Fire Endurance Modeling of Wood Structural Systems

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Abstract

A numerical model for predicting the fire endurance of gypsum wall board-potected wood floor/ceiling assemblies entitled SAWFT is described. Assemblies consisting of solid wood joist construction or metal-plate-connected wood trusses connected by plywood sheathing are addressed with this model. The degradations of wood members and metal-plate-connectors within the model are based upon elevated temperature/mechanical tests with Southern Pine lumber and metal-connector plates. Comparisons of fire endurance times predicted with the model and measured from ASTM E 119 assembly tests are favorable. Although the model provides a framework for the analysis of a broad spectrum of wood construction and fire conditions, as described herein, the degradation rules are based upon a limited set of elevated temperature tests and application of the model is currently limited to assembly situations that correspond to the test data supporting the model. Challenge remains in fire endurance and heat transfer model development to predict the degradation in integrity of gypsum wallboard protection.

Keywords: Fire, Truss, Floor, Assemblies

Introduction

Historically, fire resistant construction practices for buildings have been controlled by prescriptive standards specified in building codes. Compliance with these standards has been established in-part by showing acceptable performance in full scale fire endurance testing of assemblies under ASTM E 119 and other similar international standards. Prescriptive standards are convenient in that they free building designers from technical knowledge of fire, but by their nature, prescriptive codes, stifle innovation.

Performance-based fire safety regulations in building codes are being investigated and adopted world-wide. They offer the possibility of a wide array of solution strategies for providing fire safety (Bukowski and Babrauskas 1994). New Zealand moved from a prescriptive to a performance-based building code several years ago (Buchanan and Barnett 1995). Japan and the United Kingdom have been working towards performance-based codes since 1982 (Tanaka 1994; Bukowski and Babrauskas 1994). Performance-based codes appear to be the future in building code evolution.

As stated by Tanaka (1994), "fire models are indispensable to a performance based design system..." Although, fire models have a role even in a prescriptive code environment as an alternative or supplement to standardize fire endurance testing, the main motivation for model development is the eventual application of the full flexibility of fire models with performance-based codes.

This research program has aimed at providing a more rational basis for assessing and understanding fire endurance of wood floor/ceiling assemblies. The SAWFT *(Structural Analysis of Wood Frames and Trusses)* fire endurance model reported here applies to lumber joist or metal-plate-connected wood truss structural systems with sheathing and wall board protection. A basic premise of the reseach approach is that knowledge of assembly fire endurance behavior can be established from relatively inexpensive tests of individual wood members and connections, and combined with structural mechanics and heat transfer algeorithms to predict the response of wood assemblies. Model development encompassed component testing to establish wood and connection degrade rules development of the structural analysis/fire degrade software, and evaluation of the working model. The assembly model is an extension of a single truss model previously developed. Although the theory of the model applies to a wide range of wood types, sizes, and fire conditions, component testing supporting the model has been confined to testing 2 by 4 wood members of one species and grade, and metal plate connections under load and elevated temperature. This presentation of the research builds on a previous paper by Cramer (1995).

Tests for Defining Thermal Degradation of Lumber and Connections

Tension and bending performance of lumber and metal plate connections were evaluated under surface temperatures ranging from ambient temperatures to 325°C in a tension test apparatus at the USDA Forest Products Lab. Tension and combined tension-bending tests were conducted on No.

1D grade Southern Pine lumber with actual cross section dimensions of 38- by 89-mm (here after called 2 by 4 reflecting the nominal dimensions in English units) and the same lumber spliced with metal connector plates. The metal connector plates consisted of 20 gage (0.9-mm) thick steel plates with punched teeth approximately 3.2-mm wide by 8.5-mm long. The focus of the test program was on tensile performance because bottom chord truss members that are under tensile stress often control truss fire endurance. In the middle of the tension test apparatus, a furnace of 2.1-m by 1.4-m in plan subjected 1.8-m of the specimen length to a specified-time temperature curve.

