A Method for Calculating the Fire Resistance of Exposed Timber Decks

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ABSTRACT

A simple calculation method was developed to estimate the fire resistance of timber decks. The method consists of equations to determine the time to thermal penetration and the time to structural failure. The former is based on a recommendation in Eurocode 5, Part 1.2. The latter resulted from a transformed section analysis for a range of load ratios and plank thicknesses; assuming a constant charring rate, experimentally validated temperature profiles, safety factors commonly used in the U.S., and temperature effects on stiffness reported in the literature. The calculation method is in reasonable agreement with experimental data obtained in the 1960's.

KEYWORDS: calculation methods, decks, fire resistance, wood

NOTATION

a = thermal penetration depth (40 mm)
B = original plank width (mm)
b_i = width of slice i of transformed section (mm)
C_0 = constant in Equation 8 (min)
C_1 = function of R in Equation 8 (min/mm)
COV = estimated coefficient of variation of statistical distribution of ultimate F_b (--)
D = original plank thickness (mm)

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d = thickness of remaining section (mm)
d_i = thickness of slice i of transformed section (mm)
E = modulus of elasticity (GPa)
F_b = bending strength (MPa)
k_b = ratio of design F_b to average ultimate F_b (--)
k_b' = ratio of design F_b to fifth percentile ultimate F_b (--)
R = ratio of applied to design load (--)
$S(t)$ = transformed section modulus after t minutes of fire exposure (mm^3)
T = temperature (°C)
T_a = ambient and initial temperature (20 °C)
T_c = temperature at the char front (300 °C)
T_i = temperature of slice i of transformed section (°C)
x = distance from the char front (mm)
t_{sf} = time to structural failure (min)
t_{tf} = time to thermal failure (min)

Greek
β = charring rate (mm/min)
β_0 = design charring rate (mm/min)
δ = additional char depth for effective cross-section method (mm)
ξ = reduction coefficient to account for increased charring at the joints (--)
ρ_0 = oven dry density (kg/m^3)

INTRODUCTION

Heavy timber construction has long been recognized for its ability to maintain structural integrity while exposed to fire. Exposed wood structural members are popular with architects and designers of modern buildings because they have a pleasing appearance, are economical and easy to work with while providing the necessary fire endurance. The superior fire performance of heavy timbers can be attributed to the charring effect. As wood members are exposed to fire, an insulating char layer is formed that protects the core of the section. Thus, members can be designed so that a sufficient cross section of wood remains to sustain the loads for the required duration of fire exposure. A standard fire exposure is used for design purposes. In North America this exposure is described in the standard fire endurance test ASTM E 119 [1]. Many other countries use a comparable test exposure based on ISO 834 [2]. In spite of the differences between standard fire endurance tests, experimental charring rates measured in various parts of the world seem to be consistent. This justifies the use of such data for design, regardless of the origin.

The design method for fire-resistive exposed timber beams and columns used in North America is based on research that T.T. Lie conducted at the National Research Council of Canada in the 1970's [3]. The method was first recognized by the U.S. model building codes in 1984 as a National Evaluation Report [4]. In subsequent years, the method was adopted by the three model code organizations, allowing architects and builders to include fire-rated heavy timber members in their projects without conducting expensive standard fire resistance tests. Similar design methods for beams and columns exist in many other countries. The most recently developed procedures for the fire design of exposed timbers are described in *Eurocode 5:*

Design of timber structures - Part 1.2: General rules - Structural fire design. This document was published by the European Commission for Standardization (CEN) in November 1994 as a European prestandard (ENV 1995-1.2:1994) [5].

In the U.S. no design equations have been developed for exposed timber decks. The aforementioned Eurocode provides only partial design guidelines. Timber decks consist of planks that are at least 38 mm thick (2 inches nominal). The planks span the distance between supporting beams, and can be arranged in different ways depending on the available lengths [6]. The planks are typically 140 mm wide (6 inches nominal). Usually, a single or double tongue-and-groove method is used to connect adjoining planks, but splines or butted joints are also common. Based on the experience with beams and columns, it is expected that a simple calculation method for the fire endurance of timber decks would significantly increase the application and use of this type of fire-resistive construction. The objective of this paper is to develop and validate a calculation method for the fire resistance of exposed timber decks, that is comparable to the design equations currently accepted in the U.S. for the fire design of exposed wood beams and columns.

