3 STRUCTURES AND STRUCTURAL MEMBERS

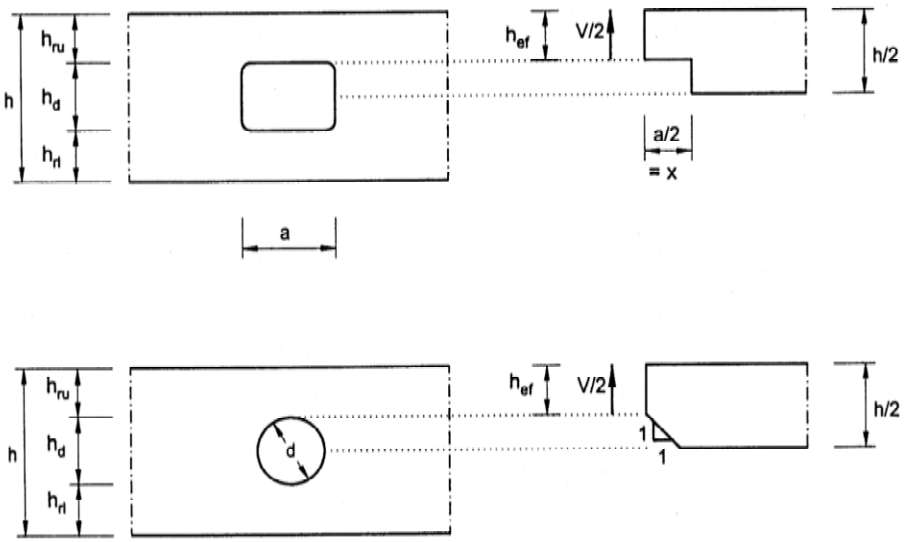
CONTENT

3.1 BEAMS WITH HOLES

ESSAY 3.1 H J Larsen
Stresses around holes in beams

The placement of holes in glulam beams represents a frequent necessity in timber construction practice in order to enable the penetration of pipes, wirings and heating tubes. The disturbance of the stress flow around a hole creates tension stresses perpendicular to grain which may reduce the load bearing capacity of beams with unreinforced holes considerably. In general a hole reinforcement is inevitable to provide sufficient shear force capacity. Invisible internal reinforcements, such as glued-in steel rods and screws, especially self-tapping screws, are very often preferable from architectural points of view.

Designers however want to have design rules. In a draft for Eurocode from 2002 there was a proposal for design rules. It was based on the assumption that the load-carrying was the same as for a corresponding notched beam, see the figure.



Dimensions of holes in beams and the notched beam approximations for rectangular and round holes.

The analogy seams obvious, however tests has shown that the method is very much on the unsafe side and it is not found in the final (2004) Eurocode 5.

Another method was put forward in a draft (2000) for the German Timber Design Code: DIN 1052. The method is below described for a round hole (the design for rectangular holes is in principle the same). The method is a classical strength of material approach: The design tension force perpendicular to the grain at the hole periphery, $F_{t,90,d}$ is compared to the design value of the resistance $R_{t,90,d}$ (not specified directly)

$$\frac{F_{t,90,d}}{R_{t,90,d}} = \frac{F_{t,90,d}}{0,5 l_{t,90} b f_{t,90,d}} \leq 1 \qquad (1)$$

where (in case of round holes)

$$l_{t,90} = 0,353 h_d + 0,5h \qquad (2)$$

is the distribution length of the assumed triangular stress distribution perpendicular to grain. b is beam width and $f_{t,90,d}$ is the design tension strength perpendicular to grain, see the figure. Rewritten as the ratio of a design stress $\sigma_{t,90,d}$ versus design strength, (1) reads:

$$\frac{\sigma_{t,90,d}}{f_{t,90,d}} \leq 1 \qquad (1b)$$

where

$$\sigma_{t,90,d} = \frac{F_{t,90,d}}{0,5 l_{t,90} b} \qquad (3)$$

The design value of the tension force is composed of two additive parts bound to the separate actions of the shear force and the bending moment

$$F_{t,90,d} = F_{t,V,d} + F_{t,M,d} \qquad (4)$$

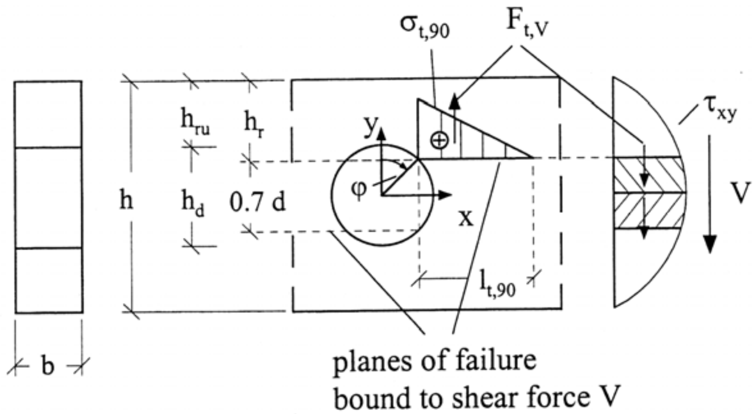
where (in the case of round holes)

$$F_{t,V,d} = \frac{V_d \cdot 0,7 h_d}{4h}\left[3 - \frac{(0,7 h_d)^2}{h^2}\right] = \eta V_d \qquad (5)$$

$$F_{t,M,d} = 0,008 \frac{M_d}{h_r} \qquad (6)$$

V_d and M_d are the absolute values of design shear force and moment at the hole edge (hole centre $\pm d/2$): sign of $\pm d/2$ to be chosen so as to give unfavourable results and

$$h_r = \min\{h_{ru} + 0{,}15h_d, h_{rl} + 0{,}15h_d\}^2 \tag{7}$$



Geometry notations of a round hole in glulam beam according to DIN 1052 and schematic illustration of the derivation of the tension force Ft,v bound to shear force V.

Further, the maximum/minimum restrictions
$h_d \le 0{,}4h$ and $h_{ro(ru)} \ge 0{,}25h$ apply.

However, the tests reported, support neither the DIN nor the Eurocode methods and they have both been withdrawn. For the time being there is no recognized design method and it is generally necessary to avoid the problem by using a reinforcement at the hole edges.

There is, however in **Paper 42-12-1** a new draft for a design method. It is copied below.

Design and construction rules for internally reinforced holes according to DIN 1052

NB Skannet tekst er endnu ikke indsat pænt

Figure 6 gives the geometry notations used in the present German timber design code DIN 1052:2008-10. In detail Fig. 6a specifies the dimensions of the hole within the beam and indicates the relevant crack planes, similarly relevant for unreinforced and reinforced holes. Figure 6 b gives the dimensional notations of the internal reinforcement rods and the respective (edge) distances. The following construction rules/limits for the permissible position and sizes of round holes (see Fig. 6a) apply:

- $\ell_v \ge h$
- $\ell_A \ge h/2$
- $h_{ro(ru)} \ge h/4$
- $h_d \le 0{,}3\ h$ (internal reinforcements, i. e screws, glued-in rods)
- $h_d \le 0{,}4\ h$ (external reinforcements, i. g. glued-on plywood plates)

For the position of the reinforcement bars (d_r = nominal /outer diameter) the following (edge) distances are prescribed:

- $a_{1,c}$: $2{,}5\ d_r \le a_{1,c} \le 4\ d_r$
- a_2: $a_2 \ge 3\ d_r$
- $a_{2,c}$: $a_{2,c} \ge 2{,}5\ d_r$

The design verification of the rod is given as

$$F_{t,90,d} \le R_{ax,d} \tag{1}$$

where

$$F_{t,90,d} = F_{t,90,k} \cdot \gamma_L = F_{t,V,d} + F_{t,M,d} \quad \left\{ \begin{array}{l} \text{design (d) resp. characteristic (k) ten-} \\ \text{sion force perpendicular to grain due to} \\ \text{stress disturbance in the hole vicinity} \end{array} \right. \tag{2a}$$

and

$$F_{t,V,d} = \frac{V_d \cdot 0{,}7h_d}{4 \cdot h}\left[3 - \frac{(0{,}7\ h_d)^2}{h^2}\right], \quad F_{t,M,d} = 0{,}008\ \frac{M_d}{h_r} \tag{2b,c}$$

$$h_r = \min\{h_{ro} + 0{,}15\ h_d\ ; h_{ru} + 0{,}15h_d\} \tag{3}$$

k_{mod} — modification factor for accumulated load duration and service class;

γ_L, γ_M — partial safety factors for loading and materials

And on the resistance side, in case of screws

$$R_{ax,d} = R_{ax,k} \cdot (k_{mod} / \gamma_M) \qquad \begin{cases} \text{axial design (d) resp. characteristic} \\ \text{(k) tension capacity of the rod} \end{cases} \tag{4a}$$

In case of (self-tapping) screws the characteristic axial capacity at 90 degrees vs. fiber direction is (head pull through situation not regarded here)

$$R_{ax,k} = \min\left\{f_{1,k} \cdot \ell_{ad} \cdot d_r \,;\, R_{t,u,k}\right\} \tag{4b}$$

and (ρ_k in kg/m^3)

$$f_{1,k} = 80 \cdot 10^{-6} \cdot \rho_k^2 \quad \text{in N/mm}^2 \qquad \begin{cases} \text{characteristic value of pull out parameter} \\ \text{of wood screw (load capacity class 3) or} \\ \text{of self tapping screw acc. to Z-9.1-519} \end{cases} \tag{4c}$$

$R_{t,u,k}$ characteristic axial steel tension load capacity

In case of glued-in rods the design capacity is

$$R_{ax,d} = \min\ \{\pi \cdot d_r \cdot \ell_{ad} \cdot f_{k1,d}\,;\ f_{y,d} \cdot A_{ef}\} \tag{5a}$$

where

$$f_{k1,d} = f_{k1,k} \cdot (k_{mod}\ \gamma_M) \qquad \begin{cases} \text{design (d) resp. characteristic (k) bond strength} \\ \text{between rod and timber (values for } f_{k1,k} \text{ given in} \\ \text{Tab. F. 23 of DIN 1052:2008, see below)} \end{cases} \tag{5b}$$

$f_{y,d} = f_{y,k} / \gamma_M$ design (d) resp. characteristic (k) yield stress of rod

The effective anchorage length ℓ_{ad} of the rod, accounting for the possible slight eccentricity of the hole vs. mid-depth and the respective minimum value are

$$\ell_{ad} = h_{ru} + 0{,}15\ h_d \quad \text{or} \quad \ell_{ad} = h_{ro} + 0{,}15\ h_d, \quad \ell_{ad,min} = \max\left\{0{,}5 d_r^2 \,;\, 10 d_r\right\} \tag{5c,d}$$

The basic idea of the design equations (1) and (2a-c) is sketched schematically in Fig. 7, which exclusively addresses the redistribution of the shear stresses which can not be transferred in undisturbed manner due to the missing cross-section in the hole area [Kolb and Epple, 1984; Blaß and Steck, 1999a-c].

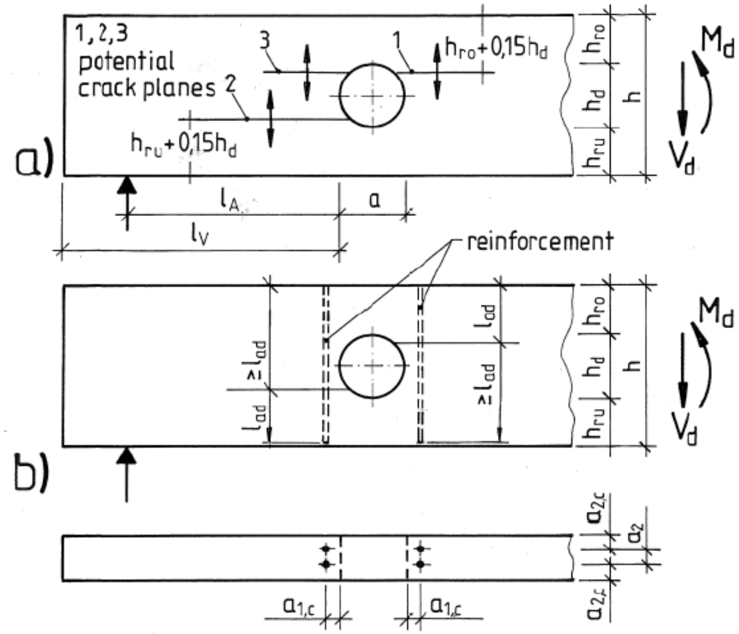


Fig. 6: Geometry notations and endangered/potential crack planes of unreinforced and reinforced glulam beams acc. to DIN 1052:2008

8-12-2 H Kolb, P Frech
Instruction for the reinforcement of apertures in glulam beams

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

Reinforcing must be done by beech plywood slabs.

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

Shear stress τ N/mm^2	Total thickness *d* of the reinforcement as a function of the beam width *B*
0,0	10 %
0,4	35 %
0,8	50 %
1,2	65 %

Intermediate values must be interpolated linearly. Slab thickness not less than10 mm.

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

Gluing with resorcinol glue, pressure about 0,6 N/mm^2

The corners have to be rounded with a radius of at least 25 mm. Normally the apertures should be symmetrical to the longitudinal axis of the beam. But at least a distance of 0,3 *H* to the above or below border has to be observed.

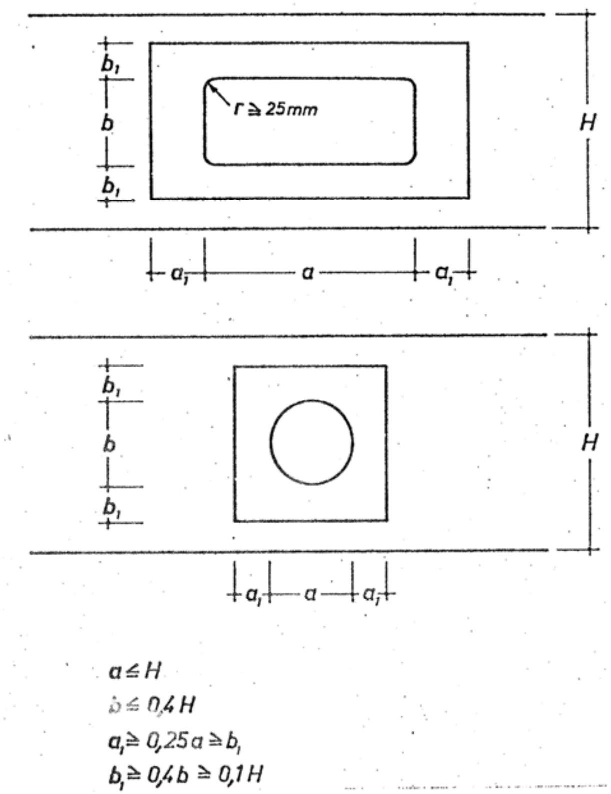
In the region of the aperture and the reinforced zones no important single loads ought to be introduced into the beams. If ducts with media the temperature of which doesn't correspond to the room temperature are directed through the apertures, the ducts have to be carefully insulated. Cross-cut ends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

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

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

Apertures are openings where at a shear stress $\tau \geq 1,2$ N/mm2 or $b \geq 0.10H$

This instruction applies only to glulam beams which are mounted under the roof, thus not exposed to weather from one or all sides.



Size of apertures and openings

28-12-3 K Riipola
Design of glulam beams with holes

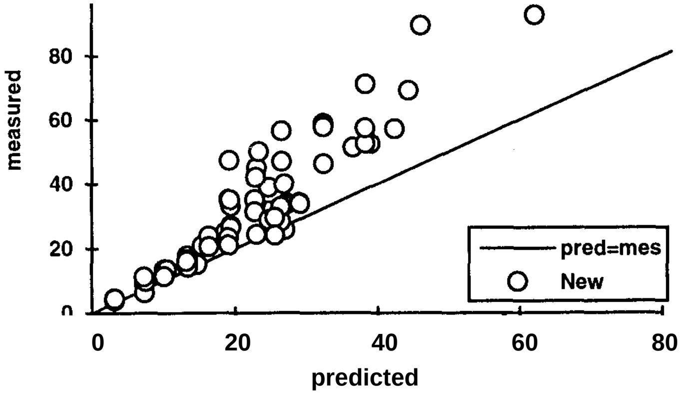
Abstract

This paper compares three design methods for glulam beams with holes. The compared methods are the German method, the method of the Swedish Glulam Handbook, and the linear elastic fracture mechanics method presented by the author in Bordeaux 1992. The linear elastic fracture mechanics method gives the best correspondence between the predicted and observed capacities. The density of the timber material is of importance for the prediction to be accurate.

Conclusions

Linear elastic fracture mechanics gives the best prediction also for such holes that have a minor effect on the capacity of the beam.

The method should apply also to other kind of holes that are not centered with the neutral axis. An application to holes not in the shear region is, in principle, also possible. About these, however, there is not sufficient experimental material to demonstrate the validity.



The measured load versus the predicted one according to the fracture mechanics method when characteristic fracture toughness values are used. Load values in kN.

35-12-1 S Aicher, L Höfflin
Glulam beams with round holes - A comparison of different design approaches vs. test data

Introduction

The design of glulam beams with holes is treated considerably different in timber design codes. Examples are the latest drafts of Eurocode 5 and of the German timber design code DIN 1052. In the first case a solution based on a linear fracture mechanics approach is stated whereas in the latter case a strength of materials design is given. In both cases the underlying mechanical models represent rather crude idealizations, especially when round holes are regarded.

The paper first shortly reveals the mechanical problem. Second, both mentioned design approaches are discussed and compared quantitatively. Thirdly, both approaches are evaluated vs. results of some recent experiments with different sized beam specimens. Some amendments of the design approaches are proposed.

Proposal for design equation changes

Any scalar changes to the DIN approach improving the agreement with the test results for large beams produce a less good agreement for beams with small dimensions. The approach, method inherent, does not reflect the experimentally obvious size effect of the fracture mechanism correctly, what is inline with former evaluations of the size effect problem (Aicher et al., 1995).

Looking at the reasons for the extreme load capacity overestimation by the EC 5 approach at least two points are important:

Factor k_n, according to

$$k_n = \frac{1}{3}\sqrt{\frac{G_{f,k}E_{0,05}}{f_{v,k}^2}}$$

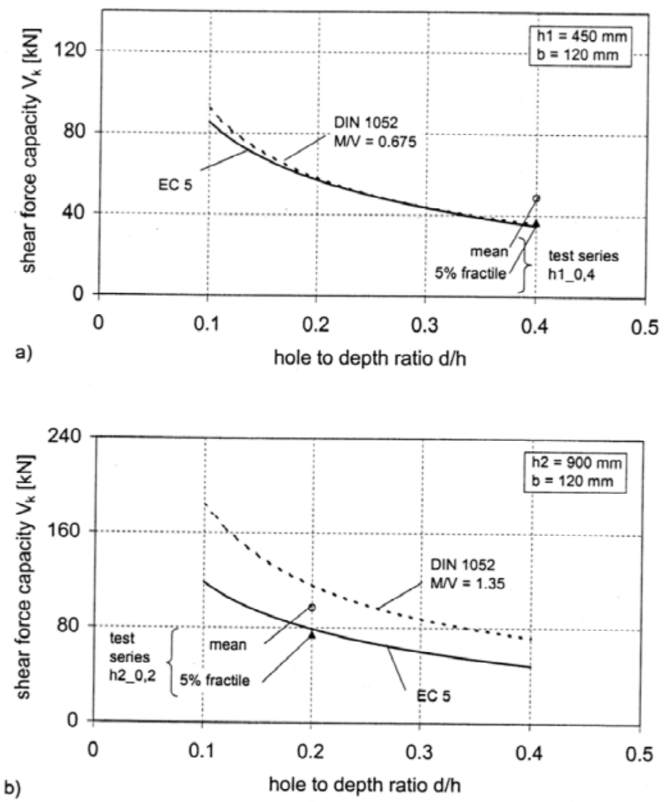
depends on (characteristic) fracture energy $G_{f,k}$ in tension perpendicular to grain. For the mean value of G_f the relationship

$$G_f = 0{,}65\rho \qquad (17)$$

was found by Larsen and Gustafson, For derivation of a characteristic value it was assumed that the ratio of characteristic to mean fracture energy corresponds to the ratio of characteristic to mean density, the latter taken to be 0,83, so:

$$\rho_k/\rho_{mean} = 0.83 = G_{f,k}/G_{f,mean} \qquad (18)$$

The mean value for the fracture energy according to eq. (17) represents a good approximation for European spruce, however eq. (18) may be considered to forward a pronounced overestimation of the 5% fractile of the G_f value. In a different investigation on fracture energy perpendicular to grain of European spruce the mean level relationship $G_f = 0{,}62\ \rho$ was found. However for the ratio of characteristic vs. mean value $G_{f,k}/G_{mean}$, a considerably smaller value of about 0,65 as compared to 0,83 in eq. (18) was obtained.



Comparison of mean and characteristic load capacities for glulam beams with a round hole depending on the hole to depth ratio according to draft of DIN 1052 and a slightly modified EC 5 draft equation vs. some test results.

There is no widely known theoretical/experimental evidence that the experimental calibration factor of 2/3 applied to the theoretical end-notched beam solution also applies to beams with round holes.

Assuming for the time being approximate validity of the linear fracture mechanics based EC 5 approach, a scalar modification, justified partly by above arguments, should give a better agreement between tests and the design equation.

In order to account more differentiated for the influence of the moment to shear force ratio and for the size effect, a Weibull theory based design model is currently developed.

36-12-2 L Höfflin, S Aicher
Weibull based design of round holes in glulam

Introduction

The bending capacity of glulam beams with holes is dominated by stresses perpendicular to grain at the periphery of the holes. The relevant material fracture resistance is tension strength and/or fracture energy perpendicular to grain. In case of rectangular holes with fairly sharp-edged corners of very small radii of curvature a linear fracture mechanics approach for modeling of the load capacity is plausible. Eurocode 5 employs for design an analogy to the fracture mechanics based design of rectangular end-notched beams. In case of round holes no quasi-singularity at the hole periphery exists, rendering at least linear fracture mechanics solutions of the design situation problematic. Nevertheless, in Eurocode 5 an attempt was made to extend the design model for rectangular holes to round holes by constructing some analogy between a tapered end-notch and a round hole.

For both types of holes, i.e. round and rectangular shaped ones, no moment contribution to the load capacity limit is considered. Especially the latter feature makes it difficult to modify the present Eurocode 5 solutions to account for arbitrary moment/shear force ratios.

This paper presents a different design model for round holes in glulam beams based on Weibull theory which shows a good agreement with experimental data.

Conclusions

The presented, easily to apply design model for glulam beams with round holes overcomes inherent, partly considerable deficiencies of present code design approaches. The model accounts for both, size effect and moment influence on load capacity in transparent manner. Hereby, it allows selective calibrations to experimental results for all relevant influential parameters.

41-12-4 H Danielsson, P J Gustafsson
Strength of glulam beams with holes - tests of quadratic holes and literature test results compilation

Background

Looking at design recommendations for glulam beams with holes in European timber engineering codes over the last decades, it can be seen that the strength design has been treated in many different ways. The theoretical backgrounds on which the recommendations are based shows fundamental differences and there are major discrepancies between the strength estimations according to the different codes as well as between tests and estimations according to codes. The contemporary version of Eurocode 5 does not state any equations concerning design of glulam beams with holes and the recommendations in the German code DIN 1052 concerning rectangular holes were withdrawn during the fall of 2007. The absence of design recommendations indicates a need for further investigations of the subject. There are, however, several tests found in the literature concerning the strength of glulam beams with holes. Two of the most recent and more comprehensive studies were presented by Höfflin in 2005 and by Aicher and Mifflin in 2006. These studies dealt exclusively with beams with circular holes. Although the test results found in literature all in all represent much work, important parameters such as mode of loading, beam size and hole placement have often been varied only within a very limit range. Among other limitations, it seems that all available test results relate to glulam beams with holes that are centrically placed with respect to the beam height.

42-12-1 S Aicher, L Höfflin
Glulam beams with holes reinforced by steel bars

Introduction

The placement of holes in glulam beams represents a frequent necessity in timber construction practice in order to enable the penetration of pipes, wirings and heating tubes. The disturbance of the stress flow around a hole creates tension stresses perpendicular to grain which reduce the load bearing capacity of beams with unreinforced holes, depending on the hole size, considerably. In general a hole reinforcement is inevitable to provide sufficient shear force capacity. Invisible internal reinforcements, such as glued-in steel rods and screws, especially self-tapping screws, are very often preferable from architectural points of view.

The paper reports first on some basic aspects of the stress distribution in the timber and on the rod forces of reinforced holes at two loading stages: i) before crack development and ii) after cross-sectional cracking at the hole edge and subsequent progressive crack extension. Second, the design and construction rules for internal reinforcements, as specified in DIN 1052:2008, are given. Third, first experiments with internally reinforced holes in structural sized glulam beams are presented. The results are discussed in comparison with experimental results on shear force capacities of beams with unreinforced holes and with regard to calculated shear capacities. A preliminary assessment of the applied design rules is given.

Conclusions

The performed tests with glulam beams with reinforced holes proved that internal, invisible reinforcements by screws or glued-in rods provide a significant, well reproduceable increase of the static short term shear force capacity vs. glulam beams with unreinforced holes. However, the investigated reinforcement configurations did not forward completely the expected load capacity level predicted by the applied DIN 1052 design model. As the final failure mode is a shear failure, as in case of unreinforced beams, it seems worthwhile to attempt a fracture mechanics approach. In ongoing tests further reinforcement and beam size configurations are regarded.

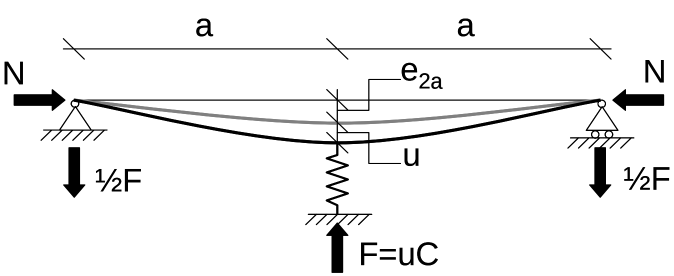
3.2 BRACING

ESSAY 3.2 H J Larsen
Bracing of compression members

The theory on which the bracing rules in Eurocode 5 are based is the following.

Single column

The free length of a single column with length $2a$ shall be reduced to a by a support in the middle. A requirement for this is that the support has sufficient strength and stiffness. The requirements are as shown in the figure found by investigating a column that in the middle has the maximum permissible deflection e_{2a}. The force F that the lateral support shall be able to exert depends among other properties on its (spring)- stiffness C.



Compression loaded column over two spans with initial mid-height deflection e_{2a} laterally supported at mid-height by a spring with stiffness C, u is the resulting deflection and F the reaction force perpendicular to the column in the elastic support.

The axial force N results in a mid point moment of $N(e_{2a}+u)$ where u is the elastic deflection. This moment is counteracted by a force $2aF/4$. Since $u = F/C$, $N\left(e_{2a} + F/C\right) = Fa/2$ or

$$F = \frac{e_{2a}}{\frac{a}{2N} - \frac{1}{C}}$$

Theoretically, the minimum stiffness of the bracing member should be

$$C = k_s \frac{N_d}{a} = 2\left(1 + \cos\frac{\pi}{m}\right)\frac{N_d}{a}$$

where m is the number of spans. For two-spans it is thus required that k_s = 2 and for several spans $k_s = 4$.

For $e_{2a}/2a$ = 1/300 that is the maximum permitted value for structural timber and for $e_{2a}/2a$ = 1/300 that is the maximum permitted the maximum permitted deviation from straightness for glulam, the required strength is given in the table depending on C.

Theoretical requirements to the strength F expressed as F/N for different C-values.

	$e_{2a}/2a$	C		
		$2N/a$	$4N/a$	∞
Structural timber	1/300	∞	1/37,5	1/75
Glulam	1/500	∞	1/63	1/125

Based on experience other values may be found in the National Application Documents.

7-2-1 J A Simon
Lateral bracing of timber struts

Synopsis

Current codes of practice give little guidance on the design of bracing and its effect on the strength of struts.
Available information on the variables affecting the behaviour of bracing is given, as well as results from a limited amount of analytical and test work done recently by the NTRI.

Object of this paper

The object of this paper is not to offer draft clauses on the design of bracing for inclusion in codes of practice, but rather to outline the problem areas and to provide a description of the 'state-of-the-art' to be used as a basis for discussion and future research work. There appear to be conflicting results on the effects of the variables involved, and considerable research effort is required before a realistic code for the design of bracing can be compiled. In the near future, the NTRI hopes to commence a project which will be designed to investigate the problem in more detail.

Conclusions

Implicit in the design of braced struts are a number of assumptions on the degree of lateral restraint offered by the bracing members. The magnitude of this restraining force depends on factors such as initial eccentricity in the strut, number of restraints, and connection stiffness. In many cases connections between rafters and purlins or battens may not have a sufficiently high stiffness to validate the design assumptions. It is therefore possible that rafters and similar members do not always have the factor of safety suggested by the design calculations.

The analysis of the forces in bracing members is currently based on 'rule-of-thumb' calculations resulting in values which bear little resemblance to those actually occurring in the erected roof. Although bracing which has been designed by experienced guesswork rather than by detailed calculation appears to have behaved adequately in the past, it is difficult to prove analytically that it is satisfactory. Some realistic design method, based on the variables involved, must therefore be developed for inclusion in codes of practice.

16-15-1 H Brüninghoff
Determination of bracing structures for compression members and beams

Introduction

To obtain a sufficient lateral stability of wooden structural members they get connected to walls, supports or bracing members being enough resistant. Those prevent too great lateral deflections rectangular to the main supporting direction of the components. Simultaneously they also can be taken to carry exterior loads such as wind.

Additional to the exterior loads there are no further forces to the bracing members as long as all parts are ideal straight and in a vertical position and if there is a perfect bringing in of loads, consequently the applied surcharge has no components rectangular to the main supporting direction of the wooden parts. In practice deviations of the ideal position have to be accepted due to unavoidable manufacturing and assembling inaccuracies. They always occur as soon as lateral thrusts due to the wind or other from the outside attacking horizontal forces are led in the plane of the supports and bracings, and thus cause deformations.

Resume

Equations are given, which allow calculating the loadings of bracing structures. The members with the risk of lateral deflection can be forced by compression forces and by bending moments. The favourable torsional rigidity of the beams is taken into consideration.

The lateral loads of compression members, e.g. flanges of trusses, and of glued laminated beams, subject to bending and of combined stressed parts can be calculated.

Simplified expressions are given for parts, subject to pure compression or bending. If the rigidity of the bracing structure is assumed, so that a given maximum of the deflection is not exceeded, very simple expressions are available.

19-15-2 M H Kessel, J Natterer
The bracing of trussed beams

Introduction

The top chord of trussed beams vertically loaded normally works in compression. The bracing system has to stabilize the chord against lateral buckling. The subject of this paper is to determine the lateral loads which act between the chord and the bracing. Before investigating the complete 3 dimensional problem, ile shall start with the simple bar to show some basic effects.

Conclusions

The distribution of the lateral load for the bracing of trusses is much more complex than is assumed in the design codes. However, from the practical point of view it seems to be too expansive always to take the real distribution of q into account when dimensioning the bracing. Therefore two strategies for dimensioning are left. The first is to determine the lateral loads by at least geometrically nonlinear finite elements, which is valid for large scale Systems only. The second is to use the code formulae and to bear in mind that a uniformly and even a sinusoidal distributed lateral load merely gives an estimation of the loads which are really applied to the bracing system.

23-15-2 T A C M van der Put
Stability design and code rules for straight timber beams

Introduction

The stability design of the Eurocode is not general and consistent enough. For instance, in the Eurocode the warping rigidity is neglected for free beams without a horizontal bracing. For braced beams however the torsional rigidity is neglected. Further the initial eccentricities are regarded for braced beams and neglected for free beams although the reversed would have been better. The given influence of the point of application of the lateral loading on l_{ef} applies only for long beams. So a more general approach is necessary. However the known calculation methods for twist-bend buckling are incomplete and often mutually contradictory and need to be extended.

By Chen and Atsuta, general equations are given for thin walled beams. However solutions are only given for pure bending with compression (thus without lateral loading). The influence of lateral loading is given by Halasz and Cziesseilski, however without initial eccentricities and without normal loading. The influence of warping is also not regarded there and thus there is no distinction between I-beams and box-beams. Larsen and Theilgaard give general equations for the case of pure bending and compression, including the influence of initial eccentricities and the failure criterion. The warping rigidity is however neglected (as also is done by all authors for rectangular beams) although there is accounted for warping deformation by the reduction of the torsional rigidity by the negative Wagner effect. This means that it is assumed that there is an unrestrained warping. However restraint warping and warping rigidity is always assumed to exist for thin-webbed beams and trusses (beams with low torsional rigidity), for instance in most regulations, because the twist-bend buckling of these profiles is calculated from the column buckling of the compressed flange, what is equivalent to a dominating warping rigidity.

In Brüninghoff's dissertation, the influence of the eccentricity of the lateral loading and the initial eccentricities are regarded for high rectangular beams. However the failure criterion is not regarded and also the warping rigidity neglected, as is only right for long rectangular beams. Because comparable general equations, including the influence of warping and the failure criterion, for the general loading case are lacking for beams and for thin-webbed beams, the derivation is given here.

26-15-1 C J Mettem, P J Moss
Bracing requirements to prevent lateral buckling in trussed rafters

Introduction

This paper describes a series of tests on trussed rafters, which were aimed at measuring their out-of-plane stiffness and hence determining directly the bracing stiffnesses and forces required to prevent their lateral buckling. The need for the work arose during a project on practical applications of bracing studies. This drew upon earlier research whose object had been to develop theories permitting calculations to be made in relation to topics such as the bracing of beams and columns, and the provision of discretely spaced braces for roof members. A series of design guides is being developed relating to stability and bracing applications, and one of these is to deal with the design of bracing for newly constructed trussed rafter roofs, as opposed to remedial bracing. It was realized that little or no experimental determinations of the out-of-plane stiffness of trussed rafters, for the purpose of confirming parameters for their stability bracing, had ever been carried out. Consequently, these tests were planned. It was intended to compare the measured lateral stiffnesses of the trusses with theoretical predictions. It was recognized that it would have been preferable to carry out more extensive practical testing, including measurements on whole-roof frameworks as three dimensional assemblies. However, this was not possible within the present scope.

The tests described were of a non-standard type. Forces representing various normal vertical design load cases were applied at the node points of the trusses. The trusses were in fact tested in a horizontal arrangement, for convenience, but these normal 'vertical' loads are described as such throughout this paper. Small loads at right angles to the plane of each truss were also applied, whilst the trusses were subjected to the main vertical loadings. These secondary loads were used, in conjunction with precise lateral deflection measurements, to evaluate the out-of-plane stiffness for each truss configuration, load case and restraint arrangement.

Concluding remarks

The experimental work described above was carried out on typical examples of commercially available medium-span trussed rafters, of 35 mm thickness and M75 grade, such as are sold in tens of thousands by normal stockists. These were not specially fabricated components, manufactured by laboratory staff. They incorporated 'initial lack of straightness' typical of such lightweight trusses. The trusses also contained punched metal plate fastener joints, whose rigidity was not complete in planwise bending, or in buckling about the minor axis. In the section of this paper dealing with the tests, it has been explained how the actual lateral stiffnesses at two important points on the rafters of the trusses were measured, whilst the components were subjected to a typical range of in-plane (vertical) design loadings.

The lateral buckling behaviour of the components under load was by no means as regular as is supposed in the normal theoretical treatments, where distinct 'second', 'third' and 'fourth' buckling modes, in discrete sinusoidal waves, can always be recognized. In order to restrain the components to behave in a manner even approaching these more ideal, theoretical conditions, a variety of sprung clamps and negative and positive restoring forces had to be applied on the test rig, through the cable-pulley system. Three of the trusses were re-tested inverted from their original positions, and the behaviour of the inverted component was not necessarily a mirror-image of the first arrangement.

As a basis for comparison, a series of linear elastic computer analyses were performed, in which the combined vertical and lateral loading arrangements were simulated. In general, these simulations indicated levels of lateral stiffness comparable to the test results, although in several instances they suggested that the tested components were in effect exhibiting one or two degrees of freedom from lateral restraint greater than was being aimed at through the test arrangement. Also, as mentioned above, the decreasing stiffnesses with increasing truss pitch, suggested by the computer model, were not reflected in the tests. This was likely to be due in part at least to some of the 'real' effects that were not modelled, including incomplete lateral stiffness of the truss plated joints, at the apex, and, for the taller trusses, at a rafter splice position.

A comparison with some simplified theoretical assumptions has been included, and this has shown that lateral stiffnesses can be predicted which again are of the same order of magnitude as both the test results and the three-dimensional model predictions. The result based on the classic Winter theory, together with assumptions regarding restraint at every third batten position, should not be compared too closely with the computer model predictions, since it was not possible exactly to reconcile each of the assumptions. Nevertheless, it is reassuring that all three methods of assessing the lateral stiffness of trussed rafters under load yield results of a similar magnitude to one another.

The adaptation of the classical discrete bracing theory for 'real' timber members with imperfections used in the comparison with theory contains several assumptions based on judgement of what is accepted in current design practice, and what codes permit, rather than being based soundly upon measurements. Throughout the project which has reviewed existing stability and bracing work in timber, there has been found to be a marked absence of experimental data, other than some very thorough beam-column tests by Larsen and Theilgaard and some measurements of imperfections in timber columns by Ehlbeck and Blass. There is a need for better experimental backing for assumptions regarding 'real' beam columns in components such as trusses, based upon careful measurements of individual members, before going too far with whole-roof tests and with further extensions of theory.

At the same time, although the British trussed rafter code, BS 5268 : Part 3 gives guidance on ad hoc bracing for domestic-scale roofs, the designer is without advice on longer spans, which are increasingly important in the roofing of commercial and industrial properties. The tests described in this paper have highlighted the probable importance of several practical effects upon likely long-term bracing performance. These include initial lack of straightness of members and of the erected components as a whole, as well as the contributions of fastener slip and creep-buckling phenomena.

As a consequence of this, a research proposal has recently been made to undertake full-scale tests to investigate the response of braced roofs to their lateral design loads, and to include an assessment of the likely long-term effectiveness of the same bracing, for stability purposes.

32-15-1 S Andreasson
Three-dimensional interaction in stabilisation of multi-storey timber frame buildings

Summary

A study has been performed with a finite element model in order to investigate the distribution of forces in multi-storey timber frame buildings under combined lateral and vertical loading. The model has been calibrated and verified against full-scale tests, performed on separate shear walls. During the tests, the vertical support forces have been monitored as well as the load and top displacement. The calibrated model has been used primarily to investigate the distribution of dead load when combined with lateral loads on shear walls and transverse walls with gypsum sheathing. The results indicate that the in-plane stiffness of diaphragms in a lateral-force-resisting system is considerable, and imply that a great portion of the dead load can be utilised to counteract uplift forces caused by lateral loading. The conclusion is that the designers have great possibilities to take advantage of dead load when stabilising the system by properly connecting the different diaphragms in order to achieve interaction in three dimensions. This is not always the practice in the current two-dimensional design approach.

Conclusions

The compression force at the end of a shear wall will always be below the sum of the dead load applied at the stud and the resultant vertical force caused by the horizontal load. On the uplift side of a shear wall, the dead load counteracting the uplift force is always greater than the dead load on that actual stud. The tributary area for the dead load counteracting the uplift force increases slightly with the number of storeys and the wall length (within reasonable limits), and more obviously with the number of sheathing layers. An increase in the anchorage stiffness gives a decreased tributary area for the dead load transferred to the point of uplift.

The possibility to take advantage of hold-downs in adjacent walls in order to counteract uplift forces in a shear wall is mainly governed by the stiffness of the connection between the walls. The capacity of the transverse walls necessary in order to transfer the uplift force from the uplift point to the anchorage is rarely a restriction in a conventionally designed system.

Dead load applied along a transverse wall can be effectively transferred to an adjacent shear wall if the shear wall is properly connected to the transverse wall and not anchored to the foundation. Hence, there is a possibility to use dead load to counteract uplift forces also when the flooring joists are supported on the transverse walls and not on the shear wall.

33-15-3 H J Larsen
Eurocode 5 rules for bracing

Background

The background for the rules for bracing in [Eurocode 5, 1993] was given in several CIB
W 18-papers e.g. **Paper 16-15-1** and **Paper7-15-1**. The present paper summarises briefly the background of the rules and gives a proposal for a modification.

37-15-4 J Munch-Andersen
Bracing of timber members in compression

Introduction

Timber members or systems of members in compression often require lateral support at intermediate nodes to prevent stability failure of the beams. The requirements to strength and stiffness of a system that can provide this support are studied. The study was initiated in connection with the preparation of a practical guideline for bracing of trusses in roof structures in accordance with the requirements in the first version from 1998 of a new generation of the Danish Code of Practice for the structural use of timber. This code has rules very similar to those in Eurocode 5. It appeared that no simple amendments to the traditional way of bracing small and medium-size roof structures for wind load would meet these requirements. However, there is no evidence that increased bracing would be necessary to avoid stability failure during snow load for normal span trusses. For long span trusses (about 20 m) made from 45 mm planks, problems have been seen that justify that sufficient bracing must be ensured.

The basis for the rules in Eurocode 5 were reviewed in order to find arguments for reducing the requirements to a level more in line with experience from practice. It appeared that the rules were not at all conservative for a single member in compression, but for a series of members - like in roof structures – there are system effects that would warrant a significant reduction. A revised version of the Danish code from 2003 includes some of the findings in this paper regarding system effects.

Conclusions

The bracing requirements for systems of timber members in compression in Eurocode 5 section 9.2.5 are somewhat on the unsafe side, even when the most conservative factors suggested are chosen.

There is a fairly simple relationship between strength and stiffness requirements to the bracing system. The designer could easily be given the possibility of choosing a favourable set for the actual purpose. This could for example be relevant if other performance requirements call for a smaller deflection than that required for the bracing. Then the load could be reduced as well.

If the requirement to the stiffness of the lateral bracing at intermediate nodes is kept at $C = 4$N/a, the force should be increased from $F = N/50$ to $F = N/37{,}5$. If the stiffness requirement is increased to $C = 6N/a$, then the present force $F = N/50$ is sufficient.

The present code value for uniform load on the bracing system for one member in compression is $q_d = N/(30l)$, which corresponds to a maximum deflection of less than $l/1000$. If the load is increased to $q_d = N/(24l)$, the permitted deflection increases to $l/500$. If other loads – like wind load – increase the deflection, the same deflection requirements should in principle be met, and consequently the stiffness of the bracing system must be increased. Since the load combination factor for wind and snow is small, the practical impact low.

The reduction of the load, when the initial deflections are smaller, is also significant for the bracing system. Therefore, there should be differentiated requirements for structural timber and laminated timber.

Significant systems effects will reduce the accumulated loads both on a common bracing system for several parallel members in compression and on the lateral bracing at intermediate nodes. It is suggested to design the bracing system for 2 times the load q for one member in compression. The accumulated force in the lateral bracing is determined to be only 2 or 3 times the force F determined for one member.

There is also an unused stiffness and strength that can be used to transfer loads locally when simple bracing systems are used for smaller structures.

42-15-3 Xiaobin Song, F Lam, Hao Huang, Minjuan He
Stability capacity and lateral bracing force of metal plate connected wood truss assemblies

Abstract

This paper presents the results of experimental and numerical studies on the critical buckling load and lateral bracing force of metal plate connected wood truss assemblies. Material property tests and full-scale tests of individual trusses and truss assemblies were conducted. The material properties test results were used as input parameters to finite element method based models, which were then verified based on the test results of individual trusses and truss assemblies. Good agreement was achieved. It was also found that the 2% design rule for the lateral bracing system overestimated the lateral bracing force in the tests. The generated database and the output of the models can be applied to more general structural configurations, and contribute to the improvement of the design methods for lateral bracing system.

Conclusion

This paper presented the results of a study on the critical buckling load and lateral bracing force of MPC wood trusses and truss assemblies. Basic material property tests and full-scale tests of MPC wood trusses and assemblies were conducted, of which the results were used as input parameter and verification for FEM based models. The following conclusions were made based on the results.

– The stiffness of individual trusses was similar despite the variation in material properties, truss plate placement and workmanship; however, the critical buckling loads of the trusses were influenced by the initial out-of-plane deformation and the out-of-plane rotational stiffness of the MPC connections of the compression members. It was also found the influence of the keeper nails on the test results of the individual trusses was immaterial.
– The 2% rule-of-thumb was found to overestimate the lateral bracing force, due to the out-of-plane rotational stiffness of the MPC connections and the randomness in the initial out-of-plane deformation of the braced webs.

It should be kept in mind that these conclusions were based on the test results of the particular truss configurations and load situations used in this study; however, this work provides a framework about how to test and evaluate the critical buckling load and lateral bracing force of MPC wood truss assemblies.

42-15-4 Chun Ni, H Rainer, E Karacabeyli
Improved method for determining braced wall requirements for conventional wood-frame buildings

Introduction
Wood-frame construction is by far the most common structural system in North America for single-family houses and low-rise multi-family dwellings, constituting over 80 % of all residential housing. In North-America, wood-frame construction can be built either by following prescriptive codes or engineering design codes. Conventional wood-frame construction refers to buildings that are designed and built according to the prescriptive rules such as those in Part 9 of National Building Code of Canada (NBCC) or the US International Residential Code (IRC). T he prescriptive rules in the codes are largely developed based on historical practice for housing and small buildings as well as pre-engineered solutions. Although buildings designed and constructed with the prescriptive rules have performed well in past earthquakes and resulted in relatively few casualties, a very few wood-frame buildings have collapsed or suffered serious damage, particularly where lack of adequate bracing created a weak first storey.

In this paper, bracing requirements for conventional wood-frame construction in the current codes are discussed. An improved method to better rationalize the bracing requirements for conventional wood-frame construction is presented. The lateral load capacities of braced walls are also discussed.

Summary and conclusions
The potential shortcomings of the bracing requirements for conventional wood-frame construction were analysed on selected building scenerios where the lengths and locations of the braced wall panels were chosen to represent as much as possible the most unfavourable case for lateral load resistance. The results showed the imbalance between the required lengths of braced walls in short and long directions of a rectangular building.

While the lateral load capacity in the long direction of the building is adequate for the twoand three-storey buildings and is in fact overly conservative for the one-storey building, the lateral load capacity in the short direction of the building may not be sufficient to resist the base shear forces with the minimum bracing requirements in the codes. The results also showed that two-storey building has the largest discrepancies between the base shear and the lateral load capacity. This is because the required minimum lengths of braced wall panels are the same for one-storey and two-storey buildings in the codes.

A new method was proposed to better rationalize the bracing requirements for conventional wood-frame construction in Canadian and the US building codes. Instead of specifying the minimum length of braced wall panels as a constant percentage of the length of a building parallel to the direction of loading considered, the new method specifies the minimum length of braced wall panels as a function of floor area of the building. This will address the imbalance between the required lengths of braced walls in the short and long directions of a rectangular building.

3.3 BRIDGES

In general Eurocode 5 Part 1-1 applies also for the design of bridges. Eurocode 5 Part 2 covers however some topics that are special for bridges. Important examples are deck plates and fatigue.

The members of the drafting groups for Eurocode bridges have, however not been very active in CIB-W18 and the few papers on bridges are general and not related to Eurocode 5.

27-12-1 M A Ritte, T G Williamson
State of the art report: glulam timber bridge design in the U.S.

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.

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 dear 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 preservative treated at manufacturing facilities in the U.S.

27-12-2 C J Mettem, J P Marcroft, G Davis
Common design practice for timber bridges in the United Kingdom

Introduction
There is a well established history and a continued use of timber for bridges in the United Kingdom. In a recently initiated survey, 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 longstanding and substantial experience in the use of timber for pre-fabricated modular bridges in developing countries. 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 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 deterrent.

TTL were extremely impressed by the manner in which the US National Timber Bridge Initiative was launched, and by its subsequent success. 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. 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.

39-12-1 K Crews
Recommended procedures for determination of distribution widths in the design of stress laminated timber plate decks

Synopsis

Stress laminated timber (SLT) decks are constructed by laminating individual pieces of timber placed side by side (on edge), until a solid deck of the desired width is achieved. The laminating is achieved by compressing individual timber members together by applying a prestress in the transverse direction, which "squeezes" the individual pieces of timber together, creating an orthotropic plate.

A number of approaches have been adopted for modelling the orthotropic behaviour of SLT plate decks. BS EN 1995-2: 2004 presents three of these as basic methods for design of SLT plate decks –orthotropic plate methods, grid (or grillage) modelling and the so called simplified method, which uses the concept of distribution width, to design the deck as a "wide beam."

This paper discusses the basis for modelling deck behaviour adopted in North America, Australia and Europe and compares the various predictions of distribution width, for a given material. Modification to some aspects of BS EN 1995-2: 2004 (Eurocode 5: Design of timber structures – Part 2: Bridges) are recommended that would lead to increased efficiencies in design using the "simplified method", based on the results of research in North America and Australia.

Conclusions and recommendations

Accurate prediction of the distribution width for design of an SLT plate deck has a major bearing on the economics of the structure, as well as being important for providing realistic assumptions for modelling the structural behaviour of the deck system. Iit can be concluded that the current simplified method for determining distribution width specified in BS EN1995-2: 2004, is conservative when compared with other international methods, which have been derived from load testing of prototype stress laminated timber bridge decks.

Both the Canadian and Australian methods for predicting the distribution width are based on simple equations that have been found to have acceptable levels of accuracy and reliability when compared with the results of full scale load testing. It is therefore recommended that these same equations be assessed for applicability for modelling the load responses of European stress laminated timber plate bridge decks. This assessment should ideally be undertaken by analysis (using the equations presented in Section 4 of this paper) of data obtained from load testing of suitable bridges constructed from European timber species. The applicability of the equations would then be determined and if necessary adjustments made to produce a simplified set of equations that replace the current provisions of Clause 5.1.3 of BS EN1995-2: 2004 for longitudinal SLT decks.

42-12-5 K Karlsson, R Crocetti, R Kliger
Mechanical properties of stress laminated timber decks - experimental study

Introduction

Timber bridges for road traffic are often designed as stress laminated timber (SLT) decks. SLT decks consist of several individual timber beams placed side by side and then stressed together. The friction generated by the prestress between the surfaces of the laminates makes it possible to consider the beams as a homogeneous timber plate. In EN 1995:2004 three models for analysis of timber bridges are suggested, namely:

Orthotropic plate theory
Modelling the deck plate by a grid
Simplified method, calculation with an effective width $b_{eff.}$

In order to analyze an orthotropic plate, knowledge about material properties in the two main directions of the plate are required. Modulus of elasticity (MOE) and Poisson‟s ratio in both directions, as well as in-plane shear modulus are required. There is a significant difference between the material parameters of a single beam and the material parameters of several beams acting as a structural system. It is of great importance that the material parameters of such a system are identified. These material parameters for SLT decks have been studied over the last two decades mainly in North America and Australia, but also in Europe to some extent. Due to practical reasons, the transverse MOE and in-plane shear MOE are often expressed as a percentage of the longitudinal MOE. The MOE in the longitudinal direction is commonly a known property for a given timber material. Modelling the deck plate by a grid also requires knowledge of the material parameters in two main directions of the plate, similar as for orthotropic plate theory. The third alternative in EN 1995-2:2004 is to design the SLT-deck with a simplified method based on the assumption of a beam with an effective width b_{eff} to carry the loads. This assumption is the basis for simplified hand calculation methods e.g. Ritter (1990), Crews (2002). The method suggested in EN 1995-2:2004 is significantly more conservative than similar international design methods (Crews, 2006).

There are several experimental methods for determining the material parameters of an SLT. Some methods include dynamic measurements, other require several test sequences. In this paper the results of a test series according to a method first suggested by Stephan Tsai in 1965 is shown. Two square timber plates are needed together with knowledge of the longitudinal MOE $E_{x.}$ Plates are subjected to pure twisting and deflection is measured in the middle of the plate.

This paper is the first part of a larger study with the aim of revising suitable design methods for SLT bridges made of Swedish glulam and designed for Swedish requirements.

This is a pilot study for obtaining material parameters for SLT decks.

Conclusion

Material parameters for SLT-decks can be seen in Table 2 for different prestress levels.

The test specimens were constructed out of glulam beams made from of Norway spruce with rather large dimensions. In EN 1995-2:2004, various MC in timber is considered for the coefficient of friction but not for material parameters for orthotropic plate theory. The influence of various MC requires more research to fully understand its influence on SLT decks. MC in the test specimens were about 9%, which affected the decks performance and influenced the capability of comparing with other studies. However, it might give an indication to behaviour of SLT bridges in extremely dry environments.

When comparing design methods, the influence of varying material parameters as input data to an orthotropic model only produced an approximate 5% difference between the high values suggested in EN 1995-2:2004 and low values obtained in this study. The difference between the simplified analysis suggested in EN 1995-2:2004 and design methods suggested by Ritter and Crews is approximately 25-35%.
The difference in required deck thickness between the two design-methods suggested in EN 1995-2:2004 is approximately 25-50%.

3.4 COLUMNS

ESSAY 3.3 H J Larsen
Timber columns

Solid columns

The first draft for the CIB code (the predecessor for Eurocode 5) was based on **Paper 2-2-1**. The departure for this paper was a straight linear elastic column loaded with a sinusoidal deviation from straightness with an eccentricity in the middle of

$$e = e_1 + e_2 = a + b\lambda \tag{1}$$

where λ is the geometrical slenderness ratio:

$$\lambda = l/i \tag{2}$$

where

l column length/buckling length
i radius of gyration

A very simple failure criterion was used:

$$\sigma_c/f_c + \sigma_m/f_m = 1 \tag{3}$$

where

σ_c axial stress
σ_m bending stress
f_c compression strength
f_m bending strength

The column factor is defined as

$$k_c = \frac{\sigma_{cr}}{f_c} \tag{4}$$

where

σ_{cr} column failure stress

σ_{cr} was determined for various assumed eccentricities used in timber codes in Canada, France, Germany, The Netherlands, Norway, Switzerland, Sweden, UK and USA. In UK $a = 0$ has been used, leading to the so-called Perry Robertson formula from 1925.

In **Paper 4-2-1** results of tests with 120 columns of Nordic Spruce of C18 and C30 and with deflection both in the weak and the strong direction are reported.

The main conclusion is that the theory described in **Paper 2-2-1** is satisfactory and that the eccentricity independent of timber grade and direction may reasonably be put at:

$$e = (0{,}1 + 0{,}005\lambda)r \tag{5}$$

where

r is the core radius.

In **Paper 20-2-2** it is criticised that the design of timber columns is based on the elastic theory assuming that collapse occurs when an elastic limit state stress is reached. Research has shown that this failure criterion is conservative and that a considerably higher load-carrying capacity may be found – especially for laterally loaded columns – by taking the plastic behaviour into account. This is also pointed out in e.g. **Paper 17-2-1 and Paper 30-2-1.**

A computer model for the ultimate load of glued laminated columns is described and used to determine characteristic values of the load-carrying capacity of timber compression members. Monte-Carlo-simulations are used to calculate the ultimate load by a second order plastic analysis, assuming for both glued laminated and solid timber columns the stress-strain-diagram shown in Figure 1.

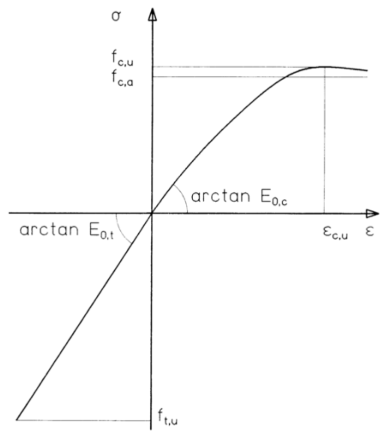


Figure 1. Stress-strain diagram. $f_{c,d} = 0{,}85 f_{c,u}$; $\varepsilon_{c,u} = 1{,}25 f_{c,u}/E_{0,c}$.

In the case of solid timber the properties were determined for cross-sections spaced 150 mm using the following structural attributes: density, knot area ratio, moisture content, and portion of compression wood. For glulam the properties were determined for cross-sections spaced 150 mm in each lamination and further the strength of finger joints were taken into consideration.

An example of a calculated k_{cr} -curve is shown in Figure 2.

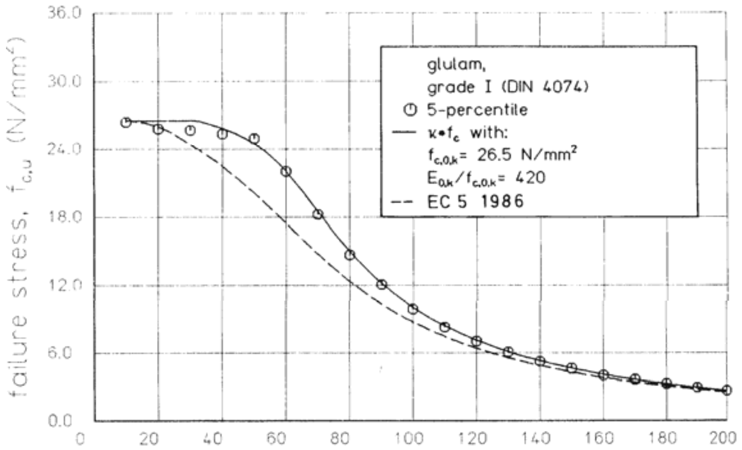


Figure 2. k_{cr} as function of the relative slenderness for timber with the assumed properties for grade 1 according to DIN 4074 together with the curve found from Eurocode 5. It is seen that Eurocode 5 is on the safe side.

