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A method for assessing the fire resistance of laminated timber beams and columns

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A method for the calculation of the fire resistance of timber beams and columns is described. The method is based partly on the results of theoretical studies and partly on the results of a large number of tests on timber beams and columns. Simple formulas for calculating fire resistance are presented. Comparison with experimental results shows good agreement between calculated and measured fire resistances.

L'auteur décrit une méthode de calcul de la résistance au feu des poutres et poteaux de bois. La méthode se fonde en partie sur les résultats d'études théoriques et en partie sur les résultats de nombreux essais sur des poutres et poteaux de bois. L'auteur présente des formules simples de calcul de la résistance au feu. Une comparaison avec des données expérimentales produit un bon accord entre les valeurs calculées et expérimentales de résistance au feu.

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Introduction

The fire resistance of a structural member may be defined as its ability to withstand exposure to fire without loss of its loadbearing function. This ability provides time to enable people to evacuate a building before it collapses in the event of fire; it is also essential for the purpose of confining a fire to the compartment where it started.

A common method to determine the fire resistance of timber structural members is by subjecting them to a standard fire test. Usually testing is costly and time consuming. Calculation methods exist but the meagre information that is available on the values of the parameters that determine the fire performance of timber structural members restricts their accuracy and their general applicability in practice.

In this paper a semi-empirical method is presented for calculating the fire resistance of timber beams and columns for a wide range of practical conditions. The method is based partly on the results of theoretical studies (Imaizumi 1962; Ödeen 1970) and partly on the results of a large number of tests on beams and columns.

Behavior of Timber Structural Members in Fire

When a structural member of timber is exposed to fire the material will generally be ignited. During the combustion of the material a char layer is formed at the exposed surface. The thickness of this layer grows continuously at a slow rate.

The formation of a char layer, which has practically no strength, means loss of load-carrying capacity of the member; the thicker the layer the greater the loss. In addition, there is also some loss of strength and rigidity of the uncharred timber because of its temperature rise. As the charring proceeds a moment will be reached when the member can no longer support its load and it will collapse.

To calculate the time for which a member is capable of supporting its load, it is necessary to know:

1. the rate of charring,
2. the temperature distribution in the uncharred part of the member, and
3. the strength and deformation properties of the material as a function of temperature.

The rate of charring of wood depends on various factors (Schaffer 1967), of which the most important are the density of the wood, its permeability, and its moisture content. From existing data (Dorn and Egner 1967; Hall 1968; Imaizumi 1962; Rogowski 1970; Schaffer 1967), it can be derived that 0.6×10^{-3} m/min is a reasonable average value for the charring rate of wood. When the wood is light and dry a value of 0.8×10^{-3} m/min is in general a better approximation of the charring rate of wood; when it is moist and dense the value is 0.4×10^{-3} m/min.

The temperature distribution in the uncharred part of the member depends on the rate of charring, on the temperature of the burning material at the surface of the uncharred wood, and on its thermal properties. The thermal properties of wood normally depend on the type of wood and its temperature (Knudson and Schniewind 1975; Ödeen 1970). This also applies to the strength and deformation properties of wood.

Calculation of Fire Resistance

Calculation of fire resistance of timber structural members based on temperature distribution in the member, material strength, and deformation is possible in principle (Knudson and Schniewind 1975). However, lack of knowledge of the various processes that take place during exposure to fire, such as the combustion processes under the char layer and outside the member, the heat transfer from the fire to the member, and the movement of moisture in the material, as well as inadequate knowledge of material properties at elevated temperatures, makes it difficult at present to determine the fire resistance accurately in this way.

The prediction of fire resistance of timber structural members can be considerably simplified and reasonably accurate results obtained if a semi-empirical method (Imaizumi 1962; Ödeen 1970; Lie 1972) is used for determining the fire resistance. In this method the following assumptions are made.

(a) The member is exposed to a standard fire, in this case one meeting the definition in ASTM E119 (American Society for Testing and Materials 1974).

(b) The fire resistance of the member is the time for which the member is capable of supporting its load. This load is a fraction k of the ultimate load.