CALCULATING FIRE RESISTANCE OF DECKS

In order to meet the requirements for a given fire resistance rating, a floor deck needs to maintain its separating function and load carrying capacity for the specified duration of exposure to standard fire conditions. The former means that the temperature rise on the unexposed side of the floor deck must not exceed critical values of 140 °C on average over the entire surface area, or 180 °C at a single location [1][2]. The latter means that the deck shall carry the specified load, and shall not deflect beyond an acceptable service limit. The two aspects of the fire design of decks are discussed in detail below.

Thermal Separation

A method for calculating the thermal penetration through decks is given in Eurocode 5, Part 1.2 [5]. The time to termination of the separating function depends on the thickness of the deck, and can be estimated from Equation (C.1) in the Eurocode

$$t_{tf} = \xi \frac{D}{\beta_0} \tag{1}$$

where t_{tf} = time to thermal failure (min);
      ξ = reduction coefficient to account for increased charring at the joints (--);
      D = thickness of the decking (mm); and
      β_0 = design charring rate (mm/min).

$\xi = 0.2$ for butted joints, $\xi = 0.4$ for single tongue and groove or spline, and $\xi = 0.6$ for double tongue and groove. There are minimum dimensions for the splines or tongues, which presumably, are derived from the test data that served as a basis for these values. The design

charring rate values, β_0, for members with a minimum dimension of 35 mm are listed below.

- Solid sawn softwoods with $\rho_0 \geq 290$ kg/m^3 . . . . $\beta_0 = 0.8$ mm/min

- Glued laminated softwoods with $\rho_0 \geq 290$ kg/m^3 . . . . $\beta_0 = 0.7$ mm/min

- Hardwoods with $\rho_0 \geq 450$ kg/m^3 . . . . $\beta_0 = 0.5$ mm/min

- Hardwoods with $\rho_0 \geq 290$ kg/m^3 . . . . $\beta_0 = 0.7$ mm/min

Interpolation is permitted for densities in between the 290 kg/m^3 and 450 kg/m^3 limits. If the minimum density requirements are not fulfilled, β_0 is estimated based on the assumption that it is proportional to the inverse of the square root of ρ_0. The design charring rate values, β_0, are higher than experimental charring rate values, β, to account for the higher rate of charring at arisses. If there is a wood subfloor and/or floor on top of the deck, the thickness of the floor and subfloor can be added to that of the deck planks to calculate t_{tf} according to Equation (1).

Structural Performance

The procedure to determine the time to structural failure of deck planks is similar to that for beams. Structural failure of a beam is assumed to occur when the following critical condition is reached

$$\frac{S(t_{sf})}{S(0)} = R k_b \tag{2}$$

where $S(t_{sf})$ = transformed section modulus after t_{sf} minutes of fire exposure (mm^3);
t_{sf} = time to structural failure (min);
$S(0)$ = section modulus of the original section (mm^3);
R = ratio of applied to design load (--);
k_b = ratio of design F_b to average ultimate F_b (--); and
F_b = bending strength (MPa).

The "design" load results in a maximum bending stress equal to the design stress, F_b, for a single member. The factor k_b is the ratio of design bending strength to average ultimate bending strength, and can be calculated from

$$k_b = k_b'(1 - 1.645 COV) \tag{3}$$

where k_b' = ratio of design F_b to fifth percentile ultimate F_b; and
COV = estimated coefficient of variation of statistical distribution of ultimate F_b (--).

With $k_b'=0.476$ for softwood and $k_b'=0.435$ for hardwood [7], and $COV=0.25$ for solid sawn lumber and $COV=0.11$ for glulam [8], values for k_b are obtained as shown in Table 1.

TABLE 1. Values for k_b

	Softwood		Hardwood	
	Solid Sawn	Glulam	Solid Sawn	Glulam
k_b	0.280	0.400	0.256	0.356

$S(t)$ is the section modulus of the transformed section that remains after t minutes of fire exposure. The section has to be transformed to account for the (partial) heating of the remaining section. This technique is common for composite beams [9], and its application for deck planks exposed to fire is described below.

