AN ABSTRACT OF THE THESIS OF

Tarimala S. Srikanth for the degree of Doctor of Philosophy in Forest Products presented on May 28, 1992.

Title: Structural Reliability of Light-Frame Wood Systems with Composite Action and Load Sharing

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Abstract Approved: ______________________
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In a majority of light-frame wood buildings, studs and joists are the basic structural components in walls and floors. The walls act as bending and compressive panels and transmit lateral wind and gravity loads to the foundation. Joists are used in floor systems, and together with the sheathing member act as an orthotropic plate in supporting live and dead loads. Mechanical fasteners form most of the joints and provide semi-rigid connection between the framing members and the sheathing. Current design methods do not incorporate the system behavior within the wall or floor, that is, both the composite action of the framing with the sheathing and load sharing between the framing members. Yet, system strength and stiffness rely on structural interaction.

This study of light-frame wall and floor systems introduces a probability-based evaluation, including the interaction of components as well as the nonlinearities

of materials and connectors. Load sharing in the wall and floor systems was modeled by a series of elastic springs with the sheathing as a distributor beam. Stochastic distributions were used to represent certain properties and loadings.

Reliability analyses of wall systems under bending and compressive loads were conducted. It was found that composite action and load sharing contribute to a reduction in failure probability. Reliability studies verified the hypothesis that wall systems are highly reliable and can sustain loads exceeding those expected under 50-year and 100-year wind loads.

Reliability levels were examined for floor systems in a bending limit state with 16-in. and 24-in. joist spacing, under a Type I extreme value distribution for live load. Floor system capacity was sensitive to the variability of the lumber modulus of rupture. The degree of load sharing was grade dependent; No.1 joists exhibited higher load sharing than No.2 joists as the coefficient of variation in strength of No.2 joists was much higher than No.1 joists. Based on these results, the 15% increase in the allowable bending stress for repetitive light-frame members as specified by the National Design Specification for Wood Construction appears to be conservative.

STRUCTURAL RELIABILITY OF LIGHT-FRAME WOOD SYSTEMS WITH COMPOSITE ACTION AND LOAD SHARING

by

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A THESIS

submitted to

Oregon State University

in partial fulfillment of
the requirements for the
degree of

Doctor of Philosophy

Completed May 28, 1992

Commencement June 1993

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Date thesis is presented May 28, 1992

ACKNOWLEDGEMENTS

I wish to express my deep sense of gratitude and appreciation to my major professor, Dr. Robert Leichti, for his continuous guidance, support, patience and encouragement through out the course of the study. Many thanks are due to the late Dr. Anton Polensek for his support and guidance.

Further acknowledgements go to my committee members Dr. Robert Ethington, Dr. Thomas Miller, Dr. John Peterson and Dr. Ezra Tice for their encouragement and help in completing my graduate program.

I would like to express my deep sense of love and affection to my mother, Smt. Vasantha, and father, Sri. Srinivasacharlu, for their encouragement and support.

Finally, a special note of appreciation to my wife, Aruna, for her support, patience and help to accomplish this task.

This research was funded by the Forest Research Laboratory, Oregon State University, and the National Science Foundation, Grant No. MSM-8614237.

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I. INTRODUCTION

For many years, wood has played an important role as a structural material in buildings. Wood framing members are the most common type of structural members used in residential and commercial buildings (Polensek 1976).

Light-frame wood dwellings are complex structural systems made up of interacting components such as walls, floors, roofs and foundations. Studs and joists, spaced at 16 in. or 24 in. are the structural components of walls and floors. A typical wall has a structural panel such as plywood as the exterior sheathing and gypsum wallboard on the interior. Unlike walls, floors usually have a structural panel attached to only one surface of the joists. Studs normally consist of 2×4 or 2×6-in. lumber and joists consist of 2×8 or 2×10-in. lumber of various grades and species, differing in strength and stiffness.

Walls, floors and roofs in light-frame structures can be visualized as building systems consisting of repetitive members acting in parallel. The framing members transmit wind, snow, earthquake, live and gravity loads to the lower stories and the foundation. Walls and floors act either as diaphragms or bending-compression panels when transmitting loads. The behaviors of the wall and floor substructures are not dependent on single-member performance because of repetitive framing and the interaction between the framing and sheathing.

The current design practice (NDS 1991) acknowledges the repetitive character of framing under bending loads. However, under axial loads, the repetitive character of the framing is ignored in structural analysis.

Composite Action

Composite action results when two or more materials contribute to the resistance of an external force. The framing member does not act alone as a beam when carrying the imposed loads. In a wall, the sheathings act with the framing member to form a composite I-beam, with the structural member acting as the web and the sheathings as the flanges. The floor has only one sheathing layer, so it acts as a series of T-beams.

The strength and stiffness of the composite section depends on the rigidity of the connection system between the layers (Kuenzi and Wilkinson 1971). In the case of light-frame construction, nails and staples are the primary structural fasteners transferring forces from the sheathing materials to the framing members (Polensek 1975). Because nailed joints are semi-rigid, relative movement (slip) can occur at the interface between the sheathing and framing. The presence of slip distinguishes the wall or a floor as a structural system with incomplete composite action. The degree to which the fasteners resist slip depends upon the load, fastener stiffness, and fastener spacing. The contribution of the covering materials to the overall strength and stiffness of the system is functionally dependent upon the slip behavior at the interface (Atherton et al 1980).

Nailed joints between the studs and sheathing members provide lateral support against buckling. If the sheathings are rigidly fastened (glued) to the framing members, the structural behavior of the system is fully composite, and I-beam properties with transformed cross sectional-areas can be used in their analysis. On the other hand, if there is no connection between the framing member and the sheathing, the two elements act independently when carrying loads. Between these two cases is the incomplete composite I-beam, involving the stiffness of the interlayer connectors, stiffness of the sheathing, the strength and stiffness of the framing member.

Load Sharing

Load sharing is a phenomenon of redundant structural systems wherein the stiffer members support a larger portion of the load than the less stiff members. The sheathing material acts as a load-distributing member bridging over the structural members, each of which has a different stiffness. Load sharing among framing members of different strength and stiffness and composite action between the framing members and the coverings are the major factors, which add strength and stiffness to the structural subsystem.

Probability-Based Design

Probability-based design provides a logical method of achieving a defined level of reliability for all structural members and can also be used as a tool for updating standards in a rational manner, ultimately resulting in design improvements. In this design method, probabilistic methods are used for the development of resistance and load factors to provide overall structural safety against variations in load and resistance. Also through probability-based design, two facets of design can be investigated (Ellingwood et al 1980): (1) for a given design, the reliability can be established dependent upon the consequences of failure, and (2) for a fixed reliability, partial safety factors can be developed.

In the past few decades, probability-based design has received considerable attention in North America.

Since 1980's, various forms of probability-based design for steel (AISC 1986) and concrete structures (ACI-318 1989) have been developed in the United States, and for wood systems in Canada (CSA 1989).

The timber design community in the United States is moving towards a probabilistic design method. A task committee on Load and Resistance Factor Design for Engineered Wood Construction completed the draft of a probability-based design format for wood systems where a single member governs performance (Murphy 1988). However, the design format does not include interaction of structural components and load sharing. As the performance of light-frame wood subsystems depend upon the strength and stiffness of the framing members, the sheathings, the intercomponent connections and the material nonlinearity, a probability-based design methodology also should include these properties.

Probability-based, limit-state design methodology is developed by combining existing design techniques of structural analysis and probability-based concepts. Load and resistance factor design (LRFD) is one such particular form of probability-based limit states design. Typically this can be described by a design criterion of form (Ellingwood et al 1980),

$$\varphi R_n \geq \sum_{i=1}^{n} \gamma_i Q_i \qquad (1)$$

where factored resistance ≥ effect of factored loads, and R_n = nominal resistance of the member corresponding to a given limit state, φ = resistance factor and reflects the degree of uncertainty associated with nominal resistance, γ_i = load factors which account for the degree of uncertainty in loading, Q_i = the nominal loads, and n = number of limit states considered.

Rationale and Significance

Light-frame wood dwellings have a long history of good performance. However, the forest resource, which provides sawn lumber for construction, is constantly changing. As these changes occur, the quality of lumber varies and the wider and longer sections become more scarce and expensive (Leichti 1986). With increasing demands placed on the timber resource, an improvement in the design methods of building systems serves as a positive step toward the efficient use of the timber resource.

Conventional design methods are based on the assumption that single members govern performance and design of light-frame wood systems. However engineering analysis and testing have shown that composite action and load sharing occur in light-frame wood systems. Composite action develops when the sheathing, which is attached to the studs or joists by nails, works in concert with the studs or joists such that the system behaves as a series of joined I-beams or T-beams rather than as simple, individual rectangular members. Load sharing is a function of lumber stiffness and involves the lateral distribution of loads via the sheathing.

Light-frame wood members are traditionally designed by the allowable stress design methodology (Breyer 1988). The starting point for the development of the allowable stress was the 5% exclusion limit, modified to account for moisture content, load duration and other applicable factors. The 5% exclusion limit and the combined effect of the modification factors yields an acceptable level of reliability for typical engineered wood systems.

Though modification factors implicitly allow for variations that occur in resistance and loading variables, the allowable stress design fails to provide uniform safety for all members in the structure. Furthermore, the deterministic approach assumes that the design variables are explicitly stated. However, in the real world structural problems are non-deterministic in character. Some uncertainty always is involved in the design of structural wood systems. Being a biological product, lumber exhibits more variability in strength and stiffness than do steel and concrete. The loads borne by the structural member and the resistance offered by the structural member are random in nature, hence the behavior of a system can be predicted with an associated probability. Although the deterministic approach has served for a number of years and yielded structures with adequate performance, the probability-based design or LRFD has some advantages to offer.

Limit state design is a more refined version of the allowable-stress design and accommodates the variations occurring in load and resistance in a rational manner. A structure is said to have reached a limit state when it is unsafe or ceases to perform its intended function (Borges 1976). The ultimate limit state corresponds to the maximum load carrying capacity and may be reached through overload causing crushing, section failure, rupture or instability. The serviceability limit state is an expression of functional requirements and may be governed by deflection or vibration criteria.

Objectives

The objective of this research is to ultimately develop a probability-based design methodology that includes the complexities of real wall and floor systems. The research described in this dissertation provides a method for evaluating system reliability for light-frame walls and floors and includes the effects of composite action and load sharing. The system reliability can be determined for a variety of load combinations and a target reliability β_0 can be chosen. This target reliability β_0 is a potential factor to assure uniform reliability across a given range of applications.

The reliability analyses presented here include the system behavior induced by load sharing and composite action. Load sharing in the wall and floor systems is modeled by a mechanical analog where in the joists or studs are elastic springs and the sheathing is a distributing beam. Composite action is taken into account by considering the interaction of the components as well as the nonlinearities of materials and connectors.

The following objectives describe the development of probability-based analysis for light-framed wall and floor systems:

1. Develop a structural analysis procedure that accounts for the incomplete composite action between the framing members and the wall or floor coverings (chapter III).

2. Develop a mathematical model that takes into account the load sharing developed in repetitive, parallel wall and floor substructures. Load-sharing factors are evaluated using a Monte Carlo simulation technique (chapter IV).
3. Develop a data base for strength and stiffness properties of the framing members to be used in reliability analysis (chapter III).
4. Using a first-order second-moment reliability method, develop a computer program that evaluates the reliability levels and failure probabilities of light-frame wall and floor systems representative of current construction (chapter VI).
5. Perform sensitivity studies to identify the dominant variables affecting the performance of wall and floor systems (chapter VI).

II. LITERATURE REVIEW

Structural Reliability

Much of the early work in structural reliability was done for reinforced concrete structures. Concepts of structural reliability and risk of failure were introduced as early as 1961 by Freudenthal for reinforced concrete structures. Benjamin (1968) discussed some of the advantages of probabilistic design compared to that of the conventional deterministic design and also introduced the decision-tree method for building design situations.

Allen (1970) conducted a probabilistic study of reinforced concrete beams in bending and suggested that there was a significant probability that a section designed as under-reinforced could become over-reinforced. The author also showed that the variability of ultimate moment expected in practice increases either when the member is thin or when the percentage of steel is high. Rowe (1970) suggested that the treatment of structural safety by a limit-state method is practical and should be a basic format for future codes of practice. Nowak (1979) studied the effect of human error on structural safety and suggested that the gross errors can be controlled by inspection and checking, proof loading, adjustment of safety factors and error proof realization of the structure.

The importance of structural safety and the advantages offered by reliability-based design led the construction industry to probability-based design methods for steel (AISC 1986) and concrete structures (ACI-318 1989).

In the past few decades, many papers on probability-based design of wood structures were published (for example, Ellingwood 1981; Foschi 1984; Bulleit 1986; Gromala and Sharp 1988; Foschi et al 1989; Bulleit 1991; Bulleit and Yates 1991).

The important applications of reliability analysis to wood members were discussed by Suddarth et al (1978). Zahn (1977) suggested reliability-based design procedures for wood structures and used a floor system as a design example treating both load and resistance as random variables. Sexsmith and Fox (1977) used a safety index to express reliability of glued-laminated beams under flexural loads. The safety index varied from 3.2 to 2.0, and they concluded that a safety index of about 2.5 appeared to be representative of current design practice.

Ellingwood (1981) established reliability benchmarks for wood design and for developing probability-based limit states design criteria. A reliability analysis for glulam beams was conducted and reliability indices were determined for different ratios of snow to dead loads and live to dead loads. He also reported that the reliability index was sensitive to the assumed distribution of resistance for small ratios of snow to dead loads and live to dead loads.

Bulleit (1986) presented an approximate reliability model for wood structural systems; the model required the probability of first member failure as an input, which had to be determined from a theoretical analysis. Later, Bulleit and Vacca (1988) developed a Markov model for wood structural systems. In doing so, the authors utilized a probability transition matrix while assuming that system failure occurs when two adjacent members fail.

Leichti and Tang (1989) studied the effect of creep on the reliability of sawn lumber and wood-composite I-beams. The authors used a Burger-body (mechanical analog) to model creep of wood, which was then used as the failure function in a second-moment reliability analysis. The authors concluded that the reliabilities change over time.

Walford (1989) discussed the conversion of the New Zealand timber design code to a limit-state design format and its advantages. Leicester (1990) discussed the timber engineering limit-states design codes for Australia and stressed the format and strategies used to develop these codes. Goodman (1990) discussed the update and status of United States progress on an LRFD specification for wood construction as it applies to single members. The authors established resistance factors in bending, tension, shear, compression, connection and stability to be used in an LRFD design. Time-effect factors using damage accumulation models were also developed.

However, most of the work reported on structural reliability concentrates on single member performance and the importance of composite action and load sharing together in wall and floor systems were seldom addressed.

Composite Action and Load Sharing in Wood Systems

The strength and stiffness added by the composite action produces substructures that perform in concert rather than as individual members. Composite action in wood systems depends upon the type of fasteners, which resist slip, the lateral load/slip stiffness, the spacing between the fasteners and finally the characteristics of the main and side members. When two identical beams are stacked one above the other, slip occurs along their contact interface on loading. When this slip is eliminated, i.e when beams are rigidly glued, stresses are redistributed. Kuenzi and Wilkinson (1971) presented a theoretical means of determining the deflections and stresses for composite wood beams having interlayer fastenings of finite rigidity and further verified their theory experimentally. This theoretical study made possible the rational design of members with incomplete composite action.

Amana and Booth (1967) investigated stressed-skin components of timber and plywood in the use of prefabricated building units. In the theoretical analysis, the authors considered plywood to be an orthotropic plate in a state of plane stress. The authors described the concept of effective breadth, stiffening factor, and the importance of slip between the plywood and timber. Basically, the stiffening factor related the deflection profile along a partially composite beam to the individual beam. It was a function of the effective width of the sheathing, stiffness of the interlayer connectors and the type of loading. They also presented theoretical solutions for the analysis and design of nailed or glued plywood stressed-skin components. However, their work did not involve any reliability study.

The behavior of wood structures with composite stiffness has been addressed by a number of researchers. Goodman and Popov (1968) provided analytical formulations which are the theoretical basis for several computer models today. Polensek (1976), based on the theoretical formulation of Amana and Booth (1967), developed a finite-element computer program FINWALL, which analyses the performance of wood-stud walls.

McCutcheon et al (1981) studied the effects of joist variability on the deflection performance of floor systems. The authors concluded that composite action reduces deflection and the effects of joist variability are minimized in assembled floor systems. McCutcheon (1986) presented a simple procedure to evaluate the composite stiffness of T-beam or I-beam, and the theoretical model agreed closely with test results.

Polensek and Gromala (1984) studied the structural performance of wood-stud walls in bending. By including statistical variations that occur in a wall system, the authors developed probability distributions that could be used in a reliability-based analysis. Polensek and Kazic (1991) examined the reliability of wall systems in bending and compression loads. The authors were able to show analytically that wall systems are highly reliable.

McCutcheon (1984) presented a beam-spring model to evaluate the performance of uniformly loaded wood floors. The proposed model accounts for composite action and the variability of joist stiffness. This technique was adopted in the load sharing analysis of wall and floor systems.

With respect to axial loads, current design methods (NDS 1991) do not acknowledge the composite action of the framing and sheathing nor load sharing among studs. Load sharing by neighboring studs and composite action developed when sheathing is attached to the studs are the major factors that contribute to the added strength and stiffness. However, the allowable bending stress (F_b) of repetitive members may be increased 15% in light-frame construction under certain conditions (spaced no more than 24 in. on center, are not less than three in number and are joined by floor, roof or other load-distributing element) (NDS 1991).

Kloot and Schuster (1963) described a procedure for estimating the load distributions in wood floors subjected to concentrated loads. Zahn (1970) developed a theory for the strength of multiple-member systems based on the weakest-link and the brittlest-link structures. The author examined four populations of wood beams and mentions a load sharing increase of about 12%. DeBonis (1980) presented a mathematical load-sharing model for three and five-member wood joist floor systems. The author established simulated distributions of the ultimate load a system can sustain. The results of the theoretical model compared well to the real load sharing systems tested to failure.

Cramer and Wolfe (1989) developed a three-dimensional load-distribution computer model for light-frame wood roof assemblies. The load-distribution model compared favorably with full-scale tests, and the authors concluded that the model gave a good prediction of load-distribution when extreme variations in truss stiffness occur. LaFave and Itani (1992) developed a load-distribution model for wood roof systems. The authors showed that the percentage of load distributed through load sharing was higher in a limber truss than in stiffer truss.

Foschi and Folz (1989) evaluated the reliability levels of light-frame wood structural systems using a finite-strip method. The authors derived a system modification factor, which includes load sharing in multiple-member systems. Rosowsky and Ellingwood (1992) addressed strength and serviceability limit-states interactions. The authors performed reliability analyses of a floor system in serviceability limit state and concluded that reliability was sensitive to the span of the joist, but was not dependent upon number of joists, the sheathing or fastener properties. Rosowsky and Ellingwood (1992) included load sharing and duration-of-load effects in analyzing system reliability of light-frame wood construction. The authors concluded that the

15% increase for members used repetitively in light-frame systems is conservative and that a value of 1.25 might be more appropriate.

Load Duration

A characteristic property of wood is that higher stresses can be sustained if applied for shorter periods of time. In 1951, Wood gave a mathematical expression for the relation of bending strength of wood to load-duration. The load-duration relationship was approximated by a hyperbolic curve. The subject of load-duration has been investigated by many scientists. More recently the effects of load-duration on wood-based composites is being examined.

Treatment of load-duration effects had to be incorporated into limit states design. Foschi et al (1989) used a damage accumulation model to interpret the duration of load effects and evaluate the reliability index in bending for a service life of 30 years. The authors concluded that a common duration-of-load factor can be applied for different species and qualities. Gromala et al (1990) derived a time-effect factor through a stochastic analysis of material properties and load distribution. This time effect factor is used to preserve a given β over a structural life time.

Ellingwood et al (1988) considered load duration effects due to dead and snow load, and presented methods to conduct reliability studies. Recently Ellingwood and Rosowsky (1991) studied the duration- of-load effects in LRFD for wood members. The authors defined two limit states, overload and progressive damage accumulation, to analyze the probability of failure. Structural loads were modeled as stochastic processes instead of random variables. Rosowsky and Ellingwood (1991) analyzed load sharing effects combined with duration-of-load in light-frame floor system. They developed system factors, which essentially took load sharing into account for use in working stress design and LRFD.

More recently, system effects and reliability were examined using an approach based on deflection, but duration-of-load effects for both strength and modulus of elasticity were not included (Rosowsky and Ellingwood 1992). Because the analysis were based on the modulus of elasticity and duration-of-load effect is unknown for modulus of elasticity, load duration was neglected.

Load-Slip Characteristics

Foschi and Bonac (1977) derived load-slip characteristics for connections with common nails. The nonlinear character of the load-slip relationship was modeled using a finite-element elasto-plastic analysis.

Polensek (1978) conducted theoretical and experimental studies to determine and establish numerical values for major properties of components and joints in wood-stud walls. The results of the slip moduli for nail joints between wall coverings will be used in the reliability analysis of a wall system. Atherton et al (1980) investigated the slip and damping characteristics of wood-plywood and wood-gypsum board nailed joints. They included the effects of specific gravity, interfacial friction between the studs and sheathing and plywood thickness on slip modulus.

Gromala (1985) developed lateral nail resistance data for commonly used sheathing materials. He also mentioned that there was no appreciable difference in strength found to exist between cyclic and uncyclic loading. Polensek (1988) studied the effects of magnitude and rate of loading, specimen assembly technique and the assumption of linear behavior in evaluating slip on damping and stiffness of nailed wood-to-sheathing joints. He concluded that standard testing techniques are needed to achieve uniformity in dynamic testing of mechanically connected joints between structural wood materials.

III. COMPOSITE ACTION IN FLOOR AND WALL SYSTEMS

Floor Systems

Wood-joist floor systems are widely used in light-frame construction. A typical joist is a nominal 2×8 or 2×10 in. solid sawn lumber. Design procedures that incorporate system performance are not available. Rather, the design principle is that each joist acts independently under a given loading and the sheathing serves only to transfer load to the nearest joist. Thus, the sheathing materials are neglected in the design of the repetitive system. This leads to a conservative design but inefficient use of the materials.

In a wood-joist floor system, slip occurs between the joist and sheathing layers because the nail connections used in the floors are not rigid (Atherton et al 1980). The sheathing members and the joists have orthotropic mechanical properties and the sheathing is not continuous because of joints or gaps between sheets. In addition, the strength and stiffness of each joist varies considerably within and between joists.

In these analyses, strength and stiffness properties of 2×8 joists were treated as random variables. The composite stiffness, which is a product of joist, sheathing and fastener stiffnesses are used in a Monte Carlo simulation model to arrive at the load-sharing factor for the system (chapter IV). Then, the composite stiffness and the load-sharing factors are eventually used in a first-order second-moment reliability analysis when evaluating the failure probabilities (chapter VI).

Theoretical Bending Model of the Wood Floor System

The objective of this section is to describe the development of the theoretical bending stress for a wood joist floor system. By assuming the sheathing and joists exhibited composite action, the floor could be evaluated as a series of T-beams. The theoretical model for composite action to compute the composite stiffness of floors is based on work done by Amana and Booth (1967). The bending stress is then used in reliability procedure along with a load-sharing model, which will be developed in chapter IV, to evaluate the failure probabilities.

A typical wood floor system is shown in Figure 1. The joists shown in the figure are assumed to be spaced at 16 in. or 24. on center and consist of Douglas-fir visual grade No.1 or grade No.2. The nails are equally spaced at 6 in. on center and the floor covering is plywood and the orientation of the grain in plywood being perpendicular to the joist. Furthermore, the floor will

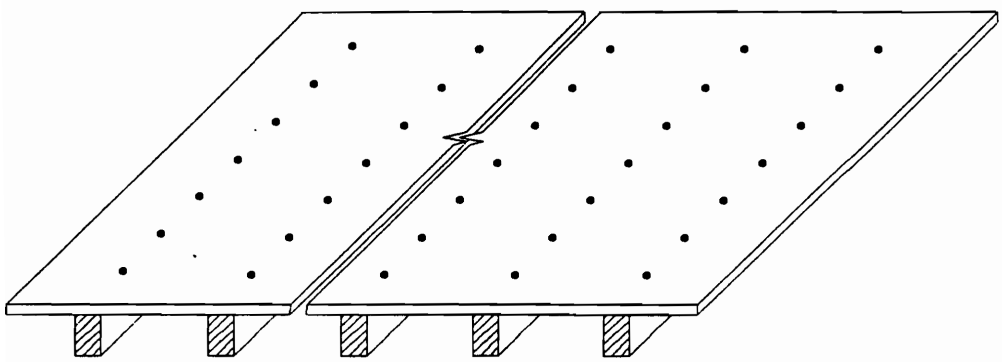


Figure 1. Part of Light-Frame Floor System with Wood Joists and Plywood Sheathing

be constructed according to the Uniform Building Code (1988) standards. The nail size conformed to Table No. 25-Q-Nailing Schedule, and plywood thickness and species group conformed to Table No.25-T-1 of this design specification.

These assumptions were used when evaluating composite stiffness of the floor system.

(1) Amount of slip at any connector is linearly proportional to the load acting at this connector.

(2) Strain distribution in each layer is linear.

(3) Shear transfer between the layers is continuous.

(4) Deflections of the layers are equal along the span.

(5) Friction between the layers is neglected.

(6) None of the layers can buckle.

Formulation of Floor Model Stiffness

The main purpose of this derivation is to obtain the composite stiffness of the floor system which will eventually be included in the bending stress equation.

The model assumes that the bending stresses act at the extreme fibers of the framing members. The critical limit state is in the joist and failure is initiated at the extreme fibers of the compressive zone as described by Malhotra and Bazan (1980). Upon further loading, stresses redistribute along the depth of the beam and the neutral axis shifts downwards towards the tensile side resulting in higher stresses and ultimately leading to a tensile failure.

The cross section of a floor system is shown in Figure 2. The cross-sectional dimensions are defined as, b_c = effective width of sheathing and is equal to joist spacing; h_c = thickness of plywood sheathing; h_j and b_j = joist depth and width respectively; $z = 0.5(h_j + h_c)$.

The total force or the shear transmitted by the nail is a function of interlayer shear flow (q_c) and nail spacing (s),

$$F = q_c s \tag{2}$$

The interlayer slip is given by

$$\Delta_s = \frac{q_c s}{k n_c} \tag{3}$$

where Δ_s = interlayer slip (in.)

F = force transmitted by the connector

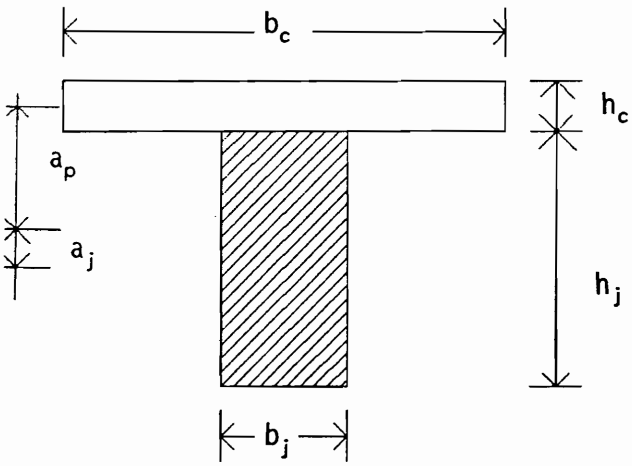
q_c = dF/dx = interlayer shear flow (lb/in.)

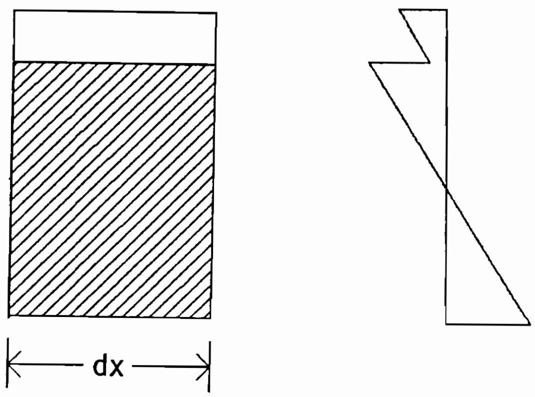
s = spacing of nails (in.)

k = slip modulus of nail (lb/in.)

n_c = number of nails per row (usually equal to 1)

(a)





(b) (c)

Figure 2. Floor section, (a) Cross-Section, (b) Elevation of Length dx, and (c) Strain Distribution

Slip strain is obtained by differentiating slip displacement with respect to distance dx.

$$\varepsilon_j = \frac{d[\Delta_s]}{dx} = \frac{s}{k}\frac{d^2F}{dx^2} \tag{4}$$

Because of the assumption that requires strain linearity in each layer, the strains at the interface of sheathing and joist can be obtained as,

$$\varepsilon_{p1} = \frac{M_p}{E_p I_p}\frac{h_c}{2} - \frac{F}{E_p A_p} \tag{5}$$

$$\varepsilon_{j1} = -\frac{M_j}{E_j I_j}\frac{h_j}{2} + \frac{F}{E_j A_j} \tag{6}$$

where ε_{p1} and ε_{j1} = strains in the lower surface of plywood member and the upper surface of the framing member respectively; M_p and M_j = moments in plywood sheathing and joist respectively; A_j and A_p = area of the joist and plywood member; E_p and E_j = modulus of elasticity of plywood and joist members; I_p and I_j = moment of inertia of plywood and joist members;

Due to the effect of interlayer slip,

$$\varepsilon_j = \varepsilon_{j1} - \varepsilon_{p1}$$

$$\varepsilon_j = -\frac{M_j}{E_jI_j}\frac{h_j}{2} + \frac{F}{E_jA_j} - \frac{M_p}{E_pI_p}\frac{h_c}{2} + \frac{F}{E_pA_p} \qquad (7)$$

As the curvatures of the joist and sheathing are equal, beam theory is used to establish the following relation.

$$-\frac{d^2w}{dx^2} = \frac{M_j}{E_jI_j} = \frac{M_p}{E_pI_p} \qquad (8)$$

Also from equilibrium of moments,

$$M = M_j + M_p + Fz \qquad (9)$$

$$M = \frac{M_j}{E_jI_j}\frac{h_j}{2} + \frac{M_p}{E_pI_p}\frac{h_c}{2} = \frac{M_j}{E_jI_j}z \qquad (10)$$

Re-arranging equations 8, 9 and substituting $EI = E_jI_j+E_pI_p$ results in,

$$\frac{M - Fz}{EI} = \frac{M_j}{E_jI_j} \qquad (11)$$

Combining equations 10 and 11, moment M can be expressed,

$$\frac{M_j}{E_jI_j}\frac{h_j}{2} + \frac{M_p}{E_pI_p}\frac{h_c}{2} = \frac{Mz - Fz^2}{EI} \qquad (12)$$

Using equation 12 in equation 7,

$$\varepsilon_j = \frac{F}{E_jA_j} + \frac{F}{E_pA_p} - \frac{Mz - Fz^2}{EI} \qquad (13)$$

Substituting equation 13 in equation 4, strain can be expressed as,

$$\frac{s}{k}\frac{d^2F}{dx^2} = F[\frac{1}{E_jA_j} + \frac{1}{E_pA_p} + \frac{z^2}{EI}] - \frac{Mz}{EI} \qquad (14)$$

Also from equations 8 and 11,

$$-\frac{d^2w}{dx^2} = \frac{M - Fz}{EI} \qquad (15)$$

Further developments to solve equations 14 and 15 are obtained from Amana and Booth (1967).

At this stage, a stiffening factor (i) is introduced, and this is the ratio of the deflection of the joist alone (w_o) to the deflection of the composite joist plus plywood.

$$i = \frac{w_o}{w} \qquad (16)$$

where w_o and w are given by (Amana and Booth 1967),

$$w_o = \sum_{n=1}^{\infty} \frac{F_n \text{Sin } \omega x}{E_jI_j\omega^2} \qquad (17)$$

$$w = \sum_{n=1}^{\infty} \frac{F_n \sin \omega x}{EI\omega^2} [1 - \frac{z^2}{G + Cz^2 n^2}] \tag{18}$$

F_n is a Fourier series coefficient, and $\omega = n\pi/L$.

$$\text{where} \quad G = \frac{EI}{E_j I_j} + \frac{EI}{E_p I_p} + z^2$$

$$C = \frac{sEI}{k} \frac{1}{z^2} \frac{\pi^2}{L^2} \tag{19}$$

Substituting equation 17 and 18 in equation 16

$$i = \frac{EI}{E_j I_j} [1 + \frac{z^2}{\frac{EI}{E_j A_j} + \frac{EI}{E_p A_p} + Cz^2}] \tag{20}$$

and further modification results in,

$$i = 1 + \frac{E_p I_p}{E_j I_j} + \frac{z^2}{1 + K_o} \frac{nA_j A_p}{[nA_p + A_j]} \frac{1}{I_j} \tag{21}$$

$$\text{where} \quad K_o = \frac{E_p A_p A_j C z^2}{EI[nA_p + A_j]}$$

$$n = \frac{E_p}{E_j} \tag{22}$$

Once (i) is evaluated, the combined moment of inertia is,

$$EI_{comb} = E_j I_j \, i \tag{23}$$

$$EI_{comb} = E_jI_j + E_pI_p + \frac{z^2}{1 + K_o}\frac{E_pA_jA_p}{[nA_p + A_j]} \tag{24}$$

Also, EI_{comb} can be represented as E_jI_w, where I_w = moment of inertia of the transformed section used and E_j = modulus of elasticity of the joist used as the base for transformation.

Then, eliminating stiffness from both sides of equation 24 yields

$$I_w = I_j + nI_p + \frac{z^2}{1 + K_o}\frac{nA_j^2A_p + n^2A_jA_p^2}{[nA_p + A_j]^2} \tag{25}$$

For a cross section such as that given in Figure 2, the centroid of the T-section from the geometric center of the flange is,

$$a_p = \frac{A_jz}{A_j + nA_p}$$

$$a_j = z - a_p = \frac{nA_pz}{A_j + nA_p} \tag{26}$$

Combining equations 25 and 26, the composite moment of inertia of the T-beam is,

$$I_w = I_j + nI_p + \frac{1}{1 + K_o}[na_p^2A_p + a_j^2A_j] \tag{27}$$

Expressions for Bending Stress

The bending stresses acting at the extreme fibers of the framing members are those associated with the limit state. These stresses are obtained from the strains and composite stiffness, which were derived earlier.

In terms of stress equation 7 is,

$$\sigma_j = -\frac{M_j}{I_j}\frac{h_j}{2} + \frac{F}{A_j} \tag{28}$$

Equation 12 can be written as,

$$\frac{M_j}{I_j} = \frac{M}{I} - \frac{Fz}{I} \tag{29}$$

Using equation 29 in 28, the stress developed at the outer fiber of the joist is,

$$\sigma_j = -\frac{M}{I_w}\frac{h_j}{2} + \frac{Fzh_j}{2I_w} + \frac{F}{A_j} \tag{30}$$

From equation 9,

$$F = \frac{1}{z}[\, M - M_j - M_p] \tag{31}$$

The interlayer moments for the individual layers can be written as,

$$\frac{M}{EI} = \frac{M_j}{E_j I_j} \tag{32}$$

$$M_j = M\frac{I_j}{I_w}, \quad M_p = M\frac{I_p}{I_w} \tag{33}$$

Substituting for M_j and M_p in equation 31, the force transmitted by the connector is,

$$F = \frac{M}{zI_w}[\, I_w + I_j + I_p] \tag{34}$$

The moment of inertia of the joist and plywood can be combined ($I_j + I_p = 47.96$), and equation 34 becomes,

$$F = \frac{M}{zI_w}[\, I_w + 47.96] \tag{35}$$

Substituting for F and z in stress equation 30,

$$\sigma_j = M\,[\frac{1.095}{I_w} + \frac{173.85}{I_w^2} + 0.023] \tag{36}$$

Material Properties

Joist properties. The data required for the floor analyses were the strength and stiffness properties of the joist members. The properties for 2×8 joist members were obtained from lumber tests (Leichti and Eskelsen, 1991). A total of 304 joist samples with a span of 123 in. of grades No.1 and No.2, were tested according to ASTM standards (ASTM 1991). Table 1 gives the mean modulus of elasticity (E) and the mean modulus of rupture (MOR). Statistical analyses on joist data were conducted, and different distribution functions were fitted to the data. The joist stiffness and strength data followed a three parameter Weibull probability density function and a Kolmogorov-Smirnov test statistic confirmed this at the 5% significance level. The method of maximum likelihood estimation Simon and Woeste (1980) was used to estimate the three parameters of Weibull distribution both for the strength and stiffness properties. The parameters of the Weibull probability density function describing the strength and stiffness properties are given in Table 2.

Modulus of elasticity and MOR are correlated and the correlation coefficients between dependent variables were established by using equation 37.

$$\rho(E,MOR) = \frac{\frac{1}{N}\sum_{i=1}^{N}[E(i) - mean(E)][MOR(i) - mean(MOR)]}{\sqrt{\frac{1}{N}\sum_{i=1}^{N}[E(i) - mean(E)]^2[MOR(i) - mean(MOR)]^2}} \qquad (37)$$

where $\rho(E,MOR)$ = correlation coefficient between E and MOR. The correlation coefficients are given in Table 3.

