U.S. Department of Housing and Urban Development
Office of Policy Development and Research

Studies on Probability-Based Design for Residential Construction

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Studies on Probability-Based Design for Residential Construction

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by

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NOTICE

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Introduction

Housing construction in the United States is based largely on accepted practices rather than engineering principles. And while the performance of residential structures has been generally acceptable, the economic losses associated with recent natural disasters has prompted a re-examination of the basis for the design of these structures. Unfortunately, such re-examinations are often done with limited, anecdotal information. In addition, engineering specifications, which apply to a wide variety of buildings, rarely consider the unique experience of residential construction and establish minimum performance expectations (i.e., target reliability) based on past design practice for "engineered" structures.

The objectives of this study were to: (1) quantify the "experience" of conventional residential construction in terms of modern concepts of reliability as used in the development of engineering specifications; (2) lay the groundwork necessary to improve probability-based design procedures as applied to residential (wood-frame) structures; (3) identify ranges of reliability that may be associated with acceptable performance such that minimum or target reliabilities may be contemplated for efficient residential building design to eliminate unacceptably low performance while maintaining a historically consistent perspective on housing value (i.e., safety vs. affordability); and (4) investigate the efficacy of a multi-story live load reduction factor to account for some degree of time independence between maximum loads realized on any one story level of a home.

The first three objectives are very much inter-related with each objective building on the previous. The fourth objective, while essentially requiring an independent study, addresses a significant issue in relation to probability-based design of residential buildings.

The focus of this study is on wood-frame (light-frame) residential structures built using nominal 2-by framing lumber, structural sheathing, and nail fasteners-materials most commonly used today. While by no means a complete analysis of all materials and methods of modern home construction, an effort has been made to consider a representative range thereby allowing generalization of the results. Where it is required or otherwise desirable that wood-frame residential structures be engineered, both allowable stress design (ASD) [AF&PA, 1996b] and Load and Resistance Factor Design (LRFD) [AF&PA, 1996a] standards are considered. Additional prescriptive design documents such as the Wood Frame Construction Manual [AF&PA, 1997] and the Residential Structural Design Guide [HUD, 2000] are also considered.

While this study may be considered as exploratory, it opens new insights into the use of reliability concepts to quantify a historically consistent representation of reliability for a specific class of structures. Interesting trends are identified and discussed in regard to design implications for modern houses. Recommendations for future studies are made at the end of this report.

Organization of Report

This report is organized into three main sections. The first section presents a historical analysis of reliability of selected members in wood-frame residential structures built in the United States over the past 100 years. This innovative study combines analysis and actual data with empirical information and engineering judgment. In addition to providing some historical perspective on relative safety, the results from this study can provide useful baseline information for future code development. The second section presents the results from an evaluation of the reliability levels associated with current design provisions. Both ASD and LRFD wood design provisions are considered, with load combinations involving dead, live, snow and wind (uplift) loads. The results from this study can be used as the basis for calibration of new design provisions or modifications to existing design provisions (e.g., partial factors). The third section presents the results of an analysis to determine load combination (coincidence) factors for multi-story residential live loads. A summary at the end of the report presents suggestions for future work including the potential for a re-evaluation of partial factors in light of (a) changes in design loads, (b) treatment of coincident loads, and (c) consideration of loading conditions specific to residential structures.

Section 1 - Reliability Analysis of Residential Wood-Frame Construction from 1900 to Present

INTRODUCTION

Purpose

Single-family housing has improved substantially over the past 100 years. These improvements, including new materials and methods of construction, have increased the rate of residential construction, decreased the cost of construction, and increased the efficiency of housing structures and systems. As a result, today's house is more cost effective to build, inhabit, and maintain.

Changes in construction methods, materials, and housing styles have likely influenced the relative performance (including safety) of structural components and assemblies. For example, lumber sizes have changed (full-size vs. dressed), growth characteristics have changed (old-growth vs. new-growth), and sheathing products have evolved (boards vs. panels). The extent to which each of these has affected structural performance over the past century is not known. Most residential structures are not engineered in the way larger structures (e.g., steel or reinforced concrete buildings) are engineered. As a result, little is known about the relative safety of these assemblies as-built today using conventional materials and framing techniques, or about how these levels of relative safety have changed over the past 100 years with changes in materials and practices.

Scope and Limitations

The purpose of this study was to quantify the reliability (relative safety) of selected structural members in single-family residential structures built in the United States over the past century. In addition to consideration of relative risk (safety) as represented by a second-moment type of reliability index, changes in this safety index with changes in construction trends (materials and procedures) were also evaluated. The components considered were some of the principal members in the structural framing system, namely floor joists, roof rafters, and connection of roof sheathing. Materials for floor joists and roof rafters were limited to solid lumber only. Engineered wood composites, laminated members, and trusses were not considered since they are relatively new. Sheathing materials included boards and panel products (plywood and oriented strand board). Finally, the study makes no attempt to quantify reliability associated with differences in "system effects" that go beyond simple "single member" structural models.

A literature review was conducted to evaluate construction practices and trends in material use and framing techniques used over the past century in the United States. The objective of this study was to quantify, in an average (or typical-use) sense, practical bounds on the reliability of these components considering the most widely used materials, member sizes, framing techniques, connection schedules, and so forth. Every effort was made to keep the focus of this study to the "common" house construction of the time. However, it is recognized that within this range of single-family homes, there may exist widely varying levels of craftsmanship, varying by geographic region, era, and even contractor. In addition, reported information on common housing construction practices may not reflect certain conditions, such as house construction in rural, non-coded regions of the United States (particularly earlier in the past century).

Trends were identified in various parameters such as roof slope, member spacing, member size, member grade, and member species. Specific consideration was given to specific loads and load combinations relevant to each of the member types. In this study, the reliability index (β or Beta) was computed using a first-order second-moment formulation. Specifically, the first-order reliability method (FORM) first presented by Rackwitz and Fiessler was employed as described in more detail elsewhere [Ang and Tang, 1975; Melchers, 1999; Rosowsky, 1997]. This method was utilized for its applicability to non-linear limit state functions of non-normal random variables. Calculations were performed using a template developed in a *MathCAD* worksheet. The parameters were varied for each case (structural element, load combination, material properties) considered.

The reliability function (or limit state function) for the floor joists and roof rafters is based on a normalized formulation in which the random variables are presented in ratio to their nominal (code-specified) values. This will be referred to as a "normalized random variable formulation" herein. The nominal values (i.e., in the denominator of each random variable) are those that would be used to design a member. This is described further in a later section. The statistics on flexural strength or modulus of rupture (MOR) and stiffness or modulus of elasticity (MOE) for the framing members in the roofs and floors, for all species, sizes, and grades considered, were taken from the In-Grade Testing Program [Green and Evans, 1987]. The evaluation of roof sheathing reliability, however, did not use a normalized random variable formulation, but rather was based on actual resistance data on sheathing uplift capacity. Hence, the limit state function was formulated in terms of actual values of the resistance and loading terms. The resistance (uplift capacity) calculation was based on the procedure in the Commentary to the National Design Specification (NDS) for Wood Construction [AF&PA, 1997].

The study was limited by the availability of useful information on materials, material properties, and conventional practices for the various time periods considered. Most references presented information in the form of allowable stresses and, in some cases, allowable spans for typical loading conditions. However, the information was incomplete or imprecise in some cases, and unavailable in others. As a result, some information had to be combined, extracted, or inferred to develop a suitable statistical database to perform the reliability analyses. It was assumed that, at any time during the past century, builders followed common practices as well as any governing building code, where it was applicable. It is known that the first part of the century lacked a universally adopted building code, and as a result construction practices were often based on experience and common rules-of-thumb.

For the purposes of this study, the century was divided into distinct regions, based largely on trends in construction practices. The discrete points in time considered were 1906, 1931 and 1997. In 1906 and 1931, design values were presented for a very small number of lumber grades for the available species. These grades (and species) did not necessarily correspond to those in use today. Therefore, a mapping was made to transform historical stresses (and grade/species categories). Much of the literature indicated that a factor of safety (on ultimate stress) was used to determine the allowable stresses. However, this factor of safety was not consistent, nor was it always reported. In addition, little information was found to describe the variability in the test data (where test data was even available). Thus, the mapping used to assign statistics to mechanical properties of historical building materials is subjective. However, it is based on whatever test data and allowable design values were available, as well as “best efforts” at interpolation using engineering judgment and information from the literature review.

LITERATURE REVIEW

Data Sources

The document of most value in developing the statistics for use in this reliability study was “A Historical Profile of Structural Materials and Methods for Home Building in the United States: 1900 to 2000,” prepared by the NAHB Research Center [HUD, 2000]. This document presented the findings of a study by NAHB to summarize housing construction trends and practices. Many other documents were also reviewed in order to obtain data on trends in housing construction. Flexural strength (MOR) and modulus of elasticity (MOE) information for floor joists and roof rafters was obtained from [AF&PA, 1997; AREA, 1929; Kidder, 1906; NAHB, 1967; NAHB, 2000; Ostrup, 1910; Thurston, 1903; Vose, 1873]. Information on roof sheathing was taken from [Fridley et al., 1995; McLain, 1997; Rammer et al., 2001; Schiff et al., 1996; Sutt et al., 2000].

Summary of Relevant Changes

Structural lumber has changed in actual dimensions over the last 100 years. These changes in dimension happened in several steps, with each step resulting in a member that was slightly smaller than the previous. At the start of the century, lumber was sawn to nearly full dimensions. That is, the cross-section of a 2 × 8 was nearly 2 inches by 8 inches in cross-sectional area. By the mid-1900’s the size of lumber had been reduced to what is now used (“dressed” sizes). That is, a 2 × 8 now measures 1.5 inches by 7.25 inches in cross-section. In the normalized limit state formulation, the section property cancels-out. The section size does, however, have an effect on the nominal (code-specified) MOR value, e.g. allowable stress from the NDS.

As virgin growth forests began to disappear and wood was increasingly harvested from managed forests, the strength characteristics of the wood began to decline. In the early part of the century, the wood supply exceeded the demand and stands of trees were in place for longer periods of time before being harvested for lumber. This created more favorable growth characteristics such as tighter grain. However, as virgin growth lumber diminished and the post-war housing boom increased the demand for wood framing members, the use of managed forests increased and the average time stands of timber were allowed to grow decreased. Consequently, the strength of the lumber began to decline. While these trends varied with species and region of the country, the overall effect was a decrease in effective grade and structural design properties.

The literature review indicated that the two most common species used in single-family home construction were Douglas Fir and Southern Yellow Pine [Kidder, 1906; NAHB, 2000; Richey, 1951]. More recently, the species groups Douglas Fir-Larch (DF-L) and Southern Pine (SP) are the most common. These are softwood species groups and have structural design properties (i.e., design stresses) that are similar in magnitude. The dominance of these species groups, in most regions of the country, has remained fairly constant over the last century.