Prior to the start of fire exposure, the entire section is at ambient temperature as shown in Figure 1a. The section modulus is equal to its initial value. Shortly after the start of exposure, charring is initiated at the bottom edge directly exposed to the fire. The temperature distribution quickly approximates a one-dimensional profile as shown in Figure 1b. As long as the thickness of the remaining section, d, is equal to or greater than 40 mm, this profile for $x \le 40$ mm can be approximated as [10]

$$T(x) = T_a + \left[T_c - T_a\right]\left[1 - \frac{x}{a}\right]^2 \quad \text{(for } d \ge a \text{ and } x \le a) \tag{4}$$

where T = temperature (°C);
      x = distance from the char front (mm);
      T_a = ambient and initial temperature (20 °C);
      T_c = temperature at the char front (300 °C);
      a = thermal penetration depth (40 mm); and
      d = thickness of the remaining section (mm).

The validity of Equation (4) was confirmed by independent test data obtained in the U.S. [11]. If the thickness of the remaining plank section is less than 40 mm, the temperature profile is affected by the boundary conditions on the unexposed side as shown in Figure 1c. Conservatively assuming that there are no heat losses through the top plane, the following equation from the final draft of Eurocode 5, Part 1.2 [10] can be used to estimate the temperature distribution:

$$T(x) = T_a + \left[T_c - T_a\right]\left[1 - 2\frac{x}{a} + 2\frac{x^2}{ad}\right] \quad \text{(for } d < a \text{ and } x \le d) \tag{5}$$

The time to structural failure is determined in an iterative manner by solving Equation (2) using a bisection technique. Upper and lower bounds are determined of a range that contains the solution. Suitable lower and upper limits are 0 and D/β_0 respectively. To determine the transformed section modulus after an estimated t minutes of fire exposure, the remaining thickness is first calculated from
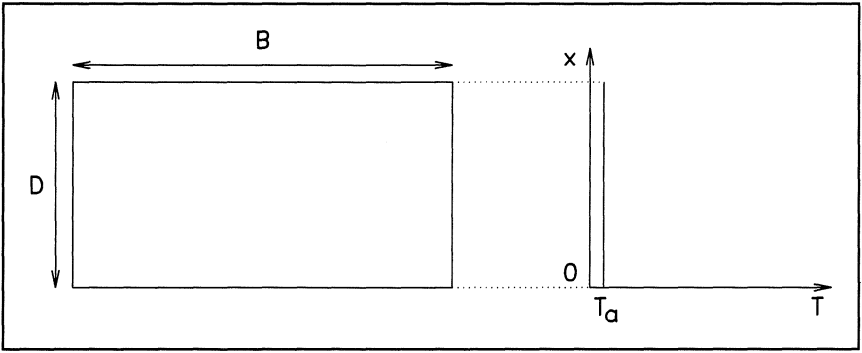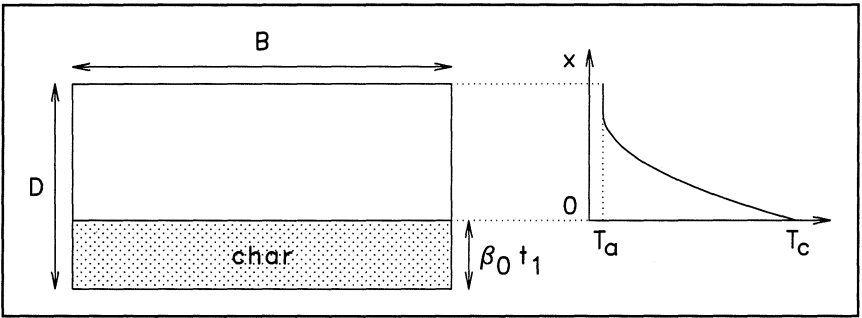
FIGURE 1a. Plank section before fire exposure