<u>Sheathing and nail properties</u>. The properties of the covering members and the stiffness modulus of the connectors were considered as deterministic. The deterministic properties were from the work by McCutcheon (1984). The plywood modulus of elasticity was taken as 1.487×10^6 lb/in^2 and the nail stiffness as 9400 lb/in. (McCutcheon 1984).

Loading Variables

The loads were treated as random variables. Dead load was assumed to follow a normal distribution with a coefficient of variation of 10%. Live load was assumed to follow a Type I extreme value distribution and was evaluated according to the Uniform Building Code (1988). The mean live load pressure on the floor model was taken as 40 psf, with coefficient of variation of 25% (Ellingwood et al 1980). These statistics are given in Appendix C.

Table 1. Mean E and MOR for Douglas-fir Joists

Joist Grade	N	E		MOR	
		Mean (psi)	CV (%)	Mean (psi)	CV (%)
No.1	153	1786.58	0.155	8681.40	0.251
No.2	151	1497.22	0.179	5643.36	0.392

Table 2. Three Parameters of Weibull Probability Function for Douglas-fir Framing Lumber

Joist Grade	E			MOR		
	η	μ_w(psi)	σ_w(psi)	η	μ_w(psi)	σ_w(psi)
No.1	3.98	809995	1083198	4.55	0.0	9448
No.2	3.30	700734	919333	1.94	1545.7	4534

Table 3. Correlation Coefficients of the Dependent Variables

Joist Grade	Variable	E	MOR
No.1	E	1.00	0.60
	MOR	0.60	1.00
No.2	E	1.00	0.53
	MOR	0.53	1.00

Wall Systems

In light-frame wood buildings, stud walls are the basic vertical structural components. The walls act primarily as bending and compressive panels and transmit lateral wind load and gravity loads into the foundations. Mechanical fasteners form most joints and provide semi-rigid connections between framing and sheathing.

This chapter introduces a probability-based design method of evaluating reliabilities in light-frame wood systems. The method includes interaction of components as well as the nonlinearities of materials and connectors. The wall system will be analyzed under bending and axial compressive loads.

Strength and stiffness properties of 2×4 studs are considered as random variables and properties of the individual stud members, along with the composite stiffness are used in a Monte Carlo simulation model to arrive at the load sharing. The composite stiffness and the load-sharing factors are eventually used in a first-order second-moment reliability analysis when evaluating the failure probabilities.

Analytical Bending Model of the Wood Wall System

In a wall system, the sheathings act with the framing member to form a composite I-beam rather than simple rectangular beam; the stud acts as the web and the sheathings as the flanges. The objective of this section is to describe the development of the theoretical bending stress for wood stud I-beam wall system which includes composite action. The theoretical model for composite action is based on the composite stiffness of walls (Polensek and Kazic 1991). The bending stress is then used in reliability procedure along with a load-sharing model which will be developed in chapter IV to evaluate the failure probabilities.

A typical wood-stud wall system is shown in Figure 3. The studs shown in the Figure 3 are assumed to be spaced at 16 in. or 24 in. on center and consist of Douglas-fir or southern pine species. The nails are equally spaced at 6 in. on center and the exterior covering consists of plywood (3/8-in. CDX) with the orientation of the grain in plywood being perpendicular to that of the studs and the interior covering consisting of 3/8 in. thick gypsum wallboard also oriented perpendicular to the studs. The wall construction was assumed to match the Uniform Building Code (1988)

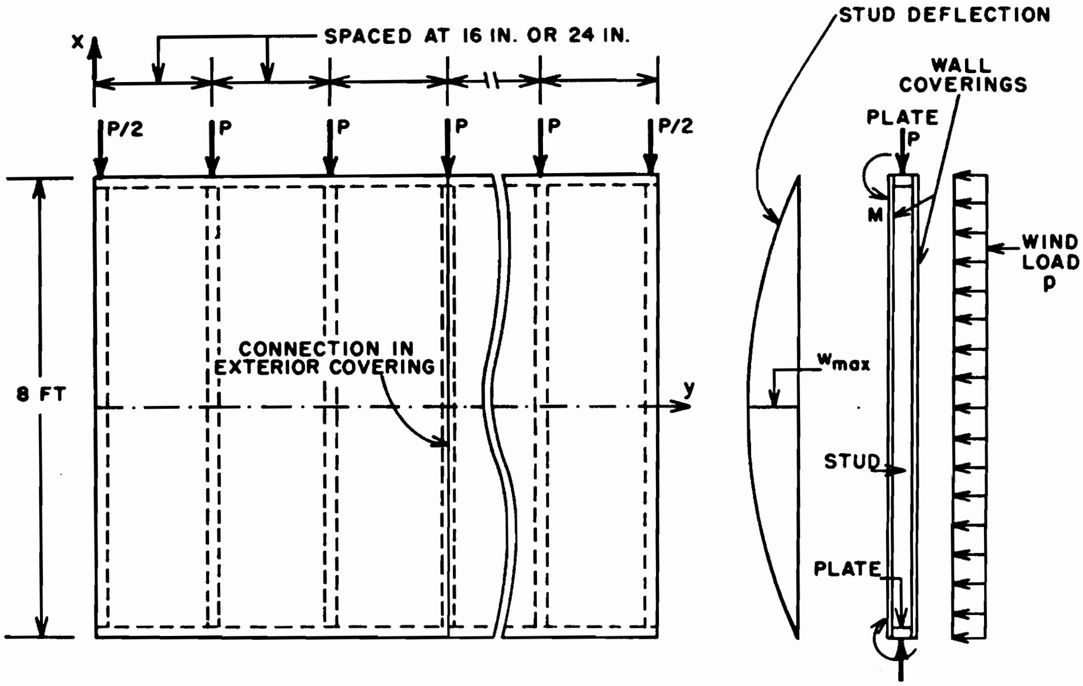


Figure 3. Wood-Stud Wall System (Polensek 1984), (a) Front view, (b) Side View with Loading and Deflection

standards. Nail size was also assumed to be in compliance with this design specification.

The assumptions of the analytical model for the properties of a composite section were similar to the assumptions used with the floor analyses.

(1) Shear transfer between the sheathing and the framing member is continuous.

(2) Amount of slip at any connector is linearly proportional to the load acting at this connector. (Load slip characteristic is linear).

(3) Distribution of strain in each layer is linear.

(4) Deflections of the layers are equal along the span.

(5) None of the layers can buckle.

Formulation of Wall Model Stiffness

The main purpose of this formulation is to obtain the composite stiffness of the wall system which will eventually be included in the bending stress equation. The cross section of a wall system is shown in Figure 4 and the cross-sectional dimensions are defined as,

h_t, h_c = thickness of plywood and gypsum sheathing; h_s and b_s = stud depth and width respectively; $z_c = 0.5(h_c + h_s)$; $z_t = 0.5(h_t + h_s)$.

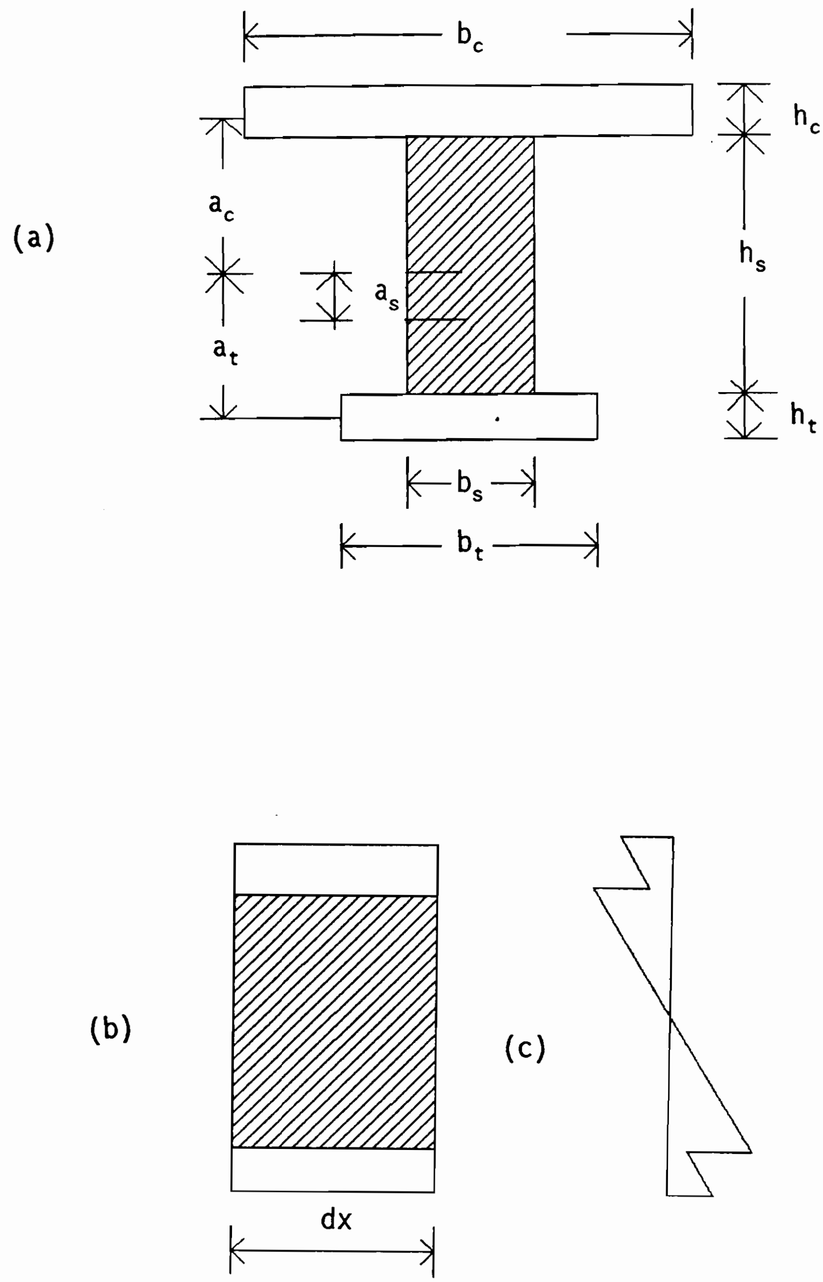


Figure 4. Wall Segment Showing Forces and Strains (Polensek and Kazic 1991), (a) Cross Section, (b) Elevation of Length dx, and (c) Strain Distribution at an Arbitrary Cross Section.

The compressive, tensile strains and the combined moment of inertia I_w of the wall model (Polensek and Kazic 1991) are,

$$\varepsilon_{sc} = -\frac{M}{EI_w}\gamma[z_c(K_{ct} + K_c) + z_t N_c] \tag{38}$$

$$\varepsilon_{st} = -\frac{M}{EI_w}\gamma[z_t(K_{ct} + K_t) + z_c N_t] \tag{39}$$

$$I_w = [I_s+n_cI_c+n_tI_t] + \gamma[A_sa_s^2 + n_cA_ca_c^2 + n_tA_ta_t^2] + \gamma A_s[N_cz_t^2 + N_tz_c^2]$$

$$\text{where} \quad \gamma = \frac{1}{1 + K_c + K_t + K_{ct}} \tag{40}$$

where ε_{sc} and ε_{st} = the strains on the compressive and tensile surfaces of the framing member, I_w = the combined moment of inertia, E = modulus of elasticity of the composite beam, subscript c = compressive flange, subscript t = tensile flange, subscript s = stud, and all other parameters are defined in the combined moment of inertia, Appendix D.

Stud walls are subjected to lateral and axial compressive loads. Axial loads from gravity, live load and lateral wind load are taken into consideration in the form of a beam-column equation.

From conventional beam equations, the midspan deflection under uniformly distributed loads is calculated as,

$$\Delta = \frac{5\ M_{max}\ L^2}{48\ EI_w} \tag{41}$$

where $M_{max} = wL^2/8$ = maximum moment. However when including the axial load for the beam-column (P), the maximum moment can be written as, (Timoshenko and Gere 1961)

$$M_{max} = \frac{w\ L^2}{8}\ [1 + \frac{5PL^2}{48EI_w}] \tag{42}$$

Expressions for Stresses and Interlayer Slip

Wall systems fail when bending or compressive stresses imposed by the axial and lateral loads exceed the established limit state. Bending stresses act at the extreme fibers of the framing member, and a simplified expression for the flexural stress at midspan of the framing members (Polensek and Kazic 1991) is,

$$\sigma_s = -\ [\frac{M_{max}}{I_w}][\gamma a_s + \gamma z_t N_c - \gamma z_c N_t + \frac{h_s}{2}] - \frac{P}{A_s} \tag{43}$$

The formulae for slip are evaluated by integrating slip strains, between the framing and sheathing interfaces. The compressive and tensile slip are evaluated from moments and are given by,

$$slip_{sc} = \int_0^x \varepsilon_{sc} dx = - [\frac{M_{max}}{EI_w}] \gamma [z_c(K_{ct}+K_c) + z_t N_c] \; L(\frac{x}{L})[1 - \frac{4}{3}(\frac{x}{L})^2] \tag{44}$$

$$slip_{st} = \int_0^x \varepsilon_{st} dx = - [\frac{M_{max}}{EI_w}] \gamma [z_t(K_{ct}+K_t) + z_c N_t] \; L(\frac{x}{L})[1 - \frac{4}{3}(\frac{x}{L})^2] \tag{45}$$

Material Properties

Stud properties. The data required for the wall analyses were the strength and stiffness properties of the stud members. The properties for 2×4 stud members were obtained from Western Wood Products Association and West Coast Lumber Inspection Bureau (1987). The stud load-deflection traces obtained in the tests were nonlinear and were approximated by two linear line

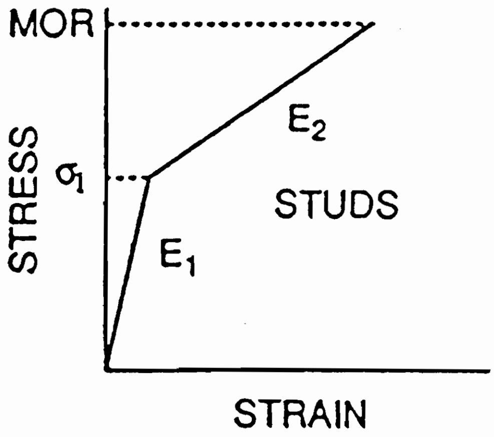


Figure 5. Idealized Nonlinear Stiffness Properties for Studs (Polensek and Kazic 1991)

segments. The segments were associated with moduli of elasticity E_1 and E_2, which are valid at the midspan deflection Δ_1 and Δ_2. The two linear segments were then transformed into the stress-strain diagrams in which the initial trace represents the modulus of elasticity E_1 and state of stress (σ_1), which corresponds to the proportional limit. The second trace represented E_2 and MOR (ultimate stress). Table 4 gives the number of data samples, the mean moduli of elasticity (E_1 and E_2) and the mean stress (σ_1 and MOR).

Douglas-fir and southern pine lumber of stud grade were considered for use in the analyses. Each species included two groups of samples, as as-grade and on-grade. The as-grade data included the physical properties of all studs in the original lot. Data for on-grade lumber represented the initial specimens after they were regraded to eliminate those that were off-grade. The difference between the two grades reflects the misgrading at lumber mills (Polensek and Gromala 1984)

Statistical analyses of the studs were conducted and different distribution functions were fitted to the data. A three-parameter Weibull distribution function represents the data well both for as-grade and on-grade Douglas-fir and southern pine studs. The Kolmogorov-Smirnov test statistic confirmed that this function represents data well for both Douglas-fir and southern pine studs at 5% significance level. The method of maximum likelihood estimation (Simon and Woeste 1980) was used to estimate the three parameters of Weibull distribution both for the strength and stiffness properties. The parameters of the Weibull probability function describing the strength and stiffness properties are given in Table 5. Histograms with overlays of the fitted distributions and the Weibull location, scale and shape parameters for the two different species of studs are shown in Figures 6 and 7.

The correlation between the dependent variables E_1, E_2, σ_1 and MOR were found by using equation 37, and the coefficients are given in Table 6.

Table 4. Mean E and MOR for As-Graded (AS) and On-Grade (ON) Douglas-fir and Southern Pine Studs

Stud type	N	E_1		E_2		σ_1		MOR	
		Mean	CV	Mean	CV	Mean	CV	Mean	CV
		(ksi)	(%)	(ksi)	(%)	(psi)	(%)	(psi)	(%)
DF-AS	440	1307	23.8	361	78.2	3922	32.7	848	84.5
DF-ON	649	1328	24.3	371	73.2	3889	31.3	880	79.9
SP-AS	420	1416	30.4	520	76.2	4798	38.2	1293	87.0
SP-ON	657	1429	27.4	530	74.0	4853	35.5	1170	89.0

Table 5. Three Parameters of Weibull Probability Function (Shape, Location and Scale) for Douglas-fir and Southern Pine Framing Lumber

Stud Type		Weibull Parameters		
		η	μ_w(psi)	σ_w(psi)
			E_1	
Douglas-fir	AS	2.36	622813	811300
	ON	2.38	602187	811300
Southern pine	AS	2.88	275920	1296500
	ON	2.79	399467	1133200
			E_2	
Douglas-fir	AS	1.30	0	390604
	ON	1.40	0	407039
Southern pine	AS	1.34	0	566139
	ON	1.39	0	580945
			σ_1	
Douglas-fir	AS	2.08	1325	3047
	ON	2.29	1256	3088
Southern pine	AS	2.13	998	4189
	ON	2.57	683	4784
			MOR	
Douglas-fir	AS	1.18	0	897
	ON	1.28	0	951
Southern pine	AS	1.16	0	1363
	ON	1.14	0	1225

Table 6. Correlation Coefficients Between Combinations of the Dependent Variables

Stud Type		Correlation Coefficients					
		E_1E_2	$E_1\sigma_1$	E_1MOR	$E_2\sigma_1$	E_2MOR	σ_1MOR
Douglas-	AS	0.263	0.638	0.421	0.00	0.575	0.162
fir	ON	0.265	0.664	0.434	0.00	0.525	0.186
Southern	AS	0.351	0.756	0.245	0.270	0.511	0.251
pine	ON	0.306	0.773	0.273	0.223	0.535	0.274

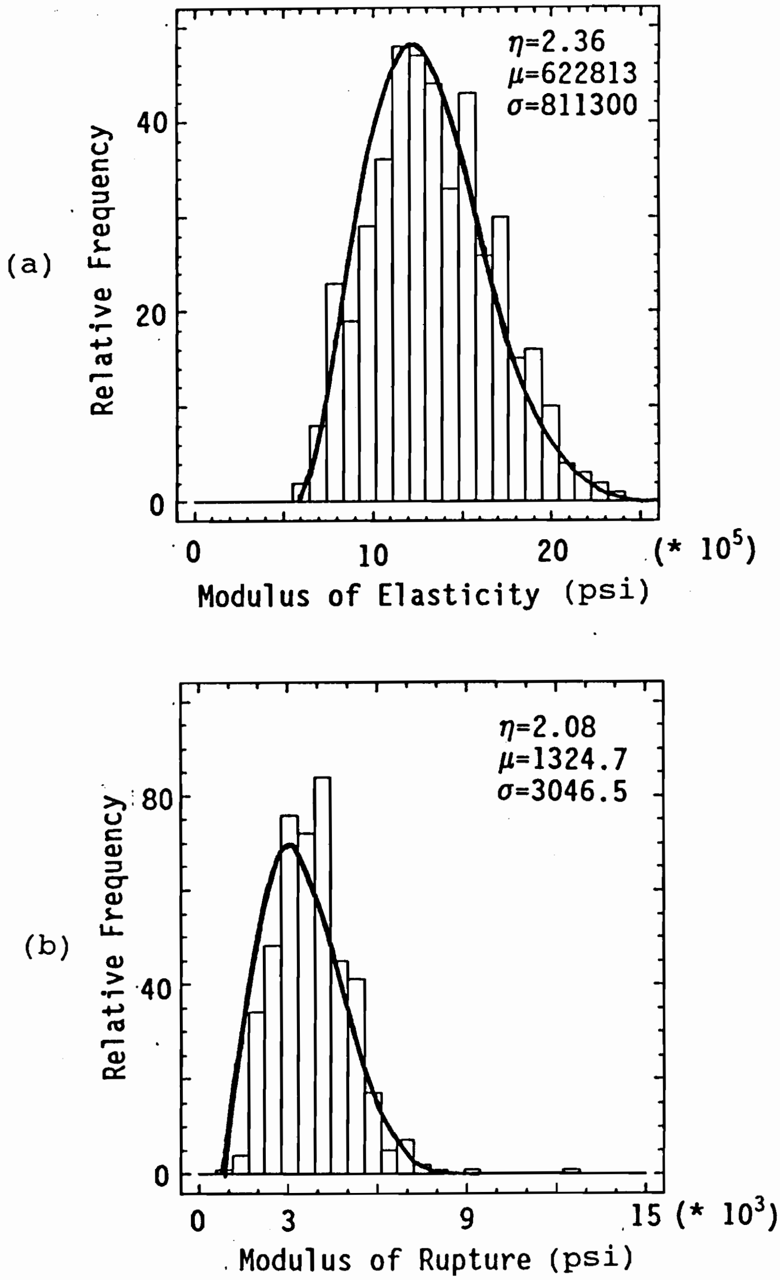
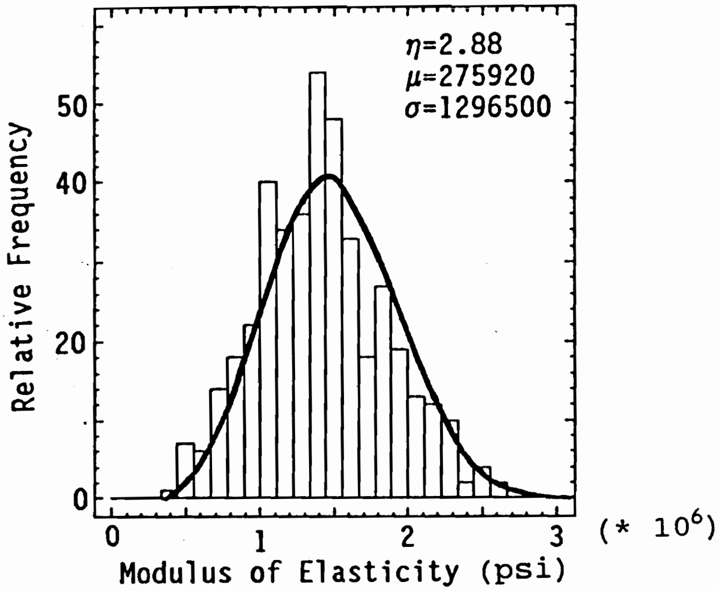


Figure 6. Frequency Histogram of 440 Observations for As-Graded Douglas-fir 2×4's with the Three-Parameter Weibull Probability Density Function Overlaid, (a) E_1 and (b) σ_1

(a)



(b)

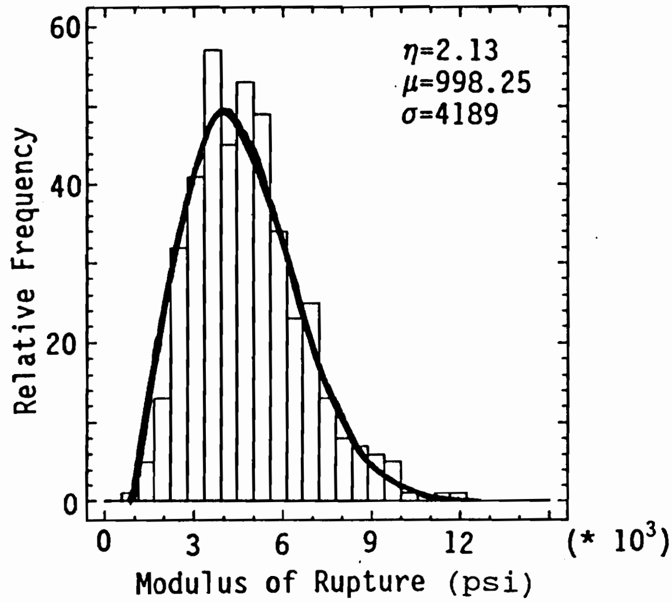


Figure 7. Frequency Histogram of 420 Observations for As-Graded Southern Pine 2×4's with the Three-Parameter Weibull Probability Density Function Overlaid, (a) E_1 and (b) σ_1

Sheathing and nail properties. Stud properties were considered as random variables, whereas elastic properties for wall coverings and joint slip moduli were taken to be deterministic. Properties of wall coverings and joint slip moduli are the average values obtained by testing (Polensek and Gromala 1984). The properties of sheathing materials and slip moduli are given in Tables 7 and 8.

The use of deterministic properties for wall coverings and joint properties was justified on several counts. First overall variation in covering properties is small compared to the variation in lumber properties. A sensitivity study of wood stud walls (Polensek 1982) showed that an increase of 50% in sheathing E produced a change of 2% in strength. In addition, variability in slip moduli has very little effect on the system performance (Polensek and Kazic 1991).

Table 7. Elastic Properties of Plywood and Gypsum Sheathing in Bending

Elastic Property	Gypsum	Plywood
E_x (ksi)	230	1730
E_y (ksi)	360	290
G_{xy} (ksi)	159	150
ν_{xy}	0.23	0.146
ν_{yx}	0.37	0.025

E_x, E_y = Modulus of Elasticity in x and y-directions.

Table 8. Slip Moduli for Nail Joints between Studs and Sheathing (Polensek and Gromala 1984)

Property	Variable	Material	
		Plywood	Gypsum
Slip Moduli (kip/in.)	k_{c1}	4.08	
	k_{c2}	1.44	
	k_{t1}		15.43
	k_{t2}		0.54
Slip (in.)	s_{c1}	0.025	
	s_{c2}	0.12	
	s_{t1}		0.002
	s_{t2}		0.54

Loading Variables

The properties of loads were treated as random variables. Dead load was represented as a normal distribution with a coefficient of variation of 10% and wind load was represented by a Type I extreme value distribution function with a coefficient of variation of 25% (Ellingwood et al 1980). More details of of the calculation of the dead load and wind load is given in Appendix B.

In the analyses of the wall systems, maximum expected wind load in the northwest United States was chosen as the load criteria for the Douglas-fir wall system, and that occurring in the southern region for the southern-pine wall system. The geographic locations were North Head, Washington and Keywest, Florida for the analysis of Douglas-fir and southern pine walls respectively. Wind load statistics used in wall analyses are given in Table 9. Wind velocities were taken from Simiu et al (1979).

Wind load, which is the primary external moment in wall analysis, is evaluated according to ANSI (A58.1-1982) minimum design loads for buildings and other structures. The lateral wind pressure on the wall models is,

$$w = c\ C_p\ K_z\ G\ V^2 \tag{46}$$

where w = lateral wind pressure, c = constant, C_p = external pressure coefficient, K_z = velocity pressure exposure coefficient, G = gust factor and V = wind velocity (mi/hr).

With the exception of wind velocity, which follows a Type I largest extreme-value distribution function, the other random variables C_p, K_z, G and V follow normal distribution. The probability distribution of the random variable defining wind load, the mean wind velocity and coefficient of variation for 50 and 100 yr are given in Table 10. The parameters a and b for the 50-yr and 100-yr winds are given in Table 11.

Table 9. Wind Load Statistics used in Wall Analyses

Location	Return Period (yr)	Wind Velocity (mi/hr)	CV (%)
North Head, WA.	50 or 100	71.47	14.2
Key West, FL.	50 or 100	51.00	33.7

Table 10. Probability Distributions of the Random Variables Defining Wind Loading

Random Variable	Probability Distribution	Mean	CV (%)
Axial Load (lb)	Normal	181.1	10.0
C_p	Normal	20.13	13.0
K_z	Normal	1.20	16.0
G	Normal	1.15	11.0
V (mi/hr)	Type I	71.47	14.2

Table 11. Mean Wind Load for Return Periods of 50 and 100 yr

Stud Type	Return Period	Wind Velocity (mi/hr)	Parameters * a	b
Douglas-fir	50	102.4	0.125	97.80
Southern pine	50	103.2	0.074	95.69
Douglas-fir	100	107.9	0.126	103.35
Southern pine	100	112.7	0.075	104.98

* defined by $f_V(v) = a \exp[-a\,(v-b) - \exp\{-a\,(v-b)\}]$

IV. LOAD SHARING WITH COMPOSITE ACTION IN LIGHT-FRAME WOOD SYSTEMS

An analytical method for evaluating the load distribution in light-frame wood structures is presented. As load distribution occurs in conjunction with composite action, the composite stiffness required for the analysis of load sharing in walls and floor systems is taken from chapter III.

Load Sharing

Load sharing occurs by the distributing action of the sheathing in a direction perpendicular to the framing members and is influenced by the relative stiffness of the framing members. Load sharing effects in light-frame structures are considered by treating the framing members as a set of elastic springs with the sheathing as a distributing beam. Composite stiffness needed for evaluating the spring stiffness for floors and walls is developed in chapter III. In the case of a wall system, the flexural rigidities of the exterior and interior sheathings were summed to model the composite flexural rigidity of the distributing beam.

$$EI_{db} = (EI)_{gypsum} + (EI)_{plywood} \qquad (47)$$

Figure 8 illustrates the real wall system and the mechanical analog of springs with a distributor beam. In the literature (for example, McCutcheon 1984), this condition is described as a beam on elastic foundation. The analog of a beam on an elastic foundation can be used as the numerical basis for analyses of both walls and floors. Gaps which disrupt the continuity of the sheathing are accounted for, through a reduced bending stiffness (McCutcheon 1984).

The beam on elastic springs is based on the model developed by McCutcheon (1984). For the theoretical analysis of load sharing in wall and floor systems, it was assumed that studs and joists were simply supported at the ends, and that they are equally spaced at 16 or 24 in. on center. A sheathing panel 96 in. long, spans 6 or 4 joists or studs with a 16 in. or 24 in. spacings. Thus, on the average a discontinuity exists at every fourth or sixth joist or stud. The flexural rigidity EI_{db} is reduced by an empirical-based constant of one-sixth or one-fourth depending on the spacing (McCutcheon 1984).

Deflection Equations for a Load-Sharing System

The deflection equations for a load-sharing system are applicable to walls and floors. A typical wall system modeled as a beam on an elastic foundation is

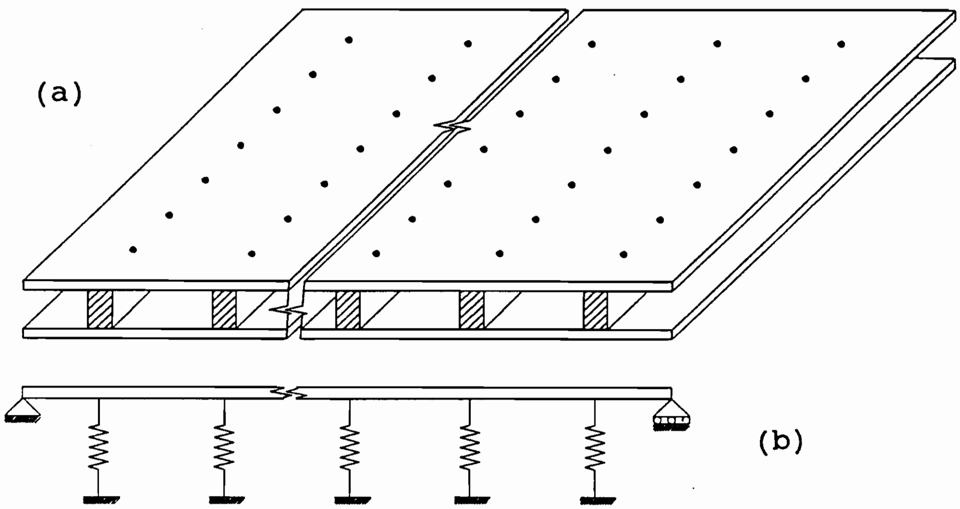


Figure 8. Wall System with Framing Members as Elastic Springs and Sheathing as a Distributor Beam, (a) Real System, (b) Mechanical Analog for Beam on Elastic Foundation

shown in Figure 8. The total flexural rigidity of the distributing members (the sheathing layers) is denoted by EI_{db}, and this includes the flexural stiffness of both the plywood and gypsum sheathings. In the case of a floor system, the tensile member (gypsum-sheathing) is neglected.

The deflection equations for the distributor beam in a wall or floor system model is developed by using the principle of superposition: the deflection equations of a simply supported beam with uniform load are superpositioned with a simply supported beam subjected to a point load.

The first case is that of a simply supported beam with a uniformly distributed load (w) as shown in Figure 9. The deflection Δ_x at any point x is given by,

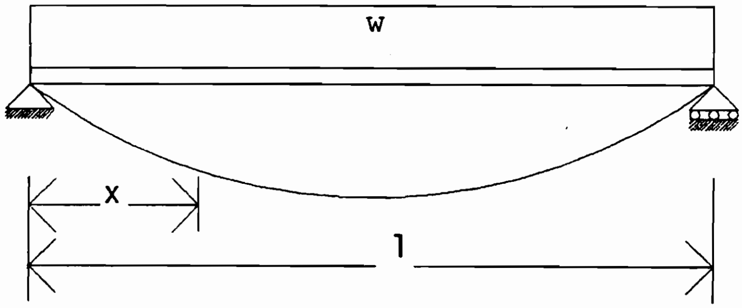
$$\Delta_x = \frac{wx}{24\ EI_{db}}\ [l^3 - 2lx^2 + x^3] \tag{48}$$

A second condition is a simply supported beam subjected to a point load R. The deflection Δ_x at any point x is given by,

$$\Delta_x = \frac{Rx(l - x)}{6l\ EI_{db}}\ [2lb - b^2 - x^2] \tag{49}$$

when $x \leq b$

(a)



(b)

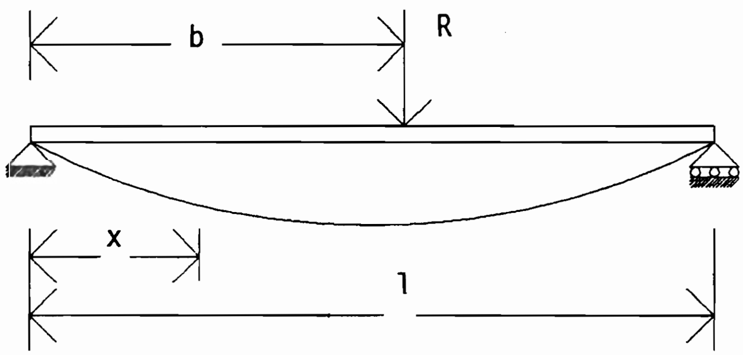


Figure 9. Bending Members, (a) Uniformly Distributed Load, (b) Concentrated Load at an Arbitrary Distance b from Left End

$$\Delta_x = \frac{Rb(l - x)}{6l\ EI_{db}} [2lx - x^2 - b^2] \tag{50}$$

when $x \geq b$

where b = distance from the support to the point load

l = length of distributing beam (wall or floor width),

E = modulus of elasticity of the beam,

I = moment of inertia of the beam,

R = reaction due to the point load,

x = distance from the support to the point where deflection is desired.

For an 8-member framing system, the deflection at spring 1 (Δ_1) is given by,

$$- \Delta_1 = -R_1 x_1{}^2 \frac{(l - x_1)^2}{3l\ (EI_{db})}$$

$$- R_2 x_1 (l - x_2) \frac{(2lx_2 - x_2^2 - x_1^2)}{6l(EI_{db})}$$

$$- R_3 x_1 (l - x_3) \frac{(2lx_3 - x_3^2 - x_1^2)}{6l(EI_{db})}$$

$$- R_4 x_1 (l - x_4) \frac{(2lx_4 - x_4^2 - x_1^2)}{6l(EI_{db})}$$

$$- R_5 x_1 (l - x_5) \frac{(2lx_5 - x_5^2 - x_1^2)}{6l(EI_{db})} \quad (51)$$

$$- R_6 x_1 (l - x_6) \frac{(2lx_6 - x_6^2 - x_1^2)}{6l(EI_{db})}$$

$$- R_7 x_1 (l - x_7) \frac{(2lx_7 - x_7^2 - x_1^2)}{6l(EI_{db})}$$

$$- R_8 x_1 (l - x_8) \frac{(2lx_8 - x_8^2 - x_1^2)}{6l(EI_{db})}$$

$$- wx_1 \frac{(l^3 - 2lx_1^2 + x_1^3)}{24\ (EI_{db})}$$

Here R_i (i = 1, 2,..., 8) = reactions (resistance) offered by the framing members,

x_i = framing spacing.

Deflections can be determined for the intermediate members or springs.