Lumber grade was a significant factor when evaluating the reliability of each component. At the start of the century, very few grading rules were in place to classify lumber. Grading rules gradually developed into very profiled categories. Thus, a challenge existed in how to map the use of (and transitions between) grading rules used in each era. The method finally adopted first identified the most common grades utilized within a given era and then compared those values to present-day values. In most cases the historical grades and categories were associated with an allowable (and sometimes an ultimate) stress. However, this process relied heavily on engineering judgment and some understanding that the strengths were effectively decreasing, as a result of both changes in growth characteristics and changes in how allowable stress values were being determined.

Structural loads were assumed to be fairly consistent during the last century. Without a unified building code, it was the responsibility of the local building authority to stipulate minimum design forces. These tended to vary, but early sources [Kidder, 1906] listed recommended safe superimposed floor loads for dwellings and tenements. These safe loads often were region-specific and ranged from 40 psf in Buffalo, NY and Denver, CO to 70 psf in Chicago, IL and St. Louis, MO. Each building authority relied heavily on experience and current trends in mandating the floor live loads they deemed prudent. The most widely cited floor live load, found in a variety of documents over the past 100 years, was 40 psf. This is the same value found in both ASCE 7 and the UBC for basic floor loading of residential structures in use today.

Combinations of structural material and loading parameters for floor joists, roof rafters, and roof sheathing were used to develop “bands” on the reliability indices (Beta) for the specific components. A summary of these bands is shown in Figure 1.1. The specific analyses and results will be discussed in detail in the following sections. It should be noted that the bands are intended to show trends in the range of reliability and average level of reliability associated with discrete points in time. Thus, the bands should not be interpreted as representing a gradual “continuum” of change. In fact, certain changes in reliability may be associated with the time frame over which new products were introduced and eventually considered as “conventional practice”.

Overview of Reliability Bands, 1906 - Present

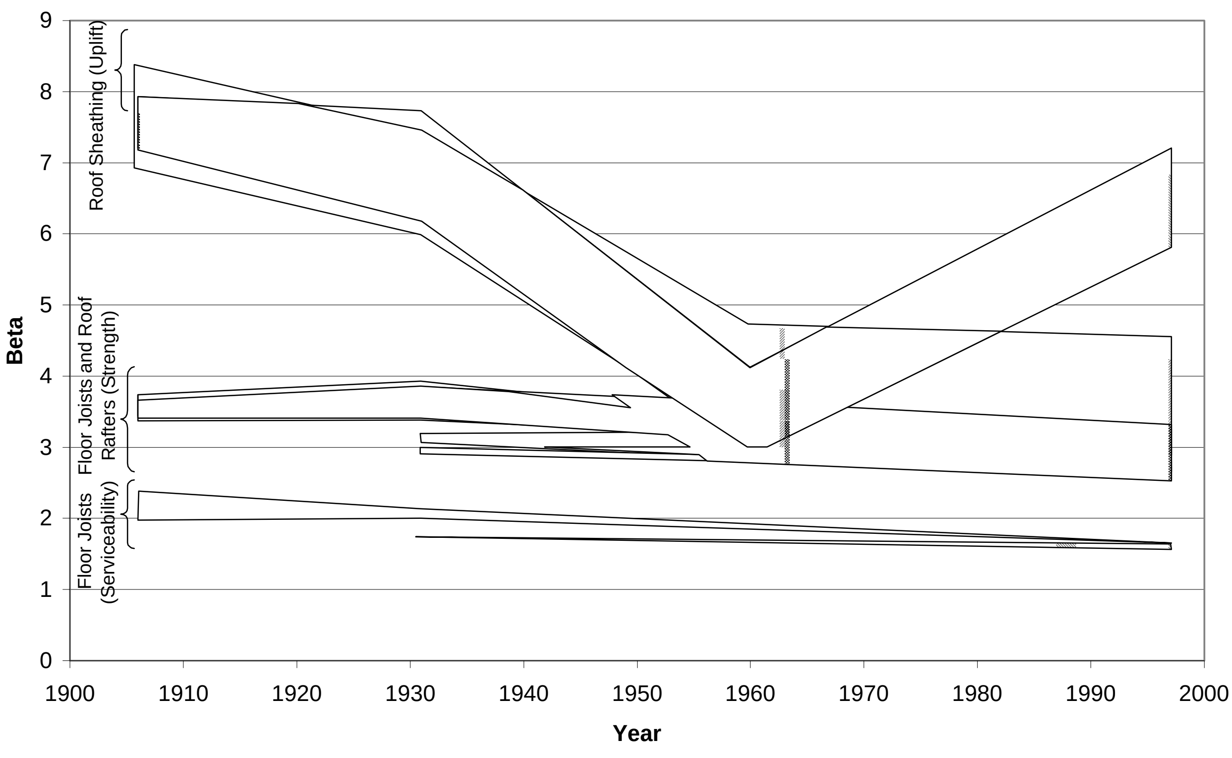


Figure 1.1
Overview of Reliability Bands, 1906 to Present

For purposes of interpretation, Beta can be described as an index of the level of relative safety exhibited by a design or structural feature. As Beta increases, the level of safety increases. In more technical terms, Beta is the number of standard deviations that the mean value of the performance function is from zero. The performance function (or limit state function) is simply a subtraction of load from structural resistance or capacity and failure is indicated when a value of zero or less is obtained. Since the design parameters that are input into the performance function are random variables (having some mean and variance) then the performance function also exhibits such statistical qualities. A more detailed description of Beta may be found elsewhere [Rosowsky, 1997].

RELIABILITY ANALYSIS OF FLOOR JOISTS

Summary of Changes

The strength of the member is a function of the species and the grade of lumber. There have been no significant changes in the method of construction that would affect floor joist reliability. Even with balloon framing, which was widely used earlier in the century, joists still rested on ledgers and partitions, much as they would today. Lateral bracing of the members was similar to that used today. While longer-span members were available, and hence continuous span floor framing may have been used more widely in the first half of the century than in modern day, this study considers only single span simply-supported floor joists.

Within the range of modest single-family homes, there seemed to be an upper and lower bounds on "quality" of home construction as early as the turn of the century. HUD [2000] reported that kit homes sold by Sears had two choices of quality level, so called "Honor Built[1]" and "Standard Built." It was assumed herein that this trend of levels of quality (providing effective bounds) has continued to the present and hence this terminology was kept. Honor Built (HB) came to refer to a member that was of higher quality but less widely used, whereas Standard Built (SB) was used to refer to members of more widely used materials, generally of lower (or more common) quality. The distinction between Honor Built and Standard Built for floor joists referred to both the *grade* of lumber used and the *spacing* at which the lumber was placed to construct the floor. Member sizes were found to be relatively consistent within a given time period and quality level. Table 1.1 summarizes the information compiled for floor joist from 1906 to the present. The most notable change was in the size of the floor joists. However, the nominal design values also evolved over the course of the century.

Grading terminology and the grade categories identified for the eras in [HUD, 2000] were utilized in selecting the grades appropriate for each species and quality level. For example, the Sears catalog of 1928 advertises lumber for its Honor Built home as "Virgin growth, dense grain" Douglas Fir lumber from the Pacific Northwest. The HUD report [HUD, 2000] also indicates that in the early 1900s there were four grade classifications. These are, from highest to lowest quality, No.1, No.2, No.3, and Culls. No.1 grade was recommended for joists and rafters, however the use of No.2 lumber was known to be more economical. This information, and the

[1]It may be more appropriate to refer to this as simply "Above Standard."

typical density of the species group, was used to equate the HB and SB members (e.g., in 1931) to select structural (SS) and No.1 by today's grading standards.

TABLE 1.1
SUMMARY OF FLOOR JOIST INFORMATION

				Nominal Design Value for Modulus of Rupture (psi)		**Nominal Design Value for Modulus of Elasticity ($\times 10^6$ psi)**	
Time	Grade	Size	Grade[1]	SP	DF-L[4]	SP	DF-L[4]
1906	HB[3]	2×8	SS	1800	1620	1.78	1.425
1931	HB	2×8	SS	1600	1600	1.6	1.6
	SB	2×8	No. 1	1200	1200	1.6	1.6
1997[2]	HB	2×10	No. 1	1300	1100	1.7	1.7
	SB	2×10	No.2	1050	990	1.6	1.6

[1]As mapped to today's grade categories.
[2]Lumber sizes assumed nominal for 1997 data.
[3]Only one grade of data was available for 1906.
[4]Includes size adjustment factor.

Limit State Formulation

Two floor joist failure modes were considered: a *strength* limit state in which the maximum stress exceeds the allowable bending stress, and a *serviceability* limit state in which the maximum deflection exceeds the allowable (live load) deflection. The limit state function was formulated as:

$$g(x) = \frac{X_1}{c_0 \cdot c_1} - \frac{1}{c_2 + c_3 \cdot c_4} \cdot \left(X_2 + c_3 \cdot X_3\right) = 0 \qquad (1.1)$$

where $x = \{X_1, X_2, X_3\}$ is the vector of basic variables and $c_0 \ldots c_4$ are deterministic quantities. The function $g(x)$ is formulated such that "failure" corresponds to the condition $g(x) < 0$. The first term in equation (1.1) is the normalized random variable corresponding to resistance (X_1) divided by constants (c_0, c_1) used to describe reductions in the resistance quantity or applicable modification factors (e.g., from the NDS). The second term contains the loading terms normalized by their nominal values. The constants c_2 and c_4 describe any load factors that would be applicable for dead and live load, respectively. The constant c_3 is the ratio of nominal loads, L_n/D_n. Many of these constants default to one in the case of allowable stress design.

Formulating the limit state equation as shown in Eqn. (1.1) permits its use for a wide range of design and/or loading conditions. The c_0 and c_1 factors could include the repetitive member factor for bending, the load duration factor, and so forth. Even before the turn of the century, it was known that wood members had the capability of carrying higher loads when they were applied for shorter durations of time [Breyer et al., 1999]. The repetitive member factor takes into account lower-bound increases in system strength and stiffness provided by transverse load distributing elements (load sharing) and composite action [AF&PA, 1997]. These adjustments were assumed to apply at all points in time considered in this study. Eqn. (1.1) is formulated such that it is independent of span, cross-sectional properties, and spacing of the members. By using a normalized random variable formulation, these parameters effectively cancel-out (that is, they appear in both the design and the checking equations). The ratio of nominal live to dead load was assumed to be four, a value typical for wood structures.

Strength

The strength limit state analysis for floor joists considered the combination of dead plus live load. The values assigned to the constants were based on the provisions of the 1997 NDS. The constant c_1 was assigned a value of 1.15 (the product of the load duration factor of 1.0 and the repetitive member factor of 1.15). Thus the nominal value was increased, thereby reducing the normalized random variable. This had the effect of reducing the effective strength (capacity) and thus reducing the reliability of the member. Table 1.2 summarizes the statistics used in calculating the flexural strength reliabilities of the floor joists.

