily measured; if this is not done, then there arises the potential for lengthy litigation in the event of a dispute on a failure. Thus, the crack width would be a useful parameter for this purpose whereas the lateral sway of a building in a 50-year return wind would not.

Finally, it should be repeated that there are many types of human response to unserviceability and that there are many non-structural parameters that affect human response to structural unserviceability. Hence, there is a strong case to be made that codes and standards should specify multiple serviceability limits rather than the format of single limits as is the current custom.

7. CONCLUSIONS

The nature of structural unserviceability has been discussed, a statistical model presented for the quantification of design and performance criteria, and some procedures for codification suggested. Probably the most interesting aspects of this study are those concerning human response to unserviceability; it was noted that this response is extremely variable, complex in nature and it is affected by many non-structural parameters.

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APPENDIX A

PROBABILITY OF FAILURE

Because relatively high probabilities of failure are involved in serviceability limit states, the choice of statistical distributions for the variables T and U is not critical. For convenience, log normal distributions will be chosen. The probability of failure is then given by

$$\begin{aligned} p_F &= \Pr(U < T) \\ &= \Phi(-\beta) \end{aligned} \tag{A1}$$

where

$$\frac{\beta = \ell n(\bar{U}/\bar{T}) + \ell n\left[\left(1 + V_2^T\right)/\left(1 + V_2^u\right)\right]^{1/2}}{\left\{\ell n\left[\left(1 + V_2^T\right)\left(1 + V_2^u\right)\right]\right\}^{1/2}} \tag{A2}$$

in which $\Phi(\,)$ denotes the cumulative distribution function of a unit normal variate.

Equation A1 may be approximated by a power function within the range of interest. For most serviceability limit states a suitable approximation is

$$p_F \approx 10^{-\beta} \tag{A3}$$

Equations (A2) and (A3) then lead to

$$p_F \approx B(\bar{U}/\bar{T})^{-n} \tag{A4}$$

where

$$n = 2.3/\left\{\ell n\left[\left(1 + V_2^T\right)\left(1 + V_2^u\right)\right]\right\}^{1/2} \tag{A5}$$

and

$$B = \left\{\left[\left(1 + V_2^T\right)/\left(1 + V_2^u\right)\right]\right\}^{-n/2} \tag{A6}$$

For coefficients of variation less than 0.3,

$$n \approx 2.3/\left[V_2^T + V_2^u\right]^{1/2} \text{ and } B \approx 1.$$

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INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

ON THE EXPERIMENTAL DETERMINATION OF FACTOR K_{DEF} AND SLIP MODULUS K_{SER} FROM SHORT- AND LONG-TERM TESTS ON A TIMBER-CONCRETE COMPOSITE (TCC) BEAM

by

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MEETING TWENTY - SEVEN

SYDNEY

AUSTRALIA

JULY 1994

On the experimental determination of factor k_{def} and slip modulus K_{ser} from short- and long-term tests on a timber - concrete composite (TCC) beam

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ABSTRACT

The factor k_{def}, introduced by Eurocode 5 to take into account the increase in deformation due to creep and moisture, is determined from a long-term test (3 years duration) on a timber and concrete composite (TCC) beam in service class 3. The difference in k_{def} values for deflections and slips is discussed. The experimental deformation-time curves, combined with moisture and shrinkage/swelling measurements, show a high influence of a hygro-mechanical effect due to the differential shrinkage between timber and concrete.
Monitoring results of the service behaviour of a real structure (twin of the tested one) during three years in service class 1 is reported.
Finally, the istantaneous slip modulus K_{ser} for TCC fasteners is determined from previously performed short-term loading tests.

INTRODUCTION

The Eurocode 5 on "Design of Timber Structures" [1], now circulating as ENV 1995-1-1, has introduced the factor k_{def} to take into account "the increase in deformation with time due to the combined effect of creep and moisture". For solid and glulam timber, the k_{def} values according to different service classes and load-duration classes are shown in table 1 (from table 4.1 in EC5).

Table 1 (from EC5): Values of k_{def} for solid/glulam timber and joints

	Service class		
Load duration class	1	2	3
Permanent	0.60	0.80	2.00
Long term	0.50	0.50	1.50
Medium term	0.25	0.25	0.75
Short term	0.00	0.00	0.30

For mechanically jointed beams, the joint slip is also prescribed to be considered as a function of the slip modulus K_{ser} , both in serviceability limit state (clause 4.2) and in ultimate limit state (clause 5.3.3).

A reference design method for mechanically jointed beams is then provided in Annex B; it may also apply "to composite members made from timber in combination with other materials" (footnote 14 at page 61).

In this paper, these two serviceability limit state parameters are determined for a timber and concrete composite (TCC) beam from experimental data obtained by long-term and in-situ tests carried out in Italy, according to the above mentioned definitions and prescriptions.

TEST PROGRAMME

I) A TCC beam full-scale model has been realized in order to perform short- and long-term tests. It is composed of two parallel glulam spruce beams (sized 125 mm x 500 mm x 6000 mm), mechanically connected to a concrete slab cast over a corrugated sheet steel (length 6.00m; width 1.50 m; slab thickness 50 mm; corrugations height 50 mm). Th fasteners are R.C. corrugated bars, d=14 mm, put into predriven holes filled with epoxy resin, with variable spacing from i_{min} = 150 mm near the beam endings until to i_{max} = 450 mm in the central zone. It is shown in fig.1 in the simply supported test configuration, over a span of 5.70 m.

The following tests have been performed on the model:

a) a long duration (3 years) test under sustained load (¼ of service design load, assumed to be the actual quasi-permanent load);

b) a short-term loading test at 1.25 times the service design load.

Midspan deflections and end slip have been measured; in the long-term test also timber moisture content and timber shrinkage/swelling deformations at different points have been measured, in addition to air temperature and R.H. registrations.

II) The above full-scale model should represent the behaviour of a real structure, realized in 1990 in Forlì (I) according to the same system. The only differences are the total span (10 m), the cross-sectional and support arrangements (see fig.2) and the fastener diameter (18 mm) and spacing (from 150 to 300 mm).

Three beams of the same floor have been monitored since november 1990 (about 6 months after erection). Midspan deflections and timber moisture content are monthly measured, while environmental conditions are continuously recorded.

RESULTS DISCUSSION

A) Long-term test results

Moisture condition and service class

The long-term test has been carried out in outdoor not covered environment. That should be enough to assign the beam to service class 3. Anyway, when looking to moisture registrations (electric resistance method, depth 40 mm) and longitudinal and transversal wood deformations during time (see respectively figg. 3, 4, 5), we can see a total range of moisture variations from 13.5% to 18.5%, confirmed by the corresponding calculated values of shrinkage/swelling coefficients of

$\alpha_{w//} = 6 \times 10^{-5}$ parallel to the grain,

$\alpha_{w\perp} = 1.2 \times 10^{-3}$ perpendicular to the grain.

According to EC5 definition of service classes, it might enter the service class 2 (it may be supported by the consideration that the timber beams are covered by the concrete slab ...).

The deflection and slip data will definitively require service class 3, as the first glance had suggested.

Deflection and slip results : determination of k_{def}

Midspan deflections and end slip are plotted against time in figg. 6 and 7 respectively. The following considerations can be drawn:

- The initial elastic values are respectively:

  $u_e = 0.87$ mm ; $v_e = 0.045$ mm.

- The maximum values during three years testing are:

  $u_{max} = 3.3$ mm ; $v_{max} = 0.31$ mm.

  with corresponding factors of increased deformation

  $k_{def}(u) = 2.8 \approx 3.0$; $k_{def}(v) = 5.9 \approx 6.0$.

- There is evidence that service class 3 is required for the tested beam.

- Both k_{def} values overcome the corresponding EC5 prescription ($k_{def} = 2.0$ for permanent load and s.c. 3). Further, k_{def} (v) is two times k_{def} (u). It is to be discussed if a distinct set of k_{def} values has to be included for joint slip. It has to be remarked that the load level in this test is deliberately not a high level. On the other side, it must be noticed that high variations in slip turned in much lower variations in deflections, that is the leading design parameter in such a structure.

- The alternating shape of both deflection and slip curves (with opposite phases) can be explained, combined with moisture and shrinkage/swelling data, referring to the dominant influence of a hygro-mechanical effect due to differential shrinkage between timber and concrete. That is, shrinking timber turns in upwards displacements of the beam sections (decreasing deflections) and increasing slips, while swelling timber turns in increasing deflections and decreasing slips.

B) Monitoring results

The midspan "apparent" deflections of the three monitored TCC beams are plotted in fig.8, while in fig.9 deep and superficial moisture content data are reported. The following considerations can be drawn:

- The first evidence is that the deflection after 3 years is about the same as the "initial" (after 6-10 months from erection) value.
- Midspan deflections show once again periodical oscillations, following (a little late) the annual moisture cycles. The same hygro-mechanical effect is likely to be responsible of this behaviour. The amplitude of annual deflection cycles is quite small, a little less than 10% of total amount.
- moisture data show that the beams lay in service class 1.
- a light increasing trend in the deflection peaks after the first year is likely to represent the only sign of a viscoelastic contribution in the total deflection.

C) Short term results : determination of K_{ser}

In fig.10 the load-deflection diagram for the loading test at 1.25 service design load (F_d = 20 kN/m) is reported. The maximum value of midspan deflection is

$$u_c\,(1.25\ F_d) = 5.3\ \text{mm} \qquad \text{(average of the two timber beams).}$$

Introducing this experimental value in the design formulae of Annex B [1], the slip modulus for the mechanical fastener can be calculated, that is

$$K_{ser} = 11700\ \text{N/mm}\ .$$

The slip modulus K_{ser} may also be obtained according to EC5, Table 4.2 (dowel-type fasteners), taking into account the different density values of the two materials. It gives

$$K_{ser} = 21500\ \text{N/mm}\ ,$$

that almost doubles the previous experimental value. The corresponding elastic deflection would be u_c = 4.25 mm.

Referring to ISO 6891/EN 26891, a k_e value of slip modulus can be obtained from unloading-deflection data, where the average elastic recovery of deflection was

$$u_e\,(-1.25\ F_d) = 4.67\ \text{mm}\ .$$

From Annex B formulae we obtain for the slip modulus

$$k_e = 16900\ \text{N/mm}\ .$$

Again, from the elastic deflection measured at loading the model for the long-term test, u_e = 0.87 mm, a new experimental value

$$k_e = 20900\ \text{N/mm}$$

is obtained.

The following considerations can be drawn:

- Calculating K_{ser} according to EC5 for the tested TCC beam overestimates the connection stiffness and underestimates the deflection.
- However, an error of about 100% on K_{ser} turns in only a 25% error on deflection.
- After the first load cycle, the slip modulus increases, getting quite near to the value predicted by EC5.

