PART II: DESIGN VALUES FOR STRUCTURAL MEMBERS

2.1-GENERAL

2.1.2-Responsibility of Designer to Adjust for Conditions of Use

The Specification identifies design value adjustment requirements for service conditions generally encountered in wood construction. However, this national standard of practice does not provide generic requirements that address all possible design applications or conditions of use. Such inclusive provisions would require the use of excessively conservative and economically prohibitive reduction factors.

Final responsibility for evaluation of the loading and exposure conditions the member or structure will be subjected to and determination of the design values that are appropriate to those conditions rests with the designer. Particular attention is required by the designer to those uses where two or more extreme conditions of service converge. An example of such a use is one where it is known that the full design load will be applied continuously, that the structural members will be consistently exposed to water at elevated temperatures, and that the structural connections will be subjected to biaxial forces and moments. Assessment of the consequences of a failure of an individual member in the structure is an integral part of the designer's responsibility of relating design assumptions and design values.

2.2-DESIGN VALUES

Design values tabulated in the Specification and its Supplement are based on normal load duration, dry conditions of service, and other normal environmental and material conditions considered represented by the product and fastener strength tests used to establish the design values.

2.3-ADJUSTMENT OF DESIGN VALUES

2.3.1-Applicability of Adjustment Factors

The Specification requires adjustment of tabulated design values for specific conditions of use, geometry and stability. Such modifications are made through application of adjustment or C factors. For structural sawn lumber and glued laminated timber, adjustment factors defined in the Specification text or footnotes to Table 2.3.1 and applied to one or more of the design values are: load duration, wet service, temperature, beam stability, size, volume, flat use, repetitive member, form, curvature, shear stress, buckling stiffness, column stability and bearing area. The uses of adjustment factors with design values for round timber piles and connections are treated separately in the Specification.

The adjustment or C factors are cumulative except where specifically indicated otherwise. Example C2.3-1 demonstrates this provision.

Example C2.3-1

A 2×6 framing member having a tabulated F_b of 1200 psi is to be used as a stud spaced 16 in. on-center in a wood foundation. Special site conditions indicate that moisture content in service will exceed 19 percent for an extended period of time. The stud will resist 6 feet of soil load. The allowable F_b' for the member is:

C_D = 0.90 (2.3.2 and Appendix B)
C_M = 0.85 (2.3.3 and Table 4A factor)
C_t = 1.00 (2.3.4)
C_L = 1.00 (2.3.7 and 3.3.3.2)
C_F = 1.30 (4.3.2 and Table 4A factor)
C_{fu} = 1.00 (4.3.3 and Table 4A factor)
C_r = 1.15 (4.3.4 and Table 4A factor)
C_f = 1.00 (2.3.8 rectangular section)
C_V = not applicable (Table 2.3.1 Footnote 3)

$$F_b' = 1200\ (0.90)\ (0.85)\ (1.00)\ (1.00) \times (1.30)\ (1.00)\ (1.15)\ (1.00) = 1372 \text{ psi}$$

In addition to the adjustment factors given in Table 2.3.1 of the Specification, other adjustments of tabulated design values for special conditions or requirements of use may be required. Such additional adjustments may include modifications for creep effects, variability in modulus of elasticity, and fire retardant treatment. Guidelines for accounting for some of these special conditions are given in the Specification.

2.3.2-Load Duration Factor, C_D

Historical Development

The characteristic behavior of wood structural members to carry a greater maximum load for short durations than for long load durations was reported as early as 1841 (177). Reductions in strength of 30 and 40 percent after a year or more under load were cited in early investigations. By the beginning of the century it was generally believed that long-term loads should not exceed about 75 percent of the proportional limit or elastic limit of the member as determined from short term tests (176). However, because proportional limit as well as ultimate strength was found to be affected by speed of testing (175), and because of the subjective element involved in determining proportional limit, ultimate strength came to be considered the most dependable base on which to establish load duration adjustments (137).

Permanent Loading Factor. By the 1930's, a factor of 9/16 the short term strength had become established as the safe working stress level for long-term loads (57). Such loads were considered to be those associated with dead and design live loads. However, an increase of 50 percent over this permanent load level was considered appropriate when designing for short load durations, such as wind loads, in conjunction with dead and live loads. These recommendations were based on the results of load duration tests on small, clear specimens in bending and compression parallel to grain, supplemented by field experience with structural members. As the short term tension strength of clear wood is about 50 percent greater than short term bending strength, use of the same working stress for tension as for bending, including the same load duration adjustments, was considered justified.

Wartime Provisions. During World War II, as part of the wartime effort to conserve scarce materials, the War Production Board authorized a twenty percent increase in design values for extreme fiber in bending, tension and horizontal shear and a ten percent increase in compression properties. A twenty percent increase in allowable design values for long columns also was authorized. These increases were based on a reevaluation of available clear wood strength data (193) and load duration considerations. In conjunction with these increases in permanent load design values, increases of 15 percent for snow, 50 percent for wind and earthquake and 100 percent for impact also were authorized provided member sizes were sufficient to carry each combination of longer term loads when the load duration adjustment applicable to that combination was considered (194). For example, wind plus live plus dead loading could be checked against a design value of 1.5 times the permanent design value provided that the live plus dead load did not exceed the permanent design value. The load duration adjustments authorized in 1943 also were applied to fastenings.

Normal Loading Established. The permanent load design values authorized by the War Production Board, including the adjustments for shorter load durations, were published in the first edition of the National Design Specification for Stress-Grade Lumber and Its Fastenings in 1944. Following World War II, one-half (10 percent) of the wartime increase for bending-tension and shear design values was retained as it was a result of reevaluation of available strength data (60,193). The remaining one-half of the war-time increase for these properties and the ten percent increase for compression parallel to grain was removed by reducing the permanent load design values for these properties by 10 percent (National Design Specification 1948). However, a new loading condition, designated "normal loading", was established as the load duration basis for wood design values. This new loading condition, applicable to live loads other than snow, wind, earthquake and impact, and equal to 1/0.9 of permanent load design values, was defined as that in which a member is fully stressed to its maximum allowable design value cumulatively or continuously for a period of ten years or less during the life of the structure in which the member is used (National Design Specification 1951). Additionally, application of 90 percent of the full maximum "normal" design load continuously during the life of the structure is a normal condition. The new normal loading base was applied to all design values for wood members except modulus of elasticity, and to design values for fastenings when the capacity of the connection was limited by the strength of the wood. One-half of the twenty percent war-time increase in allowable design values for long columns also was retained by applying the normal load concept to this design value as well.