Finally the deflection at spring 8 (Δ_8) is given by,

$$- \Delta_8 = R_1 x_1 (l - x_8) \frac{(2lx_8 - x_8^2 - x_1^2)}{6l(EI_{db})}$$

$$- R_2 x_2 (l - x_8) \frac{(2lx_8 - x_8^2 - x_2^2)}{6l(EI_{db})}$$

$$- R_3 x_3 (l - x_8) \frac{(2lx_8 - x_8^2 - x_3^2)}{6l(EI_{db})}$$

$$- R_4 x_4 (l - x_8) \frac{(2lx_8 - x_8^2 - x_4^2)}{6l(EI_{db})}$$

$$- R_5 x_5 (l - x_8) \frac{(2lx_8 - x_8^2 - x_5^2)}{6l(EI_{db})} \qquad (52)$$

$$- R_6 x_6 (l - x_8) \frac{(2lx_8 - x_8^2 - x_6^2)}{6l(EI_{db})}$$

$$- R_7 x_7 (l - x_8) \frac{(2lx_8 - x_8^2 - x_7^2)}{6l(EI_{db})}$$

$$- R_8 x_8^2 \frac{(l - x_8)^2}{3l(EI_{db})}$$

$$- w x_8 \frac{(l^3 - 2lx_8^2 + x_8^3)}{24\ (EI_{db})}$$

Each of the reactions R_i can be expressed as $R_i = -k_i \Delta_i$, where k_i is the spring stiffness of the framing member and can be expressed as the ratio of the load to the deflection of the individual framing member.

$$k_i = \frac{wL}{\Delta_i} \qquad (53)$$

In equation 53, Δ_i is the midspan deflection of a uniformly distributed framing member and is given by,

$$\Delta_i = \left(\frac{5}{384}\right)\frac{wL^4}{E_iI_w} \tag{54}$$

w = the load in lb/in., E_i = the stiffness of the individual framing member, L = length of the framing member and E_iI_w = the composite flexural rigidity which includes stiffness and geometry of all materials and connections. The term E_iI_w is derived in chapter III for floors and walls.

Substituting for R_i in the deflection equations and multiplying by $-6l(EI_{db})$, equation 51 can be written as,

$$\Delta_1[6lEI_{db} + 2k_1\, x_1^2(l - x_1)^2]$$
$$+ \Delta_2[k_2x_1(l - x_2)(2lx_2 - x_2^2 - x_1^2)]$$
$$+ \Delta_3[k_3x_1(l - x_3)(2lx_3 - x_3^2 - x_1^2)]$$
$$+ \Delta_4[k_4x_1(l - x_4)(2lx_4 - x_4^2 - x_1^2)]$$
$$+ \Delta_5[k_5x_1(l - x_5)(2lx_5 - x_5^2 - x_1^2)]$$
$$+ \Delta_6[k_6x_1(l - x_6)(2lx_6 - x_6^2 - x_1^2)] \qquad (55)$$
$$+ \Delta_7[k_6x_1(l - x_7)(2lx_7 - x_7^2 - x_1^2)]$$
$$+ \Delta_8[k_8x_1(l - x_8)(2lx_8 - x_8^2 - x_1^2)]$$
$$= wx_1l\frac{(l^3 - 2lx_1^2 + x_1^3)}{4}$$

In a matrix form, equation 55 can be expressed as,

$$\Delta_1a_{11} + \Delta_2a_{12} + \Delta_3a_{13} + \Delta_4a_{14} + \Delta_5a_{15} + \Delta_6a_{16} + \Delta_7a_{17} + \Delta_8a_{18} = w_1 \qquad (56)$$

Hence, the set of equations can be simplified to,

$$\sum_{ij}^{n} a_{ij}\, \Delta_j = w_i \qquad (57)$$

where Δ_j are deflections at each of the framing members, and the coefficients are given by,

$$a_{ij} = [6lEI_{db} + 2k_i\ x_i^2(l - x_i)^2] \quad \text{for } i = j \tag{58}$$

$$a_{ij} = [k_j x_i (l - x_j)(2lx_j - x_j^2 - x_i^2)] \quad \text{for } i < j \tag{59}$$

$$a_{ij} = [k_j x_j (l - x_i)(2lx_i - x_i^2 - x_j^2)] \quad \text{for } i > j \tag{60}$$

$$w_i = w\ x_i l \frac{[l^3 - 2lx_i^2 + x_i^3]}{4} \quad \text{for } i = 1, 8 \tag{61}$$

The main parameters required to solve equation 57 are the stiffnesses of the individual framing members, the stiffness of the sheathing, stiffness of the fasteners and the external loading function. A computer program, given in Appendix H was written to solve for the eight unknown deflection quantities. The proposed deflection equations for load sharing system was verified by comparing results with those of McCutcheon (1984) and the theoretical model agreed closely with test results.

Simulation Methods

In the analysis of load sharing with composite action in floor and wall systems, simulation of joist and stud properties were required. This was accomplished by a Monte Carlo simulation technique. However, the strength and stiffness properties of joist and stud members are correlated. Simulation of independent strength and stiffness properties would give rise to errors in modeling.

A brief discussion of simulation of correlated variables considered in the reliability analysis with composite action and load sharing is presented.

Treatment of Correlated Variables

Structural reliability concepts are developed in chapter V, based on the assumption that the random variables involved in the limit state equation were statistically independent. For weakly correlated random variables, those with correlation less than 0.2, the assumption of statistical independence is satisfactory, whereas correlation greater than 0.8, complete dependence can be assumed (Smith 1986).

A Monte Carlo simulation technique was used to generate the random samples for the framing members from Weibull density functions for the strength and stiffness, which are correlated. Simulation of correlated lumber properties required a transformation of the multivariate normal distribution that preserves the probability distribution of each property (Taylor and Bender 1988). The reason for doing this is that the simulated variable of stiffness for the framing member gives a true representation of the sample by providing a random variable in strength. Thus, the random variables in strength and stiffness are generated with the desired correlation. A detailed description of correlated variables is given in Appendix F.

The Load Sharing Simulation

A flowchart for the computer simulation of load sharing with composite action in a light-frame wood systems is given in Figure 10. The program simulated 1000 walls or floors and for each wall or floor, iterates until one framing member "fails," where "failure" was defined by

$$\left[\frac{f_{bi}}{MOR_i/2.1}\right] \leq 1.0 \qquad (62)$$

Then, load sharing was computed as the ratio of the highest lateral load carried by the weakest framing member with respect to the lateral or gravity load prescribed by the Uniform Building Code (1988). The process was repeated 1000 iterations, and the mean and coefficient of variation in load sharing were evaluated.

Results of Load Sharing

Using this approach load-sharing factors were evaluated for floor and wall systems, and the results are given in Tables 12 and 13. Load sharing was influenced by framing spacing (closer spacing yielded greater load sharing). In case of floor systems, load sharing was also affected by the joist grade (No.1 had higher levels of load sharing than No.2). The effect of grade on load sharing is related to the coefficient of variation of the joist stiffness and strength; load sharing increased as the coefficient of variation in strength of the lumber MOR decreased. This was observed in grade No.2 joist with a coefficient of variation of 39%, which yielded a load-sharing factor of 1.43 in comparison to 1.96 for grade No.1 joist with a coefficient of variation of 25%. Similar results where the system factor increased in response to lower variation in lumber strength were reported by Rosowsky and Ellingwood (1991).

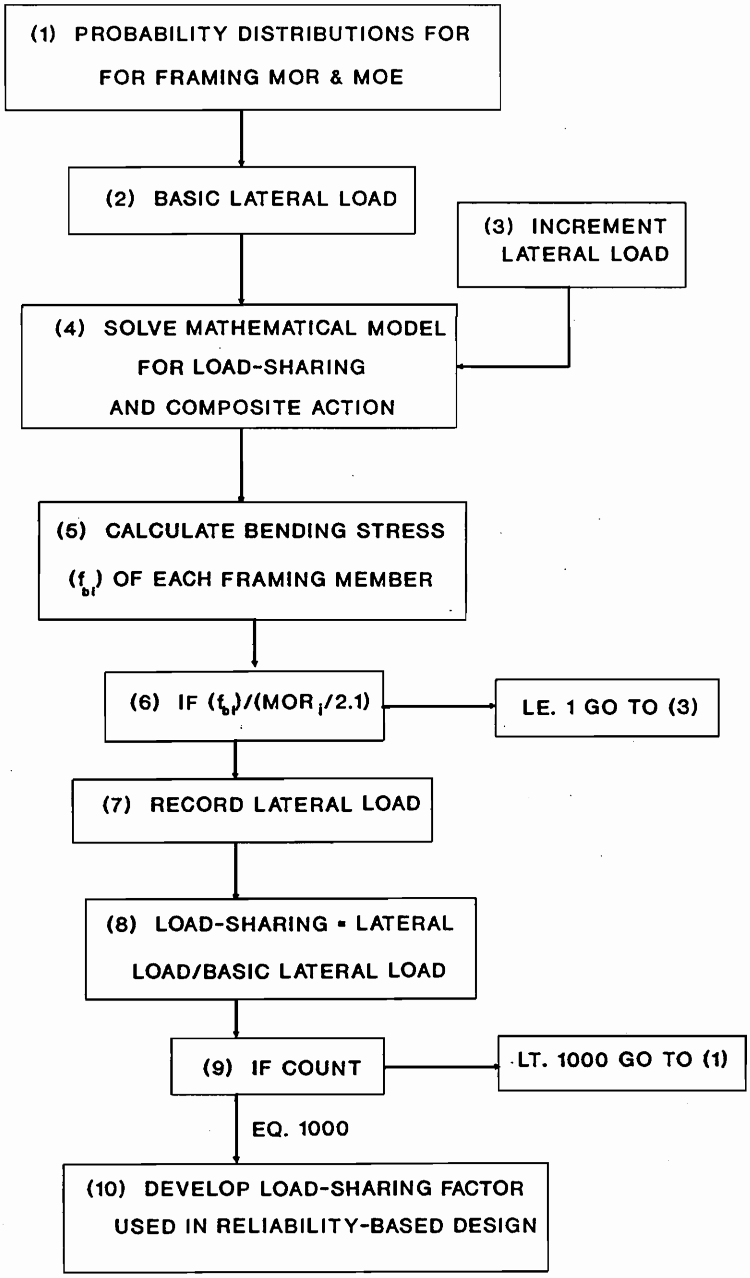


Figure 10. Flowchart for Computer Simulation of Load-Sharing Factors to be used in Reliability Analyses

Table 12. Results of Load-Sharing Simulation for Floor Systems with Douglas-fir Lumber

Joist Spacing (in)	Joist Grade	Load-Sharing Factor	CV (%)
16	No.1	1.96	17.0
24	No.1	1.66	26.0
16	No.2	1.43	28.0
24	No.2	1.19	41.0

Table 13. Results of Load-Sharing Simulation for Wall Systems with Douglas-fir and Southern Pine Lumber

Stud Type		Spacing (in)	Load-Sharing Factor	CV (%)
Douglas-fir	AS	16	1.50	27.6
		24	1.40	27.8
Southern pine		16	1.70	29.6
		24	1.59	30.8
Douglas-fir	ON	16	1.55	27.2
		24	1.43	27.1
Southern pine		16	1.73	30.4
		24	1.66	31.3

V. FORMULATION OF THE RELIABILITY ANALYSES

Structural reliability is defined as the ability of a structure to fulfill its design purpose (Thoft-Christensen and Baker, 1982). This definition implies that the structure will not reach a limit state during the specified reference period under the loading conditions encountered. The defined reference period should be similar to the anticipated useful life of the structure. Often for light-frame wood structures, a 50-year life is assumed.

The measure of structural reliability is usually expressed in terms of reliability index (β). Reliability index, also termed as the safety index, is a measure of the safety of the system.

The P_f is related to β by,

$$P_f = 1 - \Phi\ (\beta) \tag{63}$$

where Φ = standard normal probability distribution function.

Reliability Analysis of Structures

A mathematical model is first derived, which relates load and resistance variables for the limit state of interest (Ellingwood et al 1980). Suppose this relation is given by

$$g(X_1,\ X_2,\ X_3, \ldots ,\ X_n) = 0 \tag{64}$$

in which X_i = load or resistance variables and failure occurs when $g \leq 0$ for any ultimate or serviceability limit state of interest.

The P_f is obtained by integrating the density functions over the regions where $g \leq 0$. Thus,

$$P_f = \int \ldots \int f_X(x_1,\ x_2,\ x_3, \ldots ,\ x_n) dx_1\ dx_2\ dx_3 \ldots dx_n \tag{65}$$

where f_X is the joint density function for X_1, X_2, $X_3, \ldots,\ X_n$.

In most structural problems, the limit state consists of just two random variables, resistance and loading. Suppose the resistance R and loading S, which is dimensionally consistent with R, are the only two random variables occurring in the limit-state equation, then the P_f can be calculated as follows.

$$P_f = P[R - S] < 0 \tag{66}$$

Failure occurs when load exceeds resistance. In Figure 11, the lower tail of the resistance distribution intercepts with the upper tail of the load distribution. The P_f can be obtained as a function of the two distributions between the overlapping regions. By considering a small strip dx in the failure region, the P_f is given by integrating the density function.

$$P_f = \int_0^{\infty} f_S(x)\,dx \int_0^{x} f_R(x)\,dx \qquad (67)$$

where P_f = probability of failure, $f_S(x)$ and $f_R(x)$ are the load and resistance probability density functions (pdf). The notation is simplified by,

$$\int_0^{x} f_R(x)\,dx = F_R(x) \qquad (68)$$

where F_R = cumulative distribution function (cdf) in R.

Then, the probability of failure can be stated as,

$$P_f = \int_0^{\infty} F_R(x)\; f_S(x)\,dx \qquad (69)$$

Thus, for any distributions representing load and resistance, the integral of equation 69 must be evaluated to determine P_f.

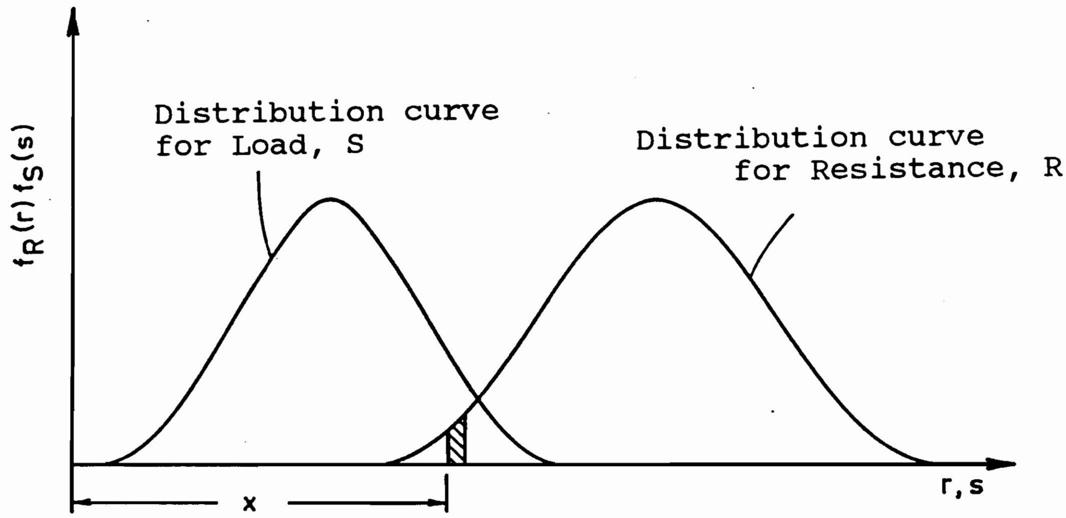


Figure 11. Structural Safety Diagram

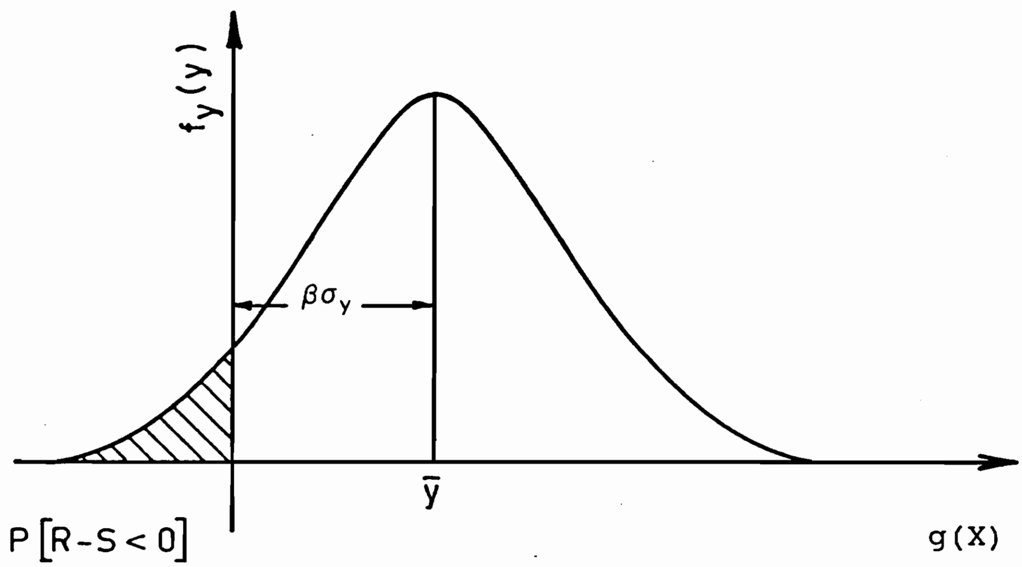


Figure 12. Graphical Representation of the Reliability Index with Failure Function, Y = R - S

If both the load and resistance follow the normal distribution and are statistically independent, then the quantity (R - S) is normally distributed as well. Thus, a new random variable Y, can be defined with mean and variance as given by equations 70 and 71.

$$\bar{Y} = \bar{R} - \bar{S} \tag{70}$$

$$\sigma_Y^2 = \sigma_R^2 + \sigma_S^2 \tag{71}$$

The P_f given the distribution Y can be evaluated as,

$$P_f = \frac{1}{\sqrt{2\pi}\,\sigma_y} \int_{-\infty}^{0} \exp\ [-\frac{1}{2}\ [\frac{y - \bar{Y}}{\sigma_y}]^2]dx \tag{72}$$

Equation 72 is the area under the normal curve between $-\infty$ and 0 and represents failure probability, thus P_f is given by,

$$P_f = \Phi[-\frac{\bar{Y}}{\sigma_y}] \tag{73}$$

From equations 63 and 73, β is given by,

$$\beta = \frac{\bar{Y}}{\sigma_y} \tag{74}$$

Then according to equation 74, β is defined as the number of standard deviations between the mean and the origin. An increase in $\bar{Y}$ leads to an increase in reliability and conversely a lower P_f. Thus, β provides a quantitative measure of structural reliability given by P_f. The shaded portion in Figure 12 represents the region where failure occurs.

In most structural reliability analyses, it is assumed that all uncertainties in design are combined in $f_X(x)$. However, in reality $f_X(x)$ are seldom known for lack of data. In some cases, the mean and variance may be the only known distributional characteristics. Also, the limit-state equation may be nonlinear with the failure boundary being a surface. In these cases, it becomes impractical to perform the integration necessary to evaluate equation 65.

Duration of Load

A duration-of-load factor was not included in the reliability analyses of wall and floor systems presented here because the random variables in load and resistance were not treated as time-dependent (stochastic process) variables. In addition, the limit-state equation in strength was minimized with respect to a certain reference period. To include load duration, the limit-state equation or the failure function must be modified by the load-duration factor corresponding to the number of years, months, days or hours for which the reliability is required.

First-Order Second-Moment Method

The numerical difficulties of integrating equation 65 led to the development of the first-order second-moment reliability analysis method. It is so called because the random variables are characterized by their first and second moments (mean and standard deviation respectively). Work on the first-order second-moment reliability method was started in the late 1960's. For this study, the reliability analysis is performed using the advanced first-order second-moment method of Rackwitz and Fiessler (1978).

The method by Rackwitz and Fiessler (1978) consists of approximating the limit-state failure function by means of a Taylor's expansion. Suppose the failure function is given by, $Y = g(X)$, where $g(X)$ consists of a single variable. Upon using a Taylor's expansion,

$$Y = g(x^*) + (X - x^*)\frac{\partial g}{\partial X^*} + \frac{(X - x^*)^2}{2}\frac{\partial^2 g}{\partial X^2} + \ldots \quad (75)$$

x^* is the value X at which the approximation is taken, $\partial g/\partial X^*$ is the first derivative of $g(X)$, evaluated at x^*. Neglecting second order terms results in a first-order approximation,

$$Y = g(x^*) + (X - x^*) \frac{\partial g}{\partial X^*} \tag{76}$$

Also the non-normal variables are transformed into an equivalent normal distribution, such that the cumulative probability and the probability density are equal for the actual and the approximating normal variables. As with other first-order second-moment methods, β is defined as the shortest distance from the failure surface to the origin. More detailed information is given by Ellingwood et al (1980) and Ang and Tang (1984). However, a brief summary of the algorithm used to evaluate β for a wall or floor system is given here.

Step 1. Establish a failure surface or define an appropriate limit-state equation.

$$Y = g(X) = g(X_1, X_2, X_3, \ldots, X_n) \tag{77}$$

Using a Taylor's expansion and neglecting second order terms, the limit-state equation becomes,

$$Y = g(x_1^*, x_2^*, x_3^*, \ldots, x_n^*) + \sum_{i=1}^{n} [X_i - x_i^*] \frac{\partial g}{\partial X_i} \tag{78}$$

here $\partial g/\partial X_i$ is the first derivative of g(X), evaluated at $X_i = x_i^*$, and x_i^* is the value X_i at which the approximation is taken, and this is usually the mean value of X or the design point value.

The mean value of $\bar{Y}$ can be obtained as,

$$Y = \sum_{i=1}^{n} [X_i - x_i^*] \frac{\partial g}{\partial X_i} \tag{79}$$

Because $g(x_1^*, x_2^*, x_3^*, \ldots., x_n^*) = 0$ on the failure surface, the standard deviation is given by,

$$\sigma_y = \sqrt{\sum_{i=1}^{n} [\frac{\partial g}{\partial X_i} \sigma_i^N]^2} \tag{80}$$

Step 2. Estimate a value for the reliability index.

Step 3. Set the initial values for all the random values as equal to the mean values of the random variables.

Step 4. Transform the non-normal variables into equivalent normal variables. This is done by equating the cdf and the pdf of the normal function to the cdf and pdf of the non-normal distribution function. This process is done in order to arrive at the mean and standard deviation of the equivalent normal distributions.

Equating the cdf's of the normal and non normal distribution functions,

$$\Phi \frac{[X_i^* - \overline{X}_i^N]}{\sigma_i^N} = F(X_i^*) \qquad (81)$$

where $F(X_i)$ = cdf of the normal function.

Rearranging equation 81,

$$\frac{[X_i^* - \overline{X}_i^N]}{\sigma_i^N} = \Phi^{-1}F(X_i^*) \qquad (82)$$

The mean value of the equivalent normal distribution is obtained from equation 82,

$$\overline{X}_i^N = X_i^* - [\Phi^{-1}F(X_i^*)]\ \sigma_i^N \qquad (83)$$

Now, equating the corresponding pdf, the standard deviation of the equivalent normal distribution is obtained,

$$\frac{1}{\sigma_i^N} \phi[\frac{X_i^* - \overline{X}_i^N}{\sigma_i^N}] = f(X_i^*) \qquad (84)$$

where ϕ = pdf of the standard normal distribution, $\overline{X}_i^N$ and σ_i^N are the mean and standard deviation of the equivalent normal distributions. $F(X_i)$ and $f(X_i)$ are the non-normal distribution and density functions of X.

The standard deviation of the equivalent normal from equation 84 can be obtained as,

$$\sigma_i^N = \frac{\phi[\Phi^{-1}F(X_i^*)]}{f(X_i^*)} \tag{85}$$

Step 5. Compute the partial derivatives $\partial g/\partial X_i$ for all the random variables in the limit-state function.

Step 6. Compute the direction cosines, called the sensitivity factors, α_i. A measure of the contribution of any variable X_i to the standard deviation σ_y is its sensitivity factor, which is the ratio,

$$\alpha_i = \frac{\frac{\partial g}{\partial X_i}\sigma_i^N}{\sigma_y} \tag{86}$$

α_i is then incorporated into equation 80. This is done in order to express σ_y as a function of α_i.

$$\sigma_y^2 = \sum_{i=1}^{n}\alpha_i\sigma_y\frac{\partial g}{\partial X_i}\sigma_i^N \tag{87}$$

or the standard deviation σ_y is,

$$\sigma_y = \sum_{i=1}^{n}\alpha_i\frac{\partial g}{\partial X_i}\sigma_i^N \tag{88}$$

Step 7. The mean value of Y and σ_y are substituted in equations 79 and 88, to obtain a new design point x_i^*

$$\beta = \frac{Y}{\sigma_y} = \frac{\sum_{i=1}^{i=n}(X_i - x_i^*)\frac{\partial g}{\partial X_i}}{\sum_{i=1}^{n}\alpha_i \frac{\partial g}{\partial X_i}\sigma_i^N} \tag{89}$$

Rearranging equation 89,

$$\sum_{i=1}^{n} \frac{\partial g}{\partial X_i}[(X_i - x_i^*) - \alpha_i \beta \sigma_i^N] = 0 \tag{90}$$

The new values of x_i^* can be computed from equation 90,

$$[(X_i - x_i^*) - \alpha_i \beta \sigma_i^N] = 0 \tag{91}$$

The value of design point x_i^* which satisfies this equation for all values of i is,

$$x_i^* = X_i - \alpha_i \beta \sigma_i^N \tag{92}$$

Step 8. Repeat steps 4 to 7 until the values of α_i stabilize.

Step 9. Compute the value of β needed for $g(x_1^*, x_2^*, x_3^*, \ldots, x_n^*) = 0$. Normally, convergence is obtained in four to five cycles.

VI. THE RELIABILITY ANALYSES OF FLOOR AND WALL SYSTEMS WITH COMPOSITE ACTION AND LOAD SHARING

To evaluate reliability levels in floor and wall systems, the reliability procedure developed in chapter V is combined with the bending stress equations developed in chapter III for floor and wall systems, and the external loading functions. Two cases are considered for the reliability analyses for wall and floor systems. The first is the reliability analysis of a system having just composite action, and the second case is a reliability analysis combined with composite action and load sharing.

Reliability Analyses of Floor Systems

Floor Systems with Composite Action

In light-frame structures, structural interactions in the form of composite action must be considered to accurately model the floor behavior. The behavior of the floor system is represented by one characteristic event. The characteristic event in the floor system corresponds to the critical limit state in the joist, and the critical limit state is described by the bending stress criteria as given by equation 62. A single limit state was deemed adequate because the stress strain relations in framing member were defined linearly.

Using the reliability procedure developed in chapter V in combination with the bending stress equation for the floor system from chapter III the failure function g(x) was formulated as,

$$g(x) = (\text{MOR} - \text{Acting Stress}) \tag{93}$$

where MOR = modulus of rupture of the joist and the acting stress on the floor system is a function of the external loading. More details about the floor properties and loading are given in Appendix C.

Minimizing equation 93 subjected to the constraint g(x) = 0, gives the minimum distance from the failure surface to the origin, which is the β.

Results of Reliability Analyses

The β and P_f for floors are given in Table 14. Including composite action in the analysis of the floor system improves the performance, β increased from 3.31 to 3.86 for 16-in. joist spacing. The corresponding P_f of a joist decreased from 1 in 2100 to 1 in 17500 for No.1 joists spaced 16 in. on center. When the 24 in. joist spacing was used with No.1 materials, β increased from 2.76 to 3.35, the corresponding P_f of a joist decreased from 1 in 330 to 1 in 2500.

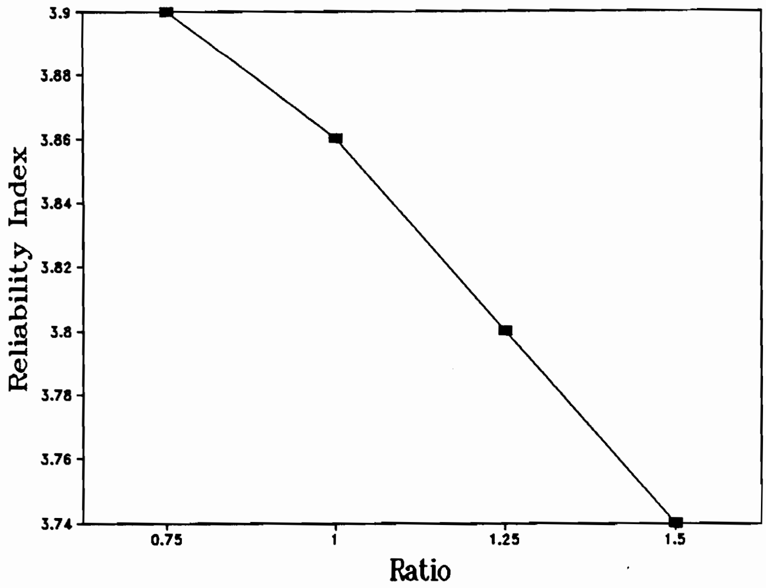
Similarly for grade No.2 joists, β increased from 2.48 to 3.30 when composite action was included. When 24-in. joist spacing was used, β increased from 1.61 to 2.55, and the corresponding P_f of joist decreased from 1 in 16 to 1 in 165. In general, by including composite action floor system performance improved. On the average, composite action lead to a 19% increase in β.

Changes in coefficient of variation for live load and stud strength were studied. Figure 13 presents the effect of coefficient of variation on β for the live load and lumber modulus of rupture. The ratio is a numerical expression relating a supposed coefficient of variation to the actual coefficient of variation. It was observed that lower variability decreased the probability of floors reaching the limit state and higher variability increased the chances of failure. For example, a 10% decrease in the coefficient of variation of the joist modulus of rupture increased β from 3.82 to 4.62. A 10% increase in the coefficient of variation of the joist modulus of rupture decreased β from 3.62 to 3.30.

Table 14. β and P_f for No.1 and No.2 Douglas-fir 2×8 Floor Systems with Composite Action and no Load Sharing

Joist Spacing (in)	CA[a]	Grade No.1		Grade No.2	
		β	P_f	β	P_f
16	Yes	3.86	$5.70*10^{-5}$	3.30	$4.80*10^{-4}$
16	No	3.31	$4.70*10^{-4}$	2.48	$6.60*10^{-3}$
24	Yes	3.35	$4.00*10^{-4}$	2.55	$5.40*10^{-3}$
24	No	2.76	$2.90*10^{-3}$	1.61	$5.40*10^{-2}$

[a] = composite action



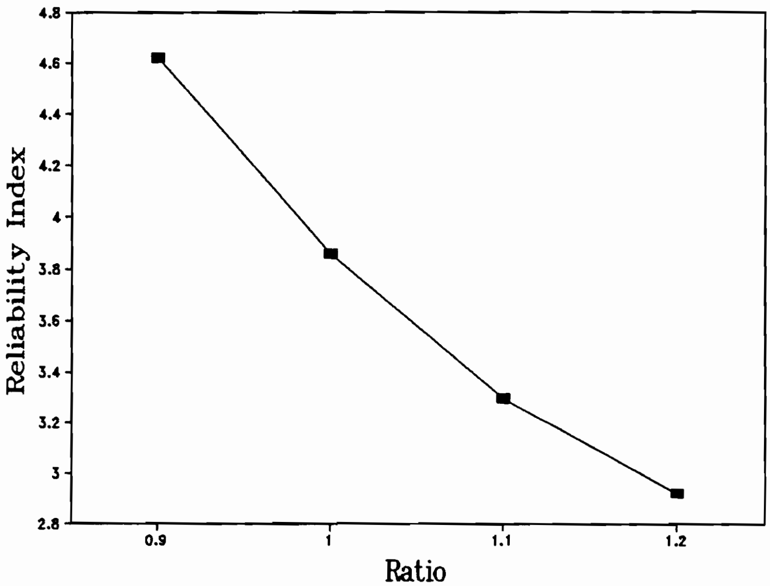


Figure 13. Effect of Change in the Coefficient of Variation (Ratio = Supposed CV/Actual CV) on Reliability Index for Floors (Douglas-fir, No.1, 2×8, 16 in. on center) with Composite Action and no Load Sharing (a) Live Load, (b) MOR

Floor Systems with Composite Action and Load Sharing

The mathematical modeling of load sharing given in chapter IV is adopted without the gypsum board covering on the tensile side. The joists in the floor system were modeled as a series of elastic springs, and the spring stiffness incorporates the composite action and material properties. The spring stiffness is given by,

$$k_i = \frac{wL}{\left(\frac{5}{384}\right)\frac{wL^4}{E_i I_w}} \tag{94}$$

here k_i is the spring stiffness of the framing member, w is the external load in lb/in., E_i is the stiffness of the individual framing member, L is length of the framing member and I_w is the composite stiffness of the floor system.

Floor systems are considered with eight joists spaced equally at 16 in. or 24 in. on center, simply supported having a span of 123 in.

$$\sum_{ij}^{8} a_{ij}\, \Delta_j = w_i \tag{95}$$

where Δ_j are deflections at each of the joist members, and the coefficients are given by,

$$a_{ij} = [k_j x_i (l - x_j)(2lx_j - x_j^2 - x_i^2)] \quad \text{for } i < j \tag{96}$$

$$a_{ij} = [k_j x_j (l - x_i)(2lx_i - x_i^2 - x_j^2)] \quad \text{for } i > j \tag{97}$$

$$w_i = w\, x_i l \frac{[l^3 - 2lx_i^2 + x_i^3]}{4} \quad \text{for } i = 1,8 \tag{98}$$

Equation 95 is solved to determine the unknown deflection quantities. Once the deflections are obtained, the stresses developed in each framing member were evaluated. Then, load sharing was computed as the ratio of the highest load carried by the weakest framing member with respect to the gravity load prescribed by the Uniform Building Code (1988).

The load-sharing factors obtained in the analyses of Chapter IV were multiplied with the MOR in the reliability analysis program to obtain failure probability of floor systems. Thus the modified failure function looks like,

$$g(x) = \{(LSF)\ MOR - Acting\ Stress\} \tag{99}$$

where LSF = load-sharing factor.

Results of Reliability Analyses

Including composite action and load sharing in the reliability analyses, improved the β of the floor system. Table 15 presents the β and P_f for 16 in. and 24 in. joist spacings for Douglas-fir joist floors for grade No.1 and grade No.2. By comparing the reliability between systems having composite action with load sharing and composite action without load sharing, it was shown that composite action with load sharing has a substantial effect on the β and P_f. The increase in reliability was almost of the same pattern in joist categories of No.1 and No.2 grades. Furthermore, it was also shown that joist grade No.1 performed much better than joist grade No.2 with respect to β. For a 16-in. joist spacing with composite action and load sharing, joist grade No.1 had a failure probability of 1 in 50,000, whereas the failure probability for joist grade No.2 was 1 in 2967.

These results indicate that including composite action and load sharing greatly reduces the failure probability of floor systems. Based on this, it can be concluded that floor types with 16 in. joist spacing or

24 in joist spacing and of either grade No.1 or grade No.2 would be safe and can easily sustain the design loads.

Table 15. β and P_f for No.1 and No.2 Douglas-fir 2×8 Floor Systems with Composite Action and Load Sharing

Joist Spacing (in)	CA[a]	LS[b]	Grade No.1		Grade No.2	
			β	P_f	β	P_f
16	No	No	3.31	$4.70*10^{-4}$	2.48	$6.60*10^{-3}$
16	Yes	No	3.86	$5.70*10^{-5}$	3.30	$4.80*10^{-4}$
16	Yes	Yes	4.24	$2.00*10^{-5}$	3.40	$3.37*10^{-4}$
24	No	No	2.76	$2.90*10^{-3}$	1.61	$5.40*10^{-2}$
24	Yes	No	3.35	$4.00*10^{-4}$	2.55	$5.40*10^{-3}$
24	Yes	Yes	3.53	$2.10*10^{-4}$	2.55[c]	$5.40*10^{-3}$

[a] = composite action,
[b] = load sharing,
[c] = stage 3 governs

Summary of Floor Systems

For typical light-frame floor systems, composite action and load sharing contribute to yield a system that reaches the assigned limit state for load at level 1.19 to 1.96 greater than allowed by the current design procedure.

Probability of failure was influenced by load sharing, composite action and variability of the framing lumber. Failure probability of the floor system with 16-in. joist spacing (Douglas-fir No.1) was 1 in 50,000 when load sharing and composite action were included. The probability of failure increased to 1 in 17,000 when only composite action was included and load sharing was neglected. When both the composite action and load sharing were neglected failure probability was in the region of 1 in 2100. These studies demonstrate that composite action and load sharing significantly contribute to the strength and stiffness of floor systems. In addition, the 15% increase in the allowable stresses for repetitive light-frame members specified by the (NDS 1991) apparently is conservative. Based on these analyses, it is concluded that floors with 16-in. or 24-in. joist spacings are safe and can safely and reliably sustain the design loads.

