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Fire Endurance Model for a Metal-Plate-Connected Wood Truss

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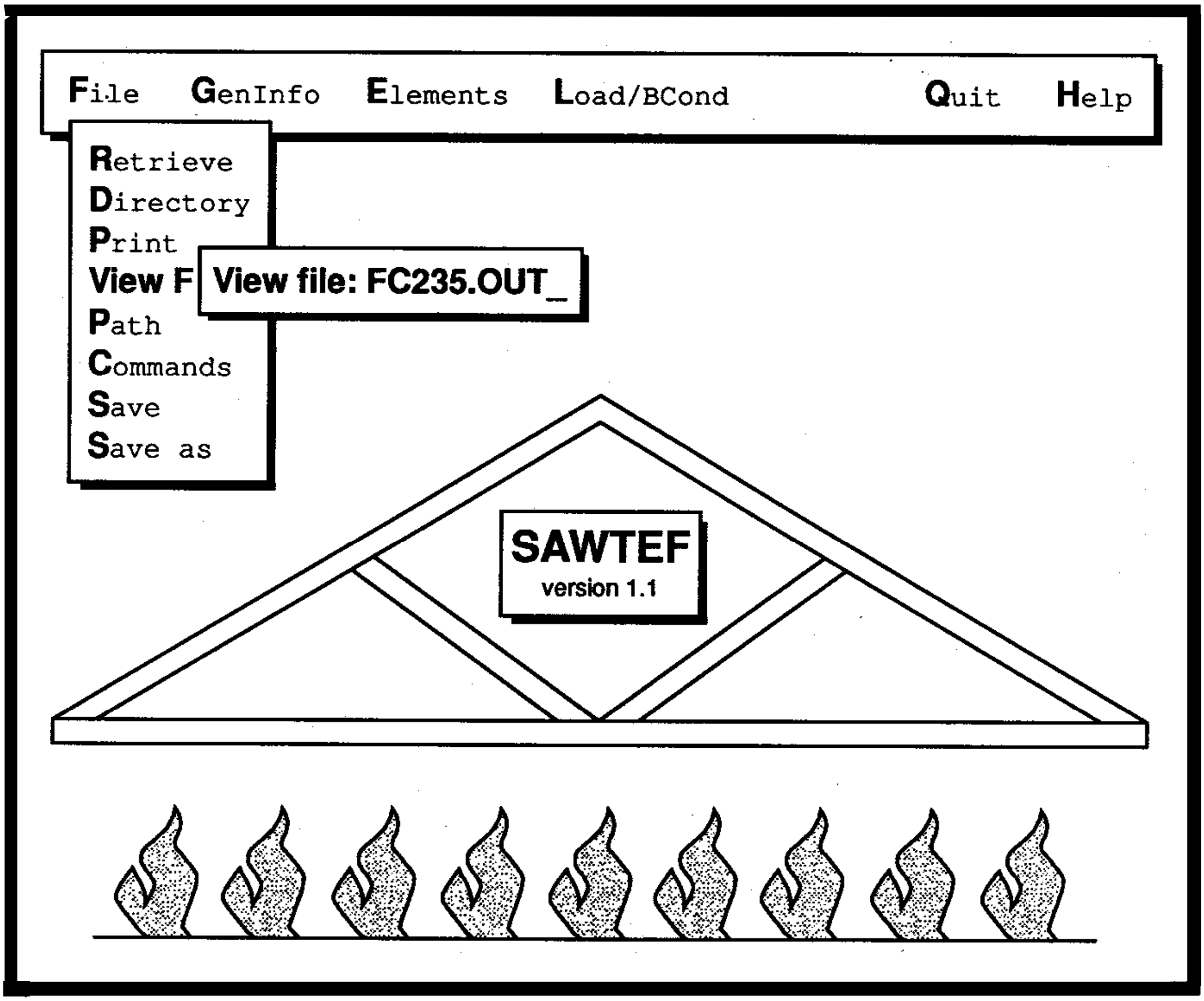

Abstract

Currently, expensive and time-conwming fire endurance tests (ASTM E 119) must be conducted on wood truss assemblies to obtain fire endurance information and code approval. A less costly, yet reliable means to establish the fire endurance of wood products and assemblies would produce a favorable environment for fire-resistant wood assemblies and increase utilization of wood. The primary objective of this study was to develop a theoretical model and user-friendly computer program that predicts the fire endurance of a metal-plate-connected wood truss (single-truss model). Extensive component fire testing was conducted to develop the necessary input and submodels for thermal degradation of the wood members and connections. The fire endurance model was evaluated using ASTM E 119 test data, and reasonably good agreement was obtained. Because the ASTM E 119 test method governs fire endurance of multiple-truss assemblies, future research will involve expanding the single-truss model to a system model.

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Fire Endurance Model for a Metal-Plate-Connected Wood Truss

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Introduction

In recent years, many wood trusses have been fire endurance tested according to ASTM E 119, the standard method of fire tests of building construction and materials (ASTM 1988). Industry has recently directed their efforts toward developing 1-h fire-rated truss floor assemblies with a directly applied single layer of gypsum board (National Evaluation Service 1989). Truss Plate Institute (TPI) has a fire testing program directed toward developing 2-h fire-rated truss assemblies. The performance of truss assemblies in a standard fire test was reviewed by Schaffer (1988) in *Fire Journal.*

Standard fire endurance tests of assemblies have been the main method of ensuring adequate fire endurance. The nature of these tests allows little development of general rules or methodologies that would enable predictions of fire endurance. As a result, expensive and time-consuming fire tests must be conducted to obtain fire endurance information and code. approval for variations in assembly design. A less costly, yet reliable means to establish the fire endurance of wood products and assemblies would produce a favorable environment for fire-resistant wood assemblies and increase utilization of wood.

Fire endurance models provide the means to better understand performance, to design for a wide range of applications, and to optimize the fire and structural performance of the structures. The development of tire endurance models would aid understanding of tire endurance and provide. a potential alternative to the expensive ASTM E 119 assembly testing. The premise of the models is that smaller, less expensive fire tests can he conducted on components, and this information can then be combined with the structural mechanics theory to predict the response of the larger assembly, the wood truss.