Short-term Load Adjustments. In conjunction with the introduction of the ten year or normal load base for tabulated design values in 1948, increases of normal design values of 1.15 for two months duration as for snow, 1.25 for 7 day duration as for construction, 1.33 for one day duration as for wind or earthquake and 2.00 for impact were established. The appropriateness and consistency of the new load duration adjustments for normal and other conditions of loading were substantiated by U.S. Forest Products Laboratory analysis (224). The relationship between strength and load duration established by this analysis is shown in Appendix B. The equation for the curve shown in Figure B1 is

$$C_D = \frac{1.75192}{(DOL)^{0.04635}} + 0.29575 \qquad \text{(C2.3-1)}$$

where:

C_D = load duration factor

DOL = duration of load, seconds

New Design Loads. The adjustments for the effect of load duration on strength established after World War II were used in wood structural design without change for the next 38 years. However, the adoption of new design snow and wind loads and related new analytical procedures by ASCE 7-88 (formerly ANSI A58.1) *Minimum Design Loads for Buildings and Other Structures* (10), impacted the design of many light frame wood structures which had a long record of satisfactory performance. Analysis of the data bases for the new loads and the probability methods used to establish them indicated the long standing assumptions that all snow loads could be categorized as 2 month loads and that peak wind design loads had a cumulative duration of 1 day were no longer appropriate.

New Provisions for Snow Loads. Snow load records showed that the duration of the ASCE 7-88 snow loads varied significantly from location to location and, in some high snow load areas, was much shorter than the traditional two months assumed. Beginning with the 1986 edition of the Specification, use of a larger load duration adjustment than 1.15 for new snow design loads is allowed when information is available on the actual duration of the load in the specific location being considered. Such adjustments are to be selected from the standard relationship characterizing the effect of load duration on strength in Appendix B (see Commentary Equation C2.3-1).

New Wind Load Adjustment. In 1987, an evaluation of the degree of conservatism in the traditional 1.33 or one day load duration adjustment for wind was initiated in response to proposed new design requirements for structures in high wind coastal areas. The proposed requirements, which called for comprehensive engineering design for ASCE 7-88 wind loads, would have substantially increased the sizes and/or numbers of structural members and connections in low rise wood frame buildings which had a long history of satisfactory performance in coastal areas when constructed in accordance with recognized good building practices. The evaluation took into account that the design wind loads given in ASCE 7-88 are based on a mean recurrence interval of once in 50 years, with the specified wind loads having a duration of from one to ten seconds, rather than the 1 day previously assumed for wood design. The standard relationship between strength and load duration (Appendix B) showed that the load duration factor associated with such short load periods was in excess of 1.85. As a result of this evaluation, a new wind load duration adjustment factor of 1.85 was established in a November 1987 revision of the Specification for engineering design of wood members based on ASCE 7-88 wind loads and related procedures.

Wind and Earthquake Adjustments in the 1991 Edition. Subsequent to the 1987 revision, studies of the duration of code specified maximum earthquake and live floor design loads also have shown that the durations of these loads are much shorter than the 1 day and 10 years periods, respectively, that traditionally have been assumed (39,121). These new findings were taken into account in conjunction with other new information and considerations in the development of load duration factors in the 1991 edition of the Specification. Other influential factors included the availability of new design values for visually graded structural lumber based on major test programs of full size, in-grade lumber in the United States and Canada; the advances made in quantification of the system contributions of framing and sheathing materials in the performance of light frame wood assemblies; and simplicity of load duration adjustments for wood strength properties.

Although new information showing the time floor live loads are at or above the code specified design level is less than 50 days (121), the conservative position of retaining a 10-year normal loading basis for such loads is continued in the 1991 edition. Further, until additional information is developed on the duration of the ASCE 7-88 snow loads, the recommendations for load duration adjustments of snow loads in the 1986 edition also are continued in the current edition.

For purposes of simplification and conservatism, load duration adjustments for ASCE 7-88 earthquake loads (longest on record less than 5 minutes, with most less than 1 minute duration of ground motion) (39) and wind loads are consolidated at 1.6. This adjustment corresponds to the general time of tests to failure used to establish the strength properties of structural wood products and is the factor used to reduce short time test results to a normal loading basis. Use of the 1.6 factor in conjunction with improved system design procedures and analytical techniques for light frame wood assemblies may be used to rationalize the performance of these assemblies under the highest wind loads. Application of the various load duration factors of the current edition are further discussed in Appendix B of the Specification.

Results of In-grade Lumber Tests. Load duration tests conducted on in-grade lumber in bending, tension and compression and reported in 1986 and 1988 indicate the standard load duration curve is a generally adequate representation of the effect of load duration on strength. In-grade bending tests on a number of species show most to be slightly less influenced by load duration than indicated by the standard curve while one species was found to be somewhat more affected (74,75,96). Similar species effects were observed earlier in load duration bending tests on clear wood specimens (224). Results from load duration tests of in-grade lumber in tension and compression on one species show a greater effect of time under load for these two properties than for in-grade lumber bending strength for the same species (96). However, the load duration effects for the direct stress properties were found to be comparable to that indicated by the standard strength-load duration curve.

The long record of satisfactory performance with wood structures designed using load duration adjustment factors, and the results of load duration tests on full-size members, substantiate the general applicability of the standard strength-load duration relationship (Appendix B). As more information is developed on the frequency of occurrence and the duration of maximum design loads specified in building regulations, use of different adjustment factors than those currently specified may be appropriate for individual load cases. Such changes would be associated with improved definition and recognition of the duration of the loads involved rather than a revision of the underlying relation between strength and time under load.