Reliability Analyses of Wall Systems

Defining the Characteristic Events

In a wood-wall system, the exterior wall covering (plywood sheathing) undergoes compressive forces while the interior covering (gypsum) is subjected to tensile forces. An evaluation of wall system data (Polensek and Kazic 1991) revealed that a wall system under axial load and lateral loadings exhibits four characteristic events. Furthermore, it is emphasized that all the four characteristic events happen under one extreme loading condition. First, a proportional limit occurs in tensile joints between the framing member and gypsum sheathing, and this is followed by a similar occurrence in the compressive joints between the plywood sheathing and the framing member. Finally the framing lumber reaches a proportional limit and then an ultimate limit. This sequence of four events was adopted in the reliability analysis and is shown in Figure 14 as a piecewise linear curve.

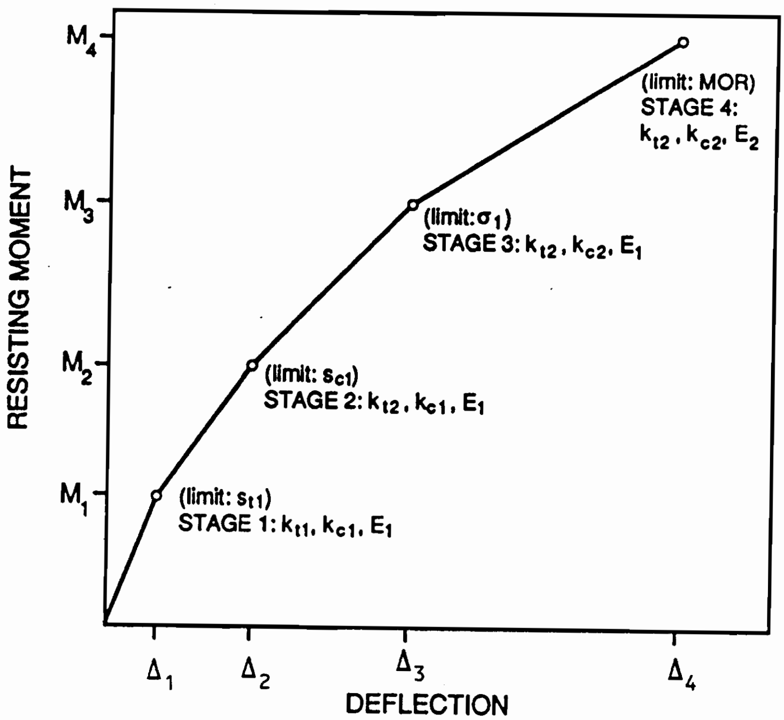
In stage 1, the limiting moment of resistance is M_1, governed by k_{t1}, k_{c1} (tensile and compressive interlayer slip moduli) and E_1 (framing member). When the limiting moment is reached the slip modulus changes from k_{t1} to k_{t2} while other properties remain the same. The resisting moment, $M_1 = M_{max}$, at this point is obtained from equation 45 with $slip_{st} = s_{t1}$, substituting the resulting M_1 into equation 43 and 41 gives the stress and the corresponding maximum deflection reached in the wall system.

In stage 2, slip moduli k_{t2}, k_{c1} and E_1 dominate, and the upper limit is defined by the compressive slip reaching $slip_{sc}$, the limiting moment of resistance is characterized by the difference M_2-M_1, in which M_2 is based on equation 44 with $slip_{sc} = s_{c1}$.

Moduli k_{t2}, k_{c2} and E_1 govern stage 3, and this is reached when the stress in the framing member reaches the proportional limit (σ_1). The limiting moment of resistance M_3 is obtained by substituting $\sigma_s = \sigma_1$ in equation 43 and solving it for M_{max}. The corresponding slips and deflection are obtained by substituting this M_{max} into equations 44, 45 and 41 respectively.

In stage 4 moduli k_{t2}, k_{c2} and E_2 govern the final stage and the upper limit is reached when the stress in the framing member becomes equal to the rupture stress (MOR). The corresponding M_4 follows from equation 43 with σ_s = MOR and slips based on equations 44 and 45.

(a)



(b)

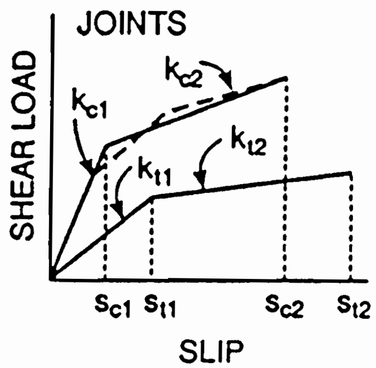


Figure 14. Characteristic Events in Wall Behavior as used for Reliability Analyses, (a) Resisting-Load Parameters, and (b) Nonlinear Stiffness Properties of Nailed Joints (Polensek and Kazic 1991)

Wall Systems with Composite Action

Any stage shown in Figure 14 is considered to fail when the resisting moment offered by the system is less than the maximum moment caused by the external loads. The failure function is expressed as the difference between the resisting moment and the moment caused by the external loads.

$$g(x) = M_m - M_{max} = 0 \qquad (100)$$

where g(x) is the failure function, M_m is the resisting moment and subscript m represents the four stages, and M_{max} is the external moment. Complete failure occurs when the stud reaches stage 4, and the intermediate stages show the amount of strength and stiffness left when the wall system is characterized by the nonlinear behavior. The failure function for any stage consists of defining the material and geometric properties of the wall system and the external moment. The evaluation of the reliability index consists of minimizing the failure function subject to the constraint g(x) = 0.

Four cases were considered in the analysis of Douglas-fir and southern pine walls. The first was a wall system with both composite action and axial load, the second a wall system with composite action but no axial load, the third with axial load but no composite action, and the fourth was a wall system with neither composite action nor axial load. In real situations the effects of axial load and composite action cannot be separated, however to study the effect of composite action and axial load on wall systems certain changes in the computer program was made. For example, when composite action and axial load were not needed the slip modulus and axial load values were given a value of 0. An example of one iteration of the failure function is given in Appendix E.

Results of Reliability Analyses

Reliability index and failure probability for all four stages of the wall system are given. Tables 16-19 are results for 16 in. stud spacing, while Tables 20-23 give the results for 24 in. stud spacing.

Effect of composite action and axial loads. By including composite action in the analysis, the wall system reliability performance was improved. Failure probabilities decreased from 3 studs in 5000 to 1 stud in 5000 for Douglas-fir framing type. Similar results were seen for the southern pine framing with failure probabilities decreasing from 4 in 1000 to 2 in 1000. It was found that in all the four cases that stage 1 failure probability was certain with a probability level equal to

1.0 or in other words that stage 2 was reached with a probability level equal to 1.0. In the case when no composite action was considered stage 1 and stage 2 probability were certain and reached stage 3 with probability equal to 1.0.

Stage 2 had a range for P_f of 1 in 25 to 1 in 83 and the corresponding β between 1.76 to 2.26, for 16 in. stud spacing, under a 50-yr wind load which includes composite action, load sharing and both the Douglas-fir and southern pine species. It should be noted that failure of stage 2 does not mean stud failure, but yielding of the stud from stage 2 to stage 3. Stud failure occurs when the system has reached stage 4.

Stage 3 had a range for P_f of 1 in 500 to 1 in 5814 and the corresponding β between 2.88 to 3.58, once again for 16 in. stud spacing, under a 50-yr wind load which includes composite action, load sharing and both the Douglas-fir and southern pine species. In all the cases, P_f and β for stage 4 was equal to stage 3. This is perhaps attributed to the high coefficient of variation occurring in MOR, which led to the framing members failure to reach stage 4.

Effect of materials. It was observed that walls with Douglas-fir framing type members had a low failure probability when compared to southern pine. However, this does not suggest that framing constructed with different species performs better, because the wind load was different. The effect of wind return period is given in the next section.

When comparisons between 16 in. and 24 in. stud spacing were made, for Douglas-fir As-graded studs, failure probability increased from 1 in 5000 to 1 in 200, which included composite action and axial load. Failure probability and β for stud spacing of 24 in. are given in Tables 20-23.

It was also seen that walls with As-graded studs performed about the same as those with On-grade studs. Errors in misgrading at lumber mills does not affect the structural reliability or performance of wall systems.

Effect of wind return period. The effect of return period in wind velocity were studied to see the behavior of wall systems. The results are presented in Tables 16-19. The reliability of walls with Douglas-fir framing type were higher when compared to southern pine due to different wind load. The 50-yr wind load considered for Douglas-fir stud walls had a mean value of 71.47 mi/hr with a coefficient of variation of 14.2%, whereas for walls with southern pine had wind load of velocity 51.0 mi/hr and a coefficient of variation of 33.7%. Though the mean wind load for walls with southern pine studs is small, the larger coefficient of variation affects the performance of the wall system and results in higher failure rate for southern pine walls than Douglas-fir stud walls. However, in all four cases failure, probability of walls constructed with either Douglas-fir or southern pine framing members is low, P_f being 1 in 250 to 1 in 50,000 range. Based on this, one can conclude that wall types with 16 in. stud spacing and of either species would be safe under a 50-yr wind load. When comparisons between 50-yr and 100-yr wind load were made, for Douglas-fir As-Graded studs, failure probability increased from 1 in 5000 to 1 in 2500, which includes composite action and axial load. It was seen that walls sustained a 100-yr wind load without any appreciable decrease in failure rate.

Sensitivity study. A sensitivity study was undertaken to find which of the parameters (wind velocity V, exposure coefficient K_z, and the pressure coefficient C_p) affected the performance of the wall system. Also, the effects of variations in stud strength and stiffness on β of wall systems were analyzed. It was found that wind velocity has a greater potential influence on the performance of walls than the pressure coefficient C_p or exposure coefficient K_z. Ratio of the supposed coefficient of variation to the actual coefficient of variation in C_p and K_z had a very little effect on the performance of walls. Reliability index versus coefficient of variation and probability of failure are shown in Figure 14 and Table 24. Wind velocity (V) and combined (V, C_p and K_z) had a major role in affecting the walls reaching the collapse stage. Lower variation in wind velocity caused the wall to perform better. A 25% less variation in wind velocity gave a failure probability of 1 in 50,000.

Changes in coefficient of variation with respect to initial conditions for stud strength and stiffness, and stud strength alone were studied on walls with Douglas-fir framing. The resulting failure probabilities and β are given in Figure 15 and Tables 25-26. As expected, it was observed that lower variability decreases the probability of studs reaching stage 4 and higher variation increases the chances of failure. Stage 1 failure probability was certain and was not affected even when the variations in strength and stiffness were reduced by 50 percent. Stage 2, stage 3 and stage 4 failure probabilities increased or decreased, in correspondence with the increase or decrease in coefficient of variation of strength and stiffness with respect to the initial conditions.

Table 16. β and P_f for Douglas-fir As-Graded Studs Spaced 16 in. on Center, Subjected to 3.29 and 3.66 lb/in. Wind Loads[a]

Analysis Conditions			50-yr Return Period		100-yr Return Period	
CA[b]	AL[c]	Stage	β	P_f	β	P_f
Yes	Yes	1	-	1.0	-	1.0
		2	2.26	$1.20*10^{-2}$	2.02	$2.17*10^{-2}$
		3	3.55	$1.93*10^{-4}$	3.38	$3.63*10^{-4}$
		4	3.55	$1.93*10^{-4}$	3.38	$3.63*10^{-4}$
Yes	No	1	-	1.0	-	1.0
		2	2.57	$5.10*10^{-3}$	2.35	$9.39*10^{-3}$
		3	3.89	$5.00*10^{-5}$	3.74	$9.20*10^{-5}$
		4	3.89	$5.00*10^{-5}$	3.74	$9.20*10^{-5}$
No	Yes	1	-	1.0	-	1.0
		2	-	1.0	-	1.0
		3	3.26	$5.57*10^{-4}$	3.08	$1.03*10^{-3}$
		4	3.26	$5.57*10^{-4}$	3.08	$1.03*10^{-3}$
No	No	1	-	1.0	-	1.0
		2	-	1.0	-	1.0
		3	3.55	$1.93*10^{-4}$	3.39	$3.50*10^{-4}$
		4	3.55	$1.93*10^{-4}$	3.39	$3.50*10^{-4}$

[a] = North Head, WA
[b] = composite action,
[c] = axial load

Table 17. β and P_f for Douglas-fir On-Grade Studs Spaced at 16 in. on Center, Subjected to 3.29 and 3.66 lb/in. Wind Loads[a]

Analysis Conditions			50-yr Return Period		100-yr Return Period	
CA[b]	AL[c]	Stage	β	P_f	β	P_f
Yes	Yes	1	-	1.0	-	1.0
		2	2.24	$1.25*10^{-2}$	2.00	$2.27*10^{-2}$
		3	3.58	$1.72*10^{-4}$	3.42	$3.13*10^{-4}$
		4	3.58	$1.72*10^{-4}$	3.42	$3.13*10^{-4}$
Yes	No	1	-	1.0	-	1.0
		2	2.55	$5.39*10^{-3}$	2.33	$9.90*10^{-3}$
		3	3.93	$4.20*10^{-5}$	3.78	$7.80*10^{-5}$
		4	3.93	$4.20*10^{-5}$	3.78	$7.80*10^{-5}$
No	Yes	1	-	1.0	-	1.0
		2	-	1.0	-	1.0
		3	3.29	$5.01*10^{-4}$	3.12	$9.04*10^{-4}$
		4	3.29	$5.01*10^{-4}$	3.12	$9.04*10^{-4}$
No	No	1	-	1.0	-	1.0
		2	-	1.0	-	1.0
		3	3.59	$1.65*10^{-4}$	3.43	$3.02*10^{-4}$
		4	3.59	$1.65*10^{-4}$	3.43	$3.02*10^{-4}$

[a] = North Head, WA
[b] = composite action,
[c] = axial load

Table 18. β and P_f for Southern Pine As-Graded Studs Spaced at 16 in. on Center, Subjected to 3.34 and 3.99 lb/in. Wind Loads[a]

Analysis Conditions			50-yr Return Period		100-yr Return Period	
CA[b]	AL[c]	Stage	β	P_f	β	P_f
Yes	Yes	1	-	1.0	-	1.0
		2	1.76	$3.92*10^{-2}$	1.46	$7.21*10^{-2}$
		3	2.88	$1.99*10^{-3}$	2.67	$3.79*10^{-3}$
		4	2.88	$1.99*10^{-3}$	2.67	$3.79*10^{-3}$
Yes	No	1	-	1.0	-	1.0
		2	2.01	$2.22*10^{-2}$	1.74	$4.09*10^{-2}$
		3	3.19	$7.11*10^{-4}$	2.99	$1.39*10^{-3}$
		4	3.19	$7.11*10^{-4}$	2.99	$1.39*10^{-3}$
No	Yes	1	-	1.0	-	1.0
		2	-	1.0	-	1.0
		3	2.68	$3.68*10^{-3}$	2.47	$6.75*10^{-3}$
		4	2.68	$3.68*10^{-3}$	2.47	$6.75*10^{-3}$
No	No	1	-	1.0	-	1.0
		2	-	1.0	-	1.0
		3	2.94	$1.64*10^{-3}$	2.74	$3.07*10^{-3}$
		4	2.94	$1.64*10^{-3}$	2.74	$3.07*10^{-3}$

[a] = Key West, FL
[b] = composite action,
[c] = axial load

Table 19. β and P_f for Southern Pine On-Grade Studs Spaced at 16 in. on Center, Subjected to 3.34 and 3.99 lb/in. Wind Loads[a]

Analysis Conditions			50-yr Return Period		100-yr Return Period	
CA[b]	AL[b]	Stage	β	P_f	β	P_f
Yes	Yes	1	-	1.0	-	1.0
		2	1.76	$3.92*10^{-2}$	1.46	$7.21*10^{-2}$
		3	2.95	$1.59*10^{-3}$	2.75	$2.98*10^{-3}$
		4	2.95	$1.59*10^{-3}$	2.75	$2.98*10^{-3}$
Yes	No	1	-	1.0	-	1.0
		2	2.02	$2.17*10^{-2}$	1.74	$4.09*10^{-2}$
		3	3.24	$5.98*10^{-4}$	3.05	$1.14*10^{-3}$
		4	3.24	$5.98*10^{-4}$	3.05	$1.14*10^{-3}$
No	Yes	1	-	1.0	-	1.0
		2	-	1.0	-	1.0
		3	2.76	$2.89*10^{-3}$	2.55	$5.39*10^{-3}$
		4	2.76	$2.89*10^{-3}$	2.55	$5.39*10^{-3}$
No	No	1	-	1.0	-	1.0
		2	-	1.0	-	1.0
		3	2.99	$1.39*10^{-3}$	2.80	$2.55*10^{-3}$
		4	2.99	$1.39*10^{-3}$	2.80	$2.55*10^{-3}$

[a] = Key West, FL
[b] = composite action,
[c] = axial load

Table 20. β and P_f for Douglas-fir As-Graded Studs Spaced at 24 in. on Center, Subjected to 4.94 and 5.48 lb/in. Wind Loads[a]

Analysis Conditions			50-yr Return Period		100-yr Return Period	
CA[b]	AL[c]	Stage	β	P_f	β	P_f
Yes	Yes	1	-	1.0	-	1.0
		2	1.13	$1.29*10^{-1}$	0.83	$2.03*10^{-1}$
		3	2.63	$4.27*10^{-3}$	2.46	$6.95*10^{-3}$
		4	2.63	$4.27*10^{-3}$	2.46	$6.95*10^{-3}$
Yes	No	1	-	1.0	-	1.0
		2	1.45	$7.35*10^{-2}$	1.17	$1.21*10^{-1}$
		3	3.00	$1.35*10^{-3}$	2.82	$2.40*10^{-3}$
		4	3.00	$1.35*10^{-3}$	2.82	$2.40*10^{-3}$
No	Yes	1	-	1.0	-	1.0
		2	-	1.0	-	1.0
		3	2.40	$8.20*10^{-3}$	2.19	$1.43*10^{-2}$
		4	2.40	$8.20*10^{-3}$	2.19	$1.43*10^{-2}$
No	No	1	-	1.0	-	1.0
		2	-	1.0	-	1.0
		3	2.70	$3.46*10^{-3}$	2.50	$6.21*10^{-3}$
		4	2.70	$3.46*10^{-3}$	2.50	$6.21*10^{-3}$

[a] = North Head, WA
[b] = composite action,
[c] = axial load

Table 21. β and P_f for Douglas-fir On-Grade Studs Spaced at 24 in. on Center, Subjected to 4.94 and 5.48 lb/in. Wind Loads[a]

Analysis Conditions			50-yr Return Period		100-yr Return Period	
CA[b]	AL[c]	Stage	β	P_f	β	P_f
Yes	Yes	1	-	1.0	-	1.0
		2	1.10	$1.36*10^{-1}$	0.80	$2.12*10^{-1}$
		3	2.70	$3.47*10^{-3}$	2.50	$6.21*10^{-3}$
		4	2.70	$3.47*10^{-3}$	2.50	$6.21*10^{-3}$
Yes	No	1	-	1.0	-	1.0
		2	1.43	$7.63*10^{-2}$	1.14	$1.27*10^{-1}$
		3	3.07	$1.07*10^{-3}$	2.88	$1.99*10^{-3}$
		4	3.07	$1.07*10^{-3}$	2.88	$1.99*10^{-3}$
No	Yes	1	-	1.0	-	1.0
		2	-	1.0	-	1.0
		3	2.44	$7.34*10^{-3}$	2.23	$1.29*10^{-2}$
		4	2.44	$7.34*10^{-3}$	2.23	$1.29*10^{-2}$
No	No	1	-	1.0	-	1.0
		2	-	1.0	-	1.0
		3	2.75	$2.98*10^{-3}$	2.56	$5.23*10^{-3}$
		4	2.75	$2.98*10^{-3}$	2.56	$5.23*10^{-3}$

[a] = North Head, WA
[b] = composite action,
[c] = axial load

Table 22. β and P_f for Southern Pine As-Graded Studs Spaced at 24 in. on Center, Subjected to 5.01 and 5.98 lb/in. Wind Loads[a]

Analysis Conditions			50-yr Return Period		100-yr Return Period	
CA[b]	AL[c]	Stage	β	P_f	β	P_f
Yes	Yes	1	-	1.0	-	1.0
		2	0.90	$1.84*10^{-1}$	0.55	$2.91*10^{-1}$
		3	2.20	$1.39*10^{-2}$	1.99	$2.33*10^{-2}$
		4	2.20	$1.39*10^{-2}$	1.99	$2.33*10^{-2}$
Yes	No	1	-	1.0	-	1.0
		2	1.18	$1.19*10^{-1}$	0.83	$2.03*10^{-1}$
		3	2.53	$5.70*10^{-3}$	2.32	$1.02*10^{-2}$
		4	2.53	$5.70*10^{-3}$	2.32	$1.02*10^{-2}$
No	Yes	1	-	1.0	-	1.0
		2	-	1.0	-	1.0
		3	2.05	$2.02*10^{-2}$	1.81	$3.51*10^{-2}$
		4	2.05	$2.02*10^{-2}$	1.81	$3.51*10^{-2}$
No	No	1	-	1.0	-	1.0
		2	-	1.0	-	1.0
		3	2.32	$1.02*10^{-2}$	2.10	$1.79*10^{-2}$
		4	2.32	$1.02*10^{-2}$	2.10	$1.79*10^{-2}$

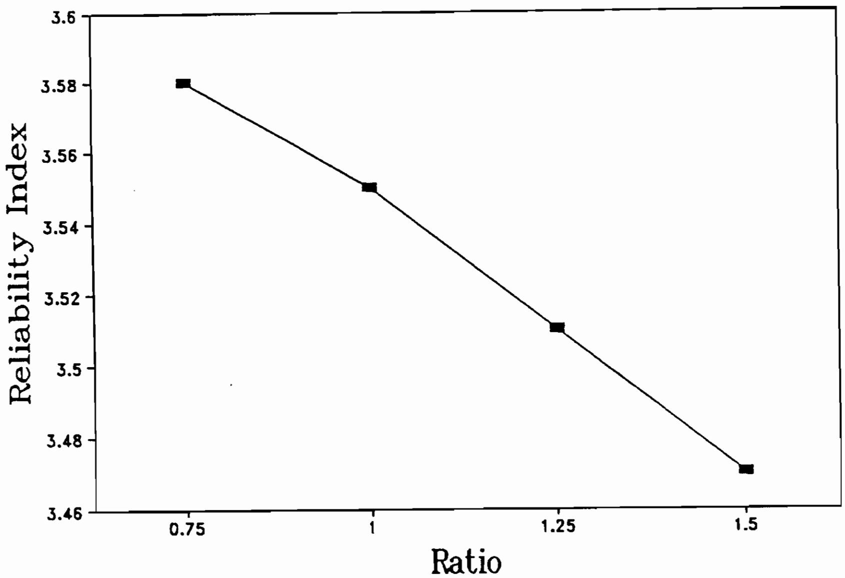
[a] = Key West, FL
[b] = composite action,
[c] = axial load

Table 23. β and P_f for Southern Pine On-Grade Studs Spaced at 24 in. on Center, Subjected to 5.01 and 5.98 lb/in. Wind Loads[a]

Analysis Conditions			50-yr Return Period		100-yr Return Period	
CA[b]	AL[c]	Stage	β	P_f	β	P_f
Yes	Yes	1	-	1.0	-	1.0
		2	0.92	$1.79*10^{-1}$	0.54	$2.95*10^{-1}$
		3	2.33	$9.90*10^{-2}$	2.09	$1.83*10^{-2}$
		4	2.33	$9.90*10^{-2}$	2.09	$1.83*10^{-2}$
Yes	No	1	-	1.0	-	1.0
		2	1.17	$1.21*10^{-1}$	0.82	$2.06*10^{-1}$
		3	2.16	$4.27*10^{-3}$	2.42	$7.76*10^{-3}$
		4	2.16	$4.27*10^{-3}$	2.42	$7.76*10^{-3}$
No	Yes	1	-	1.0	-	1.0
		2	-	1.0	-	1.0
		3	2.16	$1.54*10^{-2}$	1.93	$2.68*10^{-2}$
		4	2.16	$1.54*10^{-2}$	1.93	$2.68*10^{-2}$
No	No	1	-	1.0	-	1.0
		2	-	1.0	-	1.0
		3	2.41	$7.98*10^{-2}$	2.20	$1.39*10^{-2}$
		4	2.41	$7.98*10^{-2}$	2.20	$1.39*10^{-2}$

[a] = Key West, FL
[b] = composite action,
[c] = axial load

(a)



(b)

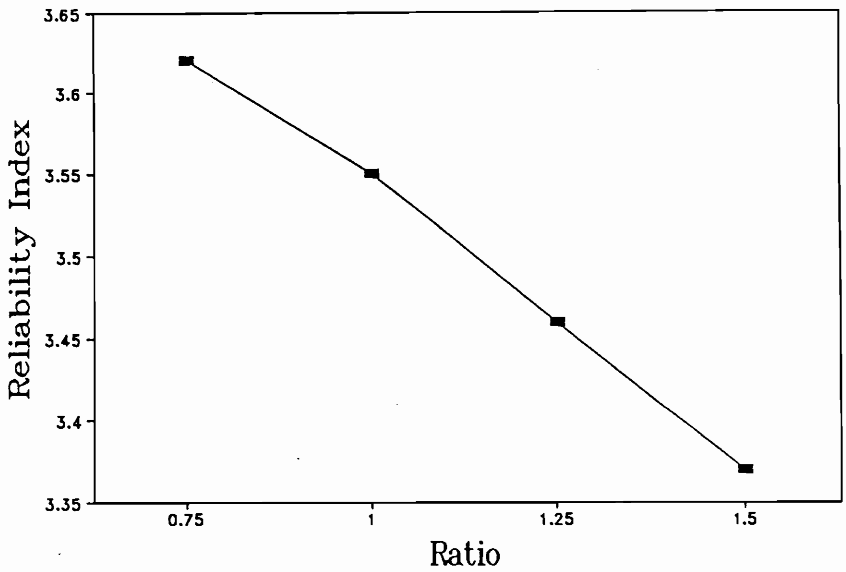
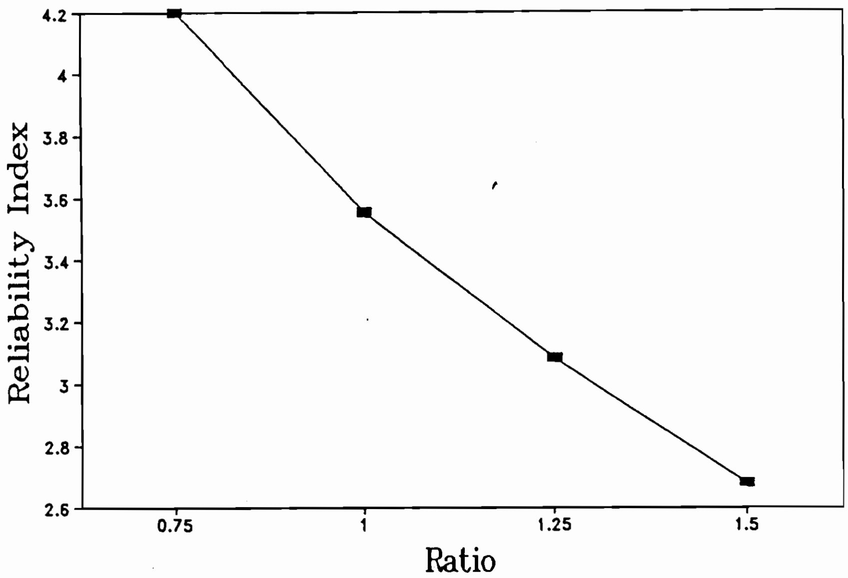


Figure 15. Effect of Change in the Coefficient of Variation (Ratio = Supposed CV/Actual CV) on Reliability Index for Walls (Douglas-fir, As-Graded, 2×4 Studs 16 in. on center) with Composite Action and no Load Sharing (a) External Pressure Coefficient C_p, (b) Exposure Coefficient K_z, (c) Wind Velocity (V), and (d) Combined (V, C_p and K_z)

(c)



(d)

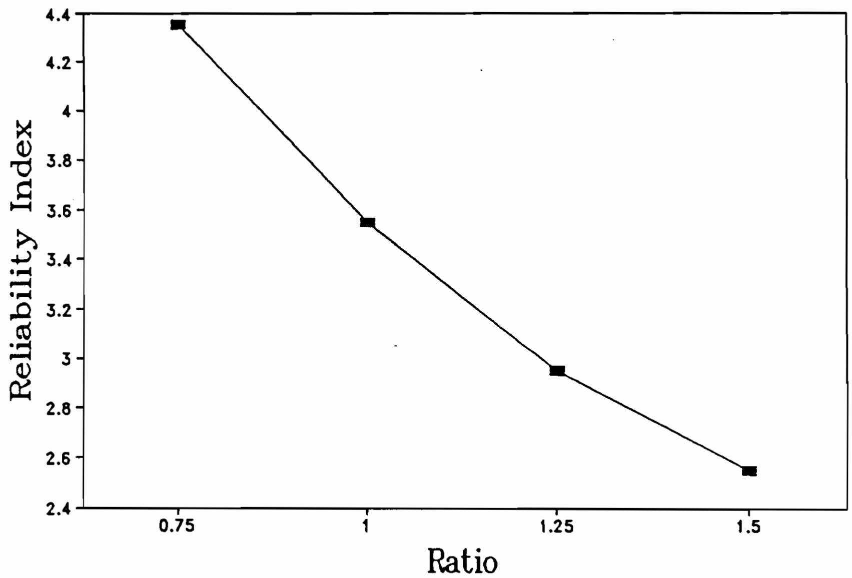


Figure 15. Continued

Table 24. Failure Probabilities[a] of Douglas-fir As-Graded Studs[b] for Percent of Change[c] in Value

Ratio	Probability of Failure[e]			
	Velocity (14.2%)[d]	C_p (13%)[d]	K_z (16%)[d]	V, C_p & K_z
0.75	$2.24*10^{-5}$	$1.72*10^{-4}$	$1.47*10^{-4}$	$7.15*10^{-6}$
1.00	$1.93*10^{-4}$	$1.93*10^{-4}$	$1.93*10^{-4}$	$1.93*10^{-4}$
1.25	$1.07*10^{-3}$	$2.24*10^{-4}$	$2.70*10^{-4}$	$1.49*10^{-3}$
1.50	$3.80*10^{-3}$	$2.60*10^{-4}$	$3.63*10^{-4}$	$5.54*10^{-3}$

[a] includes composite action and axial load
[b] studs spaced at 16 in. on center
[c] defined as the change in coefficient of variation with respect to initial conditions
[d] actual coefficient of variation
[e] under 50-yr wind load at North Head, WA.

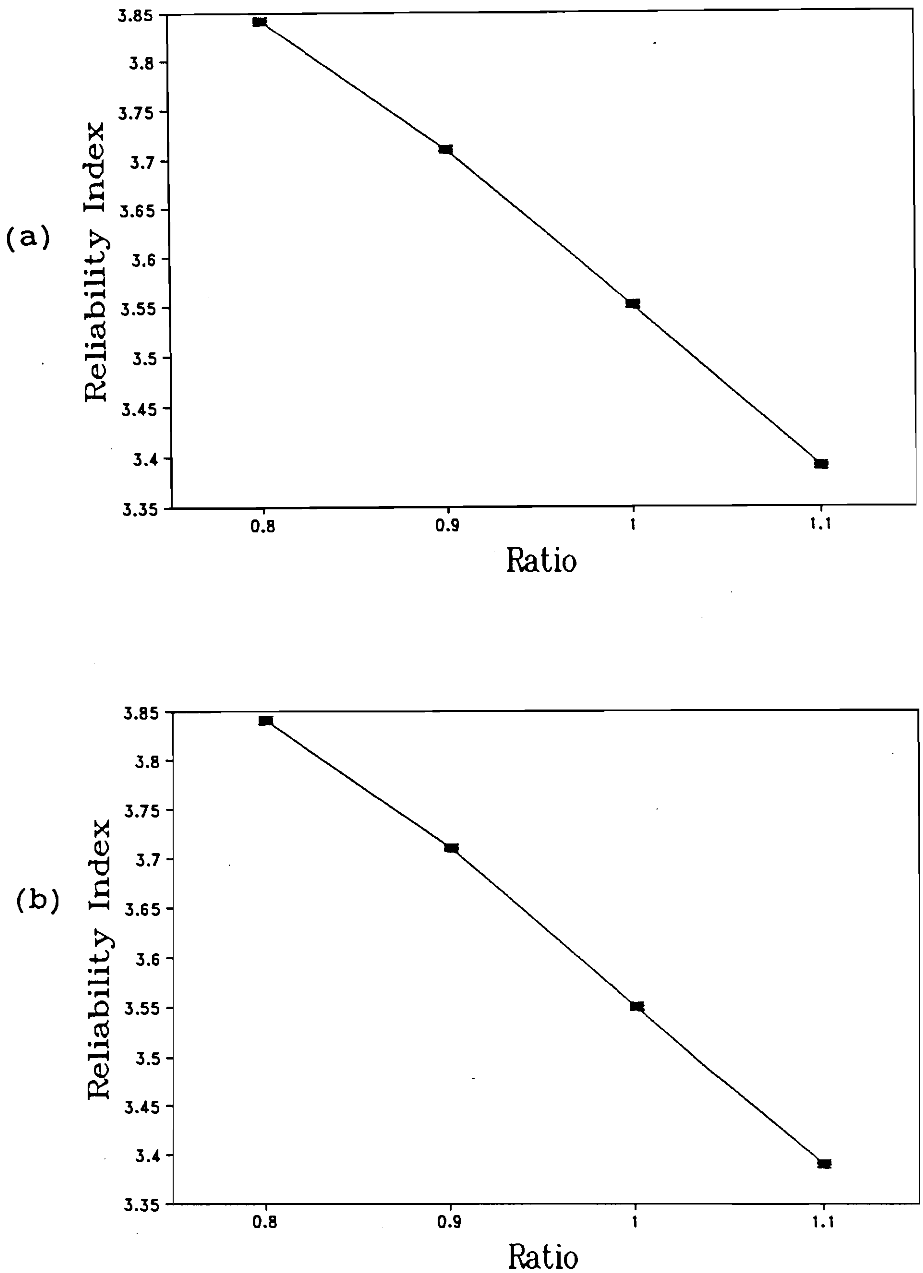


Figure 16. Effect of Change in the Coefficient of Variation (Ratio = Supposed CV/Actual CV) on Reliability Index for Walls (Douglas-fir, As-Graded, 2×4 Studs 16 in. on center) with Composite Action and no Load Sharing, (a) Combined Strength and Stiffness (E_1, E_2, σ_1 and MOR), (b) Strength (σ_1 and MOR)

Table 25. Failure Probabilities[a] of Douglas-fir As-Graded Studs[b] for Percent of Change[c] in Value of E_1, E_2, σ_1 and MOR

Ratio	Probability of Failure[d]			
	Stage 1	Stage 2	Stage 3	Stage 4
0.8	1.0	$1.10*10^{-2}$	$6.20*10^{-5}$	$6.20*10^{-5}$
0.9	1.0	$1.20*10^{-2}$	$1.04*10^{-4}$	$1.04*10^{-4}$
1.0	1.0	$1.20*10^{-2}$	$1.93*10^{-4}$	$1.93*10^{-4}$
1.1	1.0	$1.43*10^{-2}$	$3.50*10^{-4}$	$3.50*10^{-4}$
1.2	1.0	$1.54*10^{-2}$	$7.11*10^{-4}$	$7.11*10^{-4}$

[a] includes composite action and axial load
[b] studs spaced at 16 in. on center
[c] defined as the change in coefficient of variation with respect to initial conditions
[d] under 50-yr wind load at North Head, WA

Table 26. Failure Probabilities[a] of Douglas-fir As-Graded Studs[b] for Percent of Change[c] in Value of σ_1 and MOR

Ratio	Probability of Failure[d]			
	Stage 1	Stage 2	Stage 3	Stage 4
0.8	1.0	$1.20*10^{-2}$	$6.20*10^{-5}$	$6.20*10^{-5}$
0.9	1.0	$1.20*10^{-2}$	$1.04*10^{-4}$	$1.04*10^{-4}$
1.0	1.0	$1.20*10^{-2}$	$1.93*10^{-4}$	$1.93*10^{-4}$
1.1	1.0	$1.20*10^{-2}$	$3.50*10^{-4}$	$3.50*10^{-4}$
1.20	1.0	$1.20*10^{-2}$	$7.36*10^{-4}$	$7.36*10^{-4}$

[a] includes composite action and axial load
[b] studs spaced at 16. on center
[c] defined as the change in coefficient of variation with respect to initial conditions
[d] under 50-yr wind load at North Head, WA

Wall Systems with Composite Action and Load Sharing

The analytical model for load sharing given in chapter IV treated the studs in the wall system as a series of elastic springs, wherein the spring stiffness incorporates the composite action and material properties. The spring stiffness is given by,

$$k_i = \frac{wL}{\left(\frac{5}{384}\right)\frac{wL^4}{E_i I_w}} \qquad (101)$$

here k_i is the spring stiffness of the framing member, w is the external load in lb/in., E_i is the stiffness of the framing member, L is length of the framing member and I_w is the composite stiffness of the floor system.

