

# Keeping Pace with Evolving Load Standards: Recalibration of LRFD for Wood

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## Summary

This paper describes an effort in the United States to document assumptions made in the late 1980s and early 1990s in the development of the *Standard for Load and Resistance Factor Design (LRFD) for Engineered Wood Construction*, AF&PA/ASCE 16-95 and to examine how changes in *Minimum Design Loads for Buildings and Other Structures*, ASCE 7 (and elsewhere) have affected the levels of reliability implied by the LRFD provisions. Specifically, the objective of this study was to document the initial assumptions embodied in ASTM D5457-93, *Standard Specification for Computing Reference Resistance of Wood-Based Materials and Structural Connections for LRFD* and AF&PA/ASCE 16-95 as they relate to the load factors of ASCE 7-88, and to provide a complete “paper trail” of changes to ASCE 7 through the 1993, 1995, and 1998 editions to determine whether modifications are required in the underlying wood LRFD documents. In addition to changes in the load standard, this paper addresses updates in both load and resistance models used for the code calibration. Selected findings from the calibration exercise will be presented and implications for possible revisions to LRFD standards will be discussed. The results of this study were used to assist the ASCE standards committee in their revision of ASCE 16-95. Additionally,

they may be able to serve as a template for other materials (steel, concrete) as they update their LRFD specifications to reflect evolutionary changes in the ASCE 7 load standard.

## Introduction

The procedures underlying the LRFD format for engineered wood construction (AF&PA 1997) combine elements of reliability analysis with the experience gained from decades of successful use of allowable stress design procedures. However, the judgments applied in much of the early development of LRFD hinged directly on the load factors and underlying load distribution assumptions in the 1993 version of the ASCE 7 load standard. Since that time, load factors in ASCE 7 have changed several times and additional information is available regarding load distributions. Assumptions regarding resistance statistics have also evolved during the past decade. For example, early reliability analyses were based on resistance distributions derived from individual data sets. Unfortunately, this approach is confounded by often significant differences that occur between data sets. This approach also neglects the self-calibrating nature of engineered wood product specifications.

This paper presents selected results from a study to assess the reliability of wood members designed in accordance with current LRFD procedures (ASCE 16) (AF&PA 1997). Complete results may be found elsewhere (Rosowsky et al. 2004). This paper first presents the basis for reference resistance statistics intended for use in reliability-based code calibration studies. This is followed by a summary of reference statistics for wood product mechanical properties. Finally, this paper examines the range of first-order second moment (FOSM) reliability indices computed using load statistics (updated, in some cases) and the aforementioned reference resistance statistics. Selected results are presented for the flexural limit state of bending moment (only). These benchmark results are based on an analysis of single members, such as beams, joists, and rafters under several prescribed load combinations. For the cases presented here, combinations of dead, live, and snow load are considered.

### Default Resistance Statistics for Reliability Analysis

To achieve a level of consistency in reliability analyses, one goal of this project was to develop recommendations for default resistance distributions for engineered wood products. Decisions were required for:

- a. distribution form,
- b. separation factors, and
- c. coefficient of variation.

The decision related to distribution form was relatively straightforward. Based on judgments reflected in ASTM D5457 and elsewhere, the default distribution form for reliability analysis of engineered wood products is the two-parameter Weibull. However, the decisions related to separation factors and coefficient of variations (COVs) were more difficult. The following discussion covers the highlights of this decision process. Complete details may be found elsewhere (Gromala and Line 2004).

### Separation Factors

The term “separation factor” is defined herein as the ratio of the characteristic value of a given property to the published design value. For example, the most commonly referenced separation factor in wood design is 2.1, which is the explicit divisor for lumber referenced in ASTM D245 *Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber* for bending and tension. This factor is applied by dividing the characteristic value – which

for bending and tension is the lower confidence bound on the population 5th percentile – by 2.1 to obtain the reference design value.

The latest thinking is that separation factors for reliability analyses must be more stable than those achieved by analysis of individual data sets. In the early days of reliability analysis, the authors of NBS 577 (Ellingwood et al. 1980) searched for representative data sets for various properties of structural wood products to obtain these factors. Unfortunately, such a “data set driven” approach does not reflect the realities of both production variability and application of product standards to establish design values for structural wood products. The design system for lumber was recalibrated (i.e., design values were adjusted to reflect the latest available test data) when results of the North American In-Grade Lumber Testing Program were implemented in the 1991 *National Design Specification*<sup>®</sup> (NDS<sup>®</sup>) (AF&PA 1991). This recalibration realigned design values with the best available reference data, dramatically narrowing the range of ratios of actual versus assumed strengths and stiffnesses.

For other engineered wood products, in which destructive property assessment is conducted on a daily basis, this recalibration is taking place in real-time. For these products, any examination of data sets will demonstrate only slight levels of conservatism relative to the requirements of the product standard – with this conservatism generally related to avoiding “out of control” situations in the manufacturing process.

On the basis of this discussion, the separation factors (extracted from underlying product standards) shown in **Table 1** were proposed by Gromala and Line (2004). Based on these separation factors, and considering the footnotes for **Table 1**, mean-to-nominal ratios of resistance quantities for the different limit states were determined. These mean-to-nominal ratios are shown in **Table 2**.