Damage Accumulation. Adjustments of wood design values for load duration are keyed to the cumulative period of time the design load is expected to be applied to the member over its service life. Such maximum loads are considered to represent the most extreme condition expected over a long period; for example, once in a 50 year interval (10). The peak loading on the member from year to year is much lower than this design load. For this reason, coupled with the exponential relationship that exists between the strength of wood and load duration, the accumulated effects of loadings less than the maximum load the member is designed for are considered negligible and noncritical. The satisfactory performance of all types of wood structures designed on such a basis for more than 40 years attests to the appropriateness of the methodology. Recent studies (52,121) of the damage accumulation in wood structural members using new information on the intensity, frequency and duration of applied loads and one of the new models characterizing the relationship between load level and time to failure (76) further substantiate the procedure. It is reported that "practically all damage occurs when the live load intensity is equal or nearly equal to the" code specified design load (121).

2.3.2.1 Load duration factors (C_D) are applicable to all tabulated design values for lumber and glued laminated timber except modulus of elasticity and compression perpendicular to grain. The exclusion of modulus of elasticity from load duration adjustment has been a provision of the Specification since the first edition. Load duration factors are based on the effect of time under load on ultimate load-carrying capacity. Increases in deflection or deformation are a separate consideration, independent of ultimate strength. However, prior to 1977, the Specification provided for load duration adjustments to allowable compression parallel to grain design values for all columns, including those with ℓ/d ratios greater than 11 (intermediate and long columns). Because allowable design values for the latter columns were a function of modulus of elasticity, some users began to interpret this application of load duration adjustments as indicating the factors should be applied to modulus of elasticity values. Although the use of load duration adjustments for intermediate and long column allowable design values had provided for satisfactory performance over many years, this provision was removed from the Specification in the 1977 edition to obtain technical consistency and avoid misinterpretations.

Compression perpendicular to grain design values were subject to adjustment for load duration when such values were based on proportional limit test values. When the basis for such design values was changed to a deformation limit in the 1982 edition, the application of the load duration factor to compression perpendicular design values was eliminated.

Table 2.3.2 Frequently Used Load Duration Factors

Ten Year or Normal Loading. Loads traditionally characterized as normal are code specified floor loads, either uniform live or concentrated, which include furniture, furnishings, movable appliances and equipment, all types of storage loads and all people loads. Although maximum human traffic loads may be infrequent and of short duration, such as those occurring on balconies, exterior walkways and stairways, this type of loading is considered normal loading under national standards of practice.

Permanent Loads. In addition to materials of construction dead loads, foundation soil loads and concentrated loads from equipment designed as part of the structure should be considered long-term loads that will be applied continuously or cumulatively for more than ten years. Special continuous loadings related to the particular purpose or use of the structure, such as water loads in cooling towers or heavy machinery in industrial buildings, also may be associated with durations exceeding ten years.

Two Month Loads. A 2 month load duration adjustment factor of 1.15 was used for all code specified snow loads prior to 1986. New maximum snow loads published in ASCE 7-88 (10) based on probability of occurrence are significantly greater in some high snow regions than the loads previously used in those areas. Evaluation of annual snow load records available for some of these areas shows that the duration of the maximum snow load specified in ASCE 7-88 is much shorter than the two months duration previously assumed for all snow loads. The Specification provides for use of a larger snow load adjustment than 1.15 when information is available on the duration of the design snow load for a specific area.

Seven Day Loads. Where the minimum roof uniform load specified by the applicable building code exceeds the design snow load for the area and the specific building design, it is conventional practice to consider this load a construction type load for which a 7 day or 1.25 load duration factor is applicable. If the roof snow load is less than 92 percent of the minimum roof load specified, the latter will be the limiting of the two load conditions.

One Day Loads. Prior to 1987, a 1 day or 1.33 factor was used as the load duration adjustment for wind and earthquake loads. In the 1991 Specification, the load duration factor for these loads is based on a 10 minute load duration.

Ten Minutes. The 10-minute or 1.6 load duration factor is to be used with wind and earthquakes in the current Specification. The wind loads and procedures given in ASCE 7-88 are maximum loads expected to occur less than once in 50 years and to have durations of from one to 10 seconds. Peak earthquake loads are known to have cumulative durations less than 5 minutes rather than the 1 day duration traditionally assigned. The 10-minute load duration factor is a conservative adjustment for these two load conditions.

Impact Loads. Loads in this category are considered to be those in which the load duration is one second or less. Such a duration is associated with an adjustment factor of 2.0 based on the general relationship between strength and load duration (Appendix B).

Pressure treatment of wood with preservative oxides or fire retardant chemicals to retentions of 2.0 pcf or more may reduce energy absorbing capacity as measured by work-to-maximum-load in bending as much as one-third in some cases. Application of the 2.0 load duration adjustment factor for impact was discontinued for oxide treated piles and other members intended for salt water exposure in 1977, and for wood pressure treated with fire retardant chemicals in 1982.

2.3.2.2 Design of structural members in terms of size and resistance is based on the critical combination of loads representing different durations. For fully laterally supported members, this critical combination may be determined by dividing the summation of design loads by the shortest load duration factor applicable to any load included in the combination. The combination with the highest quotient is the critical total load on which the member design is to be based. Example C2.3-2 demonstrates this provision.

Example C2.3-2

Consider a fully laterally supported bending member subject to a dead load of 20 plf, a roof construction live load of 60 plf and a wind load of 40 plf. The normalized load combinations for the member are:

dead load:	(20)/0.9 = 22
dead plus roof live (7-day):	(20 + 60)/1.25 = 64
dead plus roof live plus wind:	(20 + 60 + 40)/1.6 = 75

The critical load combination is the dead plus roof live plus wind case. The actual bending stress f_b should be calculated with the full 120 plf loading with no load duration adjustment, then checked against an allowable F_b' having a C_D of 1.60.

Note that load duration adjustments are not applicable to modulus of elasticity (see Commentary 2.3.2.1), hence, a member subject to buckling should be analyzed for the critical load combination after the critical buckling design value has been calculated. This is illustrated in Example C2.3-3.