A wall system with eight studs spaced equally at 16 in. or 24 in. on center, simply supported having a span of 96 in. was considered. Equation 102 was solved to determine the unknown deflection quantities.

$$\sum_{ij}^{8} a_{ij}\,\Delta_j = w_i \qquad (102)$$

where Δ_j are deflections at each of the stud members and the coefficients are given by,

$$a_{ij} = [k_j x_i (l - x_j)(2lx_j - x_j^2 - x_i^2)] \quad \text{for } i < j \tag{103}$$

$$a_{ij} = [k_j x_j (l - x_i)(2lx_i - x_i^2 - x_j^2)] \quad \text{for } i > j \tag{104}$$

$$w_i = w\, x_i l \frac{[l^3 - 2lx_i^2 + x_i^3]}{4} \quad \text{for } i = 1,8 \tag{105}$$

Once the deflections were obtained the stresses developed in each framing member were evaluated. Then, load sharing was computed as the ratio of the highest load carried by the weakest framing member with respect to the lateral load. The four characteristic events of wall behavior included load-slip between the framing and the sheathing (stages 1 and 2) and behavior of the framing member (stages 3 and 4). Load sharing applied only to stages 3 and 4 because load sharing is a function of framing stiffness.

The load-sharing factors were multiplied with the MOR in the reliability analysis to obtain a failure probability for floor systems. Thus, the modified failure function looks like,

$$g(x) = \{(LSF)\ MOR - \text{Acting Stress}\} \tag{106}$$

where LSF = load-sharing factor. Minimizing the failure function as before, the relibility index and probability of failure were obtained.

<u>Results of Reliability Analyses</u>

Including composite action and load sharing in the analysis, improved the reliability performance of the wall systems. Tables 27-34 present the β and P_f under 50-yr and 100-yr wind load for stud walls constructed with Douglas-fir and southern pine. The increase in reliability was almost of the same pattern in stud categories of Douglas-fir and southern pine irrespective of grade.

Even for 100-yr wind loads the failure probability was very low, which indicates that wall systems can sustain loads exceeding those expected in a 100-yr wind. Although the load sharing was much higher in the southern pine stud walls, Douglas-fir stud walls performed better in comparision to southern pine. Once again, this variation was attributed to the different coefficient of variation in wind velocities.

These results indicate that composite action and load sharing greatly reduce the failure probability even though axial load contributes to an increase in the P_f of wall systems. The results also show that minor grading errors in the stud grade has a minor impact on the reliability of the system. Based on this we can conclude that wall types with 16 in. stud spacing or 24. in stud spacing and of either species would be safe and can sustain loads exceeding those expected under a 50-yr or 100-yr wind load.

Table 27. β and P_f for Douglas-fir As-Graded Studs Spaced 16 in. on Center, Subjected to 3.29 and 3.66 lb/in. Wind Loads[a]

Analysis Conditions				50-yr Return Period		100-yr Return Period	
CA[b]	AL[c]	LS[d]	Stage	β	P_f	β	P_f
Yes	Yes	Yes	1	-	1.0	-	1.0
			2	2.26	$1.20*10^{-2}$	2.02	$2.17*10^{-2}$
			3	4.03	$2.88*10^{-5}$	3.88	$5.20*10^{-5}$
			4	4.03	$2.88*10^{-5}$	3.88	$5.20*10^{-5}$
Yes	No	Yes	1	-	1.0	-	1.0
			2	2.57	$5.10*10^{-3}$	2.35	$2.33*10^{-3}$
			3	4.44	$1.24*10^{-5}$	4.31	$1.76*10^{-5}$
			4	4.44	$1.24*10^{-5}$	4.31	$1.76*10^{-5}$
Yes	Yes	No	1	-	1.0	-	1.0
			2	2.26	$1.20*10^{-2}$	2.02	$2.17*10^{-2}$
			3	3.55	$1.93*10^{-4}$	3.38	$3.63*10^{-4}$
			4	3.55	$1.93*10^{-4}$	3.38	$3.63*10^{-4}$
Yes	No	No	1	-	1.0	-	1.0
			2	2.57	$5.10*10^{-3}$	2.35	$9.39*10^{-3}$
			3	3.89	$5.00*10^{-5}$	3.74	$9.20*10^{-5}$
			4	3.89	$5.00*10^{-5}$	3.74	$9.20*10^{-5}$

[a] = North Head, WA, [b] = composite action,
[c] = axial load
[d] = load sharing

Table 28. β and P_f for Douglas-fir On-Grade Studs Spaced at 16 in. on Center, Subjected to 3.29 and 3.66 lb/in. Wind Loads[a]

Analysis Conditions				50-yr Return Period		100-yr Return Period	
CA[b]	AL[c]	LS[d]	Stage	β	P_f	β	P_f
Yes	Yes	Yes	1	-	1.0	-	1.0
			2	2.24	$1.25*10^{-2}$	2.00	$2.27*10^{-2}$
			3	4.12	$2.52*10^{-5}$	3.97	$2.36*10^{-5}$
			4	4.12	$2.52*10^{-5}$	3.97	$2.36*10^{-5}$
Yes	No	Yes	1	-	1.0	-	1.0
			2	2.55	$5.39*10^{-3}$	2.33	$9.90*10^{-3}$
			3	4.54	$9.24*10^{-6}$	4.41	$1.36*10^{-5}$
			4	4.54	$9.24*10^{-6}$	4.41	$1.36*10^{-5}$
Yes	Yes	No	1	-	1.0	-	1.0
			2	2.26	$1.20*10^{-2}$	2.02	$2.17*10^{-2}$
			3	3.55	$1.93*10^{-4}$	3.38	$3.63*10^{-4}$
			4	3.55	$1.93*10^{-4}$	3.38	$3.63*10^{-4}$
Yes	No	No	1	-	1.0	-	1.0
			2	2.57	$5.10*10^{-3}$	2.35	$9.39*10^{-3}$
			3	3.89	$5.00*10^{-5}$	3.74	$9.20*10^{-5}$
			4	3.89	$5.00*10^{-5}$	3.74	$9.20*10^{-5}$

[a] = North Head, WA, [b] = composite action,
[c] = axial load
[d] = load sharing

Table 29. β and P_f for Southern Pine As-Graded Studs Spaced at 16 in. on Center, Subjected to 3.34 and 3.99 lb/in. Wind Loads[a]

Analysis Conditions				50-yr Return Period		100-yr Return Period	
CA[b]	AL[c]	LS[d]	Stage	β	P_f	β	P_f
Yes	Yes	Yes	1	-	1.0	-	1.0
			2	1.76	$3.92*10^{-2}$	1.46	$7.21*10^{-2}$
			3	3.42	$3.13*10^{-4}$	3.24	$5.98*10^{-4}$
			4	3.42	$3.13*10^{-4}$	3.24	$5.98*10^{-4}$
Yes	No	Yes	1	-	1.0	-	1.0
			2	2.00	$2.27*10^{-2}$	1.74	$4.09*10^{-2}$
			3	3.83	$6.40*10^{-5}$	3.67	$1.21*10^{-4}$
			4	3.83	$6.40*10^{-5}$	3.67	$1.21*10^{-4}$
Yes	Yes	No	1	-	1.0	-	1.0
			2	1.76	$3.92*10^{-2}$	1.46	$7.21*10^{-2}$
			3	2.88	$1.99*10^{-3}$	2.67	$3.79*10^{-3}$
			4	2.88	$1.99*10^{-3}$	2.67	$3.79*10^{-3}$
Yes	No	No	1	-	1.0	-	1.0
			2	2.01	$2.22*10^{-2}$	1.74	$4.09*10^{-2}$
			3	3.19	$7.11*10^{-4}$	2.99	$1.39*10^{-3}$
			4	3.19	$7.11*10^{-4}$	2.99	$1.39*10^{-3}$

[a] = Key West, FL, [b] = composite action,
[c] = axial load,
[d] = load sharing

Table 30. β and P_f for Southern Pine On-Grade Studs Spaced at 16 in. on Center, Subjected to 3.34 and 3.99 lb/in. Wind Loads[a]

Analysis Conditions				50-yr Return Period		100-yr Return Period	
CA[b]	AL[c]	LS[d]	Stage	β	P_f	β	P_f
Yes	Yes	Yes	1	-	1.0	-	1.0
			2	1.76	$3.92*10^{-2}$	1.46	$7.21*10^{-2}$
			3	3.51	$2.24*10^{-4}$	3.33	$4.34*10^{-4}$
			4	3.51	$2.24*10^{-4}$	3.33	$4.34*10^{-4}$
Yes	No	Yes	1	-	1.0	-	1.0
			2	2.02	$2.17*10^{-2}$	1.74	$4.09*10^{-2}$
			3	3.87	$5.40*10^{-5}$	3.71	$1.04*10^{-4}$
			4	3.87	$5.40*10^{-5}$	3.71	$1.04*10^{-4}$
Yes	Yes	No	1	-	1.0	-	1.0
			2	1.76	$3.92*10^{-2}$	1.46	$7.21*10^{-2}$
			3	2.88	$1.99*10^{-3}$	2.67	$3.79*10^{-3}$
			4	2.88	$1.99*10^{-3}$	2.67	$3.79*10^{-3}$
Yes	No	No	1	-	1.0	-	1.0
			2	2.01	$2.22*10^{-2}$	1.74	$4.09*10^{-2}$
			3	3.19	$7.11*10^{-4}$	2.99	$1.39*10^{-3}$
			4	3.19	$7.11*10^{-4}$	2.99	$1.39*10^{-3}$

[a] = Key West, FL, [b] = composite action,
[c] = axial load
[d] = load sharing

Table 31. β and P_f for Douglas-fir As-Graded Studs Spaced at 24 in. on Center, Subjected to 4.94 and 5.48 lb/in. Wind Loads[a]

Analysis Conditions				50-yr Return Period		100-yr Return Period	
CA[b]	AL[c]	LS[d]	Stage	β	P_f	β	P_f
Yes	Yes	Yes	1	-	1.0	-	1.0
			2	1.13	$1.29*10^{-1}$	0.83	$2.03*10^{-1}$
			3	3.04	$1.18*10^{-3}$	2.86	$2.12*10^{-3}$
			4	3.04	$1.18*10^{-3}$	2.86	$2.12*10^{-3}$
Yes	No	Yes	1	-	1.0	-	1.0
			2	1.45	$7.35*10^{-2}$	1.17	$1.21*10^{-1}$
			3	3.44	$2.91*10^{-4}$	3.27	$5.38*10^{-4}$
			4	3.44	$2.91*10^{-4}$	3.27	$5.38*10^{-4}$
Yes	Yes	No	1	-	1.0	-	1.0
			2	1.13	$1.29*10^{-1}$	0.83	$2.03*10^{-1}$
			3	2.63	$4.27*10^{-3}$	2.46	$6.95*10^{-3}$
			4	2.63	$4.27*10^{-3}$	2.46	$6.95*10^{-3}$
Yes	No	No	1	-	1.0	-	1.0
			2	1.45	$7.35*10^{-2}$	1.17	$1.21*10^{-1}$
			3	3.00	$1.35*10^{-3}$	2.82	$2.40*10^{-3}$
			4	3.00	$1.35*10^{-3}$	2.82	$2.40*10^{-3}$

[a] = North Head, WA, [b] = composite action,
[c] = axial load
[d] = load sharing

Table 32. β and P_f for Douglas-fir On-Grade Studs Spaced at 24 in. on Center, Subjected to 4.94 and 5.48 lb/in. Wind Loads[a]

Analysis Conditions				50-yr Return Period		100-yr Return Period	
CA[b]	AL[c]	LS[d]	Stage	β	P_f	β	P_f
Yes	Yes	Yes	1	-	1.0	-	1.0
			2	1.10	$1.36*10^{-1}$	0.80	$2.12*10^{-1}$
			3	3.12	$9.04*10^{-4}$	2.94	$1.64*10^{-3}$
			4	3.12	$9.04*10^{-4}$	2.94	$1.64*10^{-3}$
Yes	No	Yes	1	-	1.0	-	1.0
			2	1.43	$7.64*10^{-2}$	1.14	$1.27*10^{-1}$
			3	3.53	$2.08*10^{-4}$	3.36	$3.90*10^{-4}$
			4	3.53	$2.08*10^{-4}$	3.36	$3.90*10^{-4}$
Yes	Yes	No	1	-	1.0	-	1.0
			2	1.13	$1.29*10^{-1}$	0.83	$2.03*10^{-1}$
			3	2.63	$4.27*10^{-3}$	2.46	$6.95*10^{-3}$
			4	2.63	$4.27*10^{-3}$	2.46	$6.95*10^{-3}$
Yes	No	No	1	-	1.0	-	1.0
			2	1.45	$7.35*10^{-2}$	1.17	$1.21*10^{-1}$
			3	3.00	$1.35*10^{-3}$	2.82	$2.40*10^{-3}$
			4	3.00	$1.35*10^{-3}$	2.82	$2.40*10^{-3}$

[a] = North Head, WA, [b] = composite action,
[c] = axial load,
[d] = load sharing

Table 33. β and P_f for Southern Pine As-Graded Studs Spaced at 24 in. on Center, Subjected to 5.01 and 5.98 lb/in. Wind Loads[a]

Analysis Conditions				50-yr Return Period		100-yr Return Period	
CA[b]	AL[c]	LS[d]	Stage	β	P_f	β	P_f
Yes	Yes	Yes	1	-	1.0	-	1.0
			2	0.90	$1.84*10^{-1}$	0.55	$2.91*10^{-1}$
			3	2.70	$3.47*10^{-3}$	2.48	$6.57*10^{-3}$
			4	2.70	$3.47*10^{-3}$	2.48	$6.57*10^{-3}$
Yes	No	Yes	1	-	1.0	-	1.0
			2	1.18	$1.19*10^{-1}$	0.83	$2.03*10^{-1}$
			3	3.09	$1.00*10^{-3}$	2.89	$1.93*10^{-3}$
			4	3.09	$1.00*10^{-3}$	2.89	$1.93*10^{-3}$
Yes	Yes	No	1	-	1.0	-	1.0
			2	0.90	$1.84*10^{-1}$	0.55	$2.91*10^{-1}$
			3	2.20	$1.39*10^{-2}$	1.99	$2.33*10^{-2}$
			4	2.20	$1.39*10^{-2}$	1.99	$2.33*10^{-2}$
Yes	No	No	1	-	1.0	-	1.0
			2	1.18	$1.19*10^{-1}$	0.83	$2.03*10^{-1}$
			3	2.53	$5.70*10^{-3}$	2.32	$1.02*10^{-2}$
			4	2.53	$5.70*10^{-3}$	2.32	$1.02*10^{-2}$

[a] = Key West, FL, [b] = composite action,
[c] = axial load
[d] = load sharing

Table 34. β and P_f for Southern Pine On-Grade Studs Spaced at 24 in. on Center, Subjected to 5.01 and 5.98 lb/in. Wind Loads[a]

Analysis Conditions				50-yr Return Period		100-yr Return Period	
CA[b]	AL[c]	LS[d]	Stage	β	P_f	β	P_f
Yes	Yes	Yes	1	-	1.0	-	1.0
			2	0.92	$1.84*10^{-1}$	0.54	$2.95*10^{-1}$
			3	2.86	$2.12*10^{-3}$	2.65	$4.02*10^{-3}$
			4	2.86	$2.12*10^{-3}$	2.65	$4.02*10^{-3}$
Yes	No	Yes	1	-	1.0	-	1.0
			2	1.17	$1.19*10^{-1}$	0.82	$2.06*10^{-1}$
			3	3.23	$6.19*10^{-4}$	3.05	$1.14*10^{-3}$
			4	3.23	$6.19*10^{-4}$	3.05	$1.14*10^{-3}$
Yes	Yes	No	1	-	1.0	-	1.0
			2	0.90	$1.84*10^{-1}$	0.55	$2.91*10^{-1}$
			3	2.20	$1.39*10^{-2}$	1.99	$2.33*10^{-2}$
			4	2.20	$1.39*10^{-2}$	1.99	$2.33*10^{-2}$
Yes	No	No	1	-	1.0	-	1.0
			2	1.18	$1.19*10^{-1}$	0.83	$2.03*10^{-1}$
			3	2.53	$5.70*10^{-3}$	2.32	$1.02*10^{-2}$
			4	2.53	$5.70*10^{-3}$	2.32	$1.02*10^{-2}$

[a] = Key West, FL, [b] = composite action,
[c] = axial load,
[d] = load sharing

Summary of Wall Systems

For typical light-frame wall systems, composite action and load sharing contribute to yield a system that reaches the assigned limit state for load at level 1.5 to 1.73 greater than allowed by the current design procedure. Wall system capacity was sensitive to the variability of both the lumber MOR and the wind velocity. First failures were assumed to occur at connections rather than in the lumber.

Probability of failure was influenced by load sharing and composite action. The reliability studies indicated that walls with 16 in. or 24 in. stud spacings of either Douglas-fir or southern pine were highly reliable even under 50 and 100-yr wind loads.

VII. SUMMARY AND CONCLUSIONS

In a majority of residential buildings, studs and joists are the basic structural components. The walls act primarily as bending and compression panels and transmit lateral wind load and gravity loads into foundations. Joists consisting of 2×8 or 2×10 in., lumber together with the covering member act as an orthotropic plate when carrying loads. Mechanical fasteners, such as nails, form most joints and provide semi-rigid connection between framing and sheathing.

Engineering analysis and testing have shown that composite action and load sharing occurs in light-frame wood buildings. Composite action develops when the sheathing is attached to the framing member and load sharing, which is a function of lumber stiffness, involves the lateral distribution of loads via the sheathing.

A theoretical procedure for analyzing light-frame wood systems under bending and compressive loads, was developed which takes full account of the contribution of sheathing, the semi-rigid connection between framing and sheathing, as well as the variability in the mechanical properties of the framing members. Elastic properties such as the modulus of elasticity, shear modulus and

Poisson's ratio for the materials used for the wall coverings and the stiffness modulus of the nailed connection were obtained from the existing literature and were considered deterministic.

Complete load-deflection relationships for the studs were obtained from experimental data for Douglas-fir and southern pine species. Statistical analyses confirmed that a three-parameter Weibull probability density function best represented the strength and stiffness properties. Data for joist samples were obtained from the lumber tests at the Forest Research Laboratory, Oregon State University (Leichti and Eskelsen, 1992). Once again, the strength and stiffness properties were represented by a three-parameter Weibull probability density function. A Kolmogorov-Smirnov test statistic confirmed this function at the 5% significance level.

Loads were also considered as random variables. Dead load was represented as normal distribution with a coefficient of variation of 10%. Wind load and live load were represented by a Type I extreme value distribution function. The coefficient of variation for wind loads were 14.2 and 33.7% depending on the geographic location and for live load the variation was 25% (Ellingwood et al 1980).

A reliability analysis of wall and floor systems was conducted that incorporated the effects of composite action and load sharing. Reliability levels of wall systems in bending, with 16 in. and 24 in. stud spacing under a 50-yr and 100-yr wind load were examined using the first-order second-moment reliability method. Results indicate that including composite action greatly reduces the failure probability while the axial load increases the P_f of the wall systems. For typical wall systems, including composite action and load sharing contribute to yield a system that reaches the assigned limit state for load at level 1.5 to 1.73 greater than allowed by the current design procedure. Analyses also indicate that the wall system reliability was sensitive to the variabilities of the modulus of rupture of the framing members and the wind velocity, but reliability was not impacted by errors in lumber grading. Based on this, wall types with 16-in. or 24-in. stud spacing and of either species would be safe and can sustain loads exceeding those expected under a 50-yr and 100-yr wind loads.

Reliability levels of floor systems in bending, with 16 in. and 24 in. joist spacing, under a Type I extreme value distribution for live load were examined. Including composite action and load sharing in a floor system yielded a system that reaches the assigned limit state for load at level 1.19 to 1.96 greater than allowed by the current design procedure. P_f was dependent upon load sharing, composite action and the variabilities in the strength and stiffness of the framing lumber. Grade No.1 joists exhibited a higher load sharing than grade No.2. To summarize, composite action and load sharing contributed significantly to the stiffness and strength of floor systems. The 15% increase in the allowable bending stresses for repetitive light-frame members specified by the (NDS 1991) appears to be conservative. The reliability studies demonstrated that floors with 16 or 24-in. joist spacings were safe and can easily sustain the design loads.

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APPENDICES

APPENDIX A

Notation

A_c, A_s, A_t, A_j, A_p = cross-sectional area;

a_c, a_s, a_t = cross-sectional properties;

b = distance between point load to the support;

b_c, b_t = width of compressive and tensile flange;

b_s = stud width;

b_j = joist width;

cdf = cumulative distribution function;

pdf = probability density function;

C_p = wind-pressure coefficient;

c = constant;

CV = coefficient of variation;

dx = element of cross-sectional length;

E_i = modulus of elasticity of $stud_i$;

E_j = modulus of elasticity of joist;

E_p = modulus of elasticity of plywood;

F = force transmitted by the connector;

F_n = fourier series coefficient;

$F_R(r)$ = probability distribution function in R;

$f_R(r)$ = probability density function in R;

$f_S(s)$ = probability density function in S;

$f_x(x)$ = joint density function;

$f_V(v)$ = density function in wind velocity;

f_{bi} = bending stress;

G = shear modulus;

g(x) = failure function;

h_c, h_t = thickness of plywood and gypsum wall board;

h_s, h_j = stud and joist depth;

I_w, I_j = effective moment of inertia of composite I-beam (wall) and T-beam (floor);

I_c, I_t, I_s = moment of inertia of compressive (gypsum board), tensile (plywood) and stud members;

I_p = moment of inertia of plywood member;

i = stiffening factor;

l = width of the wall or floor system;

L = length of the framing member.

$K_c, K_t, K_{ct}, N_c, N_t$ = coefficients used in defining composite stiffness;

K_z = building exposure coefficient;

$k_{c1}, k_{c2}, k_{t1}, k_{t2}$ = compression and tension slip modulus;

k_i = spring stiffness used to define framing resistance;

M, M_m = resisting moment;

M_{max} = external moment;

M_c, M_s, M_t = respective moments in gypsum, stud and plywood members;

M_p, M_j = moments in plywood and joist members;

n = E_p/E_j;

n_{cc}, n_{ct} = number of rows of connectors;

P = axial load;

P_f = probability of failure of single framing member and its coverings;

Q = gust factor;

Q_i = nominal loads;

R = resistance;

R_d = roof dead load;

R_l = roof wind load;

R_n = nominal resistance;

R_i = vertical reaction offered by framing members;

S = loading;

s, s_c, s_t = spacing of nails;

s_{c1}, s_{c2} = slip between compressive flange and framing member;

s_{t1}, s_{t2} = slip between tensile flange and framing member;

T = transformation matrix;

V = wind velocity;

V_{50}, V_{100} = 50-yr and 100-yr wind velocity;

w = uniformly distributed load (lb/in.);

X_i = load or resistance variables;

x_i = distance from the support to the framing member;

x = coordinate;

Y = random variable;

y = coordinate;

Z_i = standard normal variable;

z, z_c, z_t = distance between geometric centroids to the framing member;

Δ_s = interlayerlayer slip;

Δ_i = deflection at the ith node;

ν = Poisson's ratio;

β = reliability index;

β_0 = target reliability index;

Φ = standard normal probability distribution function;

ϕ = standard normal probability density function;

$\varepsilon_{sc}, \varepsilon_{st}$ = strains in the compressive and tensile surfaces of the framing member (stud);

ε_j = strain in the joist member;

σ = acting stress;

σ_x, σ_y = standard deviation;

σ_r, σ_s = standard deviation in resistance and load;

σ_i = stress at proportional limit;

σ_c = limiting stress in compressive flange (plywood);

σ_t = limiting stress in tensile flange (gypsum);

η = shape parameter;

μ_w = location parameter;

σ_w = scale;

μ = mean;

φ = resistance factor;

γ_i = load factors;

α_i = sensitivity factor;

APPENDIX B

Loading Variables and Their Characteristics for Wall Systems

Dead Load

The roof imposes a dead load on the wall, which is carried as an axial force by the wall studs. The distribution of the dead load is normal with a coefficient of variation of 10%, (Ellingwood et al 1980). The total dead load was calculated as the combined weights of the roof and ceiling materials.

asphalt shingles:	(2.0 psf)(17.0)2	= 68.0 lb/ft
5/8-in. plywood sheathing:	(1.9 psf)(17.0)2	= 64.6 lb/ft
1/2-in. gypsum ceiling:	(2.5 psf)(28)	= 70.0 lb/ft
insulation (loose):	(0.5 psf)(28)	= 14.0 lb/ft
Total dead load including truss weight		= 271.65/2
		= 135.83 lb/ft

With a 16 in. stud spacing the mean load carried by each stud is, (135.83)(16)/(12) = 181.1 lb, with a standard deviation of 18.11 lb.

Wind Load

Wind load was evaluated according to ANSI A58.1-1982: Minimum Design Loads for Buildings and other Structures.

The lateral wind pressure on the wall model is,

$$w = c\ C_p\ K_z\ G\ V^2\ mf \tag{B.1}$$

w = lateral wind pressure,

c = 0.00256 (air mass density at standard atmosphere with a temperature of 59° F, sea level pressure of 29.92 in., of mercury, and dimensions in mi/hr of wind speed).

C_p = external pressure coefficient,

K_z = velocity pressure exposure coefficient = 1.20,

G = gust factor = 1.15,

V = wind speed (mi/hr) at 33 ft above ground,

mf = modification factor,

As all wind velocities are measured at 33 ft above ground level, a modification factor (mf) is used to represent wind speed at a height of 6 ft level, (Simiu et al 1980).

$$V_6^{\,2} = \left(\frac{h}{33}\right)^{\frac{2}{7}} V_{33}^{\,2} \tag{B.2}$$

from equation B.2 the modification factor for a height of 6 ft is,

$$mf = \left(\frac{h}{33}\right)^{\frac{2}{7}} = 0.62 \qquad \text{(B.3)}$$

where h = height of 6 ft, V_6 and V_{33} are the velocities at 6 and 33 ft height respectively.

The windward wall is considered to govern the wall design due to higher wind load. The wind load acting per unit length on the wall in lb/in. is,

$$w = c\ C_p\ K_z\ G\ V^2\ mf\ (16/144) \qquad \text{(B.4)}$$

From North Head, WA, measured 1912 - 1952, (Simiu et al 1980) the mean wind load is,

$$\bar{V} = 71.47\ \text{(mi/hr)} \qquad \text{(B.5)}$$

with coefficient of variation (CV) = 14.2%

The 50-yr return period of mean wind load is determined as follows (Ellingwood et al 1980),

$$\overline{V_{50}} = \bar{V}\ [(1 + \frac{\sqrt{6}}{\pi})V_R\ \ln(50)] = 102.4\ \text{(mi/hr)} \qquad \text{(B.6)}$$

with a 50-yr coefficient of variation as,

$$CV_{50} = CV\ \frac{\overline{V_R}}{\overline{V_{50}}} = 10\%. \qquad \text{(B.7)}$$

The standard deviation of the 50-yr wind load is,

$(\sigma_v)_{50}$ = 10.24 mi/hr

Distribution Parameters for Wind Velocity

As the wind velocity follows a type I largest extreme value, the cumulative distribution function is given by,

$$F_v(v) = \exp\{-\exp[-a(v-b)]\} \tag{B.8}$$

where $F_v(v)$ = cumulative distribution function, the two parameters a and b of the Type I extreme value distribution are related to the mean and variance as,

$$\bar{V} = b + \frac{0.577}{a} = 102.40$$
$$\sigma_v^2 = \frac{\pi^2}{6a^2} = (10.24)^2 \tag{B.9}$$

Solving the above equations, a = 0.1252 and b = 97.8.

Substituting for the parameters a and b the density function of the Type I extreme value yields,

$$f_v(v) = 0.1252\exp\{-0.1252\ (v-97.8) - \exp\ [-0.1252(v-97.8)]\} \tag{B.10}$$

APPENDIX C

Floor System Properties and Loading Variables

This appendix describes the properties of the floor members, external loading function, the formulation of the failure function and the partial derivatives with respect to each variable used in reliability analysis.

Combined Moment of Inertia

The combined moment of inertia of the floor system was written as,

$$I_w = I_j + nI_p + \frac{1}{1+K_o}\left[na_p^2A_p + a_j^2A_j\right] \tag{C.1}$$

$$\text{where } K_o = \frac{\pi^2}{L^2}\frac{s}{k\,n}E_j\frac{nA_pA_j}{[nA_p + A_j]} \tag{C.2}$$

Joist Properties

L = Joist span = 144 in.

h = Joist height = 7.25 in.

joist width = 1.5 in.

area (A_j) = (1.5*7.25) = 10.875 in^2

joist modulus of elasticity = 1.80*10^6 lb/in^2

I_j = 47.63 in^4

Sheathing Properties

Plywood modulus of elasticity = $1.487*10^6$ lb/in^2, (McCutcheon 1984)

Plywood thickness (t) = 5/8 in.

Area (A_p) = (16*5/8) = 10.0 in^2

n = (E_p/E_j) = 0.83

z = 1/2(h+t) = 3.9375 in.

I_p = 0.325 in^4

$$a_p = \frac{A_j z}{A_j + nA_p} = 2.23 \tag{C.3}$$

$$a_j = z - a_p = 1.71$$

Nail Properties

Nail stiffness = k_n = 9400 lb/in, (McCutcheon 1984)

Nail spacing = 6 in.

Using the above values in equation C.2,

$K_o = 1.43*10^{-6}\ E_j$ and

$$I_w = 47.63 + 0.27 + \frac{1}{1 + 1.43*10^{-6}E_j} 73.07 \tag{C.4}$$

Rearranging equation C.4,

$$I_w = \frac{120.97 + 6.85*10^{-5}E_j}{1 + 1.43*10^{-6}E_j} \qquad (C.5)$$

Recall the stress equation which is of the form,

$$\sigma_j = M\,[\frac{1.095}{I_w} + \frac{173.85}{I_w^2} + 0.023] \qquad (C.6)$$

Substituting for I_w and modifying we obtain the stress acting on the floor system as,

$$\sigma_j = M\,[\frac{305.75 + 7.45*10^{-4}E_j + 4.45*10^{-10}E_j^{\,2}}{14641 + 0.0166\,E_j + 4.69*10^{-11}E_j^{\,2}} + 0.023] \qquad (C.7)$$

or σ_j = M{ A + 0.023}, where

$$A = \frac{a_o}{a_u} = \frac{305.75 + 7.45*10^{-4}E_j + 4.45*10^{-10}E_j^{\,2}}{14641 + 0.0166\,E_j + 4.69*10^{-11}E_j^{\,2}} \qquad (C.8)$$

and M = external moment.

Failure Function and Partial Derivatives

The failure function has the form,

$$g(x) = \{X_2 - (X_3 + X_4)L^2 * 0.125\,\{\,A + 0.023\}\} \qquad (C.9)$$

The partial derivatives with respect to each of the variables are,

$$\frac{\partial g}{\partial X_1} = \frac{\partial A}{\partial X_1} \left[\frac{(X_3 + X_4)\ L^2}{8} \right] \tag{C.10}$$

$$\frac{\partial g}{\partial X_2} = 1 \tag{C.11}$$

$$\frac{\partial g}{\partial X_3} = -\left[\frac{A\ L^2}{8} + \frac{0.023\ L^2}{8} \right] \tag{C.12}$$

$$\frac{\partial g}{\partial X_4} = -\left[\frac{A\ L^2}{8} + \frac{0.023\ L^2}{8} \right] \tag{C.13}$$

$$\frac{\partial A}{\partial X_1} = \frac{(\ 7.45*10^{-4}+2(4.45*10^{-10})X_1)a_u-(.016+2(4.7*10^{-11})X_1)a_o}{a_u^2} \tag{C.14}$$

Floor Loading

Dead Load

The dead load on the floor system is basically from the framing and the covering members. The distribution of the dead load is normal with a coefficient of variation of 10%, (Ellingwood et al 1980).

5/8 in. plywood covering = 1.2 psf

Framing {estimate 2x8 at 16 in. on center} = 2.1 psf

Ceiling supports {2×4 at 24 in. on center} = 0.7 psf

Ceiling { 0.5 in. gypsum wall board} = 2.5 psf

Total dead load = 6.5 psf, or = 7.0 psf

The mean dead load is 7 psf, with a standard deviation of 0.7 psf.

Live Load

Live load is evaluated according to Uniform Building Code, UBC (1988). The live load pressure on the floor model is taken as 40 psf.

Also as the live load follows a type I largest extreme value, the cumulative distribution function is given by,

$$F_V(v) = \exp\{-\exp[-a(v-b)]\} \tag{C.15}$$

where $F_V(v)$ = cumulative distribution function,

The mean live load V = 40.0 psf,

with coefficient of variation = 25.0%

Distribution Parameters for Live Load

The two parameters a and b of the Type I extreme value distribution are related to the mean and variance as follows.

$$\bar{V} = b + \frac{0.577}{a} = 40.0$$

$$\sigma_v^2 = \frac{\pi^2}{6a^2} = 10^2 \tag{C.16}$$

Solving the above equations yields

$a = 0.1283$ and $b = 35.50$.

Substituting for the parameters a and b the density function of the Type I extreme value looks like,

$$f_v(v) = 0.1283\exp\{-0.1283(v-35.5)-\exp[-0.1283(v-35.5)]\} \quad (C.17)$$

APPENDIX D

Combined Moment of Inertia

The parameters used in arriving at the combined moment of inertia and the properties of the members are described in this part of the appendix. A wall section model is shown in Figure 8. The combined moment of inertia I_w of this model is from (Polensek and Kazic 1991) is,

$$I_w = [I_s+n_c I_c+n_t I_t] + \gamma[A_s a_s^2 + n_c A_c a_c^2 + n_t A_t a_t^2] + \gamma A_s[N_c z_t^2 + N_t z_c^2]$$

$$\text{where} \quad \gamma = \frac{1}{1 + K_c + K_t + K_{ct}} \qquad \text{(D.1)}$$

here I_w is with respect to its geometric centroid, subscript c = compressive flange, t = tensile flange, s = stud. Other parameters of equation (D.1) are stated.

$$n_c = \frac{E_c}{E_s}, \quad n_t = \frac{E_t}{E_s} \qquad \text{(D.2)}$$

where E = modulus of elasticity

$$a_c = \frac{z_c A_s + (z_c+z_t) n_t A_t}{A_s + n_c A_c + n_t A_t} \qquad \text{(D.3)}$$

$$a_s = \frac{z_t n_t A_t - z_c n_c A_c}{A_s + n_c A_c + n_t A_t} \tag{D.4}$$

$$a_t = \frac{z_t A_s + (z_c + z_t) n_c A_c}{A_s + n_c A_c + n_t A_t} \tag{D.5}$$

$$K_c = \frac{\pi^2 s_c E_s n_c A_c (A_s + n_t A_t)}{L^2 \ (k_c n_{cc} A_s + n_c A_c + n_t A_t)} \tag{D.6}$$

$$K_t = \frac{\pi^2 s_t E_s n_t A_t (A_s + n_c A_c)}{L^2 \ (k_t n_{ct} A_s + n_c A_c + n_t A_t)} \tag{D.7}$$

n_{ct}, n_{cc} = number of rows of connector = 1

$$K_{ct} = \frac{\pi^4 s_c s_t E_s{}^2 n_t A_t A_s n_c A_c}{L^4 \ k_c n_{cc} k_t n_{ct} [A_s + n_c A_c + n_t A_t]} \tag{D.8}$$

$$N_c = \frac{\pi^2}{L^2} \frac{s_c}{k_c n_{cc}} \quad \frac{E_s n_c A_c n_t A_t}{A_s + n_c A_c + n_t A_t} \tag{D.9}$$

$$N_t = \frac{\pi^2}{L^2} \frac{s_t}{k_c n_{ct}} \quad \frac{E_s n_c A_c n_t A_t}{A_s + n_c A_c + n_t A_t} \tag{D.10}$$

Wall System Properties

Stud properties

Stud span (L) = 96 in.

Stud height (h_s) = 3.5 in.

Stud width = 1.5 in.

Area (A_s) = (3.5*1.5) = 5.25 in^2.

Stud modulus of elasticity (E_s)= $1.70*10^6$ lb/in^2.,

(Polensek and Kazic 1991)

I_s = 5.36 in^4.

Sheathing

Exterior sheathing - plywood. The exterior sheathing was 3/8-in. CDX plywood, with

Plywood modulus of elasticity = $1.730*10^6$ lb/in^2.,

(Polensek and Kazic 1991)

Plywood thickness (h_c) = 3/8 in.

Area (A_c) = (16*3/8) = 6.0 in^2.

$n_c = (E_c/E_s) = 1.0$

$z_c = 0.5(h_s + h_c)$ = 1.94 in.

I_c = 0.07 in^4.

slip moduli = k_{c1} = 4.08 ksi, k_{c2} = 1.44 ksi

slip, s_{c1} = 0.025 in., s_{c2} = 0.12 in.

Interior sheathing - gypsum board. The interior sheathing was assumed to be 3/8-in gypsum board with, Modulus of elasticity = $230*10^3$ lb/in^2.

