FOCUS ARTICLE

SIMPLIFIED MODELS FOR THE PROPERTIES OF DIMENSION LUMBER AND METAL-PLATE CONNECTIONS AT ELEVATED TEMPERATURES

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ABSTRACT

Structural analysis of an assembly in a fire situation differs from conventional static analysis in that the mechanical properties of the components of the assembly vary with the exposure conditions and duration of exposure. A key factor in fire-endurance modeling of a structural assembly is the successful modeling of its component properties at the exposure environment. This paper introduces an efficient and simplified technique of modeling the mechanical properties of dimension lumber and metal-plate connectors at temperatures that prevail in the plenum area of protected roof/floor-ceiling assemblies. These models serve as a useful starting point for modeling the degradation of lumber and connections exposed to elevated temperatures, but further advancement of the models to better account for the complexities of lumber exposed to combined high temperatures and loads is warranted.

Metal-plate-connected wood truss assemblies have gained wide popularity because of their cost effectiveness and versatility. Building codes require that different components of a building have fire-endurance ratings depending upon their use and type of construction. Fire-endurance ratings of wood assemblies have traditionally been evaluated from the performance of a full-scale replica of the assembly in a standard fire endurance test as outlined in ASTM E 119 (1) or other equivalent standards. The test procedure is expensive, time consuming, and design alterations in a fire-rated assembly are generally not permitted. The empirical nature of these standard test procedures does not lead to better understanding of the behavior of the wood assemblies in fire situations. A numerical model can be a useful aid in understanding and evaluating the fire endurance of an assembly.

There has been only limited progress in the analytical fire-endurance modeling of wood assemblies in recent years. This is partly because of the immense task of fully characterizing the very complex mechanical behavior of wood of various species and moisture contents, and various connections at different exposure conditions. Static structural analysis of a metal-plate-connected wood truss at high temperatures differs from the analysis at room temperature in that the mechanical and physical properties of the members vary with the exposure conditions and duration of exposure, and the assembly may undergo large deformation before failure (6). If the member characteristics can be predicted continuously for a given fire exposure condition, then the analysis procedures can be largely the same as for a static nonlinear analysis. The objective of this paper is to introduce practical models to predict the response of dimension lumber and metal-plate joints exposed to pyrolytic temperatures. A new tool for the fire-endurance modeling of a wood truss resulted when these models were integrated with a structural analysis package (5,30,34).

BACKGROUND

Wood is a cellulosic material that when exposed to a sufficient heat source will undergo thermal degradation (pyrolysis) resulting in the loss of mechanical properties. Roberts (26) and Schaffer (27,28) summarized the thermally induced changes in dry wood in an inert atmosphere for a temperature range of 50°C (122°F) to 400°C (752°F). The

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loss in the mechanical properties of wood increases with the energy absorbed, and it can be temporary or permanent depending upon the duration and severity of exposure (7,29). Gerhards (10) reviewed and compared several different studies on the effect of moisture and temperature on the mechanical properties of wood. Noren (22) concluded that lumber grades have a negligible effect on bending strength at elevated temperatures. Traditionally, the characteristics of wood properties at high temperatures have been based on test data of small defect-free specimens (12,27,32). It is not clear how these results of small defect-free specimens exposed to elevated temperatures for short durations can be correlated with the properties of structural lumber exposed to high temperatures for longer durations. Related work by Lau et al. (15) is in progress.

Different approaches have been used to evaluate the load-canying capacity of wood members exposed to fire. Knudson and Schniewind (12) modeled the section as a composite of small elements, each having distinct properties based on the temperature profile. King and Glowinski (11) assumed a section to be composed of different layers having different properties based on the temperature profile, and the load capacitv of the section was based on the total load capacity of each distinct layer. Odeen (23), Lie (16), Schaffer (29), and Woeste and Schaffer (35), used elementary beam theory along with the reduced strength and char effects as a result of tire exposure. Springer and Do (32) related the mechanical properties of a section to the mass loss induced by heat.

