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Design of Timber Structures by Testing

– A guide

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Abstract

This report presents guidelines for determining the structural properties of timber structures by testing. This method has hitherto primarily been used in connection with wooden I-beam joists, nail plate joints and shear walls.

Design by testing is applicable, for example, when no relevant calculation model is available or when existing models are inadequate.

An important point when testing timber structures is to ensure that the test objects are representative of what can and will be produced in the future. Random sampling of timber for the product to be tested is not normally feasible. The report describes one method, involving the use of machine stress grading, and where it is primarily the low-strength pieces of timber in any given timber strength class that are selected as the test specimens.

The report also demonstrates how a mathematical model can be used in connection with the evaluation of test results. Using this method, it is possible to apply the test results to structures that differ somewhat from those tested.

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Preface

This project has been performed jointly by the Norwegian Wood Technology Institute (NTI), the Finnish Technical Research Centre (VTT) and the Swedish National Testing and Research Institute (SP).

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Summary

Design by testing is practised to only a limited extent, even on a world-wide scale. Products for which this is often done are:

* different types of lightweight beams (I-beams and box beams of wood and wood-based materials);
* nail plate joints, and
* shear walls.

Design by testing is employed mainly when no appropriate calculation model is available or when non-standard material grades are to be used. Testing is then performed either on parts from the production runs or on prototypes of a particular design.

When designing timber structures by testing it is difficult, without some form of control and selection, to ensure that the performance of the structure is not overestimated. Random selection of timber for the structure to be tested is generally not realistic. It is appropriate to ensure that selection of the timber is on the safe side, and so a suitable method is to employ machine strength grading and to select only the timber at the bottom end of the class range of interest.

When evaluating the results, it is important to employ some form of model in order to enable them to be corrected and to allow them to be applied to structures that differ somewhat from those tested. This is described in the report in connection with testing of I-beams having flanges of structural timber and webs of wood fibre board.

A comparison has been carried out for the I-beams between design by calculations in accordance with Nordic timber design codes and EC5 design by testing. The results indicate the following in respect of the moment capacity:

* When designing in accordance with the EC5 or EC1 rules, design by testing results in about 5 % higher values.
* Designing in accordance with the Nordic system, design by testing gives results about 30 % higher.

1 Introduction

1.1 Background

Design by testing is an alternative to design by calculation, and is relevant in the following cases:

- Where there is no appropriate method of calculation.
- Where the design method is insufficiently reliable or is inadequate.
- Where there are no standardised material grades.
- Where design by calculation results in non-cost-effective designs through the incorporation of high safety levels.

Existing rules for design by testing are normally expressed in relatively general terms and give only very limited guidance in respect of timber structures. The Swedish handbook, *Hållfasthetsdimensionering genom provning* (Design by testing) [13], is an example of this. The Norwegian NS 3470 [7], on the other hand, is specific for timber, but provides only brief instructions for determination of structural properties by testing. To these can be added product-specific rules such as ASTM D 5055-90 [14], but these relate only to prefabricated I-beams.

NZS 3603 [15] and BS 5268 [16] describe the testing of timber structures, but are concerned only with proof loading, i.e. determining whether a structure is capable of carrying its design loads.

A questionnaire to a number of countries in Europe, North America and Australia showed that relatively little use is made of design by testing. I-beams, nail plate joints and shear walls seem to be the products to which the method is most commonly applied.

Eurocode 1, *Basis of Design* [4], sets out fundamental conditions for design by testing, linked to the entire Eurocode system. Although it describes in detail how test results are to be evaluated, it deals in only general terms with other conditions such as the design of the test object, how loading is to be applied etc. As a result, the rules in Eurocode 1 are not really suitable for practical work but should be regarded, if anything, as pointers for further standardisation work. Such work has started within RILEM[1], where a committee (TC 125) has been set up to produce detailed rules for determination of structural properties by testing. A draft [26] has been produced, containing an introduction with definitions and other information. In addition, it describes general principles in respect of planning tests, performing them and evaluating the test results. Applications for concrete, steel, timber and other structures will also be included. Both the RILEM work and the rules in Eurocode 1 are based on a report from JCSS[2], under the name of *Estimation of Structural Properties by Testing for Use in Limit State Design* [17].

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1 International Union of Testing and Research Laboratories for Materials and Structures

2 Joint Committee on Structural Safety

1.2 Specific notes on the testing of timber structures

1.2.1 Often very large differences in results between testing and calculation

Lightweight timber beams can be used as an example to explain why testing and calculation can give very different results when testing timber structures. Timber I-beams and box beams were introduced in the 1970s and have to a large extent been designed by testing. Manufacturers found at an early stage that design by testing had significant benefits. The short-term load capacity values obtained by testing were often twice as high as those obtained by calculation, for which there are several reasons.

One important reason is that the design model employed did not describe the behaviour of the structure correctly. The 1980 Swedish Building Code [27] accepted design of the flanges in accordance with the following interaction formula:

$$\frac{\sigma_n}{\sigma_{na}} + \frac{\sigma_{bx}}{\sigma_{bxa}} + \frac{\sigma_{by}}{\sigma_{bya}} \leq 1 \qquad (1.1)$$

where the index n represents the normal stress, a the permissible stress and b the bending stress.

Eurocode 5 [10] employs other design criteria, namely:

$$\sigma_{f,c,\max,d} \leq f_{m,d} \qquad (1.2)$$

$$\sigma_{f,t,\max,d} \leq f_{m,d} \qquad (1.3)$$

$$\sigma_{f,c,d} \leq k_c \cdot f_{c,0,d} \qquad (1.4)$$

$$\sigma_{f,t,d} \leq f_{t,0,d} \qquad (1.5)$$

Figure 1.1 illustrates the terms.

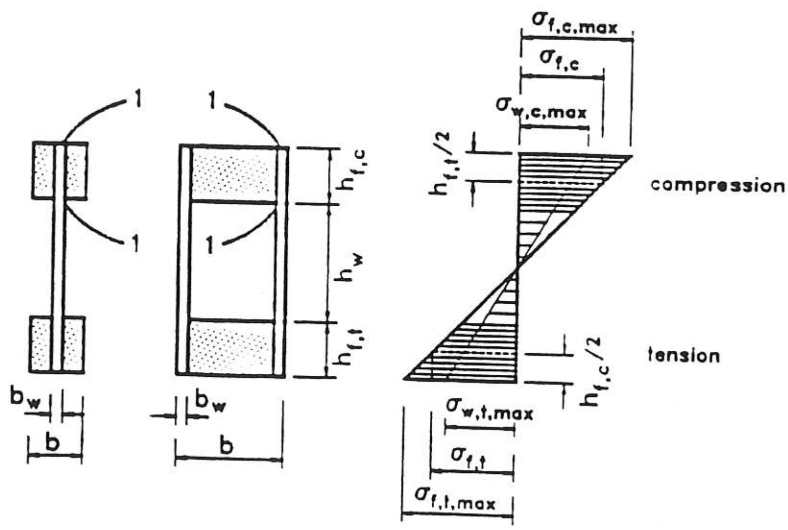

Figure 1.1 Stresses in I-beams and box beams

These criteria agree better with the results of tests: this is illustrated in more detail in Chapter 4. The reason for this is primarily that EC5 allows for the stiffening effect of the web on the flanges, which is not the case in Equation 1.1, where the flanges are regarded as struts in compression and in tension.

There are, however, at least two further factors that result in testing and calculation giving different results.

1. *The natural variations in the strength of the timber*

Calculation bases the design on characteristic values for the various materials. These values have been obtained using a standardised procedure which, in the Nordic and other countries, has involved testing in the section of the timber piece that is regarded as being the weakest. The difference in strength along a piece of timber can be considerable.

Calculation assumes that the weakest section of the timber is placed where the stress is greatest. However, this is seldom the case in structures under test unless special measures are taken. On the contrary, it can be the strongest part of the timber that is in the most stressed section.

2. *Timber quality*

Calculation uses characteristic values for the material classes used in the structure. In the case of K24 timber, the bending strength is 24 MPa, while for K30 timber it is 30 MPa.

The minimum bending strength of the timber in structures containing K24 grade material can vary from 24 MPa to over 80 MPa. If no special measures are taken, the timber in these structures can in reality very well have a characteristic strength of over 35 MPa. This can result in a difference of almost 70 % between calculation and testing.

Figures 1.2 and 1.3 further illustrate how the bending strength can vary for timber of nominally one and the same quality.

