Tensile Strength of Fire-Exposed Wood Members

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Abstract

As part of an effort to develop a fire endurance model for metal-plate-connected wood trusses, extensive testing was conducted on 2x4 lumber exposed to elevated temperatures. (Note: 2x4 refers to nominal 2- by 4-in. (standard 38- by 89-mm) lumber.) Two types of tests were conducted (a) constant temperature, increase load until failure, and (b) constant load, increase temperature until failure. The constant temperature exposures ranged from room temperature to 350°C for 30 or 60 min. The constant load tests involved two time-temperature curves, the ASTM E 119 curve and an idealized plenum curve. The constant temperature data were used to develop tensile strength thermal degrade models for Southern Pine and Spruce-Pine-Fir lumber exposed to fire. These simple models based on the temperature profile at a single point in time were able to provide reasonable estimates of the residual tensile strength of the lumber in the constant load-increase temperature tests.

Keywords: fire, tensile strength, 2x4 trusses, temperature

Introduction

Models to predict thermal degradation of the structural elements are an essential part of a tire endurance model. As part of the development of a fire endurance model for metal-plate-connected wood trusses (White and others 1993), we loaded 2x4 lumber to failure after exposing it to constant or elevated temperatures using specific time-temperature curves after applying a constant load. The constant temperature tests were conducted to obtain estimates for the parameters of the thermal degradation models. The data from the constant load tests were used to evaluate the models.

The overall testing program involved tests on 2x4 lumber with and without metal plate connectors. In this paper, only the results for tests on 2x4 lumber without metal plate connection are discussed. Results for tests on metal-plate-connected wood have been published (Shrestha and others 1995, White and others 1993, White and Cramer 1994).

Experimental Methods

The lumber was No. 1 Dense Southern Pine, KD 15 and Spruce–Pine–Fir (SPF), MSR 2100f - 1.8E. The lumber was tested dynamically for modulus of elasticity. Density and moisture content of each piece were determined. All materials were conditioned at 23°C, 50 percent relative humidity (9 percent moisture content) before testing as specified in ASTM E 119 (ASTM 1988).

The tension apparatus and furnace allow application of a tension load of up to 445 kN to a 4.9-m-long specimen. The central 1.8 m of the specimen is subjected to high temperatures or fire exposure. The tension machine is a modified tension proof tester consisting of a clamp assembly and a support frame. Tension is applied by two hydraulic cylinders, with oil supplied by a hydraulic power unit. The grips do not provide for any rotation about an axis. The rate of loading is regulated by using a manual valve to limit the flow of the hydraulic fluid. In the constant temperature tests, the initial strain rate was about 0.0003 mm/mm per min and the initial stress rate was about 3 MPa/min. (orientation of the specimen is such that the wider sides are vertical.

The furnace is lined with mineral fiber blankets and heated by eight diffuse–flame natural gas burners. All air for combustion is by natural draft through vents at the bottom of the furnace.

For the constant temperature tests, the specimens were heated in the furnace for either 30 or 60 min. Temperature levels included room temperture, 100°C, 200°C, 250°C, 275°C, 300°C, and 325°C. The heated specimens were allowed to lose moisture and were loaded while heated. Results were expressed in terms of maximum load and load-elongation curves. Some

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Table 1—Results of constant temperature tests.

Exposure T_s (°C)	t_e (min)	Specimens (n)	E_d (MPa)	ρ (kg/m³)	T_c (°C)	T_e^{a} (°C)	F_T Mean (MPa)	COV (%)	α Eq. 1
				Southern Pine					
24	—	7	15,030	518	24	24	35.2	18	1.00
100	30	9	15,634	539	90	94	38.3	36	1.09
200	30	9	14,762	560	132	155	23.4	40	0.66
200	60	4	15,844	582	156	171	24.0	13	0.68
250	30	12	15,875	582	159	190	15.8	40	0.45
250	60	5	15,720	602	210	224	11.6	35	0.33
275	30	5	14,892	526	161	199	12.4	24	0.35
300	30	11	15,124	546	193	228	8.9	55	0.25
300	60	4	15,325	531	325	317	2.4	100	0.07
325	30	5	15,584	566	188	233	8.0	41	0.23
				Spruce–Pine–Fir					
21	—	10	13,548	—	21	21	34.7	20	1.00
100	30	10	13,352	—	88	92	34.8	18	1.00
200	30	10	13,744	—	137	158	19.2	24	0.56
250	30	10	13,267	—	154	186	14.3	18	0.41
275	30	10	13,337	—	159	198	9.9	15	0.28
300	30	9	13,670	—	165	210	9.9	24	0.28

[a]Eq. (3).

specimens tested at 300°C and 325°C ignited before the load could be applied. Number of replicates ranged from 4 to 12 (Table 1).