TABLE 1.2
LOAD AND RESISTANCE STATISTICS FOR FLOOR JOIST ANALYSIS (STRENGTH)

		HB			**SB**		
		Mean-to-nominal	COV[1]	Distribution[2]	Mean-to-nominal	COV	Distribution
Load	Dead	1.05	0.10	Normal	1.05	0.10	Normal
	Live (50-yr max.)	0.90	0.20	ET-I	0.90	0.20	ET-I
Resistance (MOR)	1906, High	4.76	0.235	ET-III	-	-	-
	1906, Low	4.94	0.275	ET-III	-	-	-
	1931, High	5.36	0.235	ET-III	6.13	0.338	ET-III
	1931, Low	5.00	0.275	ET-III	5.80	0.348	ET-III
	1997, High	4.66	0.272	ET-III	5.63	0.314	ET-III
	1997, Low	5.07	0.347	ET-III	5.38	0.426	ET-III

[1]Coefficient of Variation (COV).
[2]Extreme Value Type I (ET-I); Extreme-Value Type III (ET-III).

Mechanical Properties

The resistance term for the limit state of strength is a normalized random variable comprised of the actual value, taken from the In-Grade Test Program (IGTP) [Green and Evans, 1987] for the given species/size/grade (mapped as needed), divided by the nominal value from the era being considered. An ET-III (2-parameter Weibull) distribution is assumed, with point estimate values for MOR taken from the IGTP. Used as inputs to analyze reliability, the statistics for the normalized random variable representations of resistance (MOR) are shown in Table 1.2.

Loading

The relevant load combination for floor joists is dead plus live load. Failure was considered to occur when the maximum stress in the member exceeded its capacity (allowable stress). In utilizing the normalized values for dead and live load, it was assumed that both dead and live load were uniformly distributed and that the statistical characterization of the loads remained unchanged over the last century. As described previously, the recommended design load remained constant from 1906 to the present. It is possible that occupancies, uses, weights of furnishings, etc. may have changed over the years, however these are not considered here. Used as inputs to analyze reliability, the statistics for the normalized random variable representations of dead and occupancy live load are also shown in Table 1.2.

Serviceability

The serviceability limit state analysis considered sustained live load only. The normalized random variable for the modulus of elasticity had no further adjustment since all the applicable factors in the NDS default to unity. (The load duration factor and repetitive member factor do not apply to the MOE.) Table 1.3 presents the load and resistance statistics used to evaluate the serviceability reliability of the floor joists.

TABLE 1.3
LOAD AND RESISTANCE STATISTICS FOR FLOOR JOIST ANALYSIS (SERVICEABILITY)

		HB			SB		
		Mean-to-nominal	COV	Distribution	Mean-to-nominal	COV	Distribution
Load	Live (L_s only)	0.24	0.90	Gamma	0.24	0.90	Gamma
Resistance (MOE)	1906, High	1.79	0.207	Lognormal	-	-	-
	1906, Low	1.06	0.197	Lognormal	-	-	-
	1931, High	1.153	0.211	Lognormal	0.959	0.205	Lognormal
	1931, Low	1.086	0.207	Lognormal	0.962	0.212	Lognormal
	1997, High	0.916	0.185	Lognormal	0.949	0.246	Lognormal
	1997, Low	0.919	0.192	Lognormal	0.932	0.254	Lognormal

Mechanical Properties

The resistance term in the limit state equation is a normalized random variable representation of the modulus of elasticity (MOE). As with the MOR value, this value is a ratio of the actual value (with statistics obtained from the IGTP) divided by the nominal (code-specified, where appropriate) value for that era. A Lognormal distribution was assumed for MOE. The assumed statistics for normalized MOE are shown in Table 1.3.

Loading

The sustained live load intensity, which has an average duration of eight years (length of a typical tenancy or occupancy), was assumed to follow a Gamma distribution. The statistics are shown in Table 1.3.

Results

Figure 1.2 presents the strength and serviceability reliability bands for floor joists in flexure based on an analysis of the data presented in Tables 1.2 and 1.3. Over the last century, the upper and lower bounds of the reliability bands have decreased for both strength and serviceability. Since the loading is assumed not to have varied (i.e., statistical models have not changed), these reductions in reliability are due only to changes in the resistance term of the limit state equation. Generally speaking, there will be an increase in reliability with an increase in the normalized resistance. However, this is not always the case as the COV in resistance may also change. The serviceability reliability bands were much more sensitive to small changes in normalized random variable for resistance (MOE). The nominal MOE is typically based on the mean value (i.e., no further factor of safety). This results in a normalized MOE of nearly one. The serviceability reliability bands have nearly the same shape and trend over time as the strength bands, however they are narrower and lower in magnitude.

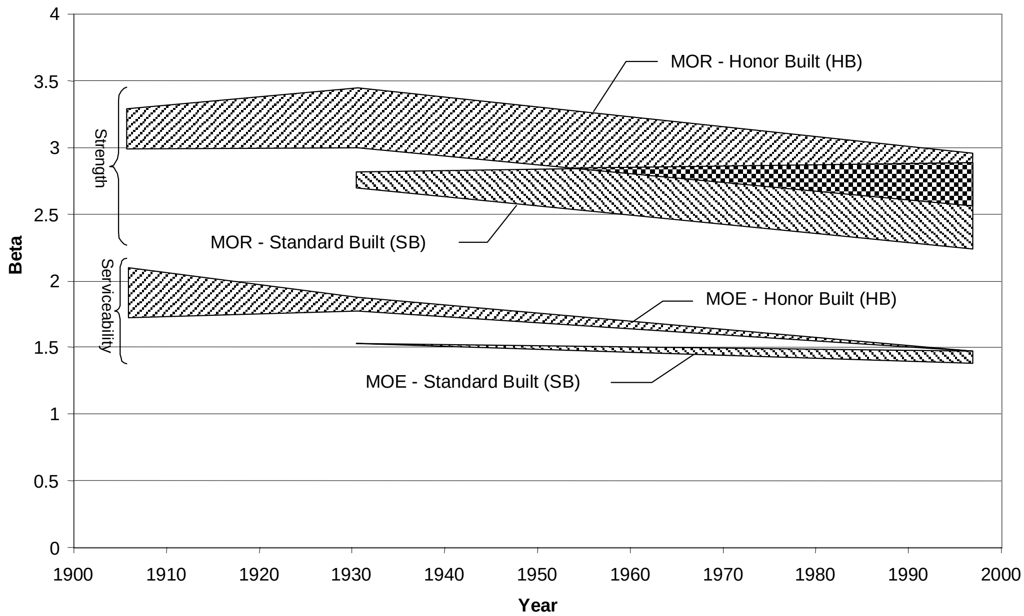


Figure 1.2
Reliability of Floor Joists, Strength (D+L) and Serviceability (L_s)

RELIABILITY ANALYSIS OF ROOF FRAMING MEMBERS

Summary of Changes

As with floor joists, the size and strength of members used for roof rafters have changed over the years. The strength of the member is a function of the species and the grade of lumber. Using a similar mapping approach as used for floor joists, statistics were obtained from the IGTP for the roof framing members considered in this study. Only solid lumber rafters were considered. Trusses were not addressed.

There have been few changes in construction methods for framing of roofs using lumber rafters. These members bear on the top plate of the supporting wall and frame up to a hip or ridge board. The methods used to connect these members, however, have changed somewhat. Pneumatically-driven nails are commonly used and light-gauge metal straps, ties, and hangers are frequently used in conjunction with nails to join the rafters to the aforementioned supporting elements. The changes in fastening methods have relatively little effect on the end restraint or the capacity of the member, and hence do not influence the strength limit state analysis. All members are assumed to be simply-supported.

A distinction between levels of quality was also made for roof rafters. The Honor Built (HB) and Standard Built (SB) terminology was maintained. Roof rafters are typically smaller in cross-section than members used for floor joists for the same span since roof live loads are typically less than those for floors and deflection limits rarely control the design of the roof member. Since the design of roof rafters is rarely controlled by deflection limitations, serviceability reliability was not considered. Differences in construction quality are reflected in the grade of lumber used and the spacing of the members. Table 1.4 summarizes the information compiled for roof rafters from 1906 to the present.

TABLE 1.4
SUMMARY OF ROOF RAFTER USAGE CHANGES

				Nominal value for MOR (psi)	
Time	Grade	Size	Grade[1]	SP	DF-L[4]
1906	HB[3]	2×6	SS	1800	1620
1931	HB	2×6	SS	1600	1600
	SB	2×6	No.1	1200	1200
1997[2]	HB	2×8	No.1	1500	1200
	SB	2×8	No.2	1200	1080

[1]As mapped to today's grade categories.
[2]Lumber sizes assumed nominal for 1997 data.
[3]Only one grade of data was available for 1906.
[4]Includes size adjustment factor.

Strength Limit State Formulation

The flexural strength limit state analysis considered the combination of dead plus roof snow load. (This combination controls, e.g., over roof live load, in many regions of the country.) The limit state function for roof rafters was formulated as described previously for floor joists. The constant c_1 was assigned a value of 1.32 (1.15*1.15) to account for the repetitive-member action and load duration effect. The nominal load ratio ($c_3 = S_n/D_n$) was assumed to be 4. Table 1.5 presents the statistics used to calculate the flexural strength reliabilities of roof rafters.

TABLE 1.5
LOAD AND RESISTANCE STATISTICS FOR ROOF RAFTER ANALYSIS (STRENGTH)

		HB			**SB**		
		Mean-to-nominal	COV	Distribution	Mean-to-nominal	COV	Distribution
Load	Dead	1.05	0.10	Normal	1.05	0.10	Normal
	Snow	0.82	0.26	Type II	0.82	0.26	Type II
Resistance (MOR)	1906, High	5.18	0.235	Type III	-	-	-
	1906, Low	5.37	0.275	Type III	-	-	-
	1931, High	5.83	0.235	Type III	5.54	0.338	Type III
	1931, Low	5.44	0.275	Type III	5.38	0.348	Type III
	1997, High	4.93	0.348	Type III	5.60	0.390	Type III
	1997, Low	4.08	0.338	Type III	5.26	0.386	Type III

Mechanical Properties

The statistics for the ultimate strength (MOR) of the 2×8 framing members were taken from the IGTP [16]. Only limited data were available for 2×6 lumber. However, data were available in the IGTP for 2×8 and 2×10 lumber of the same grade. Therefore, in order to obtain values for 2×6 lumber, two methods were employed to estimate the strength statistics. The first was linear interpolation based on member depth. The second utilized the size adjustment from ASTM D1990 [ASTM, 2000]. Both methods produced similar results. For the purposes of this study, the ultimate strength statistics for the 2×6 members, where they were not explicitly available in the IGTP, were based on the ASTM D1990 procedure.

The MOR was assumed to follow a Type III extreme value (Weibull) distribution. Statistics were obtained using the non-parametric estimates from the IGTP. The resistance (MOR) statistics used for the flexural strength limit state analysis of the roof rafters are shown in Table 1.5.

Loading

The critical load case, of the strength limit state, involves dead plus snow load. The failure definitions and load distribution assumptions were the same as those made for the floor joists and the limit state was again formulated to be independent of span and tributary area. Also, a typical roof slope of 4:12 to 5:12 was assumed. The statistics assumed for roof snow load represent an aggregate of sites in the northern U.S. (for 50-year maximum snow load). The assumed load statistics are shown in Table 1.5.