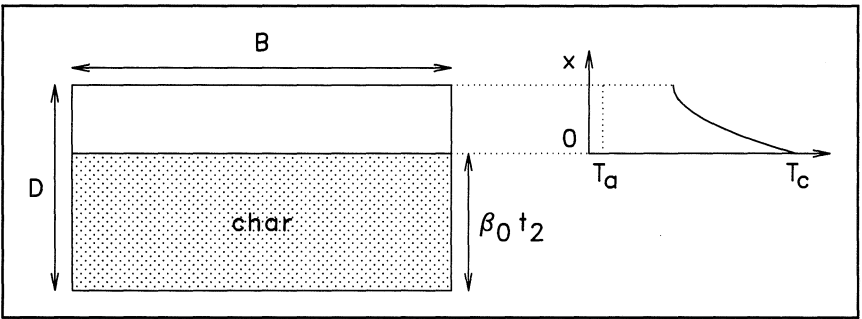**FIGURE 1b.** Plank section after t_1 min of fire exposure ($d=D-\beta_0 t_1 \geq 40$ mm)

FIGURE 1c. Plank section after t_2 min of fire exposure ($d=D-\beta_0 t_2 < 40$ mm)

$$d = D - \beta_0 t \tag{6}$$

This equation is valid for $t \geq 20$ min, and needs to be corrected for shorter exposure times to account for the initial heating and resulting ignition delay and increased rate of charring during the first 10-15 min following ignition. However, because structural failure times of less than 20 min are of little or no interest, no such correction is generally performed. The remaining section is subdivided into a number of horizontal slices with equal thickness $d_i \approx 1.6$ mm. The temperature of each slice is determined from Equations (4) or (5) as a function of the distance between the char front and the center of the slice. The width of each slice is adjusted according to

$$b_i = \frac{E(T_i)}{E(T_a)} B \tag{7}$$

where B = initial plank width (mm);
b_i = width of slice i (mm);
E = modulus of elasticity (GPa); and
T_i = temperature of slice i (°C).

The effect of temperature on the modulus of elasticity is estimated on the basis of recommendations in the literature [12], and is shown in Figure 2. Differences in E between the tension and compression zones are ignored. Figure 3 shows an example of a transformed section that is determined as outlined above. Before calculating the section modulus of the transformed section, it is verified whether the stress in the bottom slice exceeds the ultimate tensile strength adjusted for temperature. If it does, the bottom slice is assigned zero strength, and the verification is performed for the next slice inward. This process is repeated until only one slice is left, or until the slice does not fail. In the first case, the estimated failure time is obviously too high and is used as the upper limit of the solution interval for the next iteration. In the second case, the section modulus of the transformed section is calculated to determine which side of Equation (2) is the largest. If the LHS is larger, the estimated failure time provides a better value for the lower limit of the solution interval. If the LHS is smaller, it is a better value for the upper limit. A better estimate of the failure time is obtained as the midpoint of the new solution interval. This procedure is repeated until Equation (2) is fulfilled within a specified tolerance.

To obtain a design method for decks similar to the equations that are used in the U.S. for beams and columns, Equation (2) was solved for a range of load ratios and plank thicknesses. For this purpose, conservative values were assumed for β_0 (0.8 mm/min) and k_b (0.4). The results of the calculations are shown by the data points in Figure 4. The estimated failure times can be approximated as a linear function of plank thickness:
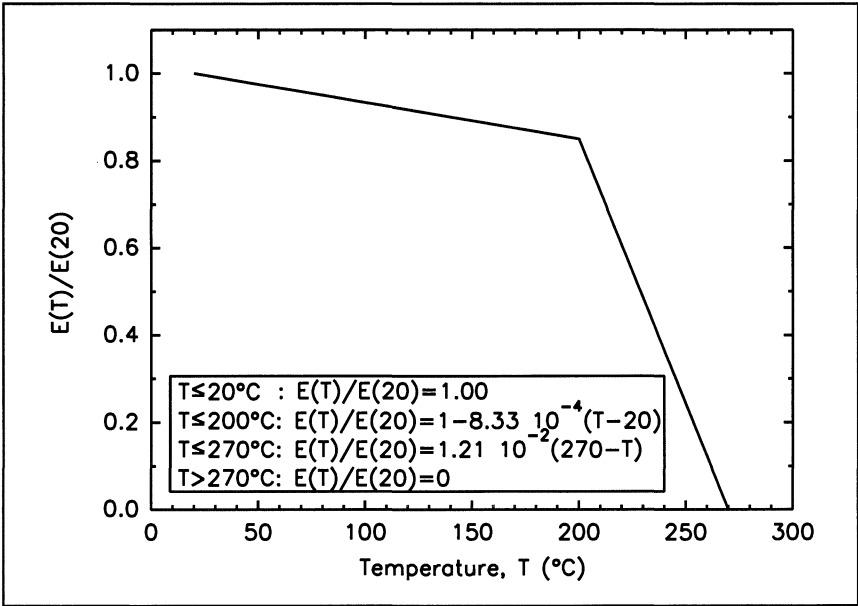
$$t_{sf} = C_1 D - C_0 \tag{8}$$