The curves have been approximated by analytical expressions that happen to be the same as those given in Eurocode 3 for steel columns,

The Eurocode 3/Eurocode 5-curves may formally be found by the method described in **Paper 2-2-**1 with an initial deviation from straightness given by:

$$\frac{e}{r} = \frac{f_m}{f_c} \beta \left(\lambda_{rel} - \lambda_{rel,0} \right) \tag{5}$$

where

β constant

λ_{rel} relative slenderness

$\lambda_{rel,0}$ the relative slenderness for the test specimens from which the compression strength is found

$$\lambda_{rel} = \frac{\lambda}{\pi} \sqrt{\frac{f_c}{E}} \tag{6}$$

$$\lambda_{rel,0} = 0,3$$

$$\beta = \begin{cases} 0,2 & \text{for structural timber} \\ 0,1 & \text{for glulam} \end{cases}$$

Since the curves have been approximated by analytical expressions of the same type as in **Paper 2-2-1**, the difference for centrally loaded columns is marginal. For laterally loaded columns, however the plastic method leads to significantly higher load-carrying capacities than the elastic approach for slenderness ratios below about 80, see Figure 3.

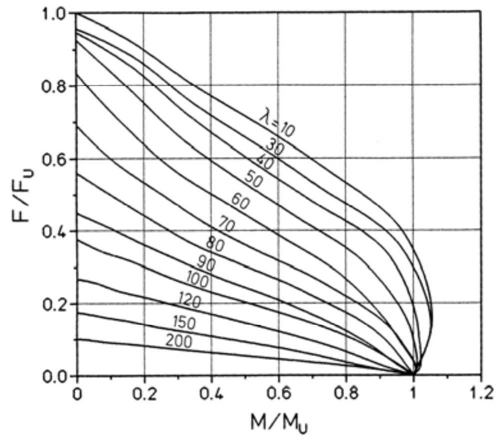


Figure 3. Combinations of axial forces and moments. *F/F_u* and *M/M_u* are in (7) and (8) denoted σ_c / f_c and $\sigma_{m,y} / f_{m,y}$.

This is, however, not taken into consideration in the general column expressions in Eurocode 5, where linear interaction expressions are prescribed, but only in the expressions for cross-section verification for combined compression and bending where for all slenderness ratios it is required that it shall be verified that

$$\left(\frac{\sigma_c}{f_c}\right)^2+\frac{\sigma_m}{f_m}\leq 1 \quad \text{Eurocode (6.19)-(6.20)} \tag{7}$$

Leicester presents in **Paper 21-2-1** a very simplistic approach to instability problems. It is based on the fact that it is generally possible to determine the load-carrying capacity for the two extremes: slender members where the elastic solution (for columns e.g. the Euler load) and members without stability problems where the strength corresponds to the failure load of the cross-section. His thesis is then that any reasonable interaction curve supported by a few test is sufficiently accurate for practice. This approach is in Eurocode 5 used for lateral beam instability.

Built-up columns

Built-up columns, Eurocode 5, Annex C) are treated in **Paper 3-2-1**. Reference is made to the general theory for built-up structures where expressions for an effective moment of inertia I_e are given. It has been shown by testing that the load-carrying capacity for slender perfect columns is determined by the Euler formula using the effective moment of inertia. It is then suggested that the load-carrying capacity can generally be based on the usual expression, but with the slenderness ratio determined from I_e and not from the total moment of inertia I. The justification for this is discussed.

Expressions for the effective moments of inertia for various types of columns: continuously jointed columns, spaced columns with glued; nailed or bolted packs, spaced columns with glued or nailed battens (Vierendeel columns) and glued or nailed lattice columns are given based on tests with the said types of columns, partly to assess the applicability of the proposed theoretical expressions and partly to determine the rigidity of the connections.

The paper contains the expressions found in Eurocode 5, Annex C.

2-2-1 H J Larsen
The design of solid timber columns

Synopsis
This report has been prepared as the basis for discussions in CIB Working Group W 18 regarding the possibilities of formulating on a uniform technical basis Codes for Timber Structures, so that differences only arise due to different loading and safety levels.

The report has been prepared on the basis of standards and supplementary material received from participants in the Working Group. So far, only solid columns have been dealt with, inter alia, because it is relatively easy – at any rate theoretically – to expand the calculation principles for solid columns to apply to composite columns as well.

3-2-1 H J Larsen
The design of built-up timber columns

See 3.8 Mechanically jointed beams

4-2-1 H J Larsen, S S Pedersen
Tests with centrally loaded timber columns

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

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

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

$$e=\left(0{,}1+0{,}005\lambda\right)k$$

λ being the slenderness ratio and k the core radius. Reference is made to figures showing both test results and theoretical values.

Further conclusions of the tests are that:
– the Euler load-carrying capacity can be determined very satisfactorily by the so-called Southwell-plot,
– the accordance between the modulus of elasticity determined by edgewise bending tests and by the Euler formula is very satisfactory. On the other hand, the correlation between the modulus of elasticity in compression (determined on 200 mm long prisms) and the modulus of elasticity in bending is weak- the compression strength and the modulus of elasticity in bending were found equal for the two grades,
– the correlation between the initial deflection of the columns in the middle and the initial eccentricity determined from the tests by the Southwell-plot is weak

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

4-2-2 B Johansson
Lateral-torsional buckling of eccentrically loaded timber columns

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

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

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

Conclusions

The study indicates that timber members subjected to bending and compression may fail by lateral-torsional buckling. However, in most timber structures the members are braced in lateral direction and hence he problem is mostly avoided. In some large structures in which the main frames are braced by secondary members, the unbraced lengths may be long enough to make lateral-torsional buckling possible

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

Basic theory

It is assumed that the member has a uniform doubly-symmetrical cross-section. The material is assumed to be elastic and the member to be straight immediately before the buckling takes place. The basic case of loading is a central thrust combined with a constant bending moment in the plane of maximum stiffness, see

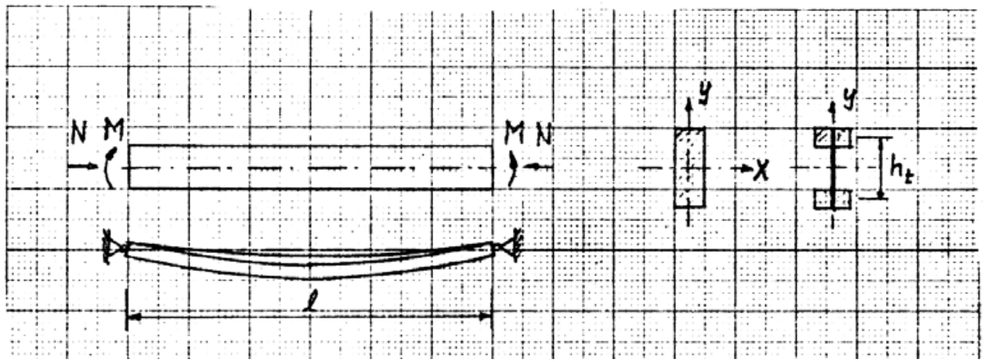


Figure 1.

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

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

in which

N_y is Euler buckling load in the lateral direction
N_t is torsional buckling load
M_{ce} is lateral buckling moment in absence of thrust.

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

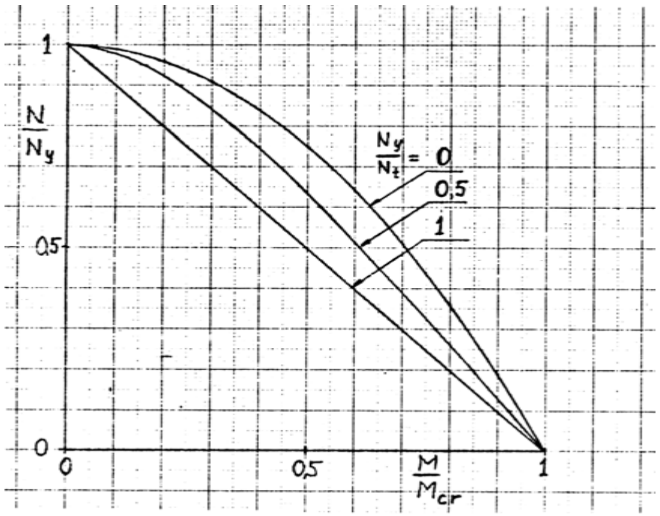


Figure 2 Interaction curves for bending moment and thrust

5-9-1 B Källsner, B Norén
Strength of a wood column in combined compression and bending with respect to creep

Introduction

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

8-15-1 H J Larsen
Laterally loaded timber columns: tests and theory

Introduction

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

The correspondence between theory and practice is good for slender structures, but for non-slender structures there are great deviations which are traditionally explained by a stress dependent modulus of elasticity. For timber, at least, there is bad correspondence between the moduli of elasticity normally measured and those that must be assumed theoretically to explain the load-carrying capacities for columns that are found by tests. For columns the correspondence between theory and tests has proved much better if the columns are assumed to be elastic to rupture but with deviations from straightness.

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

The theoretical load-carrying capacities have been derived. The expressions have been verified by tests described in the paper.

The load-carrying capacity expressions found are discussed and simple, approximated design expressions are set up.

17-2-1 T Poutanen
Model for timber strength under axial load and moment

Introduction

It is known that the generally used linear interaction formula:

$$\frac{N}{N_u}+\frac{M}{M_u}\leq 1$$

is conservative.

The author has developed a timber strength model which is different. The model is presented to raise discussion and to get test results.

The author has no resources to make tests to verify the model.

Conclusions

1. According to the model design formula 1 is 15-20 % conservative when $N,M \neq 0$.
2. Results can be derived from basic assumptions without any calibration to test results.
3. According to the model bending strength can be calculated from compression and tension strength. This calculation fits well with Finnish timber code and a test of 550 samples.
4. The *N, M*-interaction curve of the model is closely the same as that derived by Buchanan from completely different basic assumptions.
5. It is possible to derive equation for size effect from the model. This equation means that size effect can be practically completely explained by coefficient of variation of defect. Test information on size effect is limited but the model seems to fit into available tests.
6. The model defines clearly cross section deformations. This gives an opportunity to unlinear analysis of timber structures.

18-2-1 A H Buchanan, K C Johns, B Madsen
Column design methods for timber engineering

Abstract

Combined bending and axial loading is often encountered in lumber and timber members. Existing design methods are based on studies carried out many years ago, and are no longer appropriate because they do not recognize that wood with defects behaves as a non-linear ductile material in

compression, and as an elastic brittle material subject to size effects in tension.

This paper summarizes the findings of a comprehensive investigation into the behaviour of lumber subjected to eccentric axial loading carried out at two Canadian universities. The study included analytical modelling, and an extensive experimental program using full size lumber.

The results of the investigation have been used in this paper to propose improved design methods, using design charts and approximate formulae for in-plane behaviour. The discussion is extended to general loading cases and biaxial behaviour. Input information required for the design process is also discussed.

Introduction

Combined bending and axial loading is encountered in many types of timber structures. The most common are columns, chord and web members of trusses, and wall studs. Others include arches, domes and rigid frame structures.

Current design methods, developed in the 1920's, are not appropriate for modern timber engineering design. Major deficiencies include the following:

- they are mainly based on concentric loading tests on small clear wood specimens,
- they assume that realistic strength properties can be obtained from tests on small clear specimens,
- they assume that a member loaded in combined bending and compression will fail when a limiting elastic compression stress is reached. The possibility of a tension failure is ignored,
- non-linear stress-strain behaviour in compression is not considered,
- the effect of stressed volume on the strength in tension is not recognized,
- they do not recognize that knots and material variability make it impossible to load a member with perfectly concentric axial loads.

Conclusions

Four alternative design methods have been proposed for lumber and timber members subjected to combined bending and axial loading. These methods are based on the results of experimental and analytical studies into the strength of lumber under eccentric axial loading.

The proposed design methods reflect the behaviour of real materials more accurately than existing methods. They offer potential for more efficient and economical design of certain types of timber structures.

This study has identified many areas with potential for further research, the most important being the biaxial behaviour of slender timber members at ultimate loads.

19-2-1 R H Leicester
Creep buckling strength of timber beams and columns

Summary

A simple structural model is developed for predicting the strength of timber beam-columns. The model includes the effects of initial crookedness and of creep. Two different material failure criteria are used. Numerical solutions are given to illustrate the predicted performance characteristics of beam-columns and to illustrate the influence of the various structural parameters.

Introduction

The recommendations in the draft Eurocode 5 for computing the design strength of timber beam-columns are based on a study by Larsen and Theilgaard (1979). In that study the effects of timber creep are neglected and the strength of timber is defined in terms of a linear criterion that relates bending and compression stress at failure.

It is the purpose of this paper to extend the original study to include the effects of creep, and also to examine the consequences of using an alternative failure criterion. The creep model to be used is that applied in the first Australian Standard on timber engineering design AS CA65 and is also applied in the latest draft of that Standard, now AS 1720 (Standards Association of Australia, 1986). The alternative failure criterion to be examined is a bilinear one, based on experimental data obtained in studies by Buchanan (1984).

In drafting design recommendations for creep buckling, a dominating consideration is that while there is little relevant experimental data available, buckling strength is a function of a great number of complex parameters; examples of these parameters include those used for material failure and creep criteria, material variability, nonlinear material characteristics, the random dispersion of defects and initial crookedness. Thus any structural model chosen will be either oversimplified or one in which there is considerable uncertainty concerning the appropriate values to be used for important parameters.

In the following, a relatively simple structural model is developed.

Most of the equations are quite general in scope, but for clarity the discussion will be limited to the particular structural example. The structural element is a beam column of rectangular cross-section $b \times d$; the lateral restraints are arranged so that the effective buckling lengths about the major and minor axes are L_x and L_y respectively. The load is an axial load, denoted by N, applied with an eccentricity e in the y direction; this leads to an effective applied bending moment M given by

$$M = Ne$$

Concluding comment

The equations derived herein provide a relatively simple method of calculating predicted strengths for beam columns. The structural model used includes sufficient parameters to permit an empirical fit with any data that may be obtained within the foreseeable future. Input parameters include creep, initial crookedness and failure criteria. In view of the limited data available on beam-columns, it is unlikely that use of a more complex model is warranted.

It is outside the scope of this paper to discuss appropriate values for model parameters. The data on long-term performance characteristics is extremely limited; some information is given in papers by Leicester (1971b, 1972) and Cheng and Schniewind (1985). Even for short-term characteristics, the data is not extensive.

Summary

For the design of timber compression members a new method is submitted, based on a non linear moment - normal force - interaction of the cross section. The method is derived from the results of extensive simulation calculations and applies to European spruce glulam and to European softwood timber columns. The material behaviour is taken into account better than in former methods. With this method a more economic design of columns is possible. The influence of moisture content and grading is shown for centrically loaded columns.

19-12-2 H J Blass
Strength model for glulam columns

Introduction

The design specifications for compression members of the current standard DIN 1052 "Holzbauwerke, Berechnung und AusfUhrung" (timber structures, design and construction) are based on a second order elastic analysis, which underlies the ultimate loads for timber compression members given by Möhler. In the CIB Structural Timber Design Code and in Eurocode 5 the verification of the compression member is also founded on the elastic theory.

More accurate methods of calculation for timber structural members under compressive stress - particularly a second order plastic analysis - could not lead to a more efficient utilization of timber as a building material, until the stochastic scatter of the dominant influencing variables entered into the calculation, since the advantage of a more realistic mechanical model had so far been compensated by uncertainties concerning the assumed basic material properties. The introduction of the concept of reliability in connection with a probabilistic safety principle then made it possible to develop a mechanical model giving an accurate description of the member behaviour in the limit state, taking account of the statistic distribution functions of the basic variables. Whether a given level of reliability can be attained in an economical manner, depends on how exactly the mechanical model describes the load-bearing behaviour and on how well the stochastic model of the basic variables is known.

Generally, the failure probability pf can be determined as a function of the random variables 'resistance R' (e.g. load-bearing capacity of a column) and 'load effect S', as decribed in the following.

$$p_f = \int_0^\infty f_S(s) F_R(s)\, ds$$

$$p_f = \int_0^\infty \int_0^x f_S(s) f_R(r)\, dr\, ds$$

The literature provides comprehensive information on the load distribution, whereas the resistance has been investigated only to some extent. The object of this work is to determine the resistance of glued laminated compression members taking account of the geometrically and physically nonlinear behaviour of the structural components. A mechanical model of the timber compression member is obtained by a second order plastic analysis, which directly yields the load-bearing capacity of a structural member. The stochastic model of the basic variables covers the distribution functions of all governing characteristic quantities including the respective parameters of the distribution.

20-2-2 H J Blass
Design of timber columns

Introduction
The design of timber columns in the draft Eurocode 5 (1986) and in the CIB Structural timber design code (1983) is based on the elastic theory with a linear failure criterion of the cross section: collapse of the column occurs when in the critical cross section an elastic limit stress is reached.

Some research results during the last years show that this is a conservative failure criterion. Taking into account the plastic deformations of the timber when subjected to compression parallel to grain, the ultimate loads of timber compression members are considerably higher than under assumption of the elastic theory.

It is the objective of this paper to provide approximate functions for the characteristic strength of centrically and eccentrically loaded timber columns.

21-2-1 R H Leicester
Format for buckling strength

Introduction
Buckling strength is a complex function of many parameters including the following, material properties (strength stiffness, failure criterion, defect dispersion, crookedness), climate (as it affects material properties, creep), member cross-section (area, section modulus, plywood and glulam lay-up, structure of built-up members), and structural geometry (member length, end fixity conditions, method of load application). Because of these complexities, a systematic and consistent approach should be used in the presentation of equations for buckling strength in a design code; if this is not done, the code users are likely to be confused and unclear as to the type of structure being analysed and whether the results obtained are reasonable.

The following is a proposal for a code format to be applied uniformly for all types of buckling strength specifications. In so doing, it is well to bear in mind that to strive for a high degree of accuracy is inappropriate; many of the critical parameters that affect buckling strength are either unknown or vary significantly from one member to another; examples of such parameters are crookedness, defect dispersion and failure criteria.

Conclusions
The proposed format for buckling strength described by equations would appear to be applicable to define the buckling strengths of a wide range of structural members.

21-2-2 R H Leicester
Beam-column formulae for design codes

Introduction
Analytical solutions of beam-column formulae can be extremely complex and are not very suitable for use in design codes. The complexities of these formulae are confusing to the designer and imply an accuracy that is not in line with the available knowledge for any specific real structural situation.

Usually data on the buckling strength of beams and columns can be obtained without too much difficulty through direct measurement on a specific grade of a particular species of timber. From this data, the five-percentile value is extracted for design purposes. However data on the interaction between bending and axial compression strength is difficult to obtain and hence it is appropriate to use some simple empirical estimate of this interaction for design purposes.

In the following a simple interaction equation is proposed, and checked where feasible against analytical solutions and experimental data.

Conclusions
The beam-column interaction equations appear to be satisfactory for use in design codes.

30-2-1 W Lau, F Lam and J D Barrett
Beam-column formula for specific truss applications

Introduction
In Canada, there are basically two typical residential roof structures – the traditional rafters and joists (usually cut and fabricated on site) and the

metal plated wooden roof trusses (usually prefabricated). Whereas the former dominant the residential market before 1960s and are still used for roofs with unusual configurations, the latter has been playing a major role in the residential market since its introduction.

From structural theories, it is expected that lighter section can be used to fabricated roof trusses due to the load sharing effect. However, when prefabricated roof trusses were first studied in the 1960s, trusses designed according to the timber design codes resulted in member sizes larger than those obtained from the traditional rafters and joists method. A series of prefabricated roof trusses were then tested which showed satisfactory performance even though they did not meet the standard wood design procedures as set forth in CSA Standard 086.1 "Engineering Design in Wood". Consequently, the roof framing requirements in the code were rewritten stating that residential roof trusses no longer required to satisfy the wood engineering design code as long as they passed the test as described in CSA Standard S307 "Load Test Procedure for Wood Roof Trusses for Houses and Small Buildings." Since most roof trusses are different, it is impossible to test all possible truss configurations. As a result, the Truss Plate Institute of Canada (TPIC) developed a set of design procedures based on which trusses can be designed with confidence to pass the S307 procedures. According to the TPIC design procedures, an increase of 33 % of allowable stresses as specified in the working stress design version of CANS-086.1-M84 is allowed. This increase was explained as the load duration factor for structures supporting loads of 24 hours, which is the test duration of the S307 procedures. These ad hoc procedures were developed based on many years of experience and proven satisfactory in service performance.

In 1995, the Working Stress Design version of CANS-086-M84 was no longer referenced in the National Building Code of Canada (NBCC). The 1994 edition of the Limit States Design criteria in 086.1-94 did not provide for a 33 percent increase in design properties for the one day load duration. Along with changes in characteristic strengths, the removal of the 33 percent increase for one day load duration resulted in the combined stress indices of the top chords, designed using the TPIC procedures at the time, exceeding unity for trusses previously accepted in Part 9 of NBCC. Therefore, if the new Limit States Design criteria were used in designing residential trusses, member sizes would need to be increased significantly. These issues brought forth a research program engaged by the truss industry, Canadian Wood Council, Forintek Canada Corporation and University of British Columbia to develop new design procedures which are consistent with the modern limit states design philosophy and reflect the proven in-service performance of residential trusses. However, testing of two types of Part 9 trusses by Forintek Canada Corp. showed that the top chords of these two truss designs are adequate with sufficient reliability (Lum et at. 1996).

Currently, the TPIC procedures and the 086.1-94 assume a linear relationship between the axial load and bending load. It has the form of

$$\frac{M_f}{M_r} + \frac{P_f}{P_r} \leq 1$$

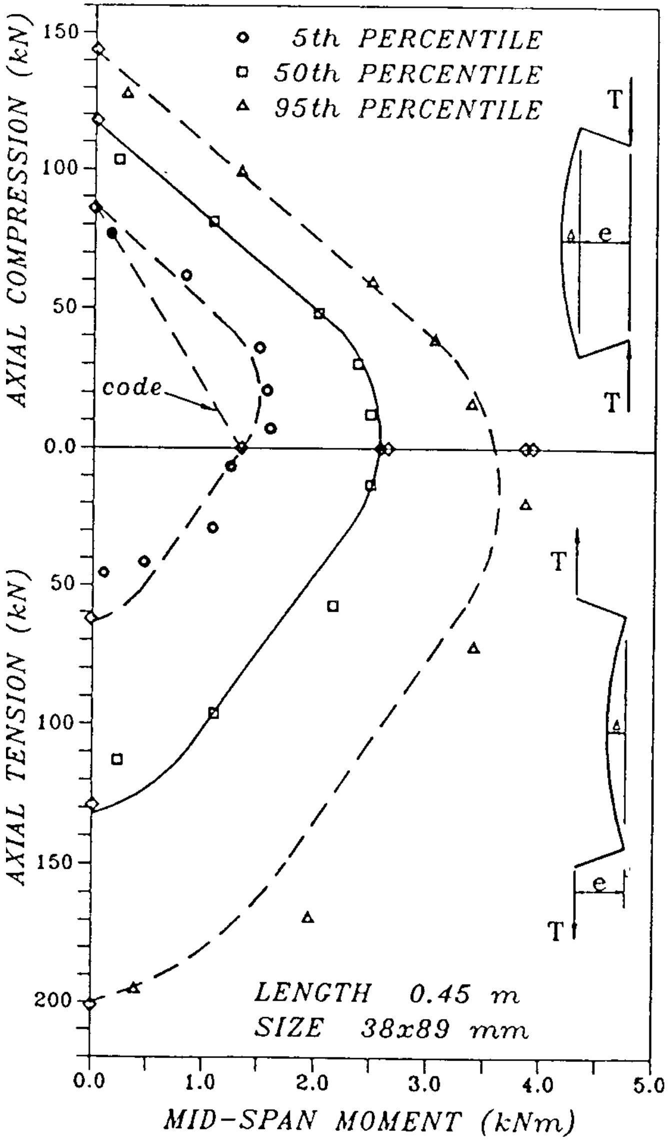
where P_f is the factored axial force, M_f is the factored bending moment, M_r, is the factored bending moment resistance, and P_r, is the factored compressive load resistance parallel to grain.

Many studies have shown that this linear interaction equation is conservative when considering dimensional lumber under combined loading.

The factored bending moment resistance, M_r, was derived based on the Canadian Wood Council Lumber Properties in-grade testing program conducted during the 1980's from which characteristic values were obtained from third-point bending test results. Modification factors such as size and treatment factors were then applied to these characteristic values to develop this factored bending moment resistance. It has been shown by many researchers that loading conditions have a significant effect on member strengths and the derived characteristic values are test method dependent.

Usually beams with higher stressed volume are expected to fail in lower loads compared to beams with less stressed volume. This dependence of member strength on the loading conditions is called stress distribution effect (or known as the load configuration effect). Madsen (1990) made a comparison of derived strengths based on different testing methods and reported a difference as much as 40%. Since the stress distribution factor was not included in the interaction equation of the current 086.1, there is a need to account for this factor to represent more realistic material behaviour under combined loading.

The objective of this paper is to provide the background and derivation to the new design procedures for wood trusses top chord members under axial compression and lateral loads which have been adopted by the 086 technical committee and will be included in the supplement No. I to 086.1-94 (1997) timber design standard.



Verifications and Conclusions

Extensive testing of columns and beam-columns were performed by Buchanan (1984). Based on his experimental results, interaction equations were developed and were compared with the size and stress distribution effects based theoretical interaction curves. Good agreement between experimental data and stress distribution based strength model was observed. In addition, the linear interaction curve used in the previous engineering code has also been shown to be over conservative in most applications. Recent test results of columns under different eccentric axial compression also confirm the non-linear behaviour of the interaction curve.

Considering some typical truss configurations manufactured as given by TPIC, a comparison of the combined stress indices computed using different criteria is shown. In general, designs based on the proposed procedures have combined stress indices that are similar to the indices computed using the CSA-086-M84/ TPIC-88 procedures. In almost all cases, the CSA-086-M84/TPIC-88 procedures produce slightly smaller CSI than the proposed procedures.

The same trusses designed according to GSA-086.1-M94 give combined stress indices which are up to 60% higher than the GSA-O86.1/TPIC-88 procedures. However, results of typical Part 9 trusses (designed using current TPIC procedures) by Forintek Canada Corp. show that they have adequate short-term capacities. Clearly the GSA-086-M84/TPIC-88 procedures are too conservative and the new procedures are more reasonable.

The new interaction equation and the associated K_m factors, which based on the stress distribution effect and experimental results, appear to better reflect the behaviour of structural lumber under combined bending and axial compression loading and thus it is appropriate for the design of top chord members of wood trusses

31-2-1 P Becker, K Rautenstrauch
Deformation and stability of columns of viscoelastic material wood

Introduction

Assuming linear visco-elasticity, deformation calculation of bending elements turns out to be very simple using a creep factor. The increase of the determined elastic deformation corresponds to the creep factor. Deformation calculation of columns proves to be involved with much more effort. Considering the problem according to theory of second order, creep deformation leads to an increasing bending moment, which again results in further elastic deformation. Superposition of creep deflection considering different classes of loads also turns out to be problematic. The object of this paper is the derivation of simple formulary, which guarantee the reliable determination of column deformations and an easy prove of the long-term stability.

Conclusion

In this paper formulas are derived, which contribute to the determination of creep deflections of timber columns under permanent and variable loading. In addition stability aspects are considered. It has to be found out by some further discussion in which way the results should be included in the codes. A consequent application of the probabilistic design concept demands, that the described conditions are satisfied by the design values of material properties and actions including the creep factors. Verification according to the codes has to make sure, that not only the short-term load bearing capacity is guaranteed.

34-2-1 W Moorkamp, W Schelling, P Becker, K Rautenstrauch Long-term experiments with columns: results and possible consequences on column design

Introduction

Although the importance of finding out about consequences of time-dependent effects to compressive members has been repeatedly emphasized, not many creep experiments have yet been performed with columns observing the time-dependent lateral deflection behaviour under axial load. Humphries/Schniewind (1982) and Cheng/Schniewind (1985) reported about tests with small clear slender ($\lambda = 173$) Douglas-fir specimens under varying climatic conditions. Their intention was the evaluation of time-to-failure of columns under high axial loads. The loads corresponded to the 1,4 to 1,8-multiple of the valid service load according to a former American code. The authors registered high failure rates during a maximum test duration of 53 days. Itani & al (1986) performed tests with structural-size slender columns ($\lambda = 121/156$) in constant climate also using Douglas-fir specimens. 70% of the individually determined Eulerload proved to be too much for the columns. None of the 60 test specimens survived the duration time of 2000 hours, most of them failed within 200 hours. Itani & al (1986) stated that the use of a former design formula might lead to unacceptable large errors in column design. Fromhold / Fridley (1998) reported about experiments under combined beam-column loading in constant climate. They concluded, that considering the creep behaviour leads to a much better evaluation of carrying performance of a wooden structural member. Some further tests have been conducted by Härtel (2000). Härtel studied the behaviour of structural-size columns under defined variable climate. He applied the limit load in service range (German code DIN 1052) to his columns and concluded, that time effects are not adequately considered in column design. Some of his measured lateral defections exceeded the tolerable deflections by multiples.

It seems important, that for creep experiments, which are performed under variable environmental conditions, structural-size specimens are taken. Results of small clear specimens can't be simply transferred to structural-size members. In addition to a much more delayed reaction to humidity changes the in homogeneity of the natural grown material wood (varying knot density, annual ring width, etc.) becomes more significant. Structural-size specimens were therefore selected for creep tests with columns reported in this paper.

34-2-2 P Becker, K Rautenstrauch Proposal for compressive member design based on long-term simulation studies

Introduction

Results of long-term experimental studies with compressive members indicate that the influence of time-dependent deformation effects to the safety of a structure is more severe than would be expected according to design rules. This especially is the case, if the portion of permanent load is high, resulting into creep deformation. researchers repeatedly emphasized this.

Creep experiments are extremely time- and money-consuming, which turns out to be problematic. There aren't any creep-tests with columns that have been conducted for more than half a year, simply too short of duration to draw any conclusion about long-term reliability under service loads. Considering this, long-term simulation studies appear as an appropriate method to observe the long-term behaviour and find out more about time-dependent effects to compressive members. Long-term simulation studies require a model, which reflects long-term material behaviour in a most possibly realistic way. As a consequence especially moist and time-dependent effects have to be covered.

Conclusions

Long-term simulation studies with compressive members have indicated, that intended reliability is not reflected by design rules. This especially applies to service classes 2 and 3 where time-dependent deformations should be considered in design process. It has been shown, that there is a difference between long-term capacity and the capacity after long-term use.

Long-term failure of compressive members is caused by different mechanisms depending, on slenderness degree:

– For slender columns long-term stability behaviour is characteristic. Because of linearity of deformations of the low-loaded slender members, long-term stability behaviour can be easily covered.
– The long-term capacity of compact columns is exclusively affected by nonlinear creep behaviour. More research is necessary to account for the exact influence.

The k_c-design-concept seems generally suitable for service class 1. For service classes 2 and 3 it is recommended not to exceed 50% of the kc-reduced design value of compression strength by design values of permanent and long-term acting loads.

The prove according to theory of second order is too conservative concerning initial pre-deflection and required reduction of E-modulus as given in the present code. Proposals for a more realistic coverage have been submitted. The verification should contain the consideration of creep deformations. For service classes 2 and 3 the design requirements are not suitable in the present form because of possible nonlinear material behaviour. The k_c-method should then be taken instead.

35-2-1 R Hartnack, K-U Schober, K Rautenstrauch
Computer simulations on the reliability of timber columns regarding hygrothermal effects

Introduction

The evaluation of the load-bearing, capacity of timber columns is based on the interaction of many effects. In particular, hygrothermal long-term effects considerably influence the reliability depending on load and are a decisive criterion for the design of wooden struts.

Nowadays it is almost impossible to evaluate the load-bearing capacity under the influence of hygrothermal long-term effects, because of the long-term nature of such experiments. Furthermore, high costs speak against a purely experimental approach if we consider the dimensions relevant to buildings as well as the large number of specimens as a result of wide-spreading parameters. Computer simulations are a cost reducing and substantially faster alternative for determining long-term effects of the load-bearing capacity behaviour.

For this purpose, software was developed at the Bauhaus-University Weimar, Chair of Timber and Masonry Engineering, which is based on the theory of finite elements (isoparametric beam elements). It considers both geometrical and physical non-linearity. The influence of the hygrothermal long-term effects was taken into account with the help of a supplementation to a computer simulations program. The adaptation of the underlying rheological model is a result of the simulation of internationally published test series. The results are consistent and verify the model sufficiently.

Summary

The wood moisture has a decisive influence on creep deformation of wooden building components and consequently on the load-bearing capacity when stability of the struts is endangered. The investigations point out, that in particular the course of the wood moisture in cross-section has a decisive influence, meaning that the values of the deformations are more dependent on the moisture fluctuations than on the level of the average wood moisture.

The actual values of the relative humidity and the resulting wood moisture are represented by approximated functions. It is apparent that the direct approximation of the climate does not match the real result very well because the fluctuation margin misses. After modifications, satisfying agreements can be realised, and appropriate climate scenarios can be created for all service classes provided in ENV 1995-1.

36-2-1 K Rautenstrauch, R Hartnack
The reliability of timber columns based on stochastic principles

Introduction

As already mentioned in publication Paper 35-2-1, the reliability of timber columns depends on a multitude of influencing variables and on the interaction of these variables. Firstly, the sector of hygrothermal long-time effects and basic principles of the calculating model was specified. In addition to hygrothermal long-time effects described the spreading of influences and of material parameters which are to be described on the basic of probabilistic principles, have a decisive influence on the evaluation of the reliability of timber columns.

Due to the multitude of influencing variables a procedure solely based on experiments is hardly possible, so that the computer simulations of the structural behaviour presented in this work represent a reasonable and an efficient alternative. Thus the parameter of the influencing sector and also the parameter of the resistance's sector could be taken into account, based on probabilistic principles. Consequentially the load-bearing capacities, calculated by the software ISOBEAM are also distributed probabilistically and they must be analysed accordingly.

The concepts of modern standards are also based on semi-probabilistic principles, so that the computer aided simulation models, presented in the following paper, could be made to verify or calibrate the semi-probabilistic methods of design.

Summary
A tool has been created which is in a position to calculate the load-bearing capacity of wooden columns with the help of virtual experiments. The material parameters of wood are determined on pure probabilistic principles and they are respectively allocated to the structural member. Action is treated in the same way, but it is partly estimated as deterministic on the basis of reasonable simplifications. According to measured values of German climate stations the climatic boundary conditions are taken into account by approximated cyclic specifications.

The chosen approach by using computer simulations makes a confrontation and a comparison to the load-bearing capacity of standard possible, so that you are able to state facts as to the actual safety. Furthermore, the influence of creep on the reliability of timber columns is investigated in this study. Especially in the constructional convenient regions of slenderness ratio you can see a high influence. The influence of the load degree and the fractile are also investigated. The influence of hygrothermal long-time effects increases with an increasing load-degree and with a decreasing fractile value.

To evaluate the load-bearing capacity of timber columns, the presented procedure on the basis of a probabilistic simulation model is excellently suitable for an economic supply of results, because such investigations are always based on a long-term observation period.

38-2-1 R Hartnack, K Rautenstrauch
Long-term load bearing of wooden columns influenced by climate – view on code

Introduction
As already mentioned in Paper 35-2-1 and Paper 36-2-1, the reliability of timber columns depends on an large number of influences and their interaction. First of all the field of hygrothermal long-time effects and the principles of the simulation model was described in detail in [4]. Furthermore, the influence of the actions and the material parameters which spread on basis of stochastic principles, on the reliability of timber columns were explained.

The wide spread of the material parameters of the construction material wood especially leads to a broad spectrum of investigations. Both the expectable high costs, the great effort of time and great amount of specimens speak against a pure experimental procedure. Therefore virtual experiments were used. The objective of these investigations was to create a design criterion under different boundary conditions to enable an easy use. In the following the development and the use will be explained in principle.

Summary
Virtual experiments represent an excellent alternative to real experiments if the used material model is sufficiently verified and a great amount of specimen is necessary because of stochastic aspects. Using the presented computer programme ISOBEAM, variation investigations like these were done with the objective to compare the safety level demanded by Code with the safety level reached by virtual experiments. As a result of the virtual experiments it can generally be stated that the additional condition of DIN 1052 seems to be appropriate in order to take hygrothermal long-time effects for high load degrees into account. Furthermore, design proposals are discussed which can describe the mentioned long-term effects for designing with a minimum equal suitability alternative to the procedure of Code.

3.5 COMPOSITE BEAM FASTENERS

34-7-5 M Grosse, S Lehmann, K Rautenstrauch
Testing connector types for laminated timber-concrete composite elements

Introduction
The idea of combining the constructional favourable characteristics of concrete and timber is not new. The mainly tension-stressed timber and compression-stressed concrete offer good load-carrying behaviour. The connection between timber and concrete is of fundamental importance for stiffness and losd-carrying performance. Since these systems have usually been realised as timber-concrete composite beams which are mainly taken for bridges or revitalisation of timber beam floors, dowel type connectors were normally studied and taken for the transfer of shear forces.

At the Bauhaus-University Weimar a research program was initiated to develop new connectors for transferring shear forces in laminated timber-concrete-composite plates. In the context of this paper the joints with flat-steel-locks, punched steel sheets and concrete cams are introduced.

An overview of shear tests, performed to examine the load-slip characteristics and bending tests of full sized composite floor elements is also given.

36-7-1 L Jorge, H Cruz, S Lopes
Shear tests in timber-LWAC joints with screw-type connections

Introduction
The use of timber-concrete slabs has been considered as a suitable alternative to traditional timber pavements, in terms of structural behaviour, acoustic and thermal insulation and fire resistance.

Typically these structures are made with normal-weight concrete (NWC), very seldom using light-weight aggregates concrete (LWAC).

The use of LWAC will most likely imply a different performance of the timber-concrete slab, due to the expected different short term and long term behaviour of the joints. Although the accepted failure modes for the joints with NWC would involve fasteners and timber only, it should be checked whether the same is valid when LWAC is used. The interest and relevance of developing further research work on the use of light-weight concrete in composite timber-concrete slabs has been pointed out by several authors.

The lower modulus of elasticity of this concrete, directly related to its lower density, will imply a higher global deformation of the composite structure, which has to be counterbalanced by its lower self weight.
The self-weight reduction has other advantages, like less need for prop, a better fire performance and a higher acoustic insulation to impact sounds as compared to NWC composite slabs.

With the purpose to study the suitability of using LWAC in timber-concrete slabs, composite shear tests on composite specimens were carried out. Special screws were used to connect timber to concrete.

Concluding remarks
The tests carried out indicate that the use of LWAC, in timber-concrete joints without interlayer leads to a strength reduction of the joint capacity but to a similar stiffness than the use of NWC.

On the contrary, in the presence of the interlayer, the use of LWAC conducts to higher slip modulus and to similar strength than the use of NWC. This is particularly promising for repair works since the studied interlayer simulates the floor boards existing in most old buildings that would generally benefit from reduced dead loads.

The modification of the screw (by introducing the washer) did not work as expected. However, the results suggest that this may be of some help in the case of using concrete admixtures even less dense and less resistant than the ones considered in the present study.

For standard screws (no washer), the presence of interlayer did not affect joint strength Although resumed to one type of fastener only, test data suggest that the use of lightweight aggregates concrete may be suitable for timber-concrete composite slabs and perfectly competitive alternative to the use of normal-weight concrete.

36-7-3 S Lehman, K Rautenstrauch
Nail-laminated timber elements in natural surface-composite with mineral bound layer

Introduction
Recently the idea was focused on combining favourable characteristics of mineral surface layer and timber in composite elements. Thereby tension-stressed timber and compression-stressed mineral surface layer, for in-

stance concrete, offer good load behaviour. The connection between timber and concrete is of fundamental importance for stiffness and load- carrying performance. Since these systems have usually been realised as timber-concrete composite beams mainly taken for bridges or revitalisation of timber beam floors, dowel type connectors were usually studied and taken for transfer of shear forces.

A research program was initiated at the Bauhaus-University Weimar, to test natural adhesion and modified timber surface structure for the transfer of shear forces in timber-mineral surface layer -composite slabs. It is a matter of saw-rough and additionally profiled lamellas of nail-laminated timber elements in combination with usual mineral surface layer. An overview of shear tests („slip-block-tests") and tension tests, performed to examine the load-slip is also given. Test data and figures are presented. Based on these test results the behaviour of the connection method is described.

Summary

Aim of the work was to examine by means of experiment the bond behaviour of nail-laminated timber elements and mineral surface layer without additional connecting devices. With the simplest formation of the composite joint, already a typical normal concrete on rigid wood surface, shear stresses can be transferred by $f_v = 0.38$ N/mm^2. The coefficient of friction between wood and concrete with the value $\mu = 0{,}9$ of normal concrete on even nail-laminated timber element well-known in the literature could be observed.

With normal concrete and profiled lamellas the shear stress amounted to $f_v = 0.42$ N/mm^2. The adhesive tensile tests confirmed the realisations of Griffith that the actual tensile strength is smaller than the theoretical tensile strength received by the shear tests. Particularly high tensile strength f_{jt} supplied the nail-laminated timber elements variant of the type P with a concrete surface layer (HZ/BP). The average tensile strength for this variant amounted about 0.18 N/mm^2. Compared with other bonds of material the reached tensile strength approximately corresponds to those of bond by stone on mortar.

36-7-4 A Dias, J W G van der Kuilen, H Cruz
Mechanical properties of timber-concrete joints made with steel dowels

Introduction

The effectiveness of a timber-concrete composite structure is highly dependent on the stiffness of the joints that connect the timber and the concrete. Many different types of joints have been developed and are already used in practice. One of the most used, for its simplicity and availability, are the dowel type fasteners. A research project was started in Coimbra University and Delft University of Technology to improve the knowledge on the mechanical behaviour of these joints. Part of that research project consists of timber-concrete shear tests with dowel type fasteners with different conditions and materials. The results and analysis of those tests are presented. The results are also compared with the values computed using the equations given in EC5 part 1 and part 2.

Conclusions

The results presented here show that the load-carrying capacity of timber-concrete joints is, among others, affected by the compression strength of concrete, a parameter that until now has been disregarded. On the other hand, no indications were found of any influence of the concrete strength on the slip modulus of the joints.

Both slip modulus and load-carrying capacities are affected by the timber density and embedding strength. However, the effect on slip modulus seems to be much higher.

The effect of an interlayer is again higher for the slip modulus than for the load carrying capacity. In both cases the mechanical properties decrease if the interlayer is present.

This research found some indications that the effect of friction of smooth dowels may not be always negligible on these kind of joints, however further research is needed to confirm it.

The models given in the EC5 for the load capacity of these joints seem to be able to evaluate their actual load-carrying capacities. The values from the models are always conservative when compared with the experimental data. Nevertheless these results also show that the properties of the concrete could be considered in the models, giving values with increased accuracy. Considering these results, the load carrying capacity of joints with concretes of higher classes, for instance of strengths of 60 MPa and more, could benefit from an increase of up to 20 %.

The ability of the model presented in EC5 to evaluate the slip modulus also seems to be good. However the values given by those models are always around 20% higher than the ones obtained from tests. That may indicate that the factor 2 given in EC5 part 2 (discarding the deformations of the concrete) is too high on this situation.

37-7-15 M Grosse, K Rautenstrauch
Numerical modelling of timber and connection elements used in timber-concrete composite constructions

Introduction

Experiments are an important basis of civil engineering. Experimental tests are needed to define physical properties of building materials as well as to make reliable statements concerning the structural safety of components considering scattering influences. It has to be noticed, that limited information can be gained from test with conventional measurement techniques. Most often only the applied load, deflections of few points relative to a reference and integral strains at some sections can be determined. In spite of the enormous effort in applying measurement techniques, the flux of forces within the member can not be determined directly. It can only be interpreted with an engineering understanding of the measuring data and the responses of a structure. Furthermore, the information won from the experimental test only apply to the fixed boundary conditions and are not transferable to real installation conditions without limitations. They are two alternatives that improve the meaningfulness of tests:

1. Increasing the concentration of measuring points: This can not be realized with classic mechanical or electronic measurement techniques. Concentration of the measuring points is easily possible using contactless methods, such as the close-range photogrammetry.
2. Accompanying numeric simulation of the tests: The distribution of forces, stresses and strains across the member can be calculated and visualized by verifying a mathematical model based on the experimental results. In the model it doesn't cause any difficulties to vary boundary conditions and material properties. With this procedure it is possible to improve the test programme and also to optimize the examined member.

Summary

A constitutive material model for softwood, based on time-independent multi-surface plasticity was introduced in the paper. This model is able to represent the effects of all nine material-specific micro-mechanical failure modes, mostly arising independently, on the stress deformation-relationship of wood. Furthermore a compression test was presented; determining the structural response to longitudinal compression and investigating the behaviour of contact connections and the fracture energy dissipated during kink-band formation. In addition a material model for concrete, needed to simulate the load carrying behaviour of timber concrete-composite structures was introduced. The efficiency of FE-simulations using these material-models was demonstrated by three examples.

39-7-3 A Döhrer, K Rautenstrauch
Connectors for timber-concrete composite-bridges

Introduction

The construction method using timber-concrete composites has been developed in the field of building engineering. This technology will also be interesting for bridge constructions in the future. Combining concrete in the compression zone with timber in the tension zone to a composite structure, the favourable properties of both materials could be used efficiently. Especially the combination of log-glued laminated beams with concrete slabs will establish new opportunities for the building of road bridges.

Many problems occurring at timber road bridges could be solved by this new type of hybrid timber-concrete composite-bridges. The concrete deck provides an ideal constructive wood preservation. The distribution of high loads per axle and the transmission of horizontal loads can be realised much easier by the concrete deck. Construction details proved in building of concrete bridges and accepted by the national highway administrations could be transferred. In comparison to simple concrete bridges the superstructure is much lighter, so costs in the fields of foundations could be reduced. Using precast concrete units a high prefabrication level could be realised.

The development and use of sufficient stiff connectors between timber and concrete is very important for those composite bridge constructions. EC5 previously only included regulations concerning bolted steel fasteners and grooves. Tests are demanded for other connections. For these tests it is necessary to take the different time-, temperature- and moisture-dependent behaviour of both composite materials into consideration, especially under conditions of service class 2.

At the Bauhaus-University Weimar, short-time shear tests, long-time shear tests and push-out tests under cyclic loading with three different types of connectors were arranged. The paper will present the results of these tests. The decrease of the stiffness influenced by long-term loading under conditions of service class 2 and under cyclic loading will also be discussed. These results could contribute to verify the shear stiffness approach of the connection between timber and concrete being the fundamental construction parameter for this new type of hybrid road bridges.

Discussion and conclusion

The construction method using timber and concrete in a composite action is an expedient alternative to the conventional bridge building. To make this method negotiable further investigations are required in the fields of connectors being suitable for bridge building. The paper focused on systematic tests on three types of suitable shear connectors. Shear tests have been arranged under short-time, long-time and dynamic loading. First the initial values of stiffness and ultimate loads were determined. Then the influences of creeping under conditions of service class 2 and of more than 2 million load cycles were investigated. Based on the experimental research the following conclusions can be drawn:

All tested connectors are suitable for bridge building. They performed superior stiffness and higher ultimate loads in short-time shear tests in comparison to joints being well-known from building construction. Grooves and stud connectors behaved well under fatigue loading exhibiting no loss of ultimate loads and stiffness. Fatigue tests at X-connectors and long-time tests have still been running and will be analysed periodically. However, the presented tests have only been done for orientation. More extensive tests are necessary to get statistically confirmed values of stiffness and ultimate loads. Such tests will be performed with the stud connector at the Bauhaus-University next month.

Finally, the tests results will be verified by a computation model using a complex 3-dimensional nonlinear material model for timber that had been developed at the Bauhaus-University Weimar.

40-7-5 E Lukaszewska, M Fragiacomo, A Frangi
Evaluation of the slip modulus for ultimate limit state verifications of timber-concrete composite structures

The timber-concrete composite structure consists of timber joists and beams effectively interconnected to a concrete slab cast on top of the timber members. This type of structure was developed as an effective method for strength and stiffness upgrading of existing timber floors. Thanks to the several advantages over traditional timber floors such as increased strength and stiffness under gravity load, better seismic resistance, effective acoustic separation and improved fire resistance, the composite structure is also used in new construction.

The structural behaviour of timber-concrete composite members is mainly governed by the shear connection between timber and concrete. Almost all connection systems are flexible, i.e. they cannot prevent a relative slip between the bottom fibre of the concrete slab and the top fibre of the timber beam. As a consequence of that, conventional principles of structural analysis cannot be applied to solve the composite beam. The Eurocode 5-Part 1-1, Annex B provides a simplified calculation method for mechanically jointed beams with flexible elastic connection. This method is based on the approximate solution of the differential equation for beams with partial composite action. As the shear connection is usually characterised by non-linear load-slip relationship, two different slip moduli are considered for design purposes: k_{ser} for the serviceability limit state (SLS) and k_u for the ultimate limit state (ULS) design. The slip modulus k_{ser} which corresponds to the secant value at 40% $k_{0,4}$ of the load-carrying capacity of the connection is usually evaluated by push-out tests according to EN 26891. For the slip modulus k_u, the use of the secant value at 60 % $k_{0,6}$ is recommended. However, if experimental data are unavailable, the Eurocode 5-Part 1-1 suggests the use of the formulae for timber-to-timber connections by multiplying the corresponding values of slip modulus k_{ser} by two. The slip modulus k_u may then be taken as 2/3 of k_{ser}. Depending on the type of connection, this assumption may or may not be adequate, leading in some cases to too conservative or not conservative results. Ceccotti et al. for example, reported a significant (50 %) discrepancy between experimental and analytical properties of the connection, and recommended the use of the actual connection properties obtained from push-out tests for the design of timber-concrete composite systems.

As shear connections markedly affect the efficiency of timber-concrete composite structures, a large number of shear tests on different connection systems as well as bending tests on composite beams have been performed worldwide. Test results can be used in conjunction with finite element (FE) numerical simulations to check whether the hypothesis of assuming the slip modulus for ULS as two-thirds of the slip modulus for SLS is adequate.

The first part of the paper presents the values of the slip moduli $k_{0,4}$ and $k_{0,6}$ as well as their ratios measured in experimental shear tests for different connection systems. The slip moduli are also compared with the analytical values computed using the approximate formulae suggested by Eurocode 5. In the second part of the paper, analytical calculations are compared with bending test results and numerical simulations for different timber-concrete composite beams. The analytical calculations are carried out for loads at SLS and ULS using the formulas in Annex B of Eurocode 5 and the slip moduli $k_{0,4}$ and $k_{0,6}$, respectively. The aim of the analysis is to check whether the use of the aforementioned secant shear moduli leads to accurate results for SLS and ULS design. Finally, based on the values of the ratios calculated from experimental tests, the assumption of the Eurocode that k_u may then be taken as 2/3 of k_{ser} is verified and if necessary improved.

Concluding remarks
The primary conclusions are reported in the following:
- The ratios a between the experimental values of secant slip moduli $k_{0,4}$ and $k_{0,6}$ for ULS and SLS verifications varied between 0.2 and 1.15, showing significant differences from the value of 2/3 recommended by the Eurocode 5 for timber joints.
- The ratio α should be assumed equal to one for notched connection details characterized by linear shear force-relative slip relationship, and in the range 0.9 to 1 for stiff connectors such as inclined screws. The value of 2/3 seems to be appropriate for vertical screws and dowels, even though in some cases it may drop significantly.
- The use of the Eurocode 5 formulae for the evaluation of $k_{0,4}$ and $k_{0,6}$ may lead to significant underestimation (up to 90 %) or overestimation (up to 40 %) of the experimental values.
- The use of the Annex B formulas of Eurocode 5 for composite beam with flexible connections leads to reasonably accurate solutions when the slip moduli at 40 % and 60 % of the shear strength evaluated in shear tests are used. The errors on deflection and stresses at both SLS and ULS design loads, in fact, do not generally exceed 10-20%. The slip and shear forces suffer from a larger error.
- The use in the Annex B formulas of the slip moduli $k_{0,4}$ and $k_{0,6}$ .analytically evaluated leads to larger differences (20-40 %, almost doubled) on deflection and stresses at both SLS and ULS.

Based on these outcomes, it is therefore recommended that the design of timber-concrete composite beams at SLS and ULS be carried out using the experimental values of the slip moduli at 40 % and 60 % of the shear strength, $k_{0,4}$ and $k_{0,6}$. The latter values, in fact, could be significantly different from the value of 2/3 $k_{0,4}$ recommended by the Eurocode 5 for connections between timber members. The analytical formulae for the evaluation of $k_{0,4}$ and $k_{0,6}$ should be used only for preliminary design since they leads to significant differences on the experimental values of slip moduli and, therefore, on the deflection, slip and stresses of the composite beam.

42-7-6 U Kuhlmann, P Aldi
Prediction of the fatigue resistance of timber-concrete-composite connections

Introduction
Lack of knowledge about the fatigue behavior of the interface between timber and concrete may be regarded as one of the difficulties of the further spread of timber-concretecomposite structures (TCC-structures) for the construction of road bridges. The first objective of this study is an analysis of the fatigue strength of TCC-beams, in which the joint between the two single sections of timber and concrete is achieved by either grooved connections or crosswise glued-in rebars. These fastening methods seem to be suitable for bridges due to their relative high stiffness as well as strength values. An investigation about the fatigue resistance of such kind of connections might also help to spread the application of TCC-beams for the construction of small or middle span composite bridges.

Load standards, like EN1991-2 [1] or the German DIN-FB 101, state a number of fatigue verifications for road bridges, which have to be included in the design calculation of bridges. For the timber parts of these structures, certain information about fatigue strength is included in Annex A of EN 1995-2, or in DIN 1074, Anhang C. Within these codes the information is limited to pure timber elements, timber-timber joints and steel-timber connections. There is no information about the fatigue resistance of connections between timber and concrete.

Conclusions
Neither S-N curves nor other analytical means are currently available to predict the lifetime of timber-concrete-composite structures under pulsating load. Two new series of investigations based on shear fatigue tests have been carried out with the aim to obtain a sufficient number of data about the fatigue behavior of the timber-concrete shear interface in composite structures. A first preliminary S-N curve for the notch in grooved

TCC structures, based on symmetrical push-out tests, has been presented. A number of composite beam tests were carried out to confirm the results obtained with the push-out tests. Furthermore a comparison with the rule given in EN 1995-2 indicates the possibility to use the available S-N line for timber shear faituge failure for grooved connections with the investigated geometry. This would lead to a safe prediction of the number of cycles to failure for the connection and will be useful for the fatigue verification of TCC structures in road bridges.

3.6 LATERAL INSTABILITY

ESSAY 3.4 H J Larsen
Lateral instability of beams

High slender beams can fail due to lateral deflection and torsion even when they are loaded in pure bending as shown in Figure 1.

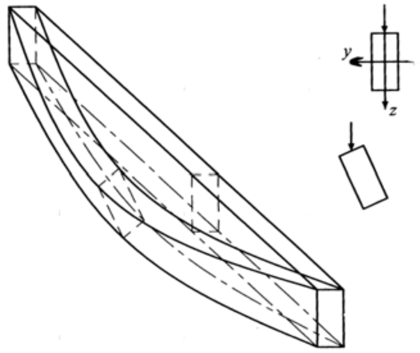


Figure 1. Deflected beam.

To calculate the load-carrying capacity of a straight elastic and simply supported beam, it is assumed that the torsion is prevented at the end supports and that the beam is loaded by two equal bending moments $M = M_y$ at the beam end. Under these conditions the beam will be stable for loads below a critical value $M = M_y$. If this moment is exceeded, the beam will deflect as shown in figure 1 and 2.