Results

The reliability bands obtained for the roof rafters are shown in Figure 1.3. These bands follow the same trends as the strength reliability bands for the floor joists. However, the rafter reliability bands are narrower than the floor joist bands, especially in the latter part of the century. Also, a noticeable decrease in reliability is observed for roof rafters from the beginning of the century to the end. There is also less overlap in the bands (HB and SB) for roof rafters than there was for floor joists.

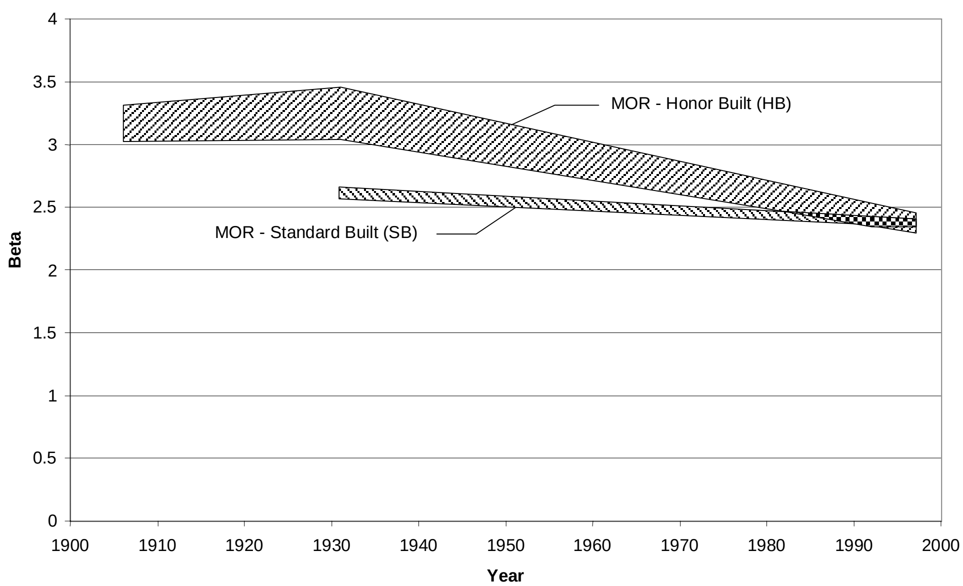


Figure 1.3
Reliability of Roof Rafters, Strength (D+S)

RELIABILITY ANALYSIS OF ROOF SHEATHING

Summary of Changes

Roof sheathing has evolved considerably over the past century, both in materials (boards, panels) and fastener type/schedules. The reliability band developed in this section therefore attempts to capture these changes and variations in construction methods/materials. No distinction in quality (i.e., HB vs. SB) is made, however. The most dominant species for the roof sheathing materials continued to be Douglas Fir and Southern Pine. Changes in roof sheathing have included size, material, fastener spacing, and fastener size. In 1906 roof sheathing was typically full sawn planking in 6 and 8-inch widths. In 1931, planks were still used but they were more commonly 3/4-inch-thick. A common rule of thumb was that the penny size of a common nail used to attach the boards was at least the thickness of the wood being secured, in eighths of an inch [Richey, 1951]. For example, a board that was 3/4-inch thick would be fastened with a 6d nail. This rule of thumb was followed when evaluating the uplift resistance for wood planks in this study. The area tributary to each fastener is a function of both the nail spacing and the spacing of the framing members. Roof rafters were commonly spaced at 16- to 24-inches-on-center. In a 6-inch wide plank, two nails were commonly used, while three were used in an 8-inch wide plank. The spacing of the planks varied with the type of roof covering materials used. However, when considering the load path from the applied wind suction to the supporting framing, any load distribution provided by the roof covering materials was neglected. Thus, the area tributary to each fastener was not affected by spacing between planks.

From the mid-1950s to the mid-1960s, the use of plywood increased for roof and floor sheathing [HUD, 2000]. Today, oriented strand board is increasingly used as a roof sheathing material. As a consequence of the larger panel size (typically 4 ft. × 8 ft.), and hence the relatively smaller number of fasteners used, the tributary area to each fastener increased over that for boards. The thickness of the panel varies based on the framing member spacing and also slightly by the type of panel product. The length of penetration of the nail into the framing member is a function of the thickness of the panel and is used to evaluate the pullout resistance (withdrawal capacity) of fasteners.

Strength Limit State Formulation

The critical failure mode for roof sheathing is removal (uplift) due to wind suction forces. The uplift capacity is a function of nail withdrawal as well as fastener spacing. A FORM analysis was performed as before, however the limit state function was not expressed in terms of normalized variables. (This was not possible since no uniform design procedure is accepted for roof sheathing, and hence an acceptable 'nominal' value could not be defined.) Instead, the roof sheathing uplift limit state function was expressed in terms of actual values for load and resistance. The load was based on code-specified actions on a simple gable-end house (see footnote in Table 1.6), while the resistance quantities were obtained from tests conducted at Clemson University. The limit state function for roof sheathing uplift is written as:

$$g(x) = c_o \cdot \frac{6900 \cdot G^{2.5} \cdot D \cdot L_p}{A_T} - (W - D) = 0 \tag{1.2}$$

The first term (resistance) in eqn. (1.2) is essentially equation C12.2-1 of the NDS commentary [AF&PA, 1997] multiplied by a factor of safety of five (typical for fasteners in withdrawal[2]).

[2]Actual nail withdrawal values are highly variable and are dependent on a variety of in-service conditions. In addition, significant variation exists between field data and laboratory data [AF&PA, 1997; Sutt et al., 2000].

This equation forms the basis for the withdrawal values presented in Table 12.2A of the NDS, and is an empirical relationship derived from laboratory tests. The equation gives load per inch of penetration, and so is multiplied by the depth of penetration L_p to account for the sheathing thickness and specific nail used. The term c_o is provided to account for any applicable modification factors. The resistance is divided by the tributary area, A_T, so that the resistance is expressed in units of *psf*, consistent with the applied loads. Table 1.6 presents the statistics used to calculate the roof sheathing (uplift) reliabilities.

TABLE 1.6
LOAD AND RESISTANCE STATISTICS FOR ROOF SHEATHING ANALYSIS (WITHDRAWAL)

		Nominal [1,2]	Mean-to-nominal	Mean	COV	Distribution
	Dead	4.0 psf	0.95	3.8 psf	0.05	Normal
Load	Wind (coastal)[3]	71 psf	0.48	34.1 psf	0.26	Type I
	Wind (inland)[3]	28 psf	0.78	21.8 psf	0.37	Type I
Resistance	variable[4]	-	-	varies[1,4]	0.20	Lognormal

Notes:
[1] Dead load consists of weight of panel, roll felt, and asphalt roofing material.
[2] Nominal wind values are computed for roof suction loads at the eaves of a simple gable structure with an average roof height of 15 ft.
[3]Coastal wind environment defined by 130 mph, Exposure C per ASCE 7-98; inland wind environment defined by 90 mph, Exposure B per ASCE 7-98. Coastal (hurricane) wind load statistics were adapted from event-based simulation (Rosowsky and Huang, 2000); inland ("non-hurricane") site statistics adapted from (Ellingwood et al., 1980).
[4]See Figure 1.4A and B for structural parameters associated with each time period; panel uplift capacity data obtained from tests conducted at Clemson University (Schiff et al., 1996).

Mechanical Properties

The equation presented in the commentary to the NDS is an empirical fit to actual nail withdrawal values reduced by a factor of safety of five. Thus, an estimate of the mean value is obtained by multiplying the allowable capacity provided by the empirical equations by the safety factor. Rammer, Winistorfer, and Bender [2001] found that smooth shank nail withdrawal capacities are best fit by a lognormal distribution. Results from nail withdrawal tests performed at Clemson University and elsewhere have shown that a COV of about 20 percent is typical for smooth shank nails in withdrawal [e.g., Schiff et al., 1996]. Little information is available on the capacities of ring shank and annularly threaded nails, as these products are not highly standardized. While their withdrawal capacities are recognized to be higher than smooth shank nails, the NDS does not provide any strength increase for these deformed shank fasteners. Thus, only smooth-shank nails are considered here.

Loading

The loads acting on the roof sheathing are applied wind load (suction) and the dead load of the roof covering and sheathing materials, which counteracts the uplift load due to wind. See Table 1.6 for assumed statistics on these quantities.

A brief investigation into changes in roof sheathing and roof covering material weights for residential structures revealed few changes. A value of 4 psf was taken as a representative nominal dead load. A COV of 0.05 was assumed. Note that this is lower than the 0.10 typically used for dead loads, however in this counteracting situation, the smaller COV is more conservative. For the same reason, a mean-to-nominal dead load of 0.95 was used (instead of 1.05).

The nominal (design) wind load was determined using ASCE 7-98 [ASCE, 1998]. A coastal (hurricane) region with a basic wind speed of 130 mph and exposure C and an inland (typical) region with a basic wind speed of 90 mph and exposure B were assumed. Southern Pine, which is predominant in the Southeastern United States, was the only species of framing member considered for the coastal (hurricane) region. Southern Pine and Douglas-Fir larch were both considered in the inland (typical) wind region.

Results

The results (reliability bands) obtained for roof sheathing subject to wind uplift are shown in Figures 1.4A and B. When planks were used, in the early part of the century, the reliabilities were quite high, a consequence of both the nail sizes and the effective tributary areas. The tributary area of a single nail was about 0.3 to 0.5 square feet with planks. This is much smaller that the typical values of tributary area for sheathing panels of about 0.67 to 2.0 square feet. Thus, planks had considerably higher uplift capacity. The decline in reliability between 1906 and 1931 was largely a result of the thickness of the board fastened. This affected the size of the nail used, and hence both the diameter and penetration length of the fastener.