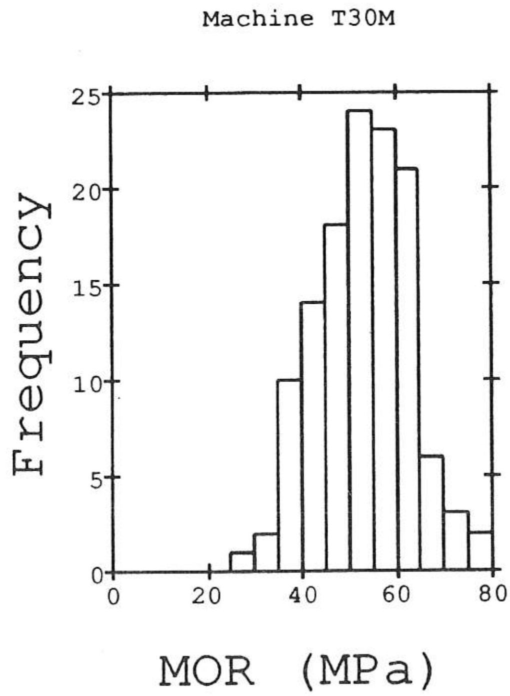

Figure 1.2 Bending strength of K30 spruce timber with a cross-section of 58 x 120 mm [18].

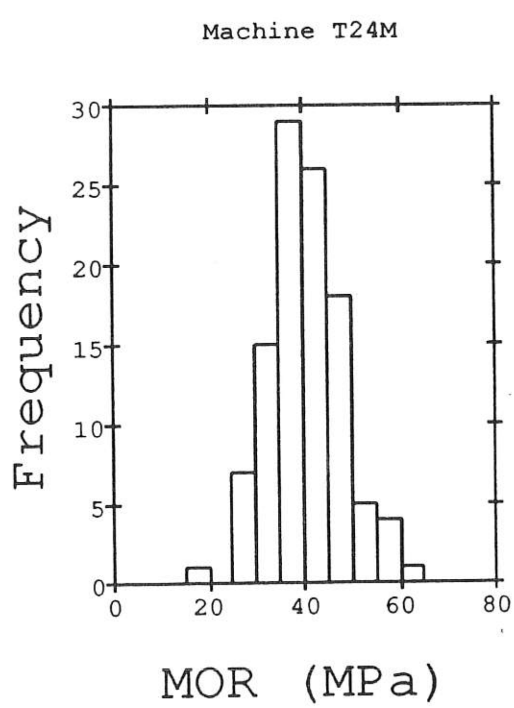

Figure 1.3 Bending strength of K24 spruce timber with a cross-section of 58 x 120 mm [18].

1.2.2 Materials and joints having different long-term characteristics

A feature of wood, joints in wood and wood-based materials is that their strength declines with the time under load. In addition, significant creep can occur.

This is accommodated in calculation by the incorporation of conversion factors based on the material, moisture conditions and load duration.

It is not clear how the various long-term characteristics of the materials and joints should be dealt with in design by testing, and this will probably be a difficult problem to resolve. As an example, we can consider a structure consisting of timber and steel joint materials. In the case of the structural timber, the long-term strength is 60-75 % of its short-term strength, which is allowed for in design by calculation by a conversion factor. Steel, however, suffers no such deterioration in its strength. Strictly, this structure has the same resistance to failure both in the timber and in the steel only if the load acts for 50 years. However, during actual testing, the failure will probably occur in the steel. For determination of structural properties of timber structures with steel joints by testing, one solution to this problem can be to reinforce the steel.

In a nailed joint (using long, thin nails), it can be approximately assumed that the failure strength $f_{nailjoint} \sim \sqrt{f_{wood}}$, where f_{wood} = the strength of the wood. If, after long-term loading, f_{wood} falls to $0.6 \cdot f_{wood}$, the strength of the joint falls by 23 % to $0.77 \cdot f_{nailjoint}$. If the strength of the nail joint $f_{nailjoint}$ is increased by about 20 % over the minimum necessary value, the structure can be regarded as uniformly strong for the purposes of short-term testing.

It must be emphasised that such reinforcement of steel or nailed joints is to be regarded as an extraordinary measure which must be employed only in connection with testing.

Creep is another long-term effect in wood and mechanical wood joints. It results in a relaxation on heavily stressed parts and an increase of the stresses in other parts, leading to some redistribution of forces and moments. Further redistribution can arise if parts of the timber having a lower modulus of elasticity are in the stressed section. As a result, the conditions in those sections indicated by calculation as being heavily stressed become somewhat more favourable. A lower modulus of elasticity is often associated with a lower strength, but the relaxation of the load can mean that there is still adequate safety against long-term failure despite the low strength. From a test point of view, this can result in short-term testing not leading to failure in the same position as would occur during long-term loading. It is not possible to give any general indication of the magnitude of this effect or whether it has any practical importance and can therefore be considered in calculations.

1.3 Objective

The objective of this work has been to formulate a proposal for design by testing rules that are specific for timber. It has also included improving the status of design by testing as a serious approach by means of a formalised, relevant system of rules. However, it has been found difficult during the work to produce detailed rules for testing products of which the structure and intended application are not known.

The result of the project has been a guide, in which determining the necessary sizing and strength of I-beams has been used to illustrate the method of working.

1.4 Scope

The project has comprised the following elements:

- Contacts with institutions in other countries throughout the world in order to determine the extent to which determination of structural properties of wood structures by testing is used and what rules are applied.

- Gathering the factors that are important to consider in determination of structural properties of timber structures by testing.

- Comparison of various methods of evaluating the results of determination of structural properties by testing.

2 General aspects of design by testing

2.1 Conditions

2.1.1 The purpose and extent of testing

In the National Board of Housing, Building and Planning's proposal for a manual of determination of structural properties by testing [13], the Board states:

'The objective of testing can be to provide information for:

- *describing a load model, e.g. a model that indicates the effect of wind on the distribution of snow loading arising as a result of particular design of a building,*

- *describing the performance of a design and of a load effect model for given loads, e.g. a model of the performance of a design in response to dynamic loading or a model of load-carrying capacity, e.g. in connection with the buckling of thin sheet structures.*

The objective of testing can also be to investigate a combination of the above, e.g. determining the performance and load-carrying capacity of a structure with given loads.

Testing can involve investigation of an entire element (e.g. a beam), part of an element (e.g. a beam support) or a detail of a design (e.g. a means of fastening that is cast into a beam). In addition, testing can investigate all effects that are of significance for the design, or only certain effects.

Test results can be used for ***one*** *particular design, for several designs of* ***one*** *particular type and size (e.g. a corrugated sheet roof of a particular type and size) or generally for elements having the same design (e.g. corrugated sheet in a more general application).*

Testing can be performed on:

- *parts taken from normal production;*
- *a prototype of a structure;*
- *a model of structures or a part of the structure.*

It can be seen that, in general terms, there are many different objectives of design by testing. However, as far as timber structures are concerned, experience over the last couple of decades shows that determination of structural properties by testing is employed primarily for determining the load-carrying capacity of relatively simple structural elements intended to be manufactured in large numbers, when the relatively expensive testing can be offset against the benefits obtained in the form of higher design values than would result from traditional calculations.

2.1.2 Scale

The use of models of timber structures should normally be avoided, as there is a significant volume effect in connection with the strength of wood materials. This can be illustrated by the depth factor in Eurocode 5 [5] which, for example, means that the formal bending strength of a 75 mm high wooden beam is 15 % higher than that of a 150 mm high beam. In reality, the difference may be greater or smaller, depending on how the timber has been sorted and on its quality.

2.2 Planning the testing

2.2.1 A test plan

A plan should normally be prepared, describing the objective of testing and the various steps involved. According to [13], such a plan should contain the following points:

a. The extent of the information expected to be provided by testing and its applicational validity.

b. A description of the characteristics of the design and other features that can affect its performance at the particular limit conditions.

c. Calculation models.

d. A specification of the intended characteristics of the test items.

e. Measurements etc. to be performed on each test item prior to testing.

f. Specification of the loads and load procedures, together with other external influences during testing.

g. Details of support and load devices.

h. Measurement points and instrumentation equipment for recording forces, movement, deformation etc.

2.2.2 Calculation model

The planning, together with processing of the results, should normally be based on a mathematical model that will enable the results to be applied to other structures similar to those tested. This mathematical model need not be complete, but it should describe the important relationships between the parameters involved.

When dealing, for example, with lightweight I-section beams, the model can consist of an expression for the bending strength of the edges (the stress in the outermost fibres of the wooden flanges at failure) as a function of the load, the geometry of the beam and the stiffness characteristics of the material of the web. The model allows the results to be applied to beams of somewhat different geometry than those tested.

[4] describes the models schematically in the following manner:

$$R = D \cdot R_t(X, W) \qquad (2.1)$$

where:

X represents stochastic variables (e.g. strength values, cross-sectional areas etc.)

W are deterministic quantities (e.g. characteristic strength values, spans and other quantities that vary so little that they can be regarded as deterministic)

R_t is the theoretical model

R is the quantity measured during the tests (e.g. load-carrying capacity)

D is the unknown coefficient that is to be determined by testing.

An example of a mathematical model and its use are given in Chapter 4.

2.2.3 Test items

In principle, test items can be of two types: *specially manufactured* or *taken from normal production*.

The objective of testing is often to determine design parameters for structures that are to be mass-produced. However, mass production has seldom started when testing is carried out: on the contrary, it is often the case that production proper can be started only after the results of testing are available. It is therefore almost always necessary to test items that have been specially made in one way or another.