The performance of a plate-wood connection at elevated temperatures depends on the performance of the metal plate, the wood beneath the plate, and other factors (14,17,25). Structural steel (A36 steel) loses about 30 percent of its original strength and stiffness properties by approximately 500°C (932°F), and then the loss increases rapidly with the temperature (20). Sheet metal is likely to behave similarly at elevated temperatures. Recent experimental work (33,34) at the USDA Forest Products Laboratory (FPL) indicated that the metal plates serve as a protective layer by reflecting some radiation while the teeth conduct thermal energy to even greater depth. The plated surface showed signs of delayed charring. Temperature measurements indicated that the temperature immediately beneath the plate was lower than the exposed surface temperature, while the inner-wood cross-section temperature was higher at the plated region than at the inner cross section of the exposed section away from the joint.

EXPERIMENTAL WORK

The mechanical properties required in a detailed, finite-element-based analysis of metal-plate-connected wood trusses are longitudinal modulus of elasticity (MOE), modulus of rupture (MOR), tensile strength, compressive strength, and stiffness parameters of the metal-plate connector. As the exposure temperature of the truss assembly increases, these properties degrade as a function of exposure conditions and duration of exposure. Predictions of these properties are essential at regular time intervals in order to analyze the truss for a given exposure condition. For this purpose, a series of tension and combined tension-bending tests were conducted on dimension lumber and metal-plate-connected tension splices. The specimens consisted of No. 1D Southern Pine, actual 38- by 89-mm (nominal 2- by 4-in.) dimension lumber (hereafter called 2- by 4-in.). The metal-plate connectors were comprised of 20 gage (0.9 mm) Grade A steel, with 3.2-mm- (1/8-in.) wide and 8.5-mm- (1/3 -in.) long teeth punched to have an average tooth density of 14,570 teeth/m^2 (9.4 teeth/in.^{2}). The major axis of each plate was oriented parallel or perpendicular to the longitudinal direction of lumber. The plate sizes used were 76- by 191-mm (3-by 7.5-in.) for the parallel orientation, and 178- by 76-mm (7-by 3-in.) for the perpendicular orientation. All lumber specimens were tested nondestructively for dynamic MOE. The test specimens were conditioned at 22°C (72°F) and 50 percent relative humidity before the test to achieve a lumber equilibrium moisture content of 12 percent.

The tests were conducted in a special tension/furnace apparatus (34), where a furnace was housed between the frames of a tension apparatus such that the middle 1.83 m (6 ft.) of the specimen length was subject to a specified time-temperature relationship. The time-temperature condition for each test was either constant temperature with increasing load or constant load with increasing temperature, Constant temperature tests consisted of heating the specimens under constant temperature for 30 minutes and then applying a ramp load to failure. The number of test replicates varied through the course of the test program, but generally 5 to 10 replicates were tested at each temperature level of ambient (22°C; 72°F), 100°C (212°F), 200°C (392°F), 250°C (482°F), 300°C (572°F), and 325°C (617°F).

In the second type of test, the specimens were subjected to a constant load of 50 or 100 percent of the design load, and then were exposed to a predefined time-temperature environment. The lumber design values were obtained from the 1986 National Design Specification for Wood Construction (21). The plate design values were obtained from the evaluation report for these particular proprietary steel gusset plates published by the ICBO Evaluation Service, Inc. (subsidiary of the International Conference of Building Officials, Whittier, Calif.) based on information provided by the manufacturer. Several replicates were tested for these situations. The most severe exposure consisted of the time-temperature curve from the ASTM E 119 test standard (1). A less severe exposure simulated temperatures measured in the plenum of a gypsum-protected truss assembly subjected to the ASTM E 119 test conditions. The idealized plenum exposure condition consisted of the exposure temperatures 65°, 93°, 188°, 260°, and 327°C (149°, 199°, 370°, 500° and 621°F) at 10, 20, 30, 45, and 60 minutes, respectively. These tests provided load-deformation and time-to-failure data.

DEGRADATION OF LUMBER PROPERTIES

Our primary goal was to develop an efficient model for the temperature-induced degradation of the components of a metal-plate-connected wood truss that can be integrated with a structural analysis package (4,5,30), while maintaining the accuracy of the model within the bounds of experimental results. The models developed here were based on the available test results for limited test conditions while targeting the factors that are likely to influence the degradation process.

