BEHAVIOR OF WOOD EXPOSED TO FIRE: A REVIEW AND EXPERT JUDGMENT PROCEDURE FOR PREDICTING ASSEMBLY FAILURE

by

Kenneth Edward Bland

A Thesis

Submitted to the Faculty

of the

WORCESTER POLYTECHNIC INSTITUTE

in partial fulfillment of the requirements for the

Degree of Master of Science

in

Fire Protection Engineering

by

May, 1994

APPROVED:

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Abstract

Wood has been the material of choice for owners and builders of residential and low-rise commercial buildings for years. Many incentives exist to use wood construction including structural integrity, appearance, cost, ease of construction, and energy savings. However, wood by its very nature is combustible, which warrants attention to insure that the built environment is safe. Over the years, fire research has been performed on wood and wood based products to evaluate and document characteristics useful towards achieving this goal. This research has provided a wealth of information on topics, such as how fast a flame spreads across the surface of wood; how much smoke is produced during combustion; at what rate does wood char and what variables influence this rate; and, at what rate is heat released.

This paper summarizes research on the structural performance of wood elements and assemblies exposed to fire, and reviews methodologies available to predict performance. Two factors significantly influence the structural performance of a wood assembly when exposed to fire; the load which it is supporting and the intensity of the fire. A review of the derivation of design values, safety factors and ultimate capacities of wood strength and stiffness characteristics is provided. Research significant to the prediction of structural behavior of wood exposed to fire is summarized, including the effect of elevated temperatures on design values and char rates. A review of the testing and empirical methodologies developed from exposing various wood elements and assemblies to fire is provided with examples.

Lastly, fire testing and research to date has not adequately provided a means to integrate the combined effects of live load, fuel load, and membrane protection. A procedure is presented which uses expert judgement of fire researchers, aided by empirical methodologies, to assign probabilities to the performance of an assembly exposed to fire. Although this methodology introduces subjective reasoning, the basis for the experts' conclusions is substantiated by their knowledge of factors influencing fire growth and reaction by assemblies. Conclusive results are not developed from the expert data, although improvements to the methodology for future exercises are discussed.

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LIST OF SYMBOLS

A - cross sectional area
B - coefficient incorporating specific gravity
B - joist modulus of rupture
B - breadth (smaller dimension)
b - initial joist width
C - specific heat capacity
CS - percent of original compressive strength
c - adjustment factor (ASTM D2555)
c - hardwood-softwood classification
D - depth (larger dimension)
d - net surface depth (page 15)
d - joist depth
d - depth of CCA penetration
E - Modulus of elasticity (psi)
E - activation energy
EL - 5% clear wood exclusion limit (ASTM D2555)
F - size factor
F_b - extreme fiber bending (psi)
$F_{c\perp}$ - compression perpendicular to the grain (psi)
F_c - compression parallel to the grain (psi)
F_t - tension parallel to the grain (psi)
F_v - horizontal shear (psi)
J - Joule's constant
k - ratio of design load to ultimate load
L_v - heat of gasification
M - moisture content
M - bending moment
MC - moisture content
m - elapsed exposure time
m - reciprocal of the char rate (min/mm)
m - mass of wood sample
m_o - mass of dry wood sample
p - oven-dry density or specific gravity
q_n - net heat flux to uncharred wood
R - gas constant
R - thickness of char layer
r - total applied load/breaking load
S - section modulus (in^3)
S - depth/breadth
s - standard deviation

T - constant furnace temperature (K)
TS - percent of original tensile strength
t_{b3} - time to reach critical dimension (three sided beam)
t_{b4} - time to reach critical dimension (four sided beam)
t_{c3} - time to reach critical dimension (three sided column)
t_{c4} - time to reach critical dimension (four sided column)
T_c - temperature in degrees celsius
T_p - average pyrolysis temperature of wood
T_o -initial temperature of wood
t - time to failure
t - time
t_f - time duration of fire
u - percentage moisture content
v- char rate (in/min)
w - moisture content
x - average strength value of the design property (ASTM D2555)
x - char layer depth
x_c - char depth measured from original surface
Y - the average measured design value for the species (ASTM D 2555)
z - load factor
α - strength reduction due to internal heating
β - char rate
γ - exposed joist fire performance factor relating normal temperature strength to high temperature strength
δ - finite thickness of residual wood which is weakened by the elevated temperature and moisture.
ρ - density