2.3.2.3 Load combination adjustment factors provided in ASCE 7-88 and in building codes account for the reduced probability that two or more loads, other than dead loads, acting concurrently will each attain its maximum at the same time. Such adjustment factors for load combinations are applicable to all materials. The effect of load duration on the individual strength properties of wood is unique, relative to other major construction materials, in terms of its magnitude and application to both ultimate strength and proportional limit stresses. Adjustment of wood design values for load duration is an adjustment to the working stress irrespective of whether one or more

Example C2.3-3

Consider a column subject to a 350 lb dead load, a 1300 lb contents live load and a 400 lb roof live load. Assume an 8 ft column height, 2x4 spruce-pine-fir stud grade:

C_D	F_c^*	C_P	F_c'	$F_c'A$	
0.90	638	0.585	373	1959 > 350	**ok**
1.00	709	0.545	386	2027 > 1650	**ok**
1.25	886	0.461	409	2145 > 2050	**ok**

The critical load combination is the dead plus live plus roof live case.

external loads are applied. Reductions allowed in design loads on the basis of load combination probabilities are to be applied concurrently with adjustment of design values for load duration. The former affects the magnitude of the actual load while the latter establishes the maximum resistance available to carry that load. Example C2.3-4 demonstrates this provision.

Example C2.3-4

Consider a column subject to a 400 lb dead load, a 1000 lb contents live load, a 600 lb snow load, and a 400 lb wind (additive) load. Assume an 8 ft column height, 2x4 spruce-pine-fir stud grade:

C_D	F_c^*	C_P	F_c'	$F_c'A$	
0.90	638	0.585	373	1959 > 400	**ok**
1.00	709	0.545	386	2027 > 1400	**ok**
1.15	815	0.492	401	2105 > 2000	**ok**
1.60	1134	0.377	427	2241 > 1800	**ok**

Note that the ASCE 7-88 load combination factor of 0.75 applies to the dead plus live plus snow plus wind case: (400 + 1000 + 600 + 400)(0.75) = 1800. The critical load combination is, therefore, the dead plus live plus snow case.

2.3.3-Wet Service Factor, C_M

The different moisture service conditions for which design values or design value adjustments are provided in the Specification are:

Sawn lumber:

dry, 19 percent or less (Tables 4A - 4E)
wet, over 19 percent (factors, Tables 4A - 4E)

Glued laminated timber:

dry, less than 16 percent (Tables 5A - 5C)
wet, 16 percent or greater (factors, Tables 5A - 5C)

Round timber piles:

wet or dry (Table 6A)

Fastenings:

dry, 19 percent or less (tabulated by type)
wet, 30 percent or more (Table 7.3.3)
partially seasoned, greater than 19 percent and less than 30 percent (Table 7.3.3)

A moisture content of 19 percent has long been recognized as an appropriate upper limit for a dry condition of service for lumber used in wood structures. This maximum level coincides with the moisture content requirement for dry dimension lumber given in the American Softwood Lumber Standard PS20-70.

Uses involving maximum moisture contents of 19 percent are traditionally considered to average 15 percent or less. Those involving maximum moisture contents of 15 percent are considered to average 12 percent or less. Glued laminated timber and wood panel products have moisture content less than 16 percent at manufacture and therefore design values for these products are appropriate for maximum service conditions associated with such a moisture content.

Moisture contents of 19 percent or less are generally obtained in covered structures or in members protected from the weather, including wind blown moisture. Roof, wall and floor framing, and attached sheathing are considered to be such dry applications except where special conditions exist (62,66). These dry conditions of service are generally associated with an average relative humidity of 80 percent or less. Framing and sheathing in properly ventilated roof systems, which are periodically exposed to relative humidities over 80 percent for short periods, meet moisture content criteria for dry conditions of use. Floor framing over properly ventilated crawl space which include a vapor retarder to cover exposed soil meet moisture content criteria for dry conditions of use (238).

The wet service factor, C_M, in the 1991 edition is an adjustment to tabulated design values for wet use conditions. The designer need only determine the expected moisture content of the product in use to establish the applicable value of C_M.

2.3.4-Temperature Factor, C_t

Prior to 1977, the effects of temperature on design values for lumber and timber were not addressed in the Specification. Decades of satisfactory experience with wood buildings and bridges have shown that adjustment of design values for ordinary temperature fluctuations encountered by these structures in service is not required. The use of wood construction in non-building industrial applications such as cooling towers where members are subjected to heavy loads and hot water exposures, as well as special cold weather building applications and use of wood for material handling of cryogenic materials, motivated the inclusion of information on temperature effects in the 1977 and subsequent editions of the Specification.

Reversible Effects. The increase in the strength properties of wood when cooled below normal temperatures and the decrease in these properties when it is heated up to 150°F are immediate and generally reversible. When the temperature of the wood returns to normal temperature levels, it recovers its original properties. In general, these reversible effects are linear with temperature for a given moisture content (72). The magnitude of the increase or decrease, however, varies with moisture content. The higher the moisture level, the larger the increase with decreasing temperature and the larger the decrease with increasing temperature.

Permanent Effects Over 150°F. Prolonged exposure to temperatures over 150°F can cause a permanent loss in strength when cooled and tested at normal temperatures. The permanent effect is in addition to the immediate or reversible effect that occurs at the exposure temperature. Permanent losses in strength resulting from exposures over 212°F are greater for heating in steam than in water (66). For temperatures over 150°F, permanent decreases in strength are greater for heating in water than in dry air.

The use of 150°F as a nominal threshold for the beginning of permanent strength loss is substantiated by available test data showing an approximate 10 percent loss in bending strength (modulus of rupture) for material exposed for 300 days in water at 150°F and then tested at room temperature (66). Exposure in air at the same temperature would result in a smaller permanent strength loss.

Temperature Adjustments. Adjustments for reversible temperature effects first given in the 1977 edition of the Specification applied to 0 percent and 12 percent in-service moisture conditions. Separate heating and cooling adjustment coefficients were given for each condition of use for modulus of elasticity and for strength properties. These adjustments, based on summary information published by the Forest Products Laboratory (65), were continued in the 1982 edition.