### Coefficients of Variation

For lumber, the in-grade test data summary (FPRS 1989) shows a range of lumber COVs from as low as 16 percent for Select Structural grade to as high as 48 percent for Utility grade. While it is tempting to use these values as published, one must examine the in-grade data more closely prior to establishing these as reference values for reliability analyses. For example, these COVs are “species wide” values.

**Table 1.**—Benchmark ratios of characteristic values ( $R_{ref}$ ) divided by published design values for Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD).<sup>a</sup>

Ratio	Bending	Tension	Compression	Shear <sup>b</sup>	Modulus of elasticity
$R_{ref}$ / ASD value	2.1	2.1	1.9	2.1	1.0
$R_{ref}$ / LRFD value <sup>c</sup>	0.97	0.97	0.88	0.97	1.0

<sup>a</sup> Assumes that property adjustments for effects such as size, moisture, load duration, etc. are accurately reflected in published design adjustment factors for each design format.

<sup>b</sup> Ratio for I-joists is 2.37 (ASD) and 1.10 (LRFD).

<sup>c</sup> LRFD value ( $\phi \times R_n$ ) defined as 2.16 times ASD value for strength properties.

**Table 2.**—Benchmark mean-to-nominal ratios (LRFD)<sup>a</sup> for various structural products.

	Bending	Tension	Compression		Shear
			Parallel	Perpendicular	
Lumber	1.29	1.37	1.09	1.38	1.11
Glulam	1.12	1.37	1.09	1.38	0.99
SCL	1.13	1.06	1.09	1.38	0.99
I-joists	1.13	n/a	n/a	n/a	1.15

<sup>a</sup> Assumes that property adjustments for effects such as size, moisture, load duration, etc. are accurately reflected in published design adjustment factors.

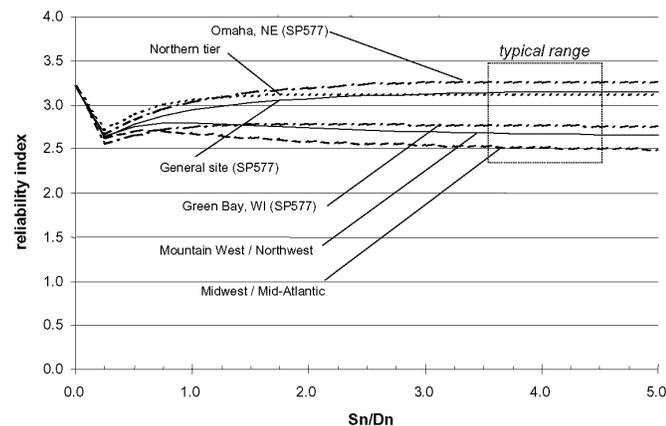
Since no one builds a single structure from species-wide lumber, one must eliminate the component of variability related to this effect. Furthermore, for lower grade material, the high COVs reflect the fact that some of the material is graded low for strength reasons (and this is reflected in the design value) while some has a low grade for wane or warp or other nonstructural reason (even though it has more than adequate strength). Since the ongoing magnitude of this practice is unpredictable, one also must reduce the COV to eliminate this effect.

Based on such considerations, and in consultation with a broad range of wood industry engineers, the default COV's shown in **Table 3** are proposed. Thus, the resistance statistics shown in **Tables 2 and 3** form the basis of the reliability analysis results reported in the remainder of this paper.

### Reliability Analysis

First-order second moment (FOSM) techniques (Ang and Tang 1984, Melchers 1999) are used to evaluate the reliability index,  $\beta$ . The development of load statistics is beyond the scope of this paper. The load statistics used in this study are summarized in **Table 4**.

The results from the FOSM analyses were plotted as a function of nominal load ratio ( $\mu$ ) for the different cases (materials, limit states, load combinations) considered. This is consistent with earlier code calibration studies (Ellingwood et al. 1980)] and provides a convenient way to



**Figure 1.**—Reliability of wood members in flexure (bending moment limit state), D + L.

**Table 3.**—Benchmark COVs for various structural products.<sup>a</sup>

	Bending	Tension	Compression	Shear	MOE
Lumber	20	25	15	20	25 <sup>b</sup>
Glulam	15	25	15	15	10
SCL	15	15	15	15	10
I-joists	15	n/a	n/a	15	10

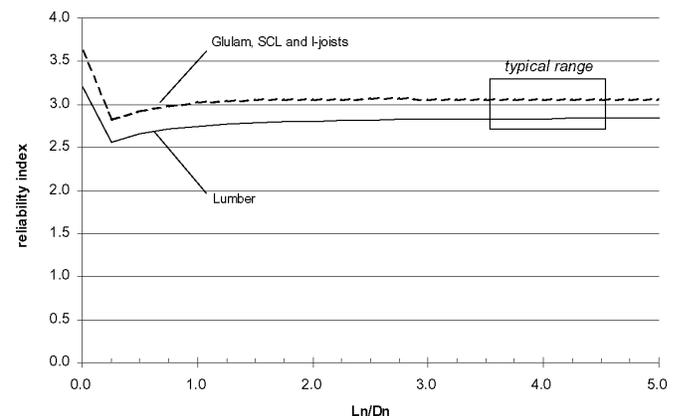
<sup>a</sup> Based on assumptions discussed herein. I-joists are assumed to be used primarily in bending applications.