CHAPTER 1

DERIVATION OF DESIGN VALUES FOR WOOD CONSTRUCTION

1.1. Introduction.

Prior to any discussion on the characteristics of wood and wood based products under fire exposure, it is appropriate to review the derivation of their strength and stiffness design. The ability of a wood structural member to support load during fire exposure, is a function of many variables including; charring rate, membrane protection, fire exposure, load ratio and the ratio of perimeter area to cross-sectional area. Because of the variability in strength between any two pieces of wood, a fairly conservative derivation method for design values has been developed. History has shown that wood frame construction is generally over designed.

Prescriptive requirements found in model building codes, are attributed more to the demonstrated performance in the field over the years, rather than to the level that engineering calculations would indicate. The success of wooden structures in withstanding natural and man made loads is due in most part to the load sharing and redundancy inherent in wood frame construction. Typical light-weight wood frame construction consists of an assembly of parallel members, connected by sheathing material which uniformly distributes load. Roof and floor assemblies act as large plates, absorbing vertical gravity loads and horizontal shear loads, transferring these loads to walls. Wall studs carry the gravity loads to the foundation, while the wall sheathing provides racking resistance from the horizontal forces. This interaction of structural assemblies results in a 3-dimension resistance to simultaneous loads.

Despite the success of prescriptive construction methodologies, a system of working stresses was necessary and today, enables wood structures to be engineered. The procedures for determining allowable design values of wood species are maintained through the national consensus process of the American Society for Testing and Materials (ASTM). Prior to 1990, there were three ASTM standards which were used to assign design values to dimension lumber. They were; ASTM D 143 [1], ASTM D 2555 [2] and ASTM D 245 [3] and are discussed in more detail in Section 1.2 , 1.3 and 1.4, respectively.

As a result of testing begun by the wood industry in 1978, new design values have been introduced as determined in accordance with ASTM D 1990 [4]. ASTM D 1990 does not provide an entirely new set of design values, but provides a new methodology for establishing the allowable stresses of specific design values. ASTM D 1990 is discussed further in Section 1.5.

The recognized source of design values for all commercially available species is the *National Design Specification for Wood Construction®*, (*NDS®*) [5] and its supplement, *Design Values for Wood Construction* [6] as published by the American Forest & Paper Association. The 1986 edition of the *NDS* [7] and the 1988 edition of the supplement [8], provide design values based entirely on ASTM D 245 and ASTM D 2555. The 1991 *NDS* and supplement provide design values based upon ASTM D 1990, ASTM D 245 and ASTM D 2555, as illustrated by Table 1.

TABLE 1.1

ASTM STANDARDS RELATED TO 1991 *NDS* DESIGN VALUE DERIVATION						
SIZE OF SPECIMEN	DESIGN VALUE					
	F_b	F_t	F_v	$F_{c\perp}$	F_c	E
2" - 4" THICK 2" and WIDER	D1990	D1990	D245 D2555	D245 D2555	D1990	D1990
5" or more THICK 5" or more WIDE	D 245 D 2555					

1.2. ASTM D 143.

ASTM D 143 establishes the testing procedures for determining the strength value of straight grained, clear, 2" x 2" wood samples of various lengths. The length dimensions vary based upon the strength value which is being tested. The rate of loading is selected to cause failure of the sample in 5 to 10 minutes. As discussed in Section 1.4.5, this rate of loading is important in establishing the short term strength properties of wood products, as discussed in Section 1.4.5.