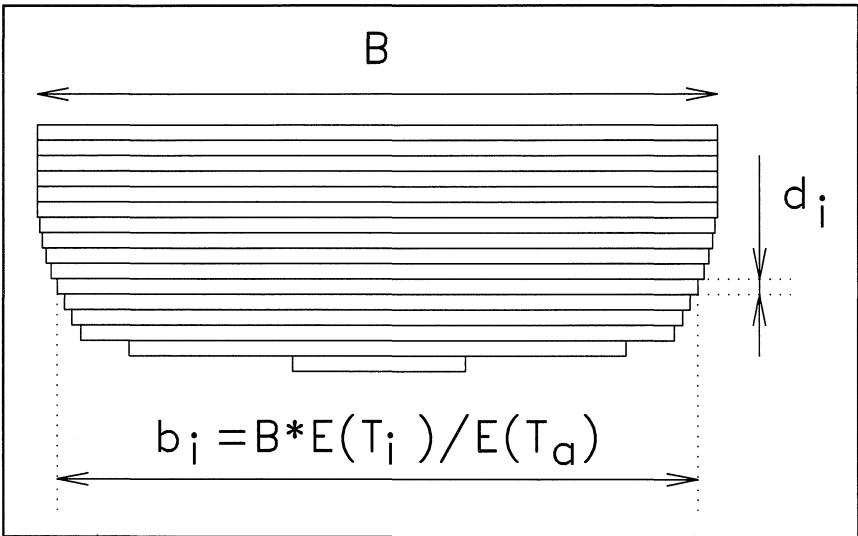**FIGURE 2.** Relative reduction of MOE as a function of temperature

FIGURE 3. Transformation of remaining section

where C_1 varies with R, and C_0 is nearly constant. The effective cross-section method in Part 1.2 of Eurocode 5 [5] can be used to obtain a functional relationship between C_1 and R, as described below.

The effective cross-section method accounts for the strength and stiffness reduction over the heated zone below the char layer by increasing the char depth so that

$$d = D - \beta_0 t - \delta \tag{9}$$

where the recommended value for δ is 7 mm. The remaining section is then assumed to have the initial strength and stiffness. Equation (2), using the effective cross-section method with unspecified δ, can be written as

$$\frac{Bd^2}{BD^2} = \left[\frac{D - \beta_0 t - \delta}{D}\right]^2 = Rk_b \tag{10}$$

This quadratic equation can be solved for t, leading to

$$t = \frac{D}{\beta_0}(1 - \sqrt{Rk_b}) - \frac{\delta}{\beta_0} \tag{11}$$

Finally, according to Equations (8) and (11), C_1 and C_0 can be expressed as

$$C_1 = \frac{1 - \sqrt{Rk_b}}{\beta_0} \quad \text{and} \quad C_0 = \frac{\delta}{\beta_0} \tag{12}$$

A linear fit according to Equation (8) through the data points that result from the transformed section analysis with $\beta_0 = 0.8$ mm/min and $k_b = 0.4$ for R between 0.05 and 1.00, give C_0 values ranging from 9.2 to 11.3 min. The highest value of C_0 is the most conservative, and corresponds to $\delta = 9$ mm for $\beta_0 = 0.8$ mm/min. Therefore, the following equation can be recommended for general design purposes