Two types of tests were conducted. In the first case, tests were conducted with either constant temperature and increasing load or constant load and increasing temperature. Constant temperature tests consisted of heating the specimen under constant temperature for 30 minutes and then applying a ramp tensile load to failure. The number of test specimens for each condition varied, but generally 5 to 10 specimens were tested at ambient, 100°C, 200°C, 250°C, 300°C and 325°C. In the second type of test, the specimens were subject to constant load of 50 or 100 percent of the designed load and exposed to a predefined time-temperature exposure that simulated temperatures measured in the plenum of a gypsum-protected truss assembly subject to ASTM E 119 test conditions.

Modeling Thermal Degrade in Lumber and Connections

The test program summarized above provided a data base for developing degrade rules for wood members and metal-plate connections. Although there have been several studies on the temperature-induced degrade of small clear wood specimens, only a couple of studies besides that described above have examined the effect of temperatures on full size lumber (Noran 1988; Lau 1994). As the data base of thermal degrade information grows, the degrade rules contained in this fire endurance model can be upgraded and expanded.

Thermal degrade of wood is extremely complex involving dehydration, shrinkage, themal expansion, and phase change leading to an increase in viscoelastic behavior and nonlinear creep deformations (Schaffer 1970). A simple characterization of this complex process is needed to yield a tractable fire endurance model. Models developed by Shrestha are currently employed in the model and are summarized below (Shrestha et al. 1995).

The degraded mechanical properties of dimension lumber for a known exposure condition of a specified duration are related to the corresponding properties at ambient temperature as shown in Eq. 1.

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

In Eq. 1, W_o = wood property at room temperature such as modulus of elasticity, modulus of rupture, and tensile strength.

W_t= corresponding wood property after exposure of time t.

t = time of exposure (min.)

T_o= room temperature at time 0 (degree C)

T_c= temperature at the center of the section (degree C)

γ = empirical function that fits Eq. 1 to the degrade test data

$\gamma_E = T_c^{0.05}/100$ for modulus of elasticity

$\gamma_{t\text{-}c} = (T_c/198)^4$ for tensile strength

$\gamma_{MOR} = (T_c/215)^6$ for bending strength (modulus of rupture)

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

Equation 1 requires knowledge of the center temperature of a section. This temperature can be determined by: conducting a detailed heat transfer analysis such as proposed by Gammon (1987), Mehaffey et al. (1994), or Fredlund 1993); using thermocouple data from components tested under similar conditions; or using a simplified heat transfer analysis as proposed by Shrestha and others (1994). The simplified heat transfer algorithm by Shrestha is used in the SAWFT fire endurance model. The temperature profile across the section of a 2 by 4 is approximately parabolic and flattens as thermal equilibrium is reached (White et al. 1993). The cumulative area under the temperature profile from each step in time is approximated by Eq. 2.

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

In Eq. 2, t and Δt = time and incremental time (min.), T_o, T_c, and T_s = initial (t=0), center, and surface temperatures (degree C) of the section. It is an unverified hypothesis that the cumulative effect of time-temperature as quantified by A_t is a key parameter in the degrade of wood members. Future research should provide an answer to this hypothesis and the degrade rules can be refined appropriately.

The displacement at the metal-wood interfaces associated with the metal-plate connections in trusses are modeled with the function shown in Eq. 3.

$$F = \frac{M_o}{1.5}\arctan\frac{1.5(K - M_1)\Delta}{M_o} + M_1\Delta \qquad (3)$$

In Eq. 3, F = force applied to the metal-wood connection surface, Δ = deflection of the metal-wood connection surface.

The three parameters in Eq. 3, K, M_1, M_0 are commonly used in North America to characterize the load-displacement curve of metal-plate connections and were first introduced by Foschi (1977). K describes the initial near-linear portion of the curve, and M_1 and M_0 characterize the nonlinear portion of the curve at higher loads. Degrade equations based on the component tests of metal-plate connections described above are shown in Eq. 4 (Shrestha et al. 1995).

$$\begin{aligned}
\frac{K^t}{K^o} &= 1.0-0.2T_g-T_o \quad for\ T_g \le 200\,^{\circ}C \\
\frac{K^t}{K^o} &= 0.8-0.5\frac{T_g-200}{100} \quad for\ T_g > 200\,^{\circ}C \\
\frac{M_o^t}{M_o^o} &= 1.0-0.1\frac{T_g-T_o}{250-T_o} \quad for\ T_g \le 250\,^{\circ}C \\
\frac{M_o^t}{M_o^o} &= 0.9-0.4\frac{T_g-250}{50} \quad for\ T_g > 250\,^{\circ}C
\end{aligned} \tag{4}$$

The superscripts in Eq. 4 refer to the connection properties at room temperature (T_0) and at the current exposure temperature (T_g). The third connection parameter, M_1, has been set to zero for all cases.