Thermally induced changes in wood are dehydration, shrinkage, thermal ex-

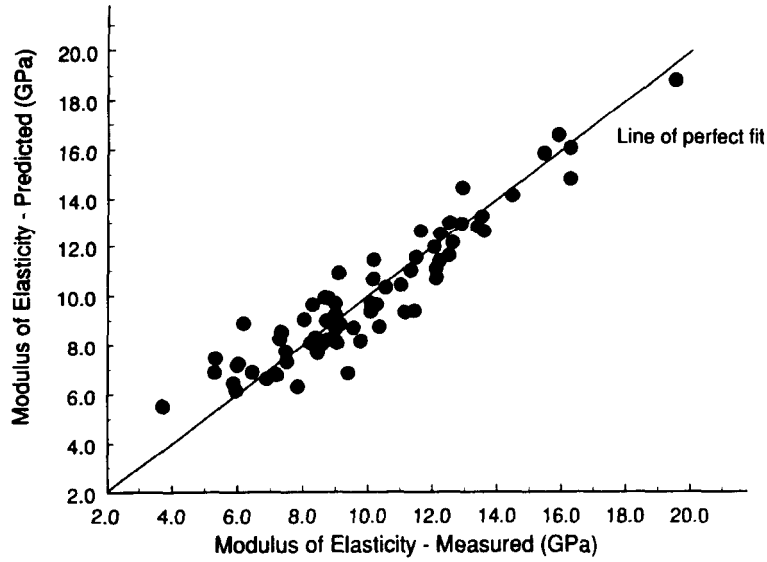

Figure 1. — A comparison of predicted (Eq. [1]) and observed longitudinal MOE of 38- by 89-mm (2- by 4-in.) lumber.

pansion, and phase change. Wood becomes increasingly viscoelastic at elevated temperatures and exhibits nonlinear creep deformation (27). All these factors influence the degradation process of dimension lumber to varying degrees depending upon the severity and duration of exposure, and the stress level of the member. These changes and behaviors should be included in a more complete and refined model to accurately predict the behavior of lumber at elevated temperatures. Modeling this very complex process in detail requires a complex model and the objective here was to develop simple, usable, and credible models without unnecessary detail. All of these complex factors are implicitly included in our simplified models, as the models introduced are based on test data that incorporate these factors to a limited degree. We have attempted to maintain similar levels of detail and rigor across all aspects of the submodels.

We hypothesized the degradation of wood properties to depend on total exposure time, exposure temperature, dimensions, exposed surface area, density, moisture content, thermal properties, and applied stress. Based on our test data, the degraded mechanical properties of dimension lumber after being exposed to elevated temperatures at known exposure conditions for a specified duration can be related to the corresponding properties at ambient temperature as

$$\frac{W_t}{W_o} = \frac{T_o\, t}{T_o\, t + A_t\, \gamma} \qquad [1]$$

where:

W_o = wood properties like longitudinal MOE (MOE_l), modulus of rupture (MOR), tensile strength, and compressive strength

W_t = corresponding wood property W_o after exposure

t = time of exposure (min.)

T_o = room temperature (°C)

T_c = temperature at the center of a section in (°C)

γ = empirical function selected to yield visual best fit between the observed and predicted data. From the FPL test data we derived the following functions to be satisfactory:

$T_c^{0.05}/100$ for E_l

$(T_c/198)^4$ for tensile and compressive strength

$(T_c/215)^6$ for MOR

A_t = area under the time-temperature profile for the member

This model provides estimates of MOE, flexural, tensile strength, and compressive strength of wood members at any time for a known exposure condition. The FPL test program did not include compression tests. Hence, the compressive strength was assumed to degrade in the same fashion as the tensile strength. Published literature (12,29) suggests some similarity in the degradation pattern of the tensile and compressive strengths of defect-free small wood specimens, but future test data will allow refinement of this simplification (15).