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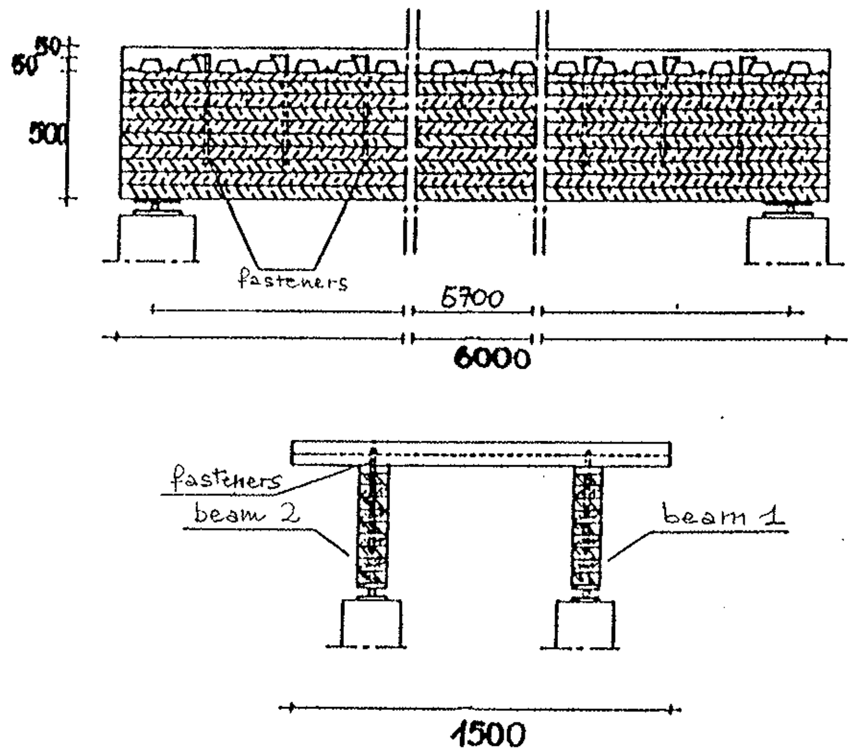
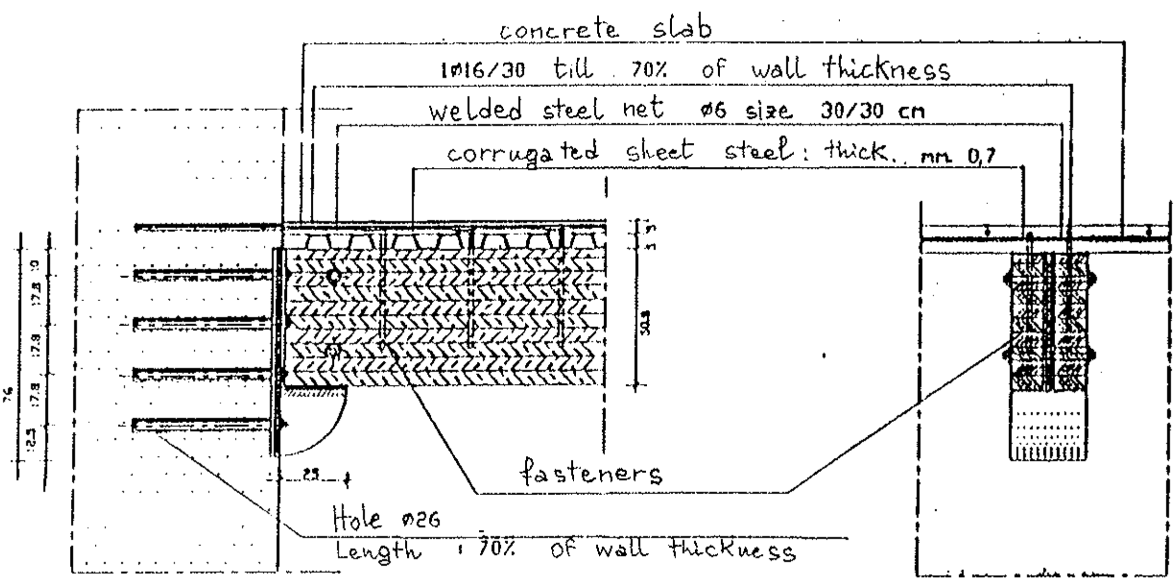


50
50
500
fasteners
5700
6000
fasteners
beam 2
beam 1
1500

Fig. 1



concrete slab
1Ø16/30 till 70% of wall thickness
welded steel net Ø6 size 30/30 cm
corrugated sheet steel: thick. mm 0,7
fasteners
Hole Ø26
Length 70% of wall thickness

Fig. 2

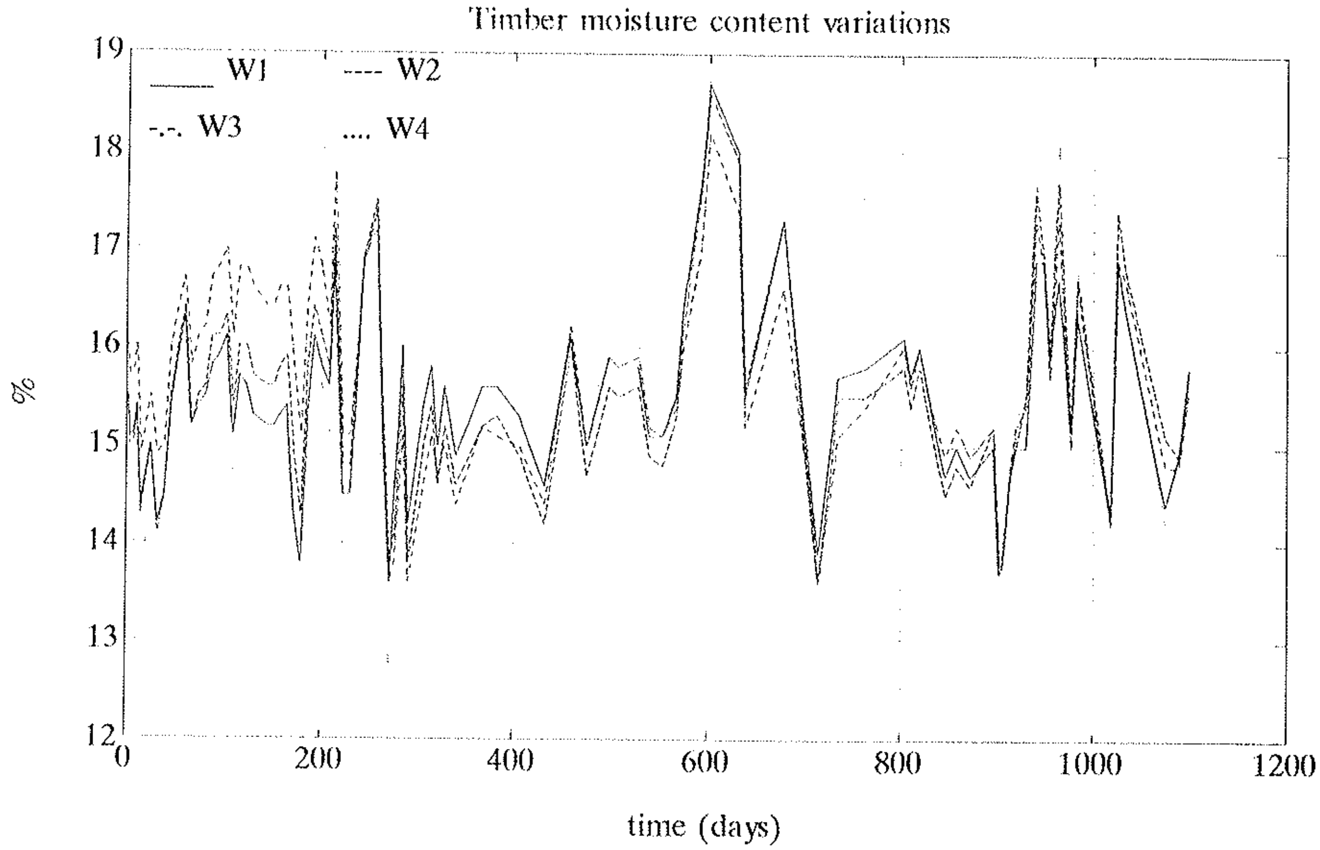

Timber moisture content variations
W1
W2
W3
W4
%
time (days)