Thickness (h_t) = 3/8 in.

Area (A_t) = (16*3/8) = 6.0 in^2.

$n_t = (E_t/E_s) = 0.14$

$z_t = 0.5(h_s + h_t) = 1.94$ in.

$I_t = 0.07$ in^4.

slip moduli = $k_{t1} = 15.43$ ksi, $k_{t2} = 0.54$ ksi

slip, $s_{t1} = 0.002$ in., $s_{t2} = 0.12$ in.

Nail spacing (s) = 6 in.

Using the above values in equations D.3 through D.10 yields

$a_c = 1.11$ in., $a_s = -0.83$ in., $a_t = 2.77$ in.

$K_c = 4.76*10^{-6}E_s$

$K_t = 3.26*10^{-7}E_s$

$K_{ct} = 1.435*10^{-12}(E_s)^2$

$N_c = 6.57*10^{-7}E_s$

$N_t = 1.74*10^{-7}E_s$

$\gamma = 0.083$

It should be noted that the stud E is a random variable, however for initial calculation purposes and to obtain the combined moment of inertia a deterministic value is used. Substituting all of the above values in equation 1,

$I_w = 16.6$ in^4.

APPENDIX E

Reliability Index Calculation-
Sample Iteration for Wall Systems

This appendix describes the external loading function, formulation of the failure function, the partial derivatives with respect to each variable, and one sample iteration used when evaluating the reliability index.

Calculation of the Reliability Index

1. Define the failure function g(x).
2. Select initial β.
3. Choose initial values for random variables.
4. Transform non-normal variables into equivalent normal variables.
5. Establish eigenvalues and eigenvectors for correlated random variables and obtain transformation matrix.
6. Perform variable transformation

$$X_{(i)} = [\sigma_X(i)^N]\ [T]\ [Y]_i + X_i^N \tag{E.1}$$

7. Plug {X} into g(x) and obtain g(y).
8. Calculate {Y} from

$$Y_{(i)} = [T]^T[\sigma_X(i)^N]^{-1} [X_i - X_i^N] \tag{E.2}$$

9. Compute the partial derivatives

$$\frac{\partial g}{\partial Y_i} = [(\sigma_X(i)^N)[T]]^T \frac{\partial g}{\partial X_i} \tag{E.3}$$

10. Compute the sensitivity factors α_i.

$$\alpha_i = \frac{\frac{\partial g}{\partial Y_i}\sigma_{yi}}{\sqrt{[\sum_{i=1}^{n}\frac{\partial g}{\partial Y_i}\sigma_{yi}]^2}} \tag{E.4}$$

where $\sigma_{yi}^2 = \lambda_i$

11. Calculate new $\{Y_i\}$.

$$Y_i = -\beta[\sigma_y(i)]\alpha_i \tag{E.5}$$

12. Compute new $\{X_i\}$.

$$X_{(i)} = [\sigma_X(i)^N] [T] [Y]_i + X_i^N \tag{E.6}$$

13. If values $\{X_i\}$ converged, then proceed to step 14, else go to step 4.
14. Evaluate g(x). If the result is close to zero, calculate the failure probability, else modify β and go to step 3.

The following is the key that is used in representing the random variables. Variable X_1 and X_2 represent the joist moduli of elasticity E_1 and E_2. Variables X_3 and X_4 represent σ_1 and MOR. Variable X_5 represents the axial load. Variable X_6 and X_7 represent the pressure coefficients C_p^+ and C_p^4. Variable X_8 represents the velocity pressure coefficient. Variable X_9 represents gust factor (G). Variable X_{10} represents wind velocity (V)

Reliability Analysis and Formulation of Failure Function

Any stage is considered to fail when the resisting moment offered by the system is less than the maximum moment caused by the external loads. The failure function is expressed as the difference between the resisting moment and the moment caused by the external loads.

$$g(x) = M_m - M_{max} = 0 \quad \text{(E.7)}$$

or in terms of stress $g(x) = MOR - \sigma = 0$

where $g(x)$ is the failure function, MOR is the modulus of rupture and σ is the acting stress. The acting stress on the wall system is calculated as follows. In this particular example, the ultimate stage, which is stage (4), is considered. Recall the moment equations from chapter 6, these are of the form,

$$M - M_3 = (\sigma - \sigma_1) I_w \frac{1}{[\frac{h_s}{2} + \gamma_3 a_s + \gamma_3 z_t N_c (\frac{K_{c1}}{K_{c2}}) - z_c N_t \frac{K_{t1}}{K_{t2}}]} \quad (E.8)$$

$$M_3 - M_2 = (\sigma_1 - \sigma_c) I_w \frac{1}{[\frac{h_s}{2} + \gamma_3 a_s + \gamma_3 z_t N_c (\frac{K_{c1}}{K_{c2}}) - z_c N_t \frac{K_{t1}}{K_{t2}}]} \quad (E.9)$$

where σ_c is the limiting stress in the compression flange (plywood).

$$\gamma_3 = \frac{1}{[1 + K_c \frac{K_{c1}}{K_{c2}} + K_t \frac{K_{t1}}{K_{t2}} + K_{ct} \frac{K_{t1}}{K_{t2}} \frac{K_{c1}}{K_{c2}}]} \quad (E.10)$$

$$M_2 - M_1 = 3.51 \frac{s_{c1} - s_{c1} \frac{K_{c1}}{K_{c2}}}{L} EI_w \frac{1}{\gamma_2 [z_c (K_{ct} \frac{K_{t1}}{K_{t2}} + K_c)] + z_t N_c} \quad (E.11)$$

$$\gamma_2 = \frac{1}{[1 + K_c + K_t \frac{K_{t1}}{K_{t2}} + K_{ct} \frac{K_{t1}}{K_{t2}}]} \quad (E.12)$$

$$M_1 = M_{max}{}^t = 3.51\, s_{t1} \frac{EI_w}{L} \frac{1}{\gamma [z_t (K_{ct} + K_t) + z_c N_t]} \quad (E.13)$$

Supplying known deterministic values,

$$M_1 = \frac{601.38 + 1.16X_1 + 0.025(10)^{-3}X_1^2}{349.12 + X_1} \tag{E.14}$$

$$M_2 = M_1 + \frac{207.33 + 1.71X_1 + 2.02(10)^{-3}X_1^2}{132.19 + X_1} \tag{E.15}$$

$$M_2 = M_1 + [C]$$

$$M_3 = M_2 + (\sigma_1 - \sigma_c)\frac{677.2 + 7.67X_1 + 18.69(10)^{-3}X_1^2}{27.2 + X_1 + 6.0(10)^{-3}X_1^2} \tag{E.16}$$

$$M_3 = M_2 + (\sigma_1 - \sigma_c)\ [B]$$

$$M = M_3 + (\sigma - \sigma_1)\frac{677.2 + 7.67X_2 + 18.69(10)^{-3}X_2^2}{27.2 + X_2 + 6.0(10)^{-3}X_2^2} \tag{E.17}$$

Recall the stress equation which is of the form,

$$\sigma = -\left[\frac{M_{max}}{I_w}\right]\left[\gamma a_s + \gamma z_t N_c - \gamma z_c N_t + \frac{h_s}{2}\right] - \frac{P}{A_s} \tag{E.18}$$

Substituting equation E.13 for moment, M_{max} in equation E.18

$$\sigma_t = -\left[\frac{3.51\ s_{t1}\ E_1}{L}\right] \frac{[a_s + z_t N_c - z_c N_t + \frac{h_s}{2\gamma}]}{z_t(K_{ct} + K_t) + z_c N_t} + \frac{P}{A_s} \qquad (E.19)$$

where σ_t is the limiting stress in the tensile flange (gypsum board).

$$\sigma_t = \frac{24.16 + 0.2586X_1 + 0.065(10)^{-3}X_1^2}{349.12 + X_1} + \frac{P}{5.25}$$

$$\sigma_t = [H] + \frac{P}{5.25} \qquad (E.20)$$

Similarly substituting equation E.11 in equation E.18,

$$\sigma_c - \sigma_t = \left[\frac{3.51 s_{c1} - s_{c1}\frac{k_{c1}}{K_{c2}}\ E_1}{L}\right] \frac{[a_s + z_t N_c - z_c N_t + \frac{h_s}{2\gamma_2}]}{z_c(K_{ct}\frac{k_{t1}}{K_{t2}} + K_c) + z_t N_c} \qquad (E.21)$$

$$\sigma_c = \sigma_t + \frac{8.33 + 0.15X_1 + 0.65(10)^{-3}X_1^2}{132.19 + X_1}$$

$$\sigma_c = \sigma_t + [D] \qquad (E.22)$$

Equation E.17 can be rearranged as,

$$\sigma = \sigma_1 + (M - M_3)\frac{27.2 + X_2 + 6.0(10)^{-3}X_2^2}{677.2 + 7.68X_2 + 18.69(10)^{-3}X_2^2} \qquad (E.23)$$

$$\sigma = \sigma_1 + (M - M_3)\ [A]$$

Substituting for M_3, M_2 and σ_1 yields

$$\sigma = \sigma_1(1 - AB) + MA - M_2A + \sigma_c AB \qquad (E.24)$$

$$\sigma = \sigma_1(1 - AB) + MA - M_1A - CA + HAB + DAB + \frac{PAB}{5.25} \qquad (E.25)$$

Parameters A, B, C, D, H are defined later, P is axial load, σ is acting stress and M is the external bending moment. The failure function now takes the form,

$$g(\ ) = MOR + \sigma_1 AB - MA + M_1A + CA - HAB - DAB - \frac{PAB}{5.25} \qquad (E.26)$$

It is also noted that B, C, D, H and M_1 are functions of random variable X_1 only which is modulus of elasticity E_1 and are defined as follows. Parameters A, B, C, D and H are seperated into numerators and denominators for simplicity in evaluating the partial derivatives.

$$A = \frac{a_o}{a_u} = \frac{677.2 + 7.67X_2 + 18.69(10)^{-3}X_2^2}{27.2 + X_2 + 6.0(10)^{-3}X_2^2} \qquad (E.27)$$

$$B = \frac{b_o}{b_u} = \frac{677.2 + 7.67X_1 + 18.69(10)^{-3}X_1^2}{27.2 + X_1 + 6.0(10)^{-3}X_1^2} \qquad (E.28)$$

$$C = \frac{c_o}{c_u} = \frac{207.33 + 1.71X_1 + 2.02(10)^{-3}X_1^2}{132.19 + X_1} \tag{E.29}$$

$$D = \frac{d_o}{d_u} = \frac{8.33 + 0.15X_1 + 0.65(10)^{-3}X_1^2}{132.19 + X_1} \tag{E.30}$$

$$H = \frac{h_o}{h_u} = \frac{24.16 + 0.2586X_1 + 0.065(10)^{-3}X_1^2}{349.12 + X_1} + \frac{P}{5.25} \tag{E.31}$$

The external moment is a given by,

$$M = \frac{wL^2}{8} = c\ C_p^4\ K_z\ G\ V^2\ \frac{96^2}{8}\ [\frac{16}{144}] \tag{E.32}$$

$$M = c\ X_7\ X_8\ X_9\ X_{10}^2\ [128]$$

Total load acting on each stud is given by,

$$P = X_5 - c\ X_6\ X_8\ X_9\ X_{10}^2[\frac{16}{12}](mf) \tag{E.33}$$

Having defined the parameters, an intial value of β = 3.0 is chosen. Set initial values or initial points for all the random variables. Note these values are arbitrary, any value can be chosen.

$X(1) = E_1 = 1700,000$

$X(2) = E_1 = 360000$

$X(3) = \sigma_1 = 3000$

$X(4) = MOR = 700$

$X(5) = R_d = 150$

$X(6) = C_p^+ = 2$

$X(7) = C_p^4 = 1$

$X(8) = K_z = 1$

$X(9) = G = 1.5$

$X(10) = V = 100$

The non-normal distributions are transformed into equivalent normal distributions such that the cumulative and probability density functions of the actual and approximating normal variable are equal.

Random variable X(1) = Modulus of Elasticity, follows a Weibull distribution, the parameters are

shape (η) = 2.36, location (μ_w) = 622813 psi,

scale (σ_w) = 811300 psi,

The cumulative distribution of the normal function is set equal to the cumulative distribution function of the Weibull. This is of the form,

$$\Phi(\xi) = 1 - \exp\left[-\left[\frac{X(1) - \mu_w}{\sigma_w}\right]^{\eta}\right] \qquad (E.34)$$

Substituting the three-parameters and the initial value,

$$\Phi(\xi) = 0.858$$
$$\xi = 1.08 \qquad (E.35)$$

Now the probability density function of the normal distribution is set equal to the probability density function of the Weibull such that the mean and standard deviation of equivalent normal distribution are,

$$\frac{1}{\sqrt{2\pi}\,\sigma_X(1)} \exp\left[-\frac{1}{2}\left[\frac{X(1) - \bar{X}^N}{\sigma_X(1)^N}\right]^2\right]$$
$$= \left(\frac{\eta}{\mu_w}\right)\left[\frac{(X(1) - \mu_w)}{\sigma_w}\right]^{(\eta-1)} \exp\left[-\frac{(X(1) - \mu_w)}{\sigma_w}\right]^{\eta} \qquad (E.36)$$

$$\sigma_X(1) =$$
$$\frac{1}{\sqrt{2\pi}\,\sigma_X(1)} \left(\frac{\eta}{\mu_w}\right)\left[\frac{(X(1) - \mu_w)}{\sigma_w}\right]^{(1-\eta)}$$
$$\frac{\exp[-0.5(\xi^2)]}{\exp\left[-\frac{(X(1) - \mu_w)}{\sigma_w}\right]^{\eta}} \qquad (E.37)$$
$$= 366714.4$$

The mean of random variable X(1) is obtained as,

$$\overline{X}(1)^N = X(1) - \xi\, \sigma_1^N = 1303948.4 \qquad (E.38)$$

Similarly random variables X(2), X(3),....X(9) are transformed into equivalent normal distribution. For random variable X(10) which follows a Type I extreme value distribution the normality is performed as follows. The parameters for 50-yr. wind load is a = 0.1252, b = 97.8.

$$\Phi(\xi) = \frac{1}{\sqrt{2\pi}\,\sigma_X(10)} \exp\left[-\frac{1}{2}\left[\frac{X(10)-\overline{X}^N}{\sigma_X(10)^N}\right]^2\right] \qquad (E.39)$$

$$= \exp[\,-\exp[\,-a(\,X(10) - b)\,]\,]$$

$$(\xi) = -0.08$$

The probability distributions are equated to arrive at,

$$\frac{1}{\sqrt{2\pi}\,\sigma_X(10)} \exp\left[-\frac{1}{2}\left[\frac{X(10)-\overline{X}^N}{\sigma_X(10)^N}\right]^2\right] \qquad (E.40)$$

$$= a \exp[\,-a\,(\,X(10 - b) - \exp[\,-a(\,X(10) - b)\,]\,]$$

$$\sigma_X(10)^N = \frac{\exp[-0.5(\xi^2)]}{\sqrt{2\pi}\, a\exp[-a(X(10)-b)-\exp[-a(X(10)-b)]]} \qquad (E.41)$$

$$= 8.94$$

The mean of random variable X(10) is obtained as,

$$\bar{X}(10)^N = X(10) - \xi\,\sigma_{10}^N = 100.7 \qquad (E.42)$$

Thus, the equivalent mean and standard deviation for all ten random variables are obtained.

From the correlated random variables, eigenvalues and the orthogonal transformation matrix which is composed of eigenvectors is obtained. An IMSL Math/Library routine is used when evaluating the eigenvectors and eigenvalues. FORTRAN program WALL (Appendix H) further explains the iteration that goes on in evaluating the reliability index.

APPENDIX F

Simulation of Correlated Lumber Properties

This section summarizes the procedure used to generate correlated random variables. The following are the steps used to determine the random vectors for modulus of elasticity (E_1), modulus of elasticity (E_2), state of stress (σ_1) and modulus of rupture (MOR).

Step 1. Obtain the appropriate distribution functions of the strength and stiffness properties, suppose this consists of $F_{E1}(x)$, $F_{E2}(x)$, $F\sigma_1(x)$ and $F_{MOR}(x)$. It should be noted that $F_{E1}(x)$, $F_{E2}(x)$, $F\sigma_1(x)$ and $F_{MOR}(x)$ correspond to the cumulative distribution function of modulus of elasticity (E_1), modulus of elasticity (E_2), state of stress (σ_1) and modulus of rupture (MOR) respectively, and x is the value assumed by the random variable. The example considered is Douglas-fir As-Graded studs with the strength and stiffness properties following a 3-parameter Weibull distribution function. The parameters of these distribution are given in Table 35.

Table 35. Three Parameters of Douglas-fir As-Graded Studs Along with the Mean and Standard Deviation

		Property			
Descriptor	Units	E_1	E_2	σ_1	MOR
shape (η)		2.36	1.30	2.08	1.18
location (μ_w)	(psi)	622813	0.0	1325	0.0
scale (σ_w)	(psi)	811300	390604	3047	897
mean	(psi)	1307000	361000	3922	848
standard deviation	(psi)	311137	386838	1284	1218

Step 2. Estimate the covariance matrix, Σ for the random variables E_1, E_2, σ_1 and MOR. This has the form of,

$$\Sigma = \begin{bmatrix} ({}^{\sigma}E_1)^2 & {}^{\sigma}(E_1E_2) & {}^{\sigma}(E_1\sigma_1) & {}^{\sigma}(E_1MOR) \\ & ({}^{\sigma}E_2)^2 & {}^{\sigma}(E_2\sigma_1) & {}^{\sigma}(E_2MOR) \\ & & {}^{\sigma}(\sigma_1)^2 & {}^{\sigma}(\sigma_1MOR) \\ & & & {}^{\sigma}(MOR)^2 \end{bmatrix}$$

where $({}^{\sigma}E_1)^2$ = variance of E_1, $({}^{\sigma}E_1E_2)$ = covariance of E_1 and E_2, and is computed as follows,

$({}^{\sigma}E_1E_2) = \rho({}^{\sigma}E_1{}^{\sigma}E_2)$, where ρ is the correlation coefficient between E_1 and E_2. ${}^{\sigma}E_1$ and ${}^{\sigma}E_2$ are the standard deviation of E_1 and E_2 respectively. The correlation coefficients are,

$\rho(E_1E_2) = 0.2634$, $\rho(E_1\sigma_1) = 0.6383$, $\rho(E_1MOR) = 0.4218$.

$\rho(E_2\sigma_1) = 0$, $\rho(E_2MOR) = 0.5750$, $\rho(\sigma_1MOR) = 0.1618$.

Using the corresponding values the symmetrical Σ matrix is,

$$\begin{bmatrix} 9.68*10^{10} & 3.13*10^{10} & 2.55*10^{8} & 1.60*10^{8} \\ & 1.49*10^{11} & 0.0 & 2.71*10^{8} \\ & & 1.65*10^{6} & 2.53*10^{5} \\ & & & 1.48*10^{6} \end{bmatrix}$$

Step 3. Σ matrix is decomposed into upper and lower triangular matrices, Cholesky method, (Meyer, 1973)

$$\sum = [L]\,[L]^T \tag{F.1}$$

The elements of {L} matrix are,

$$L_{ii} = \sqrt{\sum_{ii} - \sum_{r=1}^{i-1} L_{ir}^{2}} \tag{F.2}$$

$$L_{ij} = \frac{\sum_{ij} - \sum_{r=1}^{j-1} L_{ir}\, L_{jr}}{L_{jj}} \tag{F.3}$$

for $i > j$

Step 4. Using the covariance matrix, generate a desired number of random vectors from a multivariate normal distribution $N(0,\Sigma)$. The desired random vectors has a mean value equal to 0 and a variance equal to that found in the covariance matrix (Law and Kelton 1982).

$$x_i = \mu_i + \sum_{j=1}^{i} L_{ij} Z_j \quad \text{(F.4)}$$

where μ_i's are the mean values and are equal to 0, and Z_j are the standard normal variables $N(0,1)$.

Expanding equation F.4,

$$x_{E1} = L_{11} Z_1$$

$$x_{E2} = L_{21} Z_1 + L_{22} Z_2$$

$$x\sigma_1 = L_{31} Z_1 + L_{32} Z_2 + L_{33} Z_3$$

$$x\sigma_{MOR} = L_{41} Z_1 + L_{42} Z_2 + L_{43} Z_3 + L_{44} Z_4$$

Step 5. The normal random vectors are then divided by the standard deviation of the respective variates to obtain standard normal deviates $N(0,1)$.

$$x^{*}_{E1} = \frac{x_{E1}}{\sigma E_1} \quad \text{(F.5)}$$

$$x^*_{E2} = \frac{x_{E2}}{\sigma E_2} \quad \text{(F.6)}$$

$$x^*_{\sigma 1} = \frac{x_{\sigma 1}}{\sigma(\sigma_1)} \quad \text{(F.7)}$$

$$x^*_{MOR} = \frac{x_{MOR}}{\sigma(MOR)} \quad \text{(F.8)}$$

Step 6. The cumulative distribution is then evaluated from the standard normal resulting in vectors of correlated observations uniformly distributed between 0 and 1.

$$U_{E1} = \Phi[x^*_{E1}]$$

$$U_{E2} = \Phi[x^*_{E2}]$$

$$U\sigma_1 = \Phi[x^*\sigma_1]$$

$$U_{MOR} = \Phi[x^*_{MOR}]$$

Step 7. Finally the uniformly distributed vectors were supplied to the inverse distribution function to obtain the values of MOE_1, MOE_2, σ_1 and MOR.

$$MOE_1 = \mu + \sigma\{-\ln(1 - U_{E1}\ \}^{1/\eta}$$

$$MOE_2 = \mu + \sigma\{-\ln(1 - U_{E2}\ \}^{1/\eta}$$

$$\sigma_1 = \mu + \sigma\{-\ln(1 - U\sigma_1\ \}^{1/\eta}$$

$$MOR = \mu + \sigma\{-\ln(1 - U_{MOR}\ \}^{1/\eta}$$

APPENDIX G

Computer Program for the Computation of Load Sharing in Light-Frame Wood Systems

```
C     **************************************************
C     PROGRAM SHR.FOR
C     **************************************************
      $LARGE
       EXTERNAL FI
C      THIS PROGRAM FINDS THE LOAD SHARING BETWEEN STIFFER
C     & LIMBER JOISTS OCCURING IN A WALL OR FLOOR SYSTEMS.
C      INPUTS CONSIST OF STIFFNESS AND STRENGTH OF STUDS
C     OR JOISTS AND STIFFNESS OF PLYWOOD/GYPSUM.
C*******************************************************
*******************************************************
       DIMENSION
A(8,8),IND(8,2),B(8,1),C(8,1),E1(9000),AA(410),
     1APP1(9000),R1(9000),AR(33000),BB(410),RRIW(9000),
     2T(2,2),AQQ(8),ARR(8),ASS(8),RLSF(1001),
     4STUD(9000),G(8,8),
     5R(200),WA(9000),E2(9000),BC(8,1),F(8,8)
     OPEN(UNIT=1,FILE='D:\JUNK\NGAUSS',STATUS='OLD')
     OPEN(UNIT=6,FILE='D:\JUNK\OUT',STATUS='NEW')
C     ***************************************************
       N=8
     WRITE(*,*) 'PLEASE INPUT # OF JOISTS TO BE SIMULATED'
     READ(*,*) NUM
     LNM=NUM/8
     NUM1=2*NUM
      pi=3.141592654
     POMM=1000.0
C     *****************************************************
C     INPUT THE PDF AND CDF OF STANDARDIZED NORMAL VARIABLE
C     FOUND IN FILE CALLED AS NGAUSS
      READ (1,1000) (AA(I),BB(I),I=1,403)
1000  FORMAT(F4.2,F8.6)
C     *****************************************************
C     INPUT THE THREE PARAMETERS OF THE WEIBULL DISTRIBUTION
     READ(2,*) ETA1,AMU1,SIG1
     READ(2,*) ETA2,AMU2,SIG2
C     INPUT THE CORRELATION AND THE STANDARD DEVIATION
     READ(3,*) SDE1,SDR1
     READ(4,*) RO1
C     *****************************************************
C     GENERATION OF NORMAL RANDOM VARIABLES
```

```
      DO 100 I =1,NUM1
      J=I
      IF(I.EQ.1) J=-1
      AR(I)=GAUSS(J)
100        CONTINUE
C     GENERATION OF CORRELATED RANDOM VARIABLES
      A11=SDE1*SDE1
      A12=RO1*SDE1*SDR1
      A22=SDR1*SDR1
C     FIRST ROW
      C11=SQRT(A11)
      C21=A12/C11
C     SECOND ROW
      C22=SQRT(A22-(C21*C21))
C     **************************************************
      KK=0
       DO 34 J=1,NUM
      Z1=AR(KK+1)
      Z2=AR(KK+2)
      P1=(C11*Z1)/SDE1
      P2=(C21*Z1+C22*Z2)/SDR1
      Y1=FI(P1,AA,BB)
      Y2=FI(P2,AA,BB)
C     ********************************************
C     GENERATION OF RANDOM VARIABLES IN STRENGTH AND
       STIFFNESS (WEIBULL)
      E1(J)=AMU1+SIG1*((-ALOG(Y1))**(1./ETA1))
      R1(J)=AMU2+SIG2*((-ALOG(Y2))**(1./ETA2))
      R1(J)=R1(J)/2.1
      KK=KK+2
34     CONTINUE
C     ********************************************
C     INPUT THE SPACING BETWEEN EACH FRAMING MEMBERS
       X1=16.0
       X2=32.0
       X3=48.0
       X4=64.0
       X5=80.0
       X6=96.0
       X7=112.0
       X8=128.0
       KOUNT=1
       KKK=0
       KSUM=0
       LSUM=0
       NSUM=0
       KRUN=1
       KLAP=0
      KKR=1
      KJR=8
C     INPUT THE WIDTH OF THE FLOOR OR WALL SYSTEM
111   WID=144.0
```

```
      LOOP=0
C     **********************************************
C      EVALUATION OF COMPOSITE ACTION PROPERTIES
      CALL COMP(E1,STUD,RRIW,KKR,KJR,AK1,AK2,
     1AK3,AK4,AK5,AK6,AK7,AK8,LOOP)
C     ******************************************
C     STIFFNESS MATRIX EVALUATION
C     ********************************************
      CALL AMAT(G,WID,AK1,AK2,AK3,AK4,AK5,AK6,AK7,AK8)
      CALL MATINV(G,8,IND,8)
C     ********************************************
C     INPUT THE INITIAL DEAD AND LIVE LOAD
C     LIVE LOAD+DEAD LOAD = 47 PSF
C     TOTAL LOAD IS IN LBS/IN  (47PSF/144)144
C     INCREMENT IS DONE IN STEPS OF 0.25 PSF
      Q=30.0
112       Q=Q+0.25
      Q=(Q*144.0)/144.0
C     ********************************************
C     LOAD VECTOR MATRIX
C     *******************************************
       B(1,1)=Q*0.25*WID*X1*((WID**3)-(2.0*WID*X1*X1)
     +(X1**3))
       B(2,1)=Q*0.25*WID*X2*((WID**3)-(2.0*WID*X2*X2)
     +(X2**3))
       B(3,1)=Q*0.25*WID*X3*((WID**3)-(2.0*WID*X3*X3)
     +(X3**3))
       B(4,1)=Q*0.25*WID*X4*((WID**3)-(2.0*WID*X4*X4)
     +(X4**3))
       B(5,1)=Q*0.25*WID*X5*((WID**3)-(2.0*WID*X5*X5)
     +(X5**3))
       B(6,1)=Q*0.25*WID*X6*((WID**3)-(2.0*WID*X6*X6)
     +(X6**3))
       B(7,1)=Q*0.25*WID*X7*((WID**3)-(2.0*WID*X7*X7)
     +(X7**3))
       B(8,1)=Q*0.25*WID*X8*((WID**3)-(2.0*WID*X8*X8)
     +(X8**3))
C     ***********************************
       M1DIM=8
       M2DIM=8
       M3DIM=8
C     ***************************************
C     EVALUATION OF DEFLECTION
      CALL MULT(G,8,8,M1DIM,B,1,M2DIM,C,M3DIM)
C     **************************************
      Q=(Q*144.0)/144.0
25     FORMAT(/57('*')/)
C     **********************************************
C     STRESSES IN THE WALL SYSTEM DUE TO LOAD
C     SHARING AND COMPOSITE ACTION
686   CALL STRESS(E1,E2,R1,RRIW,C,BC,APP1,AQQ,ARR,
     1ASS,KSUM,KLAP,8)
```

```
C     ************************************
C     ******************************
      CALL TRANS(AQQ,ARR,ASS,KLAP,AVL,DVL,Q)
        IF(KLAP.EQ.1) GO TO 112
       WRITE(6,55) KRUN
55    FORMAT(' FLOOR #  ',I4/)
      WRITE(6,*) 'JOIST #   MOE   MOR   FLOOR
      STRESS DVLP  '
          DO 136 K =1,N
               KKK=KKK+1
136     write(6,54) KKK,E1(KKK),R1(KKK),APP1(KKK)
54    FORMAT(I5,2X,F10.1,3X,2F10.2)
493   WRITE(*,*) AVL,DVL
         WRITE(*,59) Q
59    FORMAT('   FLOOR FAILING LOAD (PSF)',F7.2)
      RLSF(KRUN)=Q/47.0
      WRITE(7,*) RLSF(KRUN)
           KRUN=KRUN+1
      IF(KRUN.EQ.(LNM+1)) GO TO 123
          GO TO 111
123         FAC=0.
      DO 802 KK =1,LNM
         FAC=FAC+RLSF(KK)
802      CONTINUE
      FAC=FAC/LNM
          VAR=0
      DO 803 KK=1,LNM
         VAR=VAR+((RLSF(KK)-FAC)**2)
803      CONTINUE
      SD=SQRT(VAR/LNM)
       COV=SD/FAC
       WRITE(8,809) FAC
809    FORMAT(/'LOAD SHARING FACTOR IS ',F5.2/)
       WRITE(8,807) SD,COV
807    FORMAT(/'STAN DEV IS',F7.4/,'COEFF OF VAR ',F7.5)
       STOP
       END
C     ****************************************
C     *******************************************
      SUBROUTINE TRANS(AQQ,ARR,ASS,KLAP,AVL,DVL,Q)
       DIMENSION AQQ(8),ARR(8),ASS(8)
      if(Q.EQ.100.) GO TO 77
      DO 4 K =1,7
      DO 6 J=1,7
       IF(ARR(J).LE.ARR(J+1)) GO TO 6
        TEMP=ARR(J)
       ARR(J)=ARR(J+1)
       ARR(J+1)=TEMP
6      CONTINUE
4      CONTINUE
      DO 86 JM =1,8
       IF(ARR(1).EQ.AQQ(JM)) GO TO 87
```

```
86    CONTINUE
87IF(ABS((AQQ(JM)-ASS(JM))/AQQ(JM)).GT.0.005) GO TO 88
            AVL=AQQ(JM)
            DVL=ASS(JM)
77   KLAP=0
        GO TO 89
88        KLAP=1
89      RETURN
      END
C    ******************************************
C    ******************************************
     SUBROUTINE STRESS(E1,E2,R1,RRIW,C,BC,APP1,AQQ,ARR,
     1ASS,KSUM,KLAP,N)
     DIMENSION E1(8),E2(8),R1(8),RRIW(8),
     1C(8,1),BC(8,1),APP1(8),AQQ(8),ARR(8),ASS(8)
     IF(KLAP.EQ.0) GO TO 17
         KSUM=KSUM-8
17      DO 16 I =1,N
      KSUM=KSUM+1
      DDR=RRIW(KSUM)*E1(KSUM)
     FTY=(C(I,1)*384.0*DDR)/(5.0*(144.0**4))
     APP1(KSUM)=FTY*197.25
       AQQ(I)=R1(KSUM)
       ARR(I)=R1(KSUM)
       ASS(I)=APP1(KSUM)
16       CONTINUE
     RETURN
     END
C    *********************************************
C    ******************************************
      SUBROUTINE MATINV(A,N,IND,MAX)
C    Reference (Gerstle 1974)
C     THIS SUBROUTINE COMPUTES INVERSE OF {A}
      DIMENSION A(MAX,MAX),IND(MAX,2)
      DO 550 I=1,N
      IND(I,1)=0
550   CONTINUE
      II=0
551   AMAX=-1.0
      DO 552 I =1,N
      IF(IND(I,1).NE.0) GO TO 552
      DO 554 J =1,N
      IF(IND(J,1).NE.0) GO TO 554
      TEMP=ABS(A(I,J))
      TE=TEMP-AMAX
      IF (TE.LE.0) GO TO 554
      IR=I
      JC=J
      AMAX=TEMP
554   CONTINUE
552   CONTINUE
      IF(AMAX.EQ.0.) GO TO 555
```

```
      IF(AMAX.LT.0.) GO TO 556
      IND(JC,1)=IR
C     INTERCHANGE ROWS
      IF(IR.EQ.JC) GO TO 559
      DO 560 J=1,N
      TEMP=A(IR,J)
      A(IR,J)=A(JC,J)
      A(JC,J)=TEMP
560   CONTINUE
      II=II+1
      IND(II,2)=JC
C     INVERSION BY GAUSS-JORDAN METHOD
559   PIVOT=A(JC,JC)
      A(JC,JC)=1.0
      PIVOT=1.0/PIVOT
      DO 563 J=1,N
      A(JC,J)=A(JC,J)*PIVOT
563   CONTINUE
      DO 565 I=1,N
      IF(I.EQ.JC) GO TO 565
      TEMP=A(I,JC)
      A(I,JC)=0.
      DO 566 J=1,N
      A(I,J)=A(I,J)-A(JC,J)*TEMP
566   CONTINUE
565   CONTINUE
      GO TO 551
C     REARRANGE ROWS
568   JC=IND(II,2)
      IR=IND(JC,1)
      DO 569 I=1,N
      TEMP=A(I,IR)
      A(I,IR)=A(I,JC)
      A(I,JC)=TEMP
569   CONTINUE
      II=II-1
556   IF(II.EQ.0) GO TO 570
      GO TO 568
555   WRITE(6,820)
820   FORMAT(1H1,3X,22HZERO PIVOT IN A MATRIX)
570   RETURN
      END
C    ******************************************
C    ***************************************************
      SUBROUTINE MULT(A,M1,N1,M1DIM,B,N2,M2DIM,C,M3DIM)
C     THIS SUBROUTINE COMPUTES
C     C=A*B
C     WHERE A IS ORDER OF (M1*N1)
C     WHERE B IS ORDER OF (N1*N2)
C     WHERE C IS ORDER OF (M1*N2)
C     AND WHERE A,B, AND C ARE STORED IN DIMENSIONS OF
 M1DIM,M2DIM, AND M3DIM RESPECTIVELY.
```

```
      DIMENSION A(M1DIM,8),B(M2DIM,1),C(M3DIM,1)
      DO 1 I =1,M1
      DO 1 J =1,N2
      C(I,J)=0.0
      DO 1 K =1,N1
1     C(I,J)=C(I,J)+A(I,K)*B(K,J)
      RETURN
      END
C    *****************************************
C    *****************************************
     SUBROUTINE AMAT(A,WID,AK1,AK2,AK3,AK4,AK5,AK6,AK7,AK8)
     DIMENSION A(8,8)
     EIB=3630371.0
      X1=16.0
      X2=32.0
      X3=48.0
      X4=64.0
      X5=80.0
      X6=96.0
      X7=112.0
      X8=128.0
      DO 30 I =1,8
      DO 30 J =1,8
30    A(I,J)=0.0
      A(1,1)=(6.0*EIB*WID)+(2.0*AK1*X1*X1*((WID-X1)**2))
     A(1,2)=AK2*X1*(WID-X2)*((2.0*WID*X2)-(X2*X2)-(X1*X1))
     A(1,3)=AK3*X1*(WID-X3)*((2.0*WID*X3)-(X3*X3)-(X1*X1))
     A(1,4)=AK4*X1*(WID-X4)*((2.0*WID*X4)-(X4*X4)-(X1*X1))
     A(1,5)=AK5*X1*(WID-X5)*((2.0*WID*X5)-(X5*X5)-(X1*X1))
     A(1,6)=AK6*X1*(WID-X6)*((2.0*WID*X6)-(X6*X6)-(X1*X1))
     A(1,7)=AK7*X1*(WID-X7)*((2.0*WID*X7)-(X7*X7)-(X1*X1))
     A(1,8)=AK8*X1*(WID-X8)*((2.0*WID*X8)-(X8*X8)-(X1*X1))
C
     A(2,1)=AK1*X1*(WID-X2)*((2.0*WID*X2)-(X2*X2)-(X1*X1))
      A(2,2)=(6.0*EIB*WID)+(2.0*AK2*X2*X2*((WID-X2)**2))
     A(2,3)=AK3*X2*(WID-X3)*((2.0*WID*X3)-(X3*X3)-(X2*X2))
     A(2,4)=AK4*X2*(WID-X4)*((2.0*WID*X4)-(X4*X4)-(X2*X2))
     A(2,5)=AK5*X2*(WID-X5)*((2.0*WID*X5)-(X5*X5)-(X2*X2))
     A(2,6)=AK6*X2*(WID-X6)*((2.0*WID*X6)-(X6*X6)-(X2*X2))
     A(2,7)=AK7*X2*(WID-X7)*((2.0*WID*X7)-(X7*X7)-(X2*X2))
     A(2,8)=AK8*X2*(WID-X8)*((2.0*WID*X8)-(X8*X8)-(X2*X2))
C
     A(3,1)=AK1*X1*(WID-X3)*((2.0*WID*X3)-(X3*X3)-(X1*X1))
     A(3,2)=AK2*X2*(WID-X3)*((2.0*WID*X3)-(X3*X3)-(X2*X2))
      A(3,3)=(6.0*EIB*WID)+(2.0*AK3*X3*X3*((WID-X3)**2))
     A(3,4)=AK4*X3*(WID-X4)*((2.0*WID*X4)-(X4*X4)-(X3*X3))
     A(3,5)=AK5*X3*(WID-X5)*((2.0*WID*X5)-(X5*X5)-(X3*X3))
     A(3,6)=AK6*X3*(WID-X6)*((2.0*WID*X6)-(X6*X6)-(X3*X3))
     A(3,7)=AK7*X3*(WID-X7)*((2.0*WID*X7)-(X7*X7)-(X3*X3))
     A(3,8)=AK8*X3*(WID-X8)*((2.0*WID*X8)-(X8*X8)-(X3*X3))
C
     A(4,1)=AK1*X1*(WID-X4)*((2.0*WID*X4)-(X4*X4)-(X1*X1))
```

```
      A(4,2)=AK2*X2*(WID-X4)*((2.0*WID*X4)-(X4*X4)-(X2*X2))
      A(4,3)=AK3*X3*(WID-X4)*((2.0*WID*X4)-(X4*X4)-(X3*X3))
       A(4,4)=(6.0*EIB*WID)+(2.0*AK4*X4*X4*((WID-X4)**2))
      A(4,5)=AK5*X4*(WID-X5)*((2.0*WID*X5)-(X5*X5)-(X4*X4))
      A(4,6)=AK6*X4*(WID-X6)*((2.0*WID*X6)-(X6*X6)-(X4*X4))
      A(4,7)=AK7*X4*(WID-X7)*((2.0*WID*X7)-(X7*X7)-(X4*X4))
      A(4,8)=AK8*X4*(WID-X8)*((2.0*WID*X8)-(X8*X8)-(X4*X4))
C
      A(5,1)=AK1*X1*(WID-X5)*((2.0*WID*X5)-(X5*X5)-(X1*X1))
      A(5,2)=AK2*X2*(WID-X5)*((2.0*WID*X5)-(X5*X5)-(X2*X2))
      A(5,3)=AK3*X3*(WID-X5)*((2.0*WID*X5)-(X5*X5)-(X3*X3))
      A(5,4)=AK4*X4*(WID-X5)*((2.0*WID*X5)-(X5*X5)-(X4*X4))
       A(5,5)=(6.0*EIB*WID)+(2.0*AK5*X5*X5*((WID-X5)**2))
      A(5,6)=AK6*X5*(WID-X6)*((2.0*WID*X6)-(X6*X6)-(X5*X5))
      A(5,7)=AK7*X5*(WID-X7)*((2.0*WID*X7)-(X7*X7)-(X5*X5))
      A(5,8)=AK8*X5*(WID-X8)*((2.0*WID*X8)-(X8*X8)-(X5*X5))
C
      A(6,1)=AK1*X1*(WID-X6)*((2.0*WID*X6)-(X6*X6)-(X1*X1))
      A(6,2)=AK2*X2*(WID-X6)*((2.0*WID*X6)-(X6*X6)-(X2*X2))
      A(6,3)=AK3*X3*(WID-X6)*((2.0*WID*X6)-(X6*X6)-(X3*X3))
      A(6,4)=AK4*X4*(WID-X6)*((2.0*WID*X6)-(X6*X6)-(X4*X4))
      A(6,5)=AK5*X5*(WID-X6)*((2.0*WID*X6)-(X6*X6)-(X5*X5))
       A(6,6)=(6.0*EIB*WID)+(2.0*AK6*X6*X6*((WID-X6)**2))
      A(6,7)=AK7*X6*(WID-X7)*((2.0*WID*X7)-(X7*X7)-(X6*X6))
      A(6,8)=AK8*X6*(WID-X8)*((2.0*WID*X8)-(X8*X8)-(X6*X6))
C
      A(7,1)=AK1*X1*(WID-X7)*((2.0*WID*X7)-(X7*X7)-(X1*X1))
      A(7,2)=AK2*X2*(WID-X7)*((2.0*WID*X7)-(X7*X7)-(X2*X2))
      A(7,3)=AK3*X3*(WID-X7)*((2.0*WID*X7)-(X7*X7)-(X3*X3))
      A(7,4)=AK4*X4*(WID-X7)*((2.0*WID*X7)-(X7*X7)-(X4*X4))
      A(7,5)=AK5*X5*(WID-X7)*((2.0*WID*X7)-(X7*X7)-(X5*X5))
      A(7,6)=AK6*X6*(WID-X7)*((2.0*WID*X7)-(X7*X7)-(X6*X6))
       A(7,7)=(6.0*EIB*WID)+(2.0*AK7*X7*X7*((WID-X7)**2))
      A(7,8)=AK8*X7*(WID-X8)*((2.0*WID*X8)-(X8*X8)-(X7*X7))
C
      A(8,1)=AK1*X1*(WID-X8)*((2.0*WID*X8)-(X8*X8)-(X1*X1))
      A(8,2)=AK2*X2*(WID-X8)*((2.0*WID*X8)-(X8*X8)-(X2*X2))
      A(8,3)=AK3*X3*(WID-X8)*((2.0*WID*X8)-(X8*X8)-(X3*X3))
      A(8,4)=AK4*X4*(WID-X8)*((2.0*WID*X8)-(X8*X8)-(X4*X4))
      A(8,5)=AK5*X5*(WID-X8)*((2.0*WID*X8)-(X8*X8)-(X5*X5))
      A(8,6)=AK6*X6*(WID-X8)*((2.0*WID*X8)-(X8*X8)-(X6*X6))
      A(8,7)=AK7*X7*(WID-X8)*((2.0*WID*X8)-(X8*X8)-(X7*X7))
       A(8,8)=(6.0*EIB*WID)+(2.0*AK8*X8*X8*((WID-X8)**2))
      RETURN
      END
C     ******************************************
C     *******************************************
      SUBROUTINE COMP(WA,STUD,RRIW,KKR,KJR,AK1,AK2,
     1AK3,AK4,AK5,AK6,AK7,AK8,LOOP)
      DIMENSION WA(8),E1(8),STUD(8),RRIW(8),E2(8)
       TTS=1000.*1000.
C     THIS SUBROUTINE COMPUTES THE COMPOSITE PROPERTIES
```

```
      IF(LOOP.EQ.0) GO TO 18
      KKR=KKR-8
      KJR=KJR-8
18    DO 133 MM=KKR,KJR
      RNM=120.97+((68.5*WA(MM))/TTS)
      RDM=1+((1.43*WA(MM))/TTS)
      RRIW(MM)=RNM/RDM
      DDY=RRIW(MM)*WA(MM)
      STUD(MM)=29237760.0/DDY
133    CONTINUE
      AK1=752.0/STUD(KKR)
      AK2=752.0/STUD(KKR+1)
      AK3=752.0/STUD(KKR+2)
      AK4=752.0/STUD(KKR+3)
      AK5=752.0/STUD(KKR+4)
      AK6=752.0/STUD(KKR+5)
      AK7=752.0/STUD(KKR+6)
      AK8=752.0/STUD(KKR+7)
      KKR=KKR+8
      KJR=KJR+8
      RETURN
      END
C     ******************************************
C     *********************************************
       FUNCTION FI(AS,AA,BB)
       DIMENSION AA(410),BB(410)
       CO=0.0
       IF(AS) 10,11,11
   10  AS=AS*(-1.0)
       CO=1.0
   11  DO 1 I=1,403
       IF(AA(I)-AS)1,3,2
    1  CONTINUE
    3  FI=BB(I)
       GO TO 14
    2  SA1=BB(I-1)
       SA2=BB(I)
       SB1=AA(I-1)
       SB2=AA(I)
       FI=(AS-SB1)*(SA2-SA1)/(SB2-SB1)+SA1
   14  IF(CO.EQ.1.)GO TO 15
       RETURN
15      FI=(1.0-FI)
       RETURN
       END
C     ******************************************
      FUNCTION PDF(AS,AA,BB)
      DIMENSION AA(410),BB(410)
       CO=0.0
       IF(AS-0.5)10,11,11
   10  AS=1.-AS
       CO=1.0
```

```
11    DO 1 I =1,403
      IF(BB(I)-AS)1,3,2
    1 CONTINUE
3     PDF=AA(I)
       GO TO 14
2     SA1=AA(I-1)
      SA2=AA(I)
      SB1=BB(I-1)
      SB2=BB(I)
      PDF=(AS-SB1)*(SA2-SA1)/(SB2-SB1)+SA1
   14 IF(CO.EQ.1.)GO TO 15
       RETURN
15    PDF=-1.*PDF
       RETURN
       END
C     *****************************************
C     ****************************************
       FUNCTION RAN1(IDUM)
C     Reference (Press et al 1986)
       DIMENSION R(97)
       PARAMETER (M1=259200,IA1=7141,IC1=54773)
       PARAMETER (M2=134456,IA2=8121,IC2=28411)
       PARAMETER (M3=243000,IA3=4561,IC3=51349)
       RM1 = 1./M1
       RM2 = 1./M2
      IF(IDUM.LT.0) THEN
         IFF=1
         IX1=MOD(IC1-IDUM,M1)
         IX1=MOD(IA1*IX1+IC1,M1)
         IX2=MOD(IX1,M2)
         IX1=MOD(IA1*IX1+IC1,M1)
         IX3=MOD(IX1,M3)
             DO 11 J=1,97
             IX1=MOD(IA1*IX1+IC1,M1)
             IX2=MOD(IA2*IX2+IC2,M2)
             R(J) = (FLOAT(IX1) + FLOAT(IX2)*RM2)*RM1
C             WRITE(6,6888) J,R(J)
 6888        FORMAT(I7,F15.5)
 11          CONTINUE
          IDUM=1
        ENDIF
        IX1=MOD(IA1*IX1+IC1,M1)
        IX2=MOD(IA2*IX2+IC2,M2)
        IX3=MOD(IA3*IX3+IC3,M3)
        J=1+(97*IX3)/M3
        IF(J.GT.97.OR.J.LT.1) WRITE(6,6700) J
 6700   FORMAT('MISTAKE, J=',I7,'  WHAT IS LT.1.OR.GT.97')
        RAN1 = R(J)
        R(J)=(FLOAT(IX1)+FLOAT(IX2)*RM2)*RM1
C        WRITE(6,6888) J,R(J)
        RETURN
        END
```

```
C     ********************************
C     **************************************
      FUNCTION GASDEV(IDUM)
      DATA ISET/0/
      IF(ISET.EQ.0) THEN
1     V1=2.*RAN1(IDUM)-1.
      V2=2.*RAN1(IDUM)-1.
      R=V1**2+V2**2
      IF(R.GE.1.) GO TO 1
      FAC=SQRT(-2.*LOG(R)/R)
      GSET=V1*FAC
      GASDEV=V2*FAC
      ISET=1
      ELSE
      GASDEV=GSET
      ISET=0
      ENDIF
      RETURN
      END
C     ***********************************
C     ***************************************
      FUNCTION GAUSS(IDUM)
      U1=RAN1(IDUM)
      U2=RAN1(IDUM)
      U3=RAN1(IDUM)
      IF(U1.LE.0.8638) THEN
      GAUSS=2.0*(U1+U2+U3-1.5)
      ELSE
      IF(U1.LE.0.9745) THEN
      GAUSS=1.5*(U1+U2-1.)
      ELSE
      IF(U1.LE.0.9973002039) THEN
31    X=6.0*RAN1(IDUM)-3.0
      Y=0.358*RAN1(IDUM)
      A=17.49731196
      B=2.36785163
      C=2.15787544
      D=X*X/2.0
      IF(ABS(X).LT.1.0) THEN
      G=(A*EXP(-D))-(2.0*B*(3.-X*X))-(C*(1.5-ABS(X)))
      ELSE
      IF(ABS(X).LT.1.5) THEN
      G=A*(EXP(-D))-B*((3.-ABS(X))**2)-C*(1.5-ABS(X))
      ELSE
      IF(ABS(X).LT.3.0) THEN
      G=A*(EXP(-D))-B*((3.-ABS(X))**2)
      ELSE
      IF(ABS(X).GE.3.0) THEN
      G=0.
      ENDIF
      ENDIF
      ENDIF
```

```
      ENDIF
      IF(Y.LT.G) THEN
      GAUSS=X
      ELSE
      GO TO 31
      ENDIF
      IF(U1.LE.1.0) THEN
32    V1=2.*RAN1(IDUM)-1.0
      V2=2.*RAN1(IDUM)-1.0
      W=(V1*V1+V2*V2)
      Q=(9.0-2.0*ALOG(W))/W
      S=V1*SQRT(Q)
      T=V2*SQRT(Q)
      IF(T.GT.3.0.AND.S.GT.3.0) THEN
      GAUSS=S
      ELSE
      IF(S.LE.3.0.AND.T.GT.3.0) THEN
      GAUSS=T
      ELSE
      GO TO 32
      ENDIF
      ENDIF
      ENDIF
      ENDIF
      ENDIF
      ENDIF
      RETURN
      END
```