The primary objective of study was to develop a model and user-friendly computer progmm that predicts single-truss performance under combined structural load and fire conditions. The model considers both pitched or parallel chord metal-plate-connect wood trusses. This study included the determination of the load-bearing capacities of actual 38-by 89-mm (nominal 2- by 4-in.) lumber and metal connector plates under various fire exposures. Such property data and degrade models are needed as input to the fire endurance truss model. The model requires that the temperature development within the truss assembly be specified by the user.

This study consisted of four parts:

(a) Development of model and computer program

(b) Testing of components under combined load and high temperature

(c) Evaluation and refinement of the model

(d) Initial application of the model

The model represents an initial effort to model the fire endurance behavior of trusses. Future refinements of the model will expand its applicability to multiple-truss assemblies and include heat transfer analysis of protected truss assemblies.

The long-term objective of this research is to improve the market for wood-based components and assemblies by understanding, improving, and gaining acceptance of their fire performance. By developing fire endurance computer models and increasing flexibility in fire endurance design, we hope to improve the fire safety and the commercial-industrial markets for wood-based components and assemblies.

Computer Program

The computer program SAWTEF (Structural Analysis of Wood Trusses Exposed to Fire) has two main parts: a windows-oriented pre/post processor software for data input and output interpretation and structural analysis software.

The SAWTEF program was designed for operation on an IBM-AT[1] and compatible computers operating in the MS-DOS environment. Recommended computer system requirements are a 16 MHz 386SX or faster, 1 megabyte of RAM, 1 megabyte of hard disk space, a math coprocessor, and DOS 3.1 or later version.

The model requires the following input data:

- Truss geometry, including span, slope, individual member size and length, truss plate size and orientation
- Wood member stiffness and strength values at room temperature, including tension, bending, and compression strength
- Stiffness values for the connections at room temperature in tension
- Vertical and/or lateral load magnitudes
- Temperature magnitude and distributions within the truss for an assumed fire condition

Loss of stiffness and strength of wood members or connectors because of thermal degradation of wood are estimated within the program. Alternatively, the user may input this information if it is available.

The model generates output for a single truss subject to assumed fire conditions, which includes the following for each time step and member within the truss:

- Displacements of the end of each member
- Stress conditions within each member
- Percentage of remaining load capacity

The analysis of a metal-plate-connected wood truss involves three major steps: (a) creation of the truss analog and preparation of input information, (b) execution of the analysis, and (c) interpretation of the output.

Model Development

The fire endurance truss model predicts single-truss performance for combined structural load and fire conditions. A pitched or parallel chord truss may be considered. The model considers nonlinearities introduced by nonlinear connection properties, fire degrade, and large displacements using established methods of structural and finite element analysis (Cook and others 1989). Development of the model benefitted from a parallel study conducted by the USDA Forest Service, Forest Products Laboratory (FPL), and the University of Wisconsin-Madison on the reliability of wood roof systems (Cramer and others 1990, Mtenga 1991). In that study, full-scale individual trusses and roof systems were tested to failure, and the resulting database guided our model development.

Three graduate students participated at various levels in the development of the fire endurance truss model. In unpublished works, Schellhase provided contributions to the initial development of the structural analysis portion of the algorithm, and Sankhe developed a metal-plate connection failure index. Shrestha (1992) developed the fire endurance truss computer program, SAWTEF, through development of the structural analysis, analytical heat transfer model, and thermal degrade models. Development of the model and the corresponding computer program SAWTEF are discussed in detail in Shrestha (1992).

Component Tests

This project included a component testing program, which was composed of measuring the tension and bending behavior of lumber and truss plate connections under room temperature and fire conditions. This choice of emphasis of the testing program was based on the assumption that bottom-chord members (subject to tension and bending forces) control truss fire endurance. But, failure in the model is not limited to this assumption. In the model, failure can occur as a result of compression of the top-chord members or web members.

Test Method

The testing program involved tests on 2 by 4's and on lumber with metal plate connectors. The specimens were loaded in tension in a specially made tension apparatus (Fig. 1). In the middle of the tension apparatus, there is a 2.1- by 1.4-m (7- by 4-1/2-ft) furnace. The tension/furnace apparatus is unique in terms of its size and flexibility. Although ASTM E 119 does not include tensile members as an assembly type, general characteristics are based on ASTM E 119.

The furnace is lined with mineral fiber blankets and heated by eight diffusion-flame natural gas burners. Two meters (6 ft) of the 5-m- (16-ft-) long specimens were exposed to fire or high temperatures. The orientation of the 2 by 4's was such that the wider sides were vertical. Elongation measurements were provided by an extensometer system consisting of a reference arm that goes around the furnace, a linear variable differential transducer (LVDT) at one end, and clamps with magnetic attachment plates. The strain gage length was 3.3 m (10 ft 9 in.).

[1]The use of trade or firm names in this publication is for reader information and does not imply endorsement by the U.S. Department of Agriculture of any product or service.