(c) Due to temperature rise the compressive strength and the modulus of elasticity of the uncharred part of the member is reduced. The effect of this reduction can be taken into account by using, in the calculation, reduced values of the compressive strength and the modulus of elasticity. (It is assumed that these values are a fraction α of their values before exposure to fire.)

(d) The rate of penetration of the charring can be given approximately by a constant average value β.

Beams

From these assumptions, it can be derived (Imaizumi 1962) that the critical depth d of a beam that is heated on all sides is determined by the relation

$$[1] \qquad \frac{k}{\alpha}\frac{B/D}{d/D - (1 - B/D)} = (d/D)^2$$

where B and D are the breadth (smaller side) and depth (larger side) of the beam before the fire.

When the critical depth of the beam and the rate of penetration of the charring are known, the time t_{b4} to reach this critical depth is given by

$$[2] \qquad t_{b4} = (D - d)/2\beta$$

In the derivation of [1] it is assumed that the beam is exposed to heating on all sides. In practice the top of the beam is often protected by a floor or roof construction, so that only three sides of the beam are exposed to heating. In a manner similar to that used in the case of heating on four sides, it can be derived for this case that the critical depth d of the beam, *i.e.* the depth of the unburnt part of the beam at the time of failure, is determined by

$$[3] \qquad \frac{k}{\alpha}\frac{B/D}{B/D - 2(1 - d/D)} = (d/D)^2$$

and its fire resistance t_{b3}, which is equal to the time to reach this critical depth, is given by

$$[4] \qquad t_{b3} = (D - d)/\beta$$

Columns

Relations similar to those for beams can be derived for the critical depth and fire resistance of columns.

According to standard structural design formulas for columns, the relation for short columns between the load p, stress σ, and area A of the cross section is given by

$$[5] \qquad p = \sigma A$$

It is assumed that during the exposure to fire a load p_a is applied. According to [5] the relation between this load, the stress in the column, and the dimensions of the column is given by

[6] $$p_a = \sigma BD$$

where B and D are the breadth (*larger* side) and depth (*smaller* side) of the column before the fire.

During the fire, the size of the uncharred part of the column decreases. If this part is sufficiently short, failure occurs when the stress in the column reaches a value equal to the compressive strength of the uncharred part of the column. Thus, at the time of failure:

[7] $$p_a = \alpha\sigma_c bd$$

where σ_c is the compressive strength of the wood before exposure to the fire, and b and d are the breadth and depth of the uncharred part of the column at failure. From [6] and [7] it follows that the critical depth d is determined by the relation

[8] $$\sigma BD = \alpha\sigma_c bd$$

Since $B - b = D - d$, and $\sigma/\sigma_c = k$, [8] for the critical depth can also be written as

[9] $$\frac{k}{\alpha}\frac{B/D}{d/D - (1 - B/D)} = \frac{d}{D}$$

For long columns, which fail by buckling, the critical depth can be derived using Euler's formula

[10] $$p' = \pi^2 EA/(L/r)^2$$

where p' is the buckling load, E is the modulus of elasticity of the wood before exposure to fire, L is the effective length of the column, and r is the radius of gyration.

Assuming that a load $p_a = kp'$ is applied and that the radius of gyration $r = D/\sqrt{12}$, the relation between load, column dimensions, and material properties at the start of exposure to fire is

[11] $$p_a = k\pi^2 EBD^3/12L^2$$

At the time of failure, this relation becomes

[12] $$p_a = \pi^2\alpha Ebd^3/12L^2$$

From [11] and [12], it follows that for long columns the critical depth is determined by

[13] $$\frac{k}{\alpha}\frac{B/D}{d/D - (1 - B/D)} = \left(\frac{d}{D}\right)^3$$

Equations 9 and 13 show that the critical depth is determined by an expression dependent on n where n is an exponent having a value of $n = 1$ for short columns and $n = 3$ for long columns. It is plausible that the critical depth of intermediate columns is given in a similar expression where $1 < n < 3$. Thus, the general form of the equation that determines the critical depth of a column is

[14] $$\frac{k}{\alpha}\frac{B/D}{d/D - (1 - B/D)} = \left(\frac{d}{D}\right)^n$$

where $1 \leqslant n \leqslant 3$.