If the objective of testing is to establish characteristic load-carrying capacities for a particular structure, it is essential that the structures tested are properly representative in terms of materials and construction. The items tested must not, for example, contain materials of a higher quality than can be expected in future production. This can be difficult to ensure, and requires special methods. Chapter 3 describes a procedure that has been employed by the Swedish National Testing and Research Institute for about ten years.

2.2.4 Climate classes and conditioning

The normal reference climate for testing timber structures is 20 °C and 65 % RH, equivalent to the conditions in Climate Class 1 or Service Class 1 in EC 5. However, it may be necessary for various reasons to test the structure under the conditions in which it is intended subsequently to be used. Reference [13] states the following conditioning climates for various climate classes:

Table 1.1 Conditioning climates. Temperature: 20 °C.

Climate class	Relative humidity, %
0	40
1	65
2	80
3	100

2.3 Performing the tests

How the tests are to be performed depends largely on the type of structure and the purpose of testing. However, an important element here is the loading procedure. [13] proposes the following for wood structures:

Stage 1 loading to $F = 0.4\, F_{estimated}$

Stage 2 maintain $F = 0.4\, F_{estimated}$ constant for 30 seconds

Stage 3 unload to $F = 0.1\, F_{estimated}$

Stage 4 maintain $F = 0.1\, F_{estimated}$ constant for 30 seconds

Stage 5 increase the load continuously until failure occurs or until the load that is needed to effect some other failure condition is reached, e.g. maximum acceptable deformation.

$F_{estimated}$ is the estimated average failure load. It can be obtained by calculation or by prior testing: [13] states that it should not differ by more than 20 % from the average value of the measured loads at failure.

2.4 Evaluating the results

2.4.1 Correction of the test results

It is sometimes necessary to correct the test results for further processing. It can, for example, have been found that the moisture ratio differs significantly from the intended value. After conditioning in the 20 °C, 65 % RH reference climate, the moisture ratio of the wood should be between 11 % and 13 %. If values are outside this interval, some form of correction should be applied, particularly if the moisture ratio is too low, as there will otherwise be an overestimate of the strength and stiffness. However, it can be difficult to find a suitable correction expression. As far as the short-term load capacity and bending stiffness of I-beams with wooden flanges and synthetic board webs are concerned, the following expressions have been used by SP:

$$M_{u=12} = M_u \left(\frac{34}{46 - u} \right) \tag{2.2}$$

$$EI_{u=12} = EI_u \left(\frac{70}{82 - u} \right) \tag{2.3}$$

These two expressions are based on the assumption that it is the timber that is decisive for both the short-term load capacity and the bending stiffness, which is a reasonable assumption in the case of I-beams. For them, the fiber board webs are relatively unimportant. However, in other composite structures, where the significance of the various materials for the load-carrying capacity and stiffness is more similar, it can be difficult (if not impossible) to apply appropriate corrections for differences in moisture ratios.

Corrections for differences in dimensions can sometimes be necessary or desirable. A set of nominal dimensions will normally have been determined for the item to be tested. However, testing itself often results in noting that the actual dimensions of the test items differ from the nominal dimensions. For many purposes, this can be dealt with by a calculation model (see Equation 2.1), where the stochastic variable X can allow for variations in parameters such as the cross-sectional area.

2.4.2 Statistical evaluation, characteristic values

2.4.2.1 Load-carrying capacity and stiffness values for load-carrying capacity calculations

Characteristic values can be calculated either using a distribution free method or on the basis of a suitable distribution model. A distribution free method requires a relatively large number of test items (at least 30-odd), which is often not possible for cost reasons. The normal distribution is often used as a basis for calculating characteristic values. A special method has been developed in the Nordic countries [19], which allows for the uncertainty of estimation of mean values and standard deviations from a small number of test items.

The tests result in a number of measured values n of the load-carrying capacity R or the coefficient D: see Section 2.2.2. These quantities are designated below by x: the characteristic value of them can then be calculated as follows:

$$x_k = x_{mean} (1 - k_n V_x) \tag{2.4}$$

where x_{mean} is the mean value of the measured values

V_x is the coefficient of variation, given by the standard deviation/mean value

k_n is a factor that depends on the selected fractile and the number of test items, as shown in Table 2.1.

Table 2.1 Values of k_n from [13].

n	3	5	10	20	30	∞
k_n for $V_{x,\ known}$	2.03	1.94	1.85	1.79	1.76	1.64
k_n for $V_{x,\ unknown}$	3.19	2.46	2.10	1.93	1.87	1.64

Characteristic values calculated as above give a 75 % confidence level, which means that there is a 75 % probability of underestimating the 5 % fractile value of the population.

It can be seen from Table 2.1 that it is possible to utilise prior knowledge of the coefficient of variation. This prior knowledge might have been obtained, for example, from earlier tests of a closely similar product.

Eurocode 1 [4] gives other values of k_n, but still allows the use of prior knowledge of the coefficient of variation (see Table 2.2).

Table 2.2 Values of k_n in accordance with Eurocode 1 [4].

n	3	5	10	20	30	∞
k_n for $V_{x,\ known}$	1.89	1.80	1.72	1.68	1.67	1.64
k_n for $V_{x,\ unknown}$	3.37	2.33	1.92	1.76	1.73	1.64

[13] assign the values in Table 2.2 a confidence level that varies with n, but which is always less than 75 %.

If the measured values, which we can designate by y_i, are logarithmically normally distributed, the characteristic value can be calculated as follows:

1. Convert all the measured values to logarithms, $x_i = \ln y_i$
2. Calculate the mean values, standard deviation and coefficient of variation for the logarithmic values, x_i.
3. Calculate the characteristic value using Equation 2.4.
4. Calculate the characteristic value for y_i from

$$y_k = e^{x_k} = e^{(\ln y)_k} \qquad (2.5)$$

2.4.2.2 Stiffness values for deformation calculations

The characteristic value of stiffness values such as bending stiffness and modulus of elasticity is a mean value. Normally, the random mean value is used as the characteristic value. However, the uncertainty of estimation can be allowed for by considering the coefficient of variation. The k_n factor in Equation 2.4 can be selected as shown in Table 2.3.

Table 2.3 Values of k_n from [24] based on Student's t-distribution, such that there is a 75 % probability of underestimating the mean value of the population.

n	3	5	10	20	30	∞
k_n	0.44	0.33	0.22	0.15	0.12	0

2.5 Design values

2.5.1 General

As far as the load-carrying capacity of wood and wood-based materials is concerned, there are two factors that are particularly important and which distinguish these materials from most other structural materials: the load duration and the effect of moisture in the material on its strength and stiffness properties.

Wooden materials have a relatively high strength for short-term loading, e.g. impact loading. However, the strength declines with increasing load application time. This is described in a number of ways, including the Madison Curve that presents the strength as a function of load time [20].

The strength also declines with rising moisture content, and vice versa. Under normal conditions, the moisture content of wood materials depends on the relative humidity of the surrounding air. Timber structures are normally tested after conditioning to equilibrium conditions in a climate of 20 °C and 65 % RH. This climate is taken as a reference climate, and is equivalent to (for example) Climate Class 1 in BKR [21] and NS 3470 [7] and to Service Class 1 in Eurocode 5 [5].

Laboratory testing of wood and wood-based products is normally carried out such that the load is applied at a rate that will cause failure after 5-15 minutes, which can be regarded as the reference duration.

The modulus of elasticity is also affected by the moisture content of the material and by the load duration. In addition, the load gives rise to plastic deformation, known as creep, the magnitude of which also depends on the moisture content and the duration of the load.

Determination of necessary sizes is intended ultimately to provide dimensioning values for the load-carrying capacity and stiffness properties. Design values for load-carrying capacity are obtained by dividing the characteristic value by a partial coefficient (γ_m in [1]) and multiplying it by a factor (κ_r in [1]) which allows for the reduction in strength resulting from various climatic conditions. As far as design values for stiffness are concerned, the characteristic value is instead multiplied by a factor (κ_s in [1]), which allows for creep in the material under different climatic conditions.

2.5.2 Load duration and moisture content - corrections to design values as set out in EC5

a) Design values at the failure limit

The combination of load duration and climate class determines the correction factor to be employed for calculating design strength or capacity. EC 5 [5] refers to this factor as k_{mod}. When the most unfavourable load case consists of a combination of loads of different durations, the value of k_{mod} to be chosen is that corresponding to the shortest duration load.

b) Design values at the use state

In this case, it is the deformations that are decisive. In the case of a combination of several loads, the deformation must be determined for each individual load and then calculated by a factor that depends on the duration of the load and the climate class. This factor is given by $(1 + k_{def})$. The design deformation can then be calculated as the sum of the corrected deformations of the individual loads.

2.5.3 Composite structures

Structures of which the structural properties are determined by testing are generally characterised by being constructed from several different materials. In addition, they can include mechanical joints or adhesively bonded joints. Values of κ_r and κ_s and of k_{mod} and k_{def} are given in national standards and in EC 5 for the various constituent materials and joints, but there are essentially no values for the composite structure.