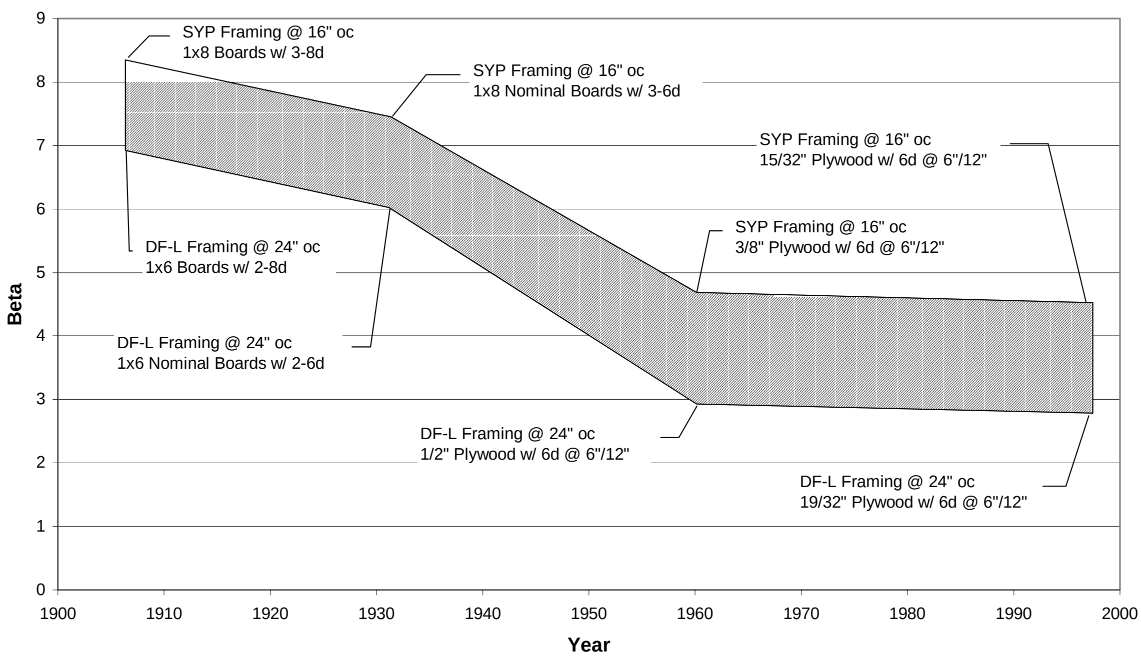


Figure 1.4A
Reliability of Roof Sheathing (W-D) in Inland Region

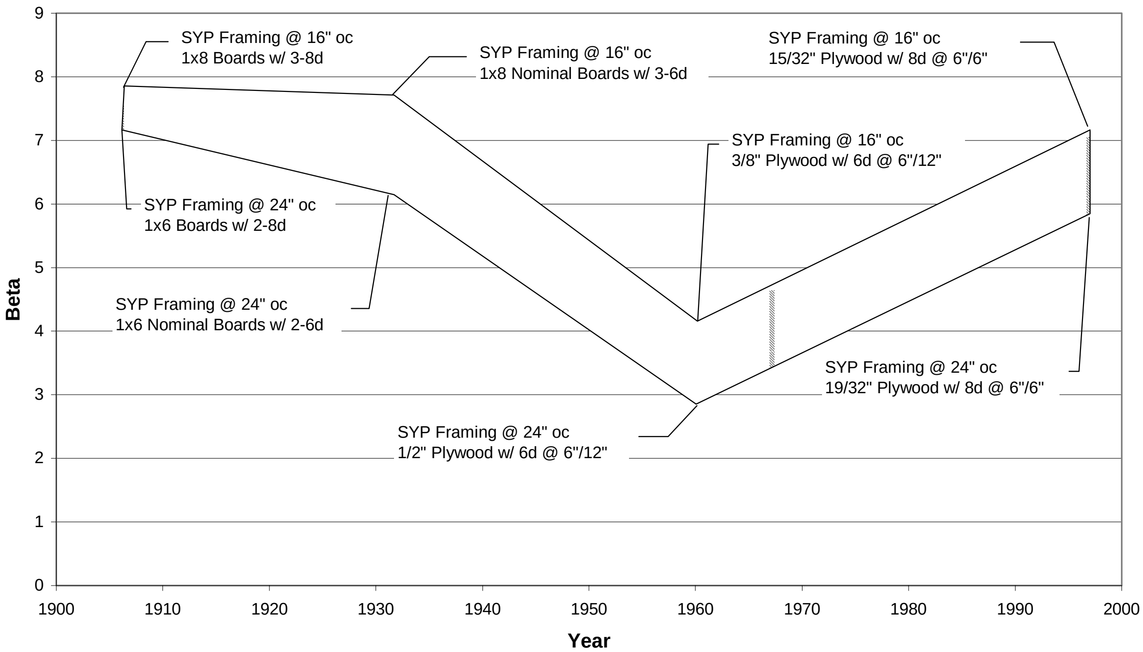


Figure 1.4B
Reliability of Roof Sheathing (W-D) in Coastal Region

A significant decrease in reliability was observed from 1931 and 1960. This is largely due to the increase in fastener tributary area when moving from planks to panels. Typical roof sheathing panels were attached using fasteners spaced six inches around the perimeter and 12 inches in the interior. Thus, the critical (i.e., interior) fastener had a relatively large tributary area. There was a slight increase in the length of penetration of the nail because of the sheathing panels were typically thinner than the earlier planks. However, the benefit realized was not sufficient enough to overcome the reduction in panel capacity resulting from the larger effective fastener tributary areas.

In the past decade, building codes have changed to require larger nails and tighter spacings in high wind regions such as South Florida. The changes were introduced as a result of losses from hurricanes Andrew and Hugo [NAHB, 1999]. The new nailing schedules for high wind regions have had the (intended) effect of significantly increasing the reliability of roof sheathing over levels associated with earlier (and less restrictive) fastener schedules, e.g., 6d nails spaced at 12 inches, more applicable to inland (non-hurricane) regions.

SUMMARY

The reliabilities of both floor joists and roof rafters have remained relatively constant from 1906 to the present. There has been some decline in the strength reliability, a result of both the reduction in conservatism used in assigning nominal design values and reductions in strength due to growth characteristics (i.e., less old-growth material, higher quality material available). There have been very few performance problems associated with the strength behavior of wood floor systems, so it is not surprising that the reliability bands have remained relatively constant over the years. The serviceability reliability for floor joists have also remained fairly constant, but have declined somewhat since the start of the century. The reliability bands are much narrower than those for strength, as a result of the greater consistency in normalized resistance (MOE) statistics across various materials.

The reliability of roof sheathing subject to wind uplift has decreased (dramatically in some cases) from 1906 to the present. This is largely a result of fastener effective tributary area. Note that two critical events (Hugo and Andrew) occurred in the late 1980s to early 1990s, a period during which the roof sheathing uplift reliability was relatively low (see the coastal case around 1960, Figure 1.4B). The upper bound of the reliability band has recovered much of what was lost when panels started replacing planks around 1960. This increase in reliability is the result of more stringent nailing schedules put in place in high wind regions after major hurricanes near the end of the century.

Section 2 - Evaluation of Reliability Associated with Current Design Provisions

INTRODUCTION, LOAD AND RESISTANCE STATISTICS

First-order reliability (FORM) methods were used to evaluate the reliability of No. 2 dimension lumber framing members designed according to the ASD [AF&PA, 1996b] and LRFD [AF&PA, 1996a] wood design provisions. FORM techniques are described elsewhere [Ang and Tang, 1975; Melchers, 1999; Rosowsky, 1997]. Additional details of this analysis may be found in [Rosowsky, 2001]. Results are presented for load combinations of dead plus occupancy live load (D+L), dead plus snow load (D+S), and dead plus counteracting wind load (W-D). Load statistics were taken from the literature [Ellingwood et al., 1980], modified in some cases based on other studies by the author, and are summarized in Table 2.1. The resistance statistics were obtained from the In-Grade Test Program (IGTP) results [Green and Evans, 1987]. Three species (Douglas-fir larch, Southern Pine, and Hem-fir) and sizes ranging from nominal 2 x 6 to 2 x 12 (all No.2 grade) were considered. The resistance statistics are shown in Table 2.2.

TABLE 2.1
LOAD STATISTICS

Load:	mean	COV	Distribution
Dead load, D	$1.05D_n$	0.10	Normal
Residential occupancy live load, L	$0.90L_n$	0.23	Type I
Snow load, S	$0.82S_n$	0.26	Type II
Wind load (inland), W	$0.47W_n$	0.37	Type I
Wind load (coastal/hurricane), W	$0.48W_n$	0.26	Type I

Notes:
[1] D_n, L_n, S_n, W_n = nominal (code-specified) dead, live, snow, wind load.
[2] Live load statistics shown for 50-year maximum (combined sustained and extraordinary) live load [Hendrickson et al., 1987; Philpot and Rosowsky, 1992].
[3] Snow load statistics shown for 50-year maximum “general site” in the northern tier of the United States [Ellingwood et al., 1980].
[4] Wind load (inland location) taken from NBS SP577 [Ellingwood et al., 1980].
[5] Nominal load values used as the basis for the statistics presented in NBS 577 correspond to values in ANSI A58.1-1972 (which generally agree with those in ANSI A58.1-1982). These values have changed in some cases, however, in the subsequent versions of ASCE 7.
[6] Wind load (coastal/hurricane region) based on results from event-based simulation [Rosowsky and Huang, 2000; Rosowsky, 2001].

TABLE 2.2
RESISTANCE STATISTICS
[Green and Evans, 1987]

N D S									
		Rn (psi)	R (psi), from IGTP				R/Rn		
	No.2	NDS	CDF	mean	std.dev	COV	mean	Std.dev.	COV
DF-L	2x6	900	ET-III	*7120*	*2421*	*0.340*	*7.91*	*2.69*	*0.340*
	2x8	900		6043	2354	0.390	6.71	2.62	0.390
	2x10	900		5322	2268	0.426	5.91	2.52	0.426
	2x12	900		*5048*	*2150*	*0.426*	*5.61*	*2.39*	*0.426*
SYP	2x6	1250		7076	2826	0.399	5.66	2.26	0.399
	2x8	1200		6306	2431	0.386	5.26	2.03	0.386
	2x10	1050		5916	1855	0.314	5.63	1.77	0.314
	2x12	975		*5611*	*1762*	*0.314*	*5.75*	*1.81*	*0.314*
HF	2x6	850		*5949*	*2088*	*0.351*	*7.00*	*2.46*	*0.351*
	2x8	850		5353	2045	0.382	6.30	2.41	0.382
	2x10	850		4559	1740	0.382	5.36	2.05	0.382
	2x12	850		*4324*	*1652*	*0.382*	*5.09*	*1.94*	*0.382*
L R F D									
		Rn (psi)	R (psi), from IGTP				R/Rn		
	No.2	LRFD	CDF	mean	std.dev	COV	mean	std.dev.	COV
DF-L	2x6	2220	ET-III	*7120*	*2421*	*0.340*	*3.21*	*1.09*	*0.340*
	2x8	2220		6043	2354	0.390	2.72	1.06	0.390
	2x10	2220		5322	2268	0.426	2.40	1.02	0.426
	2x12	2220		*5048*	*2150*	*0.426*	*2.27*	*0.97*	*0.426*
SYP	2x6	3180		7076	2826	0.399	2.23	0.89	0.399
	2x8	3050		6306	2431	0.386	2.07	0.80	0.386
	2x10	2670		5916	1855	0.314	2.22	0.69	0.314
	2x12	2480		*5611*	*1762*	*0.314*	*2.26*	*0.71*	*0.314*
HF	2x6	2160		*5949*	*2088*	*0.351*	*2.75*	*0.97*	*0.351*
	2x8	2160		5353	2045	0.382	2.48	0.95	0.382
	2x10	2160		4559	1740	0.382	2.11	0.81	0.382
	2x12	2160		*4324*	*1652*	*0.382*	*2.00*	*0.76*	*0.382*

Note: Values shown in italics were estimated using ASTM procedure (i.e., values were not available from IGTP data)