In the deflected state there will in addition to the moment about the y-axis be a moment M_z about the z-axis and a twisting moment M_x about the x-axis. Since the angles are small: $\sin\phi \sim \phi$ and $\cos\phi \sim \cos\theta \sim 1$, and

$$M_y = -EI_y \frac{d^2u}{dx^2} = M$$

$$M_z = M\phi = -EI_z \frac{d^2v}{dx^2} \tag{1}$$

$$M_x = -M\theta$$

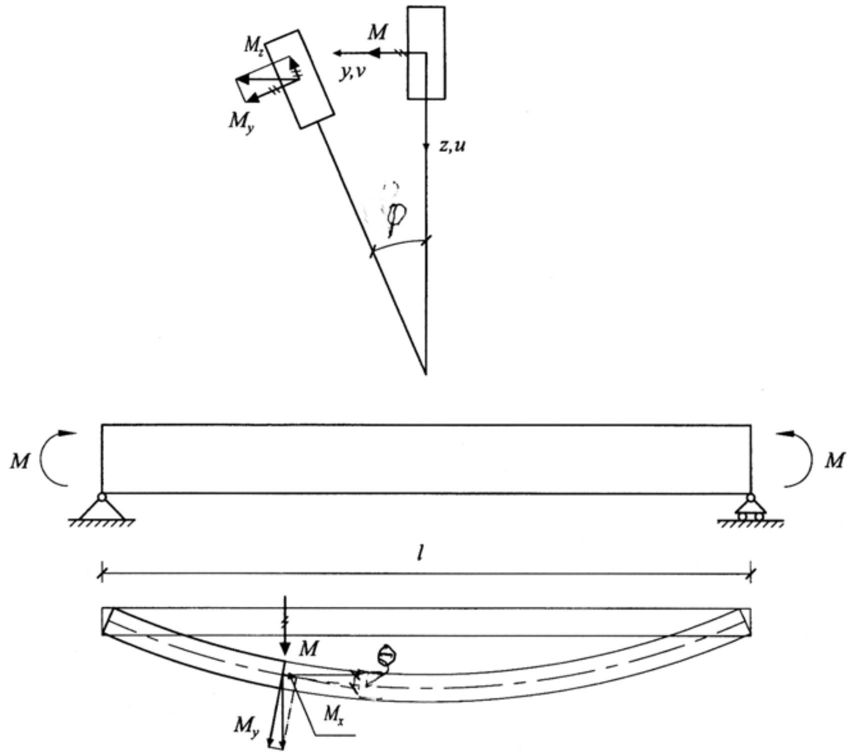


Figure 2. Beam deflected in torsion and lateral deflection instability

The differential equation for twisting (torsion) about the x-axis is:

$$M_x = GI_{tor} \frac{d\phi}{dx} - EI_w \frac{d^3\phi}{dx^3} \tag{2}$$

where
I_{tor} is the torsional moment of inertia
I_w is the warping moment of inertia

By differentiation of (2) and inserting in (1):

$$-\frac{M^2\phi}{EI_z} = GI_{tor} \frac{d^2\phi}{dx^2} - EI_w \frac{d^4\varphi}{dx^4} \tag{3}$$

A solution that fulfils the boundary conditions $\phi = 0$ for $x = 0$ and $x = l$ is:

$$\phi = \phi_0 \sin \frac{\pi x}{l} \tag{4}$$

where ϕ_0 is the twist angle at the beam middle. By inserting (4) in (3) you find:

$$\frac{M^2}{EI_z}\phi_0 \sin\frac{\pi x}{l} = GI_{tor}\phi_0\left(\frac{\pi}{l}\right)^2 \sin\frac{\pi x}{l} + EI_w\phi_0\left(\frac{\pi}{l}\right)^4 \sin\frac{\pi x}{l} \tag{5}$$

giving

$$M = M_{cr} = \frac{\pi}{l}\sqrt{EI_z GI_{tor}\left(1+\left(\frac{\pi}{l}\right)^2 \frac{EI_w}{GI_{tor}}\right)} \tag{6}$$

The last term is only of importance for open thin-walled cross-sections. For the cross-sections common in timber structures $I_W / I_{tor} \sim 0$, i.e.

$$M_{cr} = \frac{\pi}{l}\sqrt{EI_y GI_{tor}} \tag{7}$$

The case with equal end moments is one of the few where there is an analytical solution. In most cases numerical methods, e.g. based on strain energy methods are required. Examples on solutions may be found in e.g. S. P. Timoshenko and J. M. Gere: Theory of Elastic Stability
The solutions may all be written as:

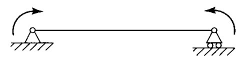
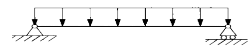
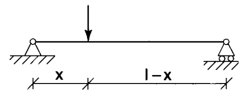
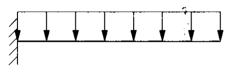
$$M_{cr} = \frac{\pi}{l_{ef}}\sqrt{EI_y GI_{tor}} \tag{8}$$

where l_{ef} is an effective length. Examples on l_{ef} are given in Table 1.

As is apparent from figures 1 and 2, the critical moment depends on the location of the load in the cross-section. The higher the location, the bigger the driving effect and the smaller the critical load. Loads acting below the axis of rotation will even have a stabilizing effect. This is taken into account in Eurocode 5 by reducing/increasing l_{ef} as shown by $0.5h$ and $2h$ respectively.

Lateral stability shall be verified both for M_y alone and for M_y together with an axial compression force N_c

Table 1. Effective length, l_{ef}, as a function of beam length l and beam depth h

	Load acts in		
	the bottom of the beam	the centerline	the top of the beam
		l	
	$0.9l - 0.5h$	$0.9l$	$0.9l + 2h$
 $\alpha = 4\frac{x}{l}\left(1-\frac{x}{l}\right)$	$0.8\alpha l - 0.5h$	$0.8\alpha l$	$0.8\alpha l + 2h$
	$0.6l - 0.5h$	$0.6l$	$0.6l + 2h$

For M_y alone, it shall be verified that

$$\sigma_m \leq k_{crit} f_m \tag{9}$$

where:
- σ_m is the bending stress
- f_m is the bending strength
- k_{crit} is a factor that takes account of the reduced load-carrying capacity when failure is caused by lateral instability

The critical bending stress may be found from (8) as:

$$\sigma_{m,crit} = \frac{M_{y,crit}}{W_y} = \frac{\pi\sqrt{EI_z GI_{tor}}}{l_{ef} W_y} \tag{10}$$

where:
- W_y is the section modulus.

For solid softwood, $\sigma_{m,crit}$ is approximately:

$$\sigma_{m,crit} = \frac{0,78 b^2}{h\, l_{ef}} E_{0,05} \tag{11}$$

Acccording to Eurocode 5

$$k_{crit} = \begin{cases} 1 & & \lambda_{rel,m} \leq 0,75 \\ 1,56 - 0,73\lambda_{rel,m} & \text{for} \quad 0,75 < & \lambda_{rel,m} \leq 1,4 \\ \dfrac{1}{\lambda_{rel,m}^2} & 1,4 < & \lambda_{rel,m} \end{cases} \tag{12}$$

where the relative slenderness $\lambda_{rel,m}$ has been introduced by

$$\lambda_{rel,m} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}} \tag{13}$$

In (12) the expression in the first line is the cross-section strength, the last expression is the instability strength according to the theory of elasticity and the middle expression is an interpolation proposed and verified in R F Hooley and B Madsen: Lateral instability of glued laminated timber beams, Journal of the Structural Division , ASCE, Vol. 90, No. ST3, 1964.

4-2-2 B Johannesson
Lateral-torsional buckling of eccentrically loaded timber columns

See 3.4 Columns

5-10-1 H J Larsen
The design of timber beams

See 3.16 Timber beams, general

19-2-1 R H Leicester
Creep buckling strength of timber beams and columns

See 3.4 Columns

21-2-2 R H Leicester
Beam-column formulae for design codes

See 3.4 Columns

29-10-1 F Rouger
Time dependent lateral buckling of timber beams

Introduction.
In **Paper 27-20-4**, Le Govic et al reported the results of a research program dealing with long term behavior of timber beams (solid timber and glulam). All along this program, beams had been tested in external protected conditions at three stress levels (2 MPa, 5 MPa, 15 MPa) corresponding to three average stress ratios (10%, 20%, 60%) with respect to the average short term characteristic strength. Whereas for low stress ratios (10%, 20%) the beams did not fail, a high percentage of failures due to lateral instabilities was observed at the highest stress level (60%). These percentages ranged from 50% for solid timber to 100% for glulam. A simple reliability calculation based on Eurocode 5 formulas lead to equivalent probabilities of failure ranging from 10% for solid timber to 25% for glulam. These calculated values were obviously much too low and justified further investigation.

In this paper, finite elements analysis combined with Eurocode 5 concepts have been done. In a first part, Timoshenko formulas are compared with finite elements results. They show a very good accordance and justify the use of finite elements concept for further analysis. In a second part, critical stresses are calculated for the tested specimens. They lead to the evaluation of probabilities of failure which are compared with the experimental values. In a third part, Eurocode 5 formulas are analysed and a new design proposal is given.

Conclusions.
A numerical method has been compared with the classical theory of Timoshenko and shows a very good accordance. It was therefore applied for deriving critical moments and subsequent critical stresses. From these results, probabilities of failure have been calculated and show a very good agreement with the experiment. EC5 equations have been investigated. A revision is proposed in two points
(1) Calculate the relative slenderness with the time dependant critical stress,
(2) Use another equation for k_{inst} versus λ_{rel},

This approach is easily applicable to columns behavior. In §5.2.1, application rule (2), equations (5.2.1c) and (5.2.1d), the MOE values should be divided by $(1 + k_{def})$.

By adopting this approach, the long term instability problems can be easily solved.

3.7 SYSTEM EFFECTS

ESSAY 3.5 H J Larsen System effects in Eurocode 5

Eurocode 5 states:

6.6 System strength
(1) When several equally spaced similar members, components or assemblies are laterally connected by a continuous load distribution system, the member strength properties may be multiplied by a system strength factor k_{sys}.
(2) Provided the continuous load-distribution system is capable of transferring the loads from one member to the neighbouring members, the factor k_{sys} should be 1,1.
(3) The strength verification of the load distribution system should be carried out assuming the loads are of short-term duration.

NOTE: For roof trusses with a maximum centre to centre distance of 1,2 m it may be assumed that tiling, battens, purlins or panels can transfer the load to the neighbouring trusses provided that these load distribution members are continuous over at least two spans, and any joints are staggered.

(4) For laminated timber decks or floors the values of k_{sys} given in Figure 6.12 should be used.

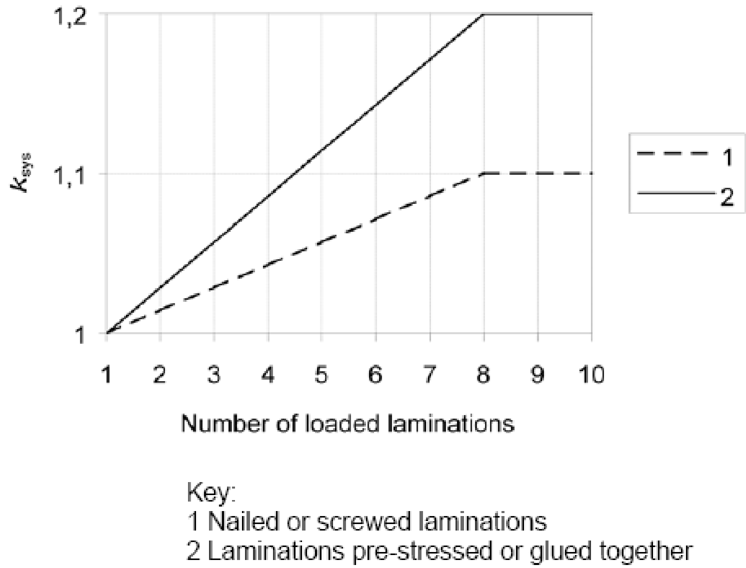


Figure 6.12 – System strength factor k_{sys} for laminated deck plates of solid timber or glued laminated members

* * *

There are 3 reasons why the strength of systems is increased.

– Initial failure in a cross-section or a joint may be counteracted or stopped because battens, laths and other secondary elements due to resulting deflections transfer part of its load to the neighbouring structures
– In statically indeterminate structures there is a possibility of load redistribution from weak to stronger elements.
– In e.g. trusses the moment distribution is characterised by localised moment peaks at supports and nodes. The probability that moment peaks and weak sections, e.g. due to knots, coincide is small.

Although it is unlikely that the secondary elements really are able to transfer a considerable part of a load on an element to the neighbouring elements, especially the situation mentioned in the NOTE is very unrealistic for trusses because of their high stiffness, Eurocode 5 takes only effect 1 into consideration and disregards effect 2, that is probably the most important and reliable effect.

22-8-1 F Rouger, J D Barrett, R O Foschi
Reliability analysis of viscoelastic floors

Introduction

This paper summarizes results of a project activity undertaken at the University of British Columbia (Canada) in cooperation with the Centre Technique du Bois et de 1'Ameublement (France). The objective of this study is to analyze the effect of joist and panel viscoelastic behaviour on reliability of wood structural systems. Since previous work has not considered the influence of creep on structural performance, this study was initiated with the objective of undertaking initial studies of the influence of creep behaviour on systems performance.

Recent studies by Foschi et al have shown that wood structural systems designed according to present codes have a somewhat lower average safety level that rnight have been anticipated considering the performance record. Load sharing factors are incorporated into wood structural design codes to account to the addition safety developed by the structural interaction of parallel members in systems such as floors roofs and trusses. The additional safety develops in part due to the redistribution of framing member stresses and due to the interaction between framing members and the panels (or cover) material and the between member variation in modulus of elasticity. Studies of the elastic behaviour of redundant member wood structures have shown that the correlation between strength and modulus of elasticity of wood members contributes to a redistribution of stresses, in which the weaker member will be subjected to lower stresses and the stronger members will have higher than average stresses

Foschi et al investigated the long-term behaviour wood roof systems subjected cyclic snow loads System time-to-failure was calculated using damage accumulation models. Damage accumulation under simulated snow loads was evaluated using member stresses derived from an elastic stress analysis The time-to-failure distribution derived from the analysis formed the basis for system reliability studies using First Order Reliability Methods.

The creep of system framing members and panels product will affect the member and panel stresses. The purpose of this paper is describe a series of analysis undertaken to assess the impact of viscoelastic behaviour of roof joist and sheathing stresses for a roof system subjected to constant and cyclic snow loads.

Conclusion

Considering linear viscoelastic behaviour in floor calculations leads to some remarks

- probabilities of failure are not affected
- times-to-failure might be reduced by 10-20%
- bending stresses are not affected.
- deflection amplification factors are large and even larger than predicted by the codes. This should be taken into account in further calibrations.

The calculation required data for both lumber and panels had to be adapted from many different sources to generate the required input files. If viscoelastic studies should be pursued, it means that much more effort must be directed to obtaining the viscoelastic material property data for wood products.

The Monte-Carlo simulation was limited to a small number of replications, the number of joists is small (3), the duration of load is short (10) years. All these limitations restrict the interpretation of the results: Additional studies would need to be completed to establish the general applicability of the conclusions derived.

Even with these limitations we must remember that a few years ago this study could not have been performed. The evolution of computers is so fast that in near future more extensive studies will be possible and work should be continue develop analytical tools and material property data required for viscoelastic behaviour studies.

25-10-2 T D G Canisius
The influence of the elastic modulus on the simulated bending strength of hyperstatic timber beams

Abstract

In the design of a timber beam, its strength is assumed to be constant along the length and to be equal to the characteristic strength. This gives rise to an increase in the safety level of the member. In order to find this increase in safety, it is necessary to find the strengths of beams under different loading conditions. In the case of statically determinate beams, the structural analysis is not affected by the variations in the elasticity modulus. The importance of considering the lengthwise variation of the elasticity modulus in calculating the strengths of hyperstatic beams is investigated in this paper. The multivariate approach of Taylor and Bender is used in gen-

erating beam properties. Finite elements are used for stress analysis. Subject to the assumptions made, it is shown that in simulating the beam strengths, the correlated elasticity modulus needs to be considered in both the generation of properties and the analysis of problems if the desired accuracy is high.

Conclusion

Details of Monte Carlo simulations carried out on several beams to determine their bending strengths were presented in this paper. The beam properties were generated using the multivariate approach of Taylor and Bender. First order Markov processes were assumed for the strengths and the elasticity moduli of the property elements along the beam length. The hyperstatic structures were analysed with a beam finite element programme.

Some of the beam property parameters were varied during the analyses to see their effect on the determined strengths. Also four types of analyses were carried out on the beams. These types depended on whether the lengthwise variation of the elasticity modulus was considered in the analysis or not and on whether the cross-correlation between the strength and stiffness was considered in the generation of beam properties or not.

The results indicate that the non-consideration of the cross-correlation between the strength and stiffness in property generation and/or the non-consideration of the elasticity modulus in structural analysis can give rise to large differences between the simulated strengths of hyperstatic beams. For example, for the clamped beam tinder a central concentrated load, the provided results show a maximum difference of 15.3% when the cross-correlation is not considered and the beam is assumed to be of uniform stiffness. This difference is with respect to an analysis that considered the non-uniformity of E and its cross-correlation with the strength. Analyses of some continuous beams showed that the differences between different types of analysis depend on both the structure and the applied loads.

Subject to the assumptions made, it can be concluded that the non-consideration of the cross-correlation coefficient and the lengthwise variation of the elasticity modulus in the generation of beam element properties and the analysis of hyperstatic structures can result in errors as large as almost 15% in their simulated bending strengths. As the effect is negligible when the bending moment is constant along the length of a beam, a similar error can occur in the Moment Configuration Factor for a hyperstatic structure.

34-8-1 M Hansson, T Isaksson
System effects in sheathed parallel timber beam structures

Abstract

The paper presents a study on the system effect of sheathed timber structures, such as roof elements. For such structures Eurocode 5 gives a system factor of 1.1. Using Monte Carlo simulations together with input from an experimental investigation of sheathed parallel timber beam structures, a parameter study of the system behaviour is performed. Parameters such as number and length of beams, type of connection between beam and sheathing and the variability in bending strength within and between members of the system are taken into account.

Systems are generated using Monte Carlo simulations and the system effect is evaluated by direct comparisons of strengths and by reliability methods, resulting in a calibration of the parameters in the code.

Conclusions and further work

The present study uses a statistical model of variability of bending strength within and between beams to investigate the magnitude of the system effect for sheathed parallel timber beam structures.

Limited experimental investigations have shown that the failure criterion for a system corresponds to the failure of two neighbouring or any three beams.

The connector between the beam and sheathing influences the behaviour of the system in such away that a nailed connection gives a higher coefficient of variation of the system strength than a glued connection.

Compared to single beams where the coefficient of variation is around 20 %, the variation in strength of a system is around 10 %. The parameter study showed that the COV for the system is more or less independent of the studied parameters.

When calibrating the partial coefficient related to the material properties, the low COV for the system results in a low γ_m. For a typical ratio between variable load and total load, the results indicate that the system effect factor k_{sys} can be set equal to 1.15. This is based on the difference in COV for the load carrying capacity for the single solid timber beam and the structural system.

In the calibration of the system effect factor the reliability index for the system is set to 4.3. However in the code the reliability index is defined as the safety of the single member in this case the solid timber beam.

35-8-1 M Hansson, T Isaksson
System effects in sheathed parallel timber beam structures part II

Abstract
This is a continuation of **Paper 34-8-1**

In Eurocode 5 a factor of 1.1 is given for such systems. In the present study Monte Carlo simulations are used to generate systems and to evaluate the influence of different parameters on the system behaviour. The variability within and between timber elements is accounted for using a model by Isaksson. The properties of the sheathing are assumed to be deterministic and the property of the joint between beam and sheathing is varied. Reliability theory is used to evaluate the system effect. For a given reliability of a single member the reliability of a system is calculated. The reliability of a system is in general lower than for a single member. The study showed a system effect somewhere between 1.2 and 1.3, i.e. the load carrying capacity of a system is 20 to 30 % higher compared to the capacity of a system when it is evaluated on a single member basis.

Conclusions
The codes generally accept that a system will have a lower reliability for the overall safety compared to the reliability of a single member. Using first order reliability method the reliability of a system based on the design of a single T-section can be compared to the reliability of a system using load sharing. Monte Carlo simulations are used to determine the load carrying capacity of single beams and systems. The parameter study shows that the different parameters only have minor influence on the system effect.

A general method based on reliability theory was used to evaluate the system effect factor. To quantify the effect, the statistical distribution of the variables in the failure function and the partial coefficients for the loads must be known. Using the Swedish code the system effect factor was found to be between 1.19 and 1.31. A simpler way to evaluate the system effect is to determine the ratio between failure load of a system and the weakest T-section in each system. For the 5th percentile and the mean value the system effect is in the interval 1.19-1.30 and 1.14-1.21 respectively. This can be compared to a factor of 1.1 given in Eurocode 5.

39-8-1 I Smith, Y H Chui, P Quenneville
Overview of a new Canadian approach to handling system effects in timber structures

Abstract
In Canada ideas are being formulated about how system design can be implemented within the national timber design code. Momentum for this comes from need to address the changing nature of the construction industry and allied professional engineering practice. The end product needs to be something that can be seamlessly transitioned into design practice. System design is not envisaged as an approach that will yield unfamiliar solutions for familiar problems. Rather it is seen as supporting creation of timber solution in new situations. This paper attempts to identify the major framework issues, starting with the question of what the term system should mean. There is no attempt to follow faddish trends that fly under the same label. It is thought that landscape level changes will shape system design in Canada, and presumably elsewhere, and will concern evolution of architectural solutions, rapid broadening in the available range of structural products (wood, non-wood and composite varieties), changing construction methods and practices, and evolution in regulatory regimes. Detailed technical issues such as what constitute acceptable 'search tree' and 'structural reliability' algorithms will need to be addressed at some point, but that should be after framework issues have been resolved.

Concluding Comments
The authors do not imagine that they have identified all the relevant issues, nor do they imagine that the approaches suggested will meet with universal support even in Canada. They hope that this "Aunt Sally" will promote debate on issues surrounding the introduction of systems level design into timber design codes. Not debating this creates the risk that such documents become obsolete and that a changing world passes them by.

43-12-3 M Frese, H J Blass
System effects in glued laminated timber in tension and bending

Introduction
In 2005, bending tests on full size glulam beams showed that the bending strength values obtained, were far too low compared with the current product standard. For that reason, a new research on glulam was carried out in-

volving a series of numerical studies on the glulam bending strength. This research resulted in a new strength model for the bending strength. The current paper aims at giving a comprehensive overview on recent developments in simulating glulam strength parallel to grain. These developments have arisen from the new research on glulam and a failure analysis on timber hall structures and now cover the tensile-to- bending-strength ratio as well as the size effect in members subject to tension and the load-carrying capacity of simple and continuous beams with up to 5 supports. Therefore, the current paper is a direct continuation of the author's CIB-W18 paper in 2008.

In general, the strength hereinafter refers to characteristic glulam bending ($f_{m,g,k}$) and characteristic glulam tensile strength ($f_{t,0,g,k}$), respectively. Both values are associated with standardised member sizes: 0.6 m times 10.8 m ($h \cdot \ell_m$) in case of bending tests and 0.6 m times 5.4 m ($h \cdot \ell_t$) in case of tensile tests (Fig. 1).

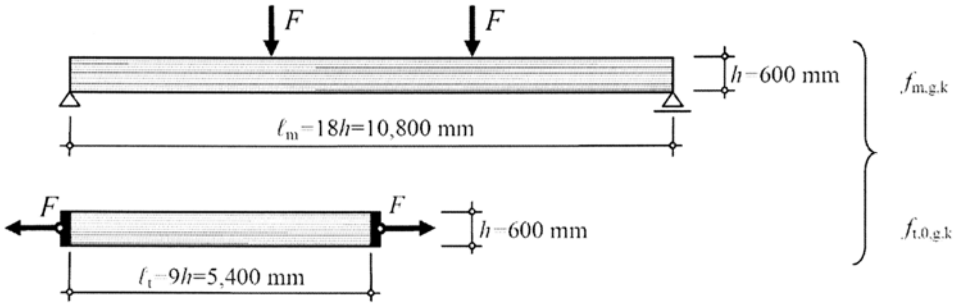


Fig. 1 Test configurations in European test standard EN 408 (2003) with the terms of European standard EN 1194 (1999); bending test (top) and tensile test (bottom).

Conclusions

- Comparative simulations of tensile and bending tests were performed to revise the tensile-to- bending-strength ratio. According to the study the tensile-to-bending-strength ratio is 0.88. The ratio applies to characteristic values of homogeneous glulam, is independent of the material strength and refers to standardised specimen dimensions for tensile and bending tests. The new found ratio is about 25 % higher than the value of about 0.70 in the European standard EN 1194 (1999). This difference may benefit the glulam industry by increasing the reference value for the characteristic tensile strength.
- A wide range of length-depth-combinations was considered for simulated tension members to create a database describing the size effect in glulam members subject to tension. In the study, the length but not the depth was found to be the dominant factor affecting the tensile strength. The influence of a member depth between 120 and 600 mm on the tensile strength is negligible. Based on Weibull's theory, a non-dimensional length factor was obtained to fit the tensile-to-reference-tensile-strength ratio for characteristic values. The ratio decreases rapidly with increasing length for short members, meaning the tensile strength shows a strong dependence on the length of short members. For a quasi-infinite member length the size effect wears off. The effective strength of a tension member 150 mm in length is about 140% of the reference value whereas the remaining tensile strength for quasi-infinite member length is approximately 80% of the reference value. The consideration of a length-dependent glulam tensile strength may increase economical design of short tension members and the reliability of very long tension members.
- Comparative simulated bending tests on simple, 2-span, 3-span and 4-span beams show that the load-carrying capacity of continuous beams is 30 - 40% higher than of comparable simple beams. In regard to the design of continuous beams the effective (or apparent) characteristic bending strength of 2-span beams is 25%, of 3-span beams 8% and of 4-span beams 15% higher than the corresponding characteristic bending strength referring to simple beams. The 25% increase in case of 2-span beams is in good agreement with analytical results based on Weibull's theory [8][9]. In contrast to the current rule in the German timber design code DIN 1052 (2008), where a 10% increase is allowed on the basis of moment redistribution, the findings of the present study may allow a more economical and consistent design of continuous beams.

43-12-4 C Faye, F Rouger, P Garcia
Experimental investigations on mechanical behaviour of glued solid timber

Context and objective:

Glued Solid timber (GS) beams are structural elements with rectangular cross section formed by bonding together two, three, four or five laminations having a thickness between 45 and 85 mm. Laminations can be finger jointed.

GS beams are used in traditional carpentry, timber frame housing, or as bracing elements, etc. Even if this product is more expensive than solid timber, the use of GS beams is increasing due to the following advantages: a better dimensional stability, moisture content around 12% due to the manufacturing process, cross section until (260*320) mm2, directly compatible with CNC machines.

Because their cross section is close to squared section (maximum cross section is L×H ≤ 260mm × 340 mm), GS beams can be loaded in flatwise (load is perpendicular to glue lines) or edgewise (load is parallel to glue lines) bending (cf. figure1)

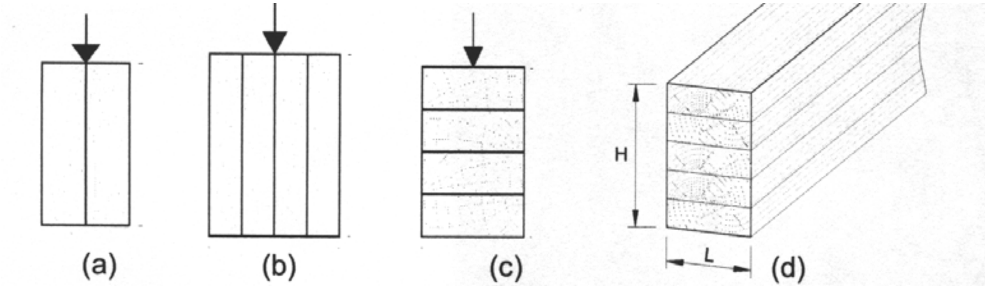


Figure 1: GS beams with two (a) and four (b) laminations in edgewise bending, GS with four (c) or five (d) laminations in flatwise bending, H×L < 260mm×320mm .

In flatwise bending, due to the fact that GS beams are composed of few laminations with large thickness, their mechanical behaviour is different from glulam. Actually, laminations in GS beams loaded in flatwise bending are not only stressed in tension but both in tension and bending. In edgewise bending, the mechanical behaviour involves system effect: laminations with higher stiffness are more stressed. Since these laminations also have higher strength, this results in a composite product with higher strength.

In this context, the aim of this paper is to determine mechanical performances of GS beams and to propose beam lay ups for GS beams based on an experimental study carried on 180 beams in Spruce and Fir and Douglas Fir (in flatwise and edgewise bending) and on bending tests on finger joints and laminations. This work is the first part of research project of 2 years. These experimental investigations will be completed by additional tests on GS beams with 4 and 5 laminations and a probabilistic finite element model taking into account the variability of the mechanical properties of the laminations (presented in a future paper).

Conclusions

Based on an experimental study carried on 180 GS beams, strength and stiffness were determined in flatwise and edgewise bending.

In edgewise bending, system effect induces an increase of strength for GS beams in comparison with laminations. This increase is less significant for high strength classes due to the weak strength of the finger joints tested in this study. System factor obtained from experiments is larger than the one given by Eurocode 5, namely k_{sys}=1,03 for GS beams with 2 laminations and k_{sys}=1,06 for GS beams with 3 laminations. In flatwise, the correspondence between the strength of laminations and the strength of GS beams is given for a reference depth of 150 mm and a depth factor of 0,1, and it is similar than the correspondence existing for glulam with a reference depth of 600 mm and a depth factor of 0,1.

The beam lays-up presented in paragraph 5 are actually discussed into working group WG3 of CEN TC124 in charge of the revision EN 14 080 which will integrate Glued Solid timber beams.

This first study will be completed by:

- Additional tests on GS beams with four and five laminations,
- A probabilistic finite element model taking into account the variability of the mechanical properties of the laminations and finger joints.

3.8 MECHANICALLY JOINTED BEAMS

ESSAY 3.6 H J Larsen Mechanically jointed beams

Mechanically jointed built-up members were used to some extent until about 1950 but are now completely replaced by glued members, e.g. glulam and light I-beams with webs of panel materials. The reason why there is still interest in the topic in CIB W18 is that the same theory applies to composite T-members with wooden webs and concrete flanges, both in new structures (e.g. bridges) and especially in old buildings where a new concrete slab on top of the existing beams can ensure an upgrading of the strength and also the fire resistance.

Figure 1 shows a symmetric, simply supported beam with two members (lamellas).

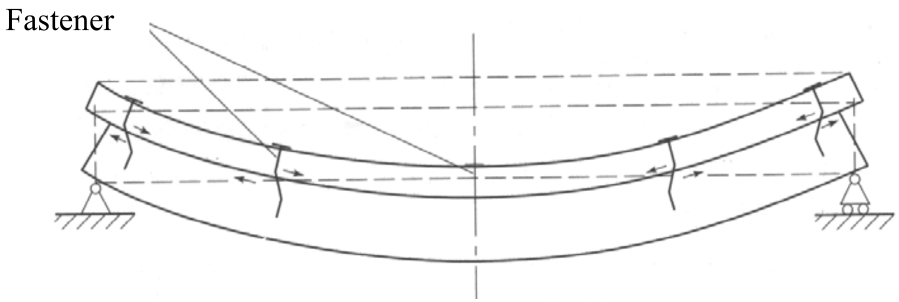


Figure 1. Simply supported composite (built-up) beam).

To transfer load between the lamellas there must be a slip in the joint, increasing from zero in the middle (due to symmetry) to a maximum value at the ends. The result is compression stresses in the top and tension in the bottom taking up any beam moment. The stiffer the connection, the more effective becomes the transfer between the two parts. If the connection is very stiff (glue), the situation corresponds to a solid beam. If the connection is very flexible, the strength and stiffness correspond to the sum of the members.

It is possible, see e.g. **Paper 3-2-1**, to derive relatively simple expressions for a general cross-section as shown in Figure 2, but in practice only T-cross-sections as shown in Figure 3, with 2 lamellas placed on top of each other are used and only these cross-sections will be dealt with in de tail in the following. However, at the end the Eurocode three expressions for I-cross-sections is derived.

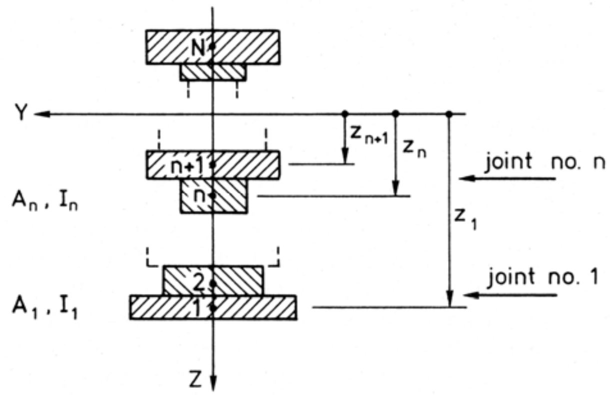


Figure 2. Cross-section with N lamellas.

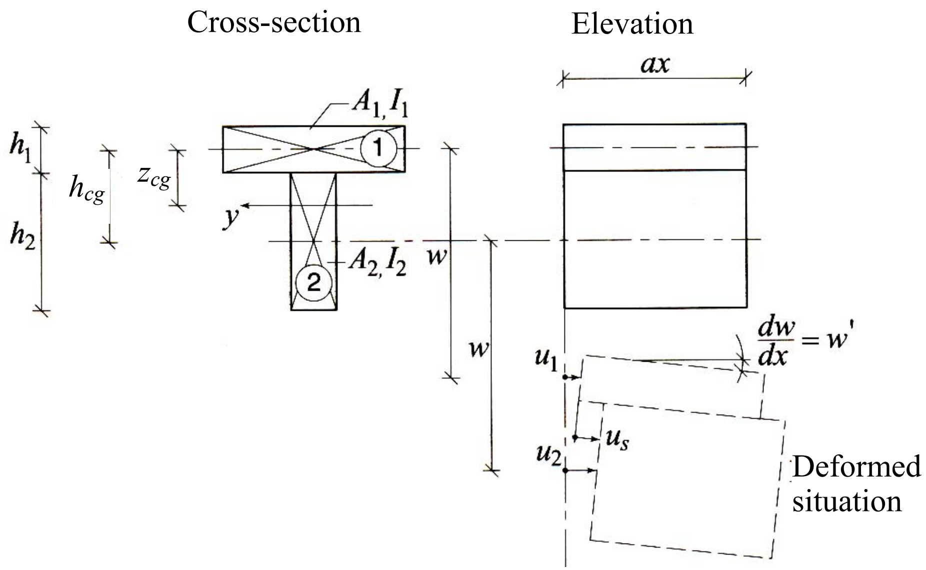


Figure 3. T- Cross-section.

T-cross-section

It is assumed that the lamellas are linear elastic. The lamination areas are A_1 and A_2. The second moments of area (moments of inertia) about there own centres of gravity are I_1 and I_2. If the modulus of elasticity vary, the theory applies if E_1 is taken as reference and the following geometrical values are used:

$$A_1,\ \frac{E_2}{E_1}A_2,\ I_1 \text{ and } \frac{E_2}{E_1}I_2. \tag{1}$$

The theory will be set up both for fasteners having a linear-elastic load-slip curve and for fasteners having a linear-elastic/ideal-plastic behaviour.

The centre of gravity is placed at

$$z_{cg} = h_{cg}\frac{A_2}{A_1 + A_2} \tag{1}$$

The total geometrical moment of inertia is

$$I = I_1 + I_2 + A_1 z_{cg}^2 + A_2\left(h_{cg} - z_{cg}\right)^2 = I_0 + h_{cg}^2 A_r \tag{2}$$

where

$$I_0 = I_1 + I_2 = \beta^2 I \tag{3}$$

$$A_r = \frac{A_1 A_2}{A_1 + A_2} \tag{4}$$

The deformation of the beam is described by the three translations u_1, u_2 and w where

u_1 translation in the beam direction of the centre of gravity of lamella number 1

u_2 translation in the beam direction of the centre of gravity of lamella number 2

w translation perpendicular to the beam axis (the same for both lamellas).

The strains are ('= differentiation with regard to x):

$$\varepsilon_1 = u_1' \text{ and } \varepsilon_2 = u_2' \tag{5}$$

The curvature is

$$\kappa = -w'' \tag{6}$$

For small values of w''

$$u_s = u_2 - u_1 + h_{cg} w' \tag{7}$$

or by differentiation

$$u_s' = u_2' - u_1' + h_{cg} w'' \tag{8}$$

u_s is the slip in the joint between lamella 1 and 2 taken positive as shown in Figure 3.

For elastic materials

$$N_1 = EA_1 u_1' \quad \text{and} \quad N_2 = EA_2 u_2' \tag{9}$$

$$M_1 = -EI_1 w'' \quad \text{and} \quad M_2 = -EI_2 w'' \tag{10}$$

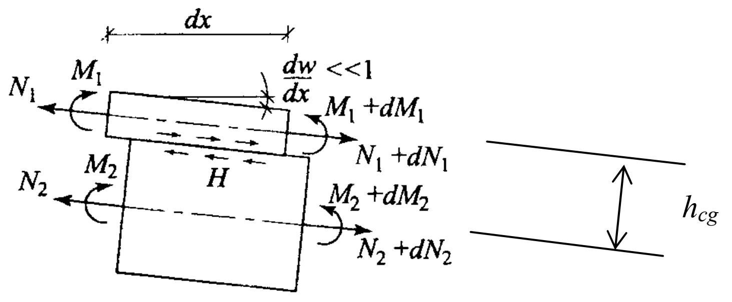


Figure 4. Forces and moments in the deformed situation.

Since there is no external axial force, equilibrium leads to:

$$0 = N = N_1 + N_2 = EA_1 u_1' + EA_2 u_2'$$

$$u_2' = -\frac{A_1}{A_2} u_1' \tag{11}$$

Moment equilibrium for the total cross-section:

$$M = M_1 + M_2 - h_{cg} N_1 = -E\left(I_1 + I_2\right) w'' - h_{cg} EA_1 u_1' = EI_0 w'' - h_{cg} EA_1 u_1' \tag{12}$$

Equilibrium for Lamella 1:

$$Hdx + N_1 + dN_1 - N_1 = 0$$

$$H = -N_1' \tag{13}$$

H is the shearing force per unit length.

Elastic behaviour of fasteners

With a fastener spacing of a, the load on one fastener is Ha and with a fastener stiffness K:

$$Ha = Ku_s \quad H = \frac{K}{a} u_s \tag{14}$$

Inserting (14) and (10) differentiated in (13) gives

$$u_1'' = -\frac{K}{aEA_1} u_s \tag{15}$$

The marked expressions (8), (11), (12) and (15) are the basic expressions from which the deflection w may be determined.

The following equation is found.

$$w'''' - \left(\frac{\gamma}{\beta}\right)^2 w'' - \frac{1}{EI_0}\left(q + M\gamma^2\right) = 0 \tag{16}$$

with

$$\gamma^2 = \frac{K}{EA_r a} \tag{17}$$

A_r is defined in (5).

Example

Simply supported beam with sinusoidal load

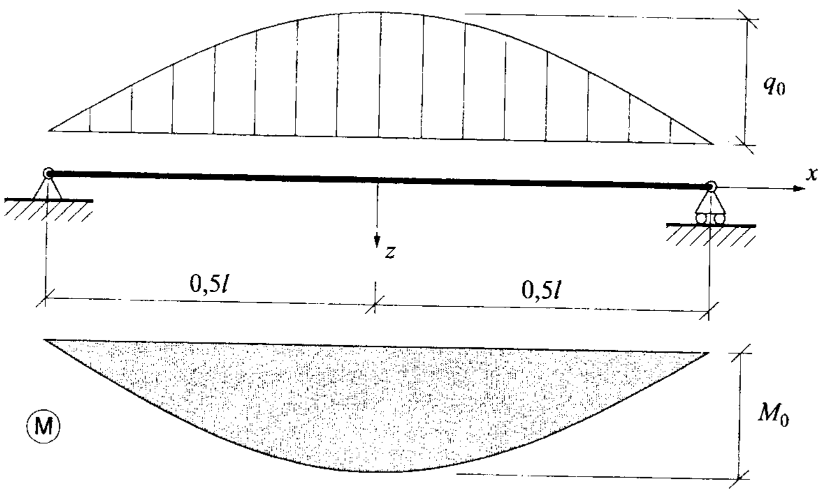


Figure 5. Simply supported beam with sinusoidal load.

The moment from the load $q = q_0 \cos\frac{\pi x}{l}$ is

$$M = \frac{q_0 l^2}{\pi^2}\cos\frac{\pi x}{l} = M_0 \cos\frac{\pi x}{l} \tag{18}$$

By (16)

$$w(x) = \frac{M_0}{EI}\left(\frac{l}{\pi}\right)^2 \frac{1+\left(\frac{\pi}{l\gamma}\right)^2}{1+\beta^2\left(\frac{\pi}{l\gamma}\right)^2}\cos\frac{\pi x}{l} = \frac{1+\mu}{1+\beta^2\mu} w_0(x) \tag{19}$$

where

$$\mu = \left(\frac{\pi}{l\gamma}\right)^2 = \frac{\pi^2 EA_r a}{l^2 K} \tag{20}$$

From this it is seen that

$$\frac{w}{w_0} = \frac{w'}{w'_0} = \frac{w''}{w_0''} = \frac{w'''}{w_0'''} = \frac{1+\mu}{1+\beta^2\mu} \tag{21}$$

The effective moment of inertia is defined as

$$I_{ef} = I\frac{1+\beta^2\mu}{1+\mu} = I_0 + \left(I - I_0\right)\frac{1}{1+\mu} \tag{22}$$

By using the effective moment of inertia, the deflections may be found by the usual methods from the theory of elasticity.

$1/(1+\mu)$ may be regarded as an effectivity factor k_{ef}. For completely stiff fasteners $\mu = 0$, i.e. $k_{ef} = 1$. For very flexible fasteners $\mu = \infty$, i.e. $k_{ef} = 0$.

Plastic behaviour of fasteners

It is assumed that that the load-slip curve is linear elastic-stiff plastic. For structural reasons fasteners are in practice placed over the full beam length, however with a concentration at the length Δl near the ends where the slip is biggest. The fasteners over the rest of the beam length are on the safe side disregarded.

It is assumed that the fastener spacing over the length Δl is constant and that the slip at least corresponds to the yield slip u_y, i.e the load per fastener is R_y . The shearing force per unit length is $H_y = R_y /a$ where a is the spacing.

Examples
A simply supported beams with uniformly distributed load built up of two members jointed by elastic-plastic fasteners with spacing a and yield load R_y is regarded.

$$M = 0,5qx\left(l - x\right) \tag{23}$$

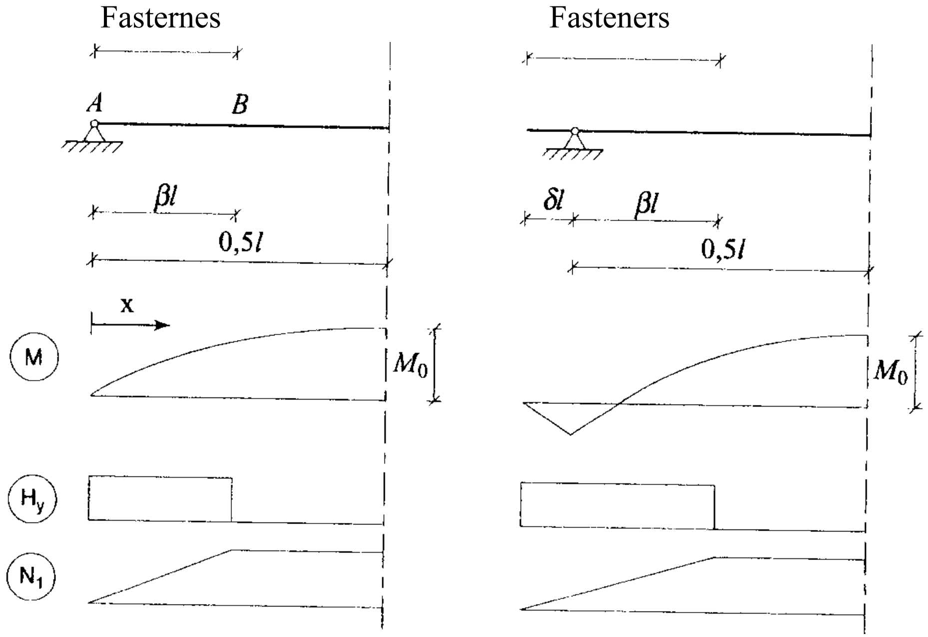


Figure 6. Simply supported beam with constant uniformly distributed load and plastic fastener over the length βl (left) or (δl + βl) (right).

No cantilever, *δl = 0*
See Figure 6, left.

$$0 \le x \le \beta l \qquad N_1 = -\frac{x}{a} R_y \tag{23}$$

$$\beta l \le x \le 0,5l \quad N_1 = -\frac{\Delta l}{a} R_y \tag{24}$$

The normal stresses in the laminations are found from the axial forces N_1 and $N_2 = - N_1$ (Note that N_1 is negative for downward load) and the moments in the laminations

$$M_1 = (M + N_1 h_{cg}) I_1 / I_0 \tag{25}$$

and

$$M_2 = (M + N_1 h_{cg}) I_2 / I_0 \tag{26}$$

The slip is found by (8):

$$u_s' = u_2' - u_1' + h_{cg} w'' = N_1 \left(\frac{1}{EA_1} + \frac{1}{EA_2} + \frac{h_{cg}^2}{EI_0} \right) - \frac{M}{EI_0} h_{cg} \tag{26}$$

Integrating and using

$u_s = 0 \quad$ for $x = 0,5l$

$u_s^+ = u_s^- \quad$ for $x = \beta l$

$$\text{A:} \quad u_s = \frac{l^2}{24E} \left(\frac{h_{cg}}{I_0} ql - 12 \frac{R_y / a}{A_r \beta^2} (1 - \beta) \beta \right) \tag{27}$$

$$\text{B:} \quad u_s = \frac{l^2}{24E} \left(\frac{h_{cg}}{I_0} ql \left(1 - 6\beta^2 + 4\beta^3\right) - 12 \frac{R_y / a}{A_r \beta^2} (1 - 2\beta) \beta \right)$$

It shall be verified that

$$u_y \le u_s \le u_{failure} \tag{28}$$

where

$$u_{failure} \sim 4u_y \tag{29}$$

If the beam has a cantilever δl, it is a good approximation just to replace Δl by $(\delta l + \beta l)$.

I-cross-section
See Figure 7.

The total moment of inertia about the y-axis is

$$I = I_0 + \frac{1}{2} A_1 h_{cg}^2 \tag{30}$$

With

$$I_0 = I_1 + I_2 + I_3 = \beta^2 I \tag{31}$$

Because of the double-symmetry $u_2 = 0$, i.e. there is only two parameters u_1 and w and (8) is replaced by

$$u_s' = -u_1' + h_{cg} w'' \tag{32}$$

The forces become

$$N_1 = -N_3 = EA_1 u_1' \quad \text{and } N_2 = 0 \tag{33}$$

$$M_1 = M_3 = -EI_1 w'' \quad \text{and } M_2 = -EI_2 w'' \tag{34}$$

The axial equilibrium is satisfied and moment equilibrium for the total cross-section leads to:

$$M = M_1 + M_2 - h_{cg} N_1 = EI_0 w'' - h_{cg} EA_1 u_1' \tag{35}$$

that is identical to (12).

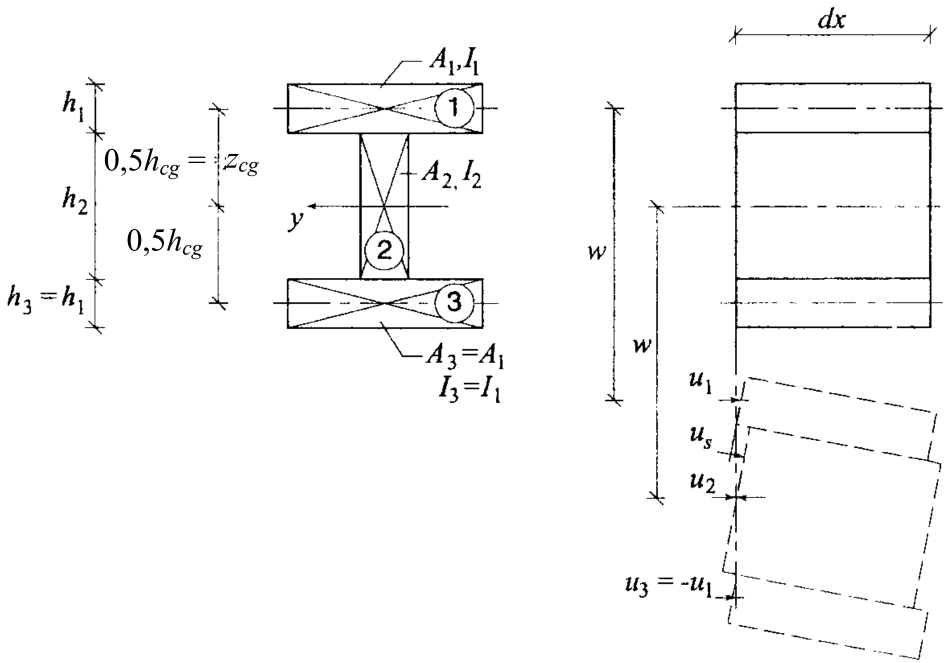


Figure 7. Double symmetrical cross-section. Geometry and deflections.

3-2-1 H J Larsen
The design of built-up timber columns

Summary
The introduction in Section 2 reviews the general basis for the computation of built-up structures where the individual components are connected by semi-rigid connections.

In Section 3 the load-carrying capacity of long perfect columns is determined. It is found that this can be expressed by the Euler formula by using a reduced effective moment of inertia, I_e. It is suggested that the load-carrying capacity can generally be based on the usual expression, but with the slenderness ratio determined from I_e and not from the total moment of inertia I. The justification for this is discussed.

Section 4 gives expressions for the effective moments of inertia for various types of columns: continuously jointed columns, spaced columns with glued or nailed battens (Vierendeel columns) and glued or nailed lattice columns.

Section 5 describes a number of tests with the said types of columns, partly to assess the applicability of the proposed theoretical expressions and partly to determine the rigidity of the connections.

In Section 6 various approximations are mentioned and assessed, and Section 7 gives a brief proposal for design rules.

Section 8 consists of a summary of the literature to which direct reference is made in the text.

32-12-1 T Wolf, O Schäfer
The bending stiffness of nail-laminated timber elements in transverse direction

Introduction
The load bearing behaviour of nail-laminated timber elements is not adequately researched yet. In the design process of nail-laminated timber elements analogies to ruled structural parts in timber work are searched to transfer familiar proves.

The transverse bending stiffness is a characteristic value, which is required at different points.
Existing approaches for the determination of the transverse bending stiffness found in literature do not reflect the real conditions.

In this paper a new analytical approach for the determination of the transverse bending stiffness is derived. Ina parameter study the resulting values of bending stiffness are examined for plausibility and the influence of the important parameters is presented. A final example shows the significance of this characteristic value in the design of nail-laminated timber elements.

Conclusion
The developed computational model enables the calculation of an analytically detectable value for the transverse bending stiffness. The knowledge of the "withdrawal stiffness" of the applied nails is necessary. This characteristic value has to be determined by withdrawal tests.

The new approach results in a lower value for the bending stiffness than estimated in former publications. The transverse bending stiffness is very low and reaches only 0,01 to 0,1 % of the bending stiffness in span direction.

It is obvious, that a sufficient transverse load distribution can not be based on the transverse bending stiffness. A sufficient transverse load distribution can only be guaranteed by the shear stiffness of the nail connection. Further investigations lead to the conclusion, that the assumption of sufficient transverse distribution concerning nail-laminated timber floors is not correct.

Assuming this conclusion, an additional prove with a point load of 1 kN in worst loading position, according to German code DIN 1055, is necessary. In this case the transverse distribution of a point load has to be known. This is why the importance of the transverse bending stiffness and the described computational model may overlap the shown prove of limiting vibration.

34-12-5 V Krämer, H J Blass
Load carrying capacity of nail-laminated timber under concentrated loads

Introduction
A laminated timber element is a plane structural component composed of single, edgewise-oriented lamellas. These lamellas are mechanically jointed usually by nails, alternatively by dowels made of hardwood. In the following, only nail-laminated timber elements are considered, which are mechanically jointed by nails and loaded by central concentrated loads.

The aim of a research project was to derive design-equations for the effective bending stiffness, for the resulting bending stresses, and for the nail loads. It would have been quite expensive to obtain the design-equations by running a large number of tests. This is why the tests were simulated on the computer. For the simulations, the parameters *l/h* (span / height of the element), the nail diameter, and the nail spacing were varied. The simulations were conducted on the basis of realistic stiffness-values of the lamellas and of the nails. The lamellas were generated on the basis of Görlacher, the nails on the basis of the deformation characteristics given in EC5. To guarantee statistically reliable results, 36 different systems of nail-laminated timber elements were simulated. Each system of the nail-laminated timber elements was simulated 500 times. The design equations were derived from the results of these 18,000 simulations.

The results of the design equations show a good agreement with the results of 6 tests.

Summary

A mechanically laminated timber element is a plane structural component which is made of single, edgewise-oriented lamellas. The lamellas are mostly jointed by nails.

The aim of a research project was to derive design- equations for the bending stiffness, for the bending stresses of the lamellas, and for the action effects of the nails. The design-equations were derived from the simulation of thousands of nail-laminated timber elements. In these simulations, the parameters f/h (span / height of the element), the diameter of the nails, and the nail spacing were varied. The simulations were conducted on the basis of realistic stiffness-values of the lamellas and the nails. The lamellas were generated on the basis of the work of Görlacher, the nails on the basis of the slip moduli given in EC5. To guarantee statistically reliable results, 36 different systems of nail laminated timber elements were simulated. Each system of nail-laminated timber elements was simulated 500 times. The design equations were derived from the 95%-fractile-values and from the mean-values of 18,000 simulations.

The results of the design equations were compared with the test results and demonstrated good agreement with each other.

34-12-7 H Kreuzinger
Mechanically jointed beams: possibilities of analysis and some special problems

Introduction

Mechanically jointed beams have a higher load capacity than the sum of the single beams. One beam must be situated over the other beam and the connectors must transport shear forces from one to the other beam. Figure 1 shows a simple bridge with a „flitch beam". The single beams may also be laminates, horizontally or vertically situated like shown in figure 2.

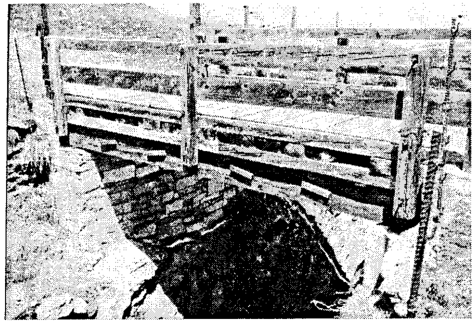


Figure 1: Flitch beam

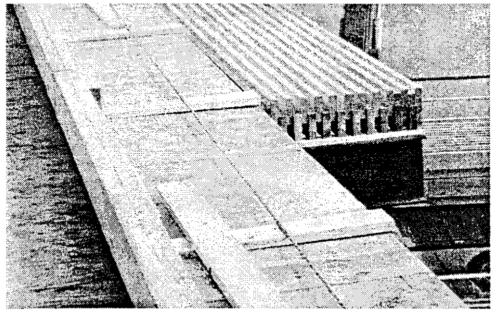


Figure 2: Mechanically jointed laminates

Regarding cross sections made of two parts - the situation is presented in figure 3:

a) two beams jointed with shear connectors
b) framework with two flanges, connected by the diagonals
c) vertically situated laminates, jointed by nails or screws

If the bending stiffness of the single beams is zero there is the situation of framework with hinges.

The connection is not only necessary to transport the shear forces from one part to the other but also to have the same deformation w of the two parts. Cross sections made of more than two parts are also possible. An example is the ribs of shells like those shown in the figures 4a and 4b. The formulas for an approximation of cross sections with more than two parts are given in E DIN 1052, May 2000.

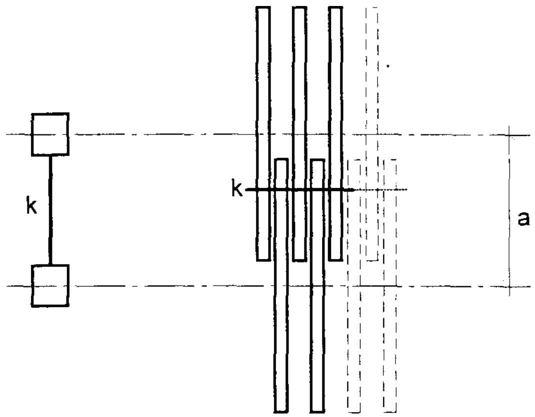


Figure 3.a. cross sections

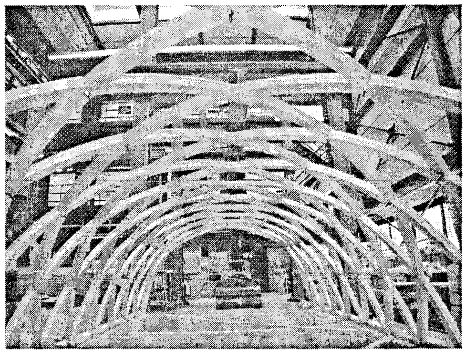


Figure 4a: ribbed timber shell /3/

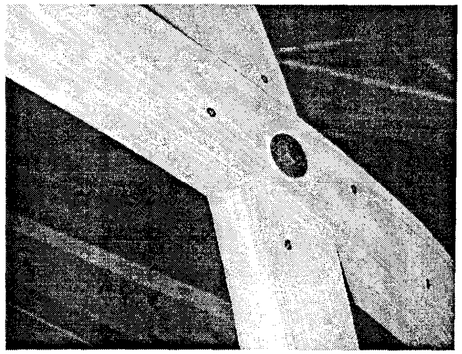


Figure 4b: node /3/

Summary

The connections of mechanically jointed beams are quite different. For example there are nails, screws or notches. They are arranged continuously or on single points along the length of the beam. The beam itself may be supported like a simple beam or may be part of a system. All these must be taken into account by calculation.