Values of k_{mod} and k_{def} must be selected on the basis of knowledge of the causes of failure and of the function of the tested structure.

As far as, for example, the short-term load-bearing capacity of I-beams is concerned, where failure almost always occurs in the flanges, it is reasonable to use the k_{mod} values given for the flanges in EC 5, or corresponding values in the national standards.

3 Materials and construction

3.1 General

As previously explained in Section 2.2.3, it is often necessary to use specially manufactured items for testing. If the objective is to provide test data for values of characteristic load-carrying capacities, the structure tested should be typical of what can be achieved in subsequent large-scale production. For structural materials other than wood, it is often possible subsequently to correct the test results if it appears that the characteristics of the materials used were not representative. This possibility is limited for timber structures.

This chapter discusses how the materials for the test structures should be selected, with the emphasis on selection of the timber. SP has developed a method of doing this that has been successfully applied for a period of about ten years.

3.2 Selection of timber

Part of the work of planning a test involves deciding which grades of materials are to be used in the structure. As far as timber is concerned, this can be one of the standardised grades, K12, K18, K24 or K30, or may also, of course, be a grade for which there is no standard. If so, special criteria need to be defined so that this non-standard quality can be sorted with satisfactory reliability.

Regardless of the grade of timber selected, it is necessary to ensure that the timber used in the test structure is representative of, or at least not better than, the timber that can normally be expected.

As described in Section 3.1, the quality grade indicates only a lower limit for the strength of the timber. K24 grade timber, which is by far the most commonly encountered, can have been produced by sorting in a number of different ways. The commonest method at present is that all timber having a characteristic strength in excess of 24 MPa is sorted, which normally means that most of it has a strength in excess of 30 MPa. It can also happen that K24 and K30 grades are sorted simultaneously: in this case, K24 will have a characteristic strength close to the nominal value.

Selecting timber of a given quality, representative of the class in question, is almost impossible, as it would necessitate selecting timber from a large number of different regions. An alternative is to attempt to control the quality so that stronger pieces of timber are avoided. The Swedish rules [11] recommend '.... *a conscious selection of the timber quality in the test pieces by selecting the material so that the requirements for the particular grade are just fulfilled*'. However, the rules do not state how this is to be done. On the other hand, excessively strict application of the criterion of '*just fulfilled*' would result in the strength of the test structure being undesirably low.

A number of methods of dealing with the problem can be considered. SS EN 28970 [25] describes a procedure for mechanical joints in wood that is based on measuring the density of the timber. This works well for this particular type of joint, where the strength is closely related to the density. However, as an indicator of the tensile and bending strength of the wood, density is a poor indicator. Nor is the size of knots a good indicator. A realistic and relatively accurate method is that of grading the timber quality using a mechanical grading machine, a method of using which is described in [22]. Briefly, this involves the following:

The timber is selected in the specified strength class, the lower limit of which is defined as the characteristic value. The following requirements must be met in each piece of timber:

1. *The bending strength at any position along the piece of timber* must exceed the lower class limit.
2. *The minimum bending strength within the timber* must be less than 1.1 times the lower class limit.
3. *The mean bending strength along the piece of timber* must not exceed 1.25 times the value of the lower class limit.

The table below illustrates what this means.

Table 3.1 Bending strength intervals (MPa) for mechanical stress grading of timber for strength classes K18, K24 and K30.

Strength class	Lower class limit	Upper class limit	Interval for mean bending strength	Lower limit for individual bending strength values	Interval for the lowest bending strength
K18	18	24	18 - 22.8	18	18 - 19.8
K24	24	30	24 - 28.8	24	24 - 26.4
K30	30		30 - 36	30	30 - 33

Trials at SP have shown that, with available stress grading machines, it is not possible to grade timber in accordance with the above criteria. The yield is so low, and such a large quantity of timber has to be graded in order to obtain sufficient quantities having the desired characteristics, that the method cannot be justified on cost grounds. Instead, SP has simplified the criteria to involve only requirements 1 and 3.

Table 3.2 shows the results of applying SP's criteria to a number of different types of timber: strength class K24 has been taken as an example. If timber with an estimated strength (machine grading) of between 24 and 1.25 x 24 MPa is sorted out, the characteristic strength of the timber will be 24 MPa, i.e. exactly the nominal value. If, however, Class K24 and better timber is sorted out, which is the normal procedure, the characteristic bending strength will be higher, as will the modulus of elasticity.

It can also be seen from the table that the yield when sorting the range 24 - 1.25 x 24 is relatively low. The spread in strength between pieces of timber is also relatively low: according to [18] and [23], it is only half as large as for K24 and better.

Table 3.2 Sorting of planed timber in a Cook Bolinder sorting machine [23]. Timber of sawfallen quality.
(f_{mk} = characteristic bending strength)
(E_m = mean modulus of elasticity)

Sorting criterion	**45 x 120 mm**			**70 x 170 mm**		
	Yield, %	**f_{mk}, MPa**	**E_m, MPa**	**Yield, %**	**f_{mk}, MPa**	**E_m, MPa**
24-30	16	24	10700	19	24	10500
>24	99	30	13200	99	26	13500

3.3 Wood-based sheet materials

In principle, the same conditions apply for wood-based sheet materials as for timber. However, the conditions are somewhat different for plywood, on the one hand, and materials such as fibreboard, chipboard and OSB on the other hand.

As far as plywood is concerned, performance characteristics can vary widely between sheets of one and the same grade. One way of avoiding the use of sheets with undesirably high quality can be to measure the modulus of bending elasticity and accept only those sheets having values between the nominal value and 1.1 times this.

For the other sheet materials, manufacturing technology is such that it is possible to control the quality relatively accurately so that the strength properties vary between only narrow limits. However, there are cost incentives to ensure that the material properties are as close to the specified values as possible, and so sheets of such materials can normally be used for determination of structural properties by testing without prior sorting. Material samples can be taken afterwards for examination and for any correction of the design strength values.

4 Application example: I-beams a comparison between different design methods

4.1 General

For the purposes of an example, this chapter describes a comparison between different design methods for I-beams. Design performance determination is carried out by testing, by calculation in accordance with Eurocode 5 and in accordance with the rules in Finland, Sweden and Norway. It is the short-term load capacity and the shear panel capacity that are investigated.

The whole consideration is based on a beam in bending, not loaded with any compressive or tensile forces. This means that it is the design conditions for tensile stress that are always decisive when compared with the conditions for compressive stress, as the tensile strength is less than the compressive strength. During 1991 and 1992, VTT tested I-beams as an industrial commission, and it is these test results that form the starting point for this comparison. The I-beams were manufactured by Pyhännän Rakennustuote Oy, which has agreed to allowing the test results to be used in this comparison. It should be noted that the test results are not identical with those that serve as a basis for Finnish type approval. Some simplification has also been applied in order better to illustrate the comparison between the different design methods.

4.2 Cross-section of the I-beam

The terms used to designate the various parts of the I-beam are shown in Figure 4.1. The ideal moment of inertia of the cross-section is indicated by I_i and can be calculated from the expression

$$I_i = \frac{b(h^3 - h_w^3)}{12} - \frac{b_w(h_b^3 - h_w^3)}{12} + \frac{E_w}{E_f}\frac{b_w h_b^3}{12} \qquad (4.1)$$

where E_w is the modulus of elasticity of the web and E_f is the modulus of elasticity of the flange.

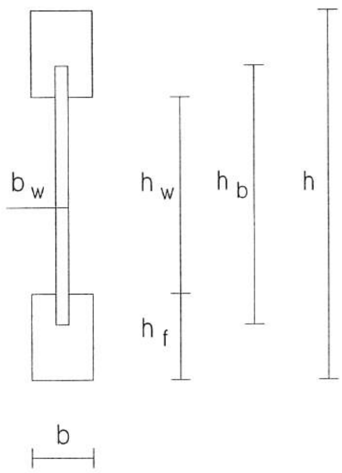

Figure 4.1 Dimensions of the I-beam cross-section

4.3 Material characteristics

The material in the flanges is spruce (*Picea abies*), mechanically stress-graded to Class T40 in accordance with the Finnish grading rules. After grading, the timber has been split, with one part being used for the upper flange of the beam and the other part being used for the lower flange of the same beam. Because of the splitting, it is assumed that the flanges are Class T30 timber.

Table 4.1 shows the assumed characteristic strength and stiffness values of the timber in the flanges. These values apply for 5-minute load durations, with the moisture content of the timber in the flange in equilibrium at a temperature of 20 °C and a relative humidity of 65 %. The values shown are based on preliminary European standard prEN 338 [9], the Finnish Building Regulations, Part B 10 [6], Norwegian standard NS 3470 [7] and the Swedish New Building Rules [1].