In the 1986 edition, the temperature adjustments were revised to reflect the results of a comprehensive analysis of available world literature by the Forest Products Laboratory (72), and expanded to include wet conditions of service (24 percent moisture condition).

1991 Factors. In the 1991 Specification, strength loss coefficients for each 1°F increase in temperature above 68°F at 0, 12 and 24 percent moisture content have been replaced with simplified temperature factors, C_t, for dry and wet service conditions and two levels of elevated temperature: 101°-125° and 126°-150°. As in previous editions, one set of factors is given for modulus of elasticity and tension parallel to the grain and a separate set for other design values. The use of temperature factors for dry (average 12 percent) and wet (greater than 19 percent) conditions of service follows the same basis used for other design value adjustments related to moisture.

Mandatory Reductions. The temperature adjustments in the 1991 Specification are mandatory when structural members are exposed to temperatures between 100°F and 150°F for extended periods of time, such as in industrial applications in which structural members are in close proximity to or in contact with heated fluids used in manufacturing processes. In general, adjustment of design values in the Specification for temperature should be considered for applications involving sustained heavy dead or equipment loads, or water immersion, or wet or high moisture content service conditions, when sustained or frequent extended exposure to elevated temperatures up to 150°F will occur.

Use of lumber or glued-laminated timber members in applications involving prolonged exposure to temperatures over 150°F should be avoided. Where such exposures do occur, adjustments for both immediate and permanent strength reductions should be made. Permanent effects should be based on the cumulative time the members will be exposed to temperature levels over 150°F during the life of the structure and the strength losses associated with these levels (66). As discussed below, roof systems and other assemblies subject to diurnal temperature fluctuations from solar radiation are not applications that normally require adjustment of tabulated design values for temperature.

Cold Temperatures. Adjustments for increasing tabulated design values for cooling below normal temperatures, which were included in Appendix C of the Specification in the three previous editions, have been eliminated in the 1991 edition. The appropriateness of such increases is difficult to establish in building design because of the variable nature of low temperature environments. Structural members that might be exposed to below freezing temperatures continuously for up to several months also are exposed to normal temperatures during periods of the year when the full design load must be resisted. Increases in design values are not applicable for this common occurring situation. To avoid misapplication of increases for below normal temperatures, and in recognition of the relatively few design situations where such increases are appropriate, temperature factors for cooling have been dropped from the Specification. For special applications such as arctic construction or transportation of cryogenic materials where the design load is always associated with low temperature environments, data from other sources may be used to make appropriate adjustments of design values (66,72).

Elevated Temperatures Encountered in Normal Service. Temperatures higher than ambient can be reached in roof systems as a result of solar radiation. The temperatures reached in such systems are a function of many variables, including hour of day, season of year, cloud cover, wind speed, color of roofing, orientation, ventilation rate, presence of insulation and thickness of sheathing. Measurements of roof system temperatures in actual buildings (89) show that structural framing members in such roofs seldom if ever reach a temperature of 150°F, and when such levels are reached the duration is very short and is confined to the face of the member on which the sheathing is attached. Even in the severest of radiation and design conditions, the temperature of structural beams, rafters and truss members in wood roofs generally do not reach 140°F. Normal temperature environments return as the sun recedes.

When wood structural members are subjected to temperatures above normal levels, the moisture content of the members decreases. At the highest temperatures reached, moisture contents in properly ventilated and maintained roof systems will approach moisture contents of 6 percent. Associated with such a decrease in moisture content is an increase in strength property which offsets the decrease resulting from temperature. For example, based on available data (72), the bending strength of a member at 140°F and 6 percent moisture is reduced 20 percent relative to strength at 70°F and 6 percent moisture content. However, the increase in strength resulting from a change in an average service moisture content of 12 percent to 6 percent is 30 percent (72), which more than offsets the immediate temperature effect. Further, structural framing members are subjected to elevated temperature exposures in the warm months of the year when uniform roof snow loads are not present.

The foregoing considerations and successful field experience are the basis for the long standing practice of applying the design values tabulated in the Specification without adjustment for temperature to structural wood roof members in systems designed to meet building code ventilation requirements. Tabulated design values also are appropriate for use with wood members directly exposed to solar radiation but otherwise surrounded by ambient air, such as members used in bridges, exterior balconies and stairways, and exterior vertical and horizontal structural framing.

2.3.5-Pressure-Preservative Treatment

Structural wood members in conditions where the moisture content will exceed 19 percent in service, or members which will be in proximity to damp wood, soil or other sources of moisture, may be susceptible to decay (88). Where such service conditions exist, it is the responsibility of the designer to determine if pressure-preservative treated wood should be used.

Application of tabulated design values for lumber and glued laminated timber pressure treated with standard preservative formulations and processes (25) has been a provision of the Specification since the first edition. Early practice with water-borne preservatives was to treat material that had been air-dried or kiln dried at temperatures of 180°F or lower. Treated material was generally air redried or redried in service. Where kiln redrying was employed, generally temperatures less than 190°F were used.

In the past several decades, kiln redrying of water-borne preservative treated wood has been increasing, particularly with the development and use of wood foundations. During the same period, use of high temperature kilns to initially dry dimension lumber also increased. As a result of studies showing that kiln redry temperatures of 170°F and higher can reduce the strength of small, clear preservative treated wood specimens (210,212,215), a redry temperature limit of 190°F for waterborne preservative treatments was added to AWPA treatment standards for sawn lumber and timber in 1988. Subsequent research showed that the effect of redrying on the strength of treated lumber was related to the grade and quality of the material, with the higher strength grades and higher strength pieces within a grade being more significantly affected than the lower strength grades and lower strength pieces within a grade (211,214).