<sup>b</sup> Value of 25% is for visually graded lumber. Use 15% for machine-evaluated (MEL) lumber and 10% for machine-stress-rated (MSR) lumber.

visualize the trends in reliability over the range of nominal load ratios. Only selected results are presented here. More complete results may be found in Rosowsky et al. (2004).

**Figure 1** presents the reliability results for the flexural members subject to the dead plus live (D+L) load combination, considering the bending (moment) limit state. Structural lumber has a lower reliability based on a single member analysis than engineered wood products (EWPs). In the case of D+L, reliabilities ranged from 2.8 to 3.1 over the load ratio range of interest (indicated by the box showing typical range).

**Figure 2** shows the reliabilities of structural lumber members in flexure (bending moment limit state) subject to D+S considering a range of snow load models and also including



**Figure 2.**—Effect of wood members in flexure (bending moment limit state), D + L.

**Table 4.**—Load statistics and distributions.

Load	Mean-to-nominal	COV	Distribution	Reference
Dead	1.05	0.10	Normal	Ellingwood et al. 1980
Live <sup>a</sup>	1.0	0.25	Extreme Type 1	Ellingwood et al. 1980
Snow <sup>b</sup>	0.82	0.26	Extreme Type 2	Ellingwood et al. 1980
Snow R1 <sup>c</sup>	0.61	0.53	Lognormal	Lee and Rosowsky 2004
Snow R2 <sup>d</sup>	0.84	0.60	Lognormal	Lee and Rosowsky 2004
Snow R3 <sup>e</sup>	0.80	0.58	Lognormal	Lee and Rosowsky 2004

<sup>a</sup> 50-year maximum total (sustained + extraordinary) occupancy live load.

<sup>b</sup> 50-year maximum snow load based on aggregation of 8 sites in northern tier states.

<sup>c</sup> 50-year maximum snow load based on new analyses (Northern tier sites).

<sup>d</sup> 50-year maximum snow load based on new analyses (Midwest/Mid-Atlantic sites).

<sup>e</sup> 50-year maximum snow load based on new analyses (Mountain West/Northwest sites).

two specific sites used to develop the SP577 statistics. (The two sites, Omaha and Green Bay, were found to be the best and worst case sites among the eight sites used to develop the aggregate site statistics. These sites' statistics are not shown in **Table 4**.) The statistics for the two sites (Omaha, NE and Green Bay, WI) also were obtained from SP577. Thus, this figure shows the effect of aggregation of site statistics implicit in the SP577 snow load model. The fact that the updated region-specific snow load models result in even lower reliabilities in some cases reflects (1) the additional years of data, (2) improved distribution fitting techniques, and (3) regional aggregation used to develop the new models.

### Summary and Conclusions

This paper presents selected results from a study to assess the reliability of wood members designed in accordance with current LRFD procedures. This study was part of a project to evaluate reliability levels associated with current LRFD design procedures in light of evolutionary changes to load and resistance factors as well as assumptions underlying resistance statistics for wood-based materials. This paper first presents the basis for reference resistance statistics intended for use in reliability-based code calibration studies. This is followed by a summary of reference statistics for wood product mechanical properties. Finally, this paper examines the range of first-order second moment (FOSM) reliability indices computed using load statistics (updated, in some cases) and the aforementioned reference resistance statistics. Results are presented for the flexural strength (moment) limit state only. These benchmark results, and those presented in the more complete paper (Rosowsky et al. 2004), are based on analysis of single members such as beams, joists, and rafters under several prescribed load combinations. The results show that the range of computed reliability indices can be significantly influenced by regional variations in load statistics. While these results also show slight differences in computed reliability indices from one wood product to another, these differences are not judged to be significant.

At the time the DOL (duration of load, now called time effect) factors were being calibrated for ASCE 16, a target reliability index  $\beta_0$  for the bending moment limit state of 2.3-2.5

(with DOL effects included) was selected for wood flexural members subject to either D+L or D+S (Ellingwood and Rosowsky 1991). For comparison, the range of reliability indices obtained for steel members in flexure designed by the LRFD Steel Specification was 2.2 to 2.7, and a study to develop LRFD for cold-formed steel construction yielded values in this range as well. The results of the present study, which presume load duration effects are properly taken into account through the time effect factor specified in ASCE 16, suggest the range of reliability indices may be considerably wider, ranging from 1.9 to 3.5. The greatest source of variability arises from variability in snow and wind load statistics from site to site. While certainly not suggesting current LRFD provisions for wood are unsafe, the results of this study suggest (1) the need for updating the load models and statistics used in on-going and future code calibration studies, and (2) the potential for site-specific load models in developing next-generation partial safety factors.

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### References

References are available upon request.

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