DEAD AND LIVE LOAD COMBINATION, D+L

A FORM analysis was conducted for flexural members (e.g., floor joists) subject to combined dead plus occupancy live (D+L) load. Load duration (DOL) effects were taken into account in an approximate way using the exponential damage rate model (EDRM) developed at the U.S. Forest Products Laboratory [Gerhards and Link, 1986; Ellingwood and Rosowsky, 1991] and assuming that damage leading to failure (by creep-rupture) is caused by a single pulse. This 'critical pulse' concept, as a reasonable approximation for cumulative damage in wood members subject to typical gravity load combinations, is described elsewhere [Bulleit and Rosowsky, 2001]. The critical pulse for the live load combination is assumed to correspond to an extraordinary live load (in combination with dead and sustained live load) having an assumed duration (Δt) of one week. Relevant load and resistance factors are taken from the NDS and LRFD. The limit state function for D+L (or D+S), including cumulative damage (DOL, critical pulse) is shown in Equation (2.1).

product of
adjustment factors

nominal
load ratio

$$g(\boldsymbol{x}) = A - B\left[\frac{\prod c_i \lambda \phi}{R / R_n}\left(\frac{1}{\gamma_D + \mu\gamma_X}\right)\left(\frac{D}{D_n} + \mu\frac{X}{X_n}\right)\right] - \ln \Delta t \tag{2.1}$$

load factors

critical pulse
duration

EDRM damage
model parameters

variable
load

The limit state given by equation (2.1) is formulated such that the failure probability (P_f) is the probability that $g(x) < 0$. The second-moment reliability index is given by $\beta = \Phi^{-1}(1\text{-}P_f)$ where

Φ^{-1} is the inverse standard normal cumulative distribution function.

The results from the FORM analyses of the flexural members subject to dead plus live load (D+L) are shown in Figure 2.1. The four graphs show results for (i) members designed according to the NDS, without consideration of DOL effects (i.e., “overload”); (ii) members designed according to the NDS, with DOL effects included (Δt = 7 days); (iii) members designed according to the NDS, with DOL effects included (Δt = 14 days); and (iv) members designed using LRFD, with DOL effects included (Δt = 7 days). Shown on each of these graphs are the average, minimum, and maximum reliabilities for a range of nominal load ratios $\mu = L_n/D_n$. Comparing the three NDS graphs illustrates the relative effect of including load duration (DOL) in the reliability analysis. Since the duration of the extraordinary live load pulse is short (on the order of 1-2 weeks typically assumed) the effect is relatively small, assuming the cumulative damage process can be reasonably approximated by the damage due to a single (critical) pulse. Comparing the second (NDS) graph and the fourth graph (LRFD) confirms the calibration exercise used in the development of the LRFD procedures for wood. It is worth noting, however, that these reliabilities for members in flexure (β = 2.0 to 2.6) are lower than those obtained in earlier calibration studies (β = 2.7 to 3.0), primarily due to differences in the strength statistics. Whereas the present study considers No. 2 grade dimension lumber of various species, previous studies [Ellingwood and Rosowsky, 1991; Hendrickson et al., 1987] used strength statistics corresponding to a particular select structural glulam data set.

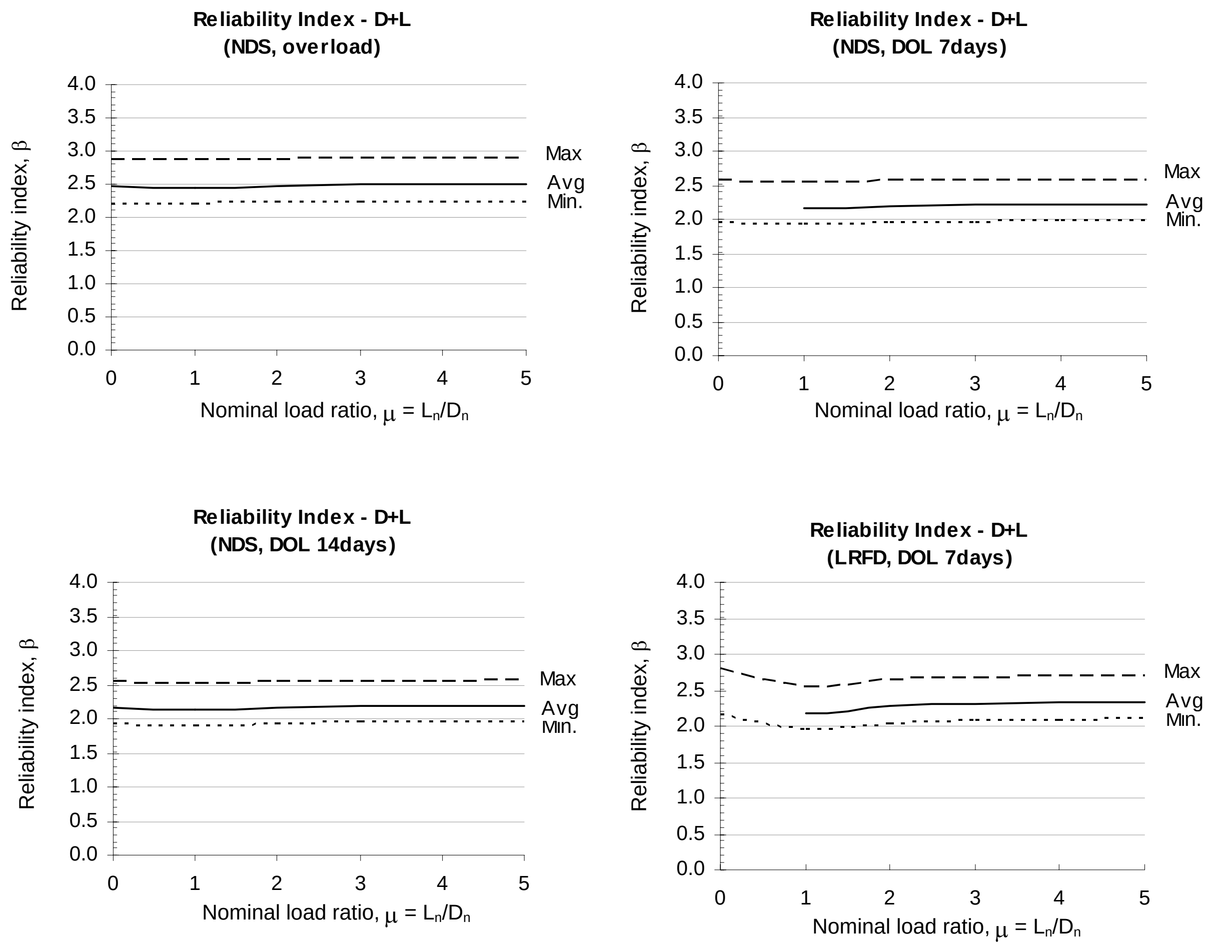


Figure 2.1
Results from Reliability Analysis of Flexural Members (D+L), ASD and LRFD

DEAD AND SNOW LOAD COMBINATION, D+S

A similar FORM analysis was performed for flexural members (e.g., rafters) subject to combined dead plus roof snow (D+S) load. Load duration effects were again taken into account using the critical pulse concept and the EDRM damage model. The critical pulse for the snow load combination is assumed to correspond to a roof snow duration (Δt) of two weeks. Relevant load and resistance factors are again taken from the ASD and LRFD wood design provisions. The limit state function is the same as the one shown previously (Eqn. 2.1).

The results from the FORM analyses of the flexural members subject to dead plus roof snow load (D+S) revealed similar ranges of reliabilities and similar trends to those observed for the dead plus live load case. These results are shown in Figure 2.2. The LRFD reliability indices (assuming a single critical pulse of duration Δt = 14 days) were somewhat higher than the corresponding ASD wood design values, reflecting one intention of the original calibration to increase the conservatism in design for snow loads [Ellingwood and Rosowsky, 1991]. As with the live load results discussed previously, it is noted that the reliabilities for the dead plus snow case are slightly lower than those obtained in the earlier calibration studies, primarily due to differences in the strength statistics. Figure 2.3 shows the effect of considering site-specific snow load statistics (see Table 2.1) on the range of reliabilities. As with the general site statistics, the site-specific statistics were obtained from eight sites considered in NBS SP577 [Ellingwood et al., 1980]. Figure 2.3 indicates that the effect of considering site-specific snow load statistics is to widen the range of reliabilities and reduces the minimum β value from about 2.2 to 2.0.

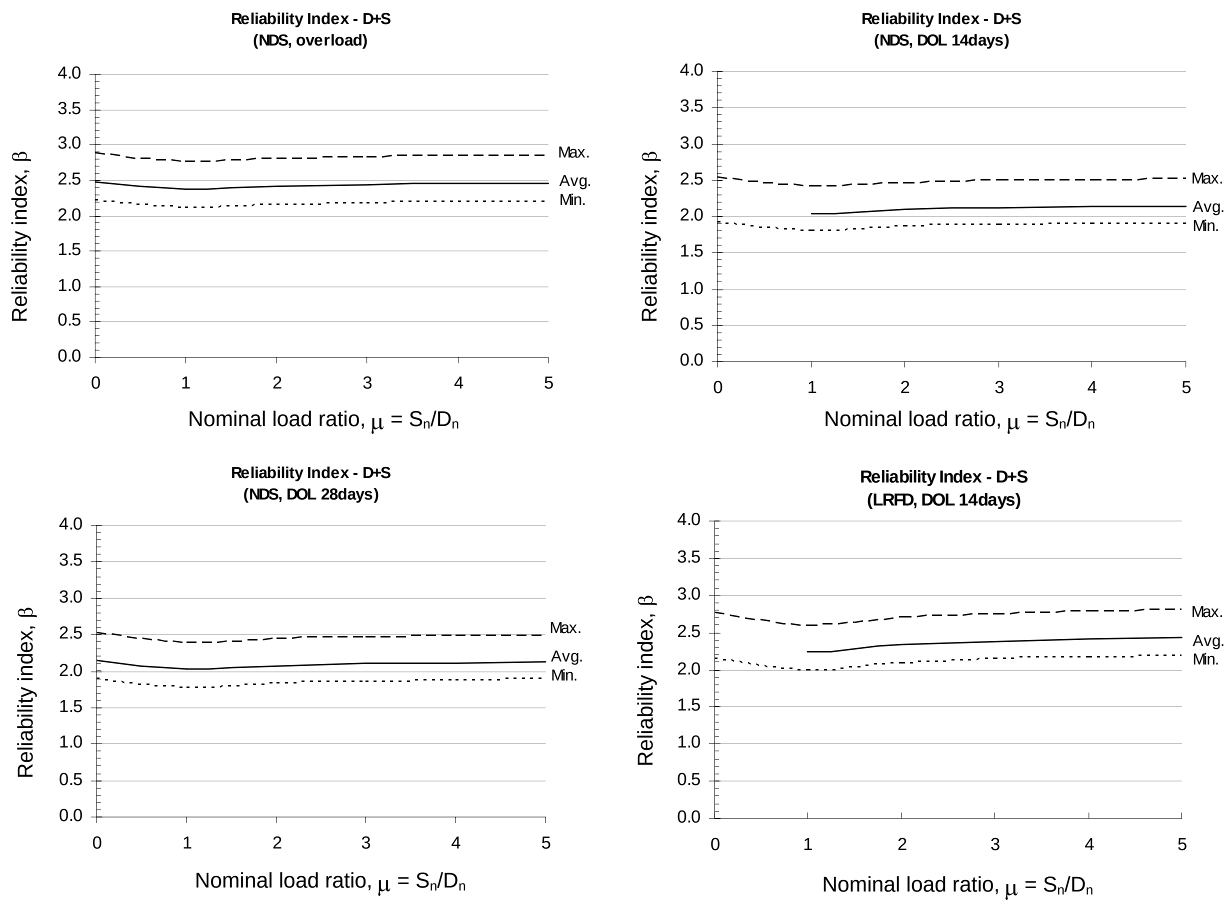


Figure 2.2
Results from Reliability Analysis of Flexural Members (D+S), ASD and LRFD