More recently it has been shown that the combination of preservative treatment of lumber that was initially dried in high-temperature kilns followed by redrying at 190°F can reduce bending and tensile strength properties up to 10 percent for some species and higher grades of material (213). As a result of these new findings, the maximum temperature for kiln drying material after treatment in AWPA Standards C2, Lumber, Timbers and Ties - Preservative Treatment by Pressure Processes, and C22, Lumber and Plywood for Permanent Wood Foundations - Preservative Treatment by Pressure Processes, recently was reduced from 190°F to 165°F (26). The provision in the Specification for use of tabulated design values with lumber that has been preservatively treated is applicable to material that has been treated and redried in accordance with AWPA 1991 Standards C2, C22, C28, Structural Glued Laminated Members and Laminations Before Gluing, Pressure Treatment or C31, Lumber Used Out of Contact with the Ground and Continuously Protected from Liquid Water. Tabulated design values for borate treatments in accordance with C31 apply to retention levels of 0.3 lbs/ft^3 and less. Borate containing treatments at retention levels of 3.0 lbs/ft^3 and higher have shown embrittlement (237).

New preservative treatments for lumber are being developed and should be tested by the manufacturer of the treatment or the company providing the treating and redrying service to evaluate the effects on lumber strength. The designer should consider potential effects on strength of lumber from new preservative treatments and should evaluate the basis and adequacy of the manufacturer's recommendations for those products not yet having a history of satisfactory performance.

Recent research has indicated that modulus of elasticity and bending design values in incised, pressure-treated lumber may be reduced compared to untreated, unincised specimens (236). Mean modulus of elasticity was reduced as much as 6 percent, mean and fifth percentile modulus of rupture were reduced as much as 21 percent and 25 percent, respectively, compared to the control group.

2.3.6-Fire Retardant Treatment

Fire retardant treated wood has been in use since before 1900 for applications ranging from ships, blimp hangers, scaffolding, doors and trim in high rise buildings, interior panelling, interior partitions, roof construction and balconies and stairways (51). Fire retardant treatments are proprietary and chemical formulations vary between manufacturers. All pressure treatments which are accepted by building codes, however, meet minimum flame spread resistance requirements (38, 91, 169). Except for certain special formulations, fire retardant treatments are intended for use in dry conditions of service.

Introduction of 10 Percent Treatment Factor for Lumber. In 1962, following the first building code acceptance of the use of fire retardant treated wood for roof construction in certain non-combustible types of buildings, the Specification recognized the structural use of such material through introduction of a 10 percent reduction in tabulated design values for lumber pressure treated with fire retardant chemicals. Available test data indicated that such a reduction, which was applied to fastener design values as well as strength properties, could be generally used with all treatments (51,81,95). The reduced strength of the fire retardant treated wood was associated with the elevated temperatures used in kiln drying after treatment. Strength properties of material air-dried after treatment were reported to be little affected (71). The 10 percent reduction factor applied to lumber design values for fire retardant treatment was continued in the Specification for the next 20 years. The satisfactory performance of components and systems made with fire retardant treated lumber over this period attested to the general adequacy of Specification provisions for the type of treatments that were commercially available.

Glued Laminated Timber. Prior to 1977, the Specification did not contain provisions specifically addressing design values for fire retardant treated glued laminated timber. Because certain species of lumber could be adequately glued after fire retardant treatment, glued laminated timber made with such treated lumber could qualify as a fire retardant treated product. The 10 percent design value adjustment for fire retardant treated lumber was assumed applicable to the properties of the treated laminations.

In 1977, the practice of using the 10 percent reduction factor for fire retardant treated lumber with glued laminated timber was discontinued. This change was based on recognition of the effect of a number of variables: the differences between pressure treatment with fire-retardant chemicals before and after gluing; and differences in the effects of individual treatments on different species and different properties. Particularly noted was information that fire retardant treatment could reduce the bending strength of glued laminated timber more than 10 percent in some cases due to the effect of the treatment on the strength of end joints in the laminations. Beginning in the 1977 edition, the Specification has referred users to the manufacturer of the treatment or to the company providing the treating and redrying service for design value recommendations for glued laminated timber pressure treated with fire retardant chemicals.

Applicable Load Duration Factors. The same load duration adjustments applicable to untreated wood have traditionally been applied to fire retardant treated wood (129). However, since 1982, the Specification has disallowed the 2.0 increase for impact loading (load duration less than one second) for fire retardant treated lumber. This change was made in recognition of the reduced energy absorbing capacity or toughness of fire retardant treated wood relative to that of untreated material. The property work-to-maximum load obtained from a bending test, which provides a measure of energy absorbing capacity, is more sensitive to treating chemicals and redrying than any of the other properties. Although impact resistance is not generally a design consideration, the reduced energy absorbing capacity of fire retardant treated wood should be recognized where it may be important in material handling, in resisting construction loads, or in resisting special service loads on completed structures. In this regard, cases have been reported of breakage of trusses made with fire retardant treated lumber when being off-loaded in bundles at the construction site and of similar occurrences with other treated products during transportation.

The use of the 1.6 (ten minute) and other load duration factors with fire retardant treated material is supported by the fact that the effects of fire retardant treatments on strength properties are evaluated using standard static tests to failure of treated and untreated wood that involve average times to failure of 5 to 10 minutes.

New Treatment Formulations. In the early 1980's, new fire retardant chemical formulations began to be introduced which had lower affinities for moisture thus reducing the corrosion potential on metal parts of the treated material when exposed to higher relative humidities. These exposure conditions were being encountered in applications considered a dry condition of service but which involved periodic exposure to humidities over 80 percent, such as those occurring in ventilated roof systems in the southeast. The lower hygroscopicity treatments have become the predominate fire retardant treatments available.

In the absence of extensive strength test data for and experience with the new fire retardant treatments, the generic 10 percent reduction in lumber design values was dropped from the Specification by amendment in 1984. Users of fire retardant treated lumber were referred to the company providing the treating and redrying service for appropriate design value adjustments. However, in response to standardization needs expressed by code agencies, users and manufacturers, new provisions for evaluating and qualifying fire retardant treated lumber for specific design value adjustments were introduced in the 1986 edition of the Specification.