Fig.3

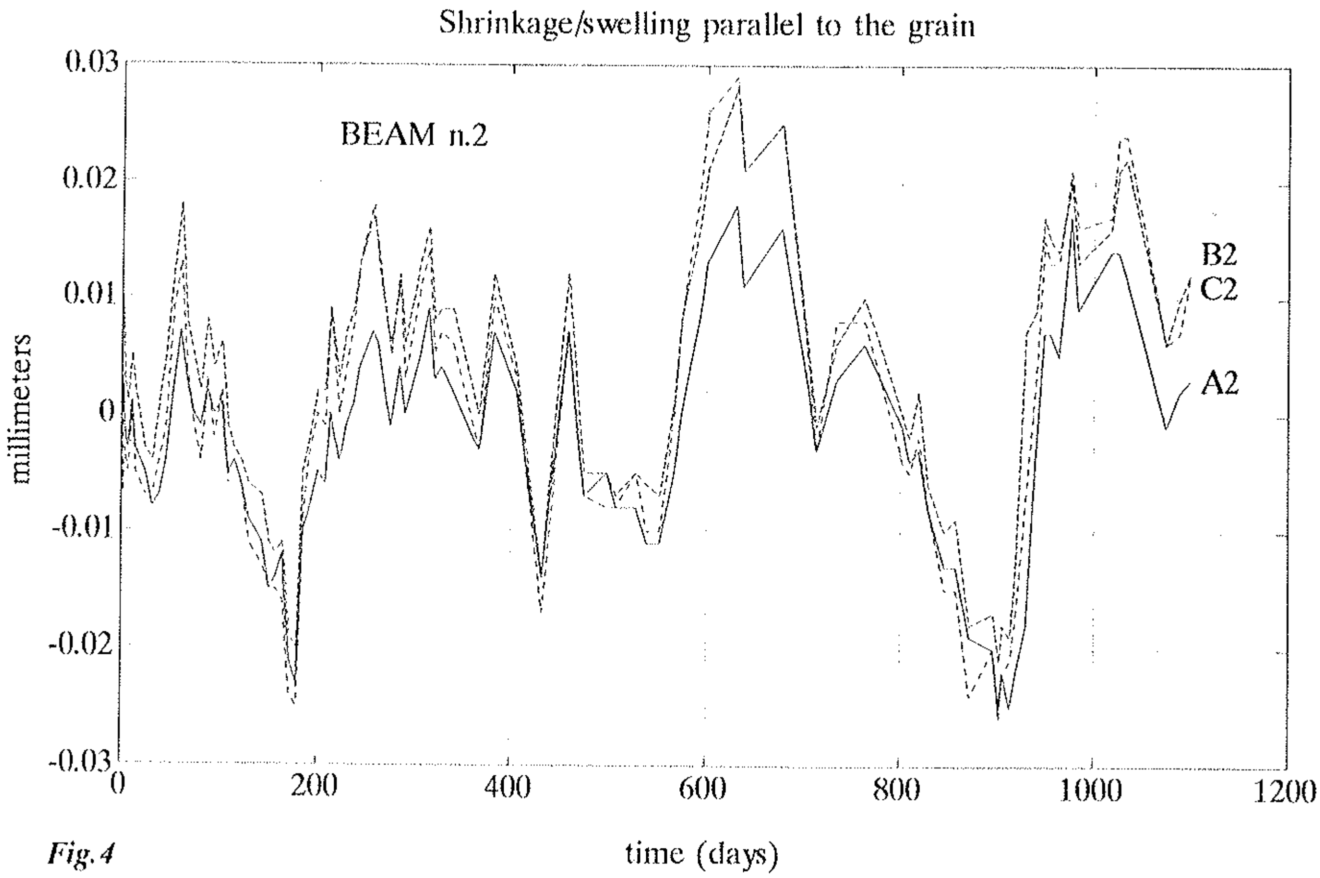
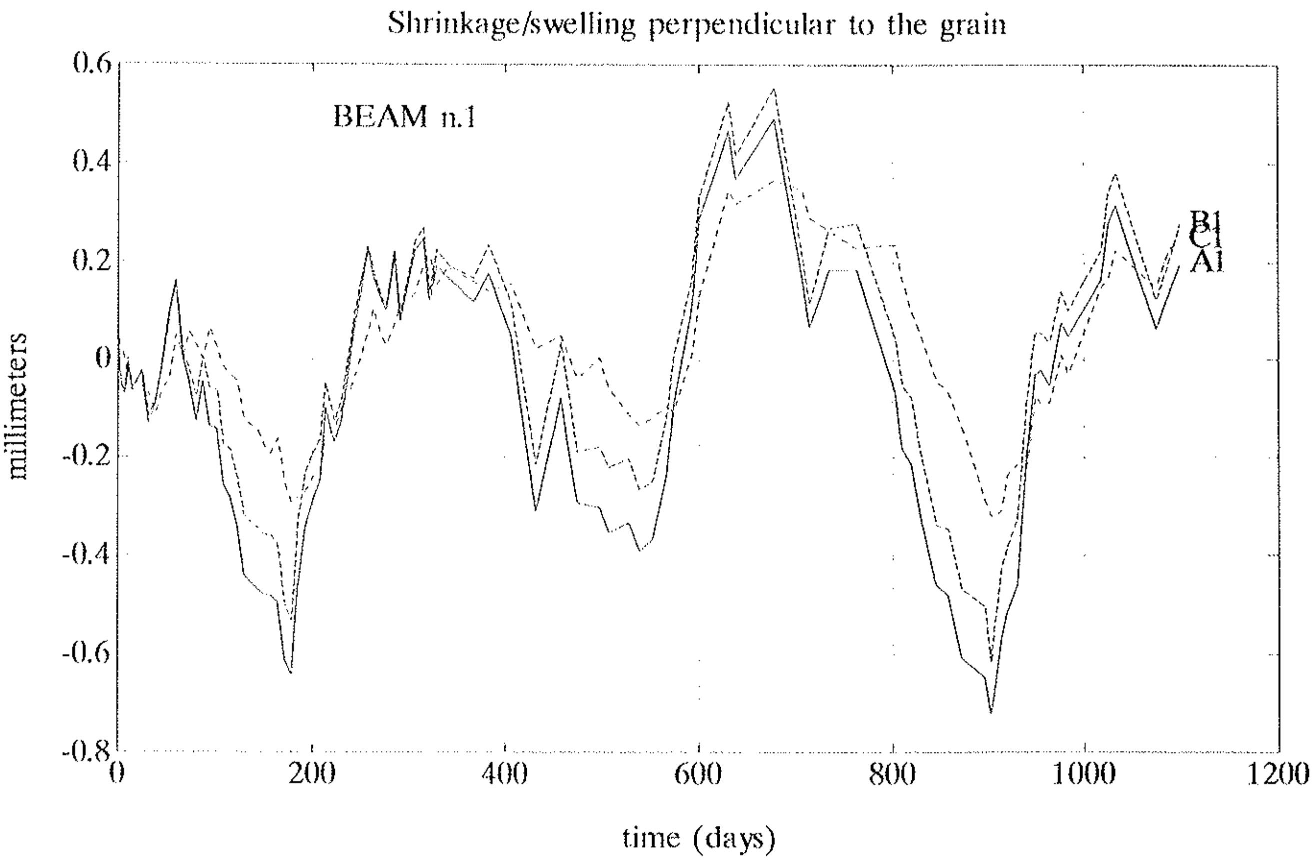
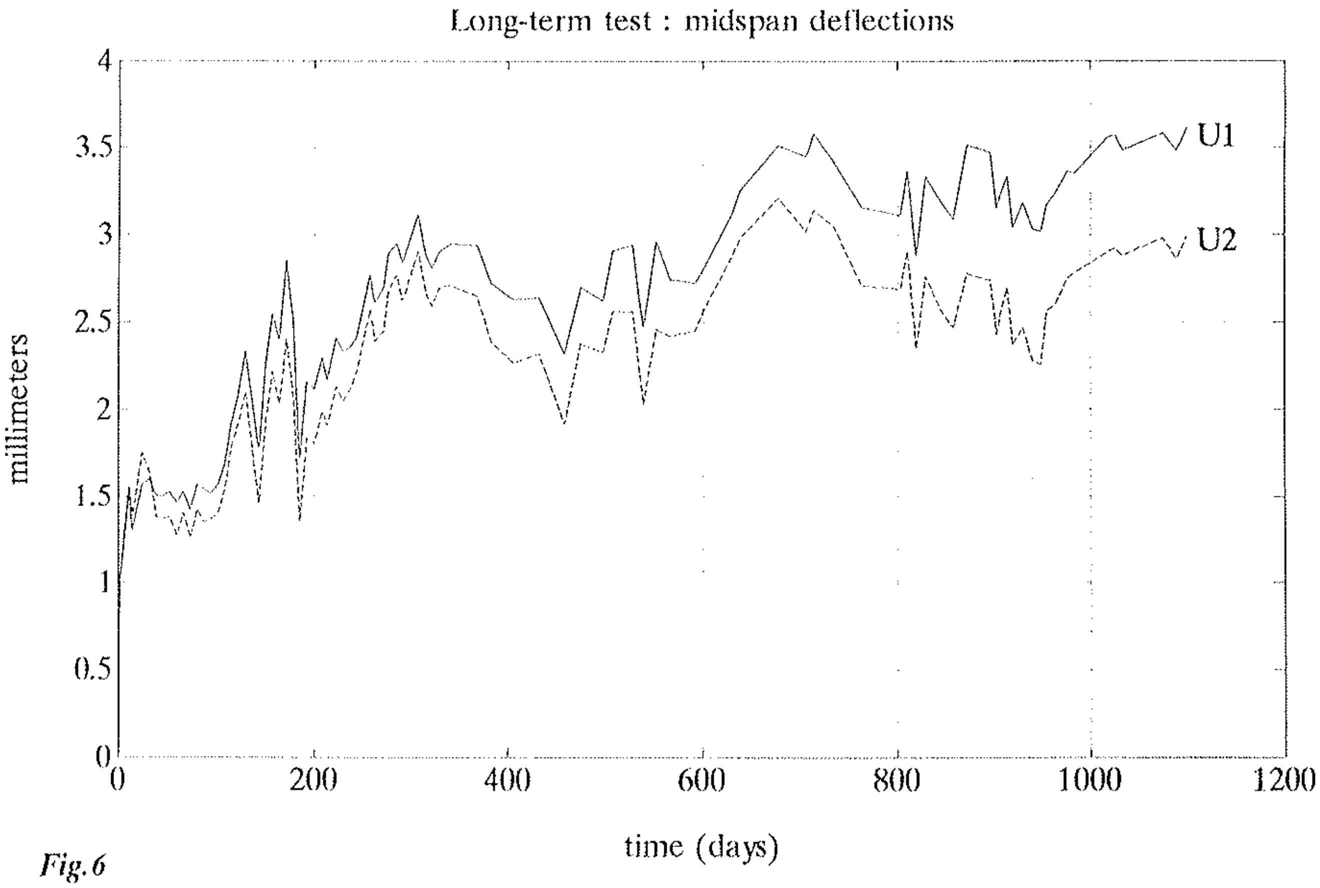

Shrinkage/swelling parallel to the grain
BEAM n.2
B2
C2
A2
millimeters
time (days)

Fig.4



Shrinkage/swelling perpendicular to the grain
BEAM n.1
millimeters
time (days)
B1
C1
A1
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
0
200
400
600
800
1000
1200

Fig. 5



Long-term test : midspan deflections
millimeters
time (days)
U1
U2
4
3.5
3
2.5
2
1.5
1
0.5
0
0
200
400
600
800
1000
1200

Fig. 6

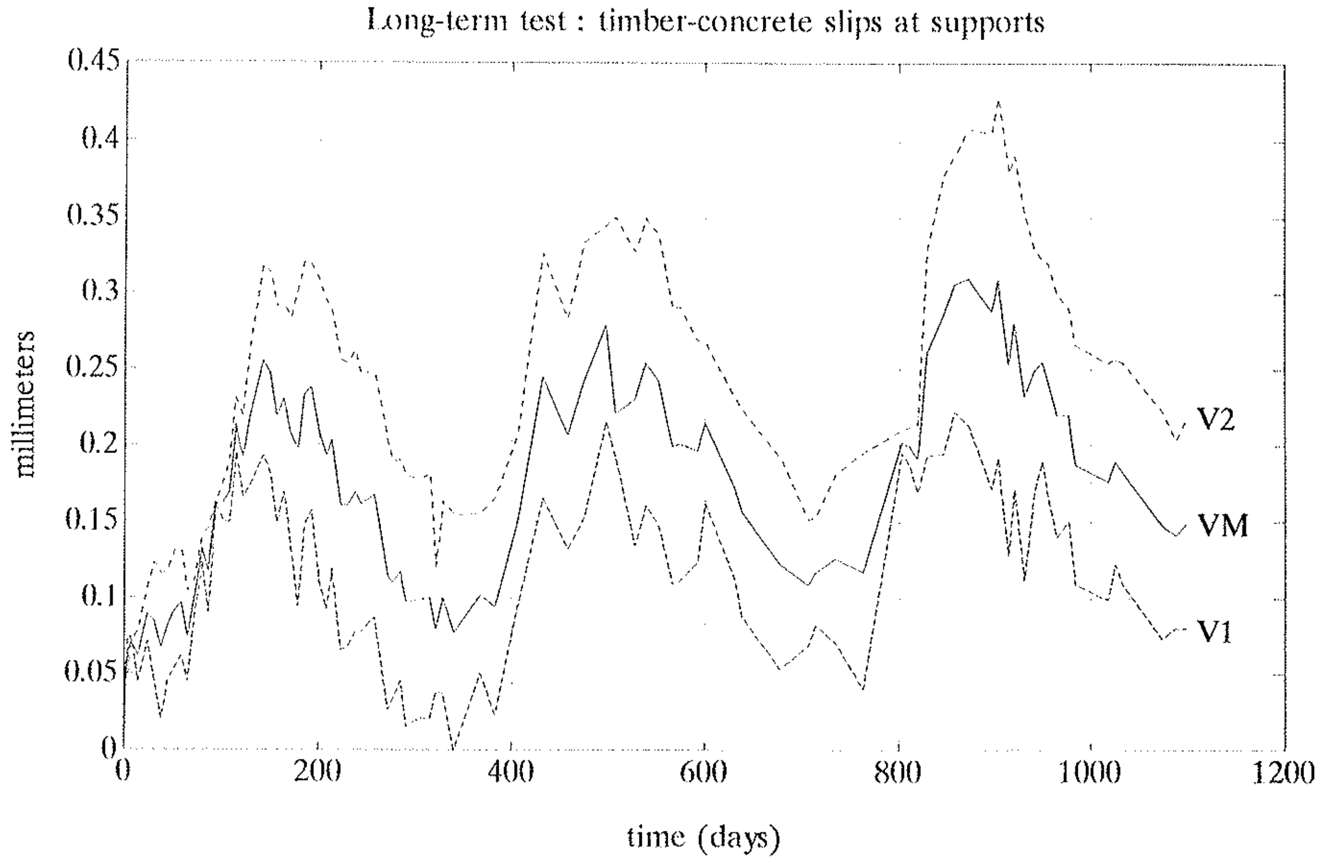

Long-term test : timber-concrete slips at supports
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
millimeters
0
200
400
600
800
1000
1200
time (days)
V2
VM
V1