Structural Analysis Model

The degrade equations described above serve to modify the properties of individual wood components and connections as a function of the time-temperature exposure of the assembly. At each time step, a geometric nonlinear and connection nonlinear structural analysis of the assembly is conducted. As previously described, the model accommodates two types of floor/ceiling assemblies, joist floor systems and metal-plate-connected truss systems. Wood members are modeled as two-dimensional linear, elastic frame members that may be subject to large end displacements. The structural model can accommodate members of nearly any size and span, but it must be remembered that the degrade roles presented earlier have only been developed for 2 by 4 lumber of one species. Metal-plate-connectors are modeled as a linkage of nonlinear springs that describe the eccentricity at member connections and the lateral resistance characteristics of the metal-connector-plates (Cramer et al. 1993).

Sheathing on the top surface of the assembly, consisting of plywood, oriented strand board or other structural products, is modeled in two ways. First, the sheathing is treated as a series of continuous beams crossing the joists or trusses. These linear, elastic beams serve to distribute load among the joists or trusses as they differentially deflect because of differences in room temperature properties, uneven load distribution or uneven high temperature exposure. Secondly, the sheathing is modeled with special elements developed at the Univ. of Texas at Austin that account for the partial composite action associated with sheathing that is nailed or glued to the joist of top chord of the truss (Warner and Wheat 1988). This model assumes that the connection between sheathing and wood is linear, elastic, there is no fiction between the sheathing and wood members, deflections are small, and there are small gaps in the sheathing at member ends. The partial composite action of the sheathing provides a slight increase in the overall stiffness of truss and (to a greater degree) joist assemblies. For simplicity, the properties of the top sheathing are assumed to be unaffected by elevated temperatures, but any structural contributions imparted by protective layers of gypsum wall board on the bottom surface of the assembly are ignored.

The structural model was originally developed for the fire endurance of a single truss (Shreatha 1992). Explicit structural modeling of the connector plates is included in the procedure. Table 1 shows a comparison of the predictive accuracy of the accepted U.S. design model with the SAWFT structural model for parallel-chord trusses whose forces and deflections were measured under nonfire conditions at design load. Table 1 provides evidence that although the SAWFT model is not perfect in modeling the performance of metal-plate-connected wood trusses, it offers a level of accuracy and realism that matches and exceeds the accepted U.S. design approach (TPI/ANSI 1995). The truss tests and data analysis were conducted independent of this research at the Univ. of Texas at Austin (King and Wheat 1988). The key to the improved accuracy achieved in Table 1 can be attributed to the metal-plate-connection model in SAWFT.

Failure of wood members in the SAWFT model is predicted using the criteria presented by Zahn (1986). The designer's form of these criteria have been adopted in the 1991 National Design Specification® for Wood Construction (AF&PA 1995) but the ultimate strength form is used in the SAWFT model. The failure criterion for metal-plate connections is indicated in Eq. 5.

$$PFI = 1-\frac{K_t}{K_o} \tag{5}$$

In Eq. 5, PFI = plate failure index,
K_t = current tangent stiffness value of the plate-wood contact surface at the given loading and temperature condition,
K_o = original tangent stiffness value of the plate-wood contact surface at zero load and prefire temperature condition.

Table 1 - Error in Computed Truss Forces and Deflections for the SAWFT Model and the Current U.S. Design Model

	SAWFT	Current U.S. Design Model
Errors in Axial Forces (13 measured forces in 5 parallel chord trusses)	18%	28%
Errors in Bending Moments (18 measured moments in 5 parallel chord trusses)	20%	70%
Errors in Truss Deflection (12 measured deflections in 5 parallel chord trusses)	8%	29%

The PFI value indicates the consumed capacity of the plate-wood contact surface in decimal percent. The values range between 0.0 at no load to 1.0 at failure. The attractive feature of this form of the failure index is that the ultimate strength values for a plate of given size and type are typically unknown, hard to obtain, but not needed in this criterion. Stiffness values can be estimated from the published literature.