$$t_{sf} = 1.25D(1 - \sqrt{0.4R}) - 11.3 \tag{13}$$

The dotted lines in Figure 4 are predictions according to Equation (13). The lines are slightly below some of the calculated data points, indicating that the recommended simplified calculation method is generally slightly conservative compared with the detailed calculations.
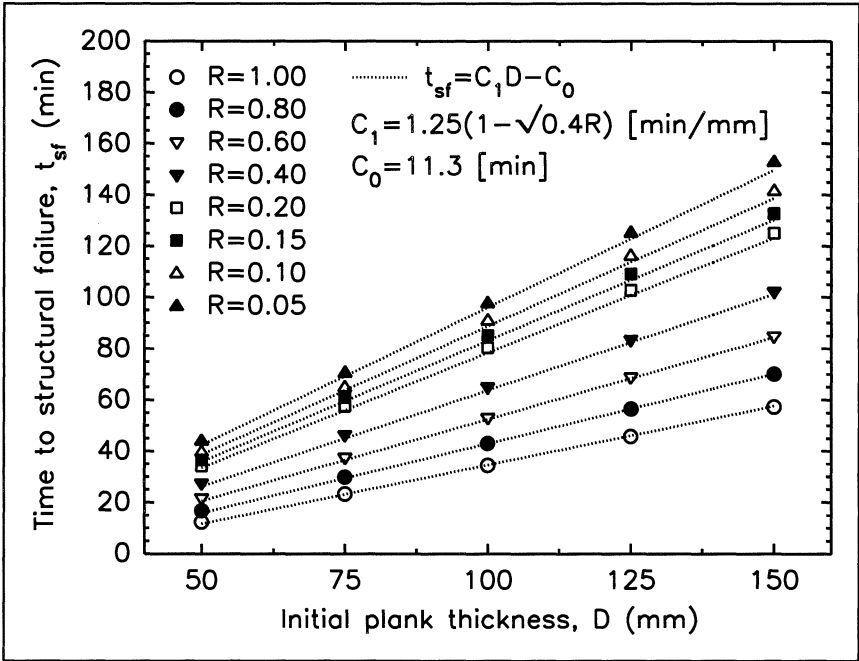
FIGURE 4. Structural fire resistance of exposed timber decks

COMPARISON WITH EXPERIMENTAL DATA

In 1964, Underwriters' Laboratories (UL) conducted a series of four tests on roof constructions for the Douglas Fir Plywood Association [13]. Two of the tests, referred to as #2 and #4, were on exposed timber decks consisting of 140 mm x 38 mm single tongue-and-groove Douglas fir planks. The decks were loaded to 46% and 59% of the design load for tests #2 and #4 respectively. The reported thermal penetration time was identical for the two tests at 20 min. Equation (1), with ξ=0.4, D=38 mm, and β_0=0.8 mm/min yields 19 min. First structural failure of a plank is not specifically mentioned in the report. However, for test #2 it is mentioned that deflection was noticeable (>32.5 mm at the center of the deck) at 13 minutes after the start of the test, and that the unsupported ends of some planks started to warp at 24 minutes. Equation (13) gives a value for t_{sf}=16 min, which is between the two reported times. For test #4, the observed times were 11 min and 18 min respectively, with t_{sf}=13 min according to Equation (13). The experimental data obtained at UL and calculated results are summarized in Table 2.

TABLE 2. Experimental data obtained at UL and calculated results

Test #	Thermal Separation		Structural Performance	
	Observed t_{tf} (min)	Calculated t_{tf} (min)	Observed t_{sf} (min)	Calculated t_{sf} (min)
2	20	19	13..24	16
4	20	19	11..18	13