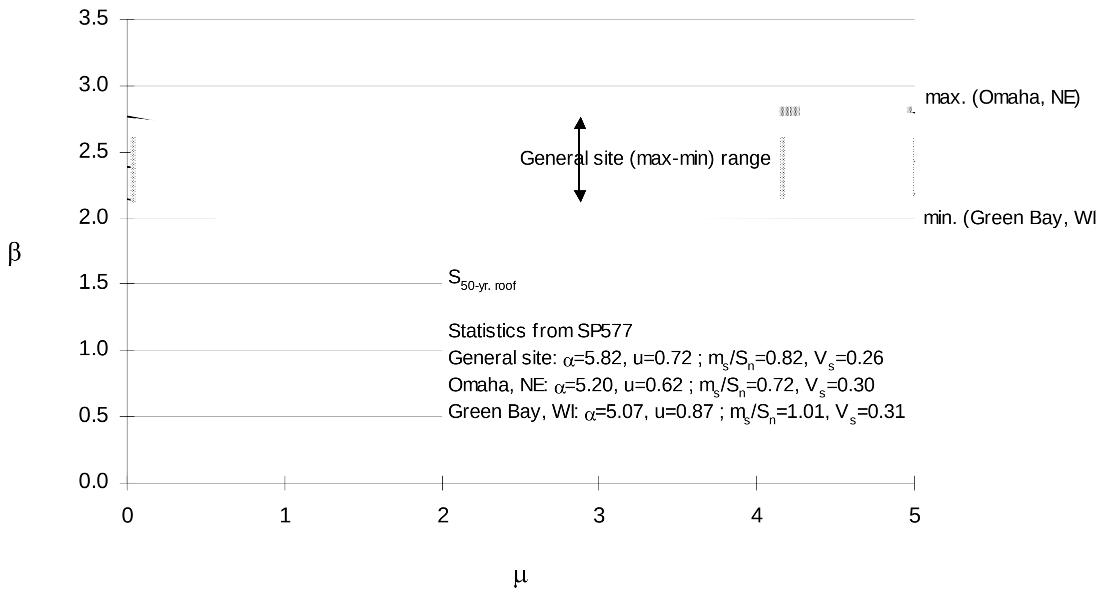
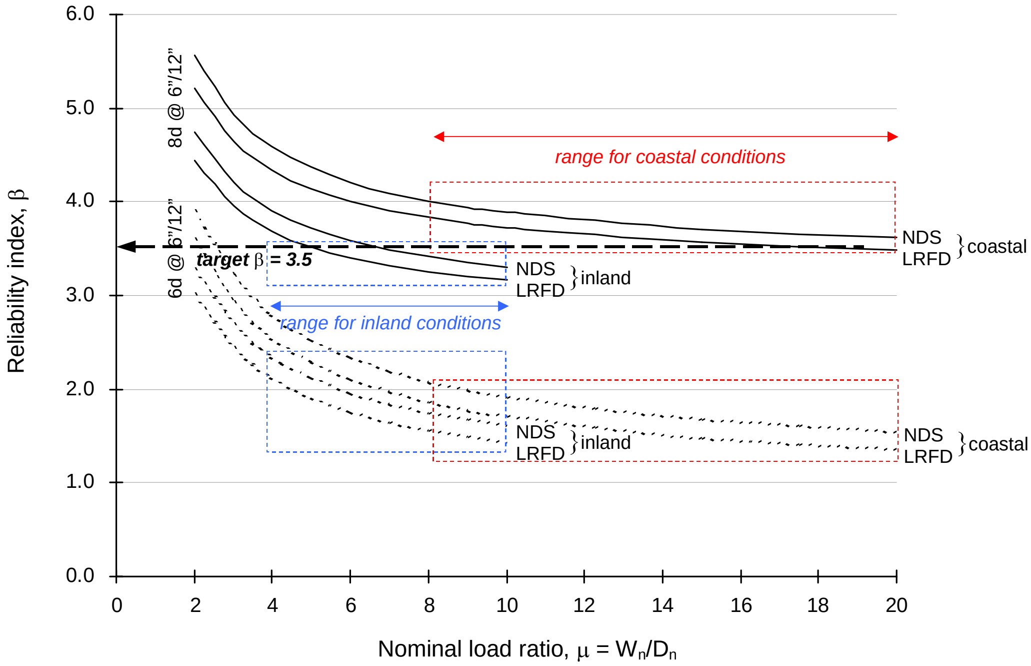


Figure 2.3
Effect of Considering Site-Specific Snow Load Statistics on Reliability of Flexural Members (D+S), LRFD, Δt = 14 days

DEAD AND WIND (UPLIFT) LOAD COMBINATION, W-D

A similar FORM analysis was performed for roof sheathing subject to uplift loads due to wind (negative pressures). The uplift load caused by wind is counteracted by the dead load and thus the load combination can be expressed as W-D. Because of the very short duration of design level wind loads, duration of load effects were not considered. Relevant load and resistance factors are again taken from the ASD and LRFD wood design provisions. The limit state function given by Eqn. 2.1 was modified to (i) remove consideration of load duration effects, and (ii) account for the load combination W-D. Failure is assumed to correspond to the failure of a single piece of roof sheathing located in a critical area of the roof.

The results from the FORM analyses of roof sheathing subject to wind uplift (W-D) are shown in Figure 2.4. Only two common nail sizes (6d and 8d), and a single nail spacing (6” around the edge of the panel and 12” on the interior of the panel), are considered. The panel uplift (ultimate) capacity statistics were obtained from testing conducted at Clemson University [Schiff et al., 1996]. The nominal panel uplift capacity was determined based on the withdrawal capacity of a single fastener located on the panel interior. (No specific procedures or recommendations exist for determining panel uplift capacities in the code.) Further information on assumptions and information sources is provided in the notes on Figure 2.4. The boxes indicated on the figure correspond to typical nominal load ratios (W_n/D_n) for wind loads on wood structures, for both inland and coastal sites.



Notes:

1. *Nominal wind loads calculated for gable end house, open exposure, using ASCE 7-98.*
2. *Statistics on inland wind load modified from values presented in NBS SP577 report (Ellingwood et al., 1980). Statistics for coastal (hurricane) wind load taken obtained from event-based analysis (Rosowsky et al., 2000).*
3. *Dead load taken as weight of sheathing, roof covering, and shingles.*
4. *Panel uplift capacity data obtained from tests conducted at Clemson University, SYP framing spaced 24 in. o.c.*
5. *Panel nominal uplift capacity determined using critical fastener tributary area (interior or panel) of 2 sq. ft., assuming 2 in. nail penetration into main member, and specific gravity G = 0.5 (framing member).*

Figure 2.4
Results from Reliability Analysis of Roof Sheathing (W-D), ASD and LRFD, Coastal and Inland Locations
(6d and 8d nails, 6"/12" spacing)

SUMMARY

Based on the results from this (preliminary) analysis, and assuming the current level of performance of these structural members and systems is satisfactory, a target reliability range of β=2.0 to 2.7 is associated with floor and roof members in flexure, while a target reliability of β=1.8 to 3.5 describes the typical range for roof sheathing attachment. Given that the performance of roof sheathing attachment has, on average, been acceptable in inland regions with the use of a minimum 6d@6"/12" nailing schedule requirement, arguments can be made in support of a design target reliability in the range of about β=2.0. Similarly, given that correctly installed roof sheathing using an 8d@6"/12" has performed reasonably well in moderately hazardous coastal regions, an argument can be made for use of a target reliability of closer to β=3.5. Based on the study of historical reliability (Section 1), reliabilities of β=3.0 or higher may better represent the outcome of "accepted practice" in residential construction.

Additional study and expert opinion will be required to develop more specific reliability targets for the purpose of residential design. Such reliability targets may vary depending on failure consequences. For example, life threatening structural failures of residential floor joists, roof rafters, and roof sheathing is fairly rare. However, economic losses associated with low reliability for roof sheathing attachment in hurricane-prone regions of the United States can be substantial. Clearly, however, more work is needed and additional lumber grades, load combinations, and limit states (e.g., tension, compression), for both members and systems (assemblies), should be considered.

Section 3 - Multistory Occupancy Live Load Analysis

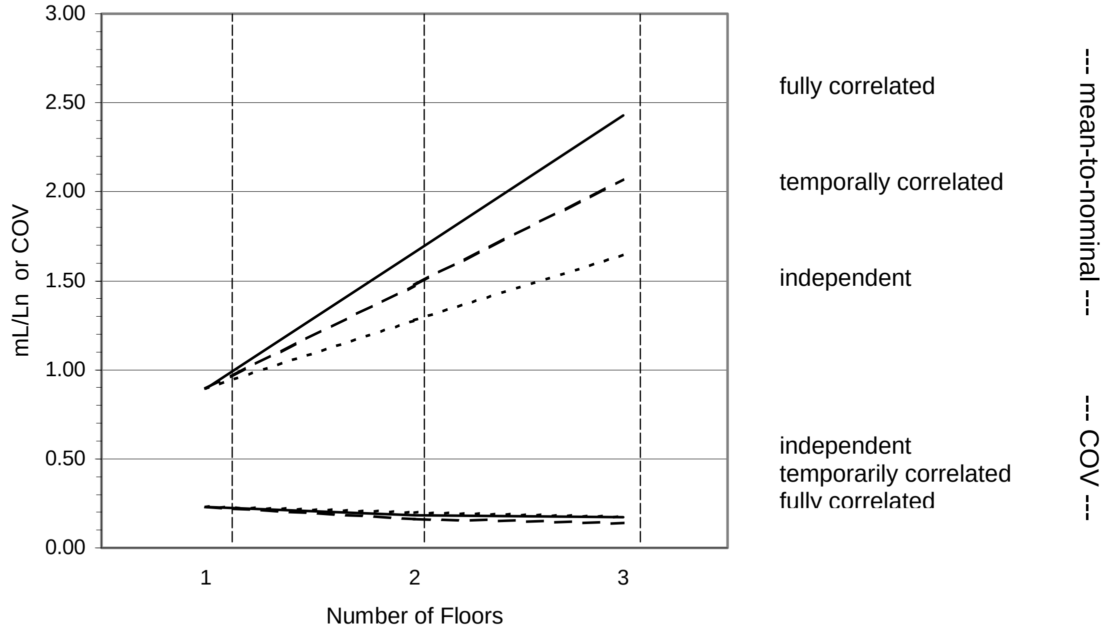
The design of lower-story columns and walls, and foundations below multi-story structures, requires an appropriate treatment of cumulative story loads. In the case of time-invariant self weights, this summation is trivial. However when the loading is time-dependent, as in the case of occupancy live loads, this summation may not be as immediately obvious. Simply adding the design live loads, for example, may result in an overly conservative assessment of the likely maximum load in the structure's lifetime. This assumption implies the maximum live loads on each floor occur at the same point in time. The nature of the event leading to the maximum live load may indeed suggest this temporal coincidence is reasonable, but the magnitudes of the loads may still be statistically independent. In some cases, it may be more reasonable to expect that the maximum is reached on one floor while the loads on the other floors are at their arbitrary point-in-time values. Obviously the number of permutations increases with the number of floors. In an office building having many floors, the assumption of independence of the floor loads seems reasonable; that is, the nature of the loading is such that it is unlikely multiple floors would see their design live load at the same time. For residential structures, however, which typically do not exceed two or three stories in height, this load coincidence issue may be relevant.