Equation [1] reflects a dependency on exposure time and thermal response of the member, while the empirical function is determined from the test data. The function considers creep, shrinkage, expansion, phase change, and other effects not explicitly treated in our simplified approach. The internal thermal response of the member must be established and can be computed from a simplified heat transfer model (30,31), an elaborate numerical solution (9,13,24,26), or measured from component tests under similar exposure conditions. Our investigations indicated that when a lumber section is exposed to elevated temperature, the center temperature gradually increases to 100°C (212°F), followed by a plateau as the moisture present evaporates, followed by rapid temperature rise as the section pyrolyzes and chars. The temperature profile across the section is approximately parabolic in shape, which gradually flattens as the entire section reaches a thermal equilibrium state (30,31,34). The cumulative area under the temperature profile from each step in time can then be approximated by

$$A_t = \sum_{t=0}^{t} \frac{1}{3}\,(T_s - 3T_o + 2T_c)\,\Delta t \qquad [2]$$

where:

t and Δt = total and incremental time in minutes, respectively

T_o, T_c, and T_s = initial, center, and surface temperature (°C) of the section, respectively

Equation **[1]** was evaluated from data gathered from a series of tension tests and combined tension-bending tests. **Figures 1, 2,** and **3** show plots of the measured versus predicted properties. The measured properties were obtained from the test results, while the corresponding predicted values were evaluated from Equation **[1],** using the

center temperature and initial properties from the test data.

For **Figure 1,** the moduli of elasticity at different exposures were computed from the observed load-displacement data obtained from the tension test results. The room temperature modulus of each specimen was obtained from a dynamic vibration modulus multiplied by a factor of 0.943 to adjust the dynamic modulus to a static MOE (30). The predicted and observed MOE data have a correlation coefficient of 0.94.

Two types of loading conditions were used to evaluate lumber strength, combined tension-bending and uniaxial tension. The combined tension-bending tests consisted of a tensile load applied at an eccentricity of 140 mm (5.5 in.) to the specimen longitudinal centerline. The stress across a section subjected to combined axial and bending forces is a combination of the axial and flexural stresses at the section and additional secondary stress resulting from the beam deflection. In reality, the stress distribution is probably not linear due to a temperature gradient across the section during fire exposures and the stress distribution is also disrupted by defects. Due to the limited test data available and the resulting uncertainties, these nonlinearities were ignored and a linear stress-strain distribution was used to compute the apparent maximum stress. **Figure 2** shows a plot of the measured versus predicted extreme fiber stress resulting from the combined loading. The scatter in **Figure 2** is due to the difficulties and uncertainties in evaluating the initial strengths and defects present. Some premature failures at the test device grips during the combined tension-bending tests suggest the influence of stress concentration in the gripping region of the specimens. Excluding the premature failure data, a linear regression analysis of the predicted and observed maximum stress at different temperatures shows a correlation coeffiient of 0.84.

For **Figure 3,** the room temperature tensile strengths were correlated with the average strength and strength-ratio factors estimated from the defects present in the specimen in accordance with ASTM D 245 (2). Including these factors, the regression of the predicted data improved to a correlation coefficient of 0.97.

DEGRADATION OF PLATE STIFFNESS PARAMETERS

Metal-plate connections exhibit a nonlinear load-slip behavior where the displacement/slip at the plate-wood interface increases exponentially with the applied load. A three-parameter equation presented by Foschi (8) has been accepted as a pseudo-standard to model the behavior of metal-plate joints. Shrestha (30) and Cramer et al. (4) noted the difficulty in the Foschi equation for certain applications and introduced an alternative equation (Eq. **[3])** while retaining the same three parameters of the Foschi equation.

$$F = \frac{M_o}{1.5}\tan^{-1}\left(\frac{1.5(K - M_1)\,\Delta}{M_o}\right) + M_1\Delta \qquad [3]$$

The constants K, M_o, and M_1 are plate stiffness (Foschi-type) parameters, F refers to force, and Δ to deformation of the joint.