For calculation different methods are available:

The analytic solution, given in the codes, is strongly valid only for single supported beams with constant cross section and connection along the length sinuslike load. Framework systems allow to calculate more complicated systems.

Regarding the deformation of the connectors as a shear deformation, it is possible to do the calculation with beams with bending- and shear stiffness (sandwich analogy).

To use the different methods and to interpret the results, the knowledge of the different distribution of shear forces over the cross section is necessary. The basic case is a beam with a shear flow as load. The shear flow in the direction of the axis acts in a distance to the axis.

There are also some peculiar problems with single loads or different boundary conditions. An example was shown using the calculation method with the sandwich analogy.

For cross sections built with two parts the sandwich analogy is theoretical correct. For cross sections consisting of more than two parts, the sandwich analogy is a good approximation calculation. The real system is also transformed in the two girders A and B like shown in figure 5. The bending stiffness of beam A is the sum of the bending stiffness of the single beams, the bending stiffness of the beam B results from the stretching stiffness and the distance of the single beams. The shear stiffness S is calculated with the values of the stiffness of the connection and the shear module of the parts. The formulas are documented in E DIN 1052, May 2000.

40-10-1 J Schänzlin, M Fragiacomo
Extension of EC5 Annex B formulas for the design of timber-concrete composite structures

Abstract

The design of timber concrete composite slabs is currently carried out using the EC5 Annex B formulas for timber-timber composite beams with flexible connections. Since timber, concrete and connection are all characterised by time-dependent phenomena such as creep and inelastic strains due to different thermal expansion or shrinkage of concrete and timber, the most critical design condition is often the limitation of the deflection in the long-term. In order to determine the effect of inelastic strains in terms of deformation and eigenstresses, the design method based on the EC5 formulas has to be extended. Furthermore, the effective creep coefficients in the composite beam differ from the pure material creep coefficients due to the stress redistribution taking place over the service time. Some modified creep values for composite structures should therefore be proposed. The paper presents two possible approaches to extend the EC5 method in order

to account for the aforementioned phenomena. The two approaches are compared to each other and with rigorous solutions. They can both be proposed for an improvement of the current design procedure of timber-concrete composite slabs and beams in accordance with the EC5 provisions.

Concluding remarks

In this paper, two different design approaches for the evaluation of the influence of inelastic strains on the behaviour of timber-concrete composite beams have been introduced and discussed. One approach is directly linked to the EC5 Annex B and based on the evaluation of an external fictitious load equivalent to the effect of inelastic strains. In the other proposal, the internal forces and deflections due to inelastic strains and vertical load are separately evaluated and then superimposed. Therefore such proposal can be used in conjunction with any design method for composite beams under vertical load accounting for the flexibility of the connection system.

In addition, effective creep coefficients have been introduced since the redistribution of the internal forces during the service life affects the relationship between creep strain and elastic strains. The different temporal development of the creep strains has also to be considered, since normally concrete creeps faster than timber. As a consequence of that, the stresses in timber can reach their maximum value within the period between 3 to 7 years, especially if concrete shrinkage is avoided or reduced by using special admixtures.

However several questions are still open for the design of timber-concrete composite structures, such as the value of the factored load coefficient γ_F for shrinkage to be used in ultimate limit state design. Only in DIN Fachbericht 104 (2003) on composite bridges a γ_F value of 1.0 is proposed. The use of this proposal for timber-concrete structures is questionable, since the steel profile provides a ductile behaviour for the steel-concrete composite beam. Additional stresses due to increasing concrete shrinkage can then be resisted by plasticization of the steel leading to an increase in deflection but with very little reduction in load-bearing capacity. However timber is not as ductile as steel, and the increase in stresses due to concrete shrinkage and difference in creep coefficient may lead to brittle failure of the timber beam in tension.

40-10-2 U A Girhammar
Simplified design method for mechanically jointed beams

Abstract

A simplified analysis and design method for composite beams with partial interaction that predicts the deflections and internal actions and stresses is proposed. The method is general in nature and can be applied to arbitrary boundary and loading conditions, and material and geometry parameters. A corresponding approximate method for mechanically jointed beams is given in Eurocode 5, Annex B. This method was originally developed on the basis of only simply supported end conditions. However, for other boundary conditions Eurocode 5 gives recommendations that generally are not as accurate as those obtained from the method presented in this paper. An effective bending stiffness is introduced that reflects the influence of the interlayer slip and depends on the partial composite action (or shear connector) parameter and the relative bending stiffness parameter, which also are defined in the paper. The effective beam length of the problem equals the buckling length for the corresponding column buckling problem. The proposed method is applied to a number of practical cases and the approximate results are compared with the exact values. The results of the approximate analysis procedure are found to be extremely good for beam deflections and usually very good for internal actions and stresses, except for interlayer slip forces. For the simply supported beam case where the Eurocode 5 method is applicable all values coincide. For other boundary conditions, the error in the Eurocode 5 procedure can be up to 36 % depending on the recommended value for the effective beam length.

The proposed method was derived independently and is presented in a somewhat different way, but is basically the same as the present Eurocode 5 method. Except for, in the author's opinion, being simpler to apply, the proposed method clarifies the "correct" way to choose, for different boundary conditions, the effective beam length to be used in either procedure.

Summary and conclusions

A simplified analysis and design method for composite members with partial interaction that predicts the deflections and internal actions and stresses has been proposed. The principle of the approximate method for partially composite beams is to replace the fully composite bending stiffness (EI_∞) with the partially (effective) composite bending stiffness (EI_{eff}) in the expressions for deflections and internal actions and stresses in the corresponding fully composite beam. The effective bending stiffness reflects the influence of the interlayer slip and depends on the partial composite action (or shear connector) parameter (slip modulus, cross-section material and geometry, and beam length) and relative bending stiffness parameter (relation between the non- and fully composite bending stiffnesses of the beam). The effective beam length of the problem equals the buckling length for the corresponding column buckling problem. The methods are general in nature and can be applied to arbitrary support and loading conditions, and material and geometry parameters.

The proposed approximate method has been applied to a number of simple practical cases and the results obtained have been compared with the exact values. The applicability of the simplified analysis procedure was found to be very good (usually an error less than 5 %), except for the interlayer shear stresses (an error of about 10-20 %). The proposed method gives identical results for composite beams with interlayer slip as the approximate method given in Eurocode 5 for simply supported members.

This approximate analysis procedure for designing composite beams with interlayer slip is well suited for designers and should be considered for inclusion in Eurocode 5. Specifically, the principle for the choice of the most accurate effective beam length should be introduced in the code.

The derived approximate analysis procedure can also be extended to include multi-layered composite beams, in particular three-layered members, multiple stringer or joist systems with decking or sheathing, and to include gaps in the decking or sheathing.

41-10-1 T G Williamson, B Yeh
Composite action of i-joist floor systems

Abstract
In 1968, APA conducted field-glued plywood-joist floor system tests and established floor composite action factors based on the research results that have subsequently been in use in the United States for 40 years. Those composite action factors were based on the use of a single layer plywood floor (underlayment) glue-nailed to sawn lumber joist systems to improve the stiffness of the floor and to minimize the impact of nail pull-out and associated squeaks. Prefabricated wood I-joists, while having been used in the United States and Canada for over 30 years, are now gaining acceptance in Europe and other geographic regions such as Australasia for residential floor construction. In fact, approximately 45% of all raised wood floors constructed in North America now use 1-joists which represent over 300 million lineal meters (984 x 106 lineal feet).

As a result, APA recently conducted a study to review the effect of glue-nailed assemblies on the stiffness capacity of I-joist floor systems with oriented strand board (OSB) floor sheathing since OSB is widely used in residential floor construction today. Bending tests on full scale-floor sections as well as T-beam sections with panels glue-nailed to the I-joist frame were conducted. Based on this research, a new composite action factor for glue-nailed I-joist floor systems was established.

APA also conducted an intensive testing program to assess the EA-perpendicular (axial stiffness) properties for OSB. An impact study was then conducted to investigate the effect of this new composite action factor on the allowable spans of I-joist floor systems when the axial stiffness of OSB floor sheathing in the direction perpendicular to the strength axis of the panel is reduced, as evidenced by these recent tests conducted by APA. This paper presents the results of these studies and provides recommendations for determining the spans of wood I-joists used in floor systems.

Conclusions and Recommendations
Results obtained from this limited study confirm that the composite action factor currently used by the wood 1-joist industry in the U.S. is conservative. A composite action factor of 0.55 seems justifiable for glue-nailed I-joist floor systems with unglued T&G. While these molts are rational and as expected, additional floor assembly tests may be considered in the future to expand the database and gain more confidence in the results.

The increase in the composite action factor results in an increase of the 1-joist floor spans in residential floor applications up to 102 mm (4 in.). However, this increase needs to be considered in conjunction with other factors contributing to the composite floor stiffness, such as the recent reduction in the *EA*-design value of OSB floor sheathing. The net effect supports the recommendation of the North American wood I-Joist industry that no changes to the existing allowable spans are required. However, if other factors that contribute to the composite floor stiffness are changed in the future, the I-Joist spans need to be re-evaluated, including the composite action factors.

43-7-3 H J Larsen, H Riberholt, A Ceccotti
Design of mechanically jointed composite concrete-timber beams taking into account the plastic behaviour of the fasteners

Introduction
Mechanically jointed built-up members were used to some extent until about 1950 but were then completely replaced by glued members, e.g. glulam and light I-beams with webs of panel materials. Mechanically jointed beams with traditional nail and screw connections have the disadvantage that the slip between the lamellas shall be rather big in order to achieve a composite effect and this typically results in large deflections.

The reason why there is still interest in the topic in CIB W18 is that the same theory applies to composite T-members with wooden web and concrete flange, both in new structures (e.g. bridges) and especially in old buildings where a new concrete slab on top of existing beams can ensure an upgrading of strength, vibration characteristics and sound insulation.

The calculation of the internal forces and moments is generally made assuming elastic behaviour either by the method laid down in Eurocode 5 (beam theory) or by more advanced design methods, e.g. Finite element methods. When the internal forces and moments are determined it is tacitly assumed that the load-carrying capacity may be found by using the usual elastic failure criteria for the individual members. It is shown in this paper that this assumption is in most cases very much on the safe side for the calculation of the shear load-carrying capacity and that a much better estimate may be found by taking into account the plastic properties of most fasteners. A further advantage by the proposed design method is that it can be used to optimise the fastener pattern

This paper will describe the theory and verify it by comparing reported test results to the results from calculations. The requirements for using the method, e.g. fastener stiffness and deformation capacity will be determined.

At the end a proposal for changes in Eurocode 5 is given.

Conclusions
There is good agreement between the measured and the elastic deformations calculated according to the linear-elastic method given in Eurocode 5, Annex B.

For stiff, brittle fasteners, there is good agreement between the load-carrying capacity found by testing and the smaller of the bending strength and the fastener shear capacity calculated according to Eurocode 5, Annex B.

For fasteners with plastic properties the predicted load-carrying capacity

– is smaller than the load found by an elastic calculation according to Eurocode 5, Annex B assuming that failure takes place when the stress in the bottom of the web exceeds the bending strength;
– is greater (normally much greater) than found by an elastic calculation according to Eurocode 5 assuming that failure takes place when the shearing force per fastener between the timber web and the concrete web exceeds the fastener shear strength;
– corresponds to the load-carrying capacity calculated by the method described assuming that the behaviour of the fasteners is elasto-plastic. The slip at the fasteners shall be checked and shall correspond to the plastic part of the load-slip curve.

43-7-4 M Fragiacomo, D Yeoh
Design of timber-concrete composite beams with notched connections

Introduction on design of timber-concrete composite beam
Timber-concrete composite (TCC) structures must be designed so as to satisfy both serviceability (SLS) and ultimate limit states (ULS) in the short- and long-term (the end of the service life). The ULS is checked by comparing the maximum shear force in the connection, the maximum stress in concrete, and the combination of axial force and bending moment in timber with the corresponding resisting design values. The most important serviceability verification is the control of maximum deflection, which is used also for an indirect verification of the susceptibility of the floor to vibration, as suggested by Australian/New Zealand Standard 1170 Part 0.

Two problems have to be addressed when evaluating stress and deflection of a TCC beam:
(1) the flexibility of connection, which leads to partial composite action and, in general, does not allow the use of the transformed section method in design; and (2) the timedependent behaviour of all component materials, i.e. creep, mechano-sorption, shrinkage/swelling, thermal and moisture strains of timber and concrete, and creep and mechano-sorption of the connection system.

To account for the first problem, two approaches have been proposed: the linear-elastic method and the elasto-plastic method. The linear-elastic method is based on the assumption that all materials (concrete, timber and connection) remain within the linear elastic range until the first component (generally, either the timber beam or the connection) fails. This is appropriate in many cases of technical interest, particularly for TCC with very strong and stiff connectors such as notches cut in the timber and filled with concrete. A linear-elastic analysis is generally carried out for the short-term (instantaneous) verifications according to the approach suggested by Ceccotti, which is based on the use of the gamma method recommended in the Annex B of the Eurocode 5. According to the gamma method, an effective bending stiffness, $(EI)_{ef}$, given by Eq (1), is used to account for the flexibility of the timber-concrete shear connection. A reduction factor γ, which ranges from 0 for no composite action between the timber and concrete interlayers to 1 for fully composite action (and rigid connection), is used to evaluate the effective bending stiffness:

$$\left(EI\right)_{ef} = E_1 I_1 + E_2 I_2 + \gamma_1 A_1 a_1^2 + \gamma_2 A_2 a_2^2 \qquad (1)$$

where subscripts 1 and 2 refer to concrete and timber elements, respectively; E is the Young's modulus of the material; A and I are the area and the second moment of area of the element cross-section; a is the distance from the centroid of the element to the neutral axis of the composite section; and γ is the shear connection reduction factor. Using the effective bending stiffness, the maximum stresses in bending, tension and compression for both the timber and concrete elements, and the shear force in the connection can then be calculated [2]. In Eq. (1), γ_1 is calculated from Eq. (2) and γ_2 is taken as one:

$$\gamma_1 = \frac{1}{1 + \dfrac{\pi^2 E_1 A_1 s_{ef}}{K l^2}} \qquad (2)$$

$$\gamma_2 = 1 \qquad (3)$$

where s_{ef} is the effective spacing of the connectors assumed as smeared along the span of the floor beam; l is the span of the TCC floor beam; and K is the slip modulus of the connector. For verifications at ULS and SLS, different values of slip moduli, K_u and K_s, are used, defined by Eqs. (4) and (5), respectively. Such a difference between K_u and K_s arise from the shear force-relative slip relationship of the connection, which is generally nonlinear. These stiffness properties of connector are evaluated through experimental push-out shear test (Figure 1) carried out as recommended in EN 26891

$$K_u = \frac{0,6 F_m}{v_{0,6}} \qquad K_s = \frac{0,4 F_m}{v_{0,4}} \qquad (4)\ (5)$$

where F_m is the mean shear strength obtained from a push-out test, $v_{0.4}$ and $v_{0.4}$ are the slips at the concrete-timber interface under a load of 40% and 60% of the mean shear strength F_m , respectively. Figure 1 displays a typical experimental set-up of a push-out test, carried out at the University of Canterbury, New Zealand, to investigate the mechanical properties of notched connectors between LVL joists and concrete slabs.

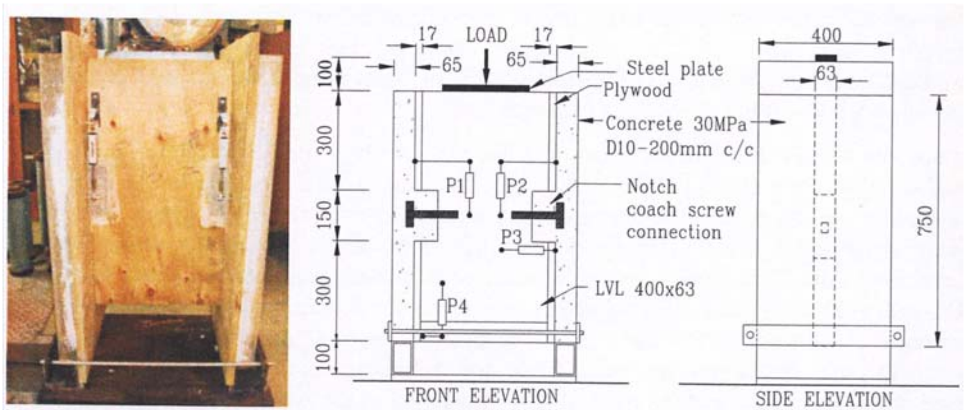


Figure 1: Symmetrical push-out test set-up (dimensions in mm)

The elasto-plastic solution has been proposed specifically for cases where the failure of he TCC is attained after extensive plasticization of the connection system, so as to allow for redistribution of the shear force from the most stressed to the less stressed connectors along the beam. This is fairly common where the connectors are low strength, low stiffness and high ductility, such as for mechanical fasteners. The failure load is evaluated by assuming a rigid-perfectly plastic behaviour of the connection.

For verifications in the long-term, the 'Effective Modulus Method' recommended by Ceccotti is used to account for the effect of creep of the different materials. The effective moduli of concrete, E_1, and timber, E_2; and slip modulus of connector, K in Eq. (1) are replaced with their respective effective moduli $E_{1,eff}$, $E_{2,eff}$ and K_{eff} given by:

$$E_{1,eff} = \frac{E_1}{1+\phi_1(t,t_0)} \quad E_{2,eff} = \frac{E_2}{1+\phi_2(t-t_0)} \quad K_{eff} = \frac{K}{1+\phi_f(t-t_0)} \qquad (6)\ (7)$$

(8)

where $\varphi_1(t,t_0)$, $\varphi_2(t-t_0)$ and $\varphi_f(t-t_0)$ are, respectively, the creep coefficient of concrete, timber, and connector, t and t_0 are, respectively, the final time of analysis (the end of the service life, usually 50 years) and the initial time of analysis (the time of application of the imposed load).

The approach discussed above neglects the effect of environmental strains caused by the different thermal expansion and shrinkage of concrete and timber on the internal forces and the deflection of TCC, resulting in an underestimation of the deflection at the end of service life. To resolve this issue, rigorous and approximated closed form solutions were derived to account for the effects of environmental strains and drying shrinkage of concrete on TCC. Such formulas were compared to each other showing good accuracy, and then used to estimate the influence of different environmental conditions, type of exposure, and size of the timber cross-section on the design of TCC beams .

A significant influence on the design was found, particularly for TCC systems with solid
timber decks and rigid connections, and for TCC floors with narrow timber joists exposed
to outdoor, sheltered environmental conditions.

Conclusions and implications for future code developments

This paper discusses the design of timber-concrete composite beams with notched connections. Analytical formulas for the prediction of the shear resistance of notched connectors were derived, based on four possible failure mechanisms:

(i) shearing of the concrete within the notch,
(ii) compression of the concrete within the notch,
(iii) shearing of the timber parallel to grain between two consecutive notches or from the first notch to the end of the beam, and
(iv) crushing of the timber parallel to the grain at the interface with the concrete.

The formulas, derived according to the New Zealand Standards and the Eurocodes, were validated against the results of an extensive experimental programme which involved several push-out specimens to failure carried out on small LVL-concrete composite blocks at the University of Canterbury, New Zealand. Such formulas can therefore be proposed in the new versions of the aforementioned regulations.

Based on an extensive experimental programme carried out at the University of Canterbury which involved tests to failure of full-scale LVL-concrete composite beams with notched connections, it was found only a minor difference between the experimental deflection at serviceability limit state and the analytical value calculated using the transformed section method, i.e. by neglecting the flexibility of the connection. This suggests a possible simplified procedure for design of timber-concrete composite beams with notched connections, i.e.:

(1) calculation of the flexural stiffness (*EI*) of the composite section using the transformed section method;
(2) calculation of the shear strength demand of the notched connection at ultimate limit using the flexural stiffness (*EI*);
(3) comparison of the shear strength demand with the strength capacity of the notched connection evaluated using the proposed analytical formulas;
(4) evaluation of the deflection at serviceability limit state by reducing the flexural stiffness (*EI*) by 13%;
(5) calculation of the stresses in concrete and timber at ultimate limit state by reducing the flexural stiffness (*EI*) by 13%.

Although the aforementioned procedure was derived and validated on a particular type of composite floor made from LVL joists with rectangular and triangular notched connections, the procedure is general and can be applied to any type of composite structure with notched connection. Further analytical-experimental comparisons are however warranted, in particular to check the accuracy of the 13% reduction factor of the flexural stiffness used together with the method of the transformed section for different types of composite floors (for example, with solid deck).

43-7-5 K Crews, C Gerber
Development of design procedures for timber concrete composite floors in Australia and New Zealand.

Part 1 – Design Methods

Introduction

Timber concrete composite (TCC) floor systems are relatively new to Australia and New Zealand and satisfactory performance requires a rigorous

design procedure addressing both ultimate and serviceability limit states. TCC structures have a degree of complexity since they combine two materials that have very different mechanical properties and respond in different ways to their environment. Furthermore, most TCC structures exhibit partial (not full) composite action and this adds to the complexity of the system.
Several design procedures are discussed in the literature. Amongst these, the Eurocode 5 C5) procedure (BSI 1991; European Committee for Standardisation 1995) is relatively straightforward and has been successfully implemented in Europe. It utilises a simplification for modelling the complex timber - concrete interaction known as the “Gamma coefficients” method, which manipulates the concrete member in order to predict the cross-section characteristics of the structure.

Concluding Comments

The design procedure presented in this paper is adapted from the design procedure of EC5 and modified to suit local practices and reflect research and development in Australia and New Zealand.
The design methodology adequately addresses the complexity of TCC structures, including
the partial composite action pro of strength checks on the cross-section components and serviceability checks with consideration to the long term performance of the structure.
Adapting the design procedure to suit Australian practices has been a challenging exercise and where assumptions have had to be made due to uncertainties, these have erred on being conservative. These assumptions are also areas for further research in order to address the uncertainties associated with them. It is anticipated that further research will include:

- shear strength of the connection – size effect of the connection strength and stiffness
- shear strength of the concrete notch – effect of the coach screw
- shear strength wood portions between the notches
- flexural shear strength of the beam – effect of deep notch and use of the net area of the shear plane,
- short-term serviceability – initial deflection and effect of concrete curing,
- long-term deflection
- influence of wood portions between the notches

Further work will also focus on making the design procedure more user-friendly wherever possible whilst preserving the safety and functionality of the design

Part 2 – Connection characterisation

Introduction

The approach adopted for design of timber concrete composite (TCC) floor systems in Australia and New Zealand is based upon extensive testing of the permitted connection types that are specified in the design procedures, identifying strength, serviceability stiffness and so called ultimate stiffness characteristic properties that are required for utilisation of the "Gamma coefficients" method, which manipulates properties of the concrete member in order to predict the cross-section characteristics of the structure. This paper presents an overview of testing undertaken to date and the derivation of characteristic properties (5th percentile for strength and 50th percentile or average for stiffness).

Conclusions

A number of shear connections have been tested using pull-out tests on full-scale specimens and load-deflection plots and stiffness for these connections have been determined. Parameters such as the type of connector, shape of notches, use of mechanical anchors and concrete properties have been investigated and analysis of this data has led to a number of conclusions

- Early research showed that use of nail plates alone as shear connectors did not prove to be effective, whilst a combination of nail plates with either screws or concrete notches was more effective - especially incorporation of concrete notches.
- A number of concrete notch type shear connections were then tested such as trapezoidal, bird-mouth type and tri-angled notch and parameters such as slant angle, use of either coach screw or normal wood screw as mechanical fastener, inclination of the mechanical fastener, inclination of the slanting face and use of low shrinkage concrete were studied.
- Use of coach screws has the advantage of deeper penetration depth inside the concrete slab in comparison to normal wood screws due to their longer length. This resulted in a single coach screw providing higher shear capacity than a combination of four wood screws.

- Interesting results were obtained from the bird-mouth type connections as these connections generally exhibited higher strength and stiffness than the trapezoidal notch connections and especially so for bird-mouth connections using 70-20 and 60-30 angle combinations.
- Tri-angled notch connections were also found to be superior to the trapezoidal notch connections, however, the complex angle sequence makes such connections difficult to fabricate.
- On the other hand, bird-mouth type connections are much easier to fabricate with a simple cutting sequence and do not need special tools for fabrication. Use of a slanted coach screw configuration in the bird-mouth notch connections provided higher stiffness; however, the effect on characteristic strength was not significant, while steel plate placed on top of the coach screw did not provide any additional strength or stiffness. It should however be noted that the coach screws in the bird-mouth notch provided only limited post peak plastic behaviour when compared to trapezoidal notch connections.
- The depth of the notch has a significant effect on both the stiffness and strength of the connections. Connections with 60 mm deep notch had superior strength and stiffness compared to the connections with 90 mm deep notch. Test results also showed that widening the slot dimension had a positive effect on strength and stiffness of the connections.
- The effect of the ratio of coach screw diameter to LVL thickness is one of the parameters that need to be further investigated. Table 1 highlights the effect of the ratio of coach screw diameter to LVL thickness and suggests that there is no advantage to using 16mm diameter screws in 48mm thick LVL beams

Whilst the variability of maximum load (strength) is considered to be acceptable, the variability of the characteristic stiffness properties highlights some of the uncertainty that is inherent in the performance of notched connections for TCC constructions. It is proposed to use the data generated to date, to refine connection performance and attempt to that stiffness variability to lower levels that could lead to more efficient design of these types of floor structures.

3.9 NOTCHED BEAMS

ESSAY 3.7 H J Larsen
Notched beams in Eurocode 5

The following simple closed-form equation for the strength is derived by means of fracture mechanics.

$$\frac{V_f}{b\alpha d} = \frac{\sqrt{G_{fy} / d}}{\sqrt{0,6(\alpha - \alpha^2) / G} + \beta\sqrt{6\left(1 / \alpha - \alpha^2\right) / E_x}} \qquad (1)$$

where

V_f shear force at fracture of the notch
G shear modulus
E_x modulus of elasticity
G_{fy} fracture energy in pure tensile splitting perpendicular to grain

To arrive at the Eurocode 5 expressions a few simplifying modifications of (1) are made:
1) The ratio *E/G* is set to 16.
2) It is assumed that $\sqrt{EG}$ is proportional to the shear strength.
3) Test results from Riberholt et al were used to introduce a factor that considers the effect of taper.

Reference:

Riberholt, H, Enquist, B, Gustafsson, P.-J. and Jensen, R. B.: Timber beams notched at the support, 1999. Report TVSM-7071. Lund University, Sweden.

21-10-1 P J Gustafsson
A study of strength of notched beams

Introduction

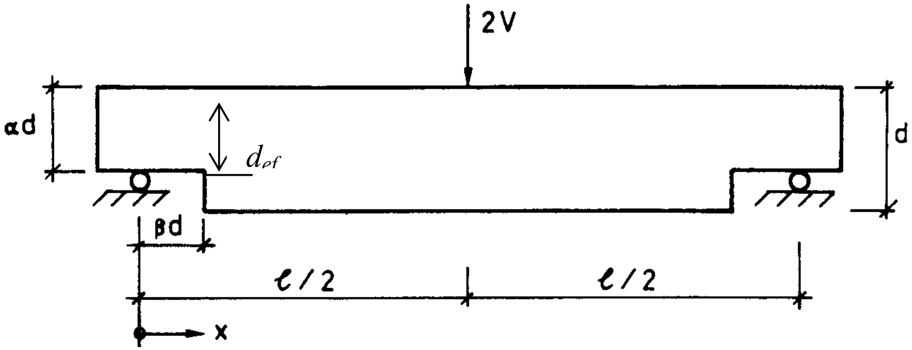


Figure 1. Beam with rectangular end-notch. Definition of α, β and d. The width of the beam is b. $d_{ef} = \alpha d$.

Short time strength of wooden beams with a rectangular end-notch on the tension side according to Figure 1is studied. A simple closed-form equation for the strength is derived by means of fracture mechanics and test results are presented. Test results from literature are compiled.

The study is inspired by differences of principle between conventional formulas used for design and theoretical results obtained by fracture mechanics and finite elements. Influence of size on strength represents such a difference.

Conclusions

Following conclusions relate to the strength of a 90º notch on the tension side of a wooden beam. By notch strength is meant $V_f / \left(bd_{ef}\right) = V_f / \left(b\alpha d\right)$.

- Several test results unanimously demonstrate a significant size-effect in notch strength. Weibull theory can hardly be regarded to give a valid explanation to this size-effect.
- Notch failure may be analysed theoretically by fracture mechanics and a closed-form expression for notch strength has been obtained. This expression seems to give results in good agreement with tests and may provide useful information during development of a design formula.
- Material properties stiffness and fracture energy are found to be decisive for notch strength. Among stiffness parameters, shear modulus, G_{xy}, and modulus of elasticity in grain direction, E_x, are of prime importance. For constant E_x/G_{xy} the notch strength is approximately proportional to $\left(G_{f,y}\sqrt{E_x G_{xy}}\right)^{0,5}$, $G_{f,y}$ being the fracture energy in tension perpendicular to grain.
- According to the theory, tensile strength perpendicular to grain is of minor importance for notch strength. Co-variation between tensile strength and other material properties may give an apparent influence of tensile strength. As for the present test results, however, no statistical correlation between notch strength and tensile strength perpendicular to grain could be found.
- The distance, βd from the support to the notch tip, which determines the bending moment at the notch tip, is of significant importance for notch strength.
- During increase in notch depth, the notch strength decreases rapidly when 1.0 > α > 0.7. For smaller values of α, the influence of α is less. For $\alpha \rightarrow 1.0$ notch strength becomes theoretically infinite, reflecting that notch failure can not occur at the tip of a non-existing notch.
- Applicability of linear elastic fracture mechanics depends on the length βd. Non-linear fracture mechanics is estimated to give good results also for small beams with a small β.
- A conventional formula, such as $1{,}5V_f/(b\ \alpha\ d) = f_v$, may in some cases overestimate notch strength very much.

22-10-1 H J Larsen and P J Gustafsson
Design of end notched beams

Introduction

The general design rules for notched beams in [Eurocode 5, 1988] have been criticized for not taking into account the effect of the depth of the beam, a factor which has been demonstrated by tests to have a significant influence on the strength.

Further the special rules for glued laminated beams are criticized for mixing global design considerations (total beam volume, load distribution etc), and problems related to small local areas.

A consistent design method based on a simplified fracture analysis was presented in **Paper 21-10-1.** The paper was discussed at the CIB W18 meeting in September 1988, and it was agreed that it should be used as basis for future versions of the CIB Structural Timber Design Code.

In the following is indicated how the code text could be formulated. The proposal only concerns the requirements for the notched part of the beams additional to the general requirements for the shear design of unnotched beams, and is thus independent of the method which might eventually be codified for beams in general (i.e. for instance whether the effect of volume and load distribution will be taken into account).

Further a draft for a test method for determination of the determining factor – the fracture energy – is given.

23-10-1 T A C M van der Put
Tension perpendicular to the grain at notches and joints

Summary
Constructive details as notches may cause high tension perpendicular to the grain and should not be applied. However rules are necessary if alternative solutions are not possible and design rules are proposed for the Dutch Code TGB-1990 as a better alternative than the Eurocode rules.

Although the background of the design rules of the American Code for notched beams is not known it is possible to derive these rules and it is shown that they are only applicable for narrow span, high beams.

Design rules are derived using the simple fracture mechanics approach. Except for splitting along the grain the method is also used for crack propagation perpendicular to the grain. The same is applied for joints at the lower edge of a beam leading to an equivalent instable crack length. Simple design rules appear to be possible based on a lowest upper bound of the strength leading to the equations of for notched beams and for eccentric joints.

23-10-2 K Riipola
Dimensioning of beams with cracks, notches and holes. an application of fracture mechanics

Abstract
A dimensioning method based on fracture mechanics is presented. Fracture energy for a beam with an infinite crack is calculated with the beam theory and the energy principle. Mode separation is done by the definition of mode II and the superposition principle. The material is assumed to be orthotropic in the stress intensity calculations. A parabolic fracture criterion is used to predict failure. To evaluate the method, some experiments in the literature are re-analyzed.

Conclusions
A fracture mechanics based method for dimensioning of beams with cracks, notches, and holes is proposed to be included in the CIB code. According to the proposal, capacity of a beam with a true or a supposed crack can be estimated as follows:

– Consider the support conditions of the beam. If the lower part can freely be separated from the upper part, a mixed mode case is present. Else, if the parts cannot be separated, (almost) pure mode II is present.
– Calculate the stress intensities according to the following equations:

$$K_I = \frac{M_a}{3th}\left[\frac{6h^2h_2^3}{h_1^3\left(h_1^3+h_2^3\right)} + \frac{M_a^2}{M_a^2}\frac{3E_x}{5G_{xy}}\frac{hh_2}{h_1}\right]^{0,5}$$

$$K_{II} = \frac{2M_a}{th}\left[\frac{2h_1h_2}{\left(h_1^3+h_2^3\right)}\right]^{0,5}$$

– Find out the fracture toughness values for the present modes and fracture systems. In the case of dimensioning, mode I system TL should be assumed, because it is weaker than mode I system RL.
– Use a suitable fracture criterion, as Wu's:

$$\frac{K_I}{K_{IC}} + \left[\frac{K_{II}}{K_{IIC}}\right]^2 = 1$$

– Solve the equation to get the critical load.

38-6-1 R H Leicester
Design specifications for notched beams in AS 1720

Abstract
The design strength of notched beams has been in the Australian Standard AS 1720.1 for more than 30 years. In this paper these design rules are examined through the use of elastic fracture mechanics. Both stress intensity factors and critical stress intensity factors are evaluated for three notch angles in a beam element.

Discussion

The information reported in this paper is of uncertain accuracy because there is limited published data available for calibration and comparison purposes. The size effect for notch slope g/a = 4 is less than expected and may be due either to the use of incorrect elastic parameters for the computations or to the fact that the sizes of the beams tested may not be large enough to contain the critical eigenfield.

The critical stress intensity factors do not follow the expected relationship $K_{AC,m} = K_{AC,v}$ or the linear interaction equation. It would be useful to use fundamental wood properties to derive an expected effect of the notch slope parameter g/a on the critical stress intensity factor K_{AC}.

41-6-1 A Asiz, I Smith
Design of inclined glulam members with an end notch on the tension face

Introduction

Notching glue laminated (glulam) timber beams at the tension side is a crucial decision that arguably should be avoided by design engineers because of the high stress concentrations that develop around such notches inducing high tension perpendicular to grain and high shear parallel to grain stresses. However, it is done in practice to facilitate construction and to reduce the total necessary depth of floors and roofs. Most current design code provisions permit design of notches at the tension side providing that they are located close to the ends of members and at a simple structural support point. The key design process is normally done through modification terms that are part of assessing the shear capacity of members. Traditional practices in this respect are empirical and of uncertain origin. Strictly the problem requires a rational fracture mechanics analysis and that is done in a few related instances. For example, the notch design equation in Canada for sawn timber members is based on a simple fracture mechanics theory. However, the same practices are not adopted in design of notched glulam members, because of concern that a simple fracture theory might be inaccurate when notches remove only a small proportion of a member's depth. Moreover contemporary notch design provisions in codes were developed mostly from empirical observations of how flat notched solid lumber beam behave. It is questionable that results apply to notching of large glulam members, particularly those with inclined configurations.

The objective of this study is to investigate the effects of small bird-mouth end-notches on failure behaviour and associated strength of inclined glulam members subjected to short-term static load. Test result is compared with current design codes taking into account various possible failure mechanisms for glulam member loaded under statically determinate four-point bending arrangement. Simple design rule is proposed regarding whether there is a need to consider notching effect for small bird-mouth end-notches.

Design recommendation

The following recommendations are proposed to CSA086-01 Committee in respect of end notches on tension faces of inclined glulam members:

- Stress concentrating effect for notches that remove not more than 8% of member depth are negligible and can be neglected in structural design.
- Stress concentrating effects of notches that remove more than 8% of member depth (up to 25%) should not be ignored in structural design, and fracture mechanic check as per solid lumber must be performed in addition to shear checking on the residual cross-section.

41-6-2 K Rautenstrauch, B Franke, S Franke, K U Schober
A new design approach for end-notched beams - view on code

Introduction

The determination of the load-carrying capacity of timber structures can be done by means of the theory of elasticity in most cases. This excludes areas of high demanding loads and stress concentration, e.g. notches and holes. For these areas, different empirical approaches on the basis of the shear resistance and the theory of elasticity have been developed over the past years and included in design codes. In the European and German Standard for the design of timber structures similar energy balanced approaches can be found. The background for these design models was the proposal of Gustafsson, who estimates the material resistance against crack initiation in dependence of the energy release rate and the resilience of the structure. In his model, he is simplifying the mixed mode ratio of the total fracture energy is approximately equal to the fracture energy for transversal tension when fracture occurs. Recent experimental and numerical investigations of the strain condition around notches and holes lead to an extension of this theoretical approach, considering also fracture mode I and fracture mode II.

Conclusions

The overall loading situation in rectangular end-notched beams consisting of transverse tension and shear influence was considered for design issues by use of critical fracture parameters for mode I and II. The parameters derived from experimental and numerical data have been calculated independently from the numerical and test results of the same specimen. The measured strain fields correspond very well with the numerically obtained structural behavior. The presented results initialized a new revised and optimized design proposal to estimate the load-carrying capacity of end-notched beams. The introduced solution and methodology to determine the capacity of rectangular end-notched beams can be easily extended to similar problems with high stress singularities.

3.10 PLATE BUCKLING

ESSAY 3.8 H J Larsen
Plate Buckling

Safe rules for plate buckling are given in Eurocode 5, clause 9.1.1 Glued thin-webbed beams and clause 9.1.2 Glued thin-flanged beams. In cases not covered with the rules a detailed buckling analysis should be made.

Theory
The following is based on **Paper 10-4-**1.

Buckling
An elastic plate loaded in compression or shear in the plane of the plate may for some load levels become unstable and deflect perpendicular to the plane. The phenomenon is called buckling and is an instability phenomenon of the Euler column buckling, however with an important difference: It is in most cases possible to increase the load after initial buckling. Initial buckling is, therefore, often regarded as a serviceability limit state.

When a rather long panel buckles it is divided up by node lines where the deflections are zero. The buckled form depends on plate form and type of loading. Typical examples are shown in figure 1. The critical load is found as for a slender column: A deflected form for the plate is assumed and by energy considerations the conditions for the deflected plate to be stable is found. The calculations will not be given here. Reference is made to e.g. Halasz & Cziesielski (1966).

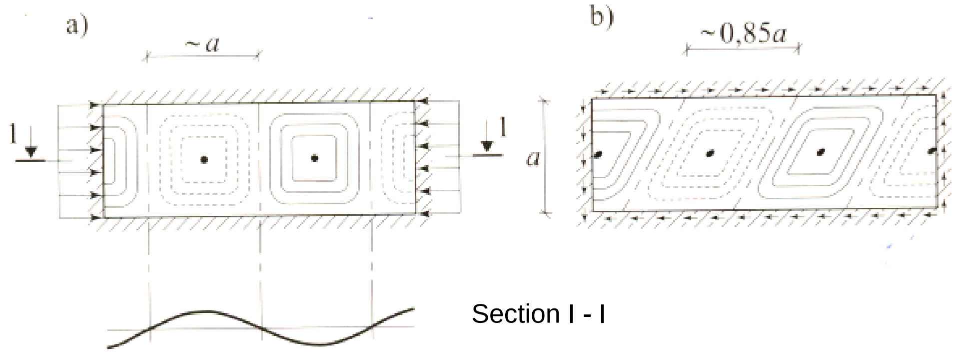


Figure 1. Buckled plate fields. a) for a constant normal (compression)stress. b) for pure shear stresses.

A rather long web may be regarded as composed of a number of fields simply supported by the flanges and webs, and over the node lines. It is loaded by in-plane stresses (σ, τ) from *N*, *M* and *V*.

Buckling – normal stresses
The case shown in Figure 2 is investigated. A plate with length l and width a, simply supported along the edges is loaded in the x-direction with compression stresses varying over the depth from σ to $\mu\sigma$ where $\mu \leq 1$. The plate is assumed to be orthotropic with main directions x and y. The critical stress may be written as

$$\sigma_{cr} = k_{buck,\sigma} \frac{\pi^2 \sqrt{(EI)_x (EI)_y}}{ta^2} \tag{1}$$

where

$(EI)_x$ bending stiffness of a strip (with length direction parallel to the y-axis) width unit width by bending about the x-axis

$(EI)_y$ as $(EI)_x$ but for bending by bending about the y-axis for a strip (with length direction parallel to the x-axis)

$k_{buck.\sigma}$ a factor depending on μ and two parameters β_1 and β_2, see figure 3-5

$$\beta_1 = \frac{l}{a} \sqrt[4]{(EI)_x / (EI)_y} \tag{3}$$

$$\beta_2 = 0{,}5 (GI)_{tor} / \sqrt{(EI)_x (EI)_y} \tag{4}$$

$(GI)_{tor}$ torsional stiffness of a plate strip with unit width.

The stiffness parameters may be calculated from the moduli of elasticity E_x and E_y, the shear modulus$G$ and the Poisson's ratios ν_{xy} and ν_{yx}

$$(EI)_x = \frac{1}{12} E_x t^3 / (1 - \nu_{xy}\nu_{yx}) \tag{5}$$

$$(EI)_y = \frac{1}{12} E_y t^3 / (1 - \nu_{xy}\nu_{yx}) \tag{6}$$

$$(GI)_{tor} = Gt^3 \big/ 3 + \left[\nu_{xy} (EI)_x + \nu_{yx} (EI)_y \right] \sim Gt^3 / 3 \tag{7}$$

For an isotropic plate is $\beta_1 = l/a$ and $\beta_2 = 2G/E$.

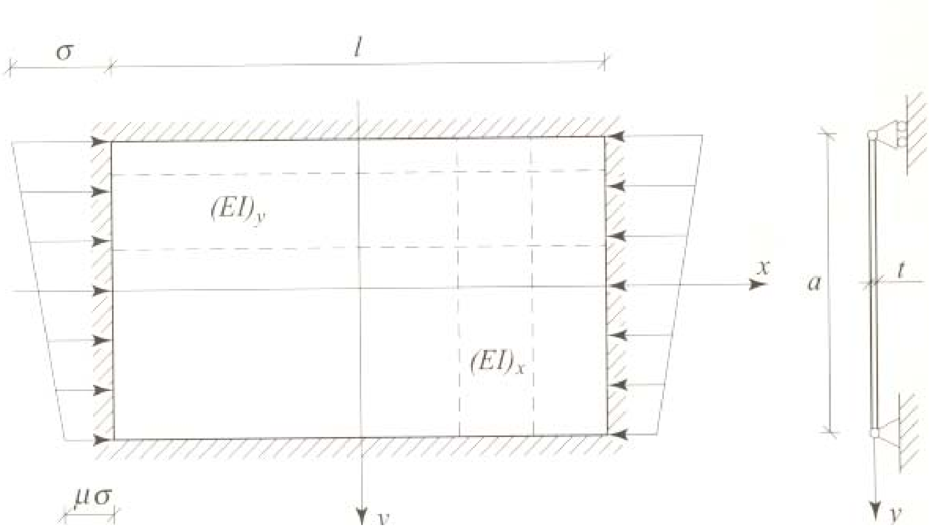


Figure 2. simply supported plate with varying normal stresses

For $\mu = 1$, the theoretical values for $k_{buck,\sigma}$ are shown in Figure 3. The "festoon" form is due to the fact that the total plate length shall be divisible by the length of the buckled plate fields.

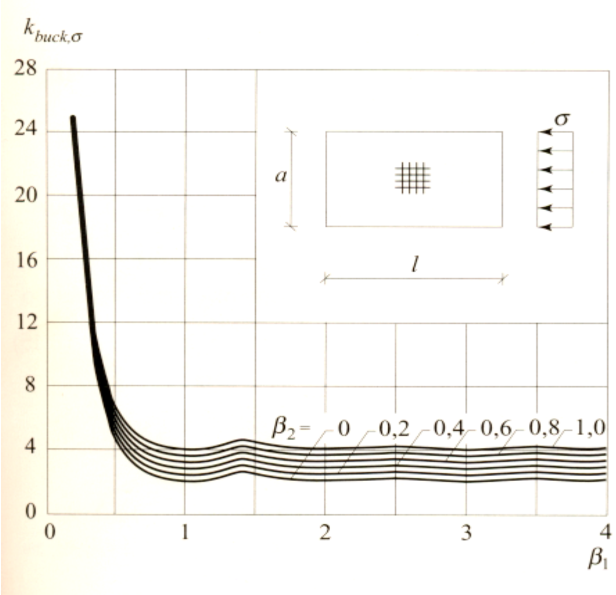


Figure 3. Theoretical values of $k_{buck,\sigma}$ for $\mu = 1$

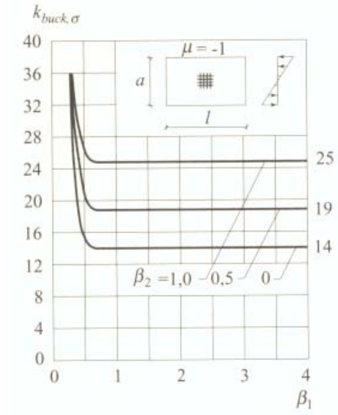


Figure 4. $k_{buck,\sigma}$ for bending ($\mu = -1$)

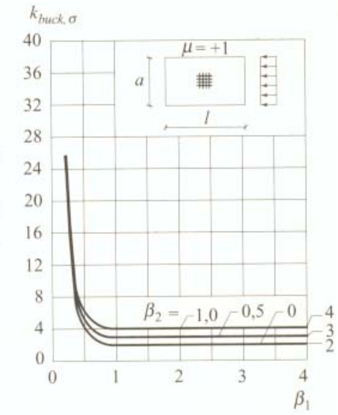


Figure 5. $k_{buck,\sigma}$ for compression ($\mu = 1$)

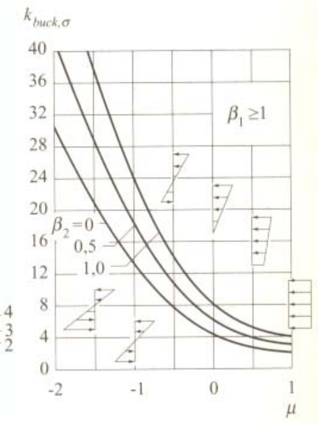


Figure 6. $k_{buck,\sigma}$ for $\beta_1 > 1$

In practice the smoothed out curves are used. They are shown in Figure 4 and 5 for $\mu = -1$ and $\mu = 1$. For $\beta > 1$, $k_{buck,\sigma}$ is almost constant. Figure 6 shows $k_{buck,\sigma}$ for μ between +1 and -2.

The following approximations may be used:

Pure bending ($\mu = -1$): $\quad k_{buck,\sigma} = 11{,}1 \cdot (1{,}25 + \beta_2)$ (8)

Pure compression ($\mu = 1$): $\quad k_{buck,\sigma} = 2 \cdot (1 + \beta_2)$ (9)

Buckling – shear stresses

For shear stresses alone, see Figure 7, the critical shear stress may be calculated as

$$\tau_{cr} = k_{buck,\tau} \frac{\pi^2 \sqrt[4]{(EI)_x^3 (EI)_y}}{ta^3}$$

Where $k_{buck,\tau}$ is given in Figure 7.

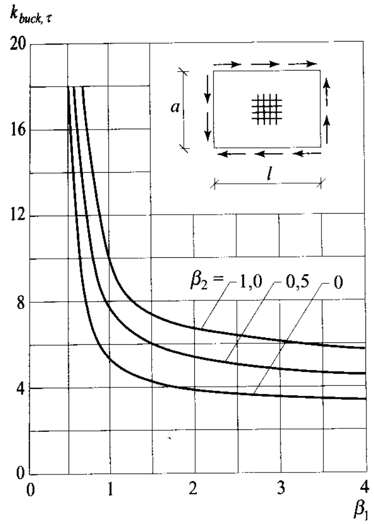


Figure 7. $k_{buck,\tau}$

Buckling – combined stresses
For both stresses according to Figures 4-6 and Figure 7 it should be verified that

$$\frac{\sigma}{\sigma_{cr}}+\left(\frac{\tau}{\tau_{cr}}\right)^2 \le 1 \qquad (10)$$

Minimum thicknesses for particle and fibre boards
In the following the thicknesses necessary to avoid buckling, even when the load-carrying capacities of the panels are fully utilised, are determined.

For isotropic panel e.g. particle boards and fibre boards, $\beta_2 \sim 1$. For rather long plates, i.e. $\beta_1 > 1$ and for pure compression the conditions become, see Figure 5:

$$\sigma_{cr} = 4\pi^2 \frac{Et^3}{12ta^2} > f_c \qquad (11)$$

$$\frac{a}{t} < 1{,}8\sqrt{\frac{E}{f_c}} \qquad (12)$$

Correspondingly for pure shear:

$$\tau_{cr} \sim 5{,}5\pi^2 \frac{Et^3}{12ta^2} > f_v \qquad (13)$$

$$\frac{a}{t} < 2{,}1\sqrt{\frac{E}{f_v}} \qquad (14)$$

Values for t typical values of E/f_c and E/f_v are given in Table 1.

Minimum thicknesses for plywood
For plywood the following notations are used:

$$\varphi = (EI)_y/(EI)_x \qquad (15)$$

$$(EI)_x = \frac{1}{1+\varphi}\frac{Et^3}{12} \qquad (16)$$

$$(EI)_x = \frac{\varphi}{1+\varphi}\frac{Et^3}{12} \qquad (17)$$

where E is the modulus of elasticity. With $E/G \sim 20$

$$\beta_2 \sim 0{,}5\frac{E}{20}\frac{t^3}{3}(1+\varphi)/\left(\frac{Et^3}{12}\sqrt{\varphi}\right) = 0{,}10\frac{1+\varphi}{\sqrt{\varphi}} \qquad (18)$$

With $0{,}5 < \varphi < 5$, β_2 will be between 0,27 and 0,20. In the following $\beta_2 = 0{,}25$ is used.

To ensure that failure will not be due to buckling in the case of pure compression

$$\sigma_{cr} \sim 2{,}5\pi^2 \frac{\sqrt{(EI)_x(EI)_y}}{ta^2} > f_c \qquad (19)$$

f_c depends on whether the stresses act in or perpendicular to the panel fibre direction. In most cases the former possibility is the worse. The limit for a/t is found by (19)

$$\frac{a}{t} = 5\frac{\sqrt[4]{(EI)_x(EI)_y}}{t\sqrt{f_c}} \qquad (20)$$

For pure shear for $\beta_2 = 0{,}25$ the condition to avoid buckling becomes

$$\tau_{cr} = \frac{4\pi^2}{ta^2}\frac{Et^3}{12}\frac{\sqrt[4]{\varphi}}{1+\varphi} > f_v \qquad (21)$$

$$\frac{a}{t} < 1{,}8\frac{\varphi^{0{,}125}}{(1+\varphi)^{0{,}5}}\sqrt{\frac{E}{f_v}} \tag{22}$$

Approximate limits are given in Table 1. They correspond to conservative estimates for the material parameters and to the situation where the panels are fully utilised. It is, therefore often possible to use thinner panels may be used.

Table 1. Limits for the ratio a/t.

	Pure compression	Pure shear
Plywood with the panel direction		
In the stress direction	20	$60/(1+0{,}1\varphi)$
Perpendicular to the stress direction	25	
Particle boards, fibre boards and MDF	30	35

If the buckling load-carrying capacity is insufficient you should
- increase the panel thickness
- put in stiffeners in the length direction to reduce a
- put in stiffeners in the cross direction to reduce l

Generally solution 1) is the most effective; the load-carrying capacity is proportional to t^2.

Solution 2) is also effective; the load-carrying capacity is proportional to $1/a^2$. In practice it may, however be difficult to realise.

Solution 3) is easy to realise but the stiffeners shall be placed rather close (spacing 0,5a to 0,7a).

References:

Halasz, R & Cziesielski, E: Berechnung und Konstruktion geleimter Träger mit Stegen aus Furnierholz. Beichte aus der Bauorschung, Heft 47. 1966.

10-4-1 J Decker, J Kuipers. H Ploos van Amstel
Buckling strength of plywood: results of tests and recommendations for calculation

Summary

Tests have been carried out on 100 specimens of Canadian Oregon Pine-plywood to verify that a reasonable good agreement exists between the buckling theories and the real behaviour of plywood. From load-deflection curves values for a critical buckling strength can be determined, which are in good agreement with theoretical values in the case of simply-supported edges. A clamped boundary condition could not be realised in such a way that the theoretical values were approximated. For design purposes this condition should not be presumed. Attention has been paid to combinations of normal and shear stresses on the basis of theoretical considerations.

17-4-2 R H Leicester, L Pham
Ultimate strength of plywood webs

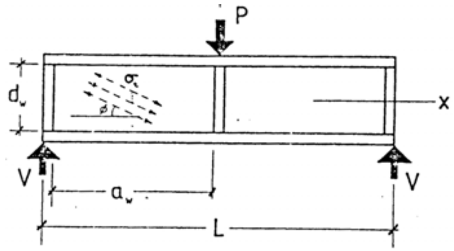
Summary

Methods are given for computing the strengths of both stiffened and unstiffened plywood webs of glued I-beams. These methods, verified by experiments with radiate pine plywood webs, indicate that the ultimate strength of plywood webs can be considerably greater than the critical elastic load.

The strength of plywood webs in I-beams, and box beams is rarely a critical design consideration. Nevertheless, in the interests of ensuring adequate reliability, it is useful to have a method for evaluating the ultimate strength of plywood webs. In the paper two specific problems are investigated: the buckling strength of an unstiffened web supporting a concentrated flange load, and the ultimate shear capacity of a stiffened plywood panel.

Simple procedures are given for estimating the critical elastic loads and the ultimate strength of both stiffened and unstiffened plywood webs for glued I-beams. The methods are obviously relevant to the design of plywood webs of other types of structures.

In practical terms, it is of interest to note that except for extreme designs it is difficult to induce failure in stiffened plywood webs of I-beams: it was also found that for many practical situations it is unnecessary to stiffen a plywood web. The reason being that once a plywood web has buckled, an I-beam will carry additional load through the action of tension membrane stresses, denoted by σα acting at an angle α to the beam axis as shown in the figure.



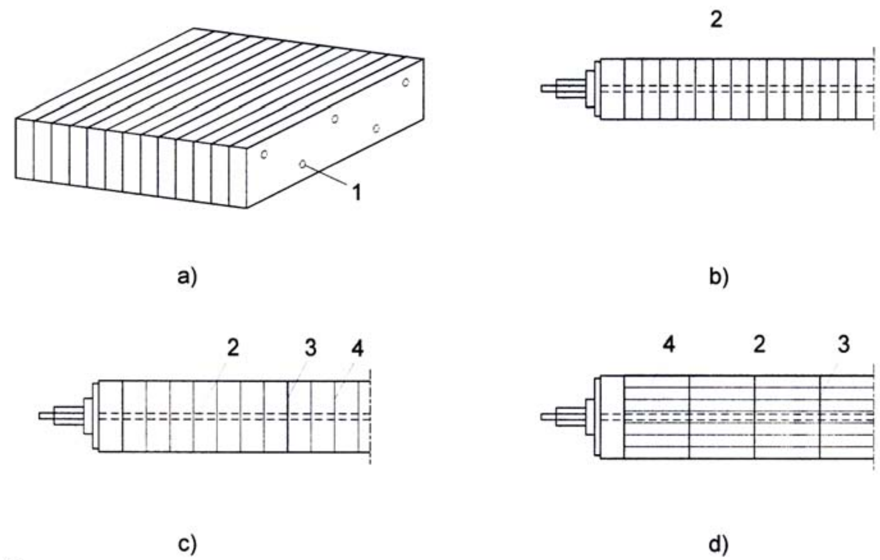
3.11 PRESTRESSING AND REINFORCING

In analogy to steel reinforced concrete, many researchers have suggested to reinforce timber with steel or fibre materials (glass fibres or carbon fibres) although there is a vital difference: As opposed to concrete timber has excellent tension in the fibre direction properties. All the same there are numerous papers on the topic. Generelly improvements are reported, mostly because of a reduction of variability, but in all cases the improvements are small and and the production costly and it would have been much more effective and cheaper to use wood as reinforcement. An exception is repair of damaged structures by gluing on reinforcement.

The same applies to the proposals to prestress timber beams.

An exception is timber bridge decks where prestressing perpendicular to grain because of the low tensile strength perpendicular to grain may be effective or necessary. Eurocode 5.2 (bridges describe several prestressed systems and gives rules for design and detailing,

Examples are shown below.