A thermocouple attached to the wide side of the 2x4 lumber was used to control the furnace temperature. A second thermocouple was inserted in a hole in the lumber so that the bead was at the center of the cross-section of the specimen.

The duration of initial heating is complicated by the effect of time on the strength properties of wood as opposed to the insulative quality of wood, which prevents rapid heating. Heating for 30 min allowed the center of the specimen to exceed 100°C for all but the 100°C exposure series. Thus, the specimens can be assumed to be without moisture. Heating for 30 min was also consistent with the likely duration of heated exposure in a l-h ASTM E 119 test of a protected truss assembly. The tests with 1 h of heating provided a more uniform temperature profile.

For the constant load tests, the specimen was first loaded to a specified load level (50 or 100 percent of design load). The specimen was then subjected to a "plenum" exposure, which represented typical temperatures in the plenum of a protected truss assembly in the ASTM E 119 test, or an "E 119" exposure, which involved direct exposure to the ASTM E 119 time–temperature curve. We derived the plenum time–temperature curve from various ASTM E 119 test results for protected truss assemblies. Some temperatures on the plenum curve were 65°C, 93°C, 188°C, 260°C, and 327°C at 10, 20, 30, 45, and 60 min; respectively. The temperature rise in the curve after 45 min was 22.2°C per 5 min. As in the constant temperature tests, a thermocouple attached to the side of the lumber was used to control the furnace temperature in the plenum tests. The ASTM E 119 time–temperature curve is 704°C, 795°C, 843°C, 892°C, and 927°C at 10, 20, 30, 45, and 60 min, respectively. Standard thermocouples in enclosed pipes were used to control the furnace in the E 119 tests. Results from the constant load tests were time–elongation curves and times of failure.

Thermal Degrade Models

The basic thermal degrade model assumes that structural failure of the heated structural element occurs when the applied load exceeds some fraction of the ultimate load capacity at room temperature. Thus, the thermal degrade factor α is

$$\alpha = \frac{F_T}{F_0} \tag{1}$$

where
F_0 = ultimate tensile stress at room temperature, MPa
F_T = ultimate tensile stress of heated element, MPa

The thermal degrade factor can be expressed as a function of fire endurance time or as a function of the structural element's characteristics at a given time. These characteristics can include temperature, duration of elevated temperatures, mass loss, applied load, species, and similar features of the structural element. When the thermal degrade factor is expressed as a function of fire endurance time, it is for a specific fire exposure, such as the ASTM E 119 time–temperature curve. The thermal degrade factor can be a single empirical value applied to the load capacity of the sructural element or multiple values applied to layers or other divisions of the cross-sectional area of the element (White 1995).

One model defined in terms of the duration of fire exposure is the model developed by Woeste and Schaffer (1981) for an unprotected wood truss floor. This model was limited to evaluating the residual strength of individual wood components. A similar model was used for exposed wood joists (Woeste and Schaffer 1981, Schaffer and others 1988). For the purpose of this paper on tensile strength, the model can be reduced to

$$\alpha = \frac{f_T}{F_0} = \frac{P/(b-2Ct_f)(d-2Ct_f)}{F_0} = \frac{1}{1+\gamma K t_f} \tag{2}$$

where

f_T = applied tensile stress, MPa
K = 2(b+d)/bd for exposure on four sides, mm^2
γ = thermal degrade parameter, mm/min
P = axial tensile force due to applied load, N
b = width of chord, mm
d = depth of beam, mm
C = char rate, mm/min
t_f= time to failure, min

In Annex A of the Eurocode 5 on a reduced strength and stiffness method for standard fire exposure, the section factor K of the residual cross-section is used to determine the modification factor for strength and stiffness properties (Konig 1994). The relationship represents a simplified measure of the temperature in the residual cross-section and the effect of creep.