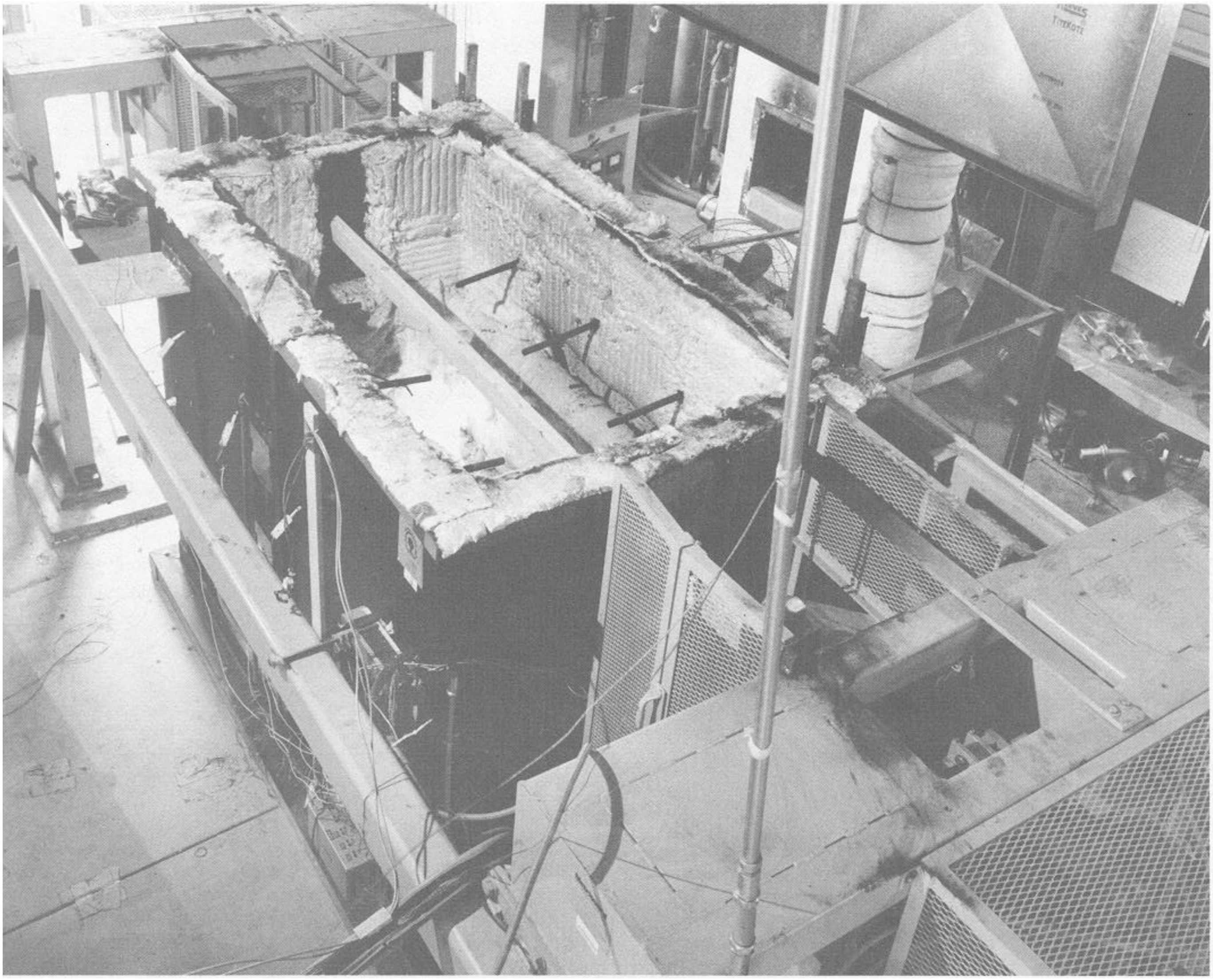

Figure 1—The Forest Products Laboratory tension apparatus and furnace (cover removed for photograph).

Two types of tests were conducted: (a) constant temperature, increase load until failure and (b) constant load, increase temperature until failure. The intent of the two types of tests was to develop thermal degrade models from the constant temperature tests and verify thermal degrade models with the constant load tests. Unloaded temperature profile tests were conducted to aid development of a method that calculated heat conduction through the wood members.

In the constant temperature tests, the specimen was heated in the furnace for 30 or 60 min at a constant temperature and then a ramp load was applied to failure. Constant furnace temperatures of 100°C, 200°C, 250°C, 275°C, and 300°C were used in the tests. Furnace temperatures were controlled using thermocouples stapled to the vertical side of the specimens. The results of the constant temperature tests were the maximum load and load-elongation curve when tested at a specified temperature.

In the constant load tests, the specimen was loaded to a constant tensile load and then exposed to fire until failure. The temperature was controlled to follow a specified time-temperature curve. The four loading and fire exposure levels for the constant load tests consisted of two load levels (50 and 100 percent of design load) and two fire exposures (ASTM E 119 and an idealized plenum time-temperature curve). The time-temperature curve of the ASTM E 119 test method represents direct-fire exposure of the wood members. 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. The plenum time-temperature curve was developed to represent the exposure of wood members within a protected assembly when it is exposed to ASTM E 119 fire exposure. This idealized plenum time-temperature curve is 65°C, 93°C, 188°C; 260°C, and 327°C at 10, 20, 30, 45, and 60 min. respectively. The curve was derived from various ASTM E 119 tests of protected truss assemblies. The results of the constant load tests were the time of failure and time-elongation curve.

The lumber tested was No. 1 Dense Southern Pine KD 15 and Spruce-Pine-Fir machine stress rated 2100f-1.8E lumber. All lumber was tested nondestructively for dynamic modulas of elasticity (MOE). The metal plate connectors were 20 gauge, Grade A steel with 8.5-mm- (1/3-in.-) long teeth, 14,570 teeth per square meter (9.4 teeth per square. inch). The two types of plates were parallel (P0) plates (75 by 190 mm (3 by 7.5 in.), 96 teeth per plate) and perpendicular (P90) plates (175 by 75 mm (7 by 3 in.), 91 teeth per plate). Parallel and perpendicular refer to the direction of the major axis of the truss plate relative to the direction of the load In all cases, the grain of the lumber was parallel to the application of the load (GO). All materials were conditioned at 22°C, 50 percent relative humidity before testing, which is specified in ASTM E 119.

In addition to the pure tension tests, the 2 by 4's and the lumber with connectors were tested under tension/bending (limited moment). The bending moment was obtained by applying the tension load to collars on each end of the. specimens so there was a 140-mm (5-1/2-in.) eccentricity in the applied load (Wolfe 1990).

Experimental Results

Experimental results discussed in this section are for the Southern Pine tested in tension. These results, the Spruce-Pine-Fir and combined tension-bending data, are discussed in greater detail in a separate FPL research paper (White and others [in preparation]).

Constant Temperature Tests

The results of the constant temperature tests were the maximum load and load-elongation curves. In the Southern Pine tests, degradation started at greater than 100°C (Figs. 2 and 3).

The temperature (T) equation used in Figures 2 and 3 was

$$T = \frac{T_s + 2T_c}{3} \quad (1)$$

where

T_s = surface temperature and
T_c = center temperature.

Equation (1) was derived from the trapezoidal rule for the area under a parabolic curve. We found that the temperature profile within the member could be represented by a parabolic curve.