Test results are reported in terms of an arithmetic average of the ultimate strength. Individual species are tested independently and the average strength for that particular species is recorded. These values are referred to as the unadjusted clear wood strength values.

1.3. ASTM D 2555.

Two methods are presented in ASTM D 2555 for assigning design values from the unadjusted clear wood strength values; Method A and Method B. Method A relies on density surveys of major commercial species and test results from ASTM D 143. Although this method provides accurate results of wood strength from species in specific geographical regions, limited survey records are available for the majority of United States timber stands. Therefore, this method is seldom used.

Alternatively, Method B utilizes the strength test data resulting from ASTM D 143. An approximate standard deviation of the unadjusted clear wood strength values, is estimated from the equation:

$$s = cY \tag{1}$$

where:

s = standard deviation
Y = the average measured value for the species, and
c = 0.16 for modulus of rupture,
0.22 for modulus of elasticity

0.18 for maximum crushing strength parallel to grain
0.14 for maximum shear strength
0.128 for compression perpendicular to the grain
0.10 for specific gravity

The values for c are based on average relationships from years of accumulated test results from ASTM D 143. Results of equation 1 are presented in tabular form in ASTM D 2555 for the predominate softwood and hardwood species found in the United States and Canada. These are unadjusted clear wood strength value.

The development of working stress design values from the unadjusted clear wood strength values considers many variables associated with "real world" lumber. The clear wood strength values for modulus of rupture (MOR), tension parallel to the grain (F_t), compression parallel to the grain (F_c) and shear strength (F_v), are reduced to a 5th percentile exclusion value. The 5th percentile exclusion value lies 1.64 standard deviations below the average strength. Determination of the 5th percentile exclusion limit value is in accordance with the equation:

$$EL = x - 1.645s \qquad (2)$$

where: EL = 5th percentile clear wood exclusion limit
x = average strength value of the design property
s = estimated standard deviation of the design property

For repetitive member assemblies, using the 5th percentile exclusion value to represent the design strength of a species, introduces a safety factor seldom recognized by designers. In reality, using the stress of approximately the 5th weakest member out of 100 is very conservative, considering the load distribution and system effects which occur in the composite structure.

Hoyle and Woeste [9] suggest that other exclusion limits may be appropriate based upon the consequence of structural failure. Rather than using the 5th percentile exclusion value as the allowable design stress for wood elements exposed to fire, another exclusion value may be more appropriate. For example, the Forest Product Laboratory, *Wood Handbook* [10] lists the mean modulus of rupture (MOR) for commercially available species. Using the MOR is less conservative than the 5th percentile value, but given assumptions used in predicting fire growth, this approach may be acceptable. The 5th percentile exclusion value provides a relatively high probability that any given member will have capacity well beyond the capacity for which it is designed. Thus, if all conditions, i.e. loading and fire, are assumed to occur simultaneously, the cumulative safety factor will be high. For this reason, it is appropriate to consider using mean values for strength when designing under fire conditions. This is consistent with Eurocode 5 [11] which allows for mean strength and stiffness design values to be used for fire design.

1.4. ASTM D 245.

ASTM D 245 provides the guidelines for adjusting the 5th percentile exclusion value for clear wood strength, since 2" x 2" green, clear, straight-grain wood is not representative of the wood available in the market place. There are a number of adjustments which are made to arrive at the working stress. The following table, which has been reprinted from ASTM D 245, summarizes the possible adjustments to each design value.

TABLE 1.2

MODIFICATION OF PROPERTIES BY GRADE AND USE FACTORS						
Kind of Allowable Stress	Size Classification	Allowable Stress modified by:				
		Grade	Rate of Growth	Density	Season-ing	Duration of load
1	2	3	4	5	6	7
Extreme fiber in bending and tension parallel to grain	1-in. nominal boards	yes	yes	yes	yes	yes
	joists and planks	yes	yes	yes	yes	yes
	beams and stringers	yes	yes	yes	no	yes
	posts and timbers	yes	yes	yes	no	yes
Horizontal Shear	all sizes	yes	no	no	yes	yes
Compression perpendicular to grain	all sizes	no	yes	yes	yes	no
Compression parallel to grain	all sizes	yes	yes	yes	yes	yes
Modulus of elasticity	all sizes	yes	no	yes	yes	no

In the following sections, each of the factors listed in Table 1.2 affecting design values are discussed.