The three parameters are selected such that Equation **[3]** will closely match experimental data. Experimental

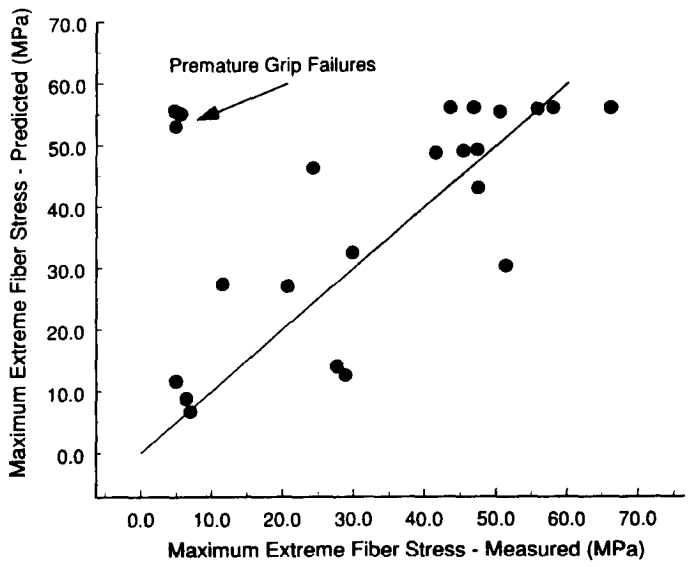

Figure 2. — A comparison of predicted (Eq. [1]) and observed maximum extreme fiber stress (tension-bending loads) of 38- by 89-mm (nominal 2- by 4-in.) lumber.

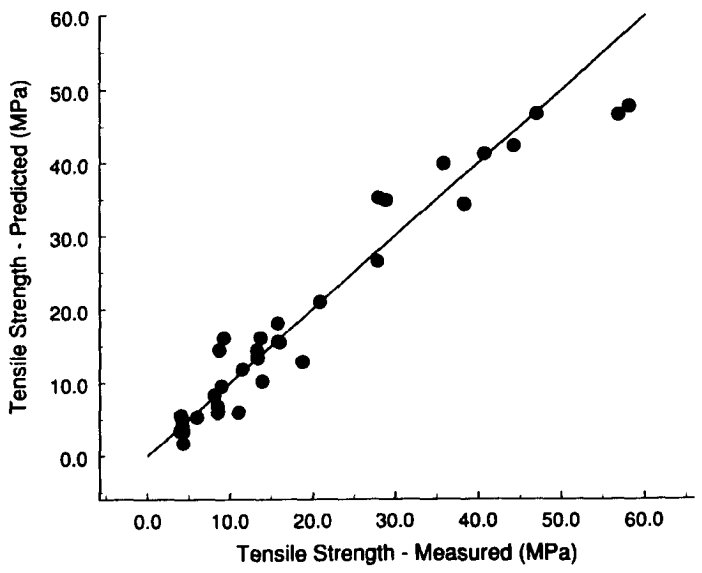

Figure 3. — A comparison of predicted (Eq. [1]) and observed maximum tensile strength of 38- by 89-mm (nominal 2- by 4-in.) lumber.

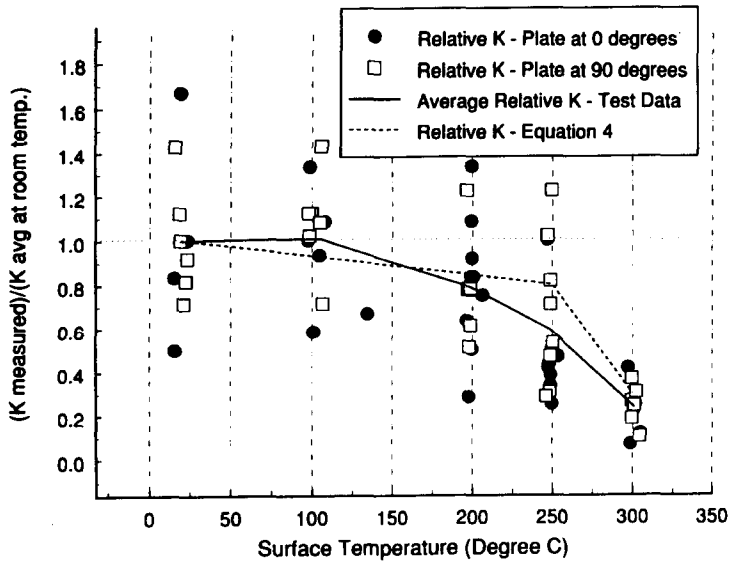

Figure 4. — A comparison of plate stiffness parameter K at different exposure temperatures.