Adjustments in the 1986 Edition. The 1986 edition provided that each treatment be qualified for certain tabulated design value adjustments by meeting specific testing, inspection and marking criteria (Appendix Q of the Specification). The prescribed test procedures utilized matched samples of treated and untreated small, clear, straight-grained material to determine the effects of treatment on strength properties. Strength test information from the same type of clear material has long been used in the establishment of design values for visually graded lumber and for development of load duration and other adjustment factors for condition of use.

Strength data for several of the treatment formulations developed in accordance with the specified testing procedures were considered in establishing design value adjustments for fire retardant treated lumber tabulated in the 1986 Specification. The factors were based on tests for the properties of extreme fiber in bending, modulus of elasticity, compression parallel to grain, tension parallel to grain and horizontal shear. Adjustment classes were established in 0.05 increment classes with the highest class being 0.90 to maintain consistency with past practice where possible. Adjustments in the 1986 edition were 0.85 and 0.80 for bending (F_b) and tension (F_t) design values respectively, with all other properties assigned a factor of 0.90. It was recognized that treatments qualifying for these adjustments might have higher treatment ratios than those tabulated for one or more properties. Further, the adjustment factor for compression perpendicular to grain was conservatively assumed equal to that for compression parallel to grain; and, as fastener loads are related to compression properties, and indirectly to shear strength because of end distance considerations, a design load adjustment factor of 0.90 for connectors was used.

Quality Marking Provisions in the 1986 Edition. The effect of fire retardant treatments on wood strength properties is affected by a number of process variables, including amount and penetration of chemicals retained and the time and temperature used to redry the material after treatment. Under AWPA Standard C20 (26), and in accordance with building code requirements, fire retardant treated lumber is required to be dried to a moisture content of 19 percent or less after treatment. The conditions used to achieve this level of drying have a significant influence on strength properties. The 1986 Specification therefore required quality marking by an approved inspection agency to validate that production material had been treated and redried in accordance with the process procedures established by the chemical manufacturer and which were used to qualify the treatment for the design value adjustments tabulated in the Specification. This required quality marking was separate from that required by building codes for flamespread classification.

Adjustment Factors Based on Ambient Temperature Conditions. The adjustment factors for fire retardant treated lumber given in the 1986 and earlier editions of the Specification were based on strength tests of material tested at normal room temperature conditions. Although treated lumber and plywood made with the older traditional formulations have given satisfactory performance in roof system applications, significant problems have been reported with plywood roof sheathing made with certain of the new fire retardant treatment formulations, particularly in town house type construction (105). A few cases of problems involving treated lumber roof framing also have been reported. Testing has now shown that fire retardant treated wood generally is more affected by prolonged exposure in service to elevated temperatures of 150°F and higher than untreated wood. Such conditions are encountered in wood roof systems where the effects of solar radiation on the roof are not offset by natural ventilation of ambient air.

The cumulative periods of time fire retardant treated wood is subject to temperatures above ambient in the life of the structure must therefore be taken into account when establishing design value adjustments for treated material used in roof applications. Although roof sheathing is subject to higher thermal loads than supporting framing, the effects of elevated temperature exposure over time must be considered for the latter as well as the former.

The major fire retardant treatment manufacturers have established design value adjustment recommendations for their present commercial formulations for both roof system and ambient temperature applications involving both treated plywood and treated lumber. The testing methodology employed for lumber evaluation is reported to be similar to that given in the 1986 Specification but includes testing after extended exposure at near maximum roof framing temperatures. These recommendations account for the effects of expected cumulative exposure to elevated temperatures over the service life of the structure.

Publication of Design Value Adjustments for Fire Retardant Treated Lumber Discontinued in the Specification. In 1990, the National Forest Products Association made the determination that design value adjustment factors for fire retardant treated lumber should no longer be published in the National Design Specification. This action was reflected in a revision to the 1986 edition, promulgated May 1990, that required the effects of fire retardant chemical treatments on strength to be considered in design, and required design values, including fastener design loads, for fire retardant treated lumber as well as glued laminated timber to be obtained from the company providing the treating and redrying service. This change in the provisions for fire retardant treated lumber was made on the basis that the design value adjustments and test procedures given in the Specification for this material did not address the effects of elevated temperature exposure, that standard methodology necessary to evaluate elevated temperature effects through accelerated testing and to convert results from such tests into long-term durability assessments was not available, and that the development and promulgation of the required methodology was outside the scope of Association functions.

The 1990 revision is carried forward in Section 2.3.6 of the 1991 Specification. It is expected that the lumber design values to be obtained from the treating companies will reflect use of testing methodologies similar to those given in Appendix Q of the 1986 edition, or equivalent; will apply to material treated in accordance with AWPA Standard C20; and will apply to material that bears agency quality monitoring marks similar to those called for in the 1986 Specification. It should be noted that use of individual company design value recommendations for fire retardant treated lumber and glued laminated timber is subject to the approval of the applicable building code jurisdiction.

2.3.7-Beam Stability Factor, C_L

The beam stability factor accounts for the tendency of laterally unsupported deep beams to rotate or buckle under bending or combined axial and bending loads. The adjustment of tabulated bending design values for beam stability or slenderness given in 3.3.3 was first incorporated into the Specification in 1968 for glued laminated timber. It was subsequently extended to cover sawn lumber beams in 1977.

Prior to the 1991 edition, the beam stability factor was not applied simultaneously with size adjustment for beams that were larger than 12 inches in depth. In the current edition, the stability factor, C_L, and the size factor, C_F, are applied simultaneously to all sawn lumber bending design values, including those for beams greater than 12 inches in depth. However, no stability or slenderness adjustment of tabulated design values for sawn lumber beams is necessary if such beams are laterally supported in accordance with the approximate rules given in section 4.4.1. These approximate rules for providing restraint against rotation and lateral displacement have been in the Specification since the 1944 edition.

The beam stability factor, C_L, is not applied simultaneously with the volume factor, C_V, for glued laminated timber bending members in the 1991 Specification. This continuation of past accepted practice considers tabulated design values for deep glued laminated timber beams to be controlled by tension zone failures whereas beam buckling characteristics are related to compression zone properties.