In the same way as for the fire resistance of beams, the fire resistance of columns heated on four sides is given by

[15] $$t_{c4} = (D - d)/2\beta$$

In this paper, the average value of $n = 2$ in [14], which is expected to be valid for intermediate columns, will be chosen for further examination.

Approximate Formulas

In the equations for the critical depth ([1], [9], and [13]) this depth appears as an implicit variable. Although solving the equations does not represent a great problem, graphical or numerical methods have to be used to calculate the critical depth and fire resistance. An attempt was made, therefore, to derive expressions in which the fire resistance is given explicitly as a function of the parameters that determine it.

For this purpose, the formulas were programmed for numerical solution and the fire resistance calculated for a wide range of practical cases. In the calculations, the average value of $\beta = 0.6 \times 10^{-3}$ m/min was used for the rate of charring, which as mentioned before was approximately representative of the average rate of charring found in a large number of experiments. For the factor α, expressing the reduction of the strength and stiffness of the uncharred part of the member due to temperature rise of the wood, a value was also used that was based on experimental results. Studies showed that the ultimate strength of various woods, at temperatures that the uncharred wood normally reaches in fires, reduces to about 0.85–0.90 of the original strength (Dorn and Egner 1960; Ödeen 1970; Tenning 1970). The reduction of the modulus of elasticity is of the same order of magnitude. Due to two-dimensional heating at the corners of a section the rate of charring at the corners is faster than the average rate of 0.6×10^{-3} m/min used in the calculations. Since the excess loss of section is relatively small, and loss of section is analogous to loss of strength or rigidity, a simple way to take into account the faster rate of charring at the corners is by choosing a somewhat lower value of α. In this study a value of $\alpha = 0.80$ was used as the reduction factor for the strength as well as for the modulus of elasticity of the uncharred part of a member.

Calculated results are given in Figs. 1–3. The results for beams are shown in Figs. 1 and 2, where t_b/D (the ratio of fire resistance to depth of the beam) is plotted as a function of B/D for various values of k. The curves in Fig. 1 are for beams heated on four sides. The solid lines give the results calculated from [1] and [2]. These lines can be described approximately by the expression:

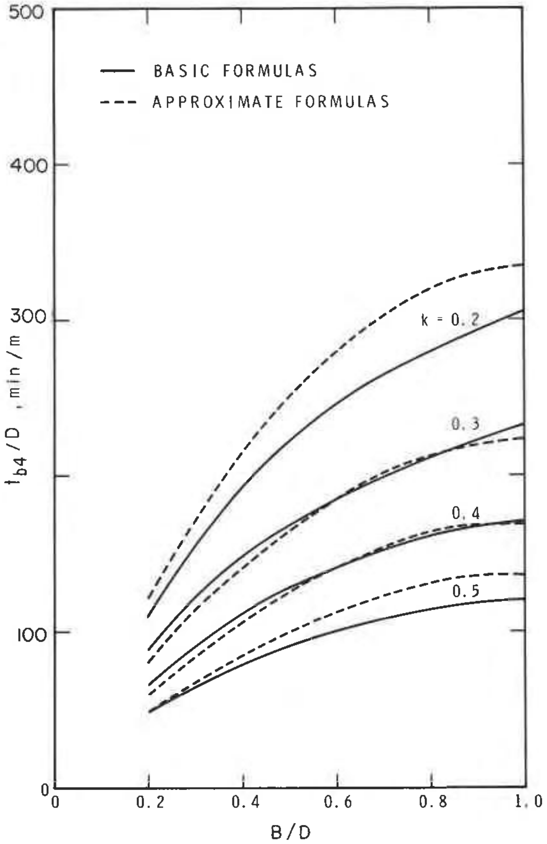

FIG. 1. Fire resistance of beams heated on four sides as a function of size and load.