SAWFT Model Evaluation

The SAWFT fire endurance model has been evaluated by comparing fire endurance times with seven full scale joist or truss assemblies that have been subject to ASTM E 119 testing. Table 2 shows the details of each assembly.

When analyzing the above assemblies for fire endurance the following input information is required by SAWFT:
* the sizes and mechanical properties of wood members and connecting plates,
* the properties of the attachment scheme between the flooring and the supporting wood members, (structural contributions of attached gypsum are assumed minor and ignored)
* the time-surface temperature exposure for wood members and connections,
* the support conditions of the assembly,
* the physical load on the assembly.

Although all of the above information cannot be directly obtained from a typical ASTM E-119 test report, that data not presented can usually be estimated from information contained in the report. When analyzing structural assemblies, the structural model chosen for analysis depends on the objective and detail provided. For example, if each truss in an assembly has identical properties, identical loading and is subject to the same time-temperature exposure, each truss will fail at the same instant as the neighboring truss. In such a situation it is logical to analyze a single representative truss (or joist) from the assembly rather than attempting to simulate the whole assembly.

Table 3 shows the predicted fire endurance times for the seven evaluation assemblies considering only a single representative component Input data to the model consisted of averages of data contained in the test report and general estimates of properties inferred form the test report for the assembly. Comparison of the measured and predicted fire endurance times in Table 3 leads to several observations. Using the average thermocouple temperatures provided in the respective test report led to fire endurance predictions that are within 10 percent of the measured values for more than half of the evaluation assemblies, but are conservative by approximately 25 percent for the remaining three assemblies. Using the near-minimum thermocouple temperatures brought the fire endurance predictions within 10 percent of the actual values for all truss assemblies but caused overprediction for the two joist floor fire endurances. Recall that the wood degrade equations were developed from tests of Southern Pine 2 by 4's and thus analysis of the Spruce-Pine-Fir and the laminated strand lumber joist floors is a dramatic extrapolation of the SAWFT model.

The model consistently predicts plate connection failure in the truss assembies unless the temperature conditions consist of moderate increases over an extended period as in FC-426. If the metal plate connections are not allowed to degrade, the fire endurance predictions improve under average thermocouple conditions suggesting that the current metal-plate connector degrade model may be overly conservative. If maximum thermocouple temperatures are used, the model becomes unacceptably conservative even though in the actual tests the maximum thermocouple temperatures may be responsible for the onset of failure.

Model Limitations

The two most significant limitations of the model for fire endurance modeling are:
* the built-in degrade rules for wood members and metal-plate-connectors are based on results from test programs that have examined wood of one species and one size, and connections under a limited set of conditions,
* time-temperature conditions for wood members resulting from a given protection scheme must be input into the model and are assumed independent of structural repsonse. There is not a linkage within the model between structural deflection and the integrity/effectiveness of the gypsum protection scheme.

Table 2 - Structural Assemblies Tested for Fire Endurance and Used for SAWFT Evaluation