This section describes one approach to developing load coincidence factors for multi-story design live loads, focusing specifically on residential occupancies. This is an uncoupled approach in which only the probabilistic nature of the load processes are considered. An alternative approach might be to perform a fully coupled reliability analysis in which both the loads and resistances are treated as random variables. The two approaches would be expected to yield similar load combination (coincidence) factors suitable for design.

Monte Carlo simulation was used to investigate the statistics of a 50-year maximum combined live load process (consisting of sustained and extraordinary pulse processes) for multi-story residential structures. Statistics for the individual pulse processes (arrival rates and magnitudes of the sustained and extraordinary live load components) were taken from previous studies [Hendrickson et al., 1987; Philpot and Rosowsky, 1992]. The specific statistics selected were for a 200-400 ft^2 tributary area, representative of an effective floor area in a residential structure. Thus, the statistics are presumed to describe a residential occupancy live load. The combination of pulse process statistics (arrival rates and magnitudes) yields a 50-year maximum combined live load (for a single story) having a mean-to-nominal load on the order of 1.0 and a COV on the order of 0.20 to 0.25.

Three cases are considered, defined as *fully correlated*, *temporally correlated*, and *independent*. The first case is a worst-case scenario, and assumes the extraordinary live loads on each floor occur at the same time *and* have the same magnitude. The second case assumes the extraordinary live load on each floor occurs at the same time, but the magnitudes are statistically independent. The third case assumes both the times of occurrence *and* the magnitudes of the extraordinary live load on each floor are statistically independent. (Note that in all cases, the sustained live loads on the different floors are independent.) As suggested, the first case (*fully correlated*) may be viewed as a worst-case scenario, and is probably unlikely. The second case (*temporally correlated*) may be realistic in the case of large gatherings such as parties, for example. The last case (*independent*) is the least conservative, but may be the most realistic for typical residential structures. Others may have their own opinions on the relative merits (or suitability) of each of these cases. Figure 3.1 shows the resulting combined (sustained plus extraordinary) 50-year live load statistics obtained by simulation for one particular tributary area and corresponding set of statistics. Results are shown for the three cases considered (fully correlated, temporally correlated, and independent).

In the recommended multistory live load combination equation in the second draft of *Structural Design Loads for One- and Two-Family Dwellings* [HUD, 2001], a load coincidence factor of 0.5 was suggested. While this seems reasonable for the *independent* case, it appears that somewhat larger factors may be needed for the *temporally correlated* (0.6-0.7) and *fully correlated* (0.8-1.0) cases, based simply on a comparison of mean-to-nominal combined live load values (see Table 3.1 and Figure 3.1). It seems the middle case may be the best compromise between “typical” and “reasonably envisioned” conditions, and thus a combination factor of 0.7 might be recommended. On this basis, the multi-story live load factor in the HUD document (HUD, 2001) was changed to 0.7. An argument could be made to reduce these factors slightly to account for the reduction in COV with increasing number of stories. For example, the COV in the temporally correlated Case 2 reduces from 0.23 to 0.14 when increasing from one to three stories. Assuming a Type I extreme value (Gumbel) distribution for the 50-year maximum combined live load, one could compare 95^{th}-percentile values to obtain a combination factor which takes this change in variability into account. These are shown in Table 3.2. Based on these results, and accepting a calibration to the 95^{th}-percentile value of the 50-year maximum combined live load, combination factors of 0.4, 0.6, and 0.9 would be recommended for the independent, temporally correlated, and fully correlated cases, respectively. This effect should be considered in future editions of the HUD document [HUD, 2001] and a similar study should be done for use with LRFD factored loads.



Notes:

fully correlated – extraordinary live loads occur at the same time, and have the same magnitude, on each floor.

temporally correlated – extraordinary live loads occur at the same time on each floor, but have different magnitudes.

independent – both occurrences and magnitudes are statistically independent on each floor.

Figure 3.1
Statistics of Maximum Combined (L_s+L_e) Residential Occupancy Live Load T_{ref}=50 yrs
Live Load Statistics form Hendrickson et al., 1987 (A_T=400 ft^2)

TABLE 3.1
COMBINED LIVE LOAD STATISTICS

Cases		mean-to-nominal			COV		
		1 story	2 stories	3 stories	1 story	2 stories	3 stories
1	Hendrickson et al., 1987 (200 ft^2), m_{Ls}/L_n = 0.23, V_{Ls} = 0.85, m_{Le}/L_n = 0.16, V_{Le} = 1.02						
	Fully correlated	1.06	2.01	2.96	0.26	0.22	0.21
	Temporally correlated	1.06	1.70	2.26	0.26	0.21	0.19
	Independent	1.06	1.51	1.89	0.26	0.21	0.18
2	Hendrickson et al., 1987 (400 ft^2), m_{Ls}/L_n = 0.23, V_{Ls} = 0.68, m_{Le}/L_n = 0.19, V_{Le} = 0.67						
	Fully correlated	0.90	1.66	2.43	0.23	0.18	0.17
	Temporally correlated	0.90	1.48	2.07	0.23	0.16	0.14
	Independent	0.90	1.28	1.65	0.23	0.20	0.17
3	Philpot and Rosowsky, 1992 (200 ft^2), m_{Ls}/L_n = 0.24, V_{Ls} = 0.90, m_{Le}/L_n = 0.16, V_{Le} = 0.84						
	Fully correlated	1.01	1.91	2.78	0.27	0.21	0.19
	Temporally correlated	1.01	1.64	2.21	0.27	0.21	0.19
	Independent	1.01	1.49	1.91	0.27	0.23	0.20
4	Philpot and Rosowsky, 1992 (400 ft^2), m_{Ls}/L_n = 0.24, V_{Ls} = 0.70, m_{Le}/L_n = 0.19, V_{Le} = 0.78						
	Fully correlated	0.99	1.93	2.84	0.22	0.19	0.18
	Temporally correlated	0.99	1.64	2.19	0.22	0.19	0.16
	Independent	0.99	1.46	1.83	0.22	0.19	0.17

Notes:
Sustained live load (L_s): ν = 0.0125/year (Poisson)
Extraordinary live load (L_e): ν = 1/year (Poisson), τ = 1 week

TABLE 3.2
LIVE LOAD COMBINATION FACTORS (κ)

Cases		1 story		2 stories		3 stories	
		95% value	κ	95% value	κ	95% value	κ
1	Hendrickson et al., 1987 (200 ft^2), m_{Ls}/L_n = 0.23, V_{Ls} = 0.85, m_{Le}/L_n = 0.16, V_{Le} = 1.02						
	Fully correlated	1.58	1.00	2.84	0.80	4.14	0.81
	Temporally correlated	1.58	1.00	2.38	0.51	3.07	0.47
	Independent	1.58	1.00	2.10	0.33	2.54	0.30
2	Hendrickson et al., 1987 (400 ft^2), m_{Ls}/L_n = 0.23, V_{Ls} = 0.68, m_{Le}/L_n = 0.19, V_{Le} = 0.67						
	Fully correlated	1.28	1.00	2.23	0.74	3.21	0.75
	Temporally correlated	1.28	1.00	1.92	0.50	2.60	0.52
	Independent	1.28	1.00	1.75	0.37	2.18	0.35
3	Philpot and Rosowsky, 1992 (200 ft^2), m_{Ls}/L_n = 0.24, V_{Ls} = 0.90, m_{Le}/L_n = 0.16, V_{Le} = 0.84						
	Fully correlated	1.53	1.00	2.65	0.74	3.76	0.73
	Temporally correlated	1.53	1.00	2.30	0.50	3.01	0.48
	Independent	1.53	1.00	2.14	0.40	2.62	0.36
4	Philpot and Rosowsky, 1992 (400 ft^2), m_{Ls}/L_n = 0.24, V_{Ls} = 0.70, m_{Le}/L_n = 0.19, V_{Le} = 0.78						
	Fully correlated	1.40	1.00	2.61	0.87	3.81	0.86
	Temporally correlated	1.40	1.00	2.22	0.58	2.85	0.52
	Independent	1.40	1.00	1.99	0.42	2.41	0.36

Notes:
two-story: $L_{n,1} + \kappa L_{n,2}$
three story: $L_{n,1} + \kappa(L_{n,2} + L_{n,3})$

Summary

This report describes a series of studies conducted in support of developing and improving probability-based design procedures for residential construction applications. The focus of this study was on wood-frame structures built using nominal 2-by framing lumber, structural sheathing, and nail fasteners—the most common materials used today. While by no means a complete analysis of all commonly used materials and methods of construction, an effort has been made to consider a representative range of conditions so that many of the results can be generalized. It is emphasized that this report represents an initial effort and is exploratory rather than conclusive in many instances. Clearly, much more work is needed.

The results in this report can be used (along with other information) as the basis for calibration of new design provisions or modifications to existing design provisions (e.g., partial factors). However, more work is needed and additional lumber grades, load combinations, and limit states (e.g., tension, compression), for both members and systems (assemblies), must be considered before a full set of target reliabilities can be recommended.

With continued attention being paid to the development of risk-consistent design procedures, particularly in the light of recent natural disasters, and on-going activities to revise and update the nominal loads in ASCE 7 (e.g.), the time is certainly right for a re-evaluation of partial safety factors in standards (such as the LRFD Standard for Engineered Wood Construction). Work is also needed to more accurately define loads on residential structures (including regional variations, load path issues, localized load effects, and load combination/coincidence issues). It is hoped that efforts will continue and that the move toward probability-based design of residential structures, particularly those located in high hazard regions, can be fully realized.

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