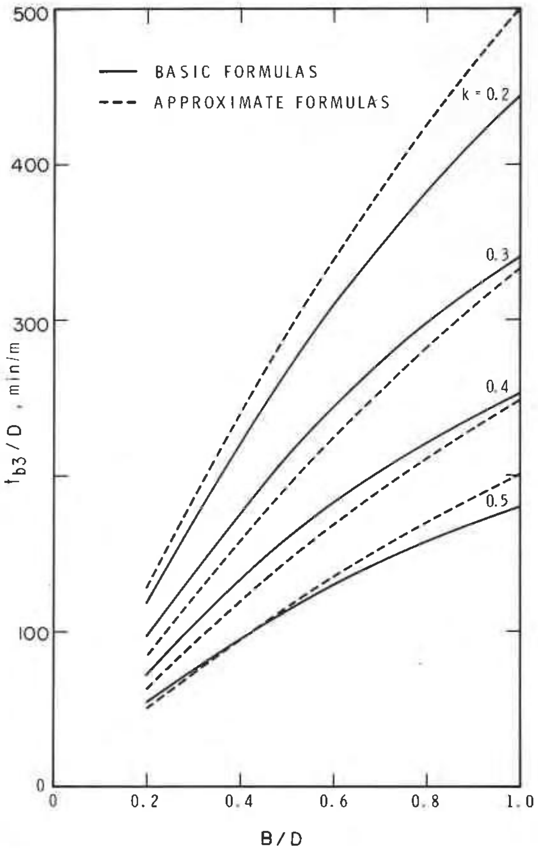

FIG. 2. Fire resistance of beams heated on three sides as a function of size and load.

[16] $t_{b4} = 33(B/k)[4 - 2(B/D)]$

Results obtained from this expression are given by the dashed lines in Fig. 1.

In Fig. 2 curves similar to those for beams heated on four sides are given for beams heated on three sides. For these beams the fire resistances, calculated from [3] and [4] and given by the solid lines in the figure, can be approximated by the expression

[17] $t_{b3} = 33(B/k)[4 - (B/D)]$

Curves for columns heated on four sides are shown in Fig. 3, where t_{c4}/D (ratio fire resistance to length of the smaller side of the column) is plotted as a function of D/B for various values of k. The solid lines give the fire resistance calculated from [14] and [15], and the dashed lines those obtained from the

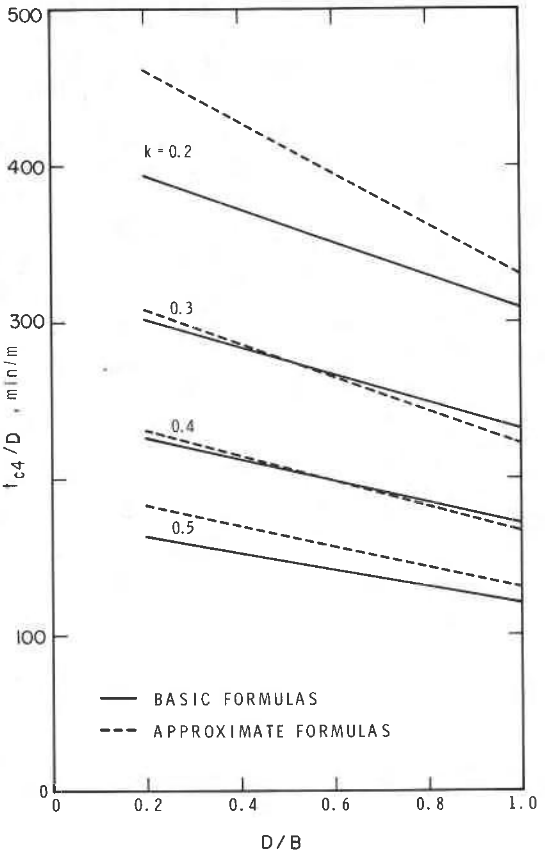

FIG. 3. Fire resistance of columns heated on four sides as a function of size and load.

approximate formula

$$[18] \qquad t_{c4} = 33(D/k)[3 - (D/B)]$$

The curves in Figs. 1–3 indicate that for values of k in the range of about 0.2–0.5, corresponding to safety factors in the range of 5–2, the formulas approximate reasonably well the calculated results. For safety factors higher than 5, which usually means that the member is substantially overdesigned, the approximate formulas considerably overestimate the fire resistance and are therefore no longer valid. It is possible, however, to derive for these cases the fire resistance of the member by using the relevant basic formulas, [1]–[15].