The maximum strength of the lumber after heating is represented as a fraction of the average maximum load obtained in the tests at room temperature (23°C) (Fig. 2). At around 300°C, the lumber had essentially no residual strength. The two curves in Figures 2 and 3 represent the 30- and 60-min heating durations. The two heating times resulted in different center temperatures. With a surface temprature of 250°C, the average. of the center temperatures at failure was 159°C in the 30-min tests and 210°C in the 60-min tests.

The stiffness (MOE) of the lumber after heating is also shown as a fraction of the average result obtained in the tests at room temperature (Fig. 3). Compared with maximum strength, there was a smaller reduction in stiffness with increasing temperature..

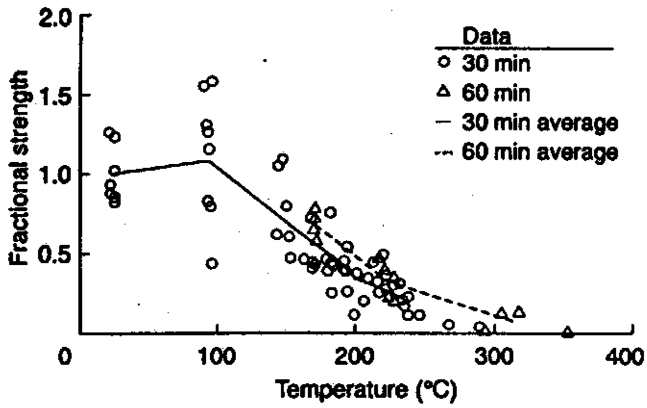

Figure 2—Loss in tensile strength of Southern Pine 2 by 4's when exposed to a constant elevated temperature for 30 to 60 min. Each data point is the result of a separate test at a controlled surface temperature. Temperature reflects the Internal temperature of the lumber at the time the load was increased until failure. Unity strength is average strength of pieces tested at room temperature.

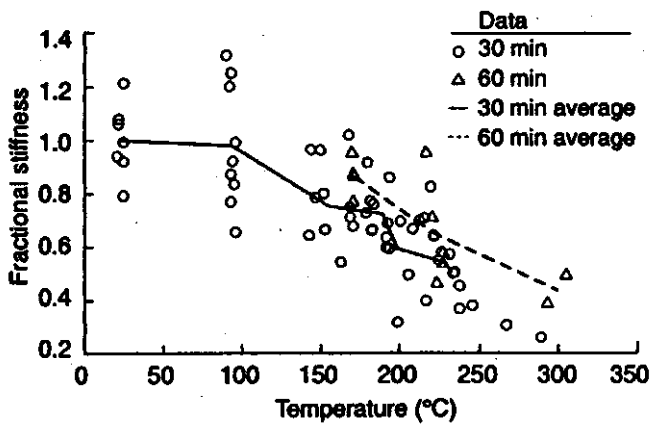

Figure 3—Loss in tensile stiffness of Southern Pine 2 by 4's when exposed to a constant elevated temperature for 30 to 60 min. Each data point is the result of a separate test at a controlled surface temperature. Temperature reflects the internal temperature of the lumber at the time the load was increased until failure. Unity stiffness is average stiffness of pieces tested at room temperature.

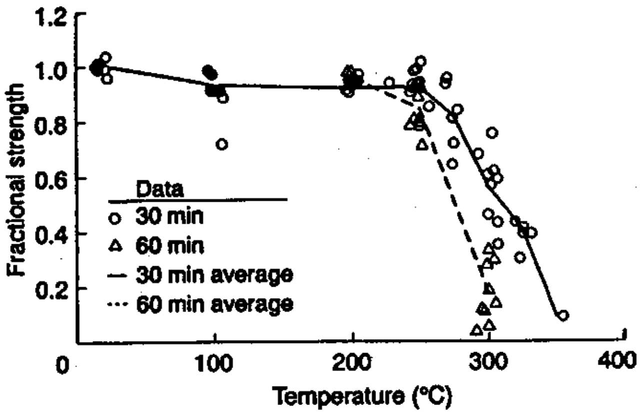

Figure 4—Loss in tensile strength of Southern Pine 2 by 4's with metal plate connection when exposed to a constant elevated temperature for 30 to 60 min. Each data point is the result of a separate test at a controlled surface temperature. Temperature is the exposed wood surface temperature. Unity strength is average strength of pieces tested at room temperature. Only failure at 300°C 60 min was in the solid member and not the connection.

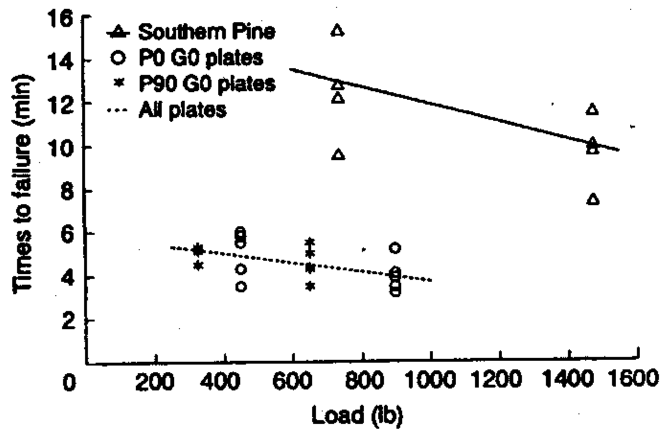

Figure 5—Time of failure compared with constant load applied for specimens exposed to ASTM E 119 fire exposure while loaded in tension. Specimens are Southern Pine 2 by 4'S and Southern Pine lumber with metal plate connection. Each type of specimen was tested with full design load and 50 percent of full design load.

The maximum load results for lumber with connectors are also shown as a fraction of the maximum load at room temperature (Fig. 4). The maximum load was the same as the room temperature tests for higher exposure temperatures in the plate tests compared with similar tests without plate connections. Figure. 4 shows the surface temperature of the wood at a location on the wide. side of the 2 by 4's (same as the plates) but at a distance from the plates. As discussed later, the temperature at the plate-wood interface may be slightly less.

The test procedures provided a realistic exposure at the expense of reliable elongation measurements that are not possible inside a large furnace. In developing the degrade models, the computed elongation of the wood was subtracted from the total elongation of the lumber with connectors to obtain the elongation resulting from the connections.