In 1969, the American Iron and Steel Institute conducted a comprehensive experimental program at Ohio State University (OSU) [14]. The program included six tests on exposed Douglas fir floor decks. The first two decks, referred to as HT1 and HT2, consisted of 38 mm x 89 mm members on edge and covered with 19 mm wood flooring. Time to thermal failure at the joints was not reported, but time to flamethrough is a reasonable estimate of the end of thermal separation. Flamethrough for the two tests was reported at 61 and 69 min respectively. Equation (1), with ξ=0.4, D=108 mm, and β_0=0.8 mm/min yields 54 min. Although there were no tongues (or splines) and grooves, ξ=0.4 is appropriate because the flooring served a similar function as tongues (or splines) and grooves in preventing premature flow of hot gases and products of combustion through the joints. The first two decks were loaded at 31% of the design load, and structural failure was reported at 62 min and 56 min for HT1 and HT2 respectively. Equation (13) gives a value for t_{sf}=61 min, which is between the two reported times. The remaining four decks, referred to as HT3 through HT6, consisted of 140 mm x 64 mm planks, covered with 19 mm wood flooring. Flamethrough for the four tests was reported at 54, 31, 35, and 49 min respectively. Equation (1), with ξ=0.4, D=83 mm, and β_0=0.8 mm/min yields 41 min, which is equal to the average of the flamethrough times for all four tests. The HT3 and HT4 decks were loaded at 22% of the design load, and structural failure was reported at 54 min for HT3 (and not reported for HT4). Equation (13) gives a value for t_{sf}=45 min, which is significantly below the reported time. The HT5 and HT6 decks were loaded at 50% of the design load, and structural failure was reported at 45 min for HT6 (and not reported for HT5). Equation (13) gives a value for t_{sf}=33 min, which again is significantly below the reported time. Note that the fuel supply to the burners instead of the temperature-time curve in the furnace was controlled during the even-numbered tests. This resulted in slightly more severe exposure conditions than in the odd-numbered tests, which were conducted strictly according to ASTM E 119. The experimental data obtained at OSU and calculated results are given in Table 3.

CONCLUSIONS

A simple calculation method was developed to estimate the times to thermal and structural failure of timber decks exposed to standard ASTM E 119 fire conditions. The method is in reasonable agreement with experimental data obtained in the U.S. Because the method is generally conservative, it is suitable in its present form to be used for design purposes.

TABLE 3. Experimental data obtained at OSU and calculated results

Test #	Thermal Separation		Structural Performance	
	Observed t_{tf} (min)	Calculated t_{tf} (min)	Observed t_{sf} (min)	Calculated t_{sf} (min)
HT1	61	54	62	61
HT2	69	54	56	61
HT3	54	41	54	45
HT4	31	41	Not Reported	45
HT5	35	41	Not Reported	33
HT6	49	41	45	33

REFERENCES

[1] "ASTM E 119 - 88: Standard Test Methods for Fire Tests of Building Construction and Materials," in *ASTM Test Standards*, pp. 674-694, ASTM, Philadelphia, PA, 1993.

[2] "ISO 834: Fire-Resistance Tests - Elements of Building Construction," International Organization for Standardization, Geneva, Switzerland, 1975.

[3] Lie, T.T., "A Method for Assessing the Fire Resistance of Laminated Timber Beams and Columns", *Canadian Journal of Civil Engineering*, 4:161-169, 1977.

[4] "Design of One-Hour Fire-Resistive Exposed Wood Members," National Evaluation Board Report No. NRB-250, Council of American Building Officials, Country Club Hills, IL, 1984.

[5] "ENV 1995-1-2:1994: Eurocode 5: Design of Timber Structures - Part 1.2: General Rules - Structural Fire Design," European Commission for Standardization (CEN), Brussels, Belgium, 1994.

[6] "Plank and Beam Framing for Residential Buildings," National Forest Products Association, Washington, DC, 1989.

[7] "ASTM D 245 - 93: Standard Practice for Establishing Structural Grades and Related Properties for Visually Graded Lumber," ASTM, Philadelphia, PA, 1993.

[8] "1991 NDS® - National Design Specification® for Wood Construction," American Forest & Paper Association, American Wood Council, Washington, DC, 1991.

[9] Hibbeler, J., "Mechanics of Materials," pp. 281-287, McMillan, New York, NY, 1991.

[10] "Eurocode 5: Design of Timber Structures - Part 1.2: General Rules - Structural Fire Design," Final Draft, CEN, Brussels, Belgium, 1993.

[11] Janssens, M., and White, R., "Communication: Temperature Profiles in Wood Members Exposed to Fire," *Journal of Fire and Materials*, 18:263-265, 1994.

[12] "Structural Fire Protection: Manual of Practice," T.T. Lie ed., p. 43, American Society of Civil Engineers, New York, NY, 1993.

[13] "Fire Resistance of Heavy Timber and Plywood Roof Constructions," Underwriters' Laboratories File NC495, Chicago, IL, 1964.

[14] Bletzacker, R., Lane, W., and Denning, D., "Fire Resistance of Construction Assemblies - Volume I," Ohio State University, Columbus, OH, 1969.