Table 4.1 Characteristic strength values (MPa) for the flange timber

	EC5 C30	**Finland T30**	**Norway T30**	**Sweden K30**
Bending strength f_{mk}	30.0	29.9	30.0	30.0
Tensile strength f_{tk}	18.0	19.5	18.0	20.0
Compressive strength f_{ck}	23.0	28.6	27.0	29.0
Modulus of elasticity:				
(load-carrying capacity) E_{Rk}	8000	7800	8750	8700
(deformation) E_k	12000	9100	12300	12000

In the case of timber subjected to bending stress, and having a cross-sectional depth less than 150 mm, and of timber subjected to tensile stress and having a longer cross-sectional dimension of less than 150 mm, the characteristic values for bending stress f_{mk} and tensile strength f_{tk}, as given in EC5, must be multiplied by a factor k_h, derived from the expression

$$k_h = \left(\frac{150}{h_f}\right)^{0,2} \qquad (4.2)$$

where h_f is the depth of the cross-section or of the longer cross-sectional dimension in mm. Equation 4.2 is valid for 40 mm $< h_f <$ 150 mm. For $h_f <$ 40 mm, k_h is given a value of 1.3.

The web consists of a hard Class B wood fibre board complying with Finnish Standard SFS 2190 [12]. Table 4.2 shows the assumed characteristic strength and stiffness values for the material, valid for load durations of five minutes and with the moisture ratio of the web in equilibrium with a temperature of 20 °C and a relative humidity of 65 %. The values given in the table are based on input to proposed European standard CEN TC 112.406 [3], the Finnish RIL 120 design codes for timber structures [10] and the Swedish New Building Regulations. Building Details A 520.237 in the Norwegian Building Research Series [2] gives values only for Class K40 particle boards. These values are identical with corresponding values in Sweden. However, as the material does not fulfil the requirements of Class K40, the Swedish Class K35 values are used.

Table 4.2 Characteristic strength values (MPa) for the material of the web

	EC5 HB	**Finland B**	**Norway K35**	**Sweden K35**
Tensile strength f_{tk}	18.0	16.8	20.0	20.0
Compressive strength f_{ck}	19.0	12.7	20.0	20.0
Panel shear strength f_{nk}	13.0	10.8	12.0	12.0
Layer shear strength f_{pk}	2.1	0.8	1.5	1.5
Modulus of elasticity:				
(load-carrying capacity) E_{Rk}	2100	3150	3500	3500
(deformation) E_k	2600	3850	4500	4500
Shear modulus				
(load-carrying capacity) G_{Rk}	900	1260	1500	1500
(deformation) G_k	1100	1540	1900	1900

The characteristic values under the Finnish rules are expressed for conditions that differ from 5-minute durations. This has been corrected by multiplying the specified values by correction factors, which explains why the bending strength of T30 in Table 4.1 is 29.9 MPa and not 30 MPa.

While the test beams were being loaded, the density and moisture ratio of the timber in the flanges and of the material in the web were also tested. The mean value and standard deviation of the density of the timber were 466 kg/m³ and 36 kg/m³ respectively. The mean moisture ratio of the timber was 13.3 %, equivalent to a relative humidity of 65 % at a temperature of 20 °C. The mean value and standard deviation of the web material were 962 kg/m³ and 21.4 kg/m³ respectively. The mean moisture ratio of the web material was 6.2 %. Calculations of the density employed the mass and volume as measured at the stated moisture contents.

4.4 Testing

Testing were performed in accordance with the Nordtest method NT BUILD 327 [8] in order to determine the moment capacity, shear capacity and bending strength. Cross-sectional dimensions as shown in Table 4.3 were investigated.

Table 4.3 Cross-sectional dimensions

Beam	**b** mm	**h** mm	**b_w** mm	**h_w** mm	**h_f** mm	**h_h** mm
300/45	45	300	6	210	45	250
300/70	45	300	6	160	70	200
400/70	45	400	6	260	70	300
450/70	45	450	6	310	70	350

The web of the test pieces used in the bending tests was jointed, with the vertical joint positioned between the point loads. The test arrangement for the bending tests is illustrated in Figure 4.2.

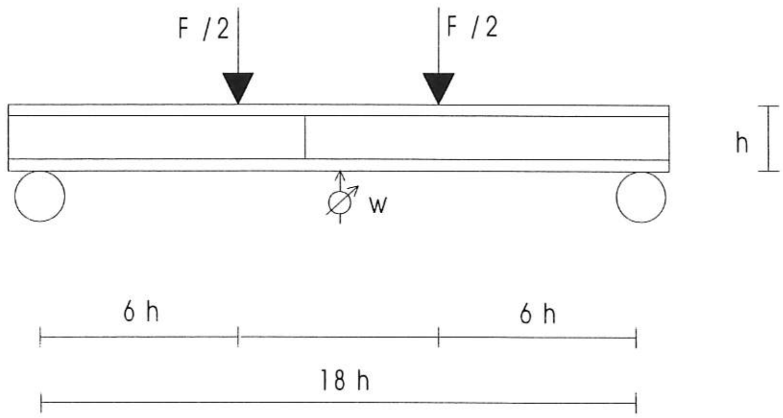

Figure 4.2 Test arrangement for bending tests

The web of the beams used for the shear tests was jointed, with the vertical joint positioned in the shear zone. A section through the joint is shown in Figure 4.3. Shear testing employed two point loads for beams 17 - 28 and a single point load, positioned at the centre, for beams 29 - 34. The test arrangement for the shear tests is shown in Figure 4.4.

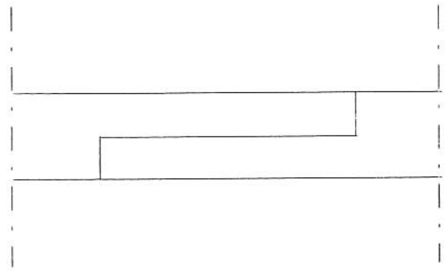

Figure 4.3 The joint in the web

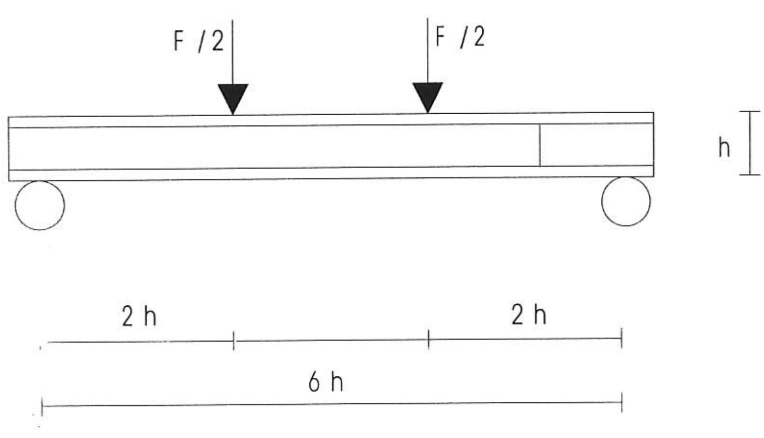

Figure 4.4 Test arrangement for the shear tests

In the tests, the load was first increased to F_1, equivalent to one-sixth of the estimated failure load F_{est}. It was then reduced to a value $F_2 = F_{est} \div 30$, after which it was increased to a value $F_3 = F_{est} \div 3$ and maintained constant for five minutes, before being increased until the beam failed. The total time for each test was about half an hour.

In the bending tests, 11 of the failures occurred in the tensile side and five of them in the compression side. All the failures were bending failures. The moment capacity, M_u, was calculated from the expression

$$M_u = \frac{F_u \cdot L}{6} \tag{4.3}$$

where F_u is the failure load and L is the span of the test beam (= 18 h). Table 4.4 shows the values of the load $\Delta F = F_3 - F_2$, deflection $\Delta w = w(F_3) - w(F_2)$ (the deflection at the middle of the beam, with allowance for deformation at the supports when the load increases from F_2 to F_3), the failure load F_u, and the moment capacity M_u, as noted during the bending test.

Table 4.4 Results of the bending tests

Beam	No.	ΔF kN	Δw mm	F_u kN	M_u kNm
300/45	1	5.33	20.6	16.3	14.7
	2	5.40	21.0	15.4	13.9
	3	5.44	23.8	14.9	13.4
	4	5.37	17.7	20.7	18.6
	5	5.36	21.1	15.4	13.9
300/70	6	7.30	20.4	24.9	22.4
	7	7.20	19.2	29.1	26.2
	8	7.35	20.8	25.0	22.5
	9	7.26	20.2	23.2	20.9
	10	7.26	20.5	20.6	18.5
400/70	11	7.32	21.9	26.8	32.2
	12	7.05	20.1	27.5	33.0
	13	7.12	23.0	23.6	28.3
	14	7.19	23.6	24.9	29.9
	15	7.26	24.3	25.4	30.5
	16	7.24	22.3	25.1	30.1

Ten of the failures in the shear tests occurred in the vertical joint in the web, while two occurred in the web but not at the joint. For six of the beams, all loaded with only one point load, the buckling of the web was so excessive that loading was terminated. Buckling also affected the capacity of many of the beams that failed through shearing of the web. None of the beams failed as a result of exceeding the planar shear strength. The shear capacity, V_u, was calculated from the expression

$$V_u = \frac{F_u}{2} \tag{4.4}$$

Table 4.5 shows the values of failure load, F_u, and shear capacity, V_u, noted during the bending tests.