Some tests were conducted at surface temperatures of 325°C and 350°C. One problem caused by using these temperatures was the tendency for the specimen to ignite. during the 30-min heating test.

Constant Load Tests

As shown in Figure 5, the times of failure corresponding to direct exposure (ASTM E 119) ranged from 7.3 to 11.4 min for the fully loaded Southern Pine 2 by 4's and 3.2 to 5.5 min for the fully loaded metal-plate-connected Southern Pine lumber.

As shown in Figure 6, the times of failure for the plenum exposure tests ranged from 57.3 to 71.6 min for the fully loaded Southern Pine. 2 by 4's and 53.8 to 63.8 min for the metal-plate-connected Southern Pine lumber.

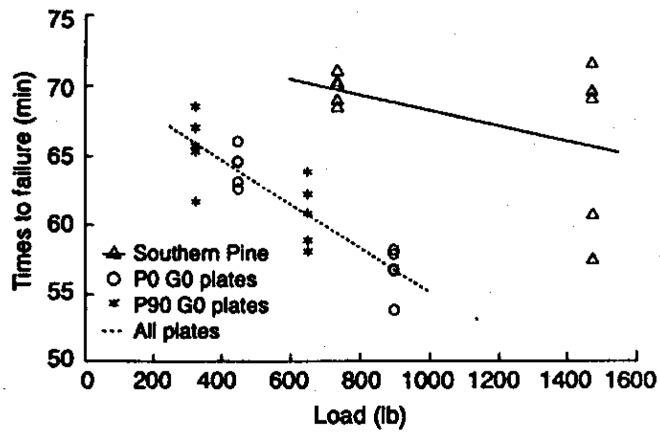

Figure 6—Time of failure compared with constant load applied for specimens exposed to an idealized plenum exposure while loaded in tension. Specimens are Southern Pine 2 by 4's and Southern Pine lumber with metal plate connection. Each type of specimen was tested with full design load and 50 percent of full design load.

Temperature Data

Of considerable interest is the effect the metal plate connector has on the temperature development beneath it. A limited number of tests were conducted in which thermocouples were placed at different depths in areas beneath the metal plates and in areas without plates. Figures 7 and 8 are results from an ASTM E 119 exposure and a plenum exposure, respectively. The thermocouples were at depths of 4.8 mm (0.19 in.) and 9.5 mm (0.38 in.). In these experiments in a test furnace with walls of ceramic fiber insulation, the temperature directly beneath the metal plate was less than

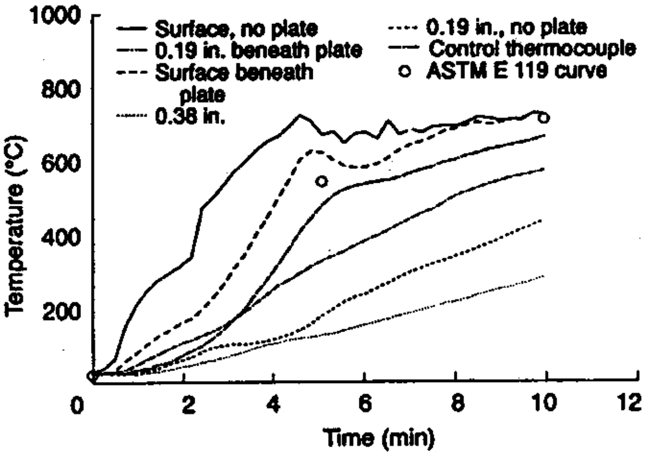

Figure 7—Temperature development at different depths when exposed to temperatures specified in ASTM E 119. Thermocouples are either beneath a metal plate (P) or not (NP). The controlled thermocouples are within sealed pipes that are used to control the furnace temperature as specified in ASTM E 119.

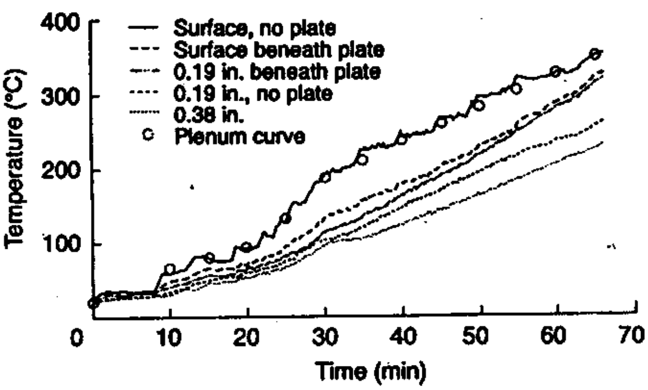

Figure 8—Temperature development at different depths when exposed to temperatures specified by an idealized plenum time-temperature curve. Thermocouples are either beneath a metal plate (P) or not (NP). The thermocouple on the wood surface and not beneath a metal plate was used to control the furnace temperature.

that of the exposed wood surface. Visual observations showed evidence of delayed charring beneath the plates. However, the temperature at the 0.19-in. (4.8-mm) depth was greater beneath the plate than in areas of exposed wood. Only one curve is shown for 0.38-in. depth (9.5-mm) because there was no temperature difference at that depth. This suggests that while the metal plate reflected some radiation, the 8.5-mm- (0.33-in.-) long metal teeth conducted energy to even greater depths. In the tests of constant temperature, reflection of radiation from the metal plates resulted in a significant delay in the temperature at the wood-plate surface reaching the exposed wood surface temperature. Because it was necessary for the metal-plate-connected lumber to be exposed to the same conditions as the 2 by 4's, the test procedure was not changed.

Theoretical Models

In this section, the theoretical models are briefly reviewed. They are discussed in detail by Shrestha (1992). These models include the basic structural model, the heat transfer model incorporated within the computer program, the thermal degrade models that define the reduction in material properties with temperature, and the criteria that define the structural failure of the truss.