Of more general use is a thermal degrade factor based on the structural element characteristics. The most simple approach is to define the thermal degrade factor in terms of temperature of the structural element. Assuming the temperature profile within the member can be represented by a parabolic curve, one possible term for the element temperature (T_e, C) is derived from the trapezoidal rule for the area under a parabolic curve.

where
T_s = surface temperature, °C
T_c = center temperature, °C

An equation for the thermal degrade factor using this mean temperature is

$$\alpha = 1 - \exp\left(D - \frac{E}{T_{eK}}\right) \tag{4}$$

where
T_{eK} = element temperature in absolute temperature, K (Eq. 3)
D, E = empirical parameters

In the model developed for structural analysis of wood trusses exposed to fire (SAWTEF) (Shrestha and others 1995), the thermal degrade models for wood included the duration of heating by including a term equal to the area under time–temperature curve for T_e minus room temperature. The model is based on an initial subset of the data discussed here. The approach of Lau and Foschi (1994) considers the histories of the applied stress and the temperature. In tests with up to 2 h of heating, Schaffer (1973) found that tensile strength decreases as a result of the duration of heating when wood is exposed to temperatures above 140°C. No effect was found at lower temperatures. We did not consider the approaches of Lau and Foschi (1994) and Shrestha and others (1995) in this analysis since our interest was in simple models based solely on the temperature profile at a single point in time.

For a more flexible model to account for the effects of the temperature distribution, the load capacity of a tensile element can be expressed as the sum of the load capacity of layers or divisions of the cross-sectional area of the structural element.

$$F_T = \sum F_i \frac{A_i}{A_t} = \sum \alpha_i F_0 \frac{A_i}{A_t} \tag{5}$$

where

F_T = ultimate tensile stress of structural element at elevated temperature T, MPa

F_i = ultimate tensile stress of layer or section i of cross-sectional area of element with temperature T_i, MPa

α_i = thermal degrade factor for layer or division i of cross-sectional area of structural element

A_i = area of layer or division i of cross-sectional area of structural element, mm^2

A_t = total area of cross section of structural element, mm^2

Similar models for load capacity of a fire-exposed wood beam involve transformed section analysis (Bender and others 1985).

For the purpose of obtaining estimates for α_i, we divided the cross-sectional area into 100 equal rectangles (10 by 10 grid) and calculated the temperature T_i as

$$T_i = (T_s - T_c)(y^2 - x^2y^2 + x^2) + T_c \qquad (6)$$

where the coordinates x,y are zero at the center and 1 at the surface. Based on the calculated temperatures, the rectangles were assigned to one of five temperature (°C) ranges (25–50, 50–100, 100–150, 150–200, 200–250, 250–350) and the corresponding cross-sectional areas, A_i (A_{35}, A_{75}, A_{125}, A_{175}, A_{225}, A_{300}), were calculated for each temperature range. Regression analysis of the constant temperature data was used to obtain estimates for the coefficients for Equation (5).

Results and Discussion

The surface and center temperatures of the specimens were recorded (Table 1). From those measurements, the element temperatures were calculated. There are some limitations to the accuracy of these measurements. Accurate surface measurements are difficult to obtain. We attempted to maintain contact between the thermocouple bead and the surface of the wood. With significant charring, this is soon not possible with the ASTM E 119 exposure. The leads to the recording devices were exposed to the elevated temperatures of the furnace interior. This results in the potential for conduction of heat along the thermocouple wire to the bead. Thus, the center temperatures recorded could be higher than the actual wood temperatures. These limitations were not shown to be significant in tests conducted specifically to obtain more complete data on the temperature profile within the member.

Average results for the constant temperature tests indicated reductions in the ultimate tensile stress (F_T) with higher element temperature (T_e, Eq. (3)) (Table 1). Longer duration of heating (t_e) resulted in higher element temperature. In the calculation of the thermal degrade factors of Table 1, the mean ultimate tensile stress of the specimens tested at room temperature was used as the estimate of F_o for all the tests. Data provided in Tables 1 and 2 include the modulus of elasticity (E_d) and density (ρ). It should be noted that these data are for lumber that had 9 percent moisture content at the beginning of the tests.

There was considerable variability in the data (coefficients of variation of 13 to 100 percent). The small increase in strength at 100°C for the Southern Pine lumber likely reflects this variability. Some of the increase in tensile strength could be due to lower moisture content. Based on the element temperatures, the residual strength of the specimens heated for 60 min was higher than that of specimens heated for 30 min (White and others 1993). However, the 60-min data are within the scatter of the 30-min data.

Regressing the thermal degrade factors with the mean element temperatures (Eq. 3) resulted in the following equation for all the data:

$$\alpha = 1 - \exp(2.67 - \frac{1523.6}{T_{eK}}) \qquad (7)$$

As discussed before (Eq. 4), an alternative model is to assume the factor is 1 for T_e less than 90°C, 0.1 for T_e greater than 240°C, and the following linear equation for the intermediate temperatures:

$$\alpha = 1.62 - 0.00633T_e \qquad (8)$$

with r^2 of 0.68 and coefficient of variation of the estimates (CV) of 31 percent.