Fig. 7

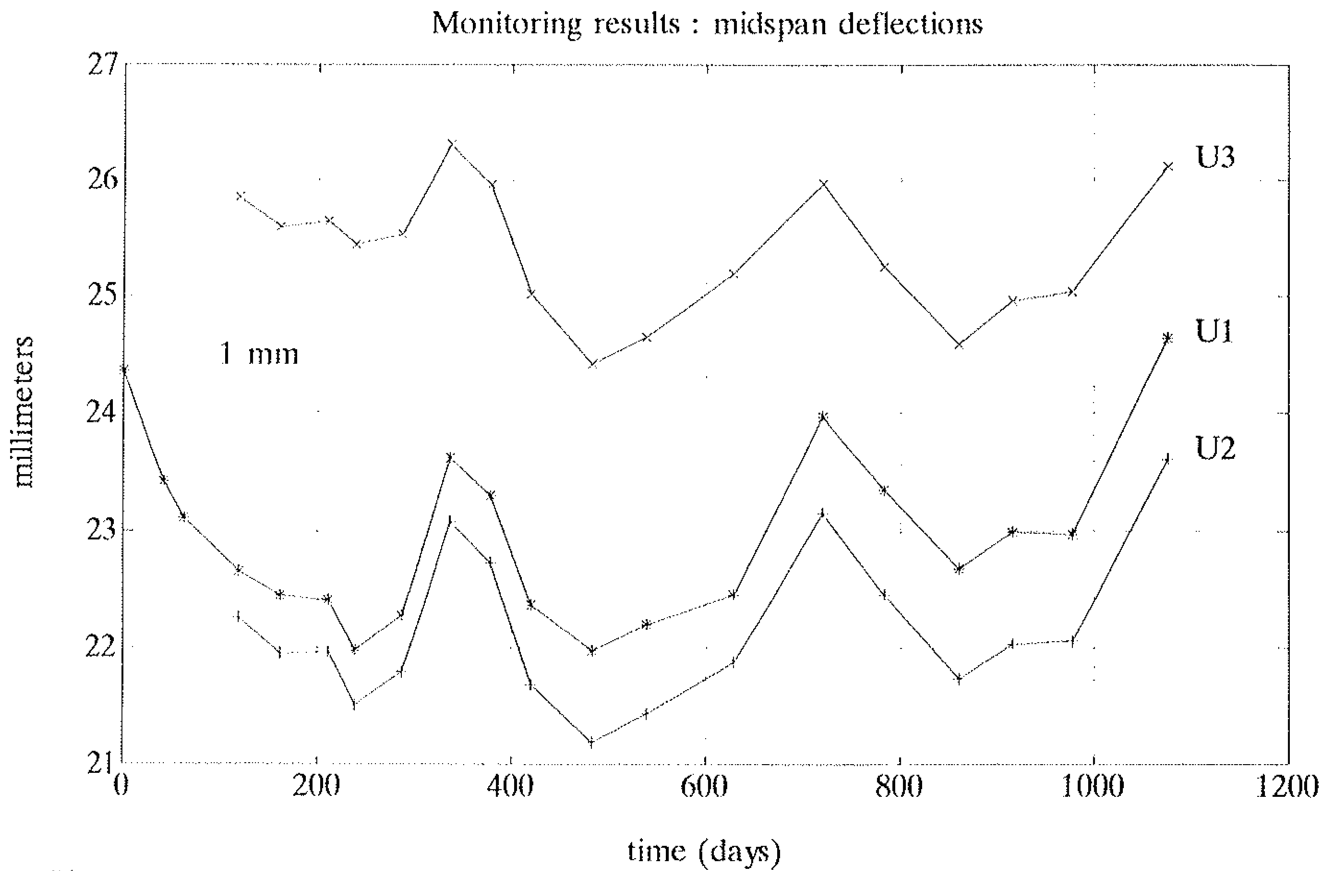
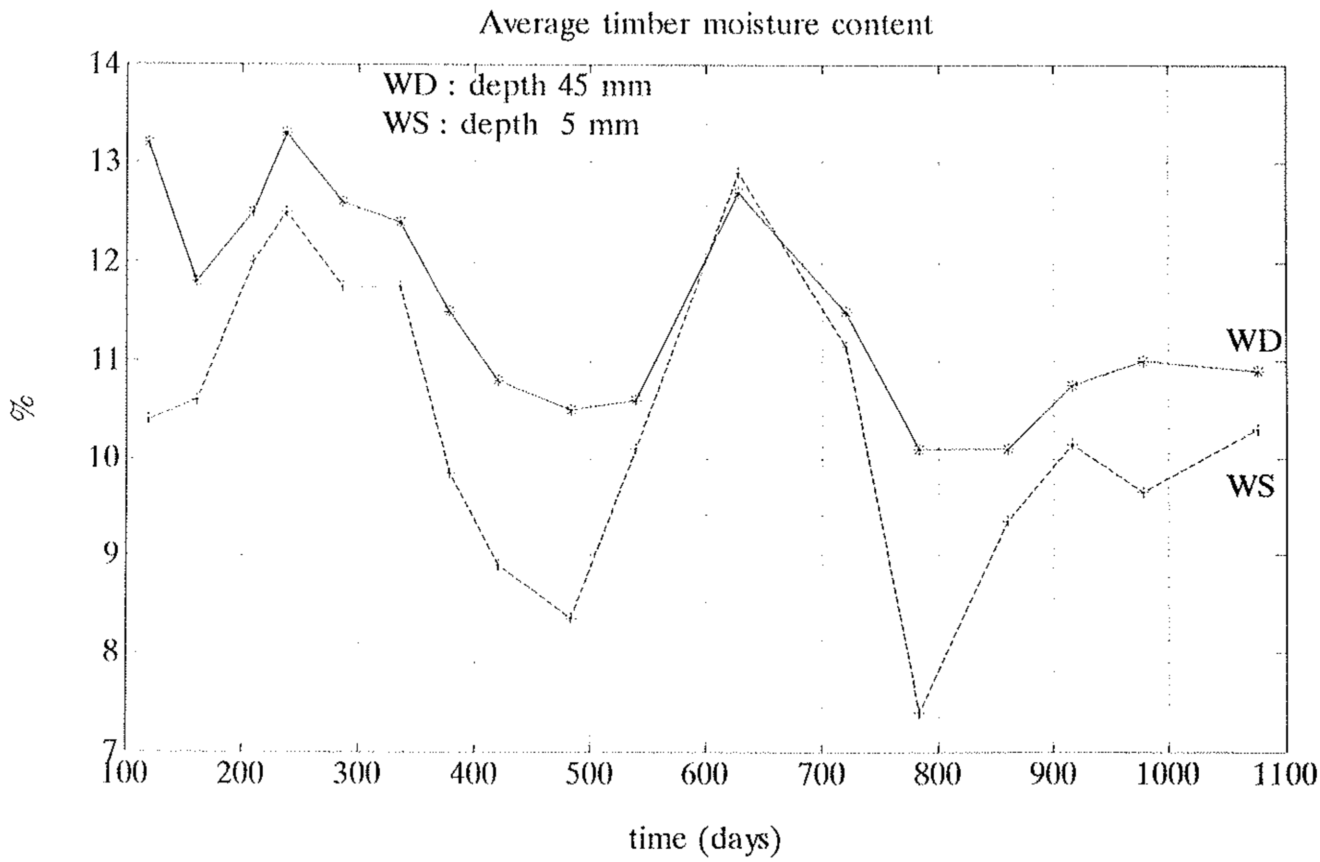
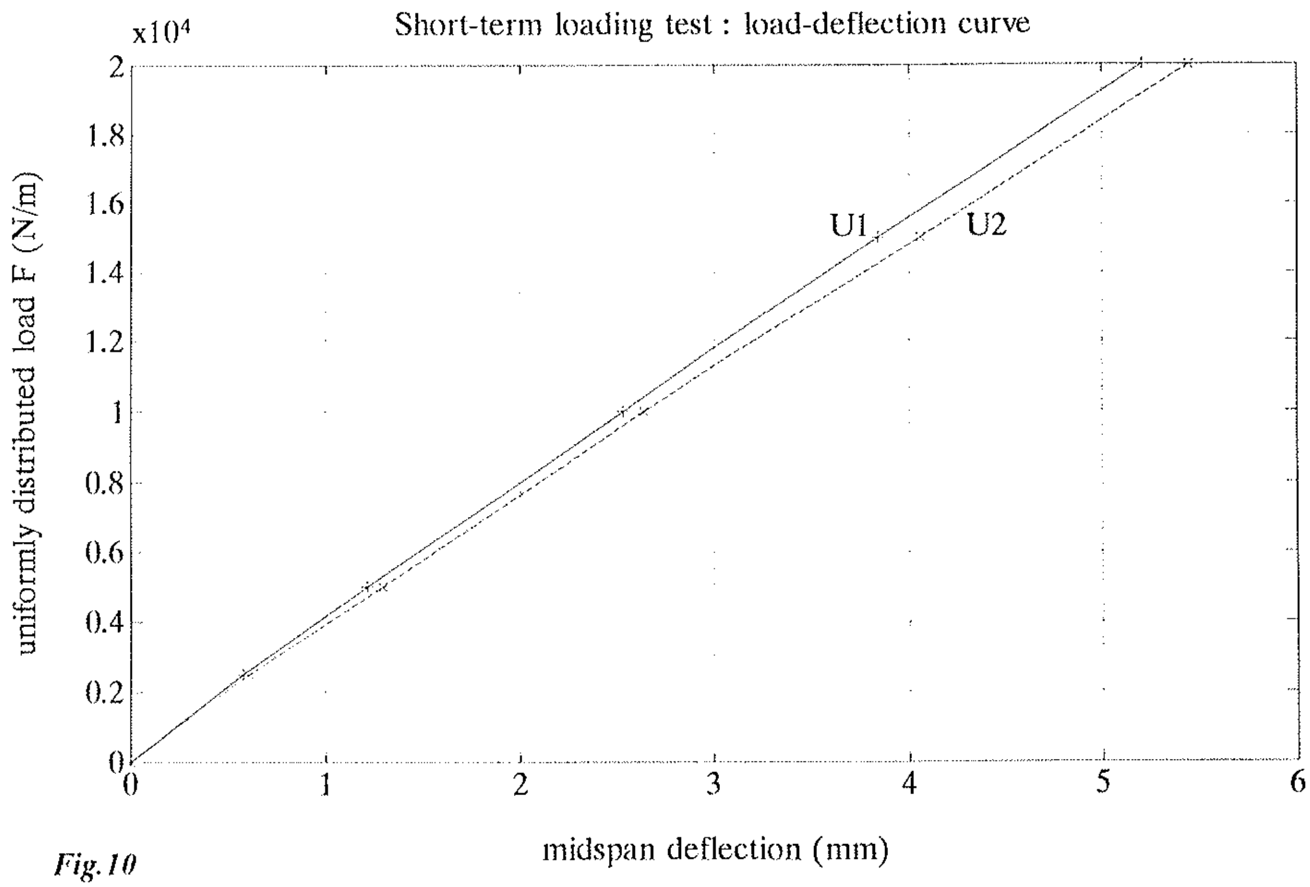

Monitoring results : midspan deflections
27
26
25
24
23
22
21
millimeters
1 mm
0
200
400
600
800
1000
1200
time (days)
U3
U1
U2

Fig. 8



Average timber moisture content
WD : depth 45 mm
WS : depth 5 mm
WD
WS
%
14
13
12
11
10
9
8
7
100
200
300
400
500
600
700
800
900
1000
1100
time (days)

Fig. 9



Short-term loading test : load-deflection curve
x10^4
uniformly distributed load F (N/m)
U1
U2
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
1
2
3
4
5
6
midspan deflection (mm)

Fig. 10

CIB-W18/27-20-3

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

SERVICEABILITY LIMIT STATES : A PROPOSAL FOR UPDATING EUROCODE 5 WITH RESPECT TO EUROCODE 1

by

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MEETING TWENTY - SEVEN

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Introduction.