Comparison of Experimental and Theoretical Results

A large number of tests have been carried out on timber beams and columns. Almost all tests on beams, however, were terminated before failure occurred, and the remainder did not provide sufficient information to make a reliable comparison between experimental and theoretical fire resistances (Dorn and Egner 1960; Imaizumi 1962; Lawson *et al.* 1951; Ödeen 1970). On the other hand, several measurements were made of various factors that determine the fire resistance, such as temperatures in the beam, rate of charring, and strength of the material. These measurements, and measurements on beams that had been exposed to fire in actual practice (Fox 1974), provided the basis for the development of the equations that have been presented.

More experimental information is available on the fire resistance of timber columns (Fackler 1961; Malhotra and Rogowski 1970; Stanke 1970; Stanke *et al.* 1973). As discussed earlier, the fire resistance of timber columns depends on a large number of factors such as timber species, moisture content, type of glue, slenderness of the column, but the most important are column size and shape and the safety factor. In the approximate formula, [18], the influence of the three last-named factors have been taken into account by separate variable quantities D, B, and k. The influence of the other factors have been assumed to be relatively small, and average values have been used for the variables affected by them. These variables include the rate of charring β, for which the average value of 0.6×10^{-3} m/min has been chosen. The influence of reduction of strength and stiffness of the uncharred wood and the faster loss of section at the corners have been taken into account by using a value for α of 0.80.

A comparison between the fire resistances calculated from the design equation, [18], and those measured during a large number of tests (Fackler 1961; Malhotro and Rogowski 1970; Stanke 1970; Stanke *et al.* 1973) is made in Fig. 4. In this figure, all results of these tests on laminated columns are shown, with the exception of those tests in which the load on the column was less than one half of the allowable load.

As shown in the derivation of the design formulas, for such loads these formulas substantially overestimate the fire resistance and are therefore not valid. The results in Fig. 4

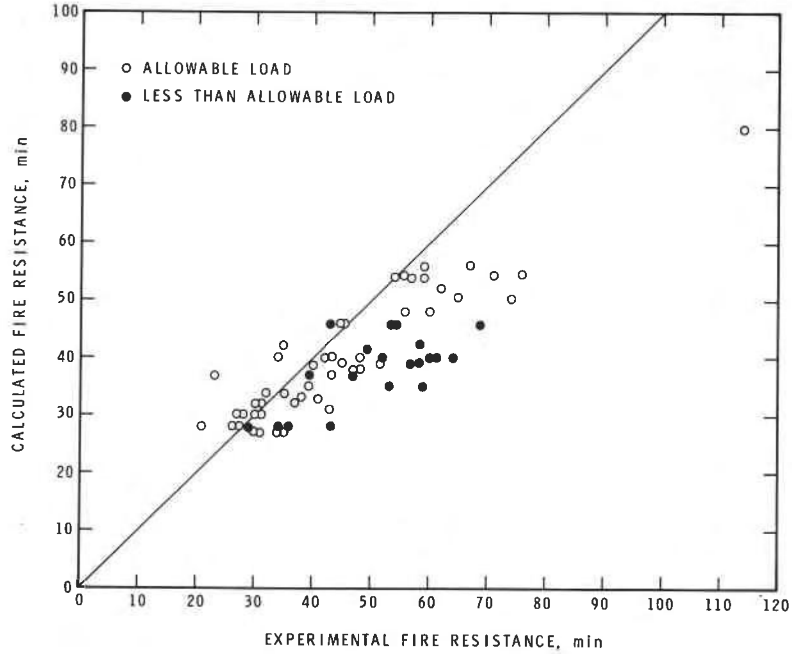

FIG. 4. Comparison between calculated and experimental fire resistances (not corrected for overdesign and column slenderness).