Assembly ID	Main Load Carrying Elements	Sheathing and Fire Protection Details
FC-214 (Factory Mutual Research 1978)	8 parallel chord trusses 305 mm deep and 5.23 m long spaced 368 to 610 mm oc and consisting of 4 by 2 Hemlock, Douglas-fir, or Southern Pine members	15.1 mm plywood flooring attached to top chords with 6 d common nails spaced 152 mm oc and two layers of 13 mm gypsum board attached directly to bottom chords.
FC-235 (Factory Mutual Research 1976)	6 parallel chord trusses 305 mm deep and 5.23 m long spaced 610 mm oc and consisting of 4 by 2 Southern Pine members	19.1 mm plywood underlayment glued to top chords and one layer of 15.9 mm gypsum board attached directly to bottom chords
FC-249 (Factory Mutual Research 1977)	6 parallel chord trusses 305 mm deep and 5.23 m long spaced 610 mm oc and consisting of 4 by 2 Southern Pine members	19.1 mm plywood flooring attached to top chords with 6 d common nails spaced 152 mm oc and one layer of 15.9 mm gypsum board attached to bottom chords via resilient channels.
FC-426 (PFS Corporation 1986)	7 parallel chord trusses 356 mm deep and 4.06 m long spaced 610 mm oc and consisting of 4 by 2 Southern Pine members	18.3 mm plywood flooring nailed and glued with construction adhesive to top chords and two layers of 15.9 mm gypsum attached to bottom chords via resilient channels
L-528 (Underwiters' Laboratories 1981)	9 parallel chord trusses 305 mm deep and 4.22 m long spaced 610 mm oc and consisting of 4 by 2 lumber members	18.3 mm plywood flooring nailed and glued to truss top chords and one layer 15.9 mm gypsum board attached to bottom chords via resilient channels.
FC-504 (PFS Corporation 1994)	13 lumber joists 38.1 mm by 235.0 mm by 4.13 m No. 2 Spruce-Pine-Fir spaced 406 mm oc.	11.9 mm underlayment over 15.1 mm plywood nailed and glued with construction adhesive to the top of the joists and one layer of 15.9 mm gypsum board attached to the bottom of the joist via resilient channels.
FC-503 (PFS Corporation 1994)	13 laminated strand lumber (LSL) joists 38.1 mm by 235.0 mm by 4.13 m spaced 406 mm oc.	11.9 mm underlayment over 15.1 mm plywood nailed and glued with construction adhesive to the top of the joists and one layer of 15.9 mm gypsum board attached to the bottom of the joist via resilient channels.

To illustrate this last limitation consider the floor joist simutations for FC-504 and FC-503. The protection scheme for each floor joist assembly was identical in design.

Assembly FC-504 was loaded to its full design load for the S-P-F joists and FC-503 was loaded to just over one-half the design load for the LSL joists. The LSL joists had a slightly greater stiffness than that for S-P-F joists established from published values. The LSL joists had approximately twice the allowable stress of the S-P-F joists. Aithough, the protection schemes were identical, the LSL system had a fire endurance time 40% greater. Both systems failed at an average plenum temperature of approximately 400°C.

It is clear that the integrity of the gypsum protection scheme was maintained longer in the LSL system, because both systems failed at the same average plenum temperature. Two possible reasons for this difference are: 1) a natural variation in the construction and performance of the gypsum protection scheme, or 2) the lower deflections associated with stronger and stiffer LSL maintained the integrity of the gypsum protection for a longer period. If either or both reasons are true, this presents a significant challenge for fire modeling. If variations in the endurance of the gypsum protection scheme are a function of construction details, then idealized heat transfer models under development will have great difficulty predicting these variations. If the fire endurance of the gypsum protection scheme depends on the structural response of the assembly, then the current version of SAWFT needs to be enhanced to link structural deflections with

Table 3 - Fire Endurance Times Measured and Computed with SAWFT for the Evaluation Assemblies

Assembly ID	Measured Fire Endurance Time (minutes)	Predicted Fire Endurance Time (minutes)			Predicted Critical Element for Avg. Temps
		Avg. Thermocouple Temperatures	Min. Thermocouple Temperatures	Avg. Temps. - No Plate Degrade	
FC-214	76	76	83	78	plate connection
FC-235	50	38	48	48	plate connection
FC-249	58	52	55	55	plate connection
FC-426	112	87	118	91	wood bottom chord
L-528	60	45	58	51	plate connection
FC-504	46.5	44	54	NA	wood joist
FC-503	65	59	68	NA	LSL joist

gypsum sheathing integrity and plenum temperatures.

SAWFT Model Application

Simulations of previously conducted ASTM E 119 tests do not illustrate the full capability of the SAWFT model. The full capability is realized when examining assemblies where conditions are not uniform. In reality, this includes all assemblies in real fires and to a lesser degree all assemblies subject to ASTM E 119 fire endurance testing. Test reports for ASTM E 119 fire endurance testing do not contain sufficient detail on nonuniformity in structural member properties and thermal conditions. Variations in wood properties and plenum temperatures are real and increased model realism is achieved by considering them.