Structural Model

The structural model is based on established methods of structural analysis and finite element analysis (Cook and others 1989). Connector elements consist of an assemblage of rigid links and springs to model the tooth-wood slip and eccentricities associated with these connectors. The truss is modeled as an assembly of wood and connector elements. The wood members are assumed to exhibit linear elastic behavior, and the metal plate connectors are assumed to have nonlinear elastic properties. The load-slip properties of the connection are specified by Foschi-type parameters (Foschi 1977). Foschi's equation for load-slip curve is

$$F = (M_0 + M_1\Delta) \quad [1 - \exp(-K\Delta/M_0)] \tag{2}$$

where

F = force,
Δ = displacement (slip), and
M_o, M_1, K = Foschi's parameters.

A large deformation-small rotation analysis was developed to accommodate the significant deformations witnessed in fire tests of assemblies.

Heat Transfer Model

The scope of this project was limited to the development of a model that will predict the structural response when the temperature development within the assembly and members is given. Recent developments in modeling heat transfer within a wood member included numerical models, such as finite difference and finite element, and all factors involved in the thermal degradation of wood and changing boundary conditions. To incorporate such models within SAWTEF was not practical. To have a more self-contained program, a limited heat transfer model of the temperature profile of a wood element was built into the program. Based on some assumptions, an analytical model with a closed-form solution was developed. The surface temperature-time curves (STCs) for the elements still need to be provided as input. The STCs for the wood and plate elements were obtained from either an ASTM E 119 test report or a separate heat transfer analysis.

The heat transfer model was developed from the conservation of energy differential equation. Heat generation includes separate terms for the moisture and pyrolysis effect. Moisture vaporization and wood pyrolysis were modeled with Arrhenius equations: By assuming that the wood member is suddenly subjected to a constant surface temperature on all four surfaces, a closed-form solution can be obtained. The temperature and heat generation terms were represented by double Fourier series. Based on the component test data, correction factors were developed for the cases when the surface temperature was not constant.

The heat transfer model provides a useful alternative when better data are not available. The user of the program has the option of inputting thermal degrade factors directly instead of using the surface temperatures. Theoretical heat transfer models are available that predict the temperature gradient in a protected wood member. A one-dimensional model was developed by Fung (1977). In a cooperative effort with FPL, the University of California developed a model for the fire endurance of wood-frame. walls (Gammon 1987). Although the wall model predicts the buckling failure of a single stud, it is primarily a heat transfer model for predicting the temperature distribution and thermal degradation within the wall assembly. Although these models are available, they are not completely satisfactory. Work is presently being done at Forintek of Canada on modeling a wood stud wall for fire endurance. In particular, Forintek is developing a better model for gypsum board. This work is critical to the future application of SAWTEF as well as other structural fire endurance models.

Thermal Degrade Models

The heat transfer model is used to predict the temperature at the center of the wood member. A parabolic temperature distribution within the member is then assumed with the specified surface temperature and the calculated center temperature. Using initial component test data, thermal degrade models were developed for the structural properties of the wood members and the metal plate connectors. The equation for the degradation of lumber properties is

$$\frac{P_t}{P_o} = \frac{T_o t_e}{T_o t_e + \gamma_p A_t} \qquad (3)$$

where

P_t = degraded property at given exposure time t_e (longitudinal MOE), tensile strength, compressive strength, or bending strength of wood),

P_0 = property corresponding to P_t measured at room temperature,

t_0 = room temperature (°C),

t_e = time or duration of exposure (min),

γ_p = correction factor: $(T_c^{0.5}/100)$ for MOE, $(T_c/198)$ for tensile and compressive strengths, and $(T_c/215)^6$ for bending strength,

T_c = center temperature (°C) at time t_e, and

A_t = area under time-temperature curve defined by temperature of Equation (1) minus the initial room temperature.

The correction factor (γ_p) is included to improve the fit of the data. Degrade data were not obtained for compression; therefore, these data were assumed to be the same as the tension data in the model. However, other research indicated that degradation in tensile properties was not the same. as that for compressive properties. For the metal plate connectors, Foschi's parameters are assumed to be linear functions of the exposed surface temperature with a break point at 200°C for the K parameter and 250°C for M_o and M_1 parameters in Equation (2).

When the thermal degradation of the components is known in detail, the thermal degrade factors can be inputted as a function of time. To develop precise thermal degrade factors, the temperature profile within the assembly and its components needs to be known in detail. The default degrade factors for the wood components are based on a uniform temperature on four surfaces. As an alternative, lumber degrade models such as that used by Schaffer and others (1986) and King and Glowinski (1988) can be used to obtain the appropriate degrade factors for the lumber. Alternative models will also be. discussed by White and others in a FPL research paper [in preparation].

Failure Criteria

Failure of a truss is assumed in this model to occur when the load on a member exceeds its residual strength or there is excessive slippage of the connection. The model provides for a maximum combined stress index (CSI) for the wood members and a maximum plate failure index (PFI) for the metal plate connectors. The CSI considers the combination of axial and bending stresses in the wood members based upon Zahn (1986). The CSI varies between 0 and 1, with greater than 1 indicating failure. The critical CSI value of 1 may be attained through a combination of bending and tensile stresses (typically occurring in the bottom chord of a truss) or a combination of bending and compressive stresses (typically occurring in the top chord of a truss). The CSI equations for each case are as follows, respectively:

$$CSI = \left(\frac{f_t}{F_t}\right) + \left(\frac{f_b}{F_b}\right) \qquad (4)$$

$$\mathrm{CSI} = \left(\frac{f_c}{F_{c1}}\right) + \left(\frac{f_b}{\theta_c F_b}\right) \qquad (5)$$

where

f_t = member tensile stress,
F_t = member tensile strength,
f_b = member bending stress,
F_b = member bending strength (MOR) assuming complete lateral support,
f_c = member compressive stress,

and θ_c is defined as

$$\theta_c = 1 - \frac{f_c}{f_{c3}} \qquad (6)$$

and F_{c1} is defined as

$$F_{c1} = \frac{F_{c2} + F_{c3}}{1.6} - \sqrt{\left(\frac{F_{c2} + F_{c3}}{1.6}\right) - \frac{F_{c2}F_{c3}}{0.8}} \qquad (7)$$

where

F_{c2} = member crushing compressive strength

and

$$F_{c3} = \frac{0.822E}{(L/d)^2} \qquad (8)$$

where

E = elastic modulus of the member,
L = member length, and
d = member depth.