1.4.1. Grade adjustments.

The most familiar adjustment to the 5th percentile exclusion value for clear wood

(CWV), as derived in ASTM D 2555, is the strength ratio for growth characteristics. With the exception of compression perpendicular to the grain, all of the design values listed in Table 1.2 are influenced by the grade of the wood. Compression perpendicular to the grain is not a function of the number or size of growth characteristics, but rather the deformation of the fibers, perpendicular to the grain.

As defined in ASTM D 245, the strength ratio represents; "the anticipated proportionate remaining strength after making allowance for the effect of maximum permitted knots, cross grain, and the like in a given grade, as compared to clear, straight-grained lumber". Assignment of the strength ratio is made by a series of equations provided in the appendix of ASTM D 245. The less severe the growth characteristic, the closer the strength ratio is to 1. For lower grade material, significant reductions in the CWV are common.

1.4.2. Rate of growth.

Column 4 of Table 1.2 allows for strength modification increases based on Rate of Growth. These increases apply to Douglas-fir from the Coast Region, redwood, and southern pine.

Earlier versions of ASTM D 245 allowed Rate of Growth modifications to be sub-divided as close grain or medium grain. Increases were only allowed for close grain. The close grain increase is based upon the number of growth rings per inch measure radially to the growth rings, with no requirements for minimum percentage of summerwood. A 7% increase in design values was permitted, with the exception of horizontal shear. No increase was allowed for modulus of elasticity. Currently, rate of growth modifications are contained in the density modifications.

1.4.3. Density.

The density stress modification indicated in column 5 of Table 1.2 is a function of the density of the growth rings and the presence of minimum summerwood per linear inch of radius. Density increases are permitted for Douglas-fir and southern pine species. When lumber meets the dense criteria, the design values, with the exception of horizontal shear, may be increased by 17% The modulus of elasticity may be increased by 5%.

1.4.4. Seasoning.

The seasoning allowable stress modifications do not apply to all sizes of lumber. For lumber less than 4" nominal in thickness, which includes joist and planks, increases for drying are permitted for each of the allowable stresses. The clear wood value derived in ASTM were taken from green wood. Wood which is enclosed as an element of a wall or floor is required to have a moisture content equal to or less than 19%, prior to enclosing BOCA® *National Building Code* (*NBC)* [12]. As wood dries from a moisture content at fiber saturation (i.e. green) to 19% moisture content, the fiber strength increases.

The exception to the seasoning adjustment are beams and stringers and posts and timbers in bending. Because these members are greater than 4 inches in thickness, it is common for shrinkage defects to occur during drying due to internal stresses. Compressive strength is not affected by these characteristics in larger structural members.

1.4.5. Duration of load.

The duration of load strength modification listed in column 7 of Table 1.2, adjusts the design values for the cumulative period of time, during which the structural element is exposed to the design load. Wood elements, such as studs, rafters and joists, have the ability to resist short duration loads at increased stress levels. Conversely, permanent loads require that allowable stress values be reduced. Since the resistance of the fibers varies with the duration of load, the methodology was developed to adjust the design values accordingly. This effect is entirely independent of the probability of the load occurring on any given structural element.

Wood [13] developed the duration of load concept, based on testing performed at the Forest Product Laboratory, United States Department of Agriculture, Madison, Wisconsin, during the 1930's and 1940's. It has long been recognized that the strength of wood is related to duration of the loading condition. The ability of wood to respond differently to varying time duration of loads is approximated by the following hyperbolic equation:

$$y = \frac{108.4}{x^{0.04635}} + 18.3 \tag{3}$$

where x is the duration of stress in seconds and y is the stress in terms of the standard data. The graphical representation of this equation is commonly know as the Madison Curve as shown in Figure 1.1.