Key:
1 Nail
2 Prestressing Member
3 Glue-line between glued laminated members
4 Glue-line between laminations in glued laminated members

a) Nail-laminated
b) Prestressed, not glued
c) Prestressed and glued
d) Prestressed glued laminated beams

22-12-4 R Candowicz, T Dziuba
Ultimate strength of wooden beams with tension reinforcement as a function of random material properties

Introduction
Ultimate strength of wooden glue-laminated beams depends on many factors mostly beefing random variables. This phenomenon results in an important scatter of ultimate strength determined experimentally. This paper presents a further step in the analysis of the problem. The digital simulation methods were continuously used in our analysis. The paper deals with the ultimate strength of rectangular cross-sections reinforced with rods made of comparatively strong material.

30-12-1 K Oiger
Experimental investigation and analysis of reinforced glulam beams

This paper discusses the results of an experimental investigation and an analysis of reinforced and prestresssed glulam beams. During many years experimental investigations have been carried out on models in the laboratory of Tallinn Technical University. The studies involved beams both, reinforced with deformed bars and prestressed with steel cables. The results show that prestressed glulam beams are applicable in some cases (in particular, by post-tensioning), though a relatively large portion of initial prestress disappears with time.

Summary
Reinforced and prestressed glulam beams could be effectively used when slender heavy loaded beams are needed. Nevertheless the long-term behaviour of prestressed glulam beams needs further research. At present more reliable and simpler solution is given by reinforced glulam beams with glued-in deformed bars.

34-12-3 M Romani, H J Blass
Design model for FRP reinforced glulam beams

Abstract

For several years possibilities to reinforce glulam beams parallel to the grain to increase bending and axial stiffness and ultimate load have been investigated. One method is to use Fibre-Reinforced Plastics (FRP) as a tensile reinforcement. Fibres used were glass fibres, aramid fibres and carbon fibres.

At the University of Karlsruhe a research project was carried out where the load-deformation behaviour of reinforced glulam beams was studied. Thin carbon FRP and aramid FRP were used as reinforcements. Within this research project a design model was developed taking into account the plastic behaviour of timber loaded in compression parallel to the grain. This paper presents the design model and test results of beams loaded to failure to verify the design model.

Summary

For the calculation of the load-carrying capacity and stiffness of tensile reinforced glulam beams a model is derived taking into account the plastic behaviour of glulam loaded in compression parallel to the grain. The model is based on an analytic solution and allows a simple calculation without any iteration steps based on design values of non-reinforced glulam. Different failure modes of FRP reinforced glulam beams are considered. Test results with reinforced beams loaded to failure show that the proposed model leads to conservative values of the load-carrying capacity. This is especially true considering the low quality of the timber used in the tests.

The test specimens mainly failed at the tension side. With a different cross-section set-up reinforced beams are possible failing in a more ductile way on the compression side. The test specimens mostly showed a significant load increase after failure of the timber facing. This was mainly caused by an effective reinforcement even after a bending failure above the FRP layer. Further research will quantify this effect and permit a more economic use of FRP reinforced glulam.

37-12-2 P Alam, M P Ansell, D Smedley
Reinforcement of LVL beams with bonded-in plates and rods – Effect of placement of steel and FRP reinforcements on beam strength and stiffness

Introduction

Two metre long laminated veneer lumber (LVL) structural beams were reinforced using four different geometrical configurations of reinforcement. Each configuration was reinforced using either grade 43 mild steel or one of three different types of pultruded fibre reinforced plastic (FRP) composite. The objective of the research is to determine the reinforcement capabilities of different reinforcing plates and rods as a function of (a) the reinforcing materials, (b) the geometrical configurations of the four phases of reinforcement and (c) the volume fraction of reinforcement. Objectives (a) and (b) were reached by laboratory testing of reinforced beams for all four types of reinforcement and objective (c) was achieved by numerical simulations for the four phases of steel reinforcement. The paper concludes with design recommendations based on density and cost considerations for all of the experimentally tested reinforced beams.

Conclusions

– Grade 43 mild steel and FRP reinforcements were tested in four phases with different geometrical configurations.
– Examination of the flexural strength and stiffness data showed that smaller volumes of plate and rod reinforcement at the outermost tension and compression fibres of the beam are as effective as full depth flitch plates for improving mechanical properties.
– Using the transformed section approach, it was found that lower modulus reinforcements such as GFRP and FULCRUM are as effective as higher modulus reinforcements such as mild steel and CFRP for improving the mechanical performance of reinforced LVL beams.
– Non-linear finite element models were developed for steel-LVL beams to predict the flexural modulus and the yield strength as a function of the reinforcement volume fraction.
– Finite element predictions for steel reinforcement showed that better flexural properties are attainable by locating small plates at the outermost fibres than similarly located rod reinforcements with equal volume fractions.
– Design recommendations regarding the geometrical arrangements of composite components and of reinforcing materials have been dis-

fraction, density and cost. Design recommendations are based on both numerical and experimental findings.

cussed with respect to enhancement of mechanical properties, volume fraction, density and cost. Design recommendations are based on both numerical and experimental findings.
– Low cost, low modulus reinforcements strategically located within LVL beams have a great deal of potential for use in practical reinforcement of structural timber.

39-12-2 K U Schober, S Franke, K Rautenstrauch
In-situ strengthening of timber structures with CFRP

Introduction
Particularly old timber structures in residential houses are designed for lower live loads as specified in performance and design standards and need new technologies to increase the load-carrying capacity of the members for state-of-the-art housing conditions. Therefore, a study of reinforcement techniques for restoration and strengthening of existing timber floors under bending loads has been carried out at the Bauhaus-University of Weimar, based on the use of carbon fibre reinforced plastics on the building site, whereby the removal of the overhanging part of the structure as well as the inserted ceiling is not necessary.

Reinforcement techniques for structural timber elements, based on the use of adhesives on site, have been applied for some decades as an extension of procedures that became very common for the repair or the upgrading of other structures. Some problems have prevented the wider use of adhesives, particularly in historical timber structures. One reason is that a life-long service life has not yet been fully proven for synthetic adhesives, since the oldest bonded joints are around sixty years and greater ages cannot be simulated by existing accelerated ageing tests. Although epoxy based adhesives have been used in most cases for on-site repair jobs, most formulations were developed for other materials. On-site application of adhesives is somewhat difficult and the consequent quality of adhesive bond is not easy to evaluate. Since properties of reinforced elements very much depend on the care put in the work, such difficulties have to be overcome and first procedures for applying and controlling were established (CEN TC 193/SCl/WGI 12003).

Conclusions
The use of CFRP as a strengthening technique can be applied without necessitating the removal of the overhanging part of the structure. This is very promising in many cases of reinforcement of old, historical structural wood parts. Upgrading traditional timber structures of old solid wood with carbon fibre reinforcement on different locations are described and discussed here based on experimental investigations. For practice related investigations effective cross-sectional data including existing cracks, knots and damages were used with a reduction of the initial bending stiffness and the moment of inertia respectively of about 17 % compared to the cross-sections without defects. The results of the experiments have highlighted the limitations of the composite structure as well as the advantages of the various reinforcement positions and present numerous interesting aspects. The wood beams reinforced with CFRP lamellas revealed more ductile behaviour with compared to un-reinforced beams. The presence of CFRP reinforcement arrests crack opening, confines local rupture and bridges local defects in the timber especially for reinforcement types other than on the bottom face.

The various theories of bonding developed so far are not able to explain comprehensively the observed effects. Chemical bonding always has been seen as the optimal form of combining two surfaces with each other, but its contribution to the overall bonding mechanism is still unclear. The properties of the glue line can be described among other methods by the analysis of the microstructure of the bond surface. This includes the adhesive penetration into the wood surface, the effect of ageing of a glue bond as well as the description of the cohesive strength of the glue line in terms of an optimization of the brittle and elastic ratio of the glue line. These methods took into account by applying a numerical model for this reinforcement types using finite element cohesive zone modelling. These investigations are currently going on.

41-10-2 M Fragiacomo, M Davies
Evaluation of the prestressing losses in timber members prestressed with unbonded tendons

Introduction
Applying prestressing to timber structures has been done in a number of cases; however, unlike concrete structures, it is not common practice. Unbonded prestressing tendons have been used to reduce the deflections of sawn timber beams, to laminate bridge decks from independent timber planks, and to reduce creep-induced deformations and provide increased strength for timber-concrete composite floors. Recently, a new construc-

tion system for multi-storey timber buildings has been proposed at the University of Canterbury, New Zealand. This system consists of frames and walls made from LVL (laminated veneer lumber) prestressed with unbonded tendons for earthquake resistance. Prestressing is mostly used to achieve connections that accommodate the inelastic seismic demand through rigid rocking motion of one member on the other, minimizing the residual damage at the end of the earthquake.

There is some scepticism among the scientific community regarding the possibility of prestressing timber members. The main issue is the behavior of the system in the long-term. The tendon relaxation, together with the time-dependent phenomena of timber such as creep, mechano-sorption, and shrinkage/swelling may in fact significantly reduce the effect of the prestressing in the long-term, particularly if the structure was built in an environment characterized by high relative humidity. A further problem is that codes of practice for timber design do not usually include specific provisions for the evaluation of the prestress losses in the long-term. The Eurocode 5 Part 2 only states that, for stress-laminated deck plates, the long-term residual prestressing stress may normally be assumed to be greater than 0.35 MPa, provided that the initial prestress is at least 1.0 MPa; the moisture content of the laminations at the time of prestressing is not more than 16%; and, the variation of the deck plate's in-service moisture content is limited by adequate protection. No formula such as that reported in the Eurocode 2 Part 1-1 (CEN 2003) for the loss of prestressing in post-tensioned concrete members is provided.

In order to address those issues, a research project was undertaken at the University of Canterbury, New Zealand. A number of LVL frames prestressed with unbonded tendons and subjected to different environmental conditions were monitored over time with the purpose of evaluating the prestressing

Concluding remarks

The paper presents the derivation of a closed-form solution for the evaluation of the prestress losses of a timber frame in the long-term. The formulas are obtained by simplifying the integral equations describing the time-dependent behavior of the prestressing steel and timber loaded parallel and perpendicular to the grain. Allowance for relaxation, creep and inelastic strains was made. The age-adjusted effective modulus method was used to transform the integral equations into algebraic equations.

The closed-form solution was compared with the experimental results measured in a long-term test performed at the University of Canterbury, New Zealand. An acceptable approximation was found, bearing in mind the scatter of experimental values, particularly the Young's moduli and creep coefficients of LVL. The experimental-analytical comparison also allowed a calibration of some coefficients, the "aging coefficients", used in the analytical solution. Based on the outcomes of the parametric study, the analytical solution was simplified further and rewritten in non-dimensional form. Such formula, easy and similar to that used for prestressed concrete structures, can be recommended for design of prestressed timber frames. The formula also allows an easy understanding of the main parameters affecting the prestress losses in the long-term, which are: (i) the ratio between the lengths of timber loaded perpendicular and parallel to the grain, and (ii) the ratio between the axial stiffness of the timber frame and the axial stiffness of the steel tendon.

42-12-3 T G Williamson, B Yeh
Laminating lumber and end joint properties for FRP-reinforced glulam beams

Abstract

Glulam beams reinforced with fibre-reinforced-polymer (FRP) are typically manufactured using graded laminating lumber with FRP reinforcement in the tension zone and have been used in numerous construction projects worldwide. Unfortunately, the lay-up combinations currently used for manufacturing FRP-einforced glulam are all proprietary even though an FRP glulam standard, ASTM D 7199, *Standard Practice for Establishing Characteristic Values for Reinforced Glued Laminated Timber (Glulam) Beams Using Mechanics-Based Models*, was published in 2007. This standard resulted from years of joint effort by APA – The Engineered Wood Association and the Advanced Engineered Wood Composites Center (AEWC) of the University of Maine.

A mechanics-based computer model, called ReLam, has been developed by the AEWC for predicting the performance of FRP-reinforced glulam beams in accordance with ASTM D 7199. This model can be used to develop a range of layup combinations for FRP-reinforced glulam beams. As the first step toward the development of these glulam lay-up combinations, mechanical properties of laminating lumber, including the tensile and compressive strength and moduli, and the end joint tensile strength must be obtained.

While the lumber properties for these lumber grades have been published in glulam industry standards, such as AITC 407, *Standard for Alternate Lumber Grades for Use in Structural Glued Laminated Timber*, the basis of those data is more than 20 years old and was in need for an update and reaffirmation. While there are end joint tension data generated daily from the quality control records of each glulam plant, those data are considered proprietary and typically unavailable for general use. In addition, the compressive strength and moduli data are not readily available. As a result, APA initiated a comprehensive study to evaluate four Douglas-fir (DF) laminating lumber grades; 302-24 tension lam, L1, L2, and L3, and two grades of end joints, 302-24 tension lam and L1, from a range of glulam plants in 2008 using current production resources in the U.S. These laminating lumber properties, including end joint data, not only confirm the historical data for these dominant DF lumber grades and end joints used in today's glulam production in the U.S., but also provide the required data for the development of lay-up combinations using ReLam. This paper presents the laminating lumber and end joint properties. In addition, the likely FRP-reinforced glulam beam layup combinations at characteristic bending strength and modulus of elasticity values of 43.4 N/mm^2 (6,300 psi) and 13800 N/mm^2 (2.0 · 106 psi), and 49.2 N/mm^2 (7,140 psi) and 15200 N/mm^2 (2.2 · 106 psi), respectively, are also provided. These FRP-reinforced glulam beams are expected to be highly competitive with steel in non-residential construction markets in the U.S.

Conclusions and Recommendations

Results obtained from this study provide the characteristic properties of Douglas-fir laminating lumber that can be used for the modelling of FRP-reinforced glulam in accordance with ASTM D 7199. These data also confirm that the currently published laminating lumber properties are adequate albeit in some cases conservative. For the purpose of the glulam modelling using the bilinear compressive stress and strain curve, further investigation on the accuracy of the estimate for the downward slope after the compression failure will be conducted.

A model analysis was conducted for the development of candidate lay up combinations with a targeted characteristic bending strength of 48.3 N/mm2 (7000 psi) and mean bending modulus of elasticity of 14480 N/mm^2 (2.1 x 106 psi) based on a reinforcement ratio of about 3% utilizing low-grade laminations. The confirmation of the properties for these layup combinations is pending the completion of full-scale beam tests.

Once the confirmation of the ReLam model is completed, APA will pursue obtaining a code evaluation report based on ICC-ES AC280 [13], *Acceptance Criteria for Fiber-Reinforced-Polymer Glued Laminated Timber Using Mechanics Based Models*, on behalf of its manufacturing members. This will allow APA members to be listed on the code evaluation report and significantly expand the use of FRP-reinforced glulam in the U.S.

43-7-7 S Giorgini, A Neale, A Palermo, D Carradine, S Pampanin, A H Buchanan
Predicting time dependent effects in unbonded post-tensioned timber frames and beams

Introduction

Development of engineered timber products such as Laminated Veneer Lumber (LVL) has caused a renewal in the development of heavy timber construction. A research programme on the application of unbonded post-tensioned construction techniques similar to concrete have been successfully proven with LVL at the University of Canterbury. This technology has been applied firstly to earthquake resistant frames and then, given the advantages that this solution offers, extended to long span beams for gravity loading. Extensive numerical and experimental research is continuing at the University of Canterbury supported by the Structural Timber Innovation Company (STIC).

Rocking connections in post-tensioned frames and walls allow discrete dissipative devices to be placed in specific locations in the structure, providing energy absorption during seismic loading, with no damage to structural members. This system has been developed throughout the last two decades for precast concrete structures, known as "PRESSS" (Precast Seismic Structural Systems) and it has been successfully transferred to timber frames and walls known as "Pres-Lam". Because the moment capacity of post-tensioned timber connections depends on the level of post-tensioning force applied by tendon, losses must be accurately assessed to ensure the structure maintains seismic integrity throughout its life. Generally the largest components of the loss are the effects of creep in the wood and relaxation of the steel tendon. The problem is also complicated by the particular column-to-beam connection where post-tensioning tendons pass through the column, loading it perpendicular to the grain, where the stiffness of the wood is much lower than parallel to the grain, so this effect can lead to considerable losses. This issue has been investigated through experimental tests.

While the Pres-Lam system developed for multi-storey timber buildings (Fig. 1a,b) uses straight tendons in the centroid of the section, the use of post-tensioning with draped or eccentric profiles becomes a suitable solution for long span timber beams, either in post-tensioned frames or as stand-alone beams. Enhanced performance of this solution has been confirmed through experimental investigations on two static schemes evaluating different tendon profiles (Fig. 1c)). The major benefit was a displacement only 60% of that for the non-post-tensioned beam. In addition to this, a significant nonlinear stiffening effect due to tendon elongation has been identified. In order to properly quantify the benefits of this technology, long-term issues have to be evaluated. Recent preliminary studies have confirmed that post-tensioning losses can be estimated around 25% in the worst environmental scenario. In order to validate the analytical approach, long term tests are ongoing at the University of Canterbury.

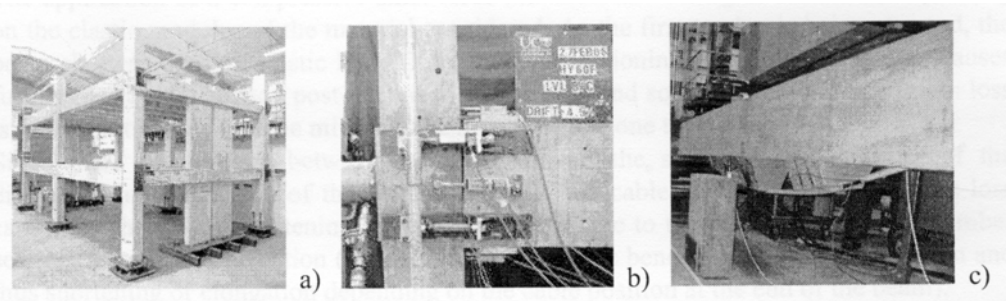


Fig. 1: Unbonded post-tensioned timber frames and beams: a) Pres-Lam timber building tested for earthquake loads [9]; b) Rocking connection; c) Post-tensioned timber beam tested for vertical serviceability loads.

The scope of this paper is to propose design procedures and simplified expressions in order to accurately quantify post-tensioning losses in long span beams and timber frames (Pres-Lam system). Both instantaneous elastic and long-term losses are investigated. By applying typical creep formulations to post-tensioned timber members with an eccentric force, some simplified expressions have been developed for beams in a form suitable for standardisation. The analytical formulation is carried out for a simply supported beam in the elastic and long-term ranges. The formulation uses a single dimensionless coefficient Θ_p in order to encompass all the involved geometric and mechanical parameters (material properties, beam geometry, steel area and the type of cable profile).

For frame systems, a simplified expression for straight concentric post-tensioning, validated through monitoring of a Pres-Lam timber building, is proposed as a viable tool for predicting post-tensioning losses taking into account column joint reinforcement. Simplified design charts are presented in order to facilitate the use of the proposed formula in a design procedure.

Conclusions

For design of post-tensioned timber beams, proper assessment of the post-tensioning losses is crucial for establishing the initial post-tensioning force and to maintain the long-term structural performance.

In this paper post-tensioning losses have been expressed for beams and frames in a common analytical formulation, expressed as function of a global mechanical parameter. Solutions based on this method have been proposed, to simplify the calculation of such complicated phenomena by practitioners.

In addition, for frames, the effect of the column reinforcement is taken into account and design charts have been created in order to further simplify the calculations.

43-7-8 M Newcombe, M Cusiel, S Pampanin, A Palermo
Simplified design of post-tensioned timber buildings

Introduction

A high performance solid timber frame system has been developed at the University of anterbury, in collaboration with the Structural Timber Innovation Company (STIC Ltd). The structural system uses post-tensioning tendons or bars to connect large timber sections (see Figure 1). The sections are constructed from engineered wood product, such as Glulam, Laminated Veneer Lumber (LVL) or Cross Laminated Timber (CLT), forming beams, columns, walls or floors. Post-tensioned timber is suitable for a wide range of building types, including commercial structures, and has the potential to compete with existing forms of construction in terms of cost, versatility and structural performance

The connection technique was adapted from post-tensioned pre-cast concrete systems. For seismic design, the combination of timber and post-tensioning is particularly efficient since it avoids potential brittle failure modes that can occur in traditional timber connections and prevents excessive frame elongation, avoiding damage to the floor system . Hence, the system fits well into recent Performance-Based Seismic Engineering (PBSE) design approaches because structural damage is minimized and, due to restoring action provided by the post-tensioning, residual deformations are insignificant.

Previous research has shown that simply designed post-tensioned timber frames respond essentially elastically to even severe earthquake loading. In such cases, for code-based seismic design, the ultimate limit state lateral forces cannot be significantly reduced due to structural ductility or hysteretic damping, in contrast to well designed concrete and steel structures. Even so, the serviceability limit state lateral load design usually governs the size of the members and the amount of post-tensioning, especially if stringent displacement limitations, according to current code provisions, are imposed to protect non-structural partitions inside a building. For an elastic design, for service or ultimate conditions, either force-based or displacement-based design can be used to determine the lateral seismic forces for post-tensioned timber frames. However, for force-based design it is essential that the allowable displacements are used to determine the natural period of the frame

The lateral resistance of post-tensioned timber frames is highly dependent on the behaviour of the beam-column connections. Analytical models have been proposed to predict the moment-rota further validation as more experimental data is obtained. Recent research has demonstrated that the shear deformation of the timber columns in the joint panel region, between the beams, significantly contributes to the overall frame flexibility. Such considerations are not required for equivalent precast concrete connections. To enable accurate design of post-tensioned timber frames, to meet specified displacement limitations, all deformation components must be quantified and accounted for.

In some situations additional non-prestressed reinforcement may be added to the beam-column connections (see Figure 1) creating what is termed a ‘Hybrid’ connection. With careful detailing, this reinforcement can improve the strength of the connections and the energy dissipation capability of the frame at large displacements.

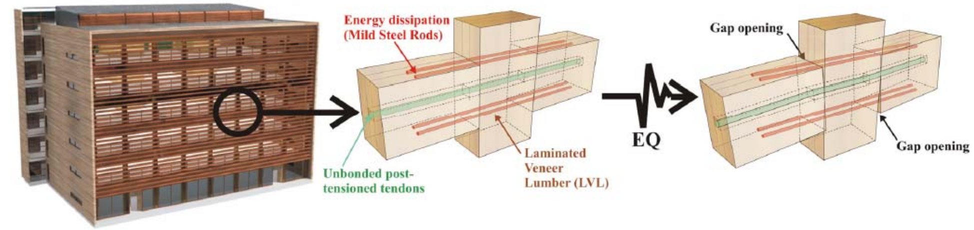


Figure 1. Post-tensioned timber frame concept

While lateral load design methodologies for post-tensioned concrete and steel frames exist , these procedures are not directly applicable to timber frames. This paper describes a simplified design procedure for posttensioned timber frames- The focus is given to characterisation of frame deformation. For simplicity, hybrid connections are not considered here.

Conclusions

A simplified design procedure has been proposed for the lateral force design of simple post-tensioned timber frames, responding within the elastic range. The procedure allows the determination of the required section sizes and post-tensioning forces to achieve the lateral strength demand of the frame, without exceeding the total allowable displacements for either serviceability or ultimate limit states. All significant deformation components are accounted for using analytical expressions.

For predicting the moment-capacity of the beam-column connections, a refined methodology is proposed that, when compared to previously published procedures for post-tensioned timber connections, more accurately predicts the peak timber stresses within the connection and can avoid iteration. Empirical relationships have been proposed hich were calibrated from detailed finite element modelling and experimental data.

3.12 RESIDENTIAL FLOORS

ESSAY 3.9 H J Larsen
Vibrations of floors

Vibrations unacceptable to the inhabitant have often been reported for residential floor design outside the very traditional fields with regard to materials and design and the problem have been in focus in CIB-W18.

For vibrations an important parameter is the structure's natural frequency, **Paper 19-8-1** gives 'designer useable' methods for predicting the dynamical behaviour of light-weight wooden joisted floors covered with semi-rigidly attached wood based sheathings of materials such as chipboard or plywood. It is demonstrated that good approximations to the fundamental natural frequencies of floors with practical combinations of edge conditions can be obtained by assuming that a floor behaves as a simple composite beam. This is the background for the simple expression given in Eurocode 5.

It is generally agreed that 8 Hz is a critical value and that a special investigation should be made for residential floors with a fundamental frequency less than 8 Hz. It is also generally agreed that the deflection under a static unit load should be limited either as an absolute value or as a function of the span.

These two parameters are not sufficient to ensure a satisfactory behaviour, and what other criteria should be used has been discussed in several papers.

Based on a Swedish proposal the unit impulse velocity response (v), i.e. the maximum initial value of the vertical floor vibration velocity (in m/s) caused by an ideal unit impulse (1 Ns) applied at the point of the floor giving maximum response has been chosen. Other proposals are

- the mean magnitude of the response caused by human footfall impact
- the frequency-weighted root-mean-square acceleration (A_r) of the response caused by a normal human footfall impact

According to Eurocode 5 it may be assumed that floors with a fundamental frequency greater than 8 Hz are satisfactory provided

$$w/F \leq a \text{ mm/kN}$$

and

$$v \leq b^{(f_1\zeta-1)} \text{ m/(Ns}^2\text{)}$$

where

w maximum instantaneous vertical deflection caused by a vertical concentrated static force F applied at any point on the floor, taking account of load distribution

v unit impulse velocity response, i.e. the maximum initial value of the vertical floor vibration velocity (in m/s) caused by an ideal unit impulse (1 Ns) applied at the point of the floor giving maximum response

ζ modal damping ratio.

Eurocode 5 gives recommended ranges of limiting values of a and b. The values to be used in a specific country should be taken from the National Application Document.

The background for the Eurocode 5 clauses may not be found in a CIB-W18 document but in e.g. Ohlsson (1988).

Reference:

Ohlsson, S: Springiness and Human-Induced Floor Vibrations; A Design Guide, 1988, Swedish Council for Building Research, Stockholm.

19-8-1 I Smith, Y H Chui
Predicting the natural frequencies of light-weight wooden floors

Abstract
This paper gives 'designer useable' methods for predicting the dynamical behaviour of light-weight wooden joisted floors covered with semi-rigidly attached wood based sheathings of materials such as chipboard or plywood. It is demonstrated that good approximations to the fundamental natural frequencies of floors with practical combinations of edge conditions can be obtained by assuming that a floor behaves as a simple composite beam. Guidance is given concerning the calculation of natural frequencies of floors simply supported at all four edges for higher order modes of vibration.

Discussion and conclusions
The objective of this paper is to give designer useable methods of predicting the dynamical response of light-weight wooden foisted floors. However, this is only one part of the overall equation needed to predict the likely acceptability of a particular design solution. The designer also requires a design criterion which relates parameters such as the fundamental natural frequency, or several of the lower natural frequencies, of a floor to the likely 'levels' of vibration that will be acceptable to the end-user(s), i.e. occupant(s) of a building. Recent work by Chui has indicated that human perception of, and tolerance thresholds on, vertical vibrations are related to quantitative measures such as amplitude, velocity and damping of the vibrations and qualitative factors related to use of a building or even a specific part of it. Root means square acceleration, r.m.s., due to a defined impulse of a magnitude related to end use conditions appears to be the best criterion. The question of how to relate an acceptance threshold for r.m.s. to parameters that are easy to calculate such as natural frequencies is the topic of ongoing studies by the second author and will be reported elsewhere.

20-8-1 Y H Chui. I Smith
Proposed code requirements for vibrational serviceability of timber floors

Abstract
This paper proposes how vibrational serviceability requirements for timber floors can be incorporated into a design code. Simple expressions are presented for estimating the fundamental natural frequency and the mean magnitude of the response caused by human footfall impact. These two parameters have been identified as the two governing parameters for assessing prospective design solutions. Limits are also given for these parameters for ensuring satisfactory floor designs.

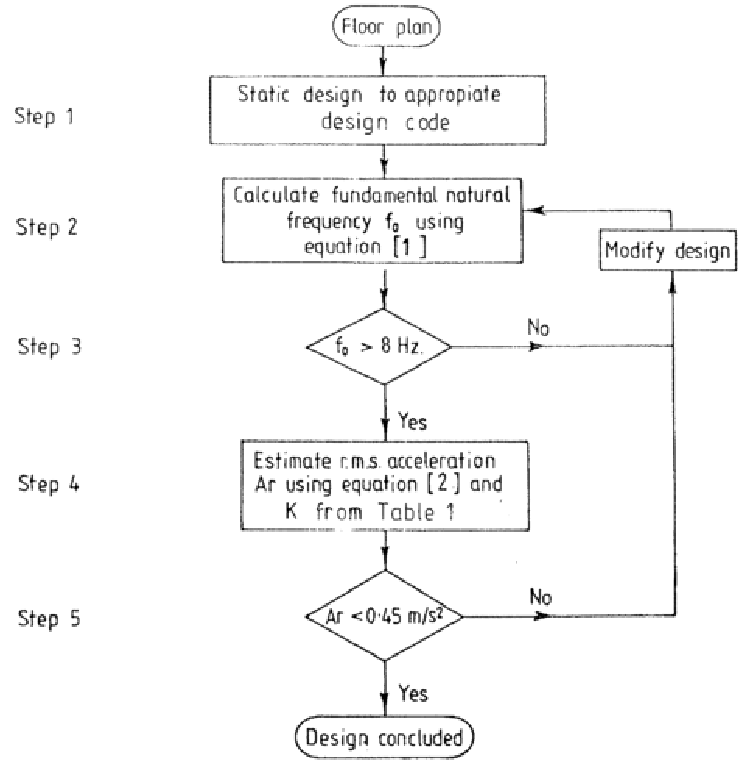


FIG.1 .Flow chart for proposed design method

Conclusion
It is proposed here that in future design of wooden floors should follow the procedures set out in Figure 1. In order to ensure that a floor is free from excessive vibration which causes discomfort to its occupants, its fundamental natural frequency (f_n) and frequency-weighted acceleration (A_r) of

the response due to a human footfall impact shall be higher than 8 Hz and smaller than 0.45 m/s^2 respectively. These two parameters ▸ may be satisfactorily estimated using the equations presented in this paper.

21-8-1 Y H Chui, I Smith
An addendum to paper 20-8-1 - proposed code requirements for vibrational serviceability of timber floors

Introduction

At the 1987 CIB-W18A Meeting, the authors proposed a possible code approach for designing against poor vibrational serviceability for lightweight wooden floors in the domestic setting. Suggestions by certain participants of the meeting have led to further work by the authors. This mainly involved additional discussion with others, who are investigating or have investigated related problems, and conducting a comparison between various design approaches.

Main differences between different design approaches are presented. Assessments of vibrational serviceability, using each approach, of a series of wooden floors covering a wide range of floor sizes, are presented.

In addition, possible extension of the proposed design equations in the previous paper (**Paper 20-8-1**) to include the stiffening effect due to a flooring with high bending stiffnesses is also discussed.

Conclusion

The additional information supports the previous contention that the design method proposed by the authors should be accepted as suitable for inclusion in future editions of the CIB Code.

21-8-2 S Ohlsson
Floor vibrational serviceability and the CIB model code

Introduction

The aim of this paper is twofold:

– To initiate a general discussion about to what extent and in which forms floor serviceability related design methods and criteria should be presented in international model codes.
– To comment upon a paper which was presented at the last meeting of CIB-Wl8A (**Paper 20-8-1)**

It has been a long time tradition in the structural design of houses and civil engineering structures to focus on the load-bearing capacity and to relate the design to different codes of practice. What to-day is known as the serviceability limit state is a rather modern concept in the construction area, relatively speaking. If we, for a moment, compare the situation with some other sector of engineering, we find that the corresponding words used often are "performance" and "comfort", etc. This is for instance the case in the automotive industry, where the "serviceability" of the product is of main importance to the design, but is normally not regulated by codes. The serviceability criteria used in this sector, on the other hand, do stimulate the technical development of the car design. As examples of such development we nowdays have cars with smaller unsprung wheel masses, individual suspensions, etc. This may not have been the case if the serviceability criteria in use had been of the same type as we use for the structural design of houses.

Research in the area of floor vibration and serviceability has been undertaken in several countries and is still going on. Reference 4 (Ohlsson, S: Springiness and Human-Induced Floor Vibrations; A Design Guide, 1988, Swedish Council for Building Research, Stockholm. ISBN 91-540-4901-6) is a design guide, which also includes a proposed design method. It has been in use among practicing engineers in Sweden and occasionally in Norway and Finland since 1984 and so far it seems to work reasonably well.

Concluding words and suggestions

Considering various aspects of the matter it is the author's opinion that a sound engineering development of floor structures will benefit if:

– The design proposal (**Paper 20-8-1**) is not accepted for inclusion in the CIB model timber code.
– The authors of the proposal contribute together with other specialists in the field of floor vibration in the process of international coordination aiming at design criteria that are independent of construction material and configuration.
– Scientists and engineers with special knowledge in any specific construction material contribute in research and development aiming towards recommendations about which structural configurations should be preferred, which detail designs that are most suitable, etc. Such work could preferably be based on the result of the abovementioned international coordination.

24-8-1 I Smith, Y H Chui
On the possibility of applying neutral vibrational serviceability criteria to joisted wood floors

Abstract
Prior discussion in CIB-W18A has not clarified whether it is possible to apply vibrational serviceability criteria that are neutral with regard to construction material and structural configuration for floors. By means of examples, it is demonstrated that responses of joisted wood floors are too complex for correct quantification, at the design stage, of parameters that indicate acceptability. Applying arbitrary conditions for imposed mass, excitation and response limits, designers have a means of assessing relative suitabilities of candidate solutions. Candidates would satisfy other (static) design criteria. "Neutral" vibrational serviceability limits in design of joisted wood floors is an ideal that is not presently attainable.

Conclusion
It is beyond a designer's ability to make quantitative vibration response predictions for joisted wood floors-. Arbitrary choices must be made regarding variables such as the amount of imposed mass, its distribution, and the excitation assumed in design. Arbitrary choices must also be made regarding limits on response. Armed with this approach sensible choices can be made on the basis of relative behaviours of a range of candidate design solutions. The candidates would satisfy static design criteria for both strength and serviceability.

Although a worthy goal to which to aspire, material and structural configuration neutral vibrational response criteria for joisted floors are not valid for contemporary engineering design codes.

30-20-2 R J Bainbridge, C J Mettem
Serviceability performance of timber floors – Eurocode 5 and full scale testing

Abstract
This paper describes recent research relating to the Serviceability Limit State (SLS) performance of timber floors.

The findings of full scale tests are summarised and related to Eurocode 5 based design of one-way spanning timber floor constructions.

Conclusions
SLS criteria related to deflections and vibration frequently govern the design of horizontal timber elements, hence it is necessary to ensure that design procedures are not unduly conservative, in order to promote efficient design and material usage.

The reasons for limiting deflections must be understood and defined by the designer, who, through modern code approaches, has greater discretion when considering serviceability.

This requires consideration of the action combinations and expected resultant consequences in terms of functionality, comfort and appearance.

It has been found that the floor design methods relating to SLS in timber floors presented in EC5 can be applied to predict test floor behaviour to a reasonable degree of accuracy, provided certain factors are known or can be approximated for inclusion in the formulae.

Since strutting provides stability to joists and enhances overall serviceability performance, both in terms of static stiffness and influence upon the complex aspects of the dynamic vibration performance, its provision as directed by current guidance should be regarded as a minimum.

It is also clear that the contribution of decking and plasterboard ceilings to the behaviour of the floor should be considered in the design process, since it is currently ignored. Whilst it is accepted that ultimate limit state design should be based on the primary structural system (i.e. the joists alone), this has implications relating to design under both static and dynamic aspects of serviceability.

32-20-1 B Mohr
Floor vibrations

Introduction
The serviceability becomes more decisive nowadays because of the use of high strength materials, plain timber construction elements (e.g. nail laminated timber decks) and longer spans. Beside the deflection requirements the dynamic aspects must be considered.

Eurocode 5 requires for residential floors having a fundamental frequency below 8 Hz "a special investigation" which is not described there. Usual wooden floors having beams and boardings may be modelled as grill girders. Such a computation of the deflection because of a concentrated static force for wooden floors is not contained in standard tables and requires a lot of time. A research project with this scope was performed in 1998 at the Technical University of Munich by Kreuzinger and Mohr.

37-20-1 Lin J Hu, Y H Chui
A new design method to control vibrations induced by foot steps in timber floors

Abstract
Design methods contained in building codes and structural design standards to prevent excessive vibration in timber floors have not been found to provide satisfactory solutions for the broad range of floor systems encountered in contemporary timber construction. This paper describes the development of a new design method. The design method consists of a dual-parameter performance criterion and the associated calculation method for the performance criterion parameters. The derivation of the performance criterion was based on extensive field testing data and an occupant survey. The two parameters selected for design purpose are static deflection at floor centre under a point load and first natural frequency. The calculation method is based on the ribbed-plate theory and considers seimirigid connections between joist and sheathing, torsional rigidity of joists, sheathing stiffness in the span and across joist directions and the contribution of lateral performance enhancing elements such as between-joist bridging, strong-back and strapping. Comparison of predicted performance with recorded rating of a group of timber floors shows that the proposed design method works reasonably well. Only about 3 of the floors were slightly mis-classified as acceptable by the method. A few acceptable floors were 'conservatively' classified as unacceptable. This has provided some confidence for adopting the proposed method for design use.

Conclusion and Recommendation
It can be concluded that the new design method shows great potential to provide rational vibration-controlled spans for a broad range of timber floor systems. The mechanics-based approach also lends itself to facilitate the acceptance of innovative building products. The new design method presented in this paper will provide a framework for the formulation of an acceptable design method for a broad range of timber floors. Additional calibration work may be required to apply the tentative design criterion to timber floor designs in other countries because it was calibrated to material properties published in the Canadian literature.

3.13 ROBUSTNESS

All structures are required to be reasonably robust, i.e. they should not be sensitive to unintended incidents or small deviations from the design assumptions e.g. small deviations from the intended geometry due to execution errors, or foundation settlements.

Robustness (sometimes called safety against disproportionate or progressive collapse) requirements are those that ensure that in case of an accident the structure do not fail disproportionate to the cause of the accident, i.e. failure in a small part of the structure must not result in failure of the whole structure .

Robustness may be ensured by the design of the statical system, e.g. by choosing statically indeterminate structures where there is more than one path for the loads to the supports, by choice of materials and joints and by control of the construction execution. It should be noted that increase of the safety factors rarely by itself has an important effect even though the Safety codes sometimes accept this as a possible way of compliance.

The codes are rather vague regarding implementation of the robustness requirements into practice and it is usually left to the designer

22-15-6 C J Mettem, J P Marcroft
The robustness of timber structures

Introduction

Structural timber is frequently used for clear spans in excess of nine metres in public buildings in Britain. Such applications are being actively promoted. For example, recreational activities and competitive sports are increasingly being enclosed by means of timber structures. This represents an ideal category of end-use for glulam. Also, with the trussed rafter market for domestic applications being largely saturated, that sector of the industry is increasingly offering solutions for roofs for non-residential groups of building. In these, clear spans of up to twenty metres are not uncommon. Glulam has far greater spanning potential, of course. Public buildings using glulam in excess of forty metres span do exist in Britain, although they are not at present commonplace.

On the continent of Europe, in Scandinavia and in North America, timber structures in excess of sixty metres span are relatively frequently found. The structural advantages, including lightness during erection and in foundation design, and good fire resistance, are well known to specialists. These features of structural timber can readily be supported with ample documentation. The ability of a timber structure to resist misuse however, and to remain stable after an accident which has affected its fabric, is less generally appreciated. In addition, the robustness of timber structures is a subject which is extremely sparsely documented. Yet it could probably be claimed that the majority of timber structures do possess an inherently high degree of robustness; or that robustness could be added with little extra detailing costs. The excellent performance of timber structures in seismic regions is almost legendary.

In Britain, there is a perceived disadvantage for timber in the way that the particular Building Regulations requirements for robustness might be achieved. There is also a lack of clarity, or even a complete gap in information, as regards provisions for meeting the specific current requirements for avoidance of disproportionate collapse in public buildings with clear spans exceeding nine metres.

Furthermore, TRADA became aware that the concept of structural robustness and the general principle of avoidance of damage disproportionate to the original cause were to be incorporated in Eurocode No 1 and in the individual material codes for Europe.

As a consequence, a three year research project was proposed to collate and to supplement information on the response of timber structures to forces leading to collapse. During the project, the ability of selected structural timber forms to resist damage would be investigated. Finally, and as quite a major apportionment of the effort, improved design guidance on the subject would be drawn up. The proposal for such a project was approved, and the work was commenced, with UK Government (DOE) partial sponsorship, in March 1989.

The purpose of this paper is to make CIB W-18 members aware of the initial studies which have been undertaken on structural robustness, with special reference to timber. In addition, the consultations and assessments being undertaken will be described, with a view to inviting further comment and suggestions. Information on the requirements and solutions, if any, in other countries which W18 members represent, would be particularly valued.

Proposals

It seems reasonable to assume that the robustness of timber structures and their ability to withstand accidents should be equal to that for other materials. However, the comparison should be made in relation to building size and building occupancy class. Without major cost implications, code writers and designers in steel and reinforced concrete can at least adopt ad hoc precautions which will provide some required additional degree of resistance. In timber, however, it is important not simply to mimic such measures, but to consider the requirements from first principles.

The imposition of new and unacceptable requirements upon timber structures arising merely for the sake of harmonization across materials would be most unfortunate. A consequence of the project might be a conscious decision that for certain classes of structure, including small buildings for example, the risk to life from accidental effects upon the building fabric is 'acceptably improbable'. Taking due account of the expense and effort in further reducing the risk of failure, it might be that design rules and details for such structures are unaltered. Nevertheless, it is important to demonstrate that this is so, rather than dismissing robustness as being of no consequence to timber structures.

In public buildings of some as yet undefined size or type, it seems unlikely that timber code writers and designers will be able to continue to evade the topic. The following steps are proposed, therefore:

– categorize buildings in terms of purpose and end use. The purpose groups included in the 'English' Building Regulations for fire regulations may provide a starting point;
– carry out a study of the availability of data and of the suitability of statistical techniques which may be employed in taking into account the criteria for reliability laid down in EC1 (risk to life; economic losses; expense to reduce risk). In the end, it seems likely that it will be necessary to take best estimates and 'state of art' guidance upon effects of accidental actions, and to apply these analytically to the common features of the forms of construction concerned.

Finally, it is felt that because of the frequently inherently robust nature of timber construction, the effects of a number of forms of accidental action can be provided for, without greatly adding to costs. This will entail acceptance of the fact that a degree of localized damage is inevitable. However, methods can be developed which should enable measures to be described to limit that damage in its extent and to ensure that there are not risks of progressive collapse.

23-15-6 C J Mettem, J P Marcroft
Disproportionate collapse of timber structures

Abstract
Regulations and Codes of Practice are placing increasing emphasis on structures being planned and designed so that they are not unreasonably susceptible to the effects of accidents. The UK Building Regulations contain one such requirement entitled disproportionate collapse" which is applicable to two building categories. Only one of these is of relevance to UK timber building manufacturers, and it covers 'single storey public buildings having a clear span exceeding nine metres.'

This paper describes, with reference to a specific form of roof construction, the differing ways in which the 'disproportionate collapse' requirement has been interpreted. It gives the authors' preferred interpretation and outlines a procedure which designers can follow to satisfy the preferred interpretation of the requirement. The specific form of roof construction considered covers non-load-sharing parallel primary frameworks supported at wall-plate level, and supporting single-span secondary construction.

Summary
In order to ensure the avoidance of disproportionate collapse for parallel-framed structures, two tasks should be brought to the attention of the designer:

– In the event that accidental loading causes local failure in a primary frame, he should be required to ensure that the adjacent primary frames are not dragged down by the failed primary frame.
– His attention should be drawn to the necessity to ensure that the undamaged areas on either side of the damaged zone retain sufficient stability to be able to withstand appropriate wind and other horizontal loads.

Both steps require that bracing is placed in a minimum of two bays, and that normally the bracing is best placed in the two end bays of the building. Step 1 could be satisfied for roof structures incorporating parallel primary frames supported at wall-plate level and supporting single-span secondary members.

31-15-2 C J Mettem, M W Milner, R J Bainbridge, V. Enjily
Robustness principles in the design of medium-rise timber-framed buildings

Abstract

One of the general principles for the structural design of buildings, which is almost universally accepted, is that exceptional conditions should be considered. For example, ENV 1991-1 has the fundamental requirement that a structure shall be designed and executed in such a way that it will not be damaged by events such as fire, explosion, impact or consequences of human errors, to an extent disproportionate to the original cause. In terms of exceptional events other than fire, this is often termed the "Robustness Requirement ".

For residential timber framed construction in the low-rise (1 to 3 storey) category, good robustness has been well established by experience, and has also been demonstrated by test. Such buildings are, by their very nature, cellular, lightweight, tied together and constructed using materials with broadly a good response to many of the forms of accidental event which are of a brief duration.

To address the concerns of the industry and regulatory authorities that better guidance is now required on what is to be expected in terms of robustness for medium-rise timber-frame, and how it can be demonstrated, a pre-feasibility study and a Stage 1 Project was carried out. This led to the planning of a programme of physical tests on a six-storey structure, the aims of which are described in this paper.

Conclusions

Tests currently in progress will permit an evaluation of the actual behaviour of a typical six storey medium-rise timber-framed structure when selected vertical load bearing wall panels are removed. It is hoped that this will verify that the inherent stiffness of cellular platform timber-frame construction can provide robustness, so that in the event of an accident, such buildings will not suffer collapse to an extent disproportionate to the cause.

This research will develop a basis for design guidance on the theoretical procedures required in the future. It is however recognised, that although these whole building tests will produce useful results, they will in some ways be restricted to buildings of a form similar to the test prototype. Therefore, if comprehensive authoritative guidance is to be produced, from which other buildings could be designed/checked against disproportionate collapse, then other tests may be required. These tests would address issues not covered in the current research, such as the comparative effects of removal of internal walls with doors and external walls with openings. Further studies to identify the strength of deep OSB and plasterboard sheathed panels would also be beneficial.

The wider-reaching impacts of such research include not only the permeation of information into codes, standards and regulations, but also enhancement of the fundamental appreciation of this form of structure that will be beneficial to future investigations of full structure and component behaviour, whether through physical test methods and procedures, or as a tool for representative numerical modelling or reliability based studies.

This subject not only has implications for design under accidental situations, but also for appreciation and quantification of material interaction and structural system effects which are applicable to development and refinement of many aspects of limit states design for timber structures as a whole.

43-7-2 A Jorissen, M Fragiacomo
Ductility in Timber Structures

General notes on ductility

At ultimate limit state, timber structural elements generally fail in a brittle manner. Such behaviour is due to the elastic-brittle stress-strain relationship of the material in both tension and shear. Non-linear load-deformation relationships characterized by some plasticization can only be achieved in timber engineering by elements loaded in compression, both parallel and perpendicular to the grain, and through connections. In bending, the load-deformation relationship is generally close to linear due to the elasticbrittle stress-strain relationship in tension, which hinders extensive plasticization in compression.

Within COST Action E55, modelling of the performance of timber structures research has been undertaken into three main subjects:

(1) System identification and exposures; (2) Vulnerability of components; and (3) Robustness of systems. From probabilistic analyses it was demonstrated that ductile structural behaviour positively influences structural robustness [2]. It is therefore of importance to investigate possible ductile behaviour of structural components for implementation into the probabilistic structural robustness analyses. This task has been undertaken by the members of COST Action E55 Working Group 2.

Ductility is an important requirement in structural design. Traditional literature references quote three main reasons for achieving ductile behaviour:

1 to ensure the failure will occur with large deformations, so as to warn the occupants in the case of an unexpected load (e.g. exceptional snow load, etc.);
2 to allow stress redistribution within a cross-section and force redistribution among different cross-sections in statically indeterminate structures (plastic analysis), so as to increase the load-bearing capacity of the structure with respect to the value calculated in elastic analysis. Plastic analysis can only be carried out for structures which exhibit a minimum amount of ductility;
3 to allow energy dissipation under seismic loading. Energy dissipation reduces the effect of the earthquake on a structure, leading to an overall better behaviour. Roughly speaking, the larger the ductility, the lower the seismic action that has to be considered in design. The seismic actions considered in design are therefore related to the ductility of the structure.

In addition, there is another important reason for ductility requirements:

4 to ensure the fulfilment of structural robustness, where the members must be able to accommodate large displacement and rotation demand caused by sudden failure of a single member within the whole structural system. According to analyses carried out in [2], only a little ductility is needed for a significant robustness increase.

It should be noted that stress redistribution in members subjected to bending is traditionally related to ductility. However, this can hardly ever be achieved for timber since it has to be ensured that plasticization in compression occurs before brittle failure in tension; consequently a tensile strength markedly higher than the compression strength is required. For clear wood this is certainly the case. For structural timber containing defects, however, this is only the case when the defects are mainly located in the compression zone. This is also the case for beams strengthened in the tensile area through ductile reinforcement for which some examples are shown in Figure 1.

Under static load conditions, design for ductile failure in the connections is desirable because this allows redistribution of forces between components and subassemblies. This is one of the means of avoiding progressive and disproportionate collapse in structural systems. In seismic design, ductile failure in the connections is desirable because mechanical connections are the only significant sources of ductility and energy absorption during cyclic loading.

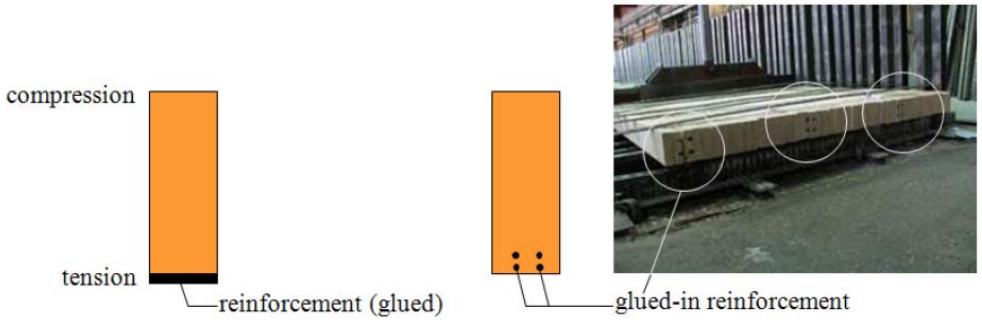


Figure 1: Reinforcement of beams (bending) for stress redistribution purposes.

Timber structures are mostly designed as a set of individual statically determinate elements connected to each other. However, most structures end up being highly statically indeterminate due to the (semi) rigidity of most connections modelled as perfectly pinned in the design phase. Consequently, it can be expected that failures in systems initiate and develop in ways that are inconsistent with the design practice or intent. This can be more problematic than, for example, in steel or reinforced concrete systems because mechanical responses of components in timber structures may be completely different from the expectations, as illustrated in Figure 2 [3]. The large differences in component response do not affect the system response at serviceability level. However, the system response at ultimate limit state can be completely different than intended during the design causing unexpected failures at unexpected load levels.

The relevance of understanding the real structural response at ultimate limit states become an important issue when designing “extrapolated structures” (large span structures with several load paths, high rise buildings) for which standard experience is not available. Load paths can “exactly” be predicted for statically determinate structures. This is much more complex for statically indeterminate structures, varying loads and system (material) properties. Due to the varying properties, it is even not certain, although pretty certain, that the intended concept of the connection being the governing parameter (capacity based design), shown in Figure 3, is achieved.

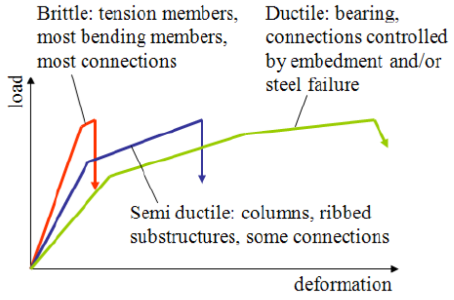


Figure 2: Typical range of responses for components in structural timber systems [3]

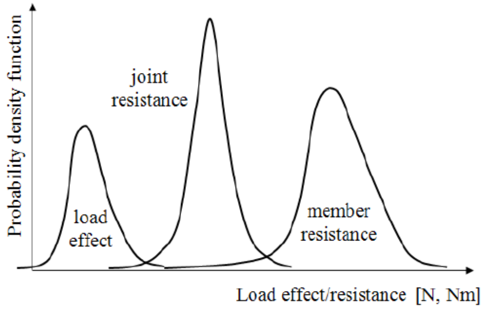


Figure 3: Capacity Based Design

Furthermore, the connections should be regarded as multiple fastener connections in strength, stiffness and ductility analyses and not as a set of single fastener connections. Depending on the connection geometry, the failure mode may change from ductile to brittle as shown in Figure 4 for dowel-type fasteners loaded perpendicular to the grain.

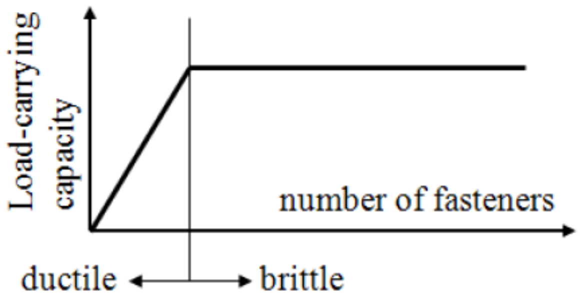


Figure 4: Changing failure mode depending on the connection geometry and on the number of fasteners for dowel-type fasteners loaded perpendicular to the grain.

Something similar happens for dowelled connections loaded parallel to the grain, where the failure mode may change from ductile for single dowelled connections to brittle for multiple dowelled connections due to stress perpendicular to the grain accumulation and/or row shear failure. In Eurocode 5, EN 1995-1-1 [4], this is taken into account by introducing an effective number of fasteners (n_{ef}).

Conclusions

The purpose of this paper on ductility is to provide acceptable input for probabilistic robustness analyses. Therefore, the results must be consistent: the ductility parameter of obviously brittle connections (with low deformation capacity) should differ significantly from the ductility parameter of connections showing large deformations. It can be concluded that this is not the case for most ductility definitions. However, this conclusion might be premature since Tables 1 and 2 only refer to tests on single bolted connections loaded parallel to the grain. Many more test results should be analysed with respect to the ductility definitions before final conclusions can be drawn.

It is also pointed out that absolute ductility definitions rather than relative definitions such as the ductility ratio should be used in timber structures and connections.

Relevance for future code development

It should be noted that quantities related to deformation capacity are needed when, for example, plastic design is carried out, as well as for seismic design. Therefore it seems important, for future development in timber engineering, to provide criteria for classification of timber connections depending on their stiffness (rigid, semi-rigid, and pinned), strength (full- and partial-strength) and ductility (ductile and brittle).

A possible parameter to classify connections is the ratio between connection strength and strength of the connected member, as suggested in Eurocode 3 Part 1-8 [20] for steel connections. In fact, ductility can be fully exploited only if the connection is partial-strength, i.e. it can transmit a force (e.g. bending moment) smaller than the strength of the members connected. If this condition is not satisfied, then failure will occur on the member side rather than in the connection and will be inherently brittle. It should be also noted that timber connections are usually regarded as pinned connections in design. However, many are semi-rigid. Only a few connection types, e.g. the connections realised with expanded tubes, are rigid (and ductile).

3.14 TAPERED AND CURVED GLULAM MEMBERS

ESSAY 3.10 H J Larsen Tapered and curved members

Tapered beams

Figures 1 and 2 show two types of tapered beams commonly used in practice, namely single and double tapered beams.

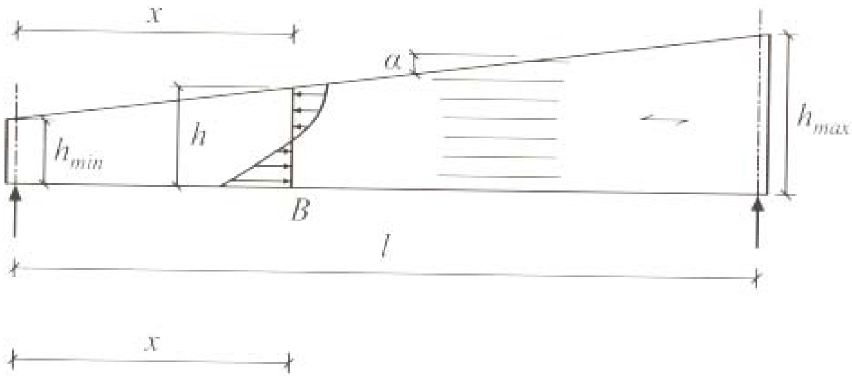


Figure 1. Single tapered beam.