Reconsider the joist floor evaluation assembly (FC-504). By randomly selecting modulus of elasticity for the joists representative of No. 2 grade Spruce-Pine-Fir and by assuming that bending strength is strongly correlated with modulus of elasticity, nonuniform joist properties will exist that are potentially similar to the real assembly tested. Using average temperatures the SAWFT full assembly model of 13 joists with randomly selected properties representative of Spruce-Pine-Fir joists and attached plywood sheathing predicts first member failure in 44 minutes. This is the same as the single joist model failure prediction, again suggesting that variations in wood properties have a relatively minor effect on fire endurance. Full collapse, however, does not occur with first member failure because the attached sheathing will provide some load distribution away from the weak and failing joist. Full collapse is predicted by the model two minutes later at 46 minutes and thus, the assembly effects provided only a minor increase in fire endurance.

Many other scenarios are possible. For example, the highest predicted deflection in the joist floor with no fire exposure, occurs in a joist that through random selection has an unusually low modulus of elasticity. If this higher deflection leads to a breakdown in the gypsum protection scheme, it is possible that this joist will be subject to a higher set of time-temperature exposures than other joists. Suppose this limber interior joist is subject to the average thermocouple temperatures of the joist floor system along with a neighboring joist. Assume that the gypsum protection of all other joists retains a higher degree of integrity and these joists are subject to only the minimum thermocouple readings from the E 119 test data. In such a scenario, first joist failure is predicted to occur in 52 minutes at the limber joist with complete collapse occurring at 54 minutes. The predicted fire endurance of the assembly has increased by 8 minutes under this nonuniform temperature exposure that simulates only a localized breakdown of the gypsum wall board protection scheme. The key to this increase in duration is that stiffer neighboring joists subject to less severe temperature exposures carry the load away from the limber joist with the severe temperature exposure. Many other scenarios and even probabilistic simulations can be examined with the SAWFT fire endurance model.

Summary and Conclusions

Anew fire endurance tool (SAWFT) has been developed that allows prediction of the fire endurance of wood floor/ceiling assemblies consisting of either metal-plate-connected trusses or lumber joists. The model is a three-dimensional structural analysis tool that computes structural performance of a wood assembly based upon the degrade of mechanical properties with exposure to elevated temperatures. The model has a built-in heat transfer algorithm to compute how high temperatures penetrate the wood members and a property degrade algorithm to predict how wood members degrade under elevated thermal exposure. The major limitations of the model are the limited applicability of the degrade routines and the lack of an interdependence between computed structural response and the resulting temperature exposures allowed by the gypsum wall board protection.

The model provides conservative, but reasonably close predictions of fire endurance of seven evaluation assemblies that were subject to E 119 full-scale testing. The model has been shown to be most sensitive to time-temperature data provided in the input emphasizing the need for an advanced heat transfer model or other reliable knowledge of the performance of gypsum wall board protection schemes when using the model. The assembly model allows examination of a wide variety of fire endurance scenarios for wood structural assemblies. Simulation of these scenarios provides insight in assembly performance that can be used to optimize designs before E 119 testing. The full capability of the model will be useful with performance-based building codes. With further development and future application, the model offers a tool that can provide supplemental information for code acceptance of wood assemblies. With further development and verification, the SAWFT model offers a computational alternative to full-scale testing.

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Acknowledgements

The development of this model has received direct and indirect contributions from many. Specifically, the thesis work of former University of Wisconsin-Madison graduate students, Dr. Deepak Shrestha and Troy Opgenorth provided key building blocks to the model. The financial support of the American Forest and Paper Association made the development of the model possible. The assistance of Dr. Marc Janssens formerly of the American Forest and Paper Association is gratefully acknowledged. The Truss Plate Institue, the Canadian Wood Council, and Trus Joist MacMillan provided valuable fire endurance data.

In: Gopu, Vijaya K.A., ed. Proceedings of the international wood engineering conference; 1996 October 28-31; New Orleans, LA. Baton Rouge, LA: Louisiana State University: Vol. 2: 249-256