When drafting Eurocode 5, the code writers had to find a compromise for formulating the load combinations. Eurocode 1 which had to define load values and load combinations were delayed for different reasons. In 1994, National Application documents are being drafted, and Eurocode 1 has been voted, at least for load combinations. This paper intends to present the way to modify Eurocode 5 in order to get compatibility among Structural Eurocodes. Basic examples illustrate this proposal.

Load classification.

In Structural Eurocodes, the assesment of structural safety refers to persistent conditions (normal use), transient conditions (execution), and accidental situations (fire, impact,...). In any case, the actions are classified according to their space variation, and to their time dependancy. In the case of normal use, actions are classified as :

- permanent actions (self weight), for which time variation is neglictible,
- variable actions (occupancy, storage, snow, wind,...) which have a significant time variation.

In the case of variable actions (Hendrickson & al., 1987, Rackwitz, 1976), the influence of time is taken into account by stochastic processes. In the case of imposed loads, the variable action (Q) is usually separated into a sustained part (Q_S) and a transient part (Q_T). For snow loads, the same analysis applies, depending on the geographical site location. Wind actions are calculated from the reference wind velocity, defined as the 10 minutes mean wind velocity 10 m above the ground. The wind actions have to be considered as short term.

In figure 1 is illustrated a stochastic process which represents an imposed load for commercial buildings (1st floor). In figure 2 is illustrated a stochastic process which represents an imposed load for offices. One can see that this kind of process is defined by :

- sustained load : mean and standard deviation of the amplitude, mean and standard variation of the load duration.
- transient load : mean and standard deviation of the amplitude, mean and standard variation of the occurence frequency.

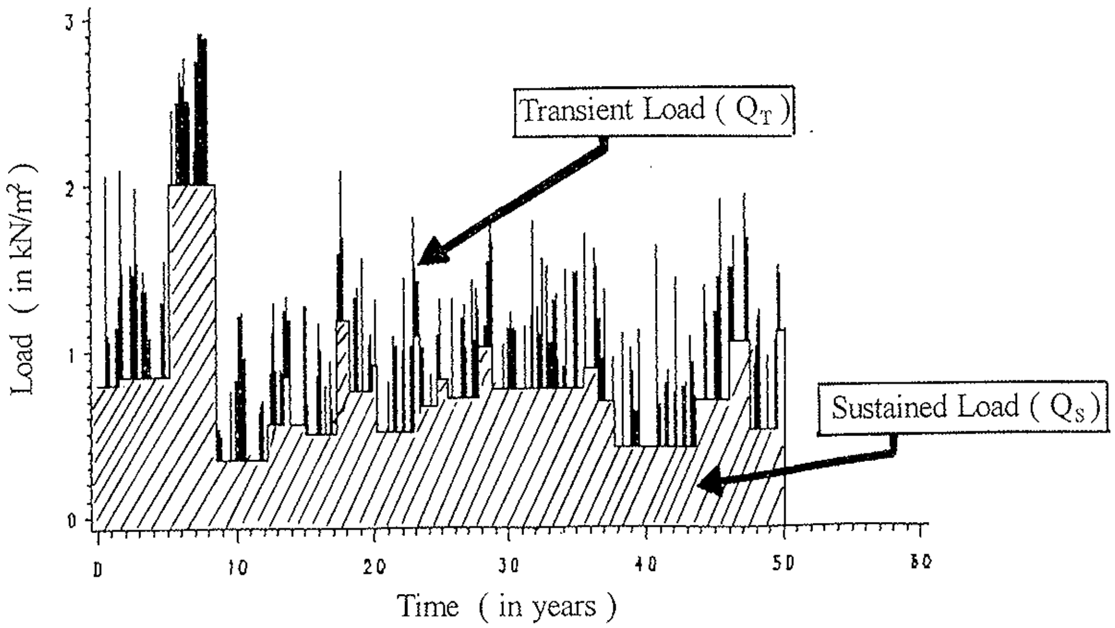

Figure 1 : *Time-dependant imposed load on commercial buildings (1st floor)*

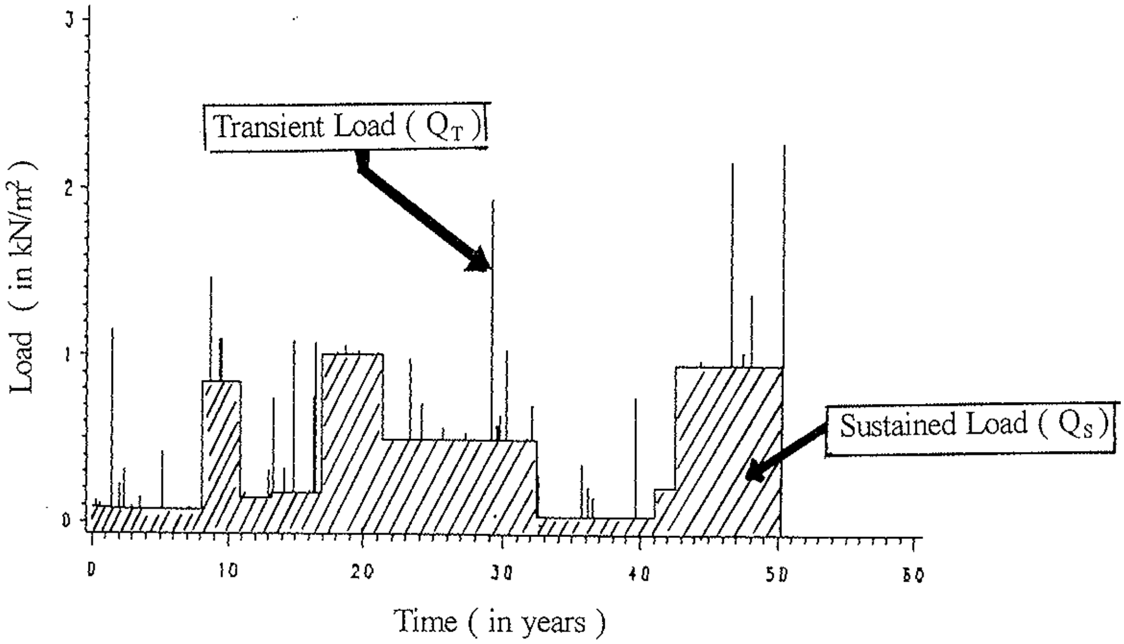

Figure 2 : *Time-dependant imposed load on offices*

When defining a load combination involving variable actions, one needs to define different representative values :

- the characteristic value (Q_k) which has to be used when a given variable action is considered as the most important one.

- the combination value (ψ_0 Q_k) which has to be used when a given action is likely to apply together with other variable actions. The coefficient ψ_0 expresses the probability of a joint occurence of two variable actions. This representative value is used in Eurocode 1 for both ultimate and serviceability limit states (see below).

- the frequent value (ψ_1 Q_k) which has to be used when irreversible behaviors are dependant on the time effect of the action (e.g. fatigue behavior).

- the quasi-permanent value (ψ_2 Q_k) which has to be used when creep behavior is involved. The coefficient ψ_2 gives the correspondance between a variable action and its equivalent so-called "permanent value". It has been calibrated according to the time dependancy of the variable action.

Load combinations.
For both Eurocode 5 and Eurocode 1, ultimate limit states load combinations are defined by :

$$E_d = \sum_j \gamma_{G,j} \, G_{k,j} + \gamma_{Q1} \, Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \, \psi_{0,i} \, Q_{k,i} \qquad (1)$$

The partial coefficients (γ) take into account the influence of the load variabilities on the safety. As explained previously, the most important variable action is taken into account by its characteristic value. The other variable actions are taken into account by their "combination values". The influence of the time dependancy on the material strength is given by :

$$R_d = \frac{R_k \, k_{\text{mod}}}{\gamma_m} \qquad (2)$$

The value of k_{mod} is defined for the shortest load duration class.

For serviceability limit states, the definition differs from Eurocode 1 to Eurocode 5. In Eurocode 1, three load combinations are defined : the characteristic, the frequent and the quasi-permenent combinations. For the design of timber structures, the characteristic load combination has to be used for the calculation of instanteneous deformations :

$$\sum_j G_{k,j} + Q_{k,1} + \sum_{i>1} \Psi_{0,i} \, Q_{k,i} \qquad (3)$$

This load combination is comparable to the one defined for the ultimate limit states, except that the partial coefficients are equal to 1 (for safety reasons). The quasi-permanent load combination has to be used to evaluate the creep part of the deformation. It is defined by :

$$\sum_j G_{k,j} + \sum_{i\geq 1} \Psi_{2,i} \, Q_{k,i} \qquad (4)$$

where, as explained previously, the ψ_2 factors transform variable actions into equivalent "quasi-permanent actions".