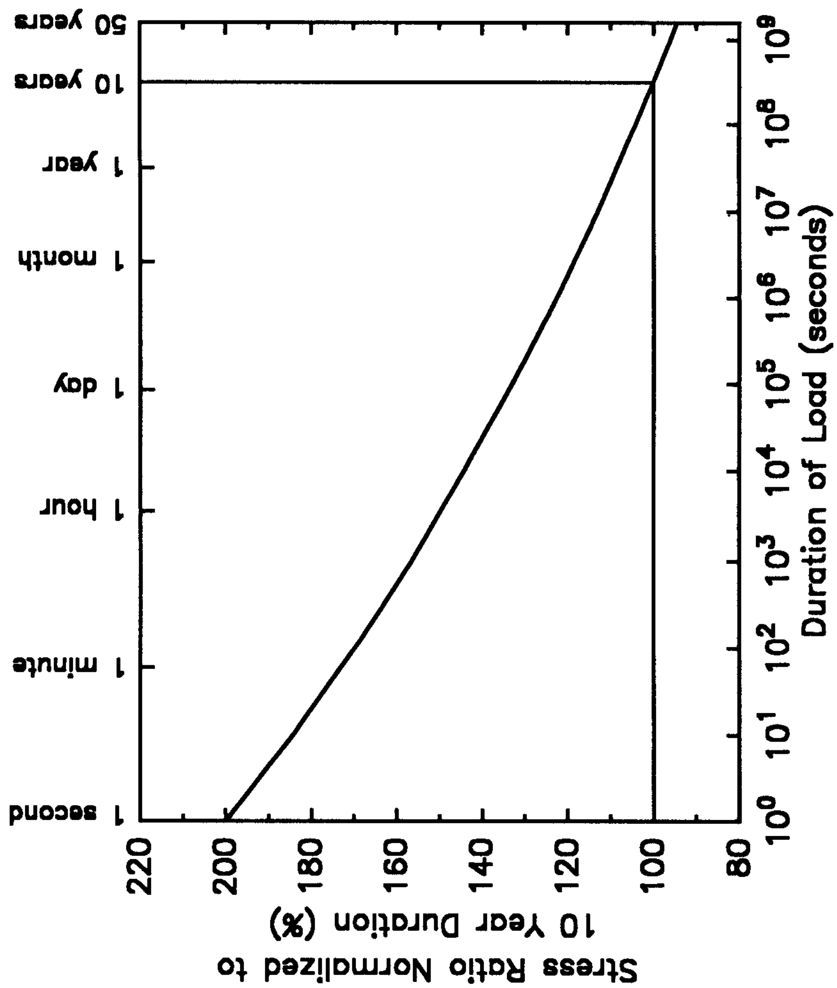

Figure 1.1 Madison Curve

The duration of load factor from the Madison Curve, used to adjust from the 5 minute test period to the 10 year cumulative loading, is approximated for the 5th percentile specimen to equal 1.62. Allowable design values are presented in the 1991 *NDS* for use in normal loading conditions or 10 year cumulative loading. The normal load period is based on the probability that during the full life of the structure, a structural element will experience a full design load for a cumulative time of ten years. Equation 3 is normalized such that the "10 year load" has a DOL of 1.0. The 1991 *NDS* provides the following common duration of load factors, derived from the Madison curve, for the expecting loading conditions specific in ASCE 7 [14]:

DOL FACTOR	LOAD CONDITION
0.90	Permanent (life)
1.00	10 years (normal)
1.15	2 months (snow)
1.25	7 days (construction)
1.60	10 minutes (seismic and wind)
2.00	instantaneous (impact)

Other DOL factors can be calculated if the time duration of the load is known.