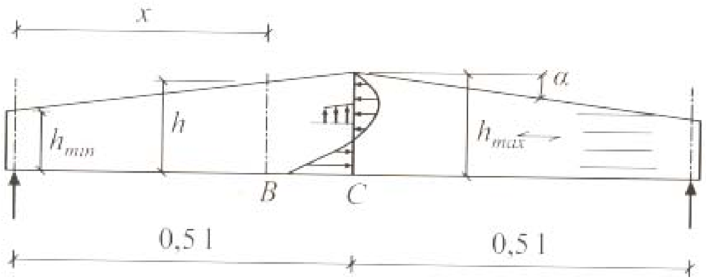


Figure 2. Double tapered beam

The stress distributions in tapered beams differ significantly from those of beams with constant depth. A bending moment results not only in stresses σ_0 in the direction of the beam axis but also in normal stresses σ_{90} perpendicular to this direction and shear stresses. The reason is that the stresses at the top shall be parallel to the surface. At the apex where the stress shall be parallel to both surface, the result is zero normal stresses and stresses – for downward load tensile stresses – perpendicular to grain in a zone under the apex.

The stress distribution is derived in **Paper 11-10-1.** The stresses shall satisfy the normal failure condition according to Hankinsson.

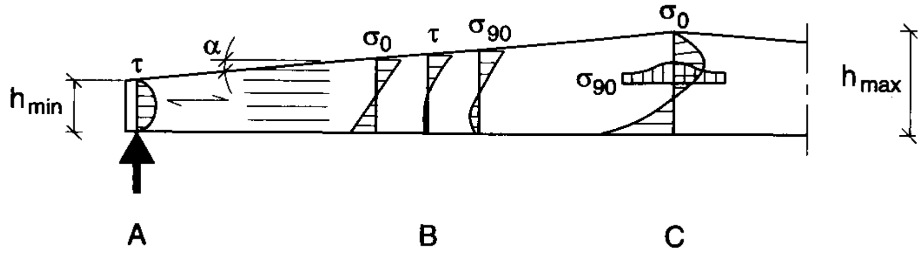


Figure 3. Stresses in a tapered beam

In the final version of Eurocode 5, the requirements are simplified. The normal stresses are calculated as for a beam with constant depth and they shall fulfil the following empirical expressions:

$$\frac{\sigma_m}{k_{m,\alpha} f_m} \leq 1 \tag{1}$$

where σ_m is the bending stress:

$$\sigma_m = \frac{6M}{bh^2} \tag{2}$$

For tensile stresses parallel to the tapered side:

$$k_{m,\alpha,t} = \frac{1}{\sqrt{1 + \left(\frac{f_m}{0.75 f_v} \tan\alpha \right)^2 + \left(\frac{f_m}{f_{t,90}} \tan\alpha \right)^2}} \tag{3}$$

For compressive stresses parallel to the tapered side:

$$k_{m,\alpha,c} = \frac{1}{\sqrt{1 + \left(\frac{f_m}{1.5 f_v} \tan\alpha \right)^2 + \left(\frac{f_m}{f_{c,90}} \tan\alpha \right)^2}} \tag{4}$$

The reduction factors $k_{m,\alpha}$ for compression and tension are shown in Figure 4 for glulam GL32h and GL24c. The reduction factors for other grades of glulam will fall between the two grades.

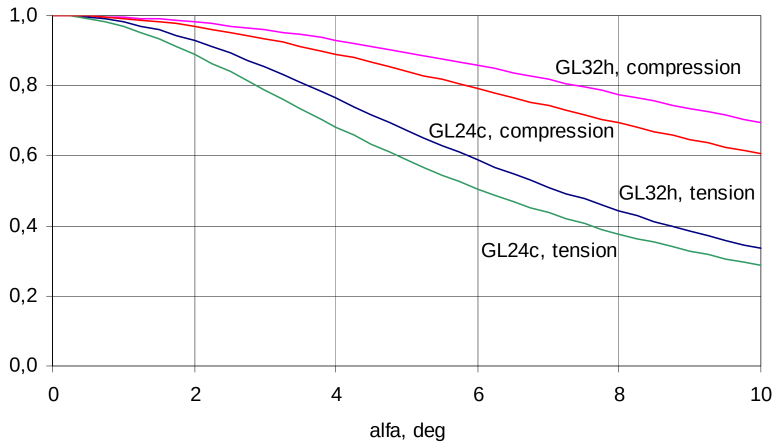


Figure 4. Reduction factors for glulam.

Curved beams

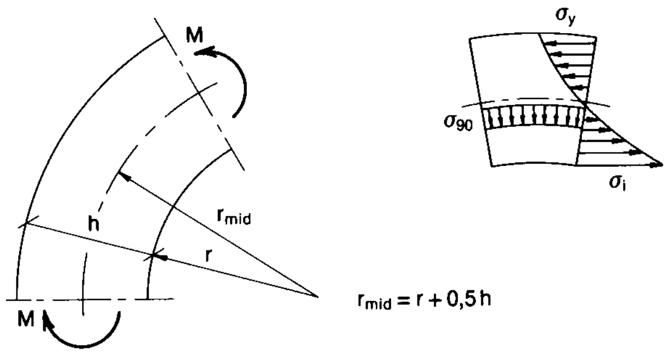


Figure 5. Stress variation in a plane curved beam with constant bending moment.

Loading a curved beam in bending will result in stresses both parallel and perpendicular to the beam, see Figure 5. The normal stresses in the convex side of the beam are smaller than the stresses at the concave side and the stresses at the concave side are larger than the stresses in a corresponding straight beam. The reason is that even if the deformations vary linearly, the strains will not because of the varying fibre lengths. This effect is disregarded in Eurocode 5, i.e. the stress is for a rectangular cross-section calculated as for a straight beam

$$\sigma_{ou} = \sigma_{in} = 6M/bh^2 \tag{5}$$

The bending stresses σ_m developed during the fabrication when the laminations with thickness t are formed to a curvature $1/r$ are theoretically rather high. In the outermost fibres

$$\sigma_m = Et/(2r) \tag{6}$$

These internal stresses reduce the load-bearing capacity of the cross-section.
For an elastic modulus $E = 12000$ N/mm^2, a lamella thickness $t = 33$ mm and a radius $r = 5000$ mm the bending stress becomes $\sigma_m = 40$ N/mm^2, i.e. corresponding to the characteristic strength. Experimental results show, however, that the built-in stresses become significantly smaller, probably due to creep that occur during the hardening process where moisture from the adhesive is added. In the ENV version of Eurocode it was that the strength values for bending, tension and compression for $r/t < 240$ should be reduced by the factor:

$$k_{curve} = 0,76 + 0,001\frac{r}{t} \ (\le 1) \tag{7}$$

In the final version this effect is disregarded.

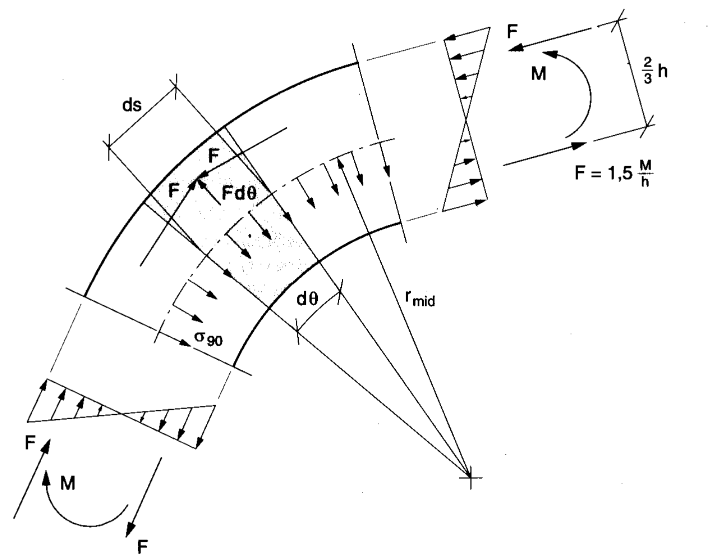


Figure 6. Internal forces and tension stresses perpendicular to the grain direction in a curved beam.

The bending moment results also in stresses perpendicular to grain. The following simplified derivation of the transversal stresses illustrates this effect. It is assumed that the normal stresses vary linearly over the beam depth, see Figure 6, i.e. the influence of the non-linear stress distribution is disregarded. The force resultant F on one half of the cross-section is $F = 1,5\,M/h$. Equilibrium of the marked element loaded by F on both cross-sections and the stress σ_{90} requires

$$F d\theta = \sigma_{90} b r_{mid} d\theta$$

$$\sigma_{90} = \frac{F}{b r_{mid}} = 1,5 \frac{M}{b h r_{mid}} \tag{8}$$

where b is the thickness (width) of the beam.

When the moment distribution tends to reduce the curvature, as is the case in Figure 8, the stresses perpendicular to the grain are tensile stresses and it is necessary to take into account that the strength perpendicular to grain depends on the stressed volume by multiplying the tensile strength perpendicular to grain by:

$$\frac{\sigma_{t,90,d}}{f_{t,90,d}} \le k_{dis} \left(\frac{V_{ref}}{V} \right)^{0.2} \tag{9}$$

where V is the stressed volume and V_{ref} is a reference volume. For glulam, $V_{ref} = 0,01$ m^3. The factor k_{dis} takes into account the stress variation over the depth. For a parabolic variation from zero at the surface to a maximum value in the middle $k_{dis} = 1,4$.

Pitched cambered beams

The stresses in the "triangle" correspond in principal to those in a curved beam – the axial stresses do not vary linearly and the moment also induces stresses perpendicular to grain – but these effects are much more pronounced especially near the apex where the normal stresses are zero because of the apex point. It is, therefore, not unusual to replace the construction by a curved beam with a separate "triangle".

The maximum normal stress is found in the bottom side of the apex section and should be calculated as

$$\sigma_{m,i} = k_0 \frac{6 M_{ap}}{b h_{ap}^2} \tag{10}$$

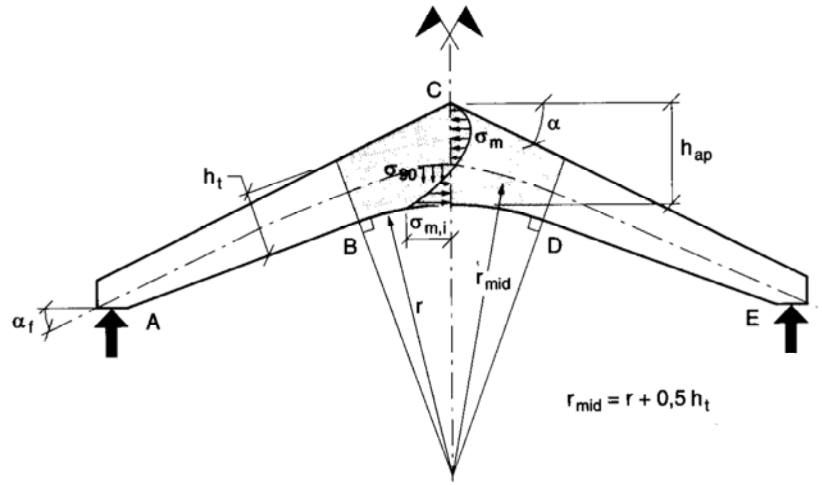


Figure 7. Pitched beam consisting in principle of two tapered beams joined by a "triangle" with curved underside.

The maximum tensile stress perpendicular to the grain direction is found just under centre line in the apex section and should be calculated as

$$\sigma_{t,90,\max} = k_{90} \frac{6 M_{ap}}{b h_{ap}^2} - 0,6 \frac{p}{b} \tag{11}$$

M_{ap} is the moment in the apex section where the beam depth is h_{ap}. The factors k_0 and k_{90} are:

$$k_0 = k_1 + k_2 \left(\frac{h_{ap}}{r_{mid}} \right) + k_3 \left(\frac{h_{ap}}{r_{mid}} \right)^2 + k_4 \left(\frac{h_{ap}}{r_{mid}} \right)^3 \tag{12}$$

$$k_{90} = k_5 + k_6 \left(\frac{h_{ap}}{r_{mid}} \right) + k_7 \left(\frac{h_{ap}}{r_{mid}} \right)^2 \tag{13}$$

with

$$\begin{aligned} k_1 &= 1 + 1,4 \tan\alpha + 5,4 \tan^2\alpha \\ k_2 &= 0,35 - 8 \tan\alpha \\ k_3 &= 0,6 + 8,3 \tan\alpha - 7,8 \tan^2\alpha \\ k_4 &= 6 \tan^2\alpha \\ k_5 &= 0,2 \tan\alpha \\ k_6 &= 0,25 - 1,5 \tan\alpha + 2,6 \tan^2\alpha \\ k_7 &= 2,1 \tan\alpha - 4 \tan^2\alpha \end{aligned} \tag{14}$$

For a curved beam with $\alpha = 0 : k_5 = 0, k_6 = 0,25$ and $k_7 = 0$, and in accordance with equation 4.4 the transversal stress becomes

$\sigma_{t,90} = 0,25 \cdot 6M/(r_{mid}bh)$.

where:
p is the uniformly distributed load acting on the top of the beam over the apex area;
b is the width of the beam;

The derivation of these expressions is given in Paper 14-12-1.

The term with p in (11) is questioned by some member countries and it is optional in the National Application Document to permit it or not. Although it is small it increases the load-carrying capacity considerably because $f_{t,90}$ is small.

11-10-1 H Riberholt
Tapered timber beams

Summary
There is proposed a method to calculate the stress distribution in tapered wood beams. The wood is assumed to be linear-elastic orthotropic.

The Norris interaction formula has been employed to compare measured and predicted bending strengths of glulam and solid clear timber beams.

There was found a good agreement between measured and predicted bending strength of glulam beams.

Conclusions
It has been demonstrated that the load capacity of tapered glulam beams can be predicted by a linear-elastic orthotropic stress analysis combined with a rupture criterion based on the maximum stresses at a point. It was found, that the Norris interaction formula gave the best prediction of the strength of glulam.

Apparently it is not possible to predict the strength of tapered solid beams on the same basis.
The test results with tapered glulam beams indicate that the Norris formula is not satisfactorily, since the basic strength parameters apparently are mutual dependent. It seems that "the shear strength is greater, when the stress perpendicular to the fibres is a compression stress than when it is a tension stress". This effect might be explained by assuming that rupture is triggered by cracks or flaws in the wood.

14-12-3 H Riberholt
Double tapered curved glulam beams

Preface
The stress analysis of double tapered curved glulam beams can not be carried out by the usual beam formulas, instead more refined methods must be applied. Among the feasible methods one can mention partly the Point Matching Technique (a collocation method) and partly the Finite Element Method.

Some authors have employed the Point Matching Technique and others have employed the Finite Element Method. Since objections may be arisen against the assumptions employed by the above mentioned authors, it is found reasonable to accomplish a stress analysis which allows for these objections. This stress analysis is carried out be means of the Finite Element Method, which is found most suitable due to its generality.

In Chapter 2 there is given a review of the differences between the methods, and in Chapter 3 the results are compared with measurements. In Chapter 4 a method for practical stress analysis is proposed.

This report aims at a stress analysis of a double tapered curved glulam beam simple supported and loaded with an uniformly distributed load.

14-12-4 E Gehri
Comment on CIB-W 18/14-12-3

Dear Mr. Sunley,
Unfortunately I will not be able to attend the CIB-meeting in Warsaw due to military service duties in my country.

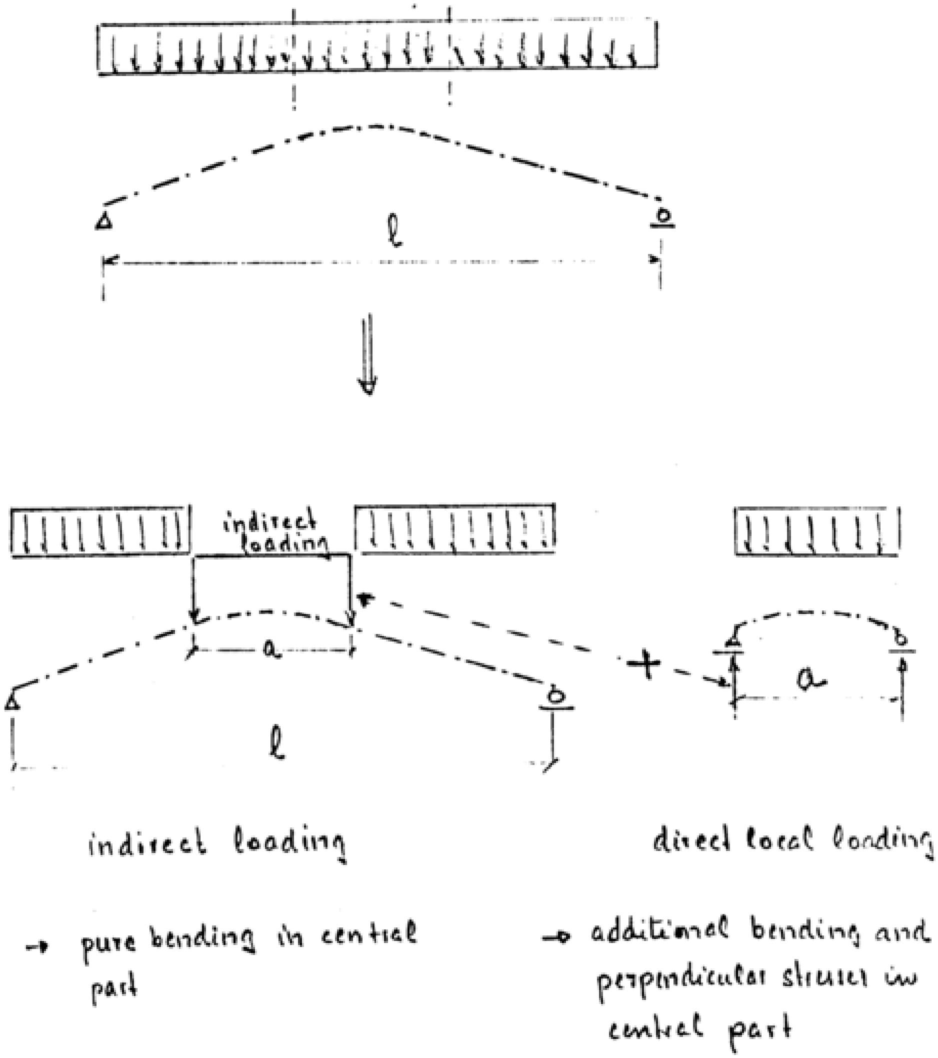
I just received the paper of Riberholt on "Double tapered curved glulam beams". This work confirms the validity of the results of Foschi and Blumer. The difference shown between PPM and FEM was greater for the radial stress, but always smaller than 10 %. Riberholt's paper shows although the influence of a uniformly distributed load of the top of the beam. For bending stresses the influence is negligible for the whole practical range of *L/H*. For the perpendicular stresses a significant decrease was found for smaller *L/H*-ratios.

The influence of a direct applied load can although be found by a simple static consideration, as explained in the sheet annexed.

The additional bending stresses due to direct local loading are very small, since they are depending on the factor $(a/k)^2$. The additional perpendicular stresses, which are proportional to the applied load, have larger influence. Assuming a linear distribution over the height of the girder, we can therefore assume that the perpendicular stress due to pure bending will be diminished by the half load direct applied. As has shown by Riberholt, for smaller *L/H*-ratios which would correspond to larger *a/z*-values there is an interest to consider the effect of direct loading.

Riberholt pointed newly out the advantages and disadvantages of presenting the maximum stresses in function of the height of the apex or of a fictive curved beam with constant height. We know from experience that in case of radial failure mode an apex means a weakening of the girder. From the engineering point of view we should therefore avoid all kinds of curved beams with apex (or apex only nailed on girder) and therefore consider the curved beam as the basic case. In my opinion we should delete the case with apex from the timber code (too big weight on this point) and make only references to corresponding publications.

With best regards,
Yours very truly, p. p. E. Gehri



24-12-2 J Ehlbeck, J Kurth
Influence of perpendicular-to-grain stressed volume on the load-carrying capacity of curved and tapered glulam beams

Introduction

According to Eurocode 5 -Draft (1987) the design of curved beams with constant depth and of double tapered beams has to take into account a volume factor k_{vol}., and a stress distribution factor k_{dis} to satisfy the design conditions covering the perpendicular-to-grain tensile stresses:

$$\sigma_{t,90,d} \le k_{vol} k_{dis} f_{t,90,d} \tag{1}$$

In (1) is:

$$k_{vol} \left(\frac{V_0}{V} \right)^{1/k_{wei}} \tag{2}$$

assuming a 2-parameter Weibull distribution,

$$k_{dis} = \frac{\sigma_{max}}{\left(\frac{1}{V} \int_V \sigma\left(x, y, z\right)^{k_{wei}} dV \right)^{1/k_{wei}}} \tag{3}$$

taking into account the stress distribution, and

$$f_{t,90,d} = k_{\mathrm{mod}} f_{t,90,k} / \gamma_M \tag{4}$$

This design method is based on the 2-parametric statistical distribution function for homogeneous and isotropic material with brittle fracture behaviour developed by Weibull 1939. It takes into account the influence on the perpendicular-to-grain tensile strength

– of the stressed volume by k_{vol},
– of the stress distribution by k_{dis}.

Proposals for design rules in EUROCODE 5

The method given in Eurocode 5 - draft (1987) is principally appropriate to design curved and double-tapered glued laminated timber beams. The advantage of this method is its simple application:

– the maximum perpendicular-to-grain tensile stress is calculated according to the application rules given in EC 5 or other relevant literature,
– k_{vol} is determined according to the application rules given in Eurocode 5 - draft (1987),

– k_{dis} is - for simplification - for all types of curved, double-tapered and cambered beams for actions giving a constant or nearly constant moment in the curved part of the beam taken as k_{dis} = 1,4.

Using these rules, the product of $k_{vol}\,k_{dis}$ leads to a realistic modification of the characteristic design strength.

Furthermore, it should be discussed to modify or change the characteristic strength values for tension perpendicular-to-grain, given in prEN 338, e.g. from 0,40 N/mm^2 to 0,55 N/mm^2 when timber is used for glulam. In case of changing the reference volume V_0 from 0,02 m^3 to a smaller volume (under discussion with respect to adequate test methods), the characteristic strength values should be modified accordingly.

3.15 THIN-WALLED ELEMENTS

11-4-1 I Smith
Analysis of plywood stressed skin panels with rigid or semi-rigid connections

Introduction

Plywood stressed skin panels consist of plywood sheets glued or mechanically fastened to the top or to both top and bottom surfaces of longitudinal timber stringers. The whole assembly acts as an integral section to resist bending, provided that the joints between the plywood and stringers are sufficiently rigid to prevent excessive slip due to longitudinal shear force induced between the plywood and the stringers. Structural economies are made possible by the utilisation of both plywood and stringers to resist bending. The size of a stringer for a given span can be less than that required for a simple joist, or a lower grade material can be used.

This paper presents a method of analysis for both single and double skin stressed skin panels, with either rigid or semi rigid connections, which is suitable for hand calculation using an electronic calculator. The analysis can be used to solve for panel deflections, and for stresses in the plywood skin(s) and timber stringers. Panels assembled by gluing are assumed to have rigid connections, and panels assembled using nails or staples are assumed to have semi rigid connections.

Using numerical examples it is demonstrated that there is good agreement between the method of analysis presented and alternative methods of analysis formulated by other workers.

Conclusions

For most practical design purposes it is acceptable to approximate the above mentioned loading and displacement definitions for a beam simply supported at both ends which carries a uniformly distributed transverse load, by the corresponding loading and displacement definitions for a beam simply supported at both ends which carries a sinusoidally distributed transverse load.

22-10-3 J König
Thin-walled wood-based flanges in composite beams

Summary

The load-bearing capacity of thin-walled wood-based flanges in composite beams can be reduced due to shear lag, buckling and flange curling. However, flanges in compression have considerable load-bearing capacity even after the flange has buckled. The effective width concept which is well established in the field of sheet metal construction can also be used in designing wood-based board material with respect to buckling. This is shown by an evaluation of tests carried out at Delft. Design formulae for the effect of flange curling are derived, and design criteria are proposed.

Introduction

In the field of timber construction, structural elements comprising thin-walled components of wood-based board material have been used for some time. These structural elements are characteristic for lightweight construction, and it is expected that they will be used more extensively in the future.

Typical examples are composite sections comprising webs of solid wood and flanges of board material. The webs may also be made up of components of board material and solid wood, see figure 1. However, owing to the small thickness of the board material, there are a number of properties which demand more accurate analysis if the material is to be utilised in the optimum manner. The flange often has the duty of distributing load in the transverse direction. In floors, consideration must also be given to the stiffness requirement which determines the thickness of the flange. In these structures it is generally only the shear lag in the board material which limits the load-bearing capacity of the flange when the flanges are very wide in proportion to the span of the beam. In other structures, for instance in roof cassettes, the load distribution task may be of subordinate importance and certain deformations may be tolerated. Finally, direct application of load on the flange can often be avoided, for instance by application of the load over the web of the composite section or on the bottom flange of the section. Flanges in compression may also be subject to buckling. When the beam is curved, or becomes curved owing to the external load, the flanges are deformed inwards towards the neutral axis of the cross section (flange curling), and the contribution of the flanges to the load-bearing capacity and stiffness of the box section is therefore reduced.

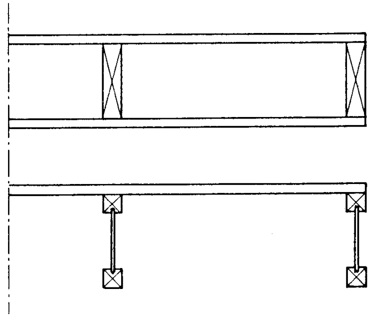


Figure 1. Examples of composite sections with thin walled components.

It is well known that thin-walled plates in compression have considerable post-critical load-bearing capacity, i.e. the load can be further increased after the critical buckling stress of the plate has been reached. This favourable behaviour has been utilised for a long time, first in aeroplane construction and since the end of the 1940s also in building construction, mainly in thin-walled structural elements of steel and aluminium sheeting. In the field of timber construction also the post-critical load-bearing capacity of buckled plate elements has been known. In order that this phenomenon may be utilised to some extent, the coefficient of safety is slightly reduced in the structural regulations for timber in most countries.

In accordance with the CIB Structural Timber Code, it is permissible to utilise the post-critical load-bearing capacity. The code lays down some values regarding the effective width of the flange, but these values are undifferentiated since neither the critical buckling load of the flange nor its compressive strength is taken into consideration. As far as the author is aware, the effect of these is reflected only in the Swiss timber code SIA 164. It is however not known whether the rules in the code had been verified for wood-based board materials when the code was written. In /6/, which gives the background to the Swiss code, no reference is made to this.

The research reports which deal with the buckling of wood-based boards or thin-walled components in structural elements mainly confine them-selves to determination of the critical buckling load. While most structural regulations for timber structures take account of the fact that the load-bearing capacity of thin-walled flanges is limited due to shear lag in the board material, there is no requirement, as far as the author is aware, that the effect of flange curling on the load-bearing capacity of the composite section should be checked.

It is shown that the effective width concept in utilising the post-critical region can also be applied to wood-based thin-walled flanges. Design rules are also given which take the effect of flange curling into consideration. The effect of shear lag in wood-based board materials is already well documented and requires no further elucidation.

3.16 TIMBER BEAMS, GENERAL

5-10-1 H J Larsen
The design of timber beams

Introduction

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

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

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

3.17 TIMBER FRAME WALLS

ESSAY 3.11 H J Larsen Racking resistance of walls

According to Eurocode 5, the racking resistance of a wall shall be determined either by test according to EN 594 or by calculations, employing appropriate analytical methods or design models.

Extract from EN 594: Timber structures - Test methods - Racking strength and stiffness of timber frame wall panels:

1 Scope
This standard specifies the test method to be used in determining the racking strength and stiffness of timber frame wall panels.
The test method is intended, primarily for panels as described, to provide:
- comparative performance values for the materials used in the manufacture of the panels and
- datum information for use in structural design.

……

5 Requirements for test panels
The dimensions of panels shall be given as given in figure 1. The edges of all sheathing materials shall be supported.

6 Test method
6.1 Principle
The test method measures the resistance to racking load of panels which can deform both vertically and horizontally in the plane of the panel.
In this test method, the bottom rail of the panel is bolted to the test rig and uplift is resisted by the sheathing fixings and also by the vertical loads on the top rail of the panel.

6.2 Apparatus
6.2.1 General
The test apparatus shall be as shown in figure 2, and shall he capable of applying, separately, both racking load F, and vertical loads F_v,. The method of application of the loads shall be such that no significant resistance to movement in the panel is induced.

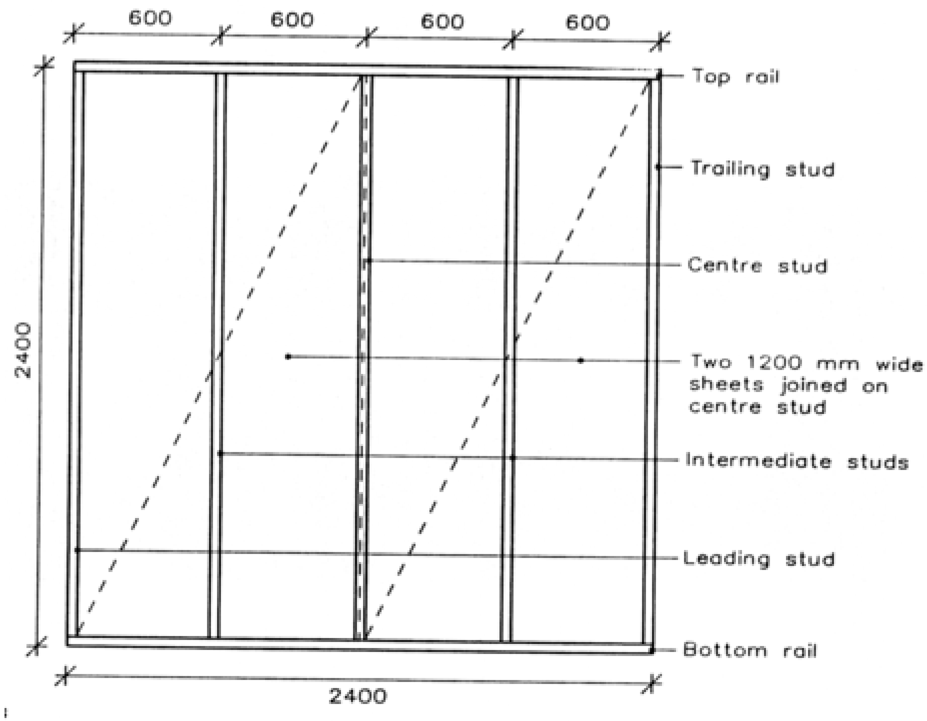


Figure 1. Detalis of test panels (sizes in mm).

……
The apparatus shall be capable of continuously recording the loads F and F_v, with an accuracy of ± 3 % of the applied loads ….

6.2.2 Base and loading frame
The base of the test rig shall provide a level bed to receive the test panel and packer. The base shall be sufficiently stiff so as not to distort during, the test.
……

6.2.3 Mounting of test panel
The panel shall be bolted through a packer to the base of the test rig with holding down bolts positioned as shown in figure 2.
……
The head binder shall be rigidly attached to the top rail of the panel. The cross-sectional dimensions and position shall be such as to provide a

firm interface between the loads and the panel and to allow the free movement of the panel sheathing, during the test.

Experience has shown that the strength and stiffness depends very much on the materials, and details in build-up and load application, and the the test method is, therefore primarily intended, to provide comparative performance values for the materials used in the manufacture of the panels, and it is difficult to see how the results can be applied in practice. In an annex guidelines are given for testing of units other than according to clause 5.

Extract from EN 594:

Annex A. The testing of units of dimensions other than 2,4 x 2,4 m

A.1 General
The purpose of this Annex is to adapt the principle of the test method:
- to other sizes of panel and;
- to combinations of panels and,
- to panels which are partially sheathed and;
- to other panel fixings.

It is intended primarily to provide performance data which may be used for quality assurance or for structural design.

A.2 Requirements for panels
The wall panels tested shall generally correspond to those used in practice as far as the essential structural details and service conditions are concerned.
…..

A.3 Apparatus
The apparatus used …. shall generally be as described in clause 6.

Testing may be relevant where there are many identical walls, but in most cases the load-carrying capacity is determined by calculation.

Eurocode 5 gives two alternative simplified methods of calculation, Method A and B. The member states decides which method should be used. With the exception of Denmark where both methods are accepted, and the UK where method B is compulsory, the member states have chosen Method A.

Method A is a very simple equilibrium method and it is clearly required that the there is a tie-down at their end, that is the vertical member at the end shall be directly connected to the construction below. The anchoring in the foundation may in many modern structures be difficult to make in practice due to heat isolation layers needed due to increased heat isolation requirements.

Method B is based on tests and do not explicitely require that the end stud is anchored but the text on this point is obscure The vertical forces necessary to ensure equilibrium is assumed to be taken by the fixing of the bottom rail to the underlying structure. The fixings shall in addition to the vertical forces prevent the sliding of the bottom rail.

Generally Method B gives lower load-carrying capacity than method A, especially for wall without vertical loads.

In **Paper 38-15-9** a unified method is proposed that gives higher load-carrying capacity than the present Method B without requiring full anchoring.

19-15-3 M Yasumura
Racking resistance of wooden frame walls with various openings

Introduction

Wooden frame construction, practically used for only small dwellings in Japan, can be applied to various uses for a building if a rational design procedure on the basis of structural calculation is developed. Although several theoretical studies have been done on the racking strength of wooden frame wall panels, they are based on the analysis of blind walls and do not explain the mechanical properties of the entire wall panel with openings. A purpose of this study is to present the useful data for calculating the lateral resistance of wooden frame construction. Twelve full-sized plywood-sheathed wall panels with various openings were subjected to the racking loads, and the influence of openings on the shear stiffness and strength was observed. Experimental results were compared with the calculated values on the basis of non-linear load-slip relation of a nail, and comparatively good agreement between experimental and calculated values was obtained.

Conclusions

The results of this study lead to the following concluding statements:

1. The current design procedure is apt to overestimate the racking resistance of a wall panel which consists of only slender blind walls and contains no small walls, and underestimate considerably the racking resistance of wall panels which contains only small openings.
2. Large vertical forces were concentrated at the studs arranged at the end of a wall panel or door openings. These studs should be connected tightly to the foundation and horizontal members when full nail action is expected. Otherwise shear force of the nails which connect a sheathing sheet to the studs subjected to large drawing force should be neglected when the racking resistance of a wall panel is computed.
3. The shear stiffness and strength of a nail fastening estimated from racking test of blind wall panels were approximately 10 to 20 percent smaller than those obtained from single shear test of nail fastenings. The effects of the angle of the fibres of sheathing material and frame members on the shear stiffness and strength of nail fasteners and those of bending deformation of frame members and the slip of the fastenings on the shear properties of wall panels should be studied more precisely to apply the load-slip relation of a nail obtained from single shear test of nail fastenings directly to the computation of the racking resistance of wall panels.
4. The solution developed previously agreed comparatively well with the experimental results. However this method has the tendency to overestimate slightly shear stiffness of wall panels having large openings of door type. This indicates that the effects of bending deformation of frame members should be considered to compute more precisely the shear stiffness of a wall panel having this type of openings. The ultimate load calculated from this solution however agreed well with the experimental results.

25-15-1 U Korin
Structural assessment of timber framed building systems

Scope

The dominant methods for home construction in Israel are based on reinforced concrete structures and block masonry. These methods are locally referred to as "conventional construction".

Socio-economical changes caused in the last few years an increase of the relative volume of low-rise home buildings. Timber framed houses, which dominate the solution for low-rise homes the world over, gradually began to penetrate the home market.

Today, timber homes account for about 10 percent of the low-rise home building (about 25 percent of the total volume of home construction). In terms of last year, we may speak of about 1500 home units of timber framed houses.

Conclusions

The enormous assessment process of building systems enable to provide the country with safe, functional and durable timber framed houses, in spite of the lack of previous experience in that type of construction.
Some of the newly built timber framed houses very successfully withstood the extremely unusual heavy weather conditions the Middle East experienced last winter. The houses were found safe and provided the occupants with good living space.

28-102-2 B Källsner
Racking strength of wall diaphragms – discussion of the Eurocode 5 approach

Abstract
The design of wall diaphragms according to Eurocode 5 is in some respects conservative and in other respects unsafe. To clarity the structural behaviour of wall diaphragms, a theoretical background is given. The design rules in Eurocode 5 are discussed and improvements of the code text are suggested. It is shown that the capacity of shear wall is influenced by different factors like boundary conditions and edge distances of fasteners.

29-15-1 F Lam, 1-1 G L Prion and M He
Lateral resistance of wood based shear walls with oversized sheathing panels

Abstract
This paper summarizes the results from the first phase of a study to develop a database comparing the structural performance of shear wall systems built with regular and 'nonstandard large dimension oriented strand board panels under monotonic and cyclic loading conditions. Different types of nail connectors have also been investigated.

Conclusions
A database comparing the structural performance of shear wall systems built with regular and nonstandard large dimension oriented strand board panels under monotonic and cyclic loading conditions was reported. Under monotonic loading, a substantial increase in both stiffness and lateral load carrying capacity was observed when comparing shear walls built with oversize and regular panels. Defining ductility as the ratio between wall displacements at maximum load and 50% of the maximum load during monotonic testing, a slight increase in ductility was observed comparing the shear walls built with oversize and regular panels. Examining the dissipated energy as the area under the load deformation curve, walls built with the regular panel dissipated more energy under cyclic loading as compared to walls built with oversized panels. The failure modes in the shear walls were substantially different under monotonic (nail withdrawal) and cyclic (nail fatigue) test conditions. Further investigation of the adopted cyclic load schedule in relation to the dynamic response of the shear wall is needed to fully understand and characterize the failure modes in shear walls.

29-15-3 D R Griffiths, C J Mettem, V Enjily, P J Steer
The racking resistance of timber frame walls: design by test and calculation

Abstract
This paper introduces work carried out over many years in the UK, on timber frame wall design. It explains why the work was necessary for the national acceptance of timber frame, and demonstrates how knowledge gained is now helping in considering designs for medium-rise timber frame buildings of up to six storeys. Alternative design and test methods are considered, and a justification is given for a top loading test, which leads to an empirically-based design method. This paper is written in preparation for a UK initiative to find a suitable design method for racking resistance to limit-state design principles. The importance of correctly modelling the timber frame behaviour under load is noted, and short design examples are included.

Summary and Conclusions
- The principal wall racking test and design method used in the UK was developed to meet a need for the structural appraisal of platform frame housing systems, which when the methods were instigated, represented a new form of domestic building construction. The limitation of deflection was significant to the design objectives. A top-loading test was accepted, after other methods had been seen to model inadequately the complex situation of the real timber frame wall.
- A detailed reduction procedure for the test results linked the test basis to a design method. This entailed the derivation of a single design value, termed the "basic racking resistance" R_b (kN/m). The racking resistance was used with the total wall length, and with modification factors for materials and wall form, to evaluate the safe racking capacity of any practical wall design desired.
- The design method was empirically based. However it was by no means ad hoc. Indeed, it was related to the test performance of nearly two hundred wall and panel units. It realistically modelled the timber frame wall, and its practical restraints, but did not take full account of

holding-down methods or of return walls, although it has been shown in research projects that these factors can in fact be modelled.

– The UK design approach has rules which allow for the combined use of structural sheathings, and semi-structural linings. These rules preclude the use of a fundamentally-based three dimensional design method, since the high stiffness and strength of internal semi-structurally lined walls must often be ignored.
– Although conceived for working stress design, the UK method could, and probably will be, adapted to a limit-state approach. This will need to look separately at serviceability and ultimate limit states. This will in turn require a comprehensive re-evaluation of the UK test data. There is a realisation that at present, design values are very much limited by deflection considerations.
– The new EN 594 test reduces the significance of stiffness in design, but at the same time is fundamentally flawed in its evaluation of this parameter. Whilst appealing in their design simplicity, the ASTM-based design methods use an unrealistic test model, and consider only wall strength. The EC5 design method concentrates on external forces, and seems not to consider the racking resistance of the wall itself. Certainly, it does not cover serviceability design, and is restrictive in its requirements for panel fixings.
– It is becoming increasingly important that both timber frame research, and the ensuing design methods are acceptable throughout Europe. In order to take full advantage of the lead shown by the UK timber frame industry, a "Eurodiaphragms" project has just been initiated. This has the objective of preparing a wall racking design method which is comprehensive in its coverage, yet simple. Most importantly, it seeks to establish a basis which will be acceptable throughout Europe. This method is to be prepared within the timetable for the revision of Eurocode 5.
– Completed research on the brickwork outer skin in the UK, both as a contributor to racking resistance, and as a wind shield, could with great benefit be transferred to a limit-state design approach. This would be a fairly substantial undertaking. However, the common between-materials safety format of the Eurocodes would help considerably in such an endeavour. Developments in medium-rise timber frame construction are stimulating a desire to investigate such masonry/timber composite action effects for taller buildings.

30-15-1 Ming He, H Magnusson, F Lam, H G L Prion
Cyclic performance of perforated wood shear walls with oversize oriented strand board panels

Abstract

This paper reports the test results obtained from a study to investigate the influence of openings on the lateral resistance of wood based shear walls built with both standard and oversize oriented strand board panels under monotonic and cyclic loading conditions. Eight walls were tested, using either standard 1.2 x 2.4 m panels or one full size 2.4 x 7.3 m oriented strand board panel. Commonly used wood frame construction details were employed, using 38 x 89 mm No. 2 and better Spruce Pine Fir framing components. Test results showed that the application of non-standard oversize panels significantly improved the performance of the perforated shear walls compared with standard 1.2 x 2.4 m panels. Door and window openings caused a significant decrease in the strength and stiffness of the walls and precipitated a change in failure mode, especially for walls with full size panels. While nail failure modes were commonly observed in walls without openings, a combination of nail and panel failures were observed in shear walls with openings. Final failures typically affected the wall performance once the plastic deformation region was reached, which was displayed in the tests in the form of either panel buckling or panel tearing at the corners of door and window openings. A newly proposed cyclic test protocol was used which consisted of fewer but more severe displacement excursions, compared to many other test protocols. This was believed to better reflect typical earthquake excitation and avoid low cycle nail fatigue failures, which were observed previously with long cyclic test protocols.

Conclusions

Openings in wood based shear walls caused a significant decrease in shear strength and stiffness of shear walls, due to the reduction in the effective sheathing area. To strengthen the racking resistance of shear walls with openings, large non-standard panels were used. When shear walls with openings were built with these oversize panels, they presented a significant improvement in shear wall performance, obtaining shear strength and stiffness values even higher than for walls without openings built with regular panels. These results imply that the impact of openings on shear walls with oversize panel is more significant in contrast with the corresponding values in shear walls with regular panel. In addition, the ductility ratios of the walls with openings were slightly lower than those without openings.

The failure modes of shear walls with openings differed from those without openings. The performance of the walls was not only governed by the behaviour of the nail connectors between panel and frame, but a combination of nail and panel failure. Panel failure modes occurred in the form of panel buckling and panel tearing around the corners of openings. Nail withdrawal was the dominant nail failure mode in the tests (except for one wall, in which nail low cycle fatigue occurred), which occurred mainly along the edges of the panels at the mid-height of the wall with regular panels and along the bottom edge of the panel in the wall with one oversize panel. Both panel and nail failure modes and locations were the same under monotonic and cyclic loading conditions. Considering the fact that panel failure locations were mainly concentrated in the corner areas of openings, localized reinforcement methods should be further investigated.

30-15-2 L Davenne, L Daudeville, N Kawai, M Yasumura
A numerical analysis of shear walls structural performances

Abstract
This paper deals with the numerical analysis of wood based shear walls made of lumber framing members and with sheathing plywood panels connected with nails. A model is presented for the simulation of the non linear response of the nailed connection under static monotonic or reversed cyclic loading. It has been implemented in the finite element code CASTEM 2000. Three kind of walls with different openings are simulated. The responses are close from the experimental results made at the Building Research Institute in Tsukuba, Japan.

Conclusion
The authors presented a finite element code to compute the response of wood frames with nailed plywood sheathing panels. A model to simulate the non linear response of joints under cyclic loading is proposed. It gives a good response in the case of static cyclic imposed displacements (CEN standard). The different strength degradations, and allowance apparitions are modeled. The response of the walls under static m loading is close to the results from the tests. The cyclic loading of walls is in progress.

The one dimensional model must now be checked in other loading cases and with other types of nail (for example, nail fatigue may appear with other kind of nails, the behaviour different than the Japanese nails tested here).

In a future work, a three dimension model with degradation couplings must be developed to better simulate the reality.

30-15-4 M Yasumura, N Kawai
Evaluation of wood framed shear walls subjected to lateral load

Abstract
The purpose of this study is to establish the test method for evaluating the seismic performance of wood-framed shear walls. The quasi-static monotonic loading and the reversed cyclic loading were applied to the nailed joints and the wood-framed shear walls sheathed with the plywood, OSB and gypsum board. The yield load, ultimate load and the maximum displacement were determined by the several procedures, and the energy dissipation of the nailed joints and the shear walls is discussed.

Conclusion
The yield load of the shear walls determined by the proposed procedure in this gives appropriate values both for the true and apparent shear deformations, while the load determined by CEN standard procedure gives 20 to 40% higher value for the a shear deformation than that for the true shear deformation. The yield load determined by method agreed well with the yield strength calculated from the experimental results of nailed joints and that with the yield theory. It also agreed well with the load corresponding to the true shear deformation of 1/300.

The ultimate load and the maximum displacement under the reversed cyclic loading was respectively 10 to 15% and 10 to 30% smaller than those of the monotonic loading. The calculated ultimate load from that of the nailed joint under the reversed cyclic load test agreed well with the experimental results.

The equivalent viscous damping of the nailed joints varied from 15 to 25% when the displacement was smaller than 1 mm and decreased to 15% in average when the displacement was larger than 5 mm in the plywood and OSB sheathed walls. This trend was not clear in the gypsum sheathed nailed joints. The equivalent viscous damping ratio of the shear walls was almost constant regardless of the displacement within the range of 10 to 20%.

The sum of the total energy dissipated in the nailed joints agreed well with the total energy dissipation of a shear wall under the monotonic loading. Sum of the total energy dissipated in the nailed joints was 30 to 90% larger than the total energy dissipation of a shear wall under the reversed cyclic loading.

32-15-3 C Ni, E Karacabeyli, A Ceccotti
Design methods for shear walls with openings

Abstract
Nailed shear wall systems provide the lateral load resistance for most wood-frame buildings. When designing shear walls containing openings, hold-down connections are normally required at the ends of each wall segment between openings. The shear wall containing openings is then designed as multiple shear wall segments. The design capacity of such a shear wall is generally determined by summing the capacities of all the shear wall segments. For applications where shear walls do not have hold down connections at the boundary of the openings, a method for determining the capacity is needed. The empirical "perforated shear wall" approach developed in Japan has been shown by number of researchers to predict conservatively the capacity of such shear walls. In this study, an alternative design method for the design of shear walls with openings is developed, and verified against available test data obtained on full-size shear walls.

Conclusion
A method for the design of shear walls with openings is presented. The predicted capacities from this method are found to be in reasonable agreement with the test results. A simplified version of the method is proposed for code implementation.

32-15-4 K Komatsu, K H Hwang, Y Itou
Static cyclic lateral loading tests on nailed plywood shear walls

Abstract
Simplified calculation method for predicting shear deformation of the model shear resistance system composed of semi-rigid jointed glulam portal frame with nailed plywood shear wall panel is proposed in this report.

Conclusions
Simplified calculation method for predicting shear deformation of the model shear resistance system in which semi-rigid jointed glulam portal frame with nailed plywood shear wall panel was proposed in this report. Load-slip relationship of nail joint, moment-rotation relationship of column leg joints and beam-column joints were all expressed in the form of 3-parameters exponential function to make nonlinear calculation possible. Coincident between observed behavior with calculated ones were good with a few exceptions. As a whole, proposed calculation method was thought to be usable for practical design purposes.

33-15-1 C Ni, E Karacabeyli, A Ceccotti
Lateral load capacities of horizontally sheathed unblocked shear walls

Abstract
Although design capacities for unblocked diaphragms are available in many design codes, capacities for unblocked shear walls are not. None the less, in applications such as exterior wall sheathing, the wood-based panels are, at times applied horizontally to the wall frame without blocking. There is a need, identified by the engineering community, for the determination of lateral load carrying capacities of unblocked shear walls. Full-scale tests were performed on unblocked shear walls. The results showed that typical failures occur along the unblocked horizontal joint where the nails either withdraw from the framing members or pull through the panel. Based on the test results, strength adjustment factors for unblocked shear walls are proposed. It was found that the factored design shear strengths for unblocked shear walls were reasonably conservative when compared to test results obtained in this study and also to the test results available in the literature.

Conclusion
Three types of unblocked shear walls were tested under ramp and reversed cyclic tests. A strength adjustment factor for unblocked shear walls is proposed to correlate specified shear strength of an unblocked shear wall to the specified shear strength of blocked shear wall of the same panel grade and thickness with 150 mm perimeter spacing. The strength adjustment factor is a function of the framing spacing and nail spacing at supported edges and at intermediate studs. Comparisons of test resin and factored

design shear strengths show that the factored design shear strengths for unblocked shear walls are reasonably conservative.

34-15-1 B Källsner, U A Girhammar, L Wu
A simplified plastic model for design of partially anchored wood-framed shear walls

Abstract
This is an introductory paper in which the structural behaviour and design of partially anchored wood-framed shear walls are studied.

A simplified plastic model is proposed for design in the ultimate limit state. The model covers only static loads and can only be applied when mechanical fasteners with plastic characteristics are used.

A few tests of shear walls have been conducted in which the influence of vertical loads and anchorage of bottom rail was investigated. A comparison between measured and calculated anchorage capacities indicates that the proposed model is suitable for design purposes.

Conclusion
The basic principles of a simplified plastic model for design of incompletely anchored wood framed shear walls have been presented. The model can be used if the sheet material is fixed by mechanical fasteners and if these sheathing-to-timber joints show plastic behaviour. The model can only be applied on static loads in the ultimate limit state.

Some introductory tests of shear walls have been conducted where the bottom rail was completely fixed to the substrate or free with respect to vertical displacements. Some of the tests included vertical loads The test results indicate that there is a reasonable agreement between measured and calculated load-carrying capacities.

The model explains the structural behaviour of incompletely anchored wood-framed shear walls and is suitable for calculations by hand. The model can be used to analyse the influence of 3-dimensional load transfer within a building.

34-15-2 S Nakajima
The effect of the moisture content on the performance of the shear walls

Abstract
Plywood sheathed or OSB sheathed shear walls were constructed by the 2x4-construction system and tested in two environments, 20°C, 65% R.H. and 20°C, 90% R.H. to clarify the effect of the moisture contents on the racking strength and stiffness of the shear walls. The nail joints of the shear walls and the sheathing materials were also tested in the two environments to clarify the effect of the moisture contents on the strength and deformation properties of the nail joints and the panel shear properties of the sheathing materials. The reduction of the yield strength of the plywood sheathed shear walls after being conditioned in the atmosphere of 20°C and 90% R.H. was 15% and that of the OSB sheathed shear walls was 10%. The test results suggested that the durability of the shear walls against water might not be adequately evaluated by the current evaluation method.

The yield strength and the initial stiffness of the shear walls were estimated from the strength and stiffness properties of the nail joints and the sheathing materials. The results of the calculation suggested that the simplified calculation method described in the 2x4 designers' manual does not sufficiently predict the effect of the moisture contents on the performance of the shear walls.

Conclusion
The reduction of the yield strength, ultimate strength and initial stiffness of the plywood sheathed shear walls after being conditioned in the atmosphere of 20°C and 90% R.H. was 15%, 13% and 21%. The reduction of the yield strength and ultimate of the OSB sheathed shear walls after being conditioned in the atmosphere of 20°C and 90% R.H. was 10% and 9%. And the initial stiffness of the OSB sheathed shear walls increase 5% after conditioned in the atmosphere of 20°C and 90% R.H. The current modification factor for the water durability issues of the shear walls should be reviewed as 1.00 for plywood seems to be too high and 0.85 for OSB seems to be too conservative.

The reduction of the panel shear modulus of rigidity due to the wet condition was 24% for plywood and 43% for OSB and the reduction of the panel shear strength was 24% for plywood and 23% for OSB. Both sheathing materials particularly OSB becomes more ductile when it is in a wet condition.

The yield shear strengths of the plywood nail joints and the OSB nailed joints in the wet condition were almost 10% higher than the yield shear strengths measured in the normal condition. Thickness swelling of the materials or the physical changes of the materials is considered to be the reason for this but the conclusion should be based on further research works.

The reduction of the initial stiffness of the shear walls due to the wet condition could be predicted by the simplified calculation method but the reduction ratio was no well estimated. The reduction of the yield strength due to the wet condition could not be predicted by the simplified calculation method. The simplified calculation method described in the 2x4 designers' manual seems not to be a good predictor to evaluate the effect of the moisture contents on the shear walls.

35-15-1 B Källsner, U A Girhammar, L Wu
On test methods for determining racking strength and stiffness of wood-framed shear walls

Abstract

It is proposed that the present standard EN 594 is revised by using, as the main alternative, a test method that evaluates the strength and stiffness of the shear wall in a pure shearing mode. This mode will render boundary and loading conditions corresponding to a fully anchored shear wall. This pure shear mode is introduced in a rational way in the wall by applying a diagonal tensile load at the top corner of the wall. This method will render basic test results that have general applicability as far as the actual sheathing and fastener materials are concerned. Different materials can then easily be compared. The necessary adjustment of the design value for shear walls applied in a specific construction with certain boundary and loading conditions is then proposed to be made either by supplementary testing or by using theoretical reduction factors that depend on these conditions and that can be obtained by analytical models for partially anchored shear walls.

The mechanical properties of the sheathing-to-timber joints have a major influence on the stiffness and strength of shear walls. Therefore, the racking test standard would need to be supplemented by a relevant method for testing of the joint characteristics. The present standard for load bearing nailed joints is not appropriate. A revised test standard is needed that incorporate testing of the parameters of special interest, such as edge distance of the fasteners and loading direction vis-a-vis the edge of the sheet. By such a test standard for joints, design values for shear walls can be evaluated using general calculation models and the results can be compared to full-scale tests.

By realising this proposal we open up for use of advanced methods in design of shear walls.

Conclusions

The design principles for shear walls given in Eurocode 5 are not satisfactory. There are shortcomings with respect to calculation of both stiffness and strength. It has been demonstrated that the structural behaviour of shear walls is very sensitive to wall geometry, boundary conditions and load configuration. The present European standard EN 594 for testing of shear walls recommends a test set-up that is very sensitive to the magnitude the applied vertical load. Therefore, the test results obtained by this test standard are not unequivocally and generally applicable.

It is proposed that the present standard EN 594 is revised by using, as the main alternative, a test method that evaluates the strength and stiffness of the shear wall in a pure shearing mode. This mode will render boundary and loading conditions corresponding to a fully anchored shear wall. This pure shear mode is introduced in a rational way in the wall by applying a diagonal tensile load at the top corner of the wall. The vertical component of the diagonal load will hold down the leading stud so it will act as fully anchored. It is recommended that the bottom rail be anchored continuously to the rigid test base.

The proposed test method will render basic test results that have general applicability as far as the actual sheathing and fastener materials are concerned. Different materials can then easily be compared. The necessary adjustment of the design value for shear walls applied in a specific construction with certain boundary and loading conditions is then proposed to be made either by supplementary testing or by using theoretical reduction factors that depend on these conditions and that can be obtained by analytical models for partially anchored shear walls.

The mechanical properties of the sheathing-to-timber joints have a major influence on the stiffness and strength of shear walls. Therefore, the racking test standard would need to be supplemented by a relevant method for testing of the joint characteristics. The present standard for load bearing nailed joints is not appropriate. A new test standard is needed that incorporate testing of the parameters of special interest, such as edge distance of the fasteners and loading, direction vis-à-vis the edge of the sheet.

By such a test standard for joints, design values for shear walls can be evaluated using general calculation models and the results can be compared to full-scale tests.

Proposal:

1) A new or revised test standard for determining basic stiffness and strength properties of sheathing-to-timber joints should be developed. Influence of edge distance and force direction should be included in this standard.
2) The main load configuration in test standard EN 594 should be changed to pure shear. This load configuration is more neutral with respect to different boundary conditions and could serve as a reference value for comparison of test results.
3) EN 594 should also include rules for testing of other load configurations that may occur in practice (e.g. the present main alternative).

By realising this proposal we open up for use of advanced methods in design of shear walls.

35-15-2U A Girhammar, L Wu, B Källsner
A plastic design model for partially anchored wood-framed shear walls with openings

Abstract

Design of shear walls has been a topic of major discussions during the Eurocode 5 work. The main problem has been that shear walls are fastened to the substrate in different ways in different countries and that this fact must be reflected in the code.

In **Paper 34-15-1**, a simplified plastic model for design of partially anchored wood-framed shear walls in the ultimate limit state was presented. The method covers static loads and can be applied when mechanical fasteners with plastic characteristics are used.

The main focus of the present paper is to extend this plastic model to design of partially anchored shear walls with openings. The method is applied to a few typical wall configurations with openings.

A few introductory tests of shear walls with openings have been conducted where the bottom rail was completely fixed to the substrate. The test results indicate that the proposed basic theory should be somewhat modified in order to obtain good agreement between measured and calculated load-carrying capacity.