In Eurocode 5, the load combination used for serviceability limit states does not correspond to any combination given in Eurocode 1. The ψ factors neither correspond to a specific material behavior. This combination might be considered as a compromise between characteristic load combinations and frequent load combinations, because the ψ_1 factors are comprised between the ψ_0 factors and the ψ_2 factors :

$$\Sigma_j \, G_{k,j} + Q_{k,1} + \Sigma_{i>1} \, \psi_{1,i} \, Q_{k,i} \qquad (5)$$

Serviceability limit states according to Eurocode 1.
According to Eurocode 1, the instantaneous deformation has to be calculated from the characteristic load combination and the creep deformation has to be calculated from the quasi-permanent load combination. This can be formulated as :

$$u_{fin} = u_{inst,(G+Q_1+\psi_{0,2}Q_2)} + \Delta u_{creep,(G+\psi_{2,1}Q_1+\psi_{2,2}Q_2)} \tag{6}$$

The creep deformation (Δu_{creep}) is equal to the final deformation minus the instananeous deformation, both being calculated from the same load combination, i.e. :

$$u_{fin} = u_{inst,(G+Q_1+\psi_{0,2}Q_2)} + u_{fin,(G+\psi_{2,1}Q_1+\psi_{2,2}Q_2)} - u_{inst,(G+\psi_{2,1}Q_1+\psi_{2,2}Q_2)} \tag{7}$$

Under an assumption of linear relationship between the actions and the corresponding deformations, equation (7) can be written :

$$u_{fin} = u_{fin,G} + \left[\psi_{2,1}u_{fin,Q_1} + (1-\psi_{2,1})u_{inst,Q_1}\right] + \left[\psi_{2,2}u_{fin,Q_2} + (\psi_{0,2}-\psi_{2,2})u_{inst,Q_2}\right] \tag{8}$$

or

$$u_{fin} = (1-\psi_{2,1})u_{inst,Q_1} + (\psi_{0,2}-\psi_{2,2})u_{inst,Q_2} + \left[u_{fin,G} + \psi_{2,1}u_{fin,Q_1} + \psi_{2,2}u_{fin,Q_2}\right] \tag{9}$$

In a general case, it is relatively easy to calculate the final deformation, given in (9). The instantaneous parts of the deformation are calculated with E_i (where i is the i^{th} member) and the final parts of the deformation are calculated with :

$$E_{fin,i} = \frac{E_i}{1 + k_{def,i}} \tag{10}$$

It means that only two calculations are necessary, regardless of the number of materials and actions involved. In this approach, the influence of the load duration class is taken into account on the action effects side (E_d) of the design equation :

$$E_d \leq C_d \tag{11}$$

The variation of the creep behavior with material and service class of the structure is taken into account on the material side (C_d). This change leads to a simplified definition of the k_{def} values related only to the permanent part of the action. The simplified values of k_{def} are given in Table 1.

Material	Service class		
	1	2	3
Timber / Glulam	0,6	0,8	2,0
Plywood	0,8	1,0	2,5
...	...	...	...

Table 1 : ***Values of*** k_{def}.

Example : I-beam in tension.

Consider an composite I-beam subjected to tension (see Figure 3). The two materials are given by their MOE's (E_1 & E_2) and by their cross-section (S_1 & S_2). This system can be considered as a parallel system:

$$\begin{cases} F_1 + F_2 = F \\ u_1 = u_2 = u \end{cases} \tag{12}$$

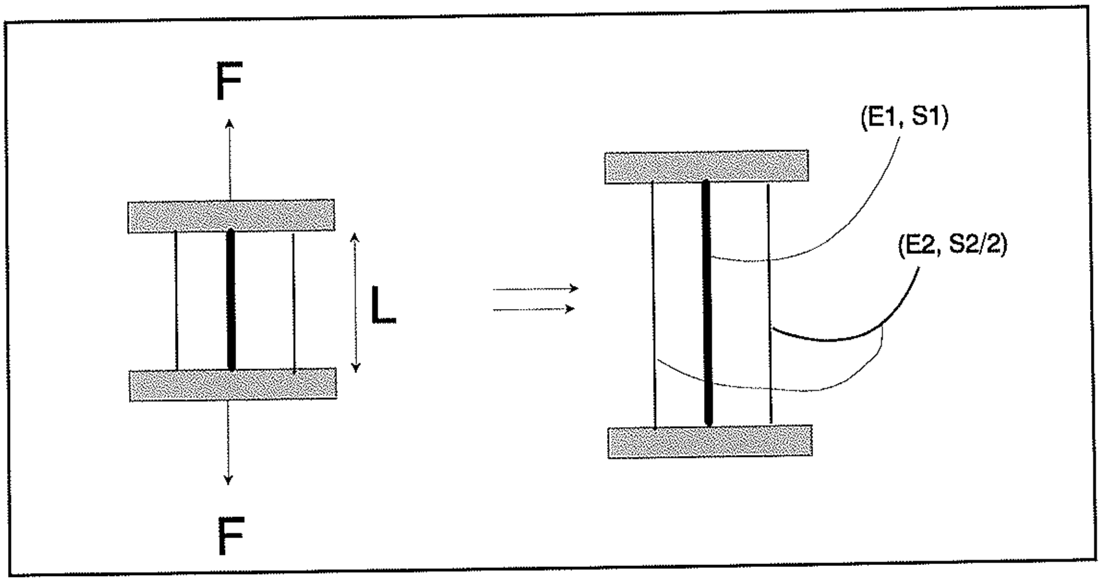

Figure 3 : ***I-beam in tension.***

The deformation (u) under an action (F) is given by :

$$u = F \frac{L}{E_1 S_1 + E_2 S_2} \tag{13}$$

where L is the length of the beam.

To evaluate the final deformation of the system under an action F, we write :

$$\begin{cases} F_1+F_2 = F \\ u_{fin,1} = u_{fin,2} = u_{fin} \end{cases} \tag{14}$$

with

$$\begin{cases} u_{fin,1} = u_{inst,1}\,(1+k_{def,1}) \\ u_{fin,2} = u_{inst,2}\,(1+k_{def,2}) \end{cases} \tag{15}$$

Equations (14)&(15) give :

$$u_{fin} = \frac{F}{\dfrac{E_1S_1}{1+k_{def,1}} + \dfrac{E_2S_2}{1+k_{def,2}}} \tag{16}$$

Thus, the final deformation defined in (9) is given by :

$$u_{fin} = \frac{L}{(ES)_{ef,i}}\left[(1-\psi_{2,1})Q_1+(\psi_{0,2}-\psi_{2,2})Q_2+\frac{1}{\dfrac{R_1}{1+k_{def,1}}+\dfrac{R_2}{1+k_{def,2}}}(G+\psi_{2,1}Q_1+\psi_{2,2}Q_2)\right] \tag{17}$$

with :

$$\begin{cases} R_1 = \dfrac{E_1S_1}{(ES)_{ef,i}} \\ R_2 = \dfrac{E_2S_2}{(ES)_{ef,i}} \end{cases} \tag{18}$$

In the particular case of a uniform section (i.e. $E_1S_1 = 0$), the final deformation is given by :

$$u_{fin} = \frac{L}{(ES)_{ef,i}}[(1-\psi_{2,1})Q_1+(\psi_{0,2}-\psi_{2,2})Q_2+(1+k_{def})(G+\psi_{2,1}Q_1+\psi_{2,2}Q_2)] \tag{19}$$

This example exhibits how to calculate the final deformation under a load combination for an I beam subjected to tension.

Serviceability limit states according to Eurocode 5.
According to equation (5), the final deformation is calculated from :

$$u_{fin} = u_{fin,G+Q_1+\psi_{1,2}Q_2} \tag{20}$$

Under the assumption of a linear relationship between the actions and the deformations, equation (19) becomes :

$$u_{fin} = u_{fin,G} + u_{fin,Q_1} + u_{fin,\psi_{1,2}Q_2} \tag{21}$$

In the case of an composite I-beam subjected to tension (see above), it becomes :

$$u_{fin} = \frac{L}{(ES)_{ef,i}}\left[\frac{G}{\frac{R_1}{1+k_{def,1,G}}+\frac{R_2}{1+k_{def,2,G}}}+\frac{Q_1}{\frac{R_1}{1+k_{def,1,Q_1}}+\frac{R_2}{1+k_{def,2,Q_1}}}+\frac{\psi_{1,2}Q_2}{\frac{R_1}{1+k_{def,1,Q_2}}+\frac{R_2}{1+k_{def,2,Q_2}}}\right] \tag{22}$$

This equation requires the use of six deformation factors, compared to two for Eurocode 1.

In the case of an homogeneous cross-section, equation (21) becomes :

$$u_{fin} = \frac{L}{(ES)_{ef,i}}\left[(1+k_{def,G})G+(1+k_{def,Q_1})Q_1+(1+k_{def,Q_2})\psi_{1,2}Q_2\right] \tag{23}$$

In a more general case (i.e. multiple members with multiple actions), it means that the final deformation calculation requires as many calculations as the number of load duration classes.

Design consequences.
Besides the simplifications of the calculation which have been explained previously, we carried out a numerical example (one material, one dead load, one live load) according to equations (17) and (22). A global creep factor (ratio between the final deformation and the equivalent instananeous deformation) has been evaluated for both Eurocode 1 and Eurocode 5

$$\theta_{(EC1 \vee EC5)} = \frac{u_{fin,(EC1 \vee EC5)}\ (ES)_{ef,i}}{L\ (G+Q)} \tag{24}$$

The ratio between these creep factors is given in Figure 4, for different dead to live loads ratios. It has been calculated for storage and for office, for solid timber (ST) and for OSB. For performing the calculation, the ψ factors (given in Table 2) and the deformation factors (given in Table 3) are necessary.

Variable action	ψ_0	ψ_1	ψ_2
Occupancy - Offices	0,7	0,5	0,3
Storage	1,0	0,9	0,8

Table 2 : ψ ***factors***

Load Duration class	Solid Timber Glulam	OSB	Action
Permanent	0,6	1,5	Self-weight
Long Term	0,5	1	Storage
Medium Term	0,25	0,5	Occupancy

Table 3 : ***Deformation factors***

The difference between these two codes ranges from -7% to +5%. The only case which is unfavorable for Eurocode 1 is OSB used for storage, which is of minor use.

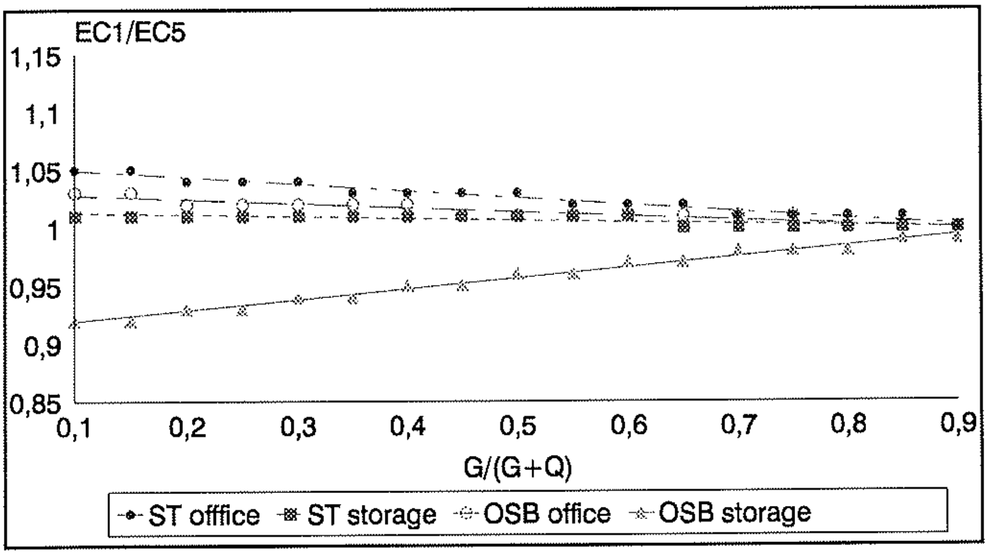

Figure 4 : ***Eurocode 1 creep factors compared to Eurocode 5.***

Conclusion.

Despite an apparent complication from the basic load combinations, Eurocode 1 serviceability limit states definitions are much more easy to undertake in a practical calculation. The numerical examples given in this paper show that a use of these formulas will not imply a increased timber consumption. Therefore, the authors propose to incorporate these concepts in the final version of Eurocode 5, which would simplify also the table of deformation factors (one line per material). An other advantage is also that timber calculations would be performed exactly in the same way as the steel or concrete calculations.