Table 4.5 Results of the shear tests

Beam	No.	F_u kN	V_u kN
300/70	17	29.8	14.9
	18	34.8	17.4
	19	33.9	17.0
	20	35.4	17.7
	21	42.7	21.4
	22	38.4	19.2
400/70	23	41.9	21.0
	24	41.1	20.6
	25	37.3	18.6
	26	35.6	17.8
	27	37.6	18.8
	28	29.7	14.8
450/70	29	26.6*	13.3*
	30	24.8*	12.4*
	31	31.5*	15.8*
	32	26.3*	13.2*
	33	26.6*	13.3*
	34	35.0*	17.5*

* Buckling of the web

4.5 Design by calculation

4.5.1 Eurocode 5

In the EU, I-beams can be designed in accordance with Eurocode 5, Design of Timber Structures [5]. For a beam having lateral support, so that it cannot buckle, the design moment capacity, M_d, is the lowest of the values obtained from the following three expressions:

$$\sigma_{md} \leq f_{md} \tag{4.5}$$

$$\sigma_{td} \leq f_{td} \tag{4.6}$$

$$\sigma_{wmd} \leq f_{wmd} \tag{4.7}$$

where f_{md} is the design value for the bending strength of the beam, f_{td} is the design value for the tensile strength of the beam and f_{wmd} is the design value for the bending strength of the web. f_{wmd} in Equation 4.7 may be replaced by the lesser of the design values for the compressive or tensile strength of the web. The bending stress in the edge of the flange, σ_{md}, the tensile stress at the centre of gravity of the flange, σ_{td}, and the bending stress in the web, σ_{wmd}, can be calculated from the expressions

$$\sigma_{md} = \frac{M_d \cdot h}{2 \cdot I_i} \tag{4.8}$$

$$\sigma_{td} = \frac{M_d (h - h_f)}{2 \cdot I_i} \tag{4.9}$$

$$\sigma_{wmd} = \frac{E_w}{E_f} \frac{M_d \cdot h_b}{2 \cdot I_i} \tag{4.10}$$

where h, h_b and h_f are as shown in Figure 4.1. The moment of inertia I_i of the cross-section is obtained from Equation 4.1. E_w is the modulus of elasticity of the web and E_f is the modulus of elasticity of the flange. As the web is jointed as shown in Figure 4.3, it is assumed that only half of its thickness is carrying the shear loads. When buckling does not occur, the design capacity in respect of panel shear, V_{pd}, is calculated from the expression

$$V_{pd} = 0{,}5 \cdot b_w \cdot h_b \cdot f_{pd} \tag{4.11}$$

where f_{pd} is the design value for the panel shear strength of the web. b_w is as shown in Figure 4.1. There is no risk of buckling when $h_w < 35\ b_w$.

The design capacity in respect of panel shear, V_{pd}, when $35\ b_w < h_w < 70\ b_w$, can be calculated from the expression

$$V_{pd} = \frac{35 \cdot b_w}{h_w} \frac{b_w}{2} h_b \cdot f_{pd} \tag{4.12}$$

where h_w is as shown in Figure 4.1.

4.5.2 Finland

In Finland, I-beams can be designed in accordance with the Finnish Building Regulations, Part B 10, Timber Structures [6].

For a beam having lateral support, so that it cannot buckle, the design moment capacity M_d can be calculated from the expression

$$\frac{\sigma_{td}}{f_{td}} + \frac{\sigma_{md} - \sigma_{td}}{f_{md}} \leq 1 \tag{4.13}$$

where f_{td} is the design value of the tensile strength of the flange and f_{md} is the design value of the bending strength of the flange. The tensile stress at the centre of gravity of the flange, σ_{td}, can be calculated from Equation 4.9, while the bending stress at the edge of the flange σ_{md}, can be calculated from Equation 4.8.

As the web is jointed as shown in Figure 4.3, it is assumed that only half of the thickness of the web is carrying the shear loads. When buckling does not occur, the design capacity in respect of panel shear, V_{pd}, can be calculated from the expression

$$V_{pd} = 0{,}5 \cdot b_w \cdot h_b \cdot f_{pd} \qquad (4.14)$$

where f_{pd} is the design value for the panel shear strength of the web. h_b and b_w is as shown in Figure 4.1. There is no risk of buckling when $h_w < 27\, b_w$.

When $h_w > 27\, b_w$, the design capacity in respect of panel shear V_{pd} can be calculated from the expression

$$V_{pd} = 0{,}5 \cdot b_w \cdot h_b \cdot f^*_{pd} \qquad (4.15)$$

where f_{pd}* is calculated from the characteristic value of panel shear strength f_{pk}*, with allowance for buckling. f_{pk}* can be calculated from the expression

$$f^*_{pk} = 3{,}3 \cdot k \cdot E_{Rk} \left(\frac{b_w}{h_w} \right)^2 \qquad (4.16)$$

where E_{Rk} is the characteristic value of the modulus of elasticity of the web. h_w is as shown in Figure 4.1. If the web is not supported, and if the relationship between the shear modulus of the web and its modulus of elasticity is equal to 0.4, k has a value of 1.30.

If Equation 4.16 gives a greater value of f_{pk}* than the characteristic value of the panel shear strength of the web, f_{pk}, f_{pd}* must be calculated on the basis of the value of f_{pk}.

4.5.3 Norway

In Norway, I-beams may be designed in accordance with Norwegian standard NS 3470, Design of timber Structures, Calculation and Design Rules [7].

For a beam having lateral support, so that it cannot tip, the design moment capacity M_d can be calculated from the expression

$$\frac{\sigma_{td}}{k_h \cdot f_{td}} + \frac{\sigma_{md} - \sigma_{td}}{k_h \cdot f_{md}} \le 1 \qquad (4.17)$$

where f_{td} is the design value of the tensile strength of the flange and f_{md} is the design value of the bending strength of the flange. The tensile stress at the centre of gravity of the flange, σ_{td}, can be calculated from Equation 4.9, while the bending stress at the edge of the flange σ_{md}, can be calculated from Equation 4.8.

In the case of timber subjected to bending stress, and having a cross-sectional height less than 200 mm, and of timber subjected to tensile stress and having a dimension of less than 200 mm for its longer cross-sectional dimension, the design values for bending stress f_{md} and tensile strength f_{td} must be multiplied by a factor k_h, derived from the expression

$$k_h = \left(\frac{200}{h_f}\right)^{0,2} \tag{4.18}$$

where h_f is the height of the cross-section or of the longer cross-sectional dimension in mm. Equation 4.18 is valid for 100 mm $<h_f<$200 mm. For $h_f<$100 mm, k_h is given a value of 1.15.

As the web is jointed as shown in Figure 4.3, it is assumed that only half of its thickness is carrying the shear loads. When buckling does not occur, the design capacity in respect of panel shear, V_{pd}, is calculated from the expression

$$V_{pd} = 0,5 \cdot b_w \cdot h_w \cdot f_{pd} \tag{4.19}$$

where f_{pd} is the design value for the panel shear strength of the web. h_b and b_w is as shown in Figure 4.1. There is no risk of buckling when $h_w < 35\ b_w$.

When $35\ b_w < h_w < 70\ b_w$, the design capacity in respect of panel shear V_{pd} can be calculated from the expression

$$V_{pd} = \frac{35 \cdot b_w}{h_w} \frac{b_w}{2} h_b \cdot f_{pd} \tag{4.20}$$

where h_w is shown in Figure 4.1.

4.5.4 Sweden

The Swedish New Building Regulations [1] give no detailed design rules. However, the 1980 Swedish Building Regulations [27] provide detailed rules for designing I-beams.

For a beam having lateral support, so that it cannot buckle, the design moment capacity M_d can be calculated from the expression

$$\frac{\sigma_{td}}{f_{td}} + \frac{\sigma_{md} - \sigma_{td}}{f_{md}} \leq 1 \tag{4.21}$$

where f_{td} is the design value of the tensile strength of the flange and f_{md} is the design value of the bending strength of the flange. The tensile stress at the centre of gravity of the flange, σ_{td}, can be calculated from Equation 4.9, while the bending stress at the edge of the flange, σ_{md}, can be calculated from Equation 4.8.

As the web is jointed as shown in Figure 4.3, it is assumed that only half of its thickness is carrying the shear loads. When buckling does not occur, the design capacity in respect of panel shear, V_{pd}, is calculated from the expression

$$V_{pd} = 0{,}5 \cdot b_w \cdot h_b \cdot f_{pd} \qquad (4.22)$$

where f_{pd} is the design value for the panel shear strength of the web. h_b and b_w is as shown in Figure 4.1. There is no risk of buckling when $h_w < 35\ b_w$.