include those of tests with various safety factors, wood species, moisture contents, types of glue, and column sizes, shapes, and slenderness ratios. Since the actual safety factor was not known precisely for each test, an average value of $k = 0.33$ has been used in the design formula. This value of k corresponds to a safety factor of 3, which is approximately equal to the safety factor customarily assumed in various fire resistance studies (Imaizumi 1962; Ödeen 1970; Stanke 1970; Stanke *et al.* 1973). Tests on sawn timber have not been included. Their performance is considerably affected by the formation of shrinkage cracks (Stanke 1970) and therefore it is not possible to make a reliable prediction of their fire resistance.

It is seen in Fig. 4 that, although most predictions err on the safe side, differences between calculated and measured fire resistances of the order of 50% occur. The greater differences, however, are for columns that were tested under a load less than the design load. It may be expected that in these cases the safety factor is higher than the factor 3 assumed in the design formula, so that in these cases the predictions are conservative. If only the tests in which the design load was applied are considered, the agreement between calculated and experimental results becomes much better.

There is still a systematic difference, however, between calculated and experimental results for the columns with higher fire resistances, but the differences are not large and are accountable. As discussed before, there are advantages to using only one formula instead of three to describe the fire resistance of columns of varying slenderness. Therefore a value of $n = 2$ was chosen in the basic equation, [14], for the calculation of fire resistance, irrespective of the column slenderness, although $n = 1$ would be more exact for short columns and $n = 3$ for long columns. As a consequence, [14] tends to overestimate the fire resistance of long columns, and to underestimate that of short columns. The overestimation of calculated fire resistances for longer columns, which in general are represented by the lower fire resistances in Fig. 4, is not clear. However, for shorter columns, represented by the higher fire resistances, the underestimation of fire resistance is pronounced.

Since slenderness of columns and overdesign have significant influence on fire resistance, the accuracy of the design formulas can be

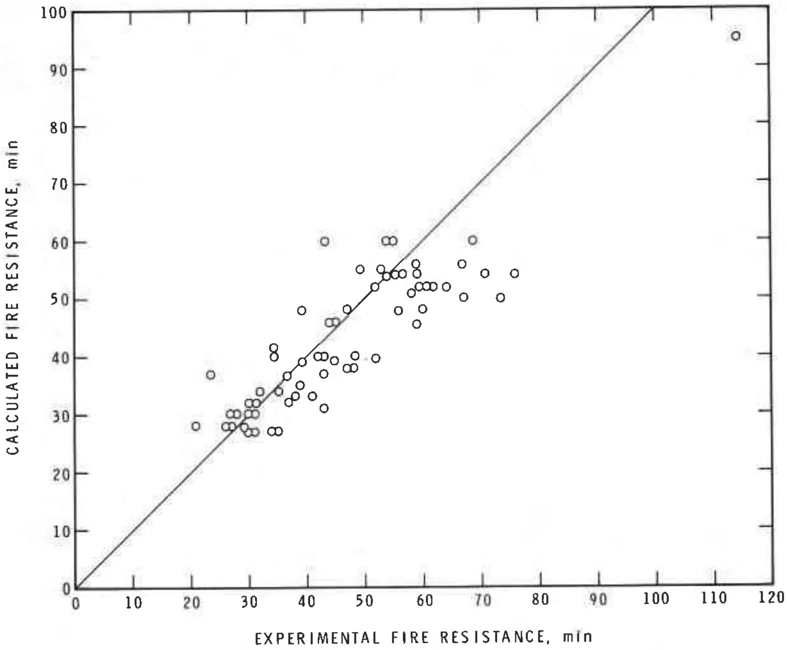

Fig. 5. Comparison between calculated and experimental fire resistances (overdesign and column slenderness are taken into account).

improved if the influence of these factors is taken into account in the formulas. A simple way of doing this is by including an empirical factor f in the calculated fire resistance for shorter columns or if the column is overdesigned.