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CIB-W18/27-20-4

INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING COMMISSION W18 - TIMBER STRUCTURES

CREEP BEHAVIOR OF TIMBER UNDER EXTERNAL CONDITIONS

by

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Introduction.
In 1990, Rouger & al. published a paper describing full scale experiments which had been initiated in order to give a basis for deformation factors given in Eurocode 5. The data corresponding to service classes 1 and 2 were fitted to a power law model, which allowed to predict the creep behavior up to 50 years. This paper presents the work which has been carried out on service class 3, requiring a specific modelling to take into account the mechano-sorptive effects. A proposal for numerical values to be incorporated in Eurocode 5 is given.

Full scale creep experiments.
A series of long term four points bending tests have been carried out in CEBTP (Centre Expérimental de Recherches et d'Etudes du Bâtiment et des Travaux Publics), in the frame of a research program involving CTBA and CEBTP, funded by the French Ministary of Research and by the AFME (Agence Française pour la Maîtrise de l'Energie). Three different load levels, corresponding to 2 MPa, 5 MPa and 15 MPa were investigated. Altogether, 75 specimens have been tested in external conditions :

- specimens of glulam, made from Spruce :

  L = 7,2 m   h = 400 mm   b = 45 or 100 mm

- specimens of solid timber (Spruce, Scots pine, Douglas Fir, Poplar)

  L =1,8 m   h = 100 mm   b = 40 mm

The specimens were sampled in order to get two qualities, corresponding to two visual grades :

C30 --> GL30 --> Category I
C22 --> GL25 --> Category II

The numbers of specimens within each strength class and stress level is given in Table 1.

Table 1 : *Sampling.*

	Solid Timber		Glulam	
Stress Level	C22	C30	GL25	GL30
2 MPa	0	3	0	3
5 MPa	12	12	3	9
15 MPa	12	12	3	6

The data which is analysed below corresponds to 800 days of creep (2,5 years). During this period, the relative humidity environment has been daily recorded.

Duration of load effects.
During this experiment, some specimens failed. In order to explain these failures, the calculation of the design strengths is necessary. According to Eurocode 5, the design strength is equal to :

$$X_d = k_h \, k_{\text{mod}} \, \frac{X_k}{\gamma_M} \tag{1}$$

where k_h is a size factor, k_{mod} is a modification that takes into account the duration of load effect on the strength, and γ_M is partial coefficient which reflects the required safety of an element. In this study, we have :

$k_h = 1,08$ $k_{mod} = 0,55$ (Service Class 3, Long Term) $\gamma_M = 1,3$

An additional requirement on the lateral instability has to be met :

$$\sigma_{m,d} \leq k_{inst} \, f_{m,d} \tag{2}$$

where k_{inst} depends on the value of the relative slenderness, given by :

$$\lambda_{rel,m} = \sqrt{\frac{L_{ef}}{\pi} \, \frac{h}{b^2} \, \frac{f_{m,k}}{E_{0,k}} \sqrt{16}} \tag{3}$$

The effective length, L_{ef}, depends on the supports and on the loading conditions. In this study, we took the most unfavorable case (i.e. $L_{ef} = L$). It should be mentionned that the relative slenderness does not take into account the time effect. It is assumed that the ratio $f_{m,k}/E_{0,k}$ does not vary with time, which is not true, and that the creep deflections do not have a second order effect on the lateral instability. The long term relative slenderness might be deducted from the short term one according to the following equation :

$$\lambda_{rel,m}(LongTerm) = \lambda_{rel,m}(ShortTerm) \, \sqrt{(1 + k_{def}) \, k_{\text{mod}}} \tag{4}$$

As described later in this paper, this influence might be significant. Therefore, the calculations of the design strengths given in Table 2 take into account the time dependance.

Table 2 : ***Design strengths including lateral instability.***

	Strength class	$f_{m,k}$ (in MPa)	$f_{m,d}$ (in MPa)	$\lambda_{rel,m}$		k_{inst}		k_{inst} $f_{m,d}$	
				S.T.*	L.T.**	S.T.*	L.T.**	S.T.*	L.T.**
Solid Timber	C22	22	10	0,69	0,81	1,0	0,95	10	9,5
	C30	30	13,7	0,73	0,85	1,0	0,92	13,7	12,6
Glulam	GL25	25	11,5	1,04	1,21	0,78	0,65	9,0	7,5
	GL30	30	13,7	1,05	1,23	0,77	0,64	10,5	8,8

* Short Term ** Long Term

It can be noticed that these design strengths are larger than 2 and 5 MPa, but lower than 15 MPa. This can explain the failure events, described in Table 3, which occured after one year.

Table 3 : ***Failure events.***			
	Strength class	Failures including lateral instabilities	
		Number of failures	Pf
Solid Timber	C22	6/12	50%
	C30	6/12	50%
Glulam	GL25	3/3	100%
	GL30	6/6	100%

Let us do a simplistic calculation, to see if these probabilities of failure are realistic. The limit state function associated to equation (2) is given by :

$$G = k_{\mathrm{mod}}\, k_{inst}\, k_h\, f_m - S = R - S \qquad (5)$$

where f_m is the bending strength (random variable),
S is the design stress (deterministic)

The approximate value of the associated safety index is equal to :

$$\beta = \frac{\mu_R - S}{\sigma_R} \qquad (6)$$

The calculations of the equivalent probabilities of failure are given in Table 4.

Table 4 : ***Safety analysis.***								
	Strength class	$f_{m,k}$	COV	$f_{m,50}$	Safety analysis (S.T.)		Safety analysis (L.T.)	
					β	P_f	β	P_f
Solid Timber	C22	22	0,3	37,8	1,10	14%	1,0	16%
	C30	30	0,3	51,5	1,70	5%	1,55	6%
Glulam	GL25	25	0,2	35,5	0,44	33%	-0,47	68%
	GL30	30	0,2	42,6	1,15	13%	0,37	36%

It can be noticed that the probabilities of failure calculated from the short term assumption are much lower than the observed ones. The long term assumption gives closer results, but still different from the experiments. This could be due to the following reasons :

- Second order effects have not been taken into account,
- Values of k_{mod} too low,
- Number of specimens too small,
- Deviations from straightness larger than required.

Creep Modelling.

The data has been analysed by using a numerical model developed by T. Toratti (1992), which combines a diffusion method, plus a modelling of creep including mechano-sorptive effects.

(1) A finite difference method is used to compute the *transient moisture content distribution* in the cross-section of the wood. The differential equation to be solved is given by :

$$\frac{du}{dt} = \frac{d(D\frac{du}{dx})}{dx} + \frac{d(D\frac{du}{dy})}{dy} \quad \textbf{(7)}$$

where u is the moisture content distribution and D is the diffusion coefficient.

A constant surface resistance of moisture movement between the air and wood surface is used to evaluate the boundary conditions :

$$g_u = S\ (\ u_{RH} - u_{surf}\ ) \quad \textbf{(8)}$$

where g_u is the moisture flow (calculated from the previous time step)
S is the surface emissivity
u_{RH} is the equilibrium moisture concentration of wood in the surrounding air (a function of the relative humidity environment)
u_{surf} is the moisture concentration at the wood surface (which gives boundary conditions).

(2) The *creep analysis* is undertaken by the method of "successive approximations". Within each time step, the following iterative scheme is computed :

- a) Compute the moisture content distribution u(x, y)
- b) Update material properties according to moisture changes
  - c) Compute stress distribution (if first iteration, value of the previous step)
    - d) Compute strain increment (due to time + moisture content + current stress level)
    - e) Compute the stress increment by inverting the constitutive equations
    - f) If change of stress increment > 0,1%, go back d)
  - g) Compute section forces
  - h) If equilibrium is verified, go to (j)
  - i) If not, adjust the strain according to the difference between the internal and external loads; go back d)
- j) Next Time step

(3) The strain increments are given by :

$$\Delta\epsilon = \Delta\epsilon_E + \Delta\epsilon_c + \Delta\epsilon_{ms} + \Delta\epsilon_u \quad \textbf{(9)}$$

where $\Delta\varepsilon_E$ is the elastic strain increment,
$\Delta\varepsilon_c$ is the normal creep strain increment,
$\Delta\varepsilon_{ms}$ is the mechano-sorptive strain increment,
$\Delta\varepsilon_u$ is the shrinkage strain increment,

The *elastic strain increment* is given by :

$$\Delta\epsilon_E = J_0 \, \Delta\sigma + \Delta J_0 \, \sigma \tag{10}$$

where J_0 is the elastic compliance
ΔJ_0 is the increment of the elastic compliance due to moisture content changes

The *normal creep strain increment* is given by a power law, which has been converted to a six Kelvin series, in order to simplify the computation.

The *mechano-sorptive creep including shrinkage* is a so-called "creep limit" model. It assumes that the irreversible strain (with respect to moisture changes) tends to reach a limit. It also assumes that the shrinkage strain decreases with the actual strain level. These strains are given by :

$$\epsilon_{ms}(t) = J^{\infty} \int_0^t \left\{ 1 - \exp\left(-c \left(\int_{t'}^{t} |du(t'')| \, dt''\right)\right\} dt' \tag{11}$$

and

$$\epsilon_u(t) = \int_0^t \{\alpha - b \, \epsilon(t')\} \, du(t') \tag{12}$$

The different components of the strain are illustrated in Figure 1.

The model parameters have been adjusted to clear wood specimens data, tested in a controlled cycling humidity environment.

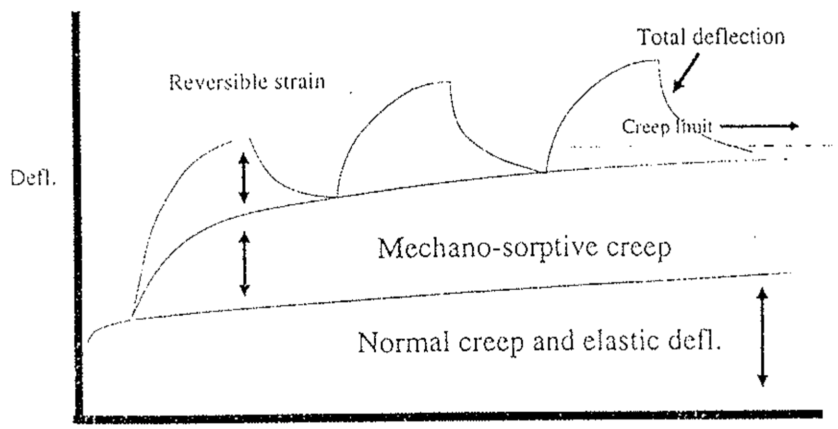

Figure 1 : *Creep Limit Model (after Toratti, 1992)*

Test data adjustements.
The daily measured humidity has been averaged on a 10 days and on a monthly period. These curves are represented in Figure 2. They were used to evaluate the moisture content distributions in the cross-sections of the full size specimens (see Figure 3). Because this model is restricted to a linear behavior with respect to the stress level, the model was fitted to low stress levels (2 and 5 MPa). The relative deformations of the specimens are represented in Figure 4 (Solid timber) and in Figure 5 (Glulam), as compared to the model predictions. It leads to the following remarks :

(1) The model gives in any case a very good prediction of the creep limit.

(2) It underestimates the shrinkage strains. This might be due to an initial guess for the moisture content distribution in the cross-sections of the pieces which was to crude (average moisture content). This might also be due to the fact that clear wood specimens were used to adjust the shrinkage parameters.