As previously stated, ASTM D 245 incorporates the DOL conversion factor into the adjustment factor. In order to determine a safety factor in the adjustment factor, the values were divided by the conversion factor of 1.62. As shown in Table 1.3, the safety factors vary based on the strength or stiffness property considered.

TABLE 1.3

CONVERSION FACTORS AND FACTORS OF SAFETY FOR SOFTWOOD & HARDWOOD BASED ON ASTM D 245				
	SOFTWOODS		HARDWOODS	
	ASTM D 245 CONVERSION FACTOR	FACTOR OF SAFETY	ASTM D 245 CONVERSION FACTOR	FACTOR OF SAFETY
BENDING STRENGTH	2.1	1.3	2.3	1.42
MODULUS OF ELASTICITY IN BENDING	1.06	1.72	1.06	1.72
TENSILE STRENGTH PARALLEL TO GRAIN	2.1	1.3	2.3	1.42
COMPRESSIVE STRENGTH PARALLEL TO GRAIN	1.9	1.17	2.1	1.3
HORIZONTAL SHEAR STRENGTH	4.1	2.53[1]	4.5	2.78

Note 1: 9/4 of this value is related to stress concentration

1.4.6. Size factor.

The 2" x 2" test samples used in ASTM D 143 are convenient for bench scale testing, but do not represent the actual lumber dimensions used in construction. A size factor, used to adjust design values from the test size to the actual width are as follows:

$$F = \left(\frac{2}{d}\right)^{\frac{1}{9}} \tag{4}$$

where: F = size factor
d = net surface depth

1.4.7. Example - design value derivation.

TABLE 1.4

DETERMINATION OF EXTREME FIBER BENDING DESIGN VALUES FOR EASTERN WHITE PINE 2" - 4" THICK, 5" & WIDER		
ASTM STANDARD	PROCEDURE	RESULT
D143	Average Modulus of Rupture from test results	4930 psi
D2555	Standard deviation from Equation 1.1	σ = 789 psi
	Clear wood 5% exclusion value from Equation 1.2	3632 psi
D245	Adjustment factor from Table 1.3 (divide by 2.1)	1730 psi
	Strength ratio for No. 2 Grade ⇒ 45% [n]	779 psi
	Seasoning Adjustment ⇒ 125%	973 psi
	Size Factor - Equation 1.5 2" x 6" ⇒ 0.894	870 psi
	Round to nearest 25 psi in accordance with the standard	875 psi

1.5 ASTM D 1990

As previously discussed in Section 1.1, some of the design values published in the 1991 *NDS* were derived in accordance with ASTM D 1990. ASTM D 1990 uses the results of full scale testing to determine allowable stresses. This methodology provides more realistic design values. Prior to testing, each piece of lumber is graded in accordance with the *National Grading Rules* [15]. The results were normalized to a standard temperature of 73°F and moisture content of 15%.

The In-Grade Testing Program (IGT), initiated in 1978, took 12 years to complete. In 1983 the Canadian lumber industry agreed to take part in the program. The testing incorporated numerous mills scattered throughout the United States and Canada. In all, some 73,000 pieces of lumber were broken. Data for each piece of lumber was recorded and evaluated. Statistical models were used to establish appropriate confidence intervals for the design values. Additional models were provided for grouping species and extrapolating results for untested species, sizes and grades.

One significant result of the IGT program was the effect of size on design values. Although size effect was always considered, the new data indicated that for the wider dimensions, the size effect was being underestimated. This has resulted in the need for the designer to adjust design values for each width, whereas, previous editions of the *NDS* grouped various widths for simplicity.