APPENDIX H

Computer Program for the Computation of

β in Wall Systems

```
C     ***************************************************
      $LARGE
C     THIS PROGRAM CALCULATES THE SAFETY OR THE RELIABILITY
C      INDEX
C      'BETA' FOR THE FOUR STATES OF FAILURE.
C      STAGE 1 : FAILURE OF STAGE 1, GYPSUM FLANGE ENTERING
C      NON-LINEAR STATE.
C     STAGE 2 : FAILURE OF STAGE 2, GYPSUM FLANGE + PLYWOOD
C      FLANGE
C      ENTERING NON-LINEAR STATE.
C     STAGE 3 : FAILURE OF STATE 3, GYPSUM FLANGE + PLYWOOD
C      FLANGE
C      + STUD ENTERING NON-LINEARITY.
C     STAGE 4 : FAILURE OF STATE 4, GYPSUM FLANGE + PLYWOOD
C      FLANGE
C      + STUD(MOR) OR THE FINAL RUPTURE STRUCTURE.
C     ****************************************************
       DIMENSION
      AA(410),BB(410),GX(10),GY(10),ALF(10),SY(10),XS1(10),
     1XS(10),XSIG(10),XM(10),GL1(10),GL2(10),YS(10),GL(10,10),
     2S(10,10),SN(10,10),T(10,10),TT(10,10),FFUN(11),TEMP(11)
       INTEGER NA,JOBNA,IZA,IER,NB,JOBNB,IZB
       REAL RA(3),RB(10),DA(2),DD(4),Z(2,2),ZZ(4,4),
      WK(3)
       CHARACTER *72 STATE1,STATE2,STATE3,STATE4,PROB1,PROB2
       OPEN(UNIT=1,FILE='D:\JUNK\NGAUSS',STATUS='OLD')
      OPEN(UNIT=2,FILE='D:\JUNK\BETA',STATUS='OLD')
      OPEN(UNIT=6,FILE='D:\JUNK\OUT',STATUS='NEW')
C      INPUT 0 OR 1 IF COMPOSITE OR NO COMPOSITE ACTION IS
      REQUIRED.
       WRITE(*,*) 'PLEASE INPUT STUD SPACING 16 IN
      OR 24 IN '
       READ(*,*) SPG
       WRITE(*,*) 'PLEASE INPUT LOAD SHARING
      FACTOR 1 OR ?  '
       READ (*,*) RLF
       RLD=(135.83/12.0)*SPG
       WRITE(*,*) 'PLEASE INPUT COMPOSITE ACTION REQ= 0.
       ELSE 1.'
       READ(*,*) COMP
       WRITE(*,*) 'PLEASE INPUT AXIAL LOAD REQ=0. ELSE 1.'
```

```
      READ(*,*) AXILO
C     READ DATA WHICH ARE IN A FILE CALLED GAUSS.
      READ(1,1000)(AA(I),BB(I),I=1,403)
1000  FORMAT(F4.2,F8.6)
C     READ DATA FROM A FILE CALLED AS BETA,
     (INITIAL AND LAST)
C     INPUT POSSIBLE BETA VALUES FOR FOUR STATES OF
      FAILURE.
C     RANGES FROM  (-4.0,1.0,3.1,AND 3.3)
      READ(2,711) STATE2
      READ(2,1050)BETIN1,BETFI1
      READ(2,711) STATE3
      READ(2,1050)BETIN2,BETFI2
      READ(2,711) STATE4
      READ(2,1050)BETIN3,BETFI3
      READ(2,711) STATE5
      READ(2,1050)BETIN4,BETFI4
C    READ THE CORRELATION COEFFICIENTS AND THIS EXISTS FOR
C     STATE 4 AND STATE 5 ONLY.
C       INPUT ONLY THE UPPER OR THE LOWER PART OF THE
     SYMMETRIC MATRIX.
      READ(2,711) PROB1
      DO 21 J =1,3
21    READ(2,*) RA(J)
      READ(2,711) PROB2
      DO 22 K=1,10
22    READ(2,*) RB(K)
1050  FORMAT(2F6.3)
C     INPUT PROPERTIES OF SHEATHING.
     WRITE(*,*) 'INPUT SHEATHING PROPERTIES'
      READ(4,*) SKC1,SKC2,SKT1,SKT2
     WRITE(*,*) 'INPUT STUD PROPERTIES'
      READ(5,*) ETA1,AMU1,SIG1
      READ(5,*) ETA2,AMU2,SIG2
      READ(5,*) ETA3,AMU3,SIG3
      READ(5,*) ETA4,AMU4,SIG4
711   FORMAT(A72)
      ECC=1730.
      ECT=230.
      CAS=5.25
      CAC=6.0
      CAT=6.0
      BETDE=(BETFI1-BETIN1)/10.
      PI=3.14159
      ERRL=0.005
      ERRS=1/(10.**9.)
C     DATA ASSOCIATED WITH WIND VELOCITY
      AMF=0.63
      CONS=0.00256
      POMM=10.**6.
     WRITE(*,*) 'INPUT WIND LOAD PARAMETERS'
      READ(7,*) VA,VB
```

```
C       VA=0.1265
C       VB=103.35
      IF(COMP.EQ.1.) GO TO 778
C     CALCULATION OF BETA AND HENCE THE FAILURE
     PROBABILITY OF
C     GYPSUM FLANGE. (STATE 2)
      WRITE(6,2000)
2000  FORMAT(//20X,'RELIABILITY INDEX BETA FOR STATE-1')
      WRITE(6,257)
C       NOTE DEAD LOAD CHANGES FOR 16 IN AND 24 IN STUD
SPACING.
C, DEAD ROOF LOAD - AXIAL FORCE X5:
      XM(2)=RLD
      XSIG(2)=RLD*0.1
C, EXTERNAL PRESSURE COEFFICIENT CP+   X6:
      XM(3)=20.13
      XSIG(3)=0.13*20.13
C     EXTERNAL PRESSURE COEFFICIENT CP4   X7:
      XM(4)=0.8
      XSIG(4)=0.12*0.8
C     EXPOSURE COEFFICIENT KZ  X8:
      XM(5)=1.2
      XSIG(5)=0.16*1.2
C     GUST FACTOR
      XM(6)=1.15
      XSIG(6)=0.11*1.15
C     INPUT OF TRANSFORMATION MATRIX T
      CALL TRMATRIX(T,TT,7)
C     INPUT OF ST.DEV. FOR RANDOM VARIABLE Y,
C     THEY ARE IN FACTS THE ROOTS OF EIGENVALUE PROBLEM
      DO 191 I =1,7
191   SY(I)=SQRT(1.0)
      KK=0
      BETA=BETIN1-BETDE
      DO 33 IJK=1,11
      BETA=BETA+BETDE
      CALL TITLE(BETA)
      KK=0
42    XS(1)=1500000.
      XS(2)=180.
      XS(3)=2.5
      XS(4)=1.
      XS(5)=1.
      XS(6)=1.6
      XS(7)=100.
41    IF(XS(1).LT.AMU1)XS(1)=AMU1*1.1
                                             C     A     L     L
WEIBULL(ETA1,AMU1,SIG1,AA,BB,XS(1),XM(1),XSIG(1))
      CALL TYPEI(VA,VB,AA,BB,XS(7),XM(7),XSIG(7))
      CALL YSPACE(GL1,GL2,YS,GL,XS,XM,XSIG,T,TT,S,7)
C     SOME VALUES WILL BE MULTIPLIED BY POMM=10**6
     BECAUSE THEY ARE TOO SMALL. LATER ON,
```

```
      THE SAME VALUES WILL BE AGAIN DIVIDED
C      BY POMM. THIS IS DONE BECAUSE OF COMPUTER INACCURACY
       SNC=(ECC*1000.)/XS(1)
       SNT=(ECT*1000.)/XS(1)
       CALL RMOI(CAS,CAC,CAT,SKC1,SKC2,SKT1,
      SKT2,SNC,SNT,XS(1),XS(1),
      1RI,AT,AC)
      AAMO=601385421.5+1158.38*XS(1)+2.05*XS(1)*
      XS(1)/10000.
       AAMU=348432.0+XS(1)
       AAM=AAMO/AAMU
       AAM1=((1158.38+2.*2.05*XS(1)/10000.)
      *AAMU-AAMO)*POMM/(AAMU*AAMU)
       BBM=1152.*CONS*XS(4)*XS(5)*XS(6)*
      XS(7)*XS(7)*SPG*AMF/144.
       W=1.*CONS*XS(4)*XS(5)*XS(6)*XS(7)*XS(7)*SPG*AMF/144.
       IF(AXILO.EQ.1.) W=0.
       P=-XS(2)+1.*CONS*XS(3)*XS(5)*
      XS(6)*XS(7)*XS(7)*SPG*AMF/12.
       CALL FACT(W,P,XS(1),RI,FACT1,FACT2)
C      EVALUATING DP/DX
       P5=-1.
       P6=1.*CONS*XS(5)*XS(6)*XS(7)*XS(7)*SPG*AMF/12.
       P8=1.*CONS*XS(3)*XS(6)*XS(7)*XS(7)*SPG*AMF/12.
       P9=1.*CONS*XS(3)*XS(5)*XS(7)*XS(7)*SPG*AMF/12.
       P10=1.*CONS*2.*XS(3)*XS(5)*XS(6)*XS(7)*SPG*AMF/12.
C      EVALUATING DW/DX
       IF(AXILO.EQ.1.) GO TO 501
       DW7=CONS*XS(5)*XS(6)*XS(7)*XS(7)*SPG*AMF/144.
       DW8=CONS*XS(4)*XS(6)*XS(7)*XS(7)*SPG*AMF/144.
       DW9=CONS*XS(4)*XS(5)*XS(7)*XS(7)*SPG*AMF/144.
       DW10=2.0*CONS*XS(4)*XS(5)*XS(6)*XS(7)*SPG*AMF/144.
C      EVALUATING DM/DX
501    DM7=1152.*CONS*XS(5)*XS(6)*XS(7)*XS(7)*SPG*AMF/144.
       DM8=1152.*CONS*XS(4)*XS(6)*XS(7)*XS(7)*SPG*AMF/144.
       DM9=1152.*CONS*XS(4)*XS(5)*XS(7)*XS(7)*SPG*AMF/144.
       DM10=2.0*1152.*CONS*XS(4)*XS(5)*XS(6)*XS(7)*
      SPG*AMF/144.
C     NOW, EARLIER MULTIPILIED VALUES BY POMM=10**6 WILL BE
       DIVIDED
       GX(1)=(AAM1/POMM)+(FACT1*P/XS(1))
       GX(2)=-FACT1*P5
       GX(3)=-FACT1*P6
       GX(4)=-DM7-(FACT2*DW7)
       GX(5)=-DM8-(FACT2*DW8)-(FACT1*P8)
       GX(6)=-DM9-(FACT2*DW9)-(FACT1*P9)
       GX(7)=-DM10-(FACT2*DW10)-(FACT1*P10)
       CALL DGDB(GL,GX,SY,T,S,XS1,XM,BETA,7)
       KK=KK+1
       CALL VEC(XS1,KK,7)
       DO 25 I=1,7
       DIFF=ABS((XS(I)-XS1(I))/XS(I))
```

```
      IF(KK.EQ.16)GO TO 43
      IF(DIFF.GT.ERRL)GO TO 40
25    CONTINUE
      GO TO 43
40    DO 26 I=1,7
26    XS(I)=XS1(I)
      GO TO 41
43    FFUN(IJK)=AAM-BBM-(FACT1*P)
      TEMP(IJK)=BETA
      WRITE(6,1055)FFUN(IJK)
1055  FORMAT(18X,'FAILURE FUNCTION G=',F15.7)
33    CONTINUE
257   FORMAT(20X,//35('*')//)
C    CALCULATION OF BETA AND HENCE THE FAILURE PROBABILITY
     OF GYPSUM FLANGE + PLYWOOD FLANGE. (STATE 2)
      WRITE(6,257)
      WRITE(6,2001)
2001  FORMAT(//20X,'RELIABILITY INDEX BETA FOR STATE-2')
      WRITE(6,257)
C, DEAD ROOF LOAD - AXIAL FORCE X5:
      XM(2)=RLD
      XSIG(2)=RLD*0.1
C, EXTERNAL PRESSURE COEFFICIENT CP+   X6:
      XM(3)=20.13
      XSIG(3)=0.13*20.13
C     EXTERNAL PRESSURE COEFFICIENT CP4   X7:
      XM(4)=0.8
      XSIG(4)=0.12*0.8
C     EXPOSURE COEFFICIENT KZ  X8:
      XM(5)=1.2
      XSIG(5)=0.16*1.2
C     GUST FACTOR
      XM(6)=1.15
      XSIG(6)=0.11*1.15
      CALL TRMATRIX(T,TT,7)
      DO 291 I =1,7
291   SY(I)=SQRT(1.0)
      KK=0
      BETDE=(BETFI2-BETIN2)/10.0
      BETA=BETIN2-BETDE
      DO 233 IJK=1,11
      BETA=BETA+BETDE
      CALL TITLE(BETA)
      KK=0
242   XS(1)=1500000.
      XS(2)=180.
      XS(3)=2.5
      XS(4)=1.
      XS(5)=1.
      XS(6)=1.6
      XS(7)=100.
C     COMPUTATION OF MEANS AND ST.DEVS OF
```

```
     EQUIVALENT NORMAL
      P.D.F.'S
241   IF(XS(1).LT.AMU1)XS(1)=AMU1*1.1
                                        C     A     L     L
WEIBULL(ETA1,AMU1,SIG1,AA,BB,XS(1),XM(1),XSIG(1))
      CALL TYPEI(VA,VB,AA,BB,XS(7),XM(7),XSIG(7))
      CALL YSPACE(GL1,GL2,YS,GL,XS,XM,XSIG,T,TT,S,7)
      SNC=(ECC*1000.)/XS(1)
      SNT=(ECT*1000.)/XS(1)
      CALL RMOI(CAS,CAC,CAT,SKC1,SKC2,SKT1,
     SKT2,SNC,SNT,XS(1),XS(1),
     1RI,AT,AC)
      CO=207292710.8+1702.5*XS(1)+2.02*XS(1)*XS(1)/1000.
      CU=132201.73+XS(1)
      C=CO/CU
      C1=((1702.5+2.*2.02*XS(1)/1000.)*CU-CO)*POMM/(CU*CU)
      AAMO=601385421.5+1158.38*XS(1)+2.05*XS(1)*
     XS(1)/10000.
      AAMU=348432.0+XS(1)
      AAM=AAMO/AAMU
      AAM1=((1158.38+2.*2.05*XS(1)/10000.)
     *AAMU-AAMO)*POMM/(AAMU*AAMU)
      BBM=1152.*CONS*XS(4)*XS(5)*XS(6)*XS(7)*
     XS(7)*SPG*AMF/144.
      W=1.*CONS*XS(4)*XS(5)*XS(6)*XS(7)*XS(7)
     *SPG*AMF/144.
      IF(AXILO.EQ.1.) W=0.
      P=-XS(2)+1.*CONS*XS(3)*XS(5)*XS(6)*
     XS(7)*XS(7)*SPG*AMF/12.
      CALL FACT(W,P,XS(1),RI,FACT1,FACT2)
C     EVALUATING DP/DX
      P5=-1.
      P6=1.*CONS*XS(5)*XS(6)*XS(7)*XS(7)*SPG*AMF/12.
      P8=1.*CONS*XS(3)*XS(6)*XS(7)*XS(7)*SPG*AMF/12.
      P9=1.*CONS*XS(3)*XS(5)*XS(7)*XS(7)*SPG*AMF/12.
      P10=1.*CONS*2.*XS(3)*XS(5)*XS(6)*XS(7)*SPG*AMF/12.
C     EVALUATING DW/DX
      IF(AXILO.EQ.1.) GO TO 502
      DW7=CONS*XS(5)*XS(6)*XS(7)*XS(7)*SPG*AMF/144.
      DW8=CONS*XS(4)*XS(6)*XS(7)*XS(7)*SPG*AMF/144.
      DW9=CONS*XS(4)*XS(5)*XS(7)*XS(7)*SPG*AMF/144.
      DW10=2.0*CONS*XS(4)*XS(5)*XS(6)*XS(7)*SPG*AMF/144.
C     EVALUATING DM/DX
502   DM7=1152.*CONS*XS(5)*XS(6)*XS(7)*XS(7)*SPG*AMF/144.
      DM8=1152.*CONS*XS(4)*XS(6)*XS(7)*XS(7)*SPG*AMF/144.
      DM9=1152.*CONS*XS(4)*XS(5)*XS(7)*XS(7)*SPG*AMF/144.
     DM10=2.0*1152.*CONS*XS(4)*XS(5)*XS(6)*XS(7)*
     SPG*AMF/144.
      GX(1)=((C1+AAM1)/POMM)+(FACT1*P/XS(1))
      GX(2)=-FACT1*P5
      GX(3)=-FACT1*P6
      GX(4)=-DM7-(FACT2*DW7)
```

```
      GX(5)=-DM8-(FACT2*DW8)-(FACT1*P8)
      GX(6)=-DM9-(FACT2*DW9)-(FACT1*P9)
      GX(7)=-DM10-(FACT2*DW10)-(FACT1*P10)
      CALL DGDB(GL,GX,SY,T,S,XS1,XM,BETA,7)
      KK=KK+1
      CALL VEC(XS1,KK,7)
      DO 225 I=1,7
      DIFF=ABS((XS(I)-XS1(I))/XS(I))
      IF(KK.EQ.16)GO TO 243
      IF(DIFF.GT.ERRL)GO TO 240
225   CONTINUE
      GO TO 243
240   DO 226 I=1,7
226   XS(I)=XS1(I)
      GO TO 241
243   FFUN(IJK)=C+AAM-BBM-(FACT1*P)
      TEMP(IJK)=BETA
      WRITE(6,1055)FFUN(IJK)
233   CONTINUE
      CALL PFL(FFUN,TEMP,AA,BB,BETA,PF)
      WRITE(6,257)
      WRITE(6,262) BETA,PF
262   FORMAT(//7X, 'RELIABILITY INDEX BETA
     FOR STATE 2 =',F5.2,//,
     1 7X,'PROBABILITY OF ENTERING STATE 3 =',E12.5//)
      WRITE(6,257)
C     CALCULATION OF BETA AND HENCE THE FAILURE
     PROBABILITY OF
C     GYPSUM FLANGE + PLYWOOD FLANGE + STUD (MOE1).
     (STATE 3)
778   WRITE(6,2002)
2002  FORMAT(//20X,'RELIABILITY INDEX BETA FOR STATE-3')
C, DEAD ROOF LOAD - AXIAL FORCE X5:
      XM(3)=RLD
      XSIG(3)=RLD*0.1
C, EXTERNAL PRESSURE COEFFICIENT CP+   X6:
      XM(4)=20.13
      XSIG(4)=0.13*20.13
C, EXTERNAL PRESSURE COEFFICIENT CP4   X7:
      XM(5)=0.8
      XSIG(5)=0.12*0.8
C, EXPOSURE COEFFICIENT KZ  X8:
      XM(6)=1.2
      XSIG(6)=0.16*1.2
C     GUST FACTOR
      XM(7)=1.15
      XSIG(7)=0.11*1.15
C     INPUT OF TRANSFORMATION MATRIX T
      CALL TRMATRIX(T,TT,8)
C      TO GET THE EIGENVECTORS AND THE EIGENVALUE, IMSL
ROUTINE IS
C     MADE USE OF.
```

```
      NA=2
      IZA=2
      JOBNA=2
      CALL EIGRS(RA,NA,JOBNA,DA,Z,IZA,WK,IER)
      DO 391 I =1,2
      DO 392 J =1,2
      T(I,J)=Z(I,J)
392   CONTINUE
391   CONTINUE
      CALL TRANSP(T,TT,SY,DA,2,8)
      KK=0
      BETDE=(BETFI3-BETIN3)/10.0
      BETA=BETIN3-BETDE
      DO 333 IJK=1,11
      BETA=BETA+BETDE
      CALL TITLE(BETA)
      KK=0
342   XS(1)=1500000.
      XS(2)=4000.
      XS(3)=180.
      XS(4)=2.5
      XS(5)=1.
      XS(6)=1.
      XS(7)=1.6
      XS(8)=100.
341   IF(XS(1).LT.AMU1)XS(1)=AMU1*1.1
                                          C     A     L     L
WEIBULL(ETA1,AMU1,SIG1,AA,BB,XS(1),XM(1),XSIG(1))
      IF(XS(2).LT.AMU3)XS(2)=AMU3*1.1
                                          C     A     L     L
WEIBULL(ETA3,AMU3,SIG3,AA,BB,XS(2),XM(2),XSIG(2))
      CALL TYPEI(VA,VB,AA,BB,XS(8),XM(8),XSIG(8))
      CALL YSPACE(GL1,GL2,YS,GL,XS,XM,XSIG,T,TT,S,8)
      SNC=(ECC*1000.)/XS(1)
      SNT=(ECT*1000.)/XS(1)
      CALL RMOI(CAS,CAC,CAT,SKC1,SKC2,SKT1,
     SKT2,SNC,SNT,XS(1),XS(1),
     1RI,AT,AC)
      BO=675221.24+7.64*XS(1)+1.86*XS(1)*XS(1)/100000.
      BU=27138.64+XS(1)+5.99*XS(1)*XS(1)/1000000.
      B=BO/BU
      B1=(7.64+2.*1.86*XS(1)/100000.)*BU
      B1=(B1-(1.+2.*5.99*XS(1)/1000000.)*BO)*POMM/(BU*BU)
      IF(COMP.EQ.1.) GO TO 706
      CO=207292710.8+1702.5*XS(1)+2.02*XS(1)*XS(1)/1000.
      CU=132201.73+XS(1)
      C=CO/CU
      C1=((1702.5+2.*2.02*XS(1)/1000.)*CU-CO)*POMM/(CU*CU)
      DOO=8331555.0+147.25*XS(1)+6.493*XS(1)*XS(1)/10000.
      DU=132201.7+XS(1)
      D=DOO/DUD1=((147.25+2.*6.493*XS(1)/10000.)*DU-DOO)*
     POMM/(DU*DU)
```

```
      HO=24165738.7+258.39*XS(1)+6.6*XS(1)*XS(1)/100000.
      HU=348432.0+XS(1)
      H=HO/HU
      H1=((258.39+2.*6.6*XS(1)/100000.)*HU-HO)*POMM/(HU*HU)
      AAMO=601385421.5+1158.38*XS(1)+2.05*XS(1)*
     XS(1)/10000.
      AAMU=348432.0+XS(1)
      AAM=AAMO/AAMU
      AAM1=((1158.38+2.*2.05*XS(1)/10000.)
     *AAMU-AAMO)*POMM/(AAMU*AAMU)
706   P=-XS(3)+1.*CONS*XS(4)*XS(6)*
     XS(7)*XS(8)*XS(8)*SPG*AMF/12.
      BBM=1152.*CONS*XS(5)*XS(6)*XS(7)*
     XS(8)*XS(8)*SPG*AMF/144.
      W=1.*CONS*XS(5)*XS(6)*XS(7)*XS(8)*XS(8)*
     SPG*AMF/144.
      IF(AXILO.EQ.1.) W=0.
      CALL FACT(W,P,XS(1),RI,FACT1,FACT2)
C     EVALUATING DP/DX
      P3=-1.
      P4=1.*CONS*XS(6)*XS(7)*XS(8)*XS(8)*SPG*AMF/12.
      P6=1.*CONS*XS(4)*XS(7)*XS(8)*XS(8)*SPG*AMF/12.
      P7=1.*CONS*XS(4)*XS(6)*XS(8)*XS(8)*SPG*AMF/12.
      P8=1.*CONS*2.*XS(4)*XS(6)*XS(7)*XS(8)*SPG*AMF/12.
C     EVALUATING DW/DX
      IF(AXILO.EQ.1.) GO TO 503
      DW5=CONS*XS(6)*XS(7)*XS(8)*XS(8)*SPG*AMF/144.
      DW6=CONS*XS(5)*XS(7)*XS(8)*XS(8)*SPG*AMF/144.
      DW7=CONS*XS(5)*XS(6)*XS(8)*XS(8)*SPG*AMF/144.
      DW8=2.0*CONS*XS(5)*XS(6)*XS(7)*XS(8)*SPG*AMF/144.
C     EVALUATING DM/DX
503   DM5=1152.*CONS*XS(6)*XS(7)*XS(8)*XS(8)*SPG*AMF/144.
      DM6=1152.*CONS*XS(5)*XS(7)*XS(8)*XS(8)*SPG*AMF/144.
      DM7=1152.*CONS*XS(5)*XS(6)*XS(8)*XS(8)*SPG*AMF/144.

DM8=2.0*1152.*CONS*XS(5)*XS(6)*XS(7)*XS(8)*SPG*AMF/144.
      GX(1)=B1*(XS(2)-H-P/5.25-D)+C1+AAM1-(H1+D1)*B
      GX(1)=GX(1)/POMM+(FACT1*P/XS(1))
      GX(2)=B
      GX(3)=B*P3/(-5.25)-(FACT1*P3)
      GX(4)=B*P4/(-5.25)-(FACT1*P4)
      GX(5)=-DM5-(FACT2*DW5)
      GX(6)=B*P6/(-5.25)-DM6-(FACT1*P6)-(FACT2*DW6)
      GX(7)=B*P7/(-5.25)-DM7-(FACT1*P7)-(FACT2*DW7)
      GX(8)=B*P8/(-5.25)-DM8-(FACT1*P8)-(FACT2*DW8)
      CALL DGDB(GL,GX,SY,T,S,XS1,XM,BETA,8)
      KK=KK+1
      CALL VEC3(XS1,KK,8)
      DO 325 I=1,8
      DIFF=ABS((XS(I)-XS1(I))/XS(I))
      IF(KK.EQ.16)GO TO 343
      IF(DIFF.GT.ERRL)GO TO 340
```

```
325   CONTINUE
      GO TO 343
340   DO 326 I=1,8
326   XS(I)=XS1(I)
      GO TO 341
343   FFUN(IJK)=B*((RLF*XS(2))-H-P/5.25-D)+AAM-
     BBM+C-(FACT1*P)
      TEMP(IJK)=BETA
      WRITE(6,1055)FFUN(IJK)
333   CONTINUE
      CALL PFL(FFUN,TEMP,AA,BB,BETA,PF)
      WRITE(6,257)
      WRITE(6,263) BETA,PF
263   FORMAT(//7X,'RELIABILITY INDEX BETA FOR STATE 3
      =',F5.2,//,
     17X,'PROBABILITY OF ENTERING STATE 4=',E12.5,//)
      WRITE(6,257)
C     CALCULATION OF BETA AND HENCE THE FAILURE PROBABILITY
     OF  STAGE 4
666   WRITE(6,2003)
2003  FORMAT(//20X,'RELIABILITY INDEX BETA FOR STATE-4 OR
      RUPTURE')
C, DEAD ROOF LOAD - AXIAL FORCE X5:
      XM(5)=RLD
      XSIG(5)=RLD*0.1
C, EXTERNAL PRESSURE COEFFICIENT CP+   X6:
      XM(6)=20.13
      XSIG(6)=0.13*20.13
C, EXTERNAL PRESSURE COEFFICIENT CP4   X7:
      XM(7)=0.8
      XSIG(7)=0.12*0.8
C, EXPOSURE COEFFICIENT KZ  X8:
      XM(8)=1.2
      XSIG(8)=0.16*1.2
C, GUST FACTOR
      XM(9)=1.15
      XSIG(9)=0.11*1.15
C     INPUT OF TRANSFORMATION MATRIX T
      CALL TRMATRIX(T,TT,10)
C       TO GET THE EIGENVECTORS AND THE EIGENVALUE, IMSL
ROUTINE IS
C     USE OF.
      NB=4
      IZB=4
      JOBNB=2
      CALL EIGRS(RB,NB,JOBNB,DD,ZZ,IZB,WK,IER)
      DO 491 I =1,4
      DO 492 J =1,4
      T(I,J)=ZZ(I,J)
492   CONTINUE
491   CONTINUE
      CALL TRANSP(T,TT,SY,DD,4,10)
```

```
      KK=0
      BETDE=(BETFI4-BETIN4)/10.0
      BETA=BETIN4-BETDE
      DO 433 IJK=1,11
      BETA=BETA+BETDE
      CALL TITLE(BETA)
      KK=0
442   XS(1)=1500000.
      XS(2)=500000.
      XS(3)=4000.
      XS(4)=4000.
      XS(5)=180.
      XS(6)=2.5
      XS(7)=1.
      XS(8)=1.
      XS(9)=1.6
      XS(10)=100.
441   IF(XS(1).LT.AMU1)XS(1)=AMU1*1.1
                                             C     A     L     L
WEIBULL(ETA1,AMU1,SIG1,AA,BB,XS(1),XM(1),XSIG(1))
      IF(XS(2).LT.AMU2)XS(2)=AMU2*1.1
      CALL WEIBULL(ETA2,AMU2,SIG2,AA,BB,XS(2),
     XM(2),XSIG(2))
C      CALL STYPEI(VC,VD,AA,BB,XS(2),XM(2),XSIG(2))
      IF(XS(3).LT.AMU3)XS(3)=AMU3*1.1
      CALL WEIBULL(ETA3,AMU3,SIG3,AA,BB,XS(3),
     XM(3),XSIG(3))
      IF(XS(4).LT.AMU4)XS(4)=AMU4*1.1
      CALL WEIBULL(ETA4,AMU4,SIG4,AA,BB,XS(4),
     XM(4),XSIG(4))
      CALL TYPEI(VA,VB,AA,BB,XS(10),XM(10),XSIG(10))
      CALL YSPACE(GL1,GL2,YS,GL,XS,XM,XSIG,T,TT,S,10)
      SNC=(ECC*1000.)/XS(2)
      SNT=(ECT*1000.)/XS(2)
      CALL RMOI(CAS,CAC,CAT,SKC1,SKC2,SKT1,
     SKT2,SNC,SNT,XS(1),XS(2),1RI,AT,AC)
      AO=675221.24+7.64*XS(2)+1.86*XS(2)*XS(2)/100000.
      AU=27138.64+XS(2)+5.99*XS(2)*XS(2)/1000000.
      A=AO/AU
      A2=(7.64+2.*1.86*XS(2)/100000.)*AU
      A2=(A2-(1.+2.*5.99*XS(2)/1000000.)*AO)*POMM/(AU*AU)
      BO=675221.24+7.64*XS(1)+1.86*XS(1)*XS(1)/100000.
      BU=27138.64+XS(1)+5.99*XS(1)*XS(1)/1000000.
      B=BO/BU
      B1=(7.64+2.*1.86*XS(1)/100000.)*BU
      B1=(B1-(1.+2.*5.99*XS(1)/1000000.)*BO)*
     POMM/(BU*BU)
      IF(COMP.EQ.1.) GO TO 707
      CO=207292710.8+1702.5*XS(1)+2.02*XS(1)*XS(1)/1000.
      CU=132201.73+XS(1)
      C=CO/CU
      C1=((1702.5+2.*2.02*XS(1)/1000.)*CU-CO)*POMM/(CU*CU)
```

```
      DOO=8331555.0+147.25*XS(1)+6.493*XS(1)*XS(1)/10000.
      DU=132201.7+XS(1)
      D=DOO/DU
      D1=((147.25+2.*6.493*XS(1)/10000.)*DU-DOO)*
     POMM/(DU*DU)
      HO=24165738.7+258.39*XS(1)+6.6*XS(1)*XS(1)/100000.
      HU=348432.0+XS(1)
      H=HO/HU
      H1=((258.39+2.*6.6*XS(1)/100000.)*HU-HO)*POMM/(HU*HU)
      AAMO=601385421.5+1158.38*XS(1)+2.05*XS(1)*
     XS(1)/10000.
      AAMU=348432.0+XS(1)
      AAM=AAMO/AAMU
      AAM1=((1158.38+2.*2.05*XS(1)/10000.)*
     AAMU-AAMO)*POMM/(AAMU*AAMU)
707   P=-XS(5)+1.*CONS*XS(6)*XS(8)*XS(9)*XS(10)*
     XS(10)*SPG*AMF/12.
      BBM=1152.*CONS*XS(7)*XS(8)*XS(9)*
     XS(10)*XS(10)*SPG*AMF/144.
      W=1.*CONS*XS(7)*XS(8)*XS(9)*XS(10)*XS(10)*
     SPG*AMF/144.
      IF(AXILO.EQ.1.) W=0.
      CALL FACT(W,P,XS(2),RI,FACT1,FACT2)
C     EVALUATING DP/DX
      P5=-1.
      P6=1.*CONS*XS(8)*XS(9)*XS(10)*XS(10)*SPG*AMF/12.
      P8=1.*CONS*XS(6)*XS(9)*XS(10)*XS(10)*SPG*AMF/12.
      P9=1.*CONS*XS(6)*XS(8)*XS(10)*XS(10)*SPG*AMF/12.
      P10=1.*CONS*2.*XS(6)*XS(8)*XS(9)*XS(10)*SPG*AMF/12.
C     EVALUATING DW/DX
      IF(AXILO.EQ.1.) GO TO 504
      DW7=CONS*XS(8)*XS(9)*XS(10)*XS(10)*SPG*AMF/144.
      DW8=CONS*XS(7)*XS(9)*XS(10)*XS(10)*SPG*AMF/144.
      DW9=CONS*XS(7)*XS(8)*XS(10)*XS(10)*SPG*AMF/144.
      DW10=2.0*CONS*XS(7)*XS(8)*XS(9)*XS(10)*SPG*AMF/144.
C     EVALUATING DM/DX
504 DM7=1152.*CONS*XS(8)*XS(9)*XS(10)*XS(10)*SPG*AMF/144.
      DM8=1152.*CONS*XS(7)*XS(9)*XS(10)*XS(10)*SPG*AMF/144.
      DM9=1152.*CONS*XS(7)*XS(8)*XS(10)*XS(10)*SPG*AMF/144.
      DM10=1152.*CONS*XS(7)*XS(8)*XS(9)*XS(10)*2.
     *SPG*AMF/144.
      GX(1)=B1*(XS(3)-H-(P/5.25)-D)-B*(H1+D1)+AAM1+C1
      GX(2)=A2*XS(4)
      GX(1)=GX(1)/POMM
      GX(2)=(GX(2)/POMM)+FACT1/XS(2)
      GX(3)=B
      GX(4)=A
      GX(5)=(-P5*B/5.25)-(FACT1*P5)
      GX(6)=(-B*P6/5.25)-(FACT1*P6)
      GX(7)=(-DM7)-(FACT2*DW7)
      GX(8)=(-DM8)-(P8*B/5.25)-(FACT1*P8)-(FACT2*DW8)
      GX(9)=(-DM9)-(P9*B/5.25)-(FACT1*P9)-(FACT2*DW9)
```

```
      GX(10)=(-DM10)-(P10*B/5.25)-(FACT1*P10)-(FACT2*DW10)
      CALL DGDB(GL,GX,SY,T,S,XS1,XM,BETA,10)
      KK=KK+1
      CALL VEC4(XS1,KK,10)
      DO 425 I=1,10
      DIFF=ABS((XS(I)-XS1(I))/XS(I))
      IF(KK.EQ.16)GO TO 443
      IF(DIFF.GT.ERRL)GO TO 440
425   CONTINUE
      GO TO 443
440   DO 426 I=1,10
426   XS(I)=XS1(I)
      GO TO 441
443   FFUN(IJK)=B*((RLF*XS(3))-H-P/5.25-D)+
     AAM-BBM+C-(FACT1*P)+(XS(4)*A)
      TEMP(IJK)=BETA
      WRITE(6,1055)FFUN(IJK)
433   CONTINUE
      CALL PFL(FFUN,TEMP,AA,BB,BETA,PF)
      WRITE(6,257)
      WRITE(6,264) BETA,PF
264   FORMAT(//7X,'RELIABILITY INDEX BETA FOR STATE 4
      =',F5.2,//,
     17X,'PROBABILITY OF FAILURE OF STATE 4 = ',E12.5,//)
      STOP
      END
C     ******************************************************
      SUBROUTINE TITLE(AK)
      WRITE(6,1000)AK
 1000 FORMAT(///,10X,'BETA=',F6.3,/)
      WRITE(6,1001)
 1001 FORMAT(' ITER',4X,'MOE1',5X,'MOE2     MOR1
MOR2       RD',4X,*'CP+   CP4  KZ   GUST  WIND',/)
      RETURN
      END
      SUBROUTINE TRAG(KM)
      WRITE(6,6000)KM
 6000  FORMAT('COMPUTATION  DIFFICULITY  IN  PROGRAM  LINE
NO.',I5)
      RETURN
      END
C    *******************************************************
      SUBROUTINE WEIBULL(WETA,WMU,WSIG,A,B,WXS,WXM,WXSIG)
      DIMENSION A(410),B(410)
      PI=3.14159
      W=(WXS-WMU)/WSIG
      IF(W.LT.ERRS) CALL TRAG(268)
      BR=1-EXP(-1.*(W**WETA))
      WKS=FI(BR,A,B)
      WXSIG=EXP(-0.5*WKS*WKS)/EXP(-1.*(W**WETA))
      WXSIG=WXSIG*(W**(1.-WETA))*WSIG/(WETA*SQRT(2.*PI))
      WXM=WXS-WKS*WXSIG
```

```
      RETURN
      END
C     ****************************************
      SUBROUTINE TYPEI(FA,FB,A,B,FXS,FXM,FXSIG)
      DIMENSION A(410),B(410)
      PI=3.14159
      BR1=FA*(FB-FXS)
      BR=EXP(-1.*EXP(BR1))
      WKS=FI(BR,A,B)
      FXSIG=EXP(-0.5*WKS*WKS)/EXP(BR1-EXP(BR1))
      FXSIG=FXSIG/(FA*SQRT(2.*PI))
      FXM=FXS-WKS*FXSIG
      RETURN
      END
      SUBROUTINE STYPEI(FA,FB,A,B,FXS,FXM,FXSIG)
      DIMENSION A(410),B(410)
      PI=3.14159
      BR1=FA*(FXS-FB)
      BR=1.0-EXP(-1.*EXP(BR1))
      WKS=FI(BR,A,B)
      FXSIG=EXP(-0.5*WKS*WKS)/EXP(BR1-EXP(BR1))
      FXSIG=FXSIG/(FA*SQRT(2.*PI))
      FXM=FXS-WKS*FXSIG
      RETURN
      END
C     ****************************************
      FUNCTION FI(AS,A,B)
      DIMENSION A(410),B(410)
      CO=0.0
      IF(AS-0.5)10,11,11
   10 AS=1.-AS
      CO=1.0
   11 DO 1 I=1,403
      IF(B(I)-AS)1,3,2
    1 CONTINUE
    3 FI=A(I)
      GO TO 14
    2 SA1=A(I-1)
      SA2=A(I)
      SB1=B(I-1)
      SB2=B(I)
      FI=(AS-SB1)*(SA2-SA1)/(SB2-SB1)+SA1
   14 IF(CO.EQ.1.)GO TO 15
      RETURN
   15 FI=-1.*FI
      RETURN
      END
C     ************************************
      SUBROUTINE BF(I)
      WRITE(44,77)I
   77 FORMAT(I20)
      RETURN
```

```
      END
C     ****************************
      SUBROUTINE TRMATRIX (TA,TTA,N)
      DIMENSION TA(10,10),TTA(10,10)
      DO 1 I=1,N
      DO 2 J=1,N
      TA(I,J)=0.0
      IF(I-J)2,100,2
  100 TA(I,J)=1.0
    2 CONTINUE
    1 CONTINUE
      DO 3 I=1,N
      DO 4 J=1,N
4     TTA(I,J)=TA(J,I)
3     CONTINUE
      RETURN
      END
C     *************************
      SUBROUTINEYSPACE(GL1,GL2,YS,GL,XS,XM,XSIG,T,TT,S,N)
      DIMENSION
      GL1(10),GL2(10),YS(10),GL(10,10),S(10,10),
     SN(10,10),
     1 TT(10,10),T(10,10),XSIG(10),XM(10),XS(10)
      DO 5 I=1,N
      DO 6 J=1,N
      S(I,J)=0.0
      IF(I-J)6,101,6
  101 S(I,J)=XSIG(I)
      KM=98
      IF(XSIG(I).EQ.0.)CALL TRAG(KM)
      SN(I,J)=1./XSIG(I)
    6 CONTINUE
    5 CONTINUE
      DO 7 I=1,N
    7 GL1(I)=XS(I)-XM(I)
      DO 8 I=1,N
      SUM=0.
      DO 9 J=1,N
    9 SUM=SUM+SN(I,J)*GL1(J)
    8 GL2(I)=SUM
      DO 10 I=1,N
      SUM=0.
      DO 11 J=1,N
   11 SUM=SUM+TT(I,J)*GL2(J)
   10 YS(I)=SUM
C, FORMATION OF TRANSPOSE OF (S*T)
      DO 12 I=1,N
      DO 13 J=1,N
      SUM=0.0
      DO 14 K=1,N
   14 SUM=SUM+S(I,K)*T(K,J)
   13 GL(J,I)=SUM
```

```
   12 CONTINUE
      RETURN
      END
      SUBROUTINE VEC(U,K,N)
      DIMENSION U(10)
      WRITE(6,1000)K,(U(I),I=1,N)
1000  FORMAT(I5,F10.1,8X,F7.2,F6.2,2F5.3,F6.3,F7.2)
      RETURN
      END
      SUBROUTINE RMOI(CAS,CAC,CAT,SKC1,SKC2,SKT1,
     SKT2,SNC,SNT,X,Y,RI,
     1AT,AC)
      PI=3.14159
      AX=X/1000.
      AY=Y/1000.
      DN=(CAS+(SNC*CAC)+(SNT*CAT))
      AS=1.94*((SNT*CAT)-(SNC*CAC))/DN
      AC=((1.94*CAS)+(1.94*2.0*SNT*CAT))/DN
      AT=((1.94*CAS)+(1.94*2.0*SNC*CAC))/DN
      CKC=(PI*PI*6.0*AX*SNC*CAC*(CAS+(SNT*
     CAT)))/(96.*96.*SKC1*DN)
      CKT=(PI*PI*6.0*AX*SNT*CAT*(CAS+(SNC*
     CAC)))/(96.*96.*SKT1*DN)
      CKCT=((PI**4.)*36.*AX*AX*CAS*SNC*CAC*SNT*CAT)
     1/((96.**4.)*SKC1*SKT1*DN)
      CNC=(PI*PI*6.0*AX*SNC*CAC*SNT*CAT)/
     (96.*96.*SKC1*DN)
      CNT=(PI*PI*6.0*AX*SNC*CAC*SNT*CAT)/(96.*96.*SKT1*DN)
      Q1=((CAS*AS*AS)+(SNC*CAC*AC*AC)+
     (SNT*CAT*AT*AT))*AX/AY
      Q2=((CNC*(SKC1/SKC2)*1.94*1.94)+
     (CNT*(SKT1/SKT2)*1.94*1.94))*
     1CAS*AX/AY
      Q3=((AX/AY)+CKC*(SKC1/SKC2)+CKT*(SKT1/SKT2)
     +CKCT*(SKT1/
     1SKT2)*(SKC1/SKC2)*(AX/AY))
      RI=5.44+(Q1/Q3)+(Q2/Q3)
      RETURN
      END
      SUBROUTINE FACT(W,P,X,RI,FACT1,FACT2)
      AXY=X
      FACT1=(W*(96.**4.)*5.)/(AXY*RI*384.)
      FACT2=(P*(96.**4.)*5.)/(AXY*RI*384.)
      RETURN
      END
      SUBROUTINE VEC3(U,K,N)
      DIMENSION U(10)
      WRITE(6,1000)K,(U(I),I=1,N)
1000  FORMAT(I5,F10.1,9X,F8.2,8X,F7.2,F6.2,2F5.3,F6.3,F7.2)
      RETURN
      END
      SUBROUTINE VEC4(U,K,N)
```

```
      DIMENSION U(10)
      WRITE(6,1000)K,(U(I),I=1,N)
 1000 FORMAT(I5,F10.1,F9.1,2F8.2,F7.2,F6.2,2F5.3,F6.3,F7.2)
      RETURN
      END
      SUBROUTINE DGDB(GL,GX,SY,T,S,XS1,XM,BETA,N)
      DIMENSION GL(10,10),GX(10),GY(10),SY(10),
     ALF(10),T(10,10),
     1 GL1(10),S(10,10),XS1(10),XM(10),YS(10)
      DO 15 I=1,N
      SUM=0.0
      DO 16 J=1,N
   16 SUM=SUM+GL(I,J)*GX(J)
   15 GY(I)=SUM
      DEL=0.0
      DO 17 I=1,N
   17 DEL=DEL+(SY(I)*GY(I))**2
      DEL=SQRT(DEL)
      DO 18 I=1,N
   18 ALF(I)=SY(I)*GY(I)/DEL
      BEE=0.0
      DO 19 I=1,N
   19 YS(I)=-1.*BETA*SY(I)*ALF(I)
      DO 20 I=1,N
      SUM=0.0
      DO 21 J=1,N
   21 SUM=SUM+T(I,J)*YS(J)
   20 GL1(I)=SUM
      DO 22 I=1,N
      SUM=0.0
      DO 23 J=1,N
   23 SUM=SUM+S(I,J)*GL1(J)
   22 XS1(I)=SUM+XM(I)
      RETURN
      END
      SUBROUTINE PFL(FFUN,TEMP,A,B,BETA,PF)
      DIMENSION FFUN(11),TEMP(11),A(410),B(410)
      DO 12 J=1,10
      IF(FFUN(J).GT.0.0.AND.FFUN(J+1).LT.0.0) GO TO 17
      IF(FFUN(J).LT.0.0.AND.FFUN(J+1).GT.0.0) GO TO 17
12    CONTINUE
17    BETA=TEMP(J)
      DO 1 I =1,403
      IF(A(I)-BETA) 1,3,2
1     CONTINUE
3     TEM=B(I)
      GO TO 14
2     TEM1=(B(I)-B(I-1))
      TEM2=(A(I)-A(I-1))
      TEM=(TEM1/TEM2)*(BETA-A(I-1))
      TEM=TEM+B(I-1)
14    PF=(1.0-TEM)
```