Conclusion

A simplified plastic model for design of incompletely anchored wood-framed shear walls has been presented. The model can be used if the sheet material is fixed by mechanical fasteners and if these sheathing-to-timber joints show plastic behaviour. The model can only be applied on static loads in the ultimate limit state.

A few introductory tests of shear walls with openings have been conducted where the bottom rail was completely fixed to the substrate. The test results indicate that the proposed basic theory should be somewhat modified in order to obtain good agreement between measured and calculated load-carrying capacity.

35-15-3 S Nakajima
Evaluation and estimation of the performance of the shear walls in humid climate

Abstract

The effect of the moisture content of the lumbers and the sheathing materials on the performance of the plywood or OSB sheathed shear walls was reported in **Paper 34-15-2**. And the effect of the moisture content on the panel shear properties of the sheathing materials and the performance of the nail joints was also reported. Additional test data for the nail joints were collected to evaluate the effect of the humid climate on the lateral resistance of the joints. Lateral nail tests were conducted for all possible combination of the surface grain direction of the wood pieces and the sheathing materials and the loading direction.

The strength reduction of the shear walls due to the high moisture content of the composing materials was predicted by analyzing the characteristics of the nail joints and the sheathing panels under the humid condition. A numerical calculation was conducted to predict the strength reduction of the shear walls. The results of the analysis were compared to the test results.

Conclusion

In most case the lateral nail resistance of the nail joints was reduced in the humid climate and the reduction level depended on the type of the sheathing materials and also the direction of the load and the assembly of the studs and sheathing panels. And the failure mode of the joints to also affected on the yield and ultimate strength of the nail joints.

And at least for the plywood sheathed shear walls the test results of the lacking test and the lateral nail resistance test indicated that the yield strength reduction of the shear walls due to the humid climate can be roughly predicted by looking at the strength reduction of the nail joints conditioned in the same humid climate.
The yield strength reduction of the shear walls caused by the humid climate was well predicted by the simplified model. This simplified model can be a good predictor to evaluate the effect of the moisture contents on the yield strength of the shear walls.

35-15-4 B Dujic, R Zarmic
Influence of vertical load on lateral resistance of timber frame walls

Abstract
Laboratory tests are the most reliable source of information about the actual response of the load-carrying walls loaded with combined vertical and horizontal load. The laboratory device, which enables testing of cantilever walls, was developed at the University of Ljubljana. Fourteen cyclic shear tests were carried out on full-scale double panel units with different timber framing (corner connectors, extra vertical stud along the vertical edge of each unit) and OSB sheathing plate. They were preloaded with different vertical loads. The research was focused on hysteretic behavior of sheathing to framing connections, lateral resistance of timber-framed panels, and mathematical modelling of their response to combined vertical and varied horizontal load.

The behavior of timber-framed walls was governed by the non-linear response of connectors and anchors. The influence of vertical load on cyclic horizontally loaded panels was studied in details. It was found that the magnitude of vertical load has strong effect on lateral resistance of the wall. At small magnitudes of horizontal load the anchorage system increases the racking resistance of the wall. Fully anchored timber-framed shear walls with tie-downs at the leading stud have higher lateral resistance and load carrying capacity than partially anchored walls. At higher magnitudes of vertical load, the anchorage system does not have much influence on lateral resistance of a shear wall.

Conclusions
The main conclusions derived from herein presented experimental research are that the shear tests on cantilever timber-framed walls are very sensitive to boundary conditions and the magnitude of vertical load. Very important is also the way of applying loading on the test specimens. The tests results show that moderate additional investment in slight variation of panel composition (anchors, tie-downs) can significantly upgrade the shear resistance of the wall at the same magnitude of vertical load. Due to weak connections of framing elements and unsuitable anchoring, the magnitude of vertical load has a beneficial influence on racking strength of the wall. The bearing wall preloaded with lower vertical load should be fully anchored with the tie-downs on the exterior studs of the frame.

The proposed non-linear spring element is capable to simulate the behaviour of all kinds of connectors and anchors with typical characteristics of hysteresis loops. The simulation of inelastic behaviour of nailed connection is of crucial importance for successful prediction of the behaviour of timber frame walls and timber structures on seismic excitation. The presented mathematical model is also suitable for modelling the response of largely fenestrated panels. The whole layout of openings can be taken into account with real distribution and dimension of OSB sheathing, including the sheathing beneath the windows as well as sheathing and glued laminated timber beams above the openings. The comparison of calculated and experimentally obtained response of the tested timber frame wall shows excellent agreement even in the shape of hysteresis loops. For accurate analysis it is important to calibrate the mathematical model parameters for mechanical connectors with experimentally obtained results.

35-15-8 N Kawai, H Okiura
Design methods to prevent premature failure of joints at shear wall corners

Introduction
In a simple method to confirm the structural performance of timber buildings with shear walls against earthquake and wind forces, shear strength of each story in one direction is assumed to be equal to the summation of those of shear walls in the same direction. And each shear wall is regarded to have the same shear strength as is obtained by experiments with preventing uplift of the shear walls. However, this assumption is available at least when the premature failure of the joints at the corners of each shear

wall is prevented.

Some calculation method to predict the tensile force and some design methods to prevent premature failure at these joints have been proposed, and one method was provided in a Notification under the Building Standard Law of Japan in 2000.

First in this paper, these calculation methods are introduced. Next, the results of lateral loading test on two-storey structures with wood framed shear walls are summarized. Finally, the calculation results by these methods are compared with the test results and numerical analysis results, and the applicability of these design methods is discussed.

Conclusions

Some design methods to calculate tensile forces of joints at the shear wall corner were introduced. To discuss the adequacy of these design method for wood frame construction, lateral loading test was conducted on a two-storey structure with plywood sheathed shear walls with openings, and the tensile forces at the joints obtained by these design methods, numerical analysis and test results were compared.

As the results, there are tendencies that numerical analysis gives larger values than the test results, and all design methods give still larger values than the numerical analysis especially for the tensile force at hold down bolts at the end of the frame. In the three design methods, precise model with rigid beam assumption seems to give nearer values to the analytical values.

There are some possible reasons for the difference, such as lack of consideration for the effect of nails from plywood to framing members, distance between hold down bolt and centre of stud, effect of suspended walls and waist-high walls.

36-15-2 N Kawai, H Isoda
Applicability of design methods to prevent premature failure of joints at shear wall corners in case of post and beam construction

Introduction

In a simple method to confirm the structural performance of timber buildings with shear walls against earthquake and wind forces, shear strength of each story in one direction is assumed to be equal to the summation of those of shear walls in the same direction. However, this assumption is available at least when the premature failure of the joints at the comers of each shear wall is prevented.

Some design methods to prevent premature failure have been proposed, and one method was provided in a Notification under the Building Standard Law of Japan in 2000. In **Paper 35-15-8** the author introduced these design methods, and reported the comparison between calculation results and test results in case of wood framed construction.

In this paper, the results of series of lateral loading tests are summarized, which were conducted on two one-storied structures and six two-storied structures by post-and-beam construction with plywood sheathed shear walls or braced frames. Next, the calculation results using the design methods are compared with the test results. Finally, the applicability of these design methods to post-and-beam construction is discussed.

Conclusions

There are some design methods to calculate tensile forces of joints at the shear wall comer. To discuss the adequacy of two of these design methods for post and beam construction, a series of lateral loading tests were conducted on two one-storied structures and six two-storied structures by post-and-beam construction with plywood sheathed shear walls or braced frames, and the tensile forces at the joints obtained by these design methods and test results were compared.

As the results, there is a tendency that the two design methods give larger values than the test results in average, and the method using rigid beam assumption gives smaller values than the method using empirical factors. The difference between the calculation results and the test results is larger for the inner posts of the frames, but it seems that both design methods are applicable for the design of joints at end posts of the frames.

There are some possible reasons for the difference, such as lack of consideration for the effect of nails from plywood to framing members and distance between hold down bolt and centre of post.

36-15-3 D M Carradine, J D Dolan, F E Woeste
Effects of screw spacing and edge boards on the cyclic performance of timber frame and structural insulated panel roof systems

Abstract

Current understanding of the effects of screw spacing and installation of perimeter edge boards on behavior of timber frame and structural insulated panel (SIP) roof systems subject to earthquake loading is limited by a lack

of test data on these systems. To assess the effects of screw spacing and edge boards on diaphragm stiffness, quasi-static cyclic stiffness tests were conducted on three 2.44 m (8 ft) deep and 7.32 in (24 ft) wide roof diaphragm assemblies, and two 6.10 m (20 ft) deep and 7.32 m (24 ft) wide roof diaphragm assemblies. Data from tests were collected and analyzed in order to determine the effects of screw spacing and the installation of an edge board versus no edge board on cyclic stiffness, strain energy, hysteretic energy, and equivalent viscous damping. A rationale for the cause of these effects was developed based on results of monotonic failure tests that provided data on fastener breakage at failure. Data from monotonic and cyclic tests were incorporated in order to make design recommendations for timber frame and SIP structures subject to earthquake forces.

Summary and Conclusions

Cyclic testing of timber frame and SIP roof systems provided data regarding ductility, strength and stiffness degradation, and energy dissipation of these assemblies. In general, timber frame and SIP roof assemblies proved to be extremely stiff structural systems that maintained their structural integrity up to cyclic displacements which coincided approximately with design loads. Cyclic characteristics of SIP and timber frame roof systems supported the assumption that SIPS create a very stiff plate that is connected to the flexible timber frame utilizing only the SIP screws. Timber frame and SIP roof assemblies tested cyclically within the elastic limit of the screws exhibited increases in cyclic stiffness and strain energy with decreases in screw spacing and installation of perimeter edge boards.

In addition to observed behavior of timber frame and SIP roof assemblies subject to cyclic loading, justification for estimating Response Modification Coefficient, *R*, for use with IBC 2003 (ICC, 2003) seismic design procedures, was based on research conducted on SIP shear walls, research regarding timber frame and SIP shear walls, and comparisons with several other building systems including nailed shear walls, shear walls with adhesive attached sheathing and plain masonry shear walls. An *R*-value of 1.5 was recommended for calculating seismic forces on timber frame and SIP buildings as part of IBC 2003 seismic design procedures. This *R*-value is based on the assumption of a brittle, stiff vertical lateral load resisting system with relatively low energy dissipation and provides conservative values for determining seismic forces on timber frame and SIP buildings.

37-15-2 B Dujic, J Pucelj, R Zamic
Testing of racking behavior of massive wooden wall panels

Abstract

Massive wooden wall panels (SPF glued lamellate panel of size 244/244 cm), produced by Slovenian company RIKO HISS Ltd., were tested by monotonous and cyclic horizontal load in combination with constant vertical load. Monotonous horizontal load was applied following the EN 594 protocol. Testing by cyclic load followed the ATC-24 protocol for the testing of steel components. This protocol was applied because the main purpose of testing was to study the behavior of steel anchors. The influence of anchoring systems on shear stiffness and strength of timber wall panels was studied. The tested wooden panels have relatively high stiffness and load-bearing capacity. Therefore, the critical elements that govern the wooden shear cantilever response to earthquake excitations are anchors connecting panels with building foundation. The test results provided the basic data on stiffness and strength of the tested anchor systems that influence the entire racking response of massive shear cantilever wall panels. These experimentally obtained mechanical properties make possible the seismic design of prefabricated system according to EC8.

Conclusion

Test results give insight in the possibilities of low cost improvement of wooden wall anchorage systems. By simple modification of anchorage a significant stiffness increase can be achieved, while load bearing capacity is limited by material properties of anchorage components and by local strength of wood panel. The modified anchors have beneficial influence on the response of wooden houses exposed to strong earthquakes meeting the serviceability criteria. The failures of anchorages exposed to large deformations are repairable without involving high costs or labour efforts what makes post earthquake interventions less demanding.

The main conclusions derived from herein presented experimental research are that the racking behaviour of cantilever massive wood walls is very sensitive to the magnitude of vertical load and the type of anchorage system. The test results show that moderate additional investment in slight variation of anchorage system can significantly upgrade the racking resistance of the wall, especially at lower magnitude of the vertical load. By using unsuitable anchoring, the magnitude of vertical load has a beneficial influence on racking strength of the wall.

37-15-3 B Källsner, U A Girhammar
Influence of framing joints on plastic capacity of partially anchored wood-framed shear walls

Abstract
In this paper a plastic lower bound method is used to study the influence of framing joints (stud-to-rail) on the load-carrying capacity of partially anchored wood-framed shear walls. The calculations show that by considering these, the load-carrying capacity can often be increased by 10 to 15 %.

As a consequence of the calculations an alternative method is presented in which the full vertical shear capacity of the wall is utilised, disregarding that the conditions of equilibrium are not always fulfilled. This method results in a load-carrying capacity that is equal to or slightly higher than the capacity of the more complicated lower bound method. The great advantage of the method is that the load-carrying capacity can be determined in a simple straightforward process without any additional checks of equilibrium conditions.

37-15-6 H Mi, Y-H Chui, I Smith, M Mohammad
Predicting load paths in shearwalls

Abstract
This paper presents an evaluation of the use of two-dimensional finite element shear wall models to predict load path in a shear wall under lateral and vertical load. Previous modeling attempts focused on predicting load-deformation response of shearwalls. Comparison is made between model predictions and test measurements of shear wall responses. The study shows that ability to accurately predict so-called racking deformations is in itself no guarantee of accurate internal force prediction.

Conclusions
Finite element models associated with different shear wall configurations were presented. These models can perform non-linear static analysis of the shear wall and are able to model the behavior of shearwalls after strength degradation. Multi-linear curves derived from tests were used to define the properties of sheathing-to-framing and framing-to-framing connections. They are suitable to model the non-linear properties of these connections, and in fact are desirable for modeling the post-peak portion of load-deformation curve. The comparison between the model predictions and test measurements indicates that the agreement of overall system load-deformation behavior is excellent. A reasonably accurate prediction of the vertical force was also achieved. But significant differences between predicted and measured horizontal forces were also found. Research is required to investigate this further. At this stage it cannot be concluded that accurate load-deformation system response implies that accurate member force prediction can be achieved using finite element model.

38-15-3 M Popovski, E Karacabeyli
Framework for lateral load design provisions for engineered wood structures in Canada

Introduction
The main sources of lateral loads on buildings are either strong winds or earthquakes. Wind and earthquake loads, however, act in a different way on the building and impose different demands related to the strength, stiffness, and deformation properties of the building. For design purposes, a building subjected to wind loads is assumed to remain within the linear elastic range, so the stiffness and strength of the lateral load resisting system are of outmost importance. On the other hand, a building subjected to earthquake loads is expected to undergo non-linear deformations. Consequently, the seismic design process should consider a careful balance of the strength, stiffness, ductility, and energy dissipation properties of the lateral load resisting system of the structure.

Basic procedures for the design of buildings subjected to seismic and wind loads are provided in the national and international model building codes and material standards. At this point, the seismic and wind design provisions for engineered wood structures have to be improved to be competitive with those already available for structures built according to the other material standards. Design provisions are also needed to ensure that wood-based lateral load resisting systems can be combined with reinforced concrete and steel systems in hybrid structures. This paper proposes a framework for development of such enhanced lateral load design provisions for engineered wood structures in Canada.

Conclusions
This paper proposes a framework for establishment of a new lateral load design section in the Canadian Standard for Engineering Design in Wood (CSA086) to respond to the changes already in place in the 2005 edition of

the National Building Code of Canada (NBCC). The proposed section includes subsections on structural systems such as shearwalls and diaphragms, heavy frames (braced and moment resisting frames), and hybrid (mixed) systems, with their appropriate *R*-factors. The information on structural performance of the lateral load resisting systems should be linked to the performance of connections used in such systems. In order to better quantify the connection behaviour, design provisions for brittle and ductile failure modes for connection under static and dynamic loads should be developed in the corresponding design section for connections. Based on the connection performance, the connection section should also establish ductility categories for connections. In principle, the design procedures should move towards adopting capacity design procedures for lateral load resisting systems. Finally, there should be an effort to find a way to implement innovative connection and structural system solutions in CSA086. This will provide designers with more options regarding the structural system when designing engineered wood-based or hybrid buildings.

38-15-4 C Ni, E Karacabeyli
Design of shear walls without hold-downs

Abstract
Hold-down connections are often used to resist the uplift loads induced by wind or earthquake loads in wood-frame construction. When there is excess amount of walls available to resist these lateral loads, hold-down connections, are not always used in conventional wood-frame construction, particularly at the end of wall segments near door and window openings. In earlier editions of Canadian Standard for Engineering Design in Wood (CSA 086) and in other design codes, hold-down (or tie-down) connections were required, in designing wood-frame nailed shear walls, at the ends and around openings of a shear wall to provide restraint against overturning moment. To provide guidance for the design of shear walls without hold-downs, a mechanics-based approach was developed and implemented in the 2001 edition of the CSA 086. In this paper, this mechanics-based approach is presented. Implementation of the mechanics-based method with complete load path in the CSA 086 is also discussed.

Conclusion
A mechanics-based method to quantify the effect of overturning restraints on the lateral load capacity of a shear wall without hold-downs is presented. Predictions based on the mechanics-based method are in reasonable agreement with the test data. For all the cases, predictions based on the mechanics-based method are conservative when compared to the test results.

By taking into consideration of dead load and shear walls without hold-downs, the design provisions give engineers more options in the design of shear walls. As it is compatible with traditional shear wall design methodology, it is easier for design professionals to use the new method. This method will also shed light on explaining the superior performance of small wood buildings in past earthquakes, and on refining current design methodologies for shear walls in other wood design codes.

38-15-5 B Källsner, U A Girhammar
Plastic design of partially anchored wood-framed wall diaphragms with and without openings

Abstract
In the European timber design code, Eurocode 5, two parallel methods for design of wood-framed wall diaphragms are given. In order to get rid of this situation a unified design method is needed.

This paper gives a theoretical and experimental background to a plastic design method that can serve as a basis for a new code proposal. The principle of the design method has previously been presented for walls without openings. In this paper the methodology is extended to wall diaphragms including openings and the method is also presented in a more straightforward format. Conducted experiments show good agreement between measured and calculated load-carrying capacity.

Conclusions
An analytical plastic model for design of partially or fully anchored wood-framed wall diaphragms is presented. The model can only be applied on wall diaphragms where the sheet material is fixed by mechanical fasteners to the timber members and where these sheathing-to-timber joints show plastic behaviour. The model covers only static loads in the ultimate limit state.

The basic idea of the method is that the full vertical shear capacity of the wall diaphragm is utilised, disregarding that the conditions of equilibrium are not always fulfilled. It has previously been shown that the method gives a load-carrying capacity that is equal to or slightly higher than the capacity obtained by means of a more complicated plastic lower bound method.

Some tests of wall diaphragms with openings have been conducted where the bottom rail was fixed to the substrate. The test results indicate good agreement between measured and calculated load-carrying capacities.

The plastic design method may serve as a basis for a new revised design method in Eurocode 5.

38-15-6 B Dujic, S Aicher, R Zarnic
Racking of wooden walls exposed to different boundary conditions

Introduction

Post earthquake observations of damaged wooden houses and the analysis of experimentally tested structural elements have developed the worldwide knowledge about the response of wooden buildings on earthquake and strong wind. One of the major problems of understanding is related to boundary conditions and the influence of vertical loading on building elements. Learning from experimental and on-site observations, researchers have developed different test protocols and test set-ups trying to simulate the natural behavior of buildings as realistically as possible. Some of these efforts are reflected in codes and standards.

Eurocode 5 (ENV 1995-1-1:1993) introduces two methods for the determination of the racking strength of cantilever wall diaphragms: i) an analytical approach and ii) an experimental approach using a test protocol according to EN 594. Both approaches are related exclusively to timber frame walls with sheathing plates. However, the current construction practice introduces many other types of wall diaphragms. Among them, very popular are one or multi-layer boards or perforated glued panels and braced walls with different diagonal strengthening.

The Eurocode 5 calculation procedure is based on lower value of plastic capacity of the fasteners which connect the sheathing plates to the timber frame. The approach can estimate only the racking strength of panels having wood based sheathing plates where frame studs are fully restrained. In the cases of partially anchored studs and low magnitudes of vertical loading, the calculation may result in values that significantly overestimate the load-bearing capacity. The testing procedure according to EN 594 requires a partially anchored wall that does not necessarily represent the actual wall diaphragm used for construction of wooden buildings. The EN 594 load protocol does not use the cyclic horizontal load to simulate the earthquake loading. It is obvious that both analytical and experimental methods addressed in Eurocode 5 need to be upgraded.

The experimental investigations on the racking behavior of different types of wall diaphragms recently carried out at University of Ljubljana justify the need for further development of test protocols and analytical methods. The above commented influences of boundary conditions and vertical load should be properly taken into account. In this paper the experimentally obtained responses of wall panels exposed both to the EN 594 protocol and to cycling loading are presented. Three different cases of boundary conditions that may occur in real structures were applied and the magnitude of the constant vertical load was varied.

The intention of the presented research results is to support the discussion about potential needs for further development of relevant codes including Eurocode 5.

Concluding remarks

The importance of a proper taking into account of the boundary conditions and of the influence of vertical and type of horizontal loading is evident from the comparison of the behavior of differently tested panels. There is a need for further development of standard protocols for wooden wall diaphragms used for structures located in earthquake prone areas. New standards should implement the concept of performance-based earthquake engineering design to obtain experimental data needed for the evaluation of the behavior factor "*q*" and to set the values of story drifts defining the limit states of the story base shear diagram.

38-15-8 J Leskeld
Linear elastic design method for timber framed ceiling, floor and wall diaphragms

Abstract

The objective of the paper is to present a linear elastic hand calculation design method for timber framed ceiling, floor and wall diaphragms developed in author's licentiate's thesis.

Fastening lay-out contributes significantly to the calculatory racking capacity and horizontal deflection of ceiling, floor and wall diaphragms. Fastening lay-out depends on the orientation of panels and frame members

and on the spacing of frame members. The panels should be orientated parallel to the frame members in order to maximise the racking capacity and to minimise the horizontal deflection of blocked and unblocked diaphragms.

Further research is recommended to experimentally analyse the fastening lay-out's contribution to the racking capacity and the horizontal deflection of unblocked ceiling, floor and wall diaphragms.

Conclusion

Simplified analysis for roof, floor and wall diaphragms given in Eurocode 5 is reviewed. Equations for wall diaphragm unit given in STEP 3 method are extended to wall diaphragms and further to ceiling and floor diaphragms. Factors taking into account the fastening lay-out are tabulated for the most typical combinations of dimensions of a panel and spacing of fasteners.

Fastening lay-out's contribution to the racking capacity and the horizontal deflection of ceiling, floor and wall diaphragms is evaluated by comparing the calculation results of different lay-outs. The accuracy of calculations is enabled by cancelling the material properties causing inaccuracy.

38-15-9 R Griffiths, B Källsner, H J Blass, V Enjily
A Unified design method for the racking resistance of timber framed walls for inclusion in Eurocode 5

Introduction

A recent European research programme sponsored by Wood Focus OY identified a need for a unified approach in Europe to the design of timber framed walls for racking resistance. Such an approach should improve on and thereby make redundant current design methods viz:

– Methods A and B in Eurocode 5
– Methods 6.1 and 6.2 in BS 5268.

Work by Källsner et al was effective in linking the principles underlying the different design approaches and, following a number of early drafts, the method presented herein was proposed. It is written so as to be a direct substitution of Methods A and B in EC5 hence the numbering of the sections. It is not viewed as a final draft since the detailed development and checks were cut short by the time scale of the research programme. However the effectiveness of the method was checked by trials on a number of hypothetical wall units the results of which are reported.

The authors hope that the design approach will attract comment and development that could be incorporated prior to submission for adoption by both Code bodies in order that Europe should have a single design approach supported by all parts of the timber frame industry because it offered advantages over existing methods. Although not an objective of the research programme, the new approach can be more easily used with other similar frame type or sandwich panel systems.

Conclusions

1. This project offered the opportunity for a more in depth study which evaluated similarities and differences in national and Eurocode approaches and built on them to derive a unified design approach that should find favour with the major protagonists.
2. With regard to timber frame walls it was quickly found that timber frame housing sold into different markets in different countries and that the market requirements were likely to have a much greater influence on the design of a building and the use of materials than the approach to design for wall racking.
3. The contributors to this project were specialists in the field of wall racking, representing wide experience in the two design approaches both from a practical and a regulatory viewpoint. Having identified the weaknesses in both methods and requirements that could in relative safety be discarded, they were able to call on recent research material which provided the key to unifying the methods. As a consequence the programme was able to progress to a higher goal and has developed a design method known as the Unified Design Method, for obvious reasons, which should be seen as sufficiently close to both original approaches, whilst omitting their weaknesses, so as to obtain acceptance by all parties. The method also takes advantage of other research which has demonstrated the large reserves of inbuilt strength in timber frame construction in order to justify increased levels in performance compared with both previous methods. As a consequence the outcome of this work will improve the opportunities for timber frame against its competitors.
4. Overall the results show the Unified Design Method (Method C) is capable of performing well in all the critical areas. Its use will significantly improve racking design values especially for typical walls with many openings where the short lengths of full height wall mean the current Method A is extremely conservative.

5. Unified Design Method (Method C) has been written in such a way that it could immediately be substituted into ENI 995-1 -1 to replace the current design methods A & B. Subsequent changes are envisaged in the detail only, covering:
   - Fastener shear values
   - Allowable spread of racking load into window soffits
   - Extent of vertical restraint from adjacent windward edge wall component
   - Extent of zone of influence for vertical load

   These factors are more subjective in nature and it is therefore important that the design proposal is judged by a wider audience than the authors.
6. The method leaves open the debate on whether or not to include brick shielding and the racking contribution of brick and plasterboard. It is recommended that these factors be addressed in National Annexes.

39-15-1 U A Girhammar, B Källsner
Effect of transverse walls on capacity of wood-framed wall diaphragms

Abstract
It is well known that the structural behaviour of a wood-framed wall diaphragm is to a large extent dependent on the 3-dimensional behaviour of the whole building. In this connection the influence of transverse walls is an issue of special interest.

A plastic design method capable of analyzing the behaviour and capacity of partially anchored wood-framed wall diaphragms has been presented in previous papers. In this paper the plastic model is applied to the case where a wall diaphragm is connected to a transverse wall. The tying-down effect of the transverse wall on the vertical uplift is studied and the effect on the horizontal load-bearing capacity of the wall diaphragm is analyzed. In an ongoing study, tests are being conducted to evaluate the strength and stiffness of these transverse walls.

The paper describes the theoretical analyses and the experimental results for sheathed wood-framed transverse walls of different geometrical configurations and with different boundary conditions. Transverse walls are studied by varying the number of sheet segments and the horizontal fixing of the top rail. The extreme cases of horizontally fixed and free top rail are investigated. The effect of the tying-down action of transverse walls on the load-carrying capacity of wall diaphragms and the agreement between theoretical and experimental results are presented.

Conclusions
A plastic lower bound method is used to study the effect of transverse walls on the load-carrying capacity of partially anchored wood-framed wall diaphragms. The load-bearing capacity for transverse walls with various number of sheet segments subjected to a vertical uplifting force is derived.

The analytical values are compared to preliminary test results and are found to agree fairly well. A more comprehensive experimental study and evaluation need to be conducted to verify the analytical models.

The tying down effect of transverse walls with the same geometrical configurations as for the wall diaphragms is demonstrated for walls with one to four sheet segments. The effect is found to be very significant.

40-15-2 U A Girhammar, B Källsner
Effect of transverse walls on capacity of wood-framed wall diaphragms – part 2

Abstract
It is well known that the structural behaviour of a wood-framed wall diaphragm is to a large extent dependent on the 3-dimensional behaviour of the whole building. In this connection the influence of transverse walls is an issue of special interest.

A plastic design method capable of analyzing the behavior and capacity of partially anchored wood-framed wall diaphragms has been presented in previous papers. In **Paper 39-15-3,** the plastic model was applied to the case where a wall diaphragm is connected to a transverse wall. The tying-down effect of the transverse wall on the vertical uplift was studied and the effect on the horizontal load-bearing capacity of the wall diaphragm was analyzed. Tests were conducted to evaluate the strength of these transverse walls when the top rail was free to displace horizontally. In this paper a corresponding study of transverse walls, but now when the top rail is fixed with respect to horizontal displacement, is presented. Also, a more general description of the analytical model is given.

The paper describes the theoretical analyses and the experimental results for sheathed wood-framed transverse walls of different number of sheet segments and with fixed top rails. The effect of the tying-down action of transverse walls on the load-carrying capacity of wall diaphragms and the agreement between theoretical and experimental results is presented.

Conclusions

A plastic lower bound method is used to study the effect of transverse walls on the loa carrying capacity of partially anchored wood-framed wall diaphragms. The load-beard capacity of transverse walls with various numbers of sheet segments and the top rail fix against horizontal displacement subjected to a vertical uplifting force is derived.

The analytical values are compared to preliminary test results and are found to agree well. A more comprehensive experimental study and evaluation need to be conducted to verify the analytical models. Also, the effect of the framing forces needs to be taken into account.

The tying down effect of transverse walls with the same geometrical configurations as for the wall diaphragms is demonstrated for walls with one and two sheet segments. The effect is found to be very significant.

40-15-3 C Ni, M Popovski, E Karacabeyli, E Varoglu, S Stiemer
Midply wood shear wall system: concept, performance and code implementation

Abstract

This paper introduces a new shear wall system referred to as "midply wall". The new wall system uses the same wall sheathing and dimension lumber as standard shearwalls used in platform-frame construction in North America. In this wall, however, the sheathing is placed in the centre of the wall between a series of pairs of studs and plates placed on both sides of the wall sheathing. As a result, nails connecting the framing members to the sheathing work in double shear in contrast to single shear in standard shearwalls. This results in substantial improvements in the overall performance of midply walls. Test results for midply walls under monotonic, cyclic and dynamic loading conditions are presented and compared with those for standard shearwalls under similar loading conditions. Implementation of the midply wall in wood design codes in North America is proposed and issues related to the code implementation are discussed.

Concluding Remarks

A new shear wall construction system named Midply Wall System is introduced. The Midply Wall System is designed to provide superior resistance to earthquake and wind loads. The improved performance is achieved by rearrangement of wall framing components and sheathing used in standard shearwalls.

Comparison of the performance of midply and typical standard shearwalls is presented. The average lateral load capacity of midply walls was more than three times higher than in typical standard shearwalls. Test results showed that the stiffness values for midply walls were approximately two to three times the average stiffness values of typical standard shearwalls. Midply walls' energy dissipation was more than 3 times higher than in typical standard shearwalls.

Implementation of the midply walls in the Canadian wood design code CSA 086-01 is also discussed. The new wall system provides architects and engineers with more options in the design of wood frame construction in cases where the demand on the shearwalls exceeds the capacity of standard shearwalls. Besides narrow shearwalls in platform frame construction, the midply wall system can be used next to garage or window openings, in seismic upgrading of existing structures, and in prefabricated housing systems for application in areas with high risk of earthquakes and hurricanes.

40-15-5 M Yasumura, E Karacabeyli
International standard development of lateral load test method for shear walls

Introduction

Shear walls are widely used for resisting member against wind and seismic load in timber structures. Evaluation of the structural performance of shear walls under static and reversed-cyclic loading has become a requirement of wind and seismic design. The working group 7 of ISO-TC 165 is preparing the International Standard to provide a test method for static and cyclic lateral loading as a basis for the development of characteristics of shear walls for use in wind and seismic design.

The standard is intended to provide static and cyclic test methods as a basis for the derivation of lateral load resisting parameters which are required in the wind and seismic design of shear walls in timber structures. This standard can be used to determine those parameters under the conditions that;

(1) the boundary conditions are designed to ensure that the full shear capacity of the wall is achieved (Method I), and
(2) the boundary conditions are designed to reflect the intended actual construction details (Method II).

In Method I, the full shear capacity of the wall specimen is achieved through application of sufficient vertical loads, and/or adequate hold-down connectors or tie-down rods (at both ends of the wall specimen in cyclic test). In Method II, the wall specimen is tested with representative boundary conditions (e.g. anchorage, hold-down connector details), and the vertical (compressive or tensile) loads that are expected to be used in actual construction.

In this paper, a Finite Element Model with non-linear joint elements connecting sheathing panels to wooden frames was developed for wood-framed shear walls and the influence of the test methods including of the vertical restraint of the end of walls, vertical loads, and the length of walls on the shear capacity are studied.

The cyclic displacement schedule in ISO Standard 16670 which was developed for cyclic testing of connections in consultation with a group of international experts is also employed in this standard. The cyclic displacement schedule is intended to produce: (a) data that sufficiently describe the elastic and inelastic cyclic properties of the wall specimen, and (b) demands representative of those imposed on the wall by earthquake,

Conclusions

The following conclusions are derived from the parameter study which employed Fnite Element Model that is verified against the static and reversed cyclic lateral load tests of wood-framed shear walls performed according to the draft.

(1) The vertical restraint of the wall with hold-down connectors, for example, is the most influential factor on the test results of strength and ductility in racking tests
(2) The effect of vertical loads on the strength and ductility of one unit wall (L=910 mm) is comparatively small, and that of two and three unit walls (L=1820 mm and 2730mm) is more significant when no vertical restraints are applied.
(3) Maximum lateral loads of the wall without vertical restrains and vertical loads whose length is one, two and three units (L=910, 1820, 2730mm) are respectively 18%, 33% and 47% of the wall with full vertical restraint.
(4) Maximum lateral loads of the wall without vertical restrains and with the vertical loads of 6kN per stud (total vertical loads of 18kN, 30kN and 42kN, respectively) whose length is one, two and three units (L=910, 1820, 2730mm) are respectively 44%, 70% and 85% of the wall with full vertical restraint.
(5) Maximum lateral loads of wall with full vertical restraint is proportional to the wall length. The length of wall does not have an important effect on the shear strength per wall length if they are tested with Method I.
(6) Maximum lateral load of wall without vertical restraint increases exponentially with the wall length. The wall length of wall has an important effect on the shear strength of wall if no vertical restrains are applied.
(7) Stiffness of loading beam has an influence on the test results when the length of walll is large and the vertical load is high. This should be noted especially when the racking test is conducted with Method II.

40-15-6 B Dujic, S Klobcar, R Zarnic
Influence of openings on shear capacity of wooden walls

Abstract

The new generations of massive cross-laminated wooden structures are recently becoming more popular in European market. The new trends are also bringing multi-storey timber structures. Special attention is paid to buildings located in earthquake prone areas of middle and south Europe. Therefore, the appropriate guidelines for designing have to be set for existing and new timber structural systems to assure their seismic resistance. In design of wood structures, the contribution of fenestrated wall segments usually is not taken into account when calculating the wall shear capacity. Some experimental and analytical studies have shown that fenestrated wall segments may contribute to the earthquake resistance of the wood-frame plywood sheathed walls. The load-bearing capacity and stiffness of fenestrated wood walls are influenced mostly by the size and layout of the openings. To evaluate the shear strength and stiffness reduction for different size and placement of openings in the wall, development of a mathematical model verified against experimental tests, is of paramount importance.

The main goals of the experimental research and parametric study presented in this paper are to provide information on how to estimate the racking strength and stiffness of cross-laminated solid wood walls with openings, and to recognize how the shape and the area of the openings influence the shear capacity and stiffness of cross-laminated wood walls. Results form the preformed parametric study are summarised in diagrams that could serve as a practical tool for estimating the influence of fenestra-

tion on the stiffness and load-bearing capacity of cross-laminated solid wood walls. The study has concluded that openings with a total area of up to 30 % of the entire wall surface do not significantly influence the load-bearing capacity of the wall. The stiffness in such case, though, is reduced for about 50 %.

Conclusion

Non-fenestrated cross-laminated wooden walls have relatively high stiffness and load-bearing capacity. Therefore, the critical elements that govern the X-lam timber shear wall response to earthquake excitations are anchors connecting the panels to the building foundation. X-lam panel with large openings has lower shear stiffness, but its load-bearing capacity is not reduced as much, because failures are mostly concentrated in anchoring areas and in the corners around openings with smashing and tearing of wood. To evaluate the trends of shear strength and stiffness reduction for different fenestrations a numerical model was utilised and verified with experimental tests on full-size X-lam walls. To reduce the number of tests a numerical parametric study was performed for 36 configurations of openings in the walls of three different lengths. The study resulted in diagrams that could serve for simple engineering design of X-lam timber walls with openings using a reduction factor based on the ratio of the openings similar to light-frame walls. The parametric study showed that the openings with the area up to 30% of the wall surface do not reduce the load-bearing capacity significantly, while the stiffness is reduced for about 50%.

Additional experimental tests and numerical analyses will enlarge the knowledge related to lateral stiffness and stability of X-lam wooden walls with openings. Additional tests, already started at University in Ljubljana, Faculty of Civil and Geodetic Engineering, will serve the verification of the empirical equations presented herein.

41-15-2 B Källsner, U A Girhammar
Plastic design of wood frame wall diaphragms in low and medium rise buildings

Abstract

Design of wall diaphragms has been a topic of major discussions during the development of the European timber design code, Eurocode 5. The main problem has been that wall diaphragms are fastened to the substrate in different ways in different countries and that this fact must be reflected in the code.

A plastic method for design of wood frame wall diaphragms is presented which can be used in case of partially or fully anchored studs and fully anchored bottom rail. In this paper the method is applied to walls that are more than one storey high. The principles of the method are demonstrated for walls with and without openings. The possibility of using transverse walls for anchoring with respect to vertical uplift is illustrated.

Conclusions

A plastic method for design of wood frame wall diaphragms is presented which can be used in case of partially or fully anchored studs and fully anchored bottom rail. In this paper the method is applied to walls that are more than one storey high. The principles of the method are demonstrated for walls with and without openings. The possibility of using transverse walls for anchoring with respect to vertical uplift is illustrated.

41-15-4 T G Williamson, B Yeh
Combined shear and wind uplift resistance of wood structural panel shearwalls

Abstract

Shearwalls constructed with wood structural panels, such as plywood and oriented strand board (OSB), have been used to resist combined shear and wind uplift forces for many years in the U.S. For example, the Southern Building Code Congress International published SSTD 10, Standard for Hurricane Resistant Residential Construction, in 1999, which provided the shear resistance table for wood structural panels. When wood structural panels are used in combined shear and wind uplift, SSTD 10-99 also tabulated the wind uplift resistance of wood structural panels with a minimum thickness of 12 mm (15/32 in.) when used in conjunction with the shear resistance table.

Working with researchers at the National Home Builders Association Research Center (NAHB RC), Norbord Industries sponsored full-scale combined shear and uplift tests, showing that the cross-grain bending of the bottom plate, which is a brittle failure mode, could be avoided by using 5.8 x 76 x 76 mm (0.229 x 3 x 3 in.) plate washers with anchor bolts. The NAHB RC tests were conducted in lateral shear and tension (uplift) sepa-

rately, and the effect of combined shear and uplift was evaluated based on an engineering analysis.

After reviewing the NAHB RC study, APA and Norbord jointly conducted full-scale combined shear and wind uplift tests at Clemson University to gather more data on this subject. The test setup at Clemson University was capable of increasing the shear and wind uplift forces simultaneously until failure was reached by using a pulley system controlling the bi-axial forces in both lateral and vertical directions. Results of the Clemson study were used to support the development of engineering standards and changes to the national building code in the U.S., and are reported in this paper.

In 2007, APA constructed new combined shear and wind uplift test equipment that is capable of bi-axial loading in both lateral and vertical directions with independent but synchronized loading mechanisms. The vertical load can be applied as either an uplift force or a downward gravity load. Research results using this new equipment are utilized to enhance the understanding of the design on the bi-axial combined shear and wind uplift. This paper describes the latest finding from this research.

Conclusions and Recommendations

Results obtained from these studies confirm the adequacy of using an engineering mechanics analysis to evaluate the resistance of shearwalls when subjected to combined shear and wind uplift provided that that 5.8 x 76 x 76 mm (0.229 x 3 x 3 in.) plate washers are installed with anchor bolts spaced at 406 mm (16 in.) or less on center so that the cross-grain bending of the bottom wall plate can be avoided. In the future, the anchor bolt spacing may be further optimized and the design values provided in this paper expanded to include other shearwall configurations.

42-15-1 U A Girhammar, B Källsner
Design aspects on anchoring the bottom rail in partially anchored wood-framed shear walls

Abstract

A plastic design method capable of analyzing the load-bearing capacity of partially anchored wood-framed shear walls has been developed by the authors. In such walls the leading stud is not fully anchored against uplift and corresponding tying down forces are developed in the sheathing-to-framing joints along the bottom rail. These joint forces will introduce crosswise bending of the bottom rail with possible splitting failure along the bottom of the rail in line with the anchor bolts. If the bottom rail fails in a brittle manner, the applicability of the plastic method can be questioned. Therefore, design recommendations with respect to the bottom rail need to be given as prerequisites for using the plastic method.

This paper deals with the design of the anchoring devices needed to tie down the bottom rail properly and to eliminate any possible brittle failure modes. It describes the experimental results for sheathed bottom rails of different designs subjected to vertical uplift forces. The effect of different washer sizes and location of the anchor bolts in the width direction on the failure load due to splitting of the bottom rail is presented. The experimental results indicate that the decisive design parameter is the distance from the edge of the washer to the loaded edge of the bottom rail.

Conclusions

The tests indicate that the failure load of the bottom rail

- increases approximately linear with respect to the washer size for fixed locations of the anchor bolts. This is valid for both single and double sided sheathing;
- increases approximately linear as the distance from the edge of the washer to the loaded edge of the bottom rail decreases, regardless of the location of the anchor bolts in single sided sheathing; and
- for double sided sheathing is twice that of single sided sheathing.

The decisive design parameter seems to be the distance from the edge of the washer to the loaded edge of the bottom rail.

43-15-1 M Yasumura
Influence of the boundary conditions on the racking strength of shear walls with an opening

Introduction

It is well known that the boundary conditions such as the vertical restraints of wall panel and the vertical loads due to live and dead loads, snow load, etc. have considerable effects on the mechanical properties of wooden shear walls. These effects are to be considered especially for the design of timber structures and the racking tests on shear walls. In this context, Draft International Standard on the static and reversed cyclic lateral loading test method proposes two boundary conditions that is designed to ensure the full shear capacity of the wall is achieved (Method I), and that to reflect the intended actual construction details (Method II). This study reviews the experimental results and Finite Element analysis of wood-framed shear walls with various opening configuration and aims at clarifying the effects of boundary condition and the vertical loads on the shear capacity of shear walls with opening to propose its design method.

Conclusion

The following conclusions are derived from the racking tests of plywood-sheathed wood framed shear walls and the Finite Element analysis.

1) Racking resistance of shear wall with an opening and full vertical restraint can be estimated from that of the shear wall of the same size without openings by multiplying by a wall coefficient.

2) Restraints of studs (e.g. hold-down connections) besides the opening can be removed if the shear wall has an opening of window or door configuration with small walls above and/or under the opening and the end studs of the wall are connected tightly to the foundation and horizontal member above or below the shear wall.

3) Racking resistance of shear wall having an opening and partial vertical restraints (in which vertical restraints besides an opening are removed) can be estimated from that of the shear wall of the same size without openings by multiplying the reduction factor defined by the formula (4) .

4) It is important to consider the loading beam stiffness and the vertical compressive load over the wall if the wall has an opening and the studs besides the opening are not connected to the foundation or horizontal members, while these incidents do not affect on the mechanical properties of shear walls if the studs are fully connected to the foundation and horizontal members.

5) Maximum tensile forces of the end stud of the wall with full restraints showed almost constant values of 30 kN regardless of the wall length, opening area and configuration. That of the wall with partial restraints decreased slightly as the increase of the opening coefficient within the range of 29 to 31 kN.

43-15-3 T Skaggs, B Yeh, F Lam, D Rammer, J Wacker
Full-scale shear wall tests for force transfer around openings

Abstract

Wood structural panel sheathed shear walls and diaphragms are the primary lateral-load resisting elements in wood-frame construction. The historical performance of light-frame structures in North America are very good due, in part, to model building codes that are designed to preserve life safety, as well as the inherent redundancy of wood-frame construction using wood structural panel shear walls and diaphragms. As wood-frame construction is continuously evolving, designers in many parts of North America are optimizing, design solutions that require the understanding of force transfer between load-resisting elements.

The North American building codes provide three solutions to walls with openings. The first solution is to ignore the contribution of the wall segments above and below openings and only consider the full height segments in resisting forces, often referred to as segmented shear wall method. The second approach, which is to account for the effects of openings in the walls using an empirical reduction factor, is known as the "perforated shear wall method". The final method, which has a long history of practical use with surprisingly little research and testing, is the "force transfer around openings method". This method is accepted as simply following "rational analysis". Typically walls that are designed for force transfer around openings result in the walls being reinforced with nails, straps and blocking in the portions of the walls with openings. The authors are aware of at least three techniques which fall under the definition of rational analysis. These techniques result in prediction of the internal forces in the walls as differing by as much as 800% in extreme cases. This variation in predicted forces is resulting in either some structures being overbuilt or some structures being less reliable than the intended performance objective.

A joint research project of APA — The Engineered Wood Association, the University of British Columbia (UBC), and the USDA Forest Products Laboratory (FPL) was initiated in 2009 to examine the variations of walls with code-allowable openings. This study examines the internal forces generated during these tests and evaluates the effects of size of openings, size of full-height piers, and different construction techniques by using the segmented method, the perforated shear wall method and the force transfer around openings method. Full-scale wall tests as well as analytical modelling were performed. The research results obtained from this study will be

used to support design methodologies in estimating the forces around the openings. This paper provides test results from 2.4 in x 3.6 in (8 feet x 12 feet) full-scale wall configurations, which will be used in conjunction with the analytical results from a computer model developed by UBC to develop rational design methodologies for adoption in the U.S. design codes and standards.

Summary and Conclusion

Twelve different wall assemblies were tested to study the effects of openings on both the global and local response of walls. Several of these assemblies were tested with multiple replications. The replications showed good agreement between each other, even when test duration was extended to ten times greater the original duration. In terms of global response, the segmented wall approach resulted in walls with the lowest load factors (based on observed global load divided by allowable capacity of the walls), followed by walls built as perforated shear walls (i.e. no special detailing for forces around openings), and finally the walls specifically detailed for force transfer around openings. In general, as opening sizes increased, the wall strength and stiffness values were negatively impacted. An observation that was not expected is that for walls with typical window openings, the walls with the narrowest piers based on minimum pier width in North American codes, resulted in higher load factors than walls with full width piers (height-to-width ratio of 2:1).

Of the twelve walls tested, internal forces were collected on eight of the assemblies. For the walls tested, the measured forces at the bottom of the windows were greater than the measured forces at the top of the window. Also, as expected, as the window opening increased and as the pier width decreased, the strap forces increased relative to the global applied force to the wall. Of these eight assemblies, one can conclude that the drag strut technique consistently underestimated the strap forces, and the cantilever beam technique consistently overestimated the strap forces. The Diekmann technique, the most computationally intensive, provided reasonable strap force predictions for the walls with window type openings. The Diekmann technique significantly over predicted the strap forces for large garage type openings.

43-15-4 B Yeh, E Keith, T Skaggs
Optimized anchor-bolt spacing for structural panel shear walls subjected to combined shear and wind uplift forces

Abstract

Wood structural panels, such as plywood and oriented strand board (OSB), have been used to construct shearwalls with a long history of success. However, most wood panel shearwalls are designed as lateral load resisting elements and their capability of resisting uplift forces induced by wind forces has been frequently overlooked. In high wind areas, such as the Gulf Coast in the U.S., the use of wood structural panel shearwalls to resist combined shear and uplift forces has gained attention in recent years due to the reduction in the need for expensive and labor-intensive metal tension straps between studs and the foundation.

The U.S. building codes and design standards have established provisions for using wood panel shearwalls to resist combined shear and uplift forces. Unfortunately, the provisions contained in the U.S. building codes and design standards require a very close spacing, 406 mm (16 in.) on center, for anchor bolts that tie bottom wall plates to the foundation regardless of the magnitude of the shear and uplift forces. This conservatism was largely due to the concern over the cross-grain bending failure on the bottom wall plate, which is a predominant failure mode when subject to combined shear and uplift forces, and the lack of full-scale wall test data to optimize the anchor bolt spacing as a function of the shear and uplift forces.

In response to the desire of builders to reduce the costs associated with the unnecessary close anchor bolt spacings prescribed in the codes and standards, and to promote the concept of advanced framing of Optimal Value Engineering (OVE) of green building movement, APA conducted a research with intent to optimize the anchor bolt spacing for wood panel shearwalls when designed for combined shear and uplift forces. Supported by the results of 26 full-scale combined shear and uplift tests, the research shows that the anchor bolt spacing can vary between 406 mm (16 in.) and 1219 mm (48 in.), depending on the magnitude of the shear and uplift forces. This paper presents the test results from the research and proposes a change to the design standards.

Conclusion

It is shown that load factors of 2.0 or greater were achieved by the assemblies selected to verify the model. As the anchor bolt spacings listed in Table 4 may be difficult to construct in the field, Tables 9 and 10 were developed. Table 9 provides anchor bolt spacings rounded down from Table 4 to 152-mm (6-inch) increments on center. The anchor bolt spacing of 406 mm (16 inches) on center were unchanged in Table 9 as this spacing is a commonly used construction spacing interval in North America, is widely published, and currently exclusively recommended for combined shear and uplift anchor bolt spacing. For practical use, Table 10 further rounds the anchor both spacing from Table 4 down to traditional spacing intervals of 406, 488, 610, 813, 914, 1067, and 1219 mm (16, 19.2, 24, 32, 36, 42, and 48 inches) used in the conventional construction in North America.

Table 9. Anchor bolt spacing (mm) rounded down to spacing with 152-mm (6-inch) increments

Nail (Common)	Nominal Unit Shear (kN/m)		Nominal Unit Uplift (kN/m)										
	SG 0.50		0	3.2	6.3	9.5	12.6	15.8	18.9	21.3	25.2	28.4	31.5
		SG 0.42	0	2.9	5.8	8.8	11.7	14.6	17.5	19.7	23.4	26.3	29.2
8d @ 102 mm (11-mm panel)	0	0	1219	1067	1016	914	762	610	610	457	406		
	5.8	5.4	1219	1067	1016	914	762	610	610	457	406		
	9.8	9.0	914	762	762	610	610	610	457	457	406		
	14.3	13.2	610	610	457	457	457	457	406				
10d @ 152 mm (12-mm panel)	0	0	1219	1067	914	914	762	610	610	508	406	406	406
	5.8	5.4	1219	1067	914	914	762	610	610	508	406		
	12.7	11.7	610	610	610	457	457	457	457	406			

1 mm = 0.0394 in., 1 kN/m = 68.5 lbf/ft

Table 10. Anchor bolt spacing (mm) rounded down to common construction spacings

Nail (Common)	Nominal Unit Shear (kN/m)		Nominal Unit Uplift (kN/m)										
	SG 0.50		0	3.2	6.3	9.5	12.6	15.8	18.9	21.3	25.2	28.4	31.5
		SG 0.42	0	2.9	5.8	8.8	11.7	14.6	17.5	19.7	23.4	26.3	29.2
8d @ 102 mm (11-mm panel)	0	0	1219	1067	914	914	813	610	610	488	406		
	5.8	5.4	1219	1067	914	914	813	610	610	488	406		
	9.8	9.0	914	813	610	610	610	610	488	488	406		
	14.3	13.2	610	610	488	488	488	406	406				
10d @ 152 mm (12-mm panel)	0	0	1219	1067	914	914	813	610	610	488	406	406	406
	5.8	5.4	1219	1067	914	914	813	610	610	488	406		
	12.7	11.7	610	610	610	488	488	488	406	406			

1 mm = 0.0394 in., 1 kN/m = 68.5 lbf/ft

Results presented in Table 10 have been incorporated into the APA System Report SR-101, *Design for Combined Shear and Uplift from Wind*, which may be used by design engineers on a voluntary basis before the provisions can be adopted into the U.S. national design standards, such as the SDPWS and the ICC-600. Since the installation details are critical to the combined shear and uplift applications, APA SR-101 also provides many practical details to ensure the wall assemblies are properly designed and constructed. It would be prudent to follow the APA SR-101 construction details closely when designing and constructing wood structural panel shearwalls for the combined shear and wind uplift applications.

While APA SR-101 still recommends the use of 15.9-mm (5/8-in.) anchor bolts as specified in the SDPWS and the ICC-600, test results obtained from this study showed that the 12.7-mm (1/2-in.) diameter anchor bolts are adequate for use in the combined shear and uplift applications when the design loads are within the range permitted in the current U.S. national design standards.

3.18 TRUSSED RAFTERS

Trussed rafters were right from the beginning a focus area for CIB-W18. Three topics were of special interest:
- simplified design methods
- bracing
- the development of deflections over time under alternating loads

Simplified design methods
Trusses, especially with punched metal plate fasteners, are used extensively in houses. They are complicated, statically indeterminate structures and until computers became generally available there was a great interest in simplified design methods making it possible to design them by hand.

4-9-1 T Feldborg and M Johansen
Long-term loading of trussed rafters with different connection systems

The long-term loading was started at the Building Research Instite in connection with short-term testing of the same truss types carried out by Mr. Arne Egerup at the Technical University.

The investigation comprises W-trusses made of 45 mm timber connected by Hydro-Nail and TCT metal plates, nailed metal plates and plywood gussets, and trusses made of two layers of 25 mm boards connected by nailed metal plates between the boards.

The supports are arranged in 3 ways, 1: below the heel joint, 2: 0,6 m from the heel joint in both sides, and 3: 1,2 m from the heel joint in one side below an extra web member.

Each type is represented by 2 trusses.

The load arrangement is shown in the figure. The trusses are designed for light roof cladding. Some types are overloaded by dead load representing heavy roof cladding.

So far the load has been varied between 8 week dead load and one week dead load + full design snow load. (Full design snow load is very rare in Denmark, and according to 30 years' measuring the duration does not exceed 5 days).

After 1 year of alternating loading it is planned to change to full design load for 4-5 years.

Deflection of the trusses is being measured at the joints and at the middle of the members of the upper and lower chords.

Temperature and relative humidity are recorded and the moisture content of wood samples placed with the trusses is measured.

6-9-3 T Feldborg, M Johansen
Deflection of trussed rafters under alternating loading during a year

Introduction
Trussed rafters are used in the greatest part of building in Denmark. Most of the trussed rafters are now factory made and connected by toothed metal plates, but also nailed connections are still used.

The common simple design calculations neglect the effect of rigidity and deformation of the joints. They therefore give a wrong picture of safety and deflections. It often results in bad utilisation of the timber and sometimes in too great deflections of the trusses.

Therefore the Technical University and the Building Research Institute have started a research programme to form a basis for more accurate and economic calculations of trussed rafters.

At the Technical University Mr. A.R. Egerup has investigated the short-term stiffness and bearing capacity of 56 trussed rafters. At the Building Research Institute the short-term behavior of the heel joint and the lower chord splice are studied, and the long-term behavior of the trussed rafters will be studied within 5 years.

This report presents the results from one year during which the load has been alternating between dead load and dead load + design snow load.

14-14-1 T Feldborg, M Johansen
Wood trussed rafter design

Summary
The tests described in this report are part of a Nordic co-operation on the development of more rational design methods for wood trussed rafters. Heel joints and splice joints made with two types of nail-plates and with nailed steel and plywood gussets were tested under short-term loading in laboratory climate. The same types of joints were used in long-term loading tests on rafters in an open pole building, in which the deformations, climate and moisture content were registered for a period of 4,5 years. On the basis of these and other Nordic tests, methods are proposed for the design of the joints and for the calculation of forces, moments and de-

flections for wood trussed rafters.

The moisture content of the trussed rafters was higher and varied more than anticipated (from 0.12 to 0.21)

The deflections increased slightly each time the timber dried out.

The deformations of the joints showed very big variation coefficients. Under long-term loading and moisture variations the slips were 3-20 times greater than under short-term loading.

The investigation showed that the slip stiffness of the joints has almost no influence on the normal forces but to some degree on the moments, and that it has a great influence on the deflections.

The rotational stiffness of a joint has a considerable effect on the moments in the parts of the timber closest to the joint.

The rotational stiffness of a heel joint is almost independent of the rotational stiffnes of the connectors in the joint but depends on the length of the joint and the normal force of the rafter.

14-14-2 R O Foschi
Truss-plate modelling in the analysis of trusses

The structural analysis of a truss requires a realistic representation of the partial fixity provided by the truss-plate joints. This degree of fixity depends on the load-slip characteristics of the plate/lumber combination, on the orientation of the plate and the area of plate covering each of the joined members. Furthermore, joint stiffness is influenced by the presence of gaps between members, local plate buckling and/or yielding. This paper presents a method of analysis of truss-plate joints which is straightforward and takes into account all of the above factors, allowing the determination of maximum truss loads based on connection capacity.

15-14-1 H Riberholt
Guidelines for static models of trussed rafters

Object

The object of the guidelines is to constitute a basis for the static modelling of timber trussed rafters. The static models are intended used for analysis of stresses and deflections.

The scope of the guidelines is primarily plane trusses with nail plate connections. But they may be applied to trusses with other types of connectors with similar static behaviour.