2.3.8-Form Factor, C_f

Adjustment of tabulated bending design values for form are based on early research at the Forest Products Laboratory (136). Results of tests of round, diamond, I and Box beam sections showed that the strengths of these members differed from that expected by their section properties and the strength of matching rectangular beams.

In the case of round beams, it was found that the average modulus of rupture of these beams as calculated by standard beam formulas was 1.15 times the modulus of rupture of matched rectangular beams. However, the section modulus ($S=I/c$) of a round beam is about 1/1.18 times smaller than that for a rectangular beam of equivalent area. Thus a beam of circular section has about the same load-carrying capacity as a rectangular beam of equal cross-sectional area; or alternatively, has a C_f of 1.18.

In the case of square sections loaded with diagonal vertical (diamond section), tests showed that the loads carried by these sections were slightly larger than those for similar sections tested with sides vertical. Although the moment of inertia, I, of a square about its diagonal is the same as that of a square about a neutral axis perpendicular to its sides, the distance from the neutral axis to the extreme fiber is $\sqrt{2}$ or 1.414 greater. Thus the diamond section, which would be expected to have a 1/1.414 lower strength than the square section by the ordinary beam formula, is assigned a C_f of 1.414.

In contrast to the circular and diamond sections, tests of I and Box sections showed that these beams had lower strengths than those expected from conventional beam equations. The behavior of the various beam sections was explained in terms of the difference in the compression parallel to grain strength and the bending strength of clear, straight grain wood and the amount of lower stressed fibers available in a beam to support higher stressed fibers near the compression face. Equations were developed at the Forest Products Laboratory to adjust tabulated bending design values, based on tests of rectangular sections, for use with I and Box beams (57,136). These equations were used in the design of wing struts and similar non-rectangular members used in wood aircraft structures (61). Because the effect of beam depth on bending strength also was considered a function of the relative support available from lower stressed fibers nearer the neutral axis, both size effect and adjustment for form were accounted for in form factor equations (136).

Form Factor Provisions in the Specification. The equivalence of circular and rectangular beams having the same cross-sectional area has been recognized in the Specification since 1944. The 1.18 form factor for circular sections, the 1.414 factor for diamond sections and a form factor equation for I and Box sections were added to the Specification in 1957. The form factor equation, which included a size effect for deep beams, was discontinued in the 1977 edition. This was a result of the introduction of a new size effect equation for sawn lumber beams in the 1973 edition and changes in commercial practice.

Application of the form factor equation for I and Box beams with built-up structural members used in buildings gradually decreased in the years following World War II as commercial lumber grades rather than aircraft grades of lumber were used as flange material. Rather than employ form factors in conjunction with bending design values to establish allowable strength values for these beams, compression parallel to grain design values were assumed to be the limiting criterion; or, more recently, design values were based on tests of full-size beams made with particular grades of flange material.

Form factors for circular and diamond sections have continued unchanged in the specification to the present edition. Because these factors are based on the strength of equivalent rectangular sections which vary with size, they are applied cumulatively with size factor, C_F.

Application of Form Factors. In view of the basis of the values of C_f for circular and diamond sections, use of these factors should be limited to naturally round material, such as piles, or to grades of material in which compression parallel to grain strength values are lower than bending (modulus of rupture) strength values.

Circular wood structural members generally will be tapered. The variable section properties of such members must be taken into account. Deflection and moment equations are available for tapered circular beams under quarter-point loading (196). A procedure for relating the deflection of round tapered beams (log beams) under uniform load to that of equivalent area (mid-span) square beams also has been developed (147).

For guidance in design of I and Box sections using special high grades of lumber, the original research publication on the subject should be consulted (136). For I and Box sections in which the top and bottom edges of the beam are not perpendicular to the vertical axis of the beam, test results from this research show the strength of such beams is equivalent to that of an I or Box section whose height equals the mean height of the original section and whose width and flange areas equal those of the original section.

2.3.9 Column Stability Factor, C_P

The column stability factor is the ℓ/d or ℓ/r based adjustment of tabulated compression parallel to grain design values to account for buckling. This factor is applied simultaneously with the size factor, C_F.

2.3.10 Bearing Area Factor, C_b

Provisions for increasing tabulated compression perpendicular design values for length of bearing have been included in the Specification since the 1944 edition. Tabulated values are based on loading a two-inch wide plate bearing on a two-inch wide by two-inch deep by six-inch long specimen. Early research at the Forest Products Laboratory on proportional limit stresses associated with bolt and washer loads showed that the smaller the width of the plate or bearing area relative to the length of the test specimen, the higher the proportional limit stress (183,217). Early research conducted in Australia and Czechoslovakia confirmed the nature and magnitude of the bearing length effect (217).

The effect of length of bearing is attributed to the resisting bending and tension parallel to grain strengths in the fibers at the edges of the bearing plate (112,217). Because of the localized nature of the edge effect, the contribution provided decreases as the length of the area under compressive load increases. Alternatively, when the bearing plate covers the entire surface of the supporting specimen (full bearing), values lower than those obtained in the standard two-inch plate test are obtained.

Formal bearing length adjustments, indexed to tabulated compression perpendicular to grain design values at a bearing length of six or more inches, were first recommended in 1935 (57). The adjustments published in the 1944 edition of the Specification and continued to the present edition are slightly revised from those first proposed in 1935, being about 10 percent lower for bearing lengths less than four inches. No adjustments are allowed for bearings less than three inches from the end of a member in recognition of the fact the adjustments assume support from two edges and the standard bearing test has a two inch unloaded length on each side of the loaded plate.

Application of Bearing Area Factors. Bearing adjustment factors are useful in special cases such as highly loaded washers, metal supporting straps or hangers on wood beams, highly loaded foundation studs bearing on wood plates and crossing wood members. In the latter case of perpendicular grain bearing on perpendicular grain, see Commentary for 4.2.6 for discussion of deformation occurring in this support condition relative to metal or end grain bearing on side or face grain.

For the case of complete surface or full bearing (bearing length equals supporting member length), such as may occur in a pressing operation, 75 percent of the tabulated compression perpendicular design value may be used.