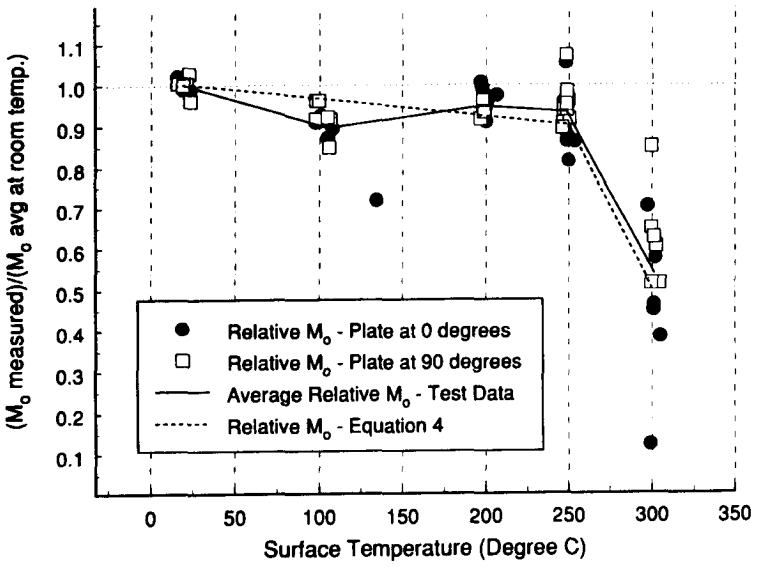

Figure 5. — A comparison of plate stiffness parameter M_o at different exposure temperatures.

data to evaluate the parameters K, M_o, and M_1 are typically gathered in accordance with CSA S347-M1980 (3) for a given plate type and wood species. These parameters have been extensively investigated at ambient temperature (14,17,18,19); however, little is known about the behavior of metal-plate connections at elevated temperatures. In this research, load-deformation data of metal-plate connections at different constant temperatures were gathered to evaluate the effect of temperature on the connection stiffness parameters.

From the load-slip data for the metal-plate connections at different exposure temperatures, the plate stiffness parameters were selected such that Equation **[3]** generated a curve that closely fit the test data. The stiffness parameters evaluated at different exposure temperatures indicated that the degradation of the plate stiffness parameters can be modeled as bilinear functions of the exposed surface temperature as shown in Equation **[4]**:

$$\frac{K^t}{K^o} = 1.0 - 0.2\frac{T_s - T_o}{200 - T_o} \quad \text{for } T_s \le 200°\text{C}$$

$$= 0.8 - 0.5\frac{T_s - 200}{100} \quad \text{for } T_s > 200°\text{C}$$

$$\frac{M_o{}^t}{M_o{}^o} = 1.0 - 0.1\frac{T_s - T_o}{250 - T_o} \quad \text{for } T_s \le 250°\text{C}$$

$$= 0.9 - 0.4\frac{T_s - 250}{50} \quad \text{for } T_s > 250°\text{C} \qquad [4]$$

K and M_o are average plate stiffness parameters and the superscripts o and t refer to the properties at room temperature (T_o) and exposure temperature (T_s), respectively.

Figures 4 and **5** show the relative plate stiffness parameters K and M_o, respectively, for two different plate orientations at different exposure temperatures normalized with respect to the corresponding average room temperature data. The third parameter, M_1, slope of the curve at large deformation, was selected to be zero for all the cases. If not zero, it is assumed to degrade in the same fashion as M_o. The solid lines represent average observed values, while the dashed lines represent the proposed model. The plots indicate a gradual loss of stiffness up to 200° to 250°C (392° to 482°F), followed by a rapid loss of stiffness. This trend is similar to the loss of stiffness of wood members, suggesting that the behavior of a metal-plate connection is largely controlled by the wood beneath the plate. The scatter in the plate stiffness parameters can be partly attributed to the difficulties in accurately measuring deflections that occur inside the oven. However, similar scatter in plate stiffnesses is common even in the case of well-controlled connection tests under ambient conditions. McCarthy and Wolf (19) report the coefficient of variation for K and M_o to be higher than 50 percent in some cases, while for M_1 it is reported as high as 127 percent. We believe that the degradations of the plate stiffness parameters are likely to depend on the duration of exposure as well. However, we did not attempt to include the exposure time in the development of the current model because of the variable nature of the data and the general lack of data defining the time effect.