15-14-2 F R P Pienaar
The influence of various factors on the accuracy of the structural analysis of timber roof trusses

Introduction

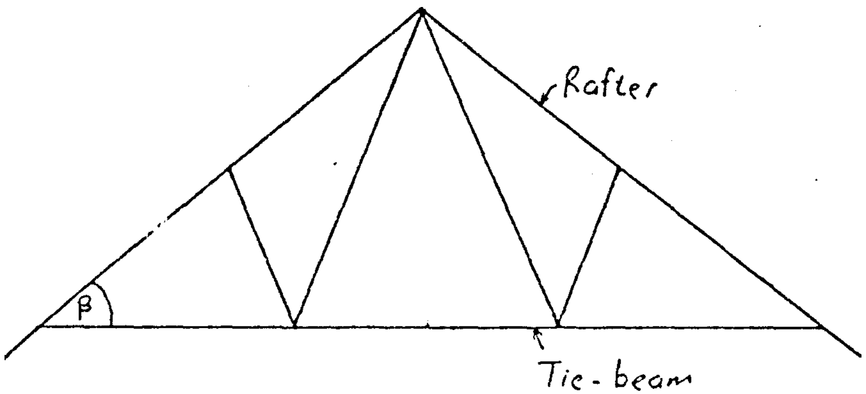
In this study, the influence of three factors on the structural analysis of timber roof trusses is investigated, namely:

- stiffness of joints (e.g. rigid, hinged, or semi-rigid, rotational and axial)
- second order effects i.e. additional bending moments which result from the deflected shape of the structure
- using wrong member sizes for the analysis.

The stiffness of joints and the second order effects can only be taken into account by sophisticated methods of analysis. It is therefore important for the designer to know what the order of the error is when using simplifying assumptions.

Because most roof trusses are not statically determinate structures, the member sizes are only known after the design process has been completed. Therefore an assumption must be made for the original analysis. The influence of incorrect member sizes on the analysis of the trusses is investigated.

In the investigation, only the W-truss is used because it is the most common configuration of timber roof trusses.



Alhough only one configuration is used, three different truss spans, (6 m, 9 m and 12 m) and four different roof pitches (100, 150, 200 and 300) are considered.

The load on the trusses is taken as that for a tiled roof, and sometimes as that of a sheeted roof.

It is generally expected that, if the sizes of the rafters and the tie-beams change, but retain the same relationship to each other, the stress distribution between the chords will change very little. However, the results given in Table 16 show that if both the tie-beam and the rafter is chosen, a size too large or too small, the forces and moments change significantly. This is because the standard sizes do not change in the same proportion as the stresses in the chords. If the one chord is too large, and the other too small, the influence is even greater and can be as much as 73 per cent, as shown in Table 16. In general, it seems as though the effect on structural analyses of the wrong choice of size decreases with an increase in pitch.

Conclusions

The use of the wrong size members in the structural analysis has a surprisingly large effect. It is therefore recommended that if member sizes are determined in a design process, it should be established that the analysis was carried out using those sizes. If not, the analysis and design process must be repeated.

General conclusions and recommendations

The influence of the following factors on the structural analysis was investigated:

– rigidity of joints
– second order effects
– incorrect assumptions of member sizes for the analysis

It is more accurate to regard joints as completely rigid than hinged (with continuity of members taken into account), although the joint is in reality only semi-rigid. Static analyses are acceptable for standard cases, but should preferably not be used for spans over 12 m and slopes under 15º. Where axial rigidities of joints are low, (e.g. bolted joints) the influence is great and must be taken into account. This can only be done using sophisticated methods of analysis which are not always readily available.

In general, the influence of the second order effects is quite small and may be ignored in ordinary design.

15-14-4 P O Reece
The design of continuous members in timber trussed rafters with punched metal connector plates

Synopsis

On the principle of superposition, bays of continuous members are regarded as simply supported at the node points with end-couples superimposed to simulate the effects of continuity in an indeterminate structure subjected to the simultaneous action of axial and lateral forces.

From the generalised theorem of three moments an expression for the maximum bending moment is derived, and this is regarded as a function of the true bending moment induced under test conditions.

Similarly, an expression is derived for the moment of resistance, and this also is regarded as a function of the true moment of resistance. By determining from published span tables the ratio of the calculated bending moment, to the calculated moment of resistance, factors are determined that represent permissible bending moment, per unit of moment of resistance as calculated.

An additional factor is introduced limiting the longitudinal strain to a fixed proportion of the strain induced at the Euler critical load in both tension and compression.

16-14-1 V Picardo
Full-scale tests on timber fink trusses made from Irish grown Sitka Spruce

Scope

Full size tests have been carried out on fifty eight trusses of the Fink configuration using home-grown (Irish) Sitka spruce timber. The timber was of the M75 grade. Metal plate fasteners with integral teeth were used at all joints using the sizes recommended by the plate manufacturers. Forty eight of the trusses were fabricated to the limiting spans for a higher Species Group (S2 - M75).

Except for three trusses all passed the acceptance criteria set in the Irish Standard. The results obtained from the tests are not included in this paper.

Only a brief summary of some of the analyses is shown in the tables at the end of the paper. On the results obtained new limiting spans were proposed for Fink trusses made from home-grown timber.

17-14-1 V Picardo
Data from full scale tests on prefabricated trussed rafters

The data shown in the following tables supplement CIB-W18 **Paper 16-14-1** presented in Norway (June 1983) and is given here at the request of various members of the Group. Because of the volume of data it was necessary to present the data in 5 separate tables as follows

Table A.1 Physical measurements of trusses
Table A.2 Plate sizes
Table A.3 Deflections from 24 hour deflection tests
Table A.4 Deflections from strength tests
Table A.5 Failure Loads and description of failure
Table A.6 Deflection recovery and span/deflection ratios at long-term design Load.

17-14-2 H Riberholt
Simplified static analysis and dimensioning of trussed rafters

Introduction

In **Paper 15-14-1** guidelines for static models of trussed rafters are given, and these are mainly thought applied to frame models. In this paper a simple calculation method is forwarded for the static analysis and dimensioning of trussed rafters.

It has been an object to forward a calculation method which is as simple as possible and still taking into account the essential effects.

Due to lack of time only W-trusses have been investigated, but it is expected that later on other types of trusses will be dealt with in a similar manner.

17-14-3 B Källsner
Simplified calculation method for W-trusses

Introduction

This paper presents a simplified calculation method for W-trusses. The method is applicable to heel joint constructions with or without wedges. The proposed construction of the heel joint with a wedge has some advantages over other heel joint constructions since the magnitude of the bending moments in the heel joint and their distribution between upper and lower chords may be influenced by altering the length of the wedge.

The greatest difficulty in the calculation of W-trusses is to take into consideration the influence on the moment distribution of the displacements of the joints. Consequently the report concentrates on the determination of the bending moments while the calculation of the axial forces has been left out. A more complete report will be published later in which dock W-trusses and WW-trusses will be treated.

18-14-1 B Källsner
Simplified calculation method for W-trusses (Part 2)

Introduction

At the CIB-W18 meeting in 1984 the author presented in **Paper 17-4-3** a calculation method for W-trusses". It was stated in this paper that a more complete report dealing with W- and WW-trusses would be published later. This complete report is now under preparation.

The calculation method presented in **Paper 17-4-3** dealt with the general case of a heel joint subjected to an eccentric support reaction. At the CIB-W18 meeting some participants suggested that the proposed analytical method was too complex for design application. The author's response to the suggestion was that it was possible to develop a simplified procedure based on the results of the study. This is demonstrated in this paper. The calculation method is simplified by neglecting some moment contributions which have only a small influence on the total bending moment distribution in the truss, and which are thus not significant.

The calculation method is presented for two different cases, a W-truss without support eccentricity at the heels and a W-truss with support eccentricity at the heels.

18-14-2 T Poutanen
Model for trussed rafter design

Summary

The paper describes a design model for trussed rafter with emphasis on stress analysis. The model is based on a two-dimensional nonlinear frame program. Special attention is paid to joint modeling. Joint eccentricities are calculated assuming that all forces are transferred through nail plates, which means no contact and friction between timber beams. The model is

used in everyday trussed rafter design. Some design experience is described.

19-14-2 H Riberholt
Simplified static analysis and dimensioning, part 2

Introduction
In **Paper17-14-2** a simple calculation method is forwarded for the determination of the internal forces and moments in a W-truss. This paper continues along the same line for WW-trusses, scissor trusses, King post trusses and pulpit trusses.

These papers are based on the moment coefficient method.

- the normal forces are calculated from a pinned joint truss model, where the distributed loads are replaced by statically equivalent concentrated nodal forces
- the moments in the chords are determined taking their continuity in consideration.

For distributed loads over several bays this may be done by means of moment coefficients k_{mom}, that are found by calibration to the results of frame programs.

For minor distributed loads over a part of a bay or for a concentrated force in a bay the moment distribution in the chords may be found by modelling them as continuous beams over several spans with nodal moments reduced in order to take into account the effect of the deflection of the truss nodes.

Moment coefficients are given depending on

- Slope of the roof
- Slip in the connections
- Eccentricities in the connections between the diagonals and the chords
- Eccentric support of the heel joint
- The dimensions of the bottom and top chord, including the stiffness of the chords
- The span of the truss
- The load on the top chord in proportion to the load on the bottom chord, e.g. heavy or light roof
- Attic load over a part of the bottom chord
- Continuous or pinned jointed chords.

The conclusion of the discussion of **Paper19-14-2** was that the time had come to change the basis for Eurocode 5 from simplified design methods to computer based methods. The basis for these methods is to a large extent established in a working group chaired by Hilmer Riberholt.

19-14-3 T Poutanen
Joint eccentricity in trussed rafters

The author has described the basic model in **Paper 18-4-2.** The model (computer program) is constantly under development. This paper describes the development level reached by August 1, 1986.

20-14-1 T Poutanen
Some notes about testing nail plates subjected to moment load

Introduction
In the CIB-conference 19, Florence, Italy the author was asked to write a paper about testing nail plate joints subjected to moment load. This paper is a preliminary reply to the request. A research project has been established under the leadership of the author lasting 30.4.1989 having the aim to define guidelines for design of the moment loaded nail plate joint. However, three suggestions will be made and offered for a discussion. A more thorough report will be published later.

20-14-2 T Poutanen
Moment distribution in trussed rafters

Introduction
A truss test procedure was developed in a thesis. 10 trusses were tested in full scale. All the trusses were quality control trusses, they came from different plants and they were selected at random from a current production lot.

It was not realized to pay attention to joint gaps. The importance of the joint gap was revealed afterwards while analyzing the results (e.g. the rafter 6 had apparently gaps with eccentric contact in several joints). These trusses were used as reference trusses in a study, which included the definition of the moment distribution with 3 models: continuous beam, pin joint frame and rigid frame. The results have been copied into this paper. The author has developed an analysis model for trusses. The results of this model are presented. The author is continuing the study of moment distribution in trussed rafters

20-14-3 A R Egerup
Practical design methods for trussed rafters

Simplified methods often give the "right" result whereas complex methods often give a conservative solution. This can he proved by mathematics or energy methods and can be seen, for example, for different types of finite elements. The simplified method, such as moment coefficients, can more easily be adjusted to, say, full-scale tests by "adjusting" the moment coefficients or the buckling lengths.

A complex method, for example a finite element model, is very rigid and members are more inter-related so that it difficult to adjust to full-scale testing".

The difference in the two types of method is that the complex method takes more effects into account. Most people agree that design method for trussed rafters must be based on a complex finite element model (frame model), but so far no one model has proved satisfactory over the whole range of trusses. Why does a complex model not give the right answer? This is because the model is still simplified and does not take all effects into account. Among the more important effects are:

1. Slip in joints
2. Rotational stiffness in joints
3. Non-linear behaviour
4. Strength distribution in joints and members
5. Real buckling length
6. Redistribution of stresses in a non-determinte structure.
7. Probability of failure in a non-determinete structure
8. Interaction formula

It is very difficult to take all these effects into account in a day-to-day design method where speed is an important factor for the industry. For example, the strength distribution along members related to a failure criterion can only be determined with a probabilistic method which will be very time-consuming.

The non-linear behaviour of the model is different for trusses with low pitches and high pitches, mainly because of different slip behaviour in the heel-joint.

Any failure criterion, the inter-action formula has proved to be very poor for determinant structures.

A complex model which takes all important effects into account is impossible to work with. However, a combination of a complex model and full-scale testing will often take all the effects into account in a more simple way and produce a reliable design.

22-14-1 H Riberholt
Guidelines for design of timber trussed rafters

Scope
The intention of this paper is to describe some methods applicable for the design of plane timber trusses. The paper gives a proposal for a structure and setting for a standard for design methods. The content of the individual sections must be considered further.

Such methods may be divided into three operations, but these do not need to be independent and they will in general not just be carried out as one sequence from the top to the bottom.
1. Static analysis of a truss.
2. Strength verification of the timber.
3. Strength verification of the connections

The intention of this paper is to describe some methods applicable for the design of plane timber trusses. The paper gives a proposal for the structure and contents of a standard for design methods. The design may be divided into three operations:
– Static analysis of a truss
– Strength verification of the timber
– Strength verification of the connections.

The paper proposes a method which can be carried out with ordinary plane frame programs. The paper concentrates on plane timber trusses with nail plate connections, but the method can also be applied to trusses with other types of connectors, if the method is adapted to their static behaviour. Further some of the ideas can be used in more general models, which for example take into account spatial behaviour and non-linearity in the material or in the geometry.

1. Static analysis
Paper 15-14-1 proposes general static models, which to a large extent have been used in succeeding CIB-papers on trusses. The paper gives valuable guidelines on the modelling of trusses and joints, the use of fictitious elements and spring elements and their stiffnesses.
.
2. Strength verification of the timber
The main assumption is that this can be carried out as described in the timber codes, for example "CIB, Structural timber design code" (No decision on drafting Eurocode 5 was taken at that time). Some effects special for timber trusses are assumed accepted, e.g. the effect of the pronounced moment peaks in the chords, see the Annex at the end of this paper.

3. Strength verification of the connections.
The strength verification of the connections is described in **Paper 19-7-7** and in **Paper 14-7-1** which describes the methods in details.

Available static analysis methods and their application
The prevailing analysis methods in practice at present and in the near future are based on plane frame programs. Their assumptions are typically: Linear elasticity, first order method (geometric non-linearity is not considered), straight beam elements. Further some programs have the option that deformations in the joints can be modelled by a prescribed slip or a spring element. More refined assumptions are not common in practical programs according to a survey carried out recently.

It is a scope of the paper to set up guidelines so that analyses with such plane frame programs reflect the real static behaviour of the truss as well as possible.

One of the parameters in the static model of a truss is the principle for modeling of the deformation in the connections. There exist the following two basic possibilities: Prescribed slip and Elastic spring element. A mixture of these two options could be favourable.

Prescribed slip
The prescribed slip method has as the' basic idea, that the nail plate connections normally are dimensioned so that their strength capacity is fully utilized in the ultimate state. From tests the deformations (slips) are known at this state, and it is the impression that for a certain type of connection the slip close to failure has a fairly constant value independent of the size of the plate. So if this slip is prescribed in the connection the static model should reflect the actual static behaviour.

If the designer wishes to reduce this slip he only has to give the plate an oversize. Therefore the guidelines should comprise a description of how the slip depends on the relative strength utilization of the nail plate connection.

The objections against this method are that the designer for each load case must know the size and direction of the forces and moments in the connections (this knowledge could arise from a previous analysis or pre analysis in the iteration) and the slip is taken into account in a rather coarse manner.

Elastic spring element
Due to the non-linear behaviour of the connections the stiffness constant must be determined as a secant-modulus. Depending on whether the analysis is for the serviceability state or ultimate state the secant must be determined from a slip at a low or a high stress level.

The objections against this method are similar to those against the slip method: The designer must for each load case know the relative strength utilization in order to determine the secant modulus. The stiffness matrix must thus be updated for each load case. (Again this knowledge could arise from a pre-analysis). The slip is taken into account in a rather coarse manner. For example the influence of the moment on the stiffness for a force will normally not be considered even though it is theoretically possible to set up such a relation.

Since both methods have advantages and disadvantages it is recommended to let the user make his own choice. So the necessary information and values of slip and stiffness constants must be available. Some values are given in literature, but it seems necessary to have the values in the nail plate certificates too.

For some special or extreme cases it would be advantageous if the guidelines could state that slip can be disregarded. This may not be so extreme, it could be sufficient to have a nail plate connection with a certain oversize (50%-100%).

Annex. Effects of the distribution of the internal forces and moments.
Since the normal forces normally are almost constant along the bay length

and since shear is normally not decisive, it is proposed to disregard the distribution effect on the tensile, compression and shear strength.

However the moment stresses will typically vary along the length of the timber parts in other ways than assumed at the determination of the bending strength values given in the relevant standards. Since the bending capacity of a timber part depends on the distribution along the length it is proposed to take this effect into account.

The research carried out in this area is scattered and how to deal with this effect has hardly found its final method of description.

The dependency between the bending strength and the distribution of the internal moment has been investigated by Madsen & Buchanan in **Paper 18-6-4**. The used theoretical description was Weibull's weakest link. The estimated values of the shape parameter *k* varied quite a lot. It is expectable to have a variation in *k* between different species and grades because *k* is related to the Coefficient of variation. The authors suggest a *k*-value of 3.5, which should give sufficient precision for engineering purposes for the type of timber in question.

In a paper from IUFRO in Oxford 1980 this dependency between bending strength and M-distribution has been investigated too. Here it is chosen to describe the strength of timber by a probability distribution of cross sections with such defects. The figure below shows the probability distribution for 3 beams with different moment distributions.

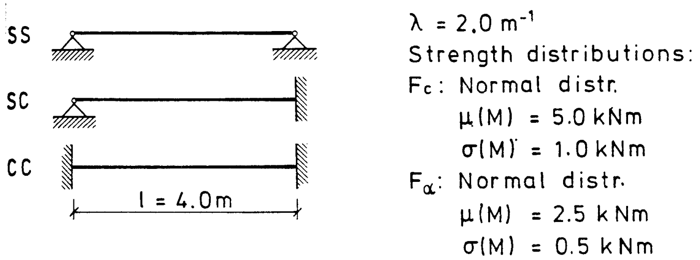


Figure. Timber beams with different supports, simple supported or clamped.

It should be noticed that the difference between the beams which are supported Simple-Clamped and Clamped-Clamped is relatively little, and for practical purposes it can be disregarded.

To illustrate the effect of using different theoretical descriptions, the ratios between the strength at max. moment of a simple-simple and a clamped-clamped beam loaded with a distributed load has been calculated based partly on a Poisson process partly on a Weibull distribution. The two ratios are 1,35 and 1,37 respectively for the 5-percentiles for a coefficient of 0,20, i.e. negligible.

By using the Weibull theory the ratio between the strength of a reference beam and any other beam can be calculated. As reference beam is chosen a simply supported beam with 2 concentrated forces in the third points. The ratios are given in the following table.

Ratios $f_m/f_{m,ref}$ for timber beams with different supports and load configurations.

Support: Simple or Clamped	Load type	Coefficient of variation: 0.10	0.15	0.20	0.30
SS	2 concentrated $a = 1/3\ l$	1.00	1.00	1.00	1.00
SS	Distributed	1.04	0.14	1.03	1.03
CC	2 concentrated $a = 1/3\ l$	1.23	1,30	1.35	1.44
CC	Distributed	1.25	1.35	1.14	1.53

It can be seen that the difference in the ratios for different load types are small as long as the moment distributions are similar. The large difference occurs when the max. moment occurs at a peak, here at the supports. So as a practical simplification it could be acceptable to allow that for strength verification of cross sections at a moment peak the bending strength could be increased by the following factor depending on the Coefficient of Variation (CoV) in per cent.

$$k = 1 + 0,4\frac{CoV}{30}$$

23-14-1 H Riberholt
Analysis of timber trussed rafters of the W type

Two simplified frame models have been investigated. The simplification has resulted in a reduction of the number of nodes and thereby the number of degrees of freedom by approximately half. Only a truss with eccentric joints has been investigated. For centric joints it is very easy to set up a similar simple model, which is in agreement with the more detailed model.

The slip in the joints can be modelled by a prescribed slip.

In the simple model the force transfer by the nail plate at the heel joint has been modelled by a pinned joint positioned at the joint between the timber parts, and the contact stresses between the chords has been modelled by a spring. The eccentric connections have been modelled in detail and at the ridge, there is introduced an eccentricity resulting in a negative moment at the ridge. The slip in the joints have been considered by a prescribed slip in the heel joint, the ridge and the splice plate with the values given in table 3.1, which are those found for a truss with normal stiffness. So the results should be compared with those in section 3.2 for a W-truss with normal joint stiffnesses.

In the extra simplified model the transfer of the internal forces at the heel joint has been modelled more crudely. In the T-joint of the top chord the eccentricity has been disregarded and so has the beneficial effect of the eccentricity in the ridge connection.

For both models analyses have been carried out for pinned joints as shown and for completely stiff joints. Further the slip in the joints have either been disregarded or taken into account by prescribed slips in the selected joints mentioned previously.

A comparison with the results for the detailed model with normal stiffnesses shows that that the agreement in CSI-values is:

Best	Poorer
Pinned joints	Completely stiff joints
Simple model	Extra simple model
Slip in the joints	No slip

Relative deviations in CSI-values obtained by the simple models with Pinned eccentric joints as compared with the detailed model with normal joint stiffnesses.

	Simple model		Extra simple model	
	No slip	Slip	No slip	Slip
Top chord	-2%	-3%	-8%	-9%
Bottom chord	-9%	-5%	-19%	-9%
Diagonals: Compressed	-13%	-20%	-13%	-20%
Tensile	-21%	-23%	-23%	-26%

The deviations given in the table should be acceptable for the simple model when slip in the joints is considered.

Comparison with the Present Danish hand calculation method

The present Danish hand calculation method is presented in **Paper17-14-2.** The distributed loads are transformed to concentrated nodal forces so that the axial forces can be determined from a lattice model, while the moments in the chords can be determined by moment coefficients, here 0.1 in the top chord, 0.1 in the outer bay of the bottom chord and 0.08 in the middle bay of the bottom chord.

The results of this hand calculation method are in table 5.1 compared with those of the models dealt with in this paper. The comparison is carried out only for a W-truss with centric connections, since the hand calculations have this as an assumption. The frame models have been analysed either by normal joint stiffnesses or by prescribed norms slips in the connections.

Table 5.1 Combined Stress Index in some cross-sections calculated by 4 different models/
methods. W-truss with centric connections.

		Hand calculation	Detailed model	Simple model	Extra simple model
Top chord	Low bay	0,99	0,92	0,83	0,74
	Up bay		0,92	0,82	0,87
	Low node	0,99	0,91	0,90	0,87
Bottom chord	K-node		0,63	0,49	0,49
	Mid bay	0,71	0,51	0,66	0,67
Diagonal	Compression	0,27	0,26	0,24	0,24
	Tension	0,35	0,39	0,32	0,32

It appears from table 5.1 that there is a reasonable agreement between the maximum CSI-values for the different timber parts. Further the frame model results are a little conservative compared with the present Danish hand calculation method. They should thus be applicable for practice, and can be employed for an optimal geometric design of the W-truss, which the hand calculation method can't be used to, because it is limited by the geometric assumptions of equal bay length.

Conclusions
For a W-truss it appears that the following conclusions are valid:

A If the rotational stiffness S_θ of a joint fulfils the requirement below, then it is a reasonable simplification to assume that the connection is completely stiff for rotations if

$$S_\theta > 3\frac{EI}{l}$$

where

$\frac{EI}{l}$ is a stiffness measure for the adjacent timber members with length l

B The relative displacements (slip) between the timber parts in the joints can be considered by spring elements or by prescribed slip. The max Combined Stress Index in the timber parts is relatively robust for practical variations in the stiffness of the joints

C Even crude frame models of the truss give Combined Stress Indices similar to those obtained by more detailed models

D The proposal to increase the bending strength of the chords at moment peaks result in a decrease in the CSI-values of typically 5-10%, and it results in truss designs, which have been used successfully over a long period

E The proposed frame models give Combined Stress Indices similar to those obtained by a Danish hand calculation method used for many years. The hand calculation method is a little more conservative.

24-14-1 A Kevarinmäki
Capacity of support areas reinforced with nail plates in trussed rafters

Summary
A low-cost and easy way to reinforce the chord is to use nail plates at the support areas of the trussed rafter. The test results of load-carrying capacity in compression perpendicular to the gain of the support areas reinforced with nail plates are shown in this paper. Although the supporting block is wooden, the bearing capacity will increase at least 30 % with nail plate reinforcement of the chord at an end support because the bearing pressure is rail type in the supporting structure. If the supporting block is steel, concrete or wood in compression parallel to the grain, the nail plate reinforcement will improve the bearing capacity of the end or intermediate support even 2 times higher.

25-14-1 A Kevarinmäki and J. Kangas
Moment anchorage capacity of nail plates in shear tests

Introduction
Eurocode 5 - draft of 1992 includes a design method for the moment anchorage stress (τ_M) of the nail plate joints. The force, anchorage stress τ_F and the moment, anchorage stress τ_M are checked separately, but also the sum of them is checked. This method is used in Norway and it has been presented in **Paper 23-7-1**. In this method the moment anchorage stress τ_M is calculated according to elastic theory, but the plastification is taken into account with a certain coefficient in the failure criteria. Norén has presented a different τ_M design method for the nail plate joints in **Paper 14-7-1**. In Noréns theory stress τ_M is calculated with the plastic theory and only the combination of τ_F and τ_M is checked at the ultimate limit state. In proposed EC 5 method the direction angle of nail plates or the direction of the grain are not taken into account if τ_M is the critical factor. In the method presented by Norén these factors have a clear influence in the failure criteria.

Comparison of these calculation methods with the results of standard shear tests is shown in this paper. The shear tests are the only standard tests of the nail plate joints where a clear combination of τ_F and τ_M occurs. The eccentricities are so big that usually τ_M is the critical factor in anchorage capacity when EC 5 method is used. The results from testing of 11 different nail plates are compared with these two theories. Altogether 220 shear tests where final failure mode was anchorage one have been analyzed and presented in this paper.

25-14-2 J Kangas, A Kevarinmäki
Design values of anchorage strength of nail plate joints by 2-curve method and interpolation

Summary
In this paper a method is presented which makes possible a more effective utilization of the anchorage capacity of nail plates than in Eurocode 5 draft 1991. The curve for design values in Eurocode 5 is taken as a minimum

one, the curve drawn through the standard test results when loaded in grain direction is taken as a maximum one and a method of interpolation between them is proposed. Test results to support the proposal are presented.

Conclusions

Test results of tests indicate, that angle γ has not an effect on the anchorage strength as expected in Eurocode 5. Hence only angles α and β are needed when basic anchorage design values are calculated.

Test results in different combinations of direction angles indicate that also $f_{a\alpha\beta}$ values can be given reliably based on standard test series. Simplest method is to interpolate between the maximum curve, when $\beta = 0°$ and the lowest test value f_{a9090}.

Interpolated values are on the safe side, if f_{a9090} is taken as a minimum value for all cases when $\beta > 45°$ and the values are interpolated linearly between $0° < \beta < 45°$.

26-14-1 E Aasheim
Test of nail plates subjected to moment

Introduction

In the test standard ISO 8969 "Timber structures - Testing of unilateral punched metal plate fasteners and joints", no test specimen for determination of moment resistance is defined. In order to model trusses, frames and other systems in a realistic way, the modelling of the joints are essential. In this respect the rotational stiffness of the contact surface is fundamental.

During CEN/TC 124/WG 1-meetings this subject has been discussed. In the document EN TC 124.116 (June 92-version) "Timber structures - Test methods - Joints made of punched metal plate fasteners" a method is introduced. The test specimen is subjected to a combination of moment and shear force, and the method includes complicated calculations to find the rotational stiffness.

Objectives

The basic objective of this paper is to introduce a test specimen for a "direct" measurement of the rotational stiffness of the contact surface of the plate and the timber.

Conclusion

In general, the results of this study show that the proposed test specimen may be used to find the rotational stiffness of the contact surface. It can be seen from table 3 that the results are in the same order of magnitude for the different test series, and the results are much more reliable than those found by specimens subjected to a combination of shear force and moment.

Concluding remarks

More research and experience is needed to use this rotational-stiffness in practical design, but the introduction of a test piece in EN TC 124.116 is recommended. The gap between the joined parts should be reduced to 5 mm, and the plate size should be chosen to ensure that failure does not occur in the plate.

26-14-2 A Kevarinmäki, J Kangas
Moment anchorage capacity of nail plates

Introduction

The, analysis of the nail plate joints anchorage moment capacity with 220 standard shear test results was presented by the authors in **Paper 25-14-1.** Now 72 new shear tests, where also the angle between grain and force (b) was a variable, have been carried out. New bending tests done in Norway have also been analyzed for comparison. The analysis of these test results, where the failure mode was anchorage failure, has been presented in this paper. The anchorage moment capacities of these specimens have been calculated according to Eurocode, 5 (modified elastic method), with the simplified plastic design method (Norén’s method) and with the accurate plastic theory solution.

Conclusions

This analysis, with the new kinds of shear and bending test pieces, supports the authors’ earlier conclusion, that the plastic theoretical τ_M design corresponds better with the test results and would be a better method than on the elastic theory based method given in ENV-Eurocode 5. The capacities of the shear test pieces were almost identical to the values calculated by the accurate plastic theory. The simplified method presented by Norén is always on the safe side and it gives generally some percents additional safety. Same safety level is obtained also by the new simplified method, which does not have geometrical limitations, presented **Paper 26-14-4**. In pure bending case also the plastic theory was 17 - 33 % conservative (while it was 35 - 54 % by the EC 5 method). According to the bending test results the moment anchorage strength τ_M is higher than the tension anchorage strength in the direction $\alpha = \beta = 0°$ ($f_{a,00}$).

The design is easier and clearer by the simplified plastic theory than by the method of ENV-EC 5. The geometrical values of the elastic method (I_P and are r_{max} are more difficult to determine than the diagonal d needed for the plastic design. There are three design criteria that must be checked in the anchorage design of EC 5. In plastic design method only one design criteria is required:

$$\left(\tau_F / f_{a,\alpha\beta,d}\right)^2 + \left(\tau_M / f_{a,00,d}\right)^2 \le 1 .$$

This equation also has the same form as many other design formulas of Eurocode 5.

26-14-3 A Kevarinmäki, J Kangas
Rotational stiffness of nail plates in moment anchorage

Introduction

Application of moment resisting rigid or semi-rigid nail plate joints in timber structures is missing at present. Including rotational stiffness and moment capacity into nail plate joint design opens following possibilities:
- it will lead to material savings and more economical nail plate structures.
- it will give more accurate and safer design in ordinary structures compared to pin joint theory.
- it enables the design of new kinds of structures, which will increase the competitiveness of the timber structures.

This kind of exact theory and design method is missing at present. In Finland the method where the moment rigidity of nail plate joints has been included is already in use, but it has simplified assumptions of material properties, which are on the safe side. ENV-Eurocode 5 is very general and insufficient. Concerning these matters measuring and determination of the nail plate joints rotational stiffness is not presented in any standard. That is the main obstacle in utilization of the nail plate joint rotational stiffness. The aim of this study was to develop a proposal for standard tests of nail plates rotational stiffness and a method for the determination of the rotational spring stiffness moduli.

In this paper the rotational stiffness of shear and bending test results of 102 nail plate joints have been analysed. The deformations between timber and the nail plates in each shear test were measured by 10 gauges. The rotational spring stiffness moduli have been calculated from these measurements. Rotational stiffnesses have been determined also from the results of Norwegian bending tests. This study showed following:
- the effect of the testing type,
- the effect of direction angles a and
- the dependence on the shape of effective nail plate areas and
- the effect of the measuring point locations to the calculated results of rotational stiffness.

Based on this research a proposal for the standard test of nail plates rotational stiffness and a method for the determination of the rotational spring stiffness modulus $K_{\varphi,ser}$ from the test results are given.

26-14-4 A Kevarinmäki
Solution of plastic moment anchorage stress in nail plates

Introduction

In Eurocode 5 the moment anchorage stress τ_M of nail plates are calculated according to the elastic theory. In the design criteria there are the factors (1.5 and 2) so the capacity is near to the plastic theoretical solution. The analysis of over 300 test pieces by the plastic theory calculation of τ_M -stresses showed that it gives much better results than the method of Eurocode 5 (**Paper 25-14-1** and **Paper 26-14-2)**. B. Norén has presented a simplified solution of the plastic theory for calculation of the plastic τ_M -stresses (**Paper 14-7-1**). The calculations of the geometrical values by the simplified plastic theory are easy compared to the elastic method. Also the design criteria become simpler by plastic theory, only one equation is needed. In Eurocode 5 three criteria's must be checked separately in the anchorage design.

The questions about the theoretical background, about differences between Norén's simplification and the exact plastic theory and about the application of Norén's method for the different shapes of effective nail plate area have been presented. This paper deals with these problems. Accurate plastic theoretical solutions are calculated by numerical integration with a computer. Differences between the accurate plastic theory and Norén's method are determined. Another alternative approximate solution for the determination of the plastic τ_M -stress has also been developed. This new method is very simple and easy to use. The method is at the same safety level as Norén's method, but all shapes of effective nail plate area can be solved by it. Norén's method is limited only to the rectangles, the right-angled triangles or the quadrilaterals where two angles are 90 degrees.

Conclusions

The comparison between Norén's simplification and the accurate plastic theory shows that Norén's method is always on the safe side in determining the plastic τ_M anchorage stress of nail plate joint areas. With the rectangle shapes of the effective nail plate areas Norén's method is 0-8 % conservative. The biggest difference (28 %) from the theoretical solution occurs in the case of right-angled triangles where the sides are equal. Difference between the theoretically and approximately calculated 'ENLPI stresses seems to be relatively high in some cases. The effect to the design result however is quite small as the cm stress is only one part in the anchorage design criteria. Usually the τ_F stress has a bigger effect on the anchorage design.

Due to Norén's methods limitations, the new alternative approximate solution is presented. The nail plate surface may be replaced by the rectangle that has the same area and height perpendicular to the longest side as the original surface has. The plastic anchorage stress τ_M may then be calculated for this fictional rectangle. This method is at the same safety level as Norén's method and it is easy to use. The new method and its formulation suit very well together with the rules of the plastic τ_M anchorage design to be included in design codes.

26-14-5 R Gupta
Testing of metal-plate-connected wood-truss joints

Abstract

A testing apparatus with computerized data acquisition and control system was developed to test metal-plate-connected) wood-truss joints. The apparatus enabled testing of several different types of joints under various loading conditions without major modifications. The apparatus shows potential as an efficient testing procedure to assess joint behavior.

26-14-6 C J Mettem, J P Marcroft
Simulated accidental events on a trussed rafter roofed building

Introduction

As discussed in previous papers (**Paper 22-15-6** and **Paper 23-15-6)**, work has been in hand for several years to address the issues arising from the ideal that timber structures should be planned and designed in such a way that they are not unreasonably susceptible to the effects of accidents. In 1989, the preliminary **Paper 22-15-6** was submitted, stating the problems in general terms, and suggesting that they are of concern to designers irrespective of whether timber or other materials are concerned, and also fundamentally irrespective of whether national or European-wide codes are to be followed. Potential disadvantages for timber were foreseen, if the fulfilment of such principles were to be deemed to be satisfied in certain ways. Naturally, this was of greatest concern to the authors with respect to the Approved Documents to the Building Regulations, which are published by the Department of the Environment in the UK, and which specifically issue guidance on "disproportionate collapse" in this context. However, it was realised that there were wider implications and that the subject is of more general interest.

The second CIB paper on the subject, presented in 1990, elaborated upon the way in which regulatory, requirements were developing. It also described, with reference to a specific form of roof construction, ways in which the "disproportionate collapse" requirement might be interpreted in terms of the acceptable extent of damage, after an accident. The form of timber roof construction evaluated in this paper was parallel framed construction, where distinct principal beams, plus purlins and bracing existed. Two requirements were brought to the attention of the designer. Firstly, that of ensuring that in the event of a local failure of a primary frame, then the adjacent primary frames would not be dragged down. Secondly, the need to ensure that the undamaged areas on either side of the damaged zone would retain sufficient stability to remain standing.

Fortunately, and possibly in part as a consequence of this work, the more recent editions of the Building Regulations Approved Documents have included revised guidance on the acceptable extent of collapse in the event of a local failure in a roof structure or its supports, for low rise public buildings. This guidance can be met with reasonable economy by well-designed versions of existing timber engineered types of building and detailing.

The ENV version of EC5 has also adopted the principle of requiring design against the eventuality of collapse.

One of the fundamental requirements of the basis of design given in EC5 states:

A structure shall also be designed in such a way that it will not be damaged by events like explosions, impacts or consequences of human errors, to an extent disproportionate to the original cause. "

However, in spite of the prominence of this principle, little or no detailed guidance is provided.

Whilst the work by TRADA Technology Limited in earlier years of the project focused upon the parallel framed structures mentioned above, it was evident that, for reasons of commercial economics, it would be important to include consideration of trussed rafter roofs. These occupy such a large proportion of the market for non-domestic roofs in the 9 to 15 metres range, that the project would have been incomplete if no account had been taken of them. It was felt necessary to address the assessment of the reaction of trussed rafter roofs to accidental events by experimental means rather than analytically, as explained below.

Concluding remarks

A full-scale testing programme was conducted upon a trussed rafter roofed building, to assess its response to simulated accidental events. The test building which was constructed represented a typical non-domestic trussed rafter roofed low rise structure, of moderate span, using masonry walls. It was roofed in a conventional manner, with trussed rafters of 10 metres span at 600mm centres. A number of variations of detail in the secondary roof construction were incorporated in the building, and for each of these, a simulated "structural accident" was replicated. Standard bracing, and in some cases additional bracing, was included.

In the case of public buildings of the type which the test structure was intended to represent, there is a distinct requirement in the Building Regulations (England and Wales) to construct in a manner such that, in the event of a failure of the roof or its supports, the building will not collapse to an extent disproportionate to the cause. Since there is also a general requirement by EC5 to design in such a manner as to avoid disproportionate damage, then the results of this project are of further and more universal relevance.

The type of accident envisaged in the simulations was the removal of a length of external load bearing masonry wall, through a vehicle impact.

As described above, the main series of tests involved the sudden removal of a 3.6 metre length of support wall, beneath the trussed rafter roof, with the ensuing gap appearing at the rnidlength of a longitudinal external load bearing masonry wall. Four tests were carried out under these conditions, each with a progressively less robust degree of secondary and longitudinal construction incorporated in the roof detailing. However, under none of the construction detailing cases covered by these four simulations did any of the major parts of the roof structure collapse to the stage of reaching the ground. This was found to be the case both for the simulated suspended ceiling arrangements and for a plasterboard ceiling. Even with interrupted wall plates, simulating unconnected butt joints over the gap, and with parts of the obligatory roof bracing removed, the roof framework remained stable, albeit with large deflections.

The four secondary components found to contribute most substantially to the robustness of the roof in resisting collapse were as follows:

1. The wall plates: Clearly their contribution depended greatly upon the relative positions of the wall plate end joints and the removed length of support wall, but if the wall plates were to remain spanning the gap after the "accident", then they would seem to make a substantial contribution to the support of the affected length of roof.
2. The "chevron" bracing: It was found that the two lines of chevron bracing made a significant contribution to the longitudinal structural rigidity of the roof, despite their incomplete triangulation. Ultimately, their capacity was limited by the performance of their nailed end joints and, with better fixings it would be possible to extend this bracing concept to obtain even greater resistance, if required.
3. The longitudinal bracings: The top and bottom chord longitudinal bracings, acting in conjunction with the other elements of the roof framework, also contributed in a surprising degree to the support of the damaged framework.
4. Other wind bracing components: The flat longitudinal wind girders, and the plasterboard ceiling, where present, also contributed in a more minor degree to the stiffness of the secondary construction.

By deduction, it was concluded that the "behaviour largely unknown" elements, particularly, the tiles, felt and battens, were contributing significantly to the ability of the damaged structure to resist collapse. This is discussed further below.

The top chord roof cladding, consisting of interlocking tiles, was found to significantly increase the capacity of the timber secondary components to transfer loads from the unsupported zone of roof onto those trussed rafters still supported at both ends. This increase in resistance was between 20% and 60 Fr, depending upon which secondary timber components were present during the associated accidental event simulation. The longitudinal flexural rigidity of the tiles and underlying felt and battens was found to be broadly comparable to that of a diaphragm of continuous 20 mm thick GS grade timber boarding laid parallel to the ridge.

Following the main series of four simulations with progressively less material incorporated in the secondary roof construction, the test building remained essentially intact and, with small repairs, further testing was possible.

Additional tests were conducted to assess:

1. The resistance to collapse of the roof, with another "accidental" gap created at mid-wall length, but with the structural contribution of the tiles removed completely (their weight being simulated).
2. The behaviour of the structure when a gap was created by the sudden removal of a length of support wall close to a gable end.
3. Finally, the resistance to collapse when portions of the gable masonry itself were removed.

The roof did not collapse at all under the first condition described above. It was able to span a 3.6 metre void in the length of the support wall with structural resistance to collapse, provided in the longitudinal direction, only by means of tiling battens and longitudinal and chevron bracing.

The final phases of testing showed most dramatically the robustness of this trussed rafter roof construction in resisting such simulated accidents. The roof showed a capacity to span a void in the support wall, close to the gable end, of over 4.3 metres. When collapse of the roof was eventually provoked, by removal of the entire half of a masonry gable wall to the side where the truss heels stood, plus the remaining gap in the longitudinal wall, then no part of the roof beyond the unsupported areas collapsed to the ground, and the damage was still considered proportionate to the original cause of the failure. Horizontal deflections of the adjacent, undamaged roof were very small in terms of such a dramatic incident.

The general conclusions are therefore that braced trussed rafter roofs of this form have been demonstrated to be very robust in the face of accidental events of the nature likely to be caused by vehicle impacts and similar events. Large areas of roof support can apparently be removed without collapse occurring which is disproportionate to the cause of the foreseen structural damage. Such roofs are likely to satisfy the very specific requirements of the UK Building Regulations for avoidance of disproportionate collapse in low rise public buildings of 9-metre span or greater. The evidence of these tests is also of a more universal relevance, in view of the principle of EC5 that disproportionate damage should be avoided in design.

30-14-1 R J Bainbridge, C J Mettern, A Reffold, T Studer
The stability behaviour of timber trussed rafter roofs - studies based on Eurocode 5 and full scale testing

Abstract

This paper describes recent research into the stability behaviour of timber trussed rafter roofs of non-domestic scale, in the light of Eurocode 5 and the latest edition of BS5268. Testing of a full scale non-domestic timber trussed rafter test roof of 11 m span is also detailed, employing diagonal bracing at rafter level to provide stability.

The comparative effects of different bracing angles are presented in terms of their influence upon the overall stiffness of the structure. Investigation is also made into the effects of variations in the fixity of the bracing members at wall plate locations.

The findings are also compared with theoretical behaviour based on a simplified three dimensional structural model and hand calculation.

Conclusions

The results of these tests have shown that the roof structure with only longitudinal binders and tile battens in place offered only minimal resistance to applied lateral loads. The introduction of even a single diagonal bracing member in a roof of this type has a large influence on the stability and lateral stiffness.

The effectiveness of the bracing is affected by the angle of slope in the brace and the robustness of connection between brace end and wall plate connections. It has been found that the relative effectiveness of the bracing can be approximated to a reasonable degree by the cosine of the angle of inclination of the bracing. Improvements in the robustness of connection details at the wall plate end of bracing provide significant enhancement to the overall roof stiffness, reducing measured roof deflections by approximately 30-40 %, dependent on angle of inclination of bracing.

It is believed that although shallow angled bracing has been shown to be structurally more efficient than steep angled bracing, 45° is still a preferable target angle. Designers should consider aspects such as the ease of installation and effects on the erection process before varying from this 'standard' detail.

32-14-1 J Nielsen
Analysis of timber reinforced with punched metal plate fasteners

Introduction

The controlling sections of chords in a truss are often influenced by a moment peak, see figure 1. At triangular trusses the maximum peak moment and maximum axial compression force in the top chord often appear at the heel joint.

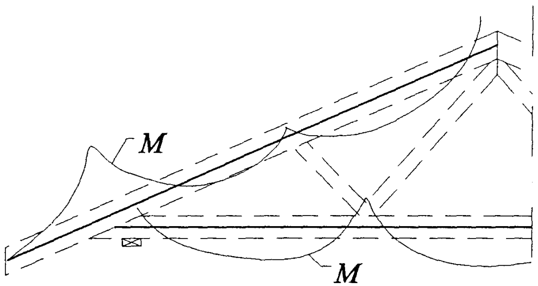


Figure 1: Variation of the moment, M, in the chords of a W-truss.

In order to obtain optimal truss designs different statistical analyses of the wood strength parameters have been made to obtain increased capacity of the sections with moment peak. Another method of increasing the moment capacity is to reinforce the dimension controlling areas with a punched metal plate of same type as used in the joints of the truss, see figure 2. An advantage of this method is that the price of the reinforcement material and the production of the reinforcement are very limited compared to the total costs of a truss. By introducing the reinforcement it may be possible to reduce the cross-sectional depths of the chords.

The reinforcing effect of punched metal plate fasteners in areas with a moment peak is analysed by tests. According to DS/EN 408 the bending capacity (bending strength) of timber shall be determined for beams loaded in 4-point bending. However, the present work is limited to analysis of the reinforcing effect in areas with moment peaks only. Therefore, the test specimens are made with beams loaded in 3-point bending, as shown in figure 3.

The tests and the test results are described in this paper. The failure types are given and some load-displacement curves are shown. Further, the tests are compared to a numerical model.

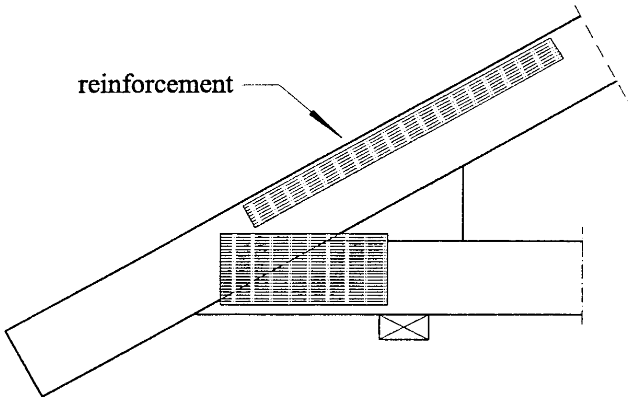


Figure 2: Heel joint with a wedge and a reinforcement.

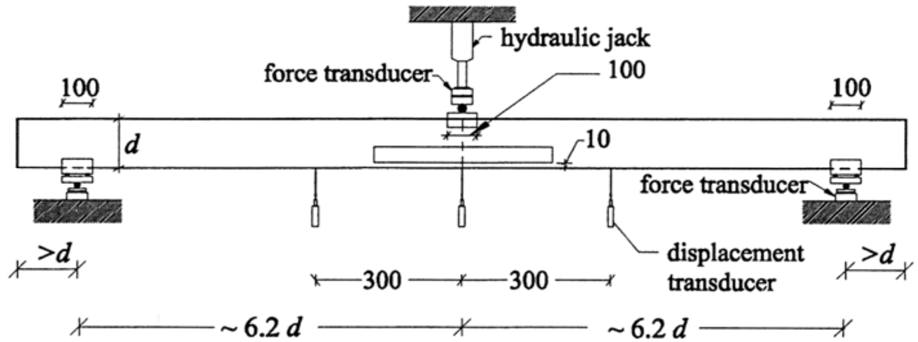


Figure 3: Test arrangement for beams in 3-point bending. Dimensions in mm.

Conclusion

Based on the tests the following conclusions can be made:

– The failure load and the stiffness of 120 mm beams with reinforcement are increased by 20 to 30%.
– The reinforcement has no effect on 170 mm beams. In further analyses it is recommended to:
– use a plate with high tensile capacity (thickness > 1 mm) in beams with a cross-sectional depth of 170mm.

– locate the reinforcement closer to the tensile edge of the timber (< 10 mm).
– limit the width of the reinforcement (< 40 - 50 mm)
– test beams with other cross-section depths and load conditions.

33-14-1 J Nielsen
Moment capacity of timber beams loaded in four-point bending and reinforced with punched metal plate fasteners

Abstract
Tests on timber beams reinforced with punched metal fasteners are continued. The reinforced beams are loaded in four-point bending. The timber is Swedish spruce with cross sectional area of 45x 120 mm and 45x 170 mm. The reinforcement is from Gang-Nail Systems, types GNA20S and GNT150S. Beams with lateral reinforcement and beams with reinforcement located at the short edge are tested. The tests and the results are described.

Conclusion
Based on the tests the following conclusions can be made:
– The reinforcement increases the strength and stiffness of 120 mm beams in strength class K18 by 20%.
– The reinforcement has no effect on the strength of 120 mm in strength class K24. However, the stiffness increases by 5-20%.
– The reinforcement has no effect on the strength of the 170 mm beams. However, the stiffness increases by 20% for 170 mm beams in strength class K18.

In further analysis is it recommended to use reinforcing plates with a less teeth density. This will increase the strength and stiffness of the reinforcement and it may also reduce the embedding difficulties as the needed embedding force is less.

36-14-1 P Ellegaard
Effect of chord splice joints on force distribution in trusses with punched metal plate fasteners

Abstract
Splice joints in timber trusses with punched metal plate fasteners (nail plates) may be assumed rotationally stiff if their deformation has no significant effect upon the distribution of member forces according to Eurocode 5: Design of Timber Structures.

Two simple guidelines for the design and location of splice joints are given in Eurocode 5 for treating the splice joints rotationally stiff. The reasonability of these guidelines is discussed in this paper.

A finite element program TrussLab where the splice joints are given semi-rigid properties is used to analyse two triangulated trusses with three and four splice joints, respectively. The influence of the splice joints on section forces, rotations in the splice joints and global displacements of the trusses is analysed.

Based on the results of the calculations it seems that the guidelines for treating splice joints rotationally stiff do not necessarily lead to more realistic truss models.

Conclusion
As could be expected model S2 (splice joint near maximum moment and nail plates utilised up to 66%) leads to smaller rotations in the splice joints and smaller global displacements compared with model S1 (splice joint near maximum moment and nail plates utilised up to 100%). Due to the larger stiffness of the splice joints in model S2 the bending moments in the splice joints are also larger than for model S 1.

However, when the splice joints are located closer to sections with zero moment (S3) the rotations and displacements are often larger compared with model S 1. Moreover, it is not clear whether the influence on the bending moment distribution is larger for model S I than for S3.

Therefore, it seems that the guidelines (especially the second one in Eurocode 5) for treating splice joints rotationally stiff do not necessarily lead to more realistic truss models compared with a situation with no limitations to the splice joints.

There are relatively large deviations between the results (moment distribution, rotations and displacements) achieved with the models S1-S3 compared with S4. To get a better agreement between real behaviour of a truss and the finite element models semi-rigidity of joints should be implemented. These joint models must include timber contact, since it is important to the stiffness and distribution of the section forces.

From a simple splice joint it is found that the rotations are mainly caused by the anchorage of the nails and not by deformations in the joint line (for GNT150S plate type).

36-14-2 M Hansson, P Ellegaard
Monte Carlo Simulation and reliability analysis of roof trusses with punched metal plate fasteners

Abstract
System effects are often found in timber structural systems e.g. due to transverse load distribution between different structural members. System effects are also related to the variation of strength within and between members. Most studies found in the literature have considered the variability between timber members, but the variability within members has been neglected mainly because of lack of data.

In this paper, Monte Carlo simulations of a W-truss are performed using a model describing the strength parameters between and within timber members. The timber members are connected by punched metal plate fasteners (nail plates). The variations in the properties of these joints have been estimated from experiments. The FE calculations are performed with TrussLab - a toolbox to MATLAB developed at Aalborg University. TrussLab takes contact between members and non-linear behaviour in the joints into account.

The system effect is estimated through a comparison of the simulations with a deterministic calculation of the roof truss using characteristic values as input to the model. The change of coefficient of variation also affects the reliability and is analysed. The total system effect is found to be between 10-20 % when timber failure is predominant and less than 10% if the total failure is caused by failure in the plates or anchorage failure in the connection between nail plate and timber member.

Summary
In the simulations, the only failure that was observed was failure in the nail plates for this specific roof truss. Therefore, additional simulations were performed ignoring failure in the nail plates. The system effect was estimated through a comparison of the simulations with a deterministic calculation of the roof truss using characteristic values as input to the model. The change of coefficient of variation also affects the reliability and was analysed. The total system effect was found to be between 10-20 % when timber failure is predominant, but less than 10% if the total failure was caused by anchorage failure in the connection between nail plate and timber member.
A more detailed study of the commercial software is desirable to determine if the joint models are accurate and if the engineering practice to increase bending strength at moment peaks is relevant.

36-14-3 R H Leicester, J Goldfinch, P Paevere, G C Foliente
Truss trouble

Abstract
Following many decades of extensive and highly successful applications of nail plated trusses in Australia, several unexpected incidents involving plated trusses occurred recently which have caused some concern and triggered investigations into the performance of these trusses. Among matters investigated have been the mechano-sorptive behaviour and structural mechanics of heel joints, creep buckling, brittle wood and quality control in fabrication and erection.

Introduction
Since the 1960s, nail plated timber roof trusses have been used extensively and successfully in Australia for housing, community halls and commercial buildings. However in 2002 the double girder truss of a golf clubhouse in Adelaide collapsed killing two persons and were it not for the fortunate placement of a side wall, would have killed another 30 people.

The collapse occurred after 7 years in service, without warning and on a quiet windless day. The only loading at the time was the dead load of the roof. Following an investigation on this collapse, the South Australian government issued a general hazard alert related to work place buildings using nail plate truss roofs. As a result a program to inspect the service condition of roof trusses was initiated. It was discovered that many existing trusses are either close to collapse, or appear to carry a risk of a collapse in the foreseeable future through teeth withdrawal due to mechano-sorptive and other effects.

Since the "problem" trusses follow conventional and apparently successful design forms, it is important to assess if there are potential future problems. The findings of some of the investigations undertaken for this purpose are discussed in the following.

Recommendations

On the basis of the investigations described herein the following recommendations can be made for the design and fabrication of nail plated timber trusses that are intended for use as critical structures:

– The strength of heel joints should be assessed by direct laboratory tests to ensure that the correct failure mode is considered.
– For girder trusses loaded on the tension chord, a formal engineering analysis for the design of the system to laterally restrain the top chord should be undertaken.
– Special quality control checks should be undertaken to ensure that local slope-of-grain, brittle heart and gum veins are not present at connector plate locations.
– Heel joints should be designed so that the dead load in service does not exceed 0.15 of the short duration ultimate strength of the joint.

40-14-1 H J Blass
Timber trusses with punched metal plate fasteners - design for transport and erection

Introduction

During transport and erection of trusses with punched metal plate fasteners unintentional forces and moments may occur which are difficult to quantify but increase with increasing truss dimensions. Hence a careful handling during transport and erection becomes ever more important with increasing truss length.

Eurocode 5 stipulates in 9.2.1 (8):

> All joints should be capable of transferring a force $F_{r,d}$ acting in any direction within the plane of the truss. $F_{r,d}$ should be assumed to be of short-term duration, acting on timber in service class 2, with the value:
>
> $F_{\mathrm{r,d}} = 1{,}0 + 0{,}1L$
>
> where $F_{r,d}$ is in kN and L is the overall length of the truss, in m.

However, this requirement does not take into account unintentional forces perpendicular to the truss plane. A design of members and connections for larger minimum forces leads to more robust members capable of better withstanding the exposure during transport and erection. This paper presents the background of a design method of trusses with punched metal plate fasteners for loads during transport and erection which is part of the German timber design code DIN 1052.

Summary

This paper presents a design proposal for the design situation transport and erection of trusses with punched metal plate fasteners. Although Eurocode 5 contains minimum loads within the truss plane for connections in trusses which are independent of the loads for the permanent design situation, these minimum loads are considered insufficient especially for larger span trusses. During transport and erection, unintentional loads occur, leading to bending moments and shear forces also perpendicular to the truss plane predominantly in the truss chord members.

The background of the proposed design is a situation, where a truss lying on a horizontal basis is lifted at the apex. A simplified analysis of the chords as continuous beams on three supports leads to bending moments and shear forces perpendicular to the truss plane to be considered in truss design. Apart from significantly larger minimum loads for the truss connections in the upper and lower chords, a minimum timber member width results from the design proposal.

Since the plate and anchorage capacities in Eurocode 5 do not provide load-carrying capacities for loads perpendicular to the truss plane, a test method is shown for providing the necessary capacities. While the bending moment from loads perpendicular to the truss plane only cause stresses parallel to the punched metal plate fasteners and hence may be considered in design with the capacities listed in Eurocode 5, the shear forces perpendicular to the truss plane necessitate a new resistance property fax of punched metal plate fasteners which is to be determined by tests for each type of fastener.

For the design of the connections between diagonals and chords, the design rules in Eurocode 5 are considered sufficient.