The model based on Equations (5) and (6) and all the data was as follows:

$$\alpha = 1.00A_{35} + 1/05A_{75} + 1.06A_{125} + 0.54A_{175} + 0.36A_{225} + 0.10A_{300} \qquad (9)$$

with CV of 29 percent. Equations similar to equations (7) to (9) were obtained when the Southern Pine and the SPF data were analyzed separately. Variations of this model can be obtained by using different ranges of temperature. The coefficients will be different depending on the ranges used to calculate the A_i values of the elements.

As expected, higher loads in the constant load tests reduced failure times (t_f) (Table 2). Based on the surface and center temperatures at the time of failure, Equations (7), (8), and (9) were used to obtain estimates of the corresponding α at time of failure (Table 3) in the

Table 2—Results of constant load tests.

Species	Exposure	Load (N)	Specimens (n)	E_d (MPa)	ρ (kg/m³)	t_f Mean (min)	t_f COV (%)
S. Pine	E 119	14,595	6	13,493	539	12.9	17
S. Pine	E 119	29,189	5	14,739	534	9.9	17
S. Pine	Plenum	14,595	5	14,795	533	69.7	2
S. Pine	Plenum	29,189	6	14,649	537	61.1	10
SPF	Plenum	18,389	5	13,395	—	67.8	10
SPF	Plenum	36,778	5	13,073	—	54.2	12

Table 3—Predictions for constant load tests.

Species	Exposure	Load (N)	T_e at failure (°C) Mean	T_e at failure (°C) Range	f_t / F_o	α Eq. 7	α Eq. 8	α Eq. 9	γ Eq. 2 (mm/min)
S. Pine	E 119	14,595	—	—	0.12	—	—	—	2.0
S. Pine	E 119	29,189	—	—	0.24	—	—	—	1.3
S. Pine	Plenum	14,595	277	255-289	0.12	0.09	0.10	0.10	1.4
S. Pine	Plenum	29,189	249	225-271	0.24	0.22	0.12	0.16	0.6
SPF	Plenum	18,389	261	238-276	0.16	0.17	0.10	0.14	1.0
SPF	Plenum	36,778	197	175-210	0.31	0.44	0.38	0.33	0.5

plenum tests, As discussed reliable surface and center temperature data were not obtained in the E 119 tests. Both Equations (7) and (9) provided estimates for α (f_t/F_o) at time of failure that were in reasonable agreement with α calculated from the applied load (Table 3). In the application of these models, there is a method to predict the center temperature of a lumber section when the exposure history of its surface is known (Shrestha and others 1994).

In Equation 2, the endurance time and the residual cross-section are used as a measure of the elevated temperature. Using the constant load data estimates for the thermal degrade parameter γ of Equation (2) were calculated (Table 3). For E 119 tests, a charring rate of 0.76 mm/min was assumed. No charring was assumed for the plenum tests. The estimates for γ for the different load levels were not the same. This indicates that these data do not support the use of Equation (2) to predict the fire endurance of wood truss components. Other variations of the time-to-structural-failure model (Woeste and Schaffer 1981) may provide more consistent estimates for the parameters over different load levels by providing a better measure of the elevated temperature. In other tests with the E 119 exposure, Southern Pine Select Structural lumber (92 by 41 mm cross section) loaded to 83 percent of allowable stress (17 percent of estimated ultimate stress) failed in 10.0 to 12.85 min (Woeste and Schaffer 1981). These results are consistent with the results in Table 2. The estimate for γ for these Southern Pine tests was 2.1 mm/min. Data for tests of Douglas Fir-Larch Select Structural lumber (92 by 41 mm cross section) (Woeste and Schaffer 1981) resulted in γ estimates of 2.6 and 2.8 for members loaded to 16.8 and 20.6 percent of the estimated ultimate stress. With the similar model for wood joists (Schaffer and others 1988), the estimates for γ were fairly close (5.6 and 4.6 mm/min) for a 535-percent increase in the applied load.

Conclusions

Lumber exposed to short-term heating (30 to 60 min) above 150°C experiences great losses in tensile strength. Short-term heating at 100°C does not result in measurable losses in tensile strength. Simple models based on the temperature profile of the lumber at a single point in time were able to provide reasonable estimates of residual tensile strength.

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