The following approximate formulas are suggested for the calculation of the fire resistance of timber beams and columns.

Beams Heated on Four Sides

[19] $$t_{b4} = 100fB[4 - 2(B/D)]$$

Beams Heated on Three Sides

[20] $$t_{b3} = 100fB[4 - (B/D)]$$

Columns Heated on Four Sides

[21] $$t_{c4} = 100fD[3 - (D/B)]$$

The factor f depends on the load and for columns also on the effective length, as given in Table 1. If a load is applied that is lighter than the allowable load the fire resistance of a member increases. The higher fire resistance of a member that is overdesigned is expressed by a higher value of f. With respect to fire, a member may be regarded as overdesigned if it is designed to resist extreme loads such as loads due to earthquake, wind, or snow. Because the probability of occurrence of earthquake or strong wind at the time of fire is very low, in the determination of the load for the time that the member is exposed to fire, the effects of earthquake and wind may be neglected. Loads due to snow have a higher probability of occurrence. It is not likely, however, that the construction, *e.g.* roofs or other surfaces, will be subjected to the full design snow load during exposure to fire. Because the magnitude of the load at the time of fire is determined by chance it cannot be predicted. It seems reasonable, however, to assume that the probability of occurrence of fire and at the

Table 1. Values of f

Load (as % of allowable load)	Member type		
	Beam	Column (L/D) > 10	Column (L/D) ≤ 10
> 75	1.0	1.0	1.2
≤ 75, > 50	1.1	1.1	1.3
≤ 50	1.3	1.3	1.5

same time a snow load greater than 50% of the design snow load is negligible. It is suggested, therefore, to assume for fire exposure that the snow load is 50% of the design snow load.

In Fig. 5, the fire resistances calculated from [21] are compared with experimental results. It is seen that some differences between calculated and experimental results of the order of 35–40% remain, but if it is taken into consideration that differences in the measured fire resistance between repeat tests of about 30% are not uncommon (Stanke 1970; Stanke *et al.* 1973), the predicted fire resistances may be regarded as reasonably accurate.

Example

It might be useful to illustrate in an example the use of the approximate formulas for the calculation of the fire resistance of timber beams and columns. In this example a beam is considered that is exposed to fire on three sides. The breadth B of the beam is 0.25 m and its depth D is 0.75 m. The load on the beam may be assumed to be 75% of the allowable load.

According to [20] the fire resistance of the beam is given by

$$t_{b3} = 100fB[4 - (B/D)]$$

According to Table 1, for a load of 75% of the allowable load, $f = 1.1$. Therefore, substitution of values into the formula gives

$$t_{b3} = 100 \times 1.1 \times 0.25[4 - (0.25/0.75)] = 101$$

Thus the calculated fire resistance of the beam is 101 min.

Acknowledgement

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Notation

A = area of cross section (m^2)

b = breadth of uncharred part of member at the time of failure (larger side of column, smaller side of beam) (m)

B = breadth of member before exposure to fire (larger side of column, smaller side of beam) (m)

d = depth of uncharred part of member at the time of failure (smaller side of column, larger side of beam) (m)

D = depth of member before exposure to fire (smaller side of column, larger side of beam) (m)
E = modulus of elasticity (N/m^2)
f = factor (values given in Table 1)
k = ratio between applied load and ultimate load
L = effective length of column (m)
n = exponent in [14], taking into account the dependence of the critical depth of columns on column length
p = load (N/m^2)
p_a = applied load (N/m^2)
p' = buckling load (N/m^2)
r = radius of gyration (m)
t_{b3} = fire resistance of beams heated on three sides (min)
t_{b4} = fire resistance of beams heated on four sides (min)
t_{c4} = fire resistance of columns heated on four sides (min)
α = a factor that takes into account the reduction of compressive strength (or modulus of elasticity) due to temperature rise in the uncharred part of column or beam
β = rate of penetration of the charring (m/min)
σ = stress (N/m^2)
σ_c = compressive strength of the wood before exposure to fire (N/m^2)