When $35\ b_w < h_w < 70\ b_w$, the design capacity in respect of panel shear V_{pd} can be calculated from the expression

$$V_{pd} = \frac{35 \cdot b_w}{h_w} \frac{b_w}{2} h_b \cdot f_{pd} \qquad (4.23)$$

where h_w is shown in Figure 4.1.

4.6 Design by testing

4.6.1 Eurocode 1

Within the EU, I-beams can be designed in accordance with Eurocode 1, Basis of Design and Actions of Structures, Part 1, Basis of Design, Annex 5, Design Assisted by Testing [4]. This Eurocode is available in a version dated October 1993.

Design by testing is based on one or more calculation models, depending on the actual type of failure. As the calculation model is not all-embracing, designs must include a correction parameter obtained from actual testing. This parameter is assumed to have a normal distribution.

For a beam having lateral support, so that it cannot buckle, the calculation model for the moment capacity M_u is assumed to be

$$M_u = D \frac{2 \cdot I}{h} f_{mk} \qquad (4.24)$$

where f_{mk} is the characteristic value of the bending strength of the flange. h is as shown in Figure 4.1, while the moment of inertia of the cross-section, I, is obtained from Equation

4.1 by assuming that $E_w = E_f$. D is the unknown parameter, obtained from the test results. Substituting Equation 4.3 in Equation 4.24, we obtain

$$D = \frac{F_u \cdot L \cdot h}{12 \cdot I \cdot f_{mk}} \qquad (4.25)$$

where F_u is the measured failure load and L is the span of the test beam (= *18 h*). Table 4.6 shows the individual values, d_i, of parameter D obtained from bending tests.

Table 4.6 Analysis results of bending tests

Beam	No.	d_i
300/45	1	1.03
	2	0.97
	3	0.94
	4	1.31
	5	0.97
300/70	6	1.27
	7	1.49
	8	1.28
	9	1.19
	10	1.05
400/70	11	1.17
	12	1.20
	13	1.03
	14	1.09
	15	1.11
	16	1.10
Mean value		1.14
Standard deviation		0.15

The characteristic value of the unknown parameter D can be calculated from the expression

$$D_k = m_{d_i} - k_n \cdot s_{d_i} \qquad (4.26)$$

where m_{di} is the mean value of the values of d_i, and s_{di} is the standard deviation of d_i. k_n is a coefficient that depends on the number of values and on whether the variation coefficient is known or unknown. Values of k_n are given in Table A5.1 in Eurocode 1, Part 1, Annex D [4]. When n = 16, k_n has a value of 1.82.

Inserting values from Table 4.6 and a value of 1.82 for k_n in Equation 4.26 gives a value of 0.867 as characteristic value for the unknown parameter D.

In order to evaluate this model, the measured values of moment capacity, M_{ui}, from Table 4.4 have been plotted in Figure 4.5 against the values, M_{ue}, obtained from the calculation model (Equation 4.24) when correction parameter D = 1.14. The line of exact accordance has also been plotted in the diagram. The relationship between M_{ui} (kNm) and M_{ue} (kNm) is

$$M_{ui} = 0{,}514 + 0{,}978 \cdot M_{ue} \qquad (4.27)$$

and the correlation coefficient is 0.934.

If the regression equation (4.27) is constrained to pass through the origin, then

$$M_{ui} = 0{,}998 \cdot M_{ue} \qquad (4.28)$$

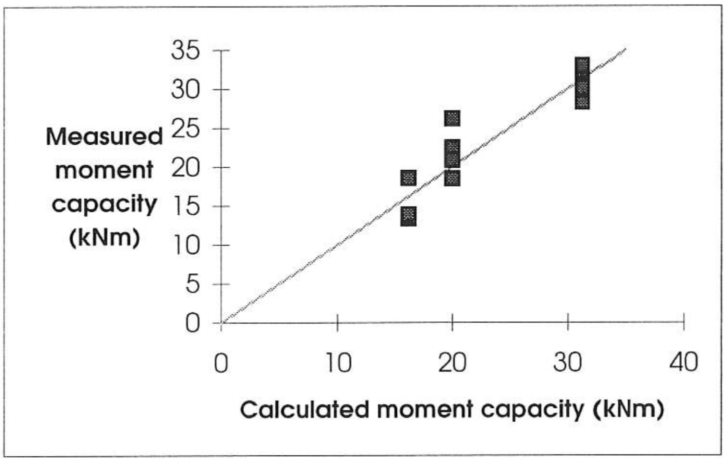

Figure 4.5 Measured and calculated values of the moment capacity, M_u, when D = 1.14.

Shear tests were carried out on 18 beams. Ten of the failures occurred in the vertical joint in the web, while two occurred in the web but not at the joint. For six of the beams, buckling of the web was so excessive that loading was terminated. The failure load for these beams was assumed to be the same as the maximum load, which assumption gives somewhat conservative results. If the assumption is accepted, it means that all the beams failed when the capacity of the web was exceeded. However, this capacity depends on the buckling of the web. When h_w <(26 = i) b_w, the calculation model for the capacity in respect of shear stress V_{pu} is assumed to be

$$V_{pu} = D \cdot b_w \cdot h_b \cdot f_{pk} \qquad (4.29)$$

and when $(26 + i)\, b_w < h_w < (52 + 2i)\, b_w$, the calculation model is assumed to be

$$V_{pu} = D\frac{(26+i)b_w}{h_w}b_w \cdot h_b \cdot f_{pk} \tag{4.30}$$

where $1 < i < 9$ x f_{pk} is the characteristic value of the panel shear strength of the web. b_w, h_b and h_w are as shown in Figure 4.1, while D is the unknown parameter obtained from the tests.

The value of i is selected as a whole number that gives the best correlation coefficient between the measured values of V_u and the values given by Equations 4.29 and 4.30 of the calculation model. Based on the tests performed, i has the value of 3. If V_{pu} in Equations 4.29 and 4.30 is substituted by V_u from Equation 4.4, while i is assumed to be equal to 3, we obtain

$$D = \frac{F_u}{2 \cdot b_w \cdot h_b \cdot f_{pk}} \tag{4.31}$$

$$D = \frac{h_w}{29 \cdot b_w} \frac{F_u}{2 \cdot b_w \cdot h_b \cdot f_{pk}} \tag{4.32}$$

Table 4.7 presents the individual values, d_i, of parameter D as obtained from the shear tests.

Table 4.7 Analysis of the shear test results

Beam	No.	d_i
300/70	17	0.96
	18	1.12
	19	1.09
	20	1.13
	21	1.37
	22	1.23
400/70	23	1.34
	24	1.31
	25	1.19
	26	1.14
	27	1.20
	28	0.95
450/70	29	0.87
	30	0.81
	31	1.03
	32	0.86
	33	0.87
	34	1.14
Mean value		1.09
Standard deviation		0.17

The value of the unknown parameter, D, is obtained from Equation 4.26. When $n = 18$, k_n has a value of 1.79.

If the mean value and standard deviation of d_i, as shown in Table 4.7, are inserted in Equation 4.26, together with the value of 1.79 for k_n, we obtain 0.786 as the characteristic value of the unknown parameter.

In order to evaluate results from the model, Figure 4.6 shows a plot of the measured values of shear capacity, V_{ui}, from Table 4.5 against the V_{ue} values obtained from Equations 4.29 and 4.30 in the calculation model when correction parameter $D = 1.09$. The line indicating exact agreement has also been drawn in.

As the calculated values of V_u differ by only 0.3 kN from each other, the relationship between V_{ui} (kN) and V_{ue} (kN) is meaningless. However, if the regression equation is plotted through the origin, we obtain

$$V_{ui} = 1{,}00 \cdot V_{ue} \qquad (4.33)$$

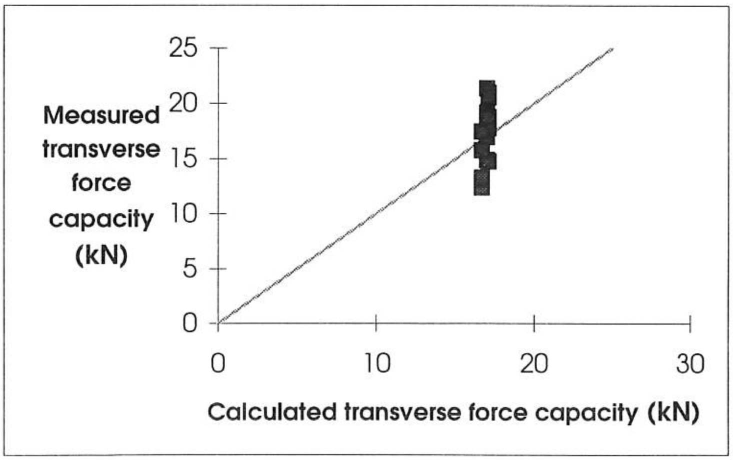

Figure 4.6 Measured and calculated values of shear capacity, V_u, when $D = 1.09$.