(3) The model has not been succesfully applied to low quality solid timber. This might be due to a non linear behavior (stress ratio of 20% to 50%). This might be due to an increased heteregoneity in this material which limits the validity of the model, which has been developed for clear wood.

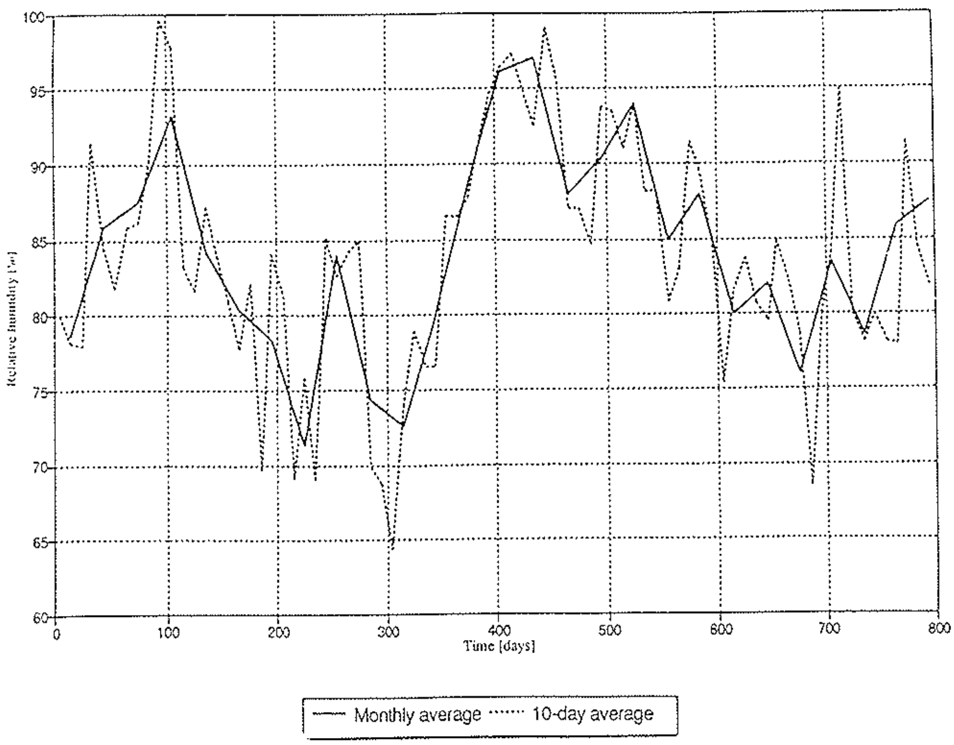

Figure 2 : ***Relative Humidity environment.***

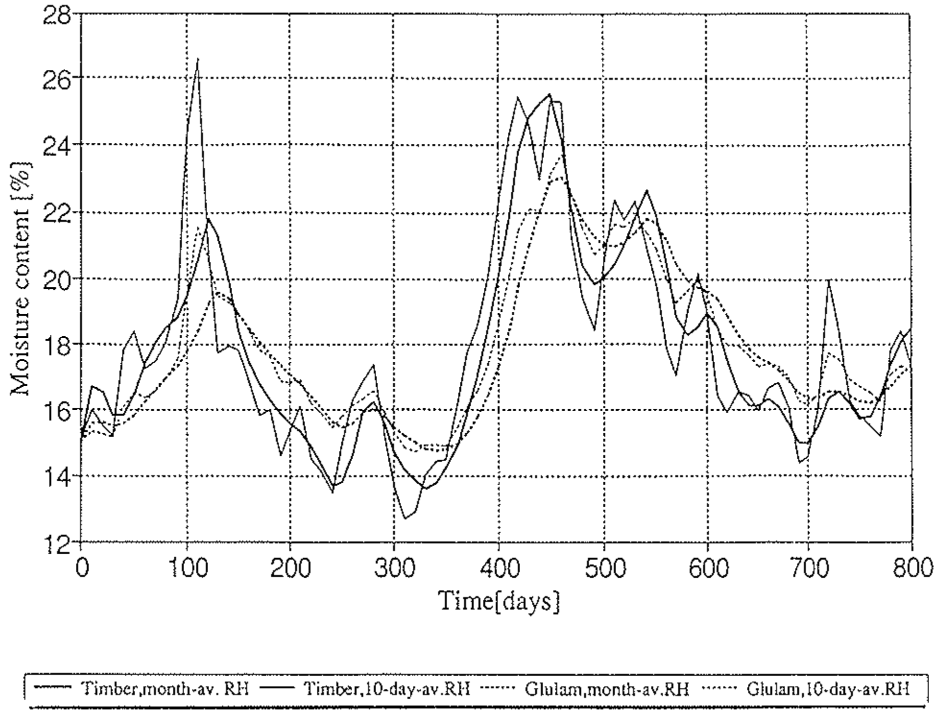

Figure 3 : *Average moisture content distributions in the test pieces*

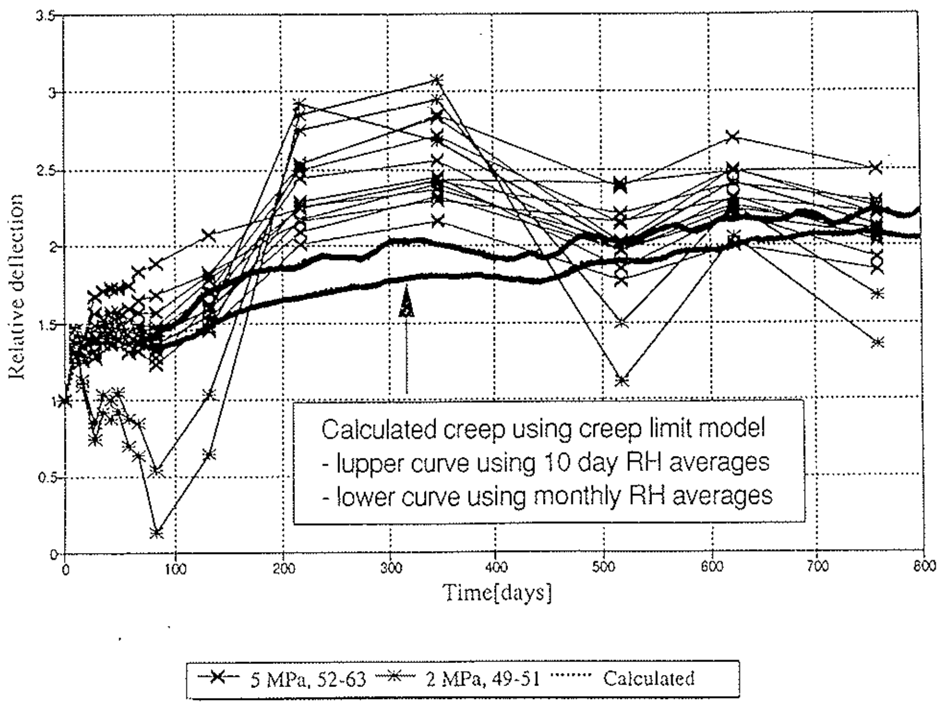

Figure 4 : *Relative deflections of solid timber (2 and 5 MPa).*

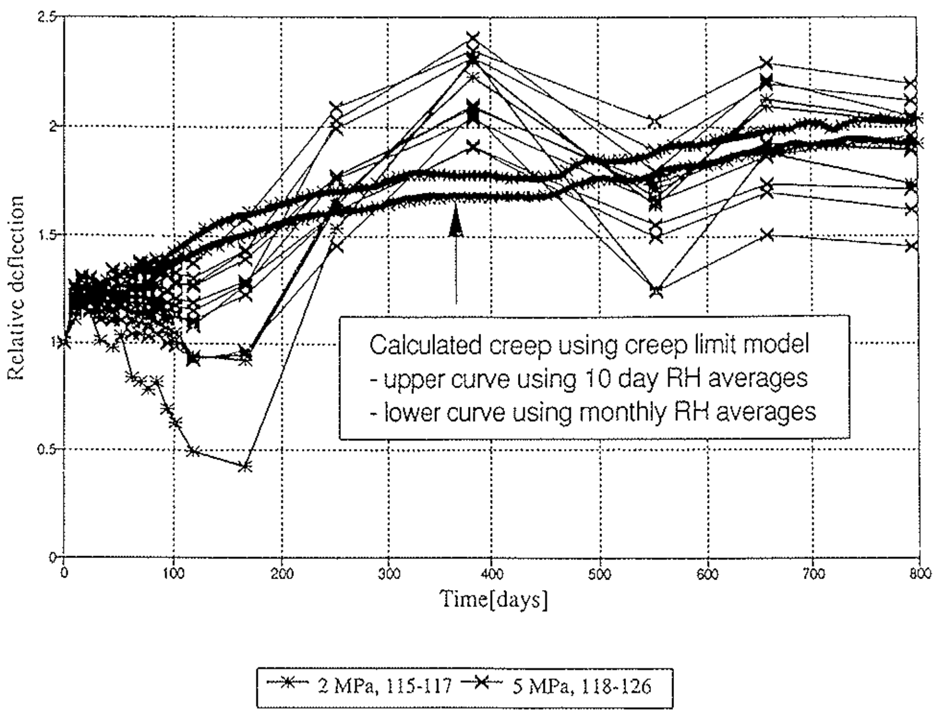

Figure 5 : ***Relative deflections of glulam (2 and 5 MPa)***

Proposals for Eurocode 5.
Based on the modelling explained above, the k_{def} values have been evaluated for a medium term load (6 months) and for a permanent load (50 years), in service class 3. These values have been compared to service classes 1 & 2 values, which have been calculated from the model given in Rouger & al. (1990). These results are compared to Eurocode 5 prescribed values in Table 5.

Table 5 : ***Deformation factors given from the modelling, as compared to EC5 values.***

Service class		1		2		3	
Material	Load duration class	Model	EC5	Model	EC5	Model	EC5
Solid timber	Medium term	0,25	0,25	0,35	0,25	1,4	0,75
	Permanent	0,9	0,6	1,3	0,8	1,7	2,0
Glulam	Medium term	0,35	0,25	0,38	0,25	0,7	0,75
	Permanent	1,4	0,6	1,25	0,8	1,7	2,0

They lead to the following proposal :

(1) In service class 1, the k_{def} values should be :

0,3	for medium term	(instead of 0,25)
1,1	***for permanent***	***(instead of 0,6)***

(2) In service class 2, the k_{def} values should be :

0,4	for medium term	(instead of 0,25)
1,3	***for permanent***	***(instead of 0,8)***

(1) In service class 3, the k_{def} values should be :

1,0	for medium term	(instead of 0,75)
1,7	***for permanent***	***(instead of 2,0)***

Eurocode 5 tends to underestimate deformation factors for constant climatic conditions, and to overestimate them for varying conditions. However, the proposed value of 1,7 has to be discussed, since the shrinkage is underestimated by the model. Based on Racher & Rouger (1994), the modified table of Eurocode 5 should only contain the permanent values.

Conclusion.
When investigating the creep behavior under varying conditions, a number of failures have been recorded. They justify a probabilistic study to check the probabilities of failures. The creep limit concept has been shown to be well appliable to test results, despite it was adjusted to clear wood specimens. The only part of the deformation which has been underestimated is the shrinkage deformation. The design proposal will tend to decrease the influence of moisture content on creep, comparing it to the current version of EC5.

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