APPLICATION OF THE DEGRADATION MODELS

The degradation models introduced have been incorporated in the computer program SAWTEF (4,30,34). In order to assess the validity of the degradation models, FPL tension tests on solid and spliced lumber exposed to variable time-temperature conditions that normally prevail in the plenum of protected assemblies in a standard ASTM E 119 fire endurance test (l), were simulated using program SAWTEF. The physical and elastic properties of lumber and the exposure time-temperature data needed in simulating the tests were obtained from the respective tests. The plate stiffness parameters for the plates and lumber properties were estimated from published literature (10,18). Failure of the assembly could be either due to an excessive slip at the plate-wood interface (30) or the total stress induced in any wood member exceeding the total capacity of the member (36).

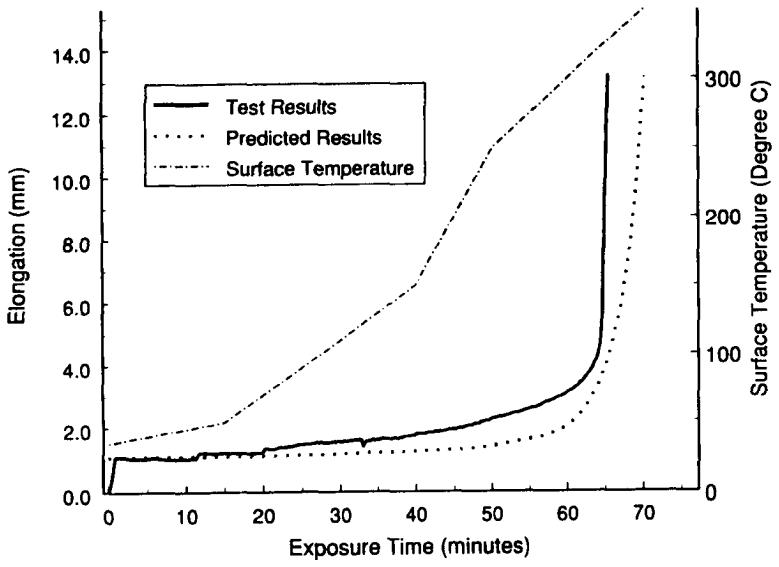

Figure 6. — Elongation of a spliced lumber specimen in tension exposed to linearly varying temperature.

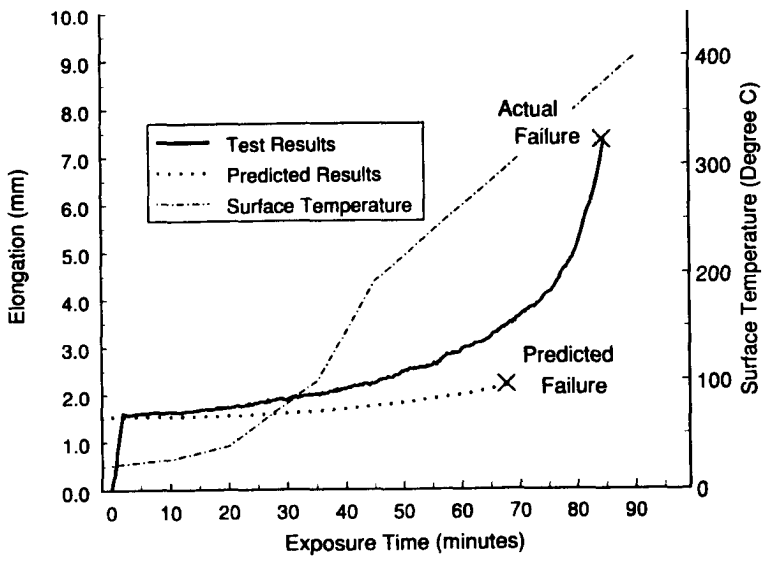

Figure 7. — Elongation of a solid lumber specimen in tension exposed to linearly varying temperature.