In order further to evaluate the model, Figure 4.7 shows the relationship between the measured values of V_u and the values produced by the calculation model (D = 1.09), plotted against the relationship between h_w *and* b_w. The dotted line indicates exact agreement.

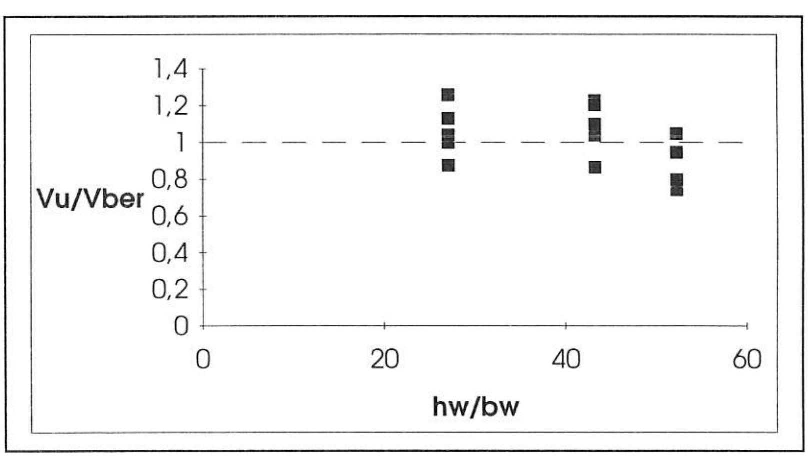

Figure 4.7 Relationship between measured and calculated values of V_u as a function of the relationship between the free height and width of the web.

4.6.2 Finland, Norway and Sweden

In Finland, I-beams can be designed in accordance with RIL-120, Design Rules for Timber Structures, Appendix 2, Design Values, and Appendix 3, Determination of structural properties by Testing [10]. In Norway, I-beams can be designed in accordance with NS 3470, Design of Timber Structures, Calculation and Design Rules, Supplement A, Rules for Testing and Approval of Timber Structures [7]. In Sweden, I-beams can be designed in accordance with the Strength Determination by Testing Manual [13]. These three design methods do not differ from each other.

When the beam is transversely supported, so that it cannot buckling, the calculation model for moment capacity, M_u, is assumed to be

$$M_u = \frac{2 \cdot I}{h} f_{mu} \tag{4.34}$$

where f_{mu} is the bending strength value of the I-beam obtained from testing. h is as shown in Figure 4.1, while the moment of inertia, I, of the cross-section is obtained from Equation 4.1 by assuming that $E_w = E_f$. Substituting Equation 4.3 in Equation 4.34 gives

$$f_{mu} = \frac{F_u \cdot L \cdot h}{12 \cdot I} \tag{4.35}$$

where F_u is the measured failure load and L is the span of the test beam (= *18 h*). Table 4.8 shows the values of f_{mu} obtained from bending tests.

Table 4.8 Analysis results of bending tests

Beam	No.	f_{mu} MPa
300/45	1	30.9
	2	29.2
	3	28.3
	4	39.3
	5	29.2
300/70	6	38.2
	7	44.7
	8	38.4
	9	35.6
	10	31.6
400/70	11	35.2
	12	36.1
	13	31.0
	14	32.7
	15	33.3
	16	32.9
Mean value		34.2
Standard deviation		4.4

f_{mk}, the characteristic value of the bending strength of the I-beam, can be calculated from the expression

$$f_{mk} = m_{f_{mu}} - k_n \cdot s_{f_{mu}} \tag{4.36}$$

where m_{fmu} is the mean value of f_{mu}, and s_{fmu} is the standard deviation of f_{mu}. k_n is a coefficient that is used to reduce the mean value to correspond to the 5 % fractile, and is obtained from the number of tests performed, n. When n = 16, k_n has a value of 1.98.

If we enter the mean value and standard deviation of f_{mu} as shown in Table 4.8 in Equation 4.36, together with a value of 1.98 for k_n, we obtain 25.5 MPa as the characteristic value of the bending strength of the I-beam.

When $h_w < 29\, b_w$, the calculation model for determining the shear strength capacity V_{pu} is assumed to be

$$V_{pu} = b_w \cdot h_b \cdot f_{pu} \tag{4.37}$$

and when $29\, b_w < h_w < 58\, b_w$, the calculation model is assumed to be

$$V_{pu} = \frac{29 \cdot b_w}{h_w} b_w \cdot h_b \cdot f_{pu} \tag{4.38}$$

where f_{pu} is the value of the shear strength of the I-beam obtained from the tests. b_w, h_b and h_w are as shown in Figure 4.1, while the limit value of $29\, b_w$ has been assumed on the basis of the test results, so that the best possible match is obtained. If V_{pu} in Equations 4.37 and 4.38 is replaced by V_u from Equation 4.2, we obtain

$$f_{pu} = \frac{F_u}{2 \cdot b_w \cdot h_b} \tag{4.39}$$

$$f_{pu} = \frac{h_w}{27 \cdot b_w} \frac{F_u}{2 \cdot b_w \cdot h_b} \tag{4.40}$$

Table 4.9 shows the values of parameter f_{pu} obtained from the shear tests.

Table 4.9 Analysis of the shear test results

Beam	No.	f_{pu} MPa
300/70	17	12.4
	18	14.5
	19	14.1
	20	14.8
	21	17.8
	22	16.0
400/70	23	17.4
	24	17.1
	25	15.5
	26	14.8
	27	15.6
	28	12.3
450/70	29	11.3
	30	10.5
	31	13.4
	32	11.2
	33	11.3
	34	14.8
Mean value		14.2
Standard deviation		2.2

The characteristic value of the shear strength of the I-beam, f_{pk}, is obtained from Equation 4.36 by replacing f_{mk} by f_{pk}. When n = 18, k_n has a value of 1.97. Inserting the mean value and standard deviation of f_{pu} from Table 4.9 in Equation 4.36, together with a value of 1.97 for k_n, we obtain 9.9 MPa as the characteristic value of the panel shear strength of the I-beam.

4.7 Evaluation

Tables 4.10 and 4.11 show comparisons of the characteristic moment capacity and the characteristic panel shear strength capacity as obtained by design by calculation and design by testing.

In the case of design by testing, the identical results obtained in Finland, Norway and Sweden are 2-3 % poorer than the corresponding results obtained by Eurocode 1. This difference is due only to the coefficient by which the standard deviation is multiplied when calculating the characteristic value: see Equations 4.26 and 4.36.

Table 4.10 The characteristic moment capacity (kNm) obtained by design by calculation and by design by testing of the I-beams investigated.

Beam	Eurocode 1		Finland		Norway		Sweden	
	Calc.	Test	Calc.	Test	Calc.	Test	Calc.	Test
300/45	11.7	12.3	9.1	12.1	9.8	12.1	9.3	12.1
300/70	15.5	15.2	12.1	14.9	13.0	14.9	12.4	14.9
400/70	22.0	23.8	18.2	23.3	19.4	23.3	18.6	23.3
450/70	25.3	28.3	21.3	27.8	22.8	27.8	21.8	27.8

In Finland, Norway and Sweden, design by testing gives moment capacities that are 1.2 - 1.3 times as high as the moment capacities obtained by design by calculation. The difference is least in Norway, as the characteristic bending and tensile strength values can be increased for small cross-sectional areas. This advantage does not result to design by testing under the Eurocode rules, which is due to the fact that the moment capacity obtained by design by calculation is higher than in Finland, Norway or Sweden. The moment capacities are calculated by various different expressions.

Table 4.11 The characteristic panel shear strength capacity (kN) obtained by design by calculation and by design by testing of the I-beams investigated.

Beam	Eurocode 1		Finland		Norway		Sweden	
	Calc.	Test	Calc.	Test	Calc.	Test	Calc.	Test
300/45	9.8	12.7	8.1	12.3	9.0	12.3	9.0	12.3
300/70	7.8	12.3	6.5	11.9	7.2	11.9	7.2	11.9
400/70	9.4	12.3	7.1	11.9	8.7	11.9	8.7	11.9
450/70	9.2	12.0	5.8	11.7	8.5	11.7	8.5	11.7

Design by testing under the Norwegian, Swedish and Eurocode rules gives shear capacities that are 1.4 times as high as the shear capacities obtained by design by calculation. This difference can be explained by the fact that, in reality, the jointed web is capable of carrying 70 % of the load capacity of an unjointed web, and not 50 %. Under the Finnish rules, design by testing gives even better results. However, this is due to the fact that the characteristic panel shear strength is lower than in Norway, Sweden or as allowed by the Eurocode system. The actual panel shear strength of the panel is unknown.

When designing I-beams of this type in accordance with the national regulations in Finland, Norway or Sweden, it is preferable to do so by design by testing. When designing them in accordance with the Eurocode rules, the benefit is insignificant. However, in order to be able to utilise the full capacity of a jointed web, the manufacturer must demonstrate what this capacity is, which can most suitably be done by design by testing.

5 References

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