The maximum PFI indicates the capacity consumed for a plate-wood contact area based upon the degrade. of stiffness of the connection. The index varies between 0 and 1, with 1 or greater indicating failure. The PFI for a given contact area is computed as

$$\mathrm{PFI} = 1 - \frac{K_1}{K_0} \qquad (9)$$

where

K_1 = current tangent stiffness value of the plate-wood connection, and

K_0 = original tangent stiffness value of the plate-wood connection.

Our investigation showed that the degrade in stiffness of the connection is a reasonable predictor of the capacity consumed. As K_1 approaches zero, the force required to cause a unit displacement in the connection also approaches zero. This corresponds to imminent failure.

Model Evaluation

Comparison With ASTM E 119 Tests

Standard fire tests of five different floor-ceiling truss assemblies were used to evaluate the model. The assemblies were designed by TPI, and the task of validating the fire rating of each assembly was assigned to independent agencies. Table 1 summarizes the characteristics of each assembly. All assemblies employed 2 by 4 Southern Pine lumber and 20 gage metal plate connectors. A typical representative truss from each of the floor-ceiling assemblies was modeled.

The test reports provided the following general information:

- Layout, geometry, and general setup, including wood member and plate sizes and design load
- Deflection measurements along the center-line, parallel to the truss, and at mid- and quarter-span points
- Visual observations of the fire-exposed surface and unexposed (to fire) surface of the assembly at regular intervals
- Temperatures of the fire-exposed surface, unexposed (to fire) surface, and plenum region of the assembly
- Illustrations and photographs showing truss geometry

Detailed information on wood member and connector properties does not generally appear in these reports. Based upon the grade of lumber and gage of the. metal plates, corresponding properties were estimated from the literature (Gerhards 1983, McCarthy and Wolfe 1987). Plate positioning was estimated from drawings and photographs. The surface temperature-time record for each member was estimated based upon plenum thermocouple data in each report.

By combining the data given with reasonable estimations of the missing data, each truss was analyzed with SAWTEF for design load and the reported plenum temperature exposure. The observed failure times for the five different floor-ceiling assemblies were compared with the SAWTEF predictions for the representative truss of the respective assembly (Fig. 9). The predicted values were 16 percent less on average than the observed failure times. Figure 9 indicates that the model generally under-predicts the actual failure time. This is reasonable since assembly performance. is being compared to predicted single-truss performance. The representative truss does not incorporate the load-sharing effects of attached sheathing (plywood or gypsum) and bracing members, which are present in the assemblies.

Several general observations are noted from the analyses of the five trusses. Deflection in the truss increased gradually through must of the test until a few minutes prior to failure.

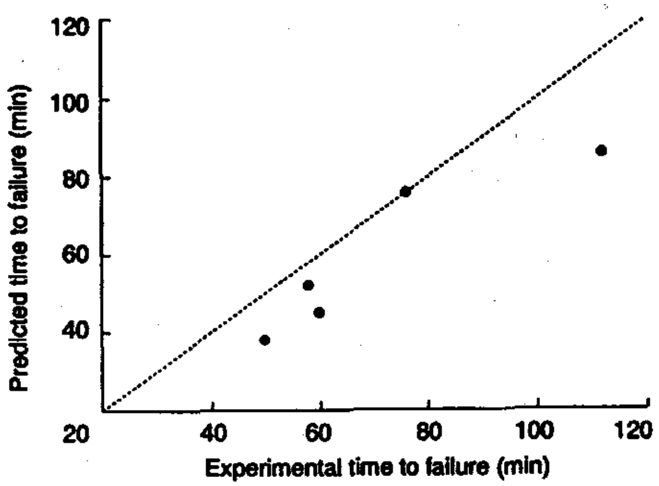

Figure *9—Comparison of observed failure times for five floor-ceiling assemblies with predicted failure times. The predicted failure times are for a single representative truss.*

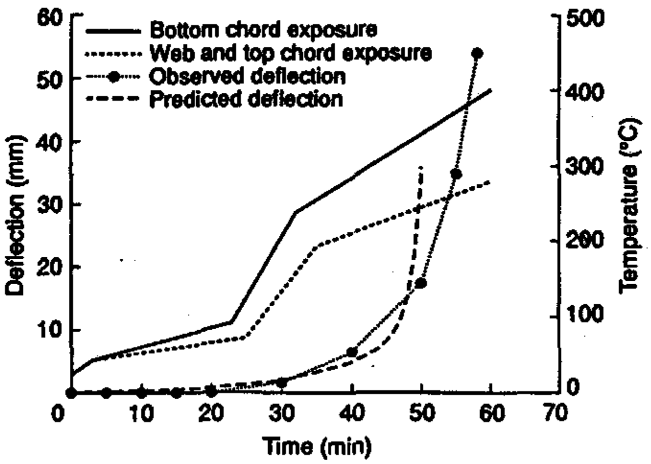

Figure 10—Surface temperature-time history and displacements for truss FC-249.

Deflections then increased exponentially when approaching collapse. As the model considers only the single truss, there is no load-sharing to provide even a momentary respite from impending collapse. The increase in deflections resulted from the loss of stiffness caused by the increasing temperature.

As stiffnes is lost, the capacity of the members is also increasingly consumed. Hence, as failure approaches, the CSI and PFI, which indicate the maximum lumber and plate capacity consumed, approach unity. Figure 10 shows the relationship. between plenum temperatures and time and truss deflection for Truss FC-249. This relationship was typical for the five truss assemblies studied.

Model Sensitivity Analysis

The sensitivity of the model was examined through analysis of different variations of a representative truss from the five assemblies listed in Table. 1. The following parameters were considered:

- Plate strength and stiffness properties (changed by plus and minus 50 percent)
- Wood strength and stiffness (changed by plus and minus 50 percent)
- Combined changes in wood and plate properties (previous two parameters applied together)
- Thickness of wood members
- Time-temperature exposure

A summary of the trends is presented here, and the actual data associated with the analyses are in Shrestha (1992). In general, single-truss performance was highly dependent on the temperature exposure. Exposures greater than 325°C typically resulted in first failure of a plated connection, and exposures less than 325°C resulted in first failure of the wood member. Variations in wood and plate properties, as large as plus or minus 50 percent, had little or no effect on time to failure, but presented a significant influence on deflection. The time-temperature exposure of the truss is currently assumed to be independent of the truss deflection. It is known that as truss deflections increase, the thermal protection membrane can be compromised, leading to a rapid increase in exposure temperature. Rules quantifying this relationship have not been established, but are needed for realism and to fully utilize the capabilities of the model.