1.6. American Lumber Standards Committee.

The American Lumber Standards Committee, as appointed by the Secretary of Commerce, maintains the *National Grading Rules*. The National Institute of Science and

Technology (NIST) and the Forest Products Laboratory, provide professional support to the ALSC for reviewing changes to design values. Independent grading agencies, certified by ALSC Board of Review and using ALSC guidelines, publish grading rules. The grading rules of an ALSC approved grading agencies are specific to the species or group of species. Grading agencies have evolved to represent species specific to naturally defined geographical growing regions. The grading agency rules are essential for the specification of lumber for both aesthetic and structural uses. The assignment of design values for a species and grade are performed by the grading agency and published in a book of rules. These values then become the source for allowable design values after review and acceptance by ALSC.

At the present time, the American Lumber Standards Committee has approved the rules of the following Rules Writing Agencies

Northeastern Lumber Manufacturers Association (NELMA)
Northern Hardwood and Pine Manufacturers Assoc. (NHPMA)
Redwood Inspection Service (RIS)
Southern Pine Inspection Bureau (SPIB)
West Coast Lumber Inspection Bureau (WCLIB)
Western Wood Products Association (WWPA)
National Lumber Grades Authority (NLGA)

1.7. Loads and Resistance.

Model building codes prescribe minimum required design loads for structural elements. Most often, loading configurations are given for combinations of loads consisting of: dead, live, snow (roof), wind and/or seismic. Special loads can be required by reference to nationally recognized load standards such as ASCE 7. Deflection limits required by the building code may also control the design of structural elements.

Typically for solid and glued laminated timber, deflection will control the design of floor joist, and extreme fiber bending strength will control the design of roof rafters. Since deflection limits are more restrictive in floor construction than roof construction, stiffness will often limit the span of floor joists. Roofs are permitted up to 2 times the deflection permitted for floor joists.

1.7.1. Extreme Fiber Bending.

The extreme fiber bending (f_b, psi) is defined as the bending moment (M, in.-lb.) divided by the section modulus (S, in^3):

$$f_b = \frac{M}{S} \tag{5}$$

In accordance with the 1991 *NDS*, No. 2, 2" to 4" thick, 5" and wider dimension lumber may have an allowable base values for single member fiber bending design value (F_b') ranging from 625- 1250 psi, depending on the species selected. Adjustment factors for design values are published in the *NDS* supplement. Those adjustment factors include, size effect and repetitive member factor (Cr), among others. A repetitive member is defined as:

"Uses are intended for the design of members in bending, such as joists,

> trusses, rafters, studs, planks, decking or similar members that are in contact or spaced not more than 24 inches on centers, are not less than 3 in number and are joined by floor, roof or other load-distributing elements adequate to support the design load."

All other applications are as single members. The *NDS* permits a 15% increase of the single member F_b when the member qualifies as repetitive. For most situations, floor joists and roof rafters use repetitive configurations.

Modulus of Rupture (MOR) is occasionally referenced in research on structural behavior. MOR is the unadjusted clear wood extreme fiber bending stress given in ASTM D 2555, representing the average strength.

1.7.2. Tension parallel to the grain.

Tension parallel to the grain, f_t, is the unit axial stress causing elongation of the wood fibers. This value with reductions to account for notches and holes, shall not exceed the allowable tension parallel to the grain, F_t.

1.7.3 Compression parallel to the grain.

Compression parallel to the grain, f_c, is the unit axial stress tending to compress the fibers. For the design of columns, the 1986 *NDS* recognizes three distinct length regions; short, intermediate and long columns. The short column region uses an allowable F_c, whereas the intermediate and long regions modify F_c based on the slenderness ratio of the column. F_c' is the notation given to the adjusted allowable compression parallel to the grain for intermediate and long columns.

As part of testing conducted at the Forest Products Laboratory on compression parallel to the grain, a new column equation was developed for the 1991 *NDS*. The new equation eliminates the three distinct failure modes described previously. A discussion of column design is given in Chapter 8.

1.7.4. Modulus of elasticity.

The values for modulus of elasticity, (E), given in the 1991 *NDS* supplement are taken from the average value derived in accordance with ASTM D 245. These values are adjusted to reflect the affects of seasoning, density and grade reductions of particular species. Duration of load adjustments do not apply to the modulus of elasticity.