Figures 6 and **7** show the comparisons of the observed and predicted elongations in the tension tests of spliced and solid lumber specimens, respectively. The spliced and solid specimens were subjected to an axial load of 16,903 N (3,800 lb.) and 28,023 N (6,300 lb.) respectively, and the middle 1.83-m (6-ft.) portion of the specimens was exposed to an idealized plenum temperature (34). The observed elongations increase gradually to a near-failure stage and then they increase exponentially. The model predicted a similar pattern. However, the simulation of the solid lumber test did not exhibit the degree of ductility observed in the test **(Fig. 7).** Throughout the test, the predicted values tend to be lower than the corresponding data observed.

The differences in the predicted and observed elongations in **Figs. 6** and **7** are most likely due to the combined effects of reductions in strength and stiffness properties, thermal expansion, shrinkage, and creep, while only the effects of reduced strength and stiffness properties are explicitly incorporated in the model. The difference in the predicted and observed elongations at failure is approximately 5.1 mm (0.2 in.) in **Figure 7** (solid lumber case). If we take a coefficient of thermal expansion and shrinkage as 1.7e-6 mm/mm/°F and 0.1 percent respectively (7) and, as an approximation, creep strain as 0.0018 mm/mm for 5 percent of ultimate load for 80 minutes of exposure at 275°C (527°F) (27), a temperature rise of 250°C (482°F) in a 3.28-m- (10 ft., 9 in.) long beam will cause a thermal expansion, shrinkage, and creep elongation of 2.5, -3.3, and 5.9 mm (0.1, -0.13, 0.23 in.), respectively. These factors sum to an extra elongation of 5.1 mm (0.2 in.) in addition to the elongation due to change in the stiffness. It is coincidental that these factors sum to the precise amount of the model's error in this prediction. But this coincidence suggests that creep modeling may be important for prolonged combined temperature/load exposures. The data upon which the model is based did not include prolonged loading periods and thus creep effects are not considered. The error shown in **Figure 7** will have some effect on the ability of a truss

model to accurately predict deflections near failure. But this error may be masked by connection deformations as shown in **Figure 6** and predictions of deflections become large and difficult to predict regardless of the inclusion of creep modeling as a truss approaches collapse. The exclusion of creep deformations may not significantly affect the ability to compute fire endurance (time to failure) of an assembly.

SUMMARY AND CONCLUSIONS

Simple models have been introduced to predict mechanical properties of structural lumber and metal-plate connectors at elevated temperatures. The models were based on component tests of structural lumber and metal-plate-connected tension splices exposed to different load and temperature conditions. These models are intended for use in the structural analysis of wood assemblies exposed to elevated temperatures. The wood degradation model recognizes exposure conditions and the subsequent thermal response of individual members as important factors, while the plate degradation model is a bilinear function of the exposure temperature.

Observations of fire-endurance tests and analyses of resulting data suggested that several complex physical phenomena influence the mechanical response of the lumber and the metal-plate connections. In particular, creep deformations became significant in prolonged high temperature combined with design load durations of 60 minutes or more. It was not our objective nor was it possible from the limited data to understand and fully describe these phenomena in the simple models presented. As more test data become available at varied exposure conditions, the degradation models presented will be further refined and validated to provide more complete and descriptive predictions of mechanical response. Already, preliminary analysis of recently gathered data from a greater assortment of metal-plate connections shows that Equation **[4]** for M_o may need some adjustment to reflect greater degradation of strength at temperatures below 250°C (33). Continuing refinements are anticipated.

The models developed do not require extensive computational effort and have been successfully implemented in a structural analysis package. This technology of defining component properties at elevated temperatures is expected to lead to a greater understanding of the fire endurance of wood assemblies.

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