Several assumptions used for simplification in modeling fire and the exposed structure trivialized the problem in this early stage of fire modeling research. Temperature conditions derived from previous ASTM E 119 tests were assumed to be identical along the single bottom chord of the truss leading to complete. simultaneous failure of the bottom chord of the truss. When failure began, no load sharing was possible because the entire length of the bottom chord collapsed at once as the ASTM E 119 temperature exceeded 350°C. Such a situation is unlikely in real fires, and even in most ASTM E 119 furnaces, it is known that regions near the walls experience some reduction in temperature compared to the midsections of the span. Furthermore, in real fires, the truss would not be as uniformly loaded as in the ASTM E 119 test procedure, thus the remaining capacity of different members in a truss would vary and allow for greater load sharing within the assembly. Unlike assemblies, a single-wood truss is almost statically determinate. This means that when one component fails, the single truss will collapse. But, most metal-plate-connected trusses are connected to other trusses by sheathing materials. As a result, failure of one component in a truss may not cause failure of the assembly.

Table 1-Summary of ASTM E 119 tests

Design number	Testing agency	Target rating (min)	Truss characteristics and protection scheme[a]
FC-214	Factory Mutual Research Corp., Norwood, MA (1978)	60	Truss: 12 in. deep, 17 ft 2 in. long Gypsum: Two layers of 1/2-in. Type FSW-1 attached directly to bottom chord Floor: Single layer of 1/2-in. plywood
FC-235	Factory Mutual Research Corp., Norwood, MA (1976)	45	Truss: 12 in. deep, 17 ft 2 in. long Gypsum: One layer of 5/8-in. Type FSW-1 attached directly to bottom chord Floor: 3/4-in. plywood with vinyl asbestos tile on top
FC-249	Factory Mutual Research Corp., Norwood MA (1977)	45	Truss: 12 in. deep, 17 ft 2 in. long Gypsum: One layer of 5/8-in. Type FCC attached to bottom chord on 1/2-in.-deep furring channels on 1/2 in.-deep furring Floor: 3/4-in. plywood
FC-426	PFS Corp., Madison, WI (1986)	120	Truss: 14 in. deep, 13 ft 4 in. long Gypsum: Two layers of 5/8-in. Type FCC attached, to bottom chord on furring channels Floor: 23/32-in. tongue and groove plywood
L-528	Underwriters Lab. Inc., Northbrook, IL (1981)	60	Truss: 12 in. deep, 13 ft 11 in. long Gypsum: One layer of 5/8-in. Type FCC attached to bottom chord on 7/8-in.-deep furring channels Floor: 5/8-in. tongue and groove. plywood glued to the top chord

[a]1 in. = 25.4 mm; 1 ft = 0.3 m.

Application of the Model

The developed fire endurance model can be used to predict the failure time of metal-plate-connected parallel-chord and pitched-chord wood trusses exposed to given time-temperature conditions. In addition, deflection of the truss, forces within the members, and mode of failure are computed. With these capabilities, the model can be. applied in parameter studies to gain insight into truss performance at elevated temperatures. The model can be used by designers in the truss industry to examine the predicted performance of a truss design to high temperatures, before proceeding with an ASTM E 119 test. Repetitions of the expansive, full-scale ASTM E 119 test may be avoided or at least minimized through use of the model. The model was compared with previously conducted ASTM E 119 assembly tests. The evaluation of the model with assembly test data has been limited, and thus sole reliance of the model predictions in situations involving life safety is not recommended. Until further verifications, the model should be used as an aid, not a replacement, when conducting tests.

The model provides an excellent platform for development of an assembly analysis. In fact, the complete capabilities of the model cannot be fully utilized until it is extended to address trusses connected by sheathing in an assembly.

We hope to eventually have the capability to completely model the performance of a truss assembly in a fire. Such a capability will enable engineers and architects to analyze fire safety requirements in the design process instead of being limited only to listed assemblies. Also, code officials and others will be able to consider the performance of the truss assembly under realistic fire exposures, determined by the combustible contents and ventilation factors of the building, rather than just the standard fire exposure.

Additional Research

This report covers the development of the fire endurance model. Currently, the development of improved fire resistant truss assemblies is being studied and funded by the USDA, Cooperative State Research Service, Competitive Research Grants Office. Our primary objectives in this study were to identify and evaluate technical innovations that will improve the fire resistance of wood truss systems without involving the protective membranes. Secondary objectives were to (a) refine the fire endurance model, (b) expand the database on the high temperature performance of metal-plate connectors, and (c) expand the database. on performance. of fire resistive coatings on wood.

The fire endurance of a structural assembly depends upon the combined action of individual trusses connected by sheathing that share in carrying the load. It is generally recognized that system effects (opposed to single-component behavior) add safety to an assembly through load redistribution mechanisms. With funding from the American Forest and Paper Association (formerly National Forest Products Association), we plan to expand the current single-truss model to a system model. This will add the contribution of attached wood-based or gypsum sheathing materials to the residual capacity of the assembly. This will include the composite action of the sheathing as well as the load-sharing effects within a system of trusses.

continued research is also needed to develop acceptable heat transfer models for wood assemblies as well as solid wood. The determination of the temperature distribution within the wood components has become a critical item in development of fire endurance models.

Conclusions

A theoretical model and a user-friendly computer program were developed for predicting the fire endurance of a metal-plate-connected wood truss. Extensive component fire testing was conducted to develop the necessary input and submodels for thermal degradation of the wood members and connections. The model was evaluated using existing ASTM E 119 test data. Reasonably good agreement was obtained. The model will be used in a current study to identify the critical factors in the fire endurance of trusses and help develop improved fire resistant wood trusses. Because the ASTM E 119 test method that governs fire endurance regulation of structural assemblies is a test of an assembly, there is a need to expand this current single truss model to a system model.

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