CHAPTER 2

MECHANICAL PROPERTIES AT ELEVATED TEMPERATURES

2.1. Introduction.

Knowledge of the strength reducing effect of high temperatures on wood fiber is necessary to understand the structural behavior of lumber when exposed to fire. It is not unusual, under non-fire conditions, for concealed spaces such as attics to reach temperatures in excess of 125°F. The slight impact and variability in load carrying capacity associated with this temperature range is typically ignored by designers. None-the-less, the 1991 *NDS* provides design value adjustments when environmental temperatures cause the structural member temperature to reach 150°F for extended periods of time. The adjustment factors assume that the occurrence of the elevated temperature is temporary. For prolonged heating above normal temperatures, further reductions in the design values may be appropriate. When member temperatures exceed 150°F, the design values provided in the 1991 *NDS* supplement are no longer valid.

2.2. Strength loss at elevated temperatures.

To evaluate the strength loss in wood members at elevated temperatures, Schaffer [16] performed tensile and compressive tests on dry (Moisture Content = 0%) specimens of Douglas-fir. The sample sizes were 1/8" thick and 1" wide. The length of the samples varied for tension and compression experiments, 10 inches and 3-3/4 inches long, respectively. The compression length was reduced to assure that the sample would fail in crushing rather than buckling. The thickness was selected such that rapid and uniform heating of the cross section would occur. Both the immediate and prolonged effect of elevated temperature were explored.

The wood temperature was regulated through heating plates containing resistive elements. Testing was performed at six temperature levels of 25, 50, 93, 140, 204 and 275°C (77, 122, 199, 284, 399, and 527°F). Schaffer had previously experimentally determined that 288°C (550°F) represented the base of the char layer, and that wood fibers in regions in excess of this temperature contributed negligible strength due to rapid charring.

Schaffer reports that loss in tensile strength is gradual from room temperature to approximately 170°C (340°F), at that point, the specimen has retained more than 92% of its original strength. At temperatures above 170°C (340°F), strength loss occurs more rapidly, with a retained strength of only 27% at 288°C (550°F). Compressive strength loss begins to decrease almost immediately upon heating with a retained strength of 15% at 288°C (550°F).

Schaffer presented an interesting discussion which supports these findings, based upon the chemical composition of wood. Wood is comprised of three constituents; cellulose, hemicellulose and lignin. The fiber of wood consists of joined cellulose molecules. The tensile strength of wood is related to the amount of cellulose present [17].

Schaffer reports that cellulose does not start to lose weight until reaching 280°C. Hemicellulose, a fiber-to-fiber bonding element, begins to lose significant weight at approximately 180°C [18] [19]. Since cellulose and hemicellulose are associated, the more significant loss of tensile strength, reported to occur at 170°C, appears to be substantiated by chemical changes. The third constituent of wood, lignin, is predominately located towards the outside of the cell wall and between cells. At approximately 55°C, the lignin begins to soften, as reported by Schaffer.

2.2.1 Strength loss associated with elevated temperature and reconditioned samples.

Knudson and Schniewind [20] tested Douglas-fir at 12% moisture content in compression and tension for immediate strength loss and retained strength after cooling. Reconditioning was accomplished by returning the sample temperature to room temperature and 12% equilibrium moisture content. Heating of 3/16" square specimens was conducted in a furnace at 8 temperatures. Again, the compression samples were shorter in length to assure a crushing failure.

Samples were tested after being heated to temperatures of 77, 122, 176, 320, 421, 500, 550°F (25, 50, 80, 116, 160, 216, 260, and 288°C). Additionally, the samples were exposed to 4 different heating periods of 5, 15, 30 or 60 minutes. Knudson and Schniewind concluded that the effect of heating time duration on elevated temperature strength or reconditioned strength, was insignificant with one exception. Samples heated to 288°C for greater than 5 minutes experienced char formation and were not