BETA SHOWING INITIAL VALUES, AND THE CORRELATION

COEFFICIENTS REQUIRED AS AN INPUT FOR WALL ANALYSIS

ALSO NGAUSS CONTAINS CDF AND PDF OF STANDARDIZED NORMAL VALUES

STAGE 1 RELIABILITY INDEX
0.01 1.00
STAGE 2
2.200 2.300
STAGE 3
3.500 3.600
STAGE 4
3.000 3.200
CORRELATION COEFFICIENT BETWEEN E_1 AND σ_1. (STATE 3)
1.0
0.6383
1.0
CORRELATION COEFFICENT BETWEEN E_1, E_2, σ_1 and MOR (STAGE 4)
1.0
0.2634
1.0
0.6383
0.0
1.0
0.4218
0.5750
0.1618
1.0

****
SHEATHING PROPERTIES
****
4.08,1.44,15.43,0.54

****
THREE PARAMETERS FOR E_1
****

2.36,622813.0,811300.0
1.30,0.0,390604.0
2.08,1325.0,3046.55
1.18,0.0,896.78

****
THREE PARAMETERS FOR E_2
****

2.38,602187.0,811300.0
1.40,0.0,407039.5
2.29,1256.2,3088.3
1.28,0.0,950.53

****
THREE PARAMETERS FOR σ_1
****

2.88,275920.0,1296500.0
1.34,0.0,566139.32
2.13,998.25,4189.0
1.16,0.0,1363.4

****
THREE PARAMETERS FOR MOR
****

2.79,399466.7,1133200.0
1.39,0.0,580945.0
2.57,682.97,4784.2
1.14,0.0,1225.0
****

APPENDIX I

Computer Program for the Computation of

β in Floor Systems

```
************************************************************
C     PROGRAM FLR.FOR
************************************************************
      $LARGE
       DIMENSION
       AA(410),BB(410),GX(10),GY(10),ALF(10),SY(10),
       XS1(10),XS(10),XSIG(10),XM(10),GL1(10),GL2(10),
       YS(10),GL(10,10),S(10,10),SN(10,10),T(10,10),
       TT(10,10),FFUN(11),TEMP(11)
       INTEGER NA,JOBNA,IZA,IER,NB,JOBNB,IZB
       REAL RA(3),RB(10),DA(2),DD(4),Z(2,2),ZZ(4,4),WK(3)
C      READ DATA WHICH ARE IN A FILE CALLED GAUSS.
      OPEN(UNIT=1,FILE='D:\JUNK\NGAUSS',STATUS='OLD')
      OPEN(UNIT=2,FILE='D:\JUNK\DATA',STATUS='OLD')
      OPEN(UNIT=6,FILE='D:\JUNK\OUT',STATUS='NEW')
      READ(1,1000)(AA(I),BB(I),I=1,403)
1000  FORMAT(F4.2,F8.6)
      READ(2,*)ETA1,AMU1,SIG1
      READ(2,*)ETA2,AMU2,SIG2
      WRITE(*,*) 'PLEASE INPUT JOIST SPACING'
      READ(*,*) SPA
      WRITE(*,*) 'IF CA REQUIRED INPUT 1, ELSE 0'
      READ(*,*) CA
      WRITE(*,*) 'INPUT LOAD SHARING FACTOR'
      READ(*,*) RLS
      WRITE(*,*) 'INPUT BETA INITIAL AND FINAL'
      READ(*,*) BETIN1,BETFI1
       BETDE=(BETFI1-BETIN1)/10.
       PI=3.14159
       ERRL=0.005
       ERRS=1/(10.**9.)
      ER=1.0/(10.**4)
      ERR=1.0/(10.**10)
      ERRE=1.0/(10.**11)
      FACT=93.70
C      DATA ASSOCIATED WITH WIND VELOCITY
       AMF=0.63
       CONS=0.00256
       POMM=10.**6.
C     TWO PARAMETERS OF TYPE I EXTREME VALUE FUNCTION
C     LIVE LOAD
      VA=0.1283
```

```
      VB=35.50
C     DEAD LOAD, RANDOM VARIABLE X3
      XM(3)=7.0
      XSIG(3)=0.7
C      INPUT OF TRANSFORMATION MATRIX T
      CALL TRMATRIX(T,TT,4)
C        TO GET THE EIGENVECTORS AND THE EIGENVALUE, IMSL
ROUTINE C  IS MADE USE OF.
C     *********************************************
C     INPUT THE CORRELATION BETWEEN MOE AND MOR
C     *********************************************
      RA(1)=1.0
      RA(2)=0.60
      RA(3)=1.0
       NA=2
       IZA=2
       JOBNA=2
       CALL EIGRS(RA,NA,JOBNA,DA,Z,IZA,WK,IER)
       DO 391 I =1,2
       DO 392 J =1,2
       T(I,J)=Z(I,J)
392    CONTINUE
391    CONTINUE
      CALL TRANSP(T,TT,SY,DA,2,4)
       KK=0
       BETA=BETIN1-BETDE
       DO 33 IJK=1,11
       BETA=BETA+BETDE
      WRITE(*,*) BETA
       CALL TITLE(BETA)
42     XS(1)=1500000.
      XS(2)=1500.0
      XS(3)=7.0
      XS(4)=40.0
41     IF(XS(1).LT.AMU1)XS(1)=AMU1*1.1
       CALL WEIBULL(ETA1,AMU1,SIG1,AA,BB,XS(1),
      XM(1),XSIG(1))
      IF(XS(2).LT.AMU2)XS(2)=AMU2*1.1
      CALL WEIBULL(ETA2,AMU2,SIG2,AA,BB,XS(2),
      XM(2),XSIG(2))
      CALL TYPEI(VA,VB,AA,BB,XS(4),XM(4),XSIG(4))
      CALL YSPACE(GL1,GL2,YS,GL,XS,XM,XSIG,T,TT,S,4)
C     SOME VALUES WILL BE MULTIPLIED BY POMM=10**6 BECAUSE
C     THEY ARE TOO SMALL. LATER ON, THE SAME VALUES
C     WILL BE AGAIN DIVIDED BY POMM. THIS IS DONE BECAUSE OF
C     COMPUTER INACCURACY
C     EVALUATING DA/DX1
      AAMO=305.75+(7.45*ER*XS(1))+(4.45*ERR*XS(1)*XS(1))
      AAMU=14641.0+0.0166*XS(1)+(4.69*ERRE*XS(1)*XS(1))
       AAM=AAMO/AAMU
      AAM1=((7.45*ER)+(2.*4.45*ERR*XS(1)))*AAMU
      TEMPO=(0.0166+(2.0*4.69*ERRE*XS(1)))*AAMO
```

```
      AAM1=(AAM1-TEMPO)*POMM/(AAMU*AAMU)
C     ******************************************
C     INPUT THE NO COMPOSITE ACTION PART HERE
      IF(CA.EQ.0) THEN
      AAMO=306.61+(0.272*XS(1))+(2.26*ER*XS(1)*XS(1))
      AAMU=14835.0+(11.69*XS(1))+(2.30*XS(1)*XS(1)*ER*10.0)
      AAM=AAMO/AAMU
      AAM1=(0.272+(2.0*2.26*ER*XS(1)))*AAMU
      TEMPO=(11.69+(2.0*2.30*XS(1)*ER*10.0))*AAMO
      AAM1=(AAM1-TEMPO)*POMM/(AAMU*AAMU)
      ENDIF
C     EVALUATING DW/DX3 AND DW/DX4
      DW3=18.0*SPA*(AAM+0.023)
      DW4=DW3
C      NOW, EARLIER MULTIPILIED VALUES BY POMM=10**6
      WILL BE C DIVIDED
      GX(1)=-(AAM1*(XS(3)+XS(4))*18.0*SPA)
      GX(1)=GX(1)/POMM
      GX(2)=1.0
      GX(3)=-DW3
      GX(4)=-DW4
      CALL DGDB(GL,GX,SY,T,S,XS1,XM,BETA,4)
       KK=KK+1
      CALL VEC(XS1,KK,4)
      DO 25 I =1,4
       DIFF=ABS((XS(I)-XS1(I))/XS(I))
       IF(KK.EQ.16)GO TO 43
       IF(DIFF.GT.ERRL)GO TO 40
25     CONTINUE
       GO TO 43
40    DO 26 I =1,4
26     XS(I)=XS1(I)
       GO TO 41
43    FFUN(IJK)=RLS*XS(2)-((XS(3)+XS(4))*SPA*
      18.0*(AAM+0.023))
       TEMP(IJK)=BETA
       WRITE(6,1055)FFUN(IJK)
33     CONTINUE
1055   FORMAT(18X,'FAILURE FUNCTION G=',F15.7)
       CALL PFL(FFUN,TEMP,AA,BB,BETA,PF)
       WRITE(6,257)
       WRITE(6,263) BETA,PF
263   FORMAT(/5X,'BETA='F7.2,/,7X,'PF='E10.4,/)
       WRITE(6,257)
257    FORMAT(20X,//35('*')//)
       STOP
       END
C      *********************************************
       SUBROUTINE TITLE(AK)
       WRITE(6,1000)AK
 1000  FORMAT(///,10X,'BETA=',F6.3,/)
       WRITE(6,1001)
```

```
1001 FORMAT(' ITER',4X,'MOE',5X,'MOR DEAD
     LOAD LIVE LOAD',/)
      RETURN
      END
      SUBROUTINE TRAG(KM)
      WRITE(6,6000)KM
 6000 FORMAT('COMPUTATION DIFFICULITY IN
     PROGRAM LINE NO.',I5)
      RETURN
      END
C     ****************************************
      SUBROUTINE WEIBULL(WETA,WMU,WSIG,A,B,WXS,WXM,WXSIG)
      DIMENSION A(410),B(410)
      PI=3.14159
      W=(WXS-WMU)/WSIG
      IF(W.LT.ERRS) CALL TRAG(268)
      BR=1-EXP(-1.*(W**WETA))
      WKS=FI(BR,A,B)
      WXSIG=EXP(-0.5*WKS*WKS)/EXP(-1.*(W**WETA))
      WXSIG=WXSIG*(W**(1.-WETA))*WSIG/(WETA*SQRT(2.*PI))
      WXM=WXS-WKS*WXSIG
      RETURN
      END
C     ****************************************
      SUBROUTINE TYPEI(FA,FB,A,B,FXS,FXM,FXSIG)
      DIMENSION A(410),B(410)
      PI=3.14159
      BR1=FA*(FB-FXS)
      BR=EXP(-1.*EXP(BR1))
      WKS=FI(BR,A,B)
      FXSIG=EXP(-0.5*WKS*WKS)/EXP(BR1-EXP(BR1))
      FXSIG=FXSIG/(FA*SQRT(2.*PI))
      FXM=FXS-WKS*FXSIG
      RETURN
      END
C     ****************************************
      FUNCTION FI(AS,A,B)
      DIMENSION A(410),B(410)
      CO=0.0
      IF(AS-0.5)10,11,11
   10 AS=1.-AS
      CO=1.0
   11 DO 1 I=1,403
      IF(B(I)-AS)1,3,2
    1 CONTINUE
    3 FI=A(I)
      GO TO 14
    2 SA1=A(I-1)
      SA2=A(I)
      SB1=B(I-1)
      SB2=B(I)
      FI=(AS-SB1)*(SA2-SA1)/(SB2-SB1)+SA1
```

```
   14 IF(CO.EQ.1.)GO TO 15
      RETURN
   15 FI=-1.*FI
      RETURN
      END
C     ************************************
      SUBROUTINE BF(I)
      WRITE(44,77)I
   77 FORMAT(I20)
      RETURN
      END
C     ****************************
C     *************************
      SUBROUTINE YSPACE(GL1,GL2,YS,GL,XS,XM,XSIG,
     T,TT,S,N)
      DIMENSION
     GL1(10),GL2(10),YS(10),GL(10,10),S(10,10),SN(10,10),
     1 TT(10,10),T(10,10),XSIG(10),XM(10),XS(10)
      DO 5 I=1,N
      DO 6 J=1,N
      S(I,J)=0.0
      IF(I-J)6,101,6
  101 S(I,J)=XSIG(I)
      KM=98
      IF(XSIG(I).EQ.0.)CALL TRAG(KM)
      SN(I,J)=1./XSIG(I)
    6 CONTINUE
    5 CONTINUE
      DO 7 I=1,N
    7 GL1(I)=XS(I)-XM(I)
      DO 8 I=1,N
      SUM=0.
      DO 9 J=1,N
    9 SUM=SUM+SN(I,J)*GL1(J)
    8 GL2(I)=SUM
      DO 10 I=1,N
      SUM=0.
      DO 11 J=1,N
   11 SUM=SUM+TT(I,J)*GL2(J)
   10 YS(I)=SUM
C, FORMATION OF TRANSPOSE OF (S*T)
      DO 12 I=1,N
      DO 13 J=1,N
      SUM=0.0
      DO 14 K=1,N
   14 SUM=SUM+S(I,K)*T(K,J)
   13 GL(J,I)=SUM
   12 CONTINUE
      RETURN
      END
      SUBROUTINE DGDB(GL,GX,SY,T,S,XS1,XM,BETA,N)
      DIMENSION
```

```
     GL(10,10),GX(10),GY(10),SY(10),ALF(10),T(10,10),
     1 GL1(10),S(10,10),XS1(10),XM(10),YS(10)
      DO 15 I=1,N
      SUM=0.0
      DO 16 J=1,N
   16 SUM=SUM+GL(I,J)*GX(J)
   15 GY(I)=SUM
      DEL=0.0
      DO 17 I=1,N
   17 DEL=DEL+(SY(I)*GY(I))**2
      DEL=SQRT(DEL)
      DO 18 I=1,N
   18 ALF(I)=SY(I)*GY(I)/DEL
      BEE=0.0
      DO 19 I=1,N
   19 YS(I)=-1.*BETA*SY(I)*ALF(I)
      DO 20 I=1,N
      SUM=0.0
      DO 21 J=1,N
   21 SUM=SUM+T(I,J)*YS(J)
   20 GL1(I)=SUM
      DO 22 I=1,N
      SUM=0.0
      DO 23 J=1,N
   23 SUM=SUM+S(I,J)*GL1(J)
   22 XS1(I)=SUM+XM(I)
      RETURN
      END
      SUBROUTINE VEC(U,K,N)
      DIMENSION U(10)
      WRITE(6,1000)K,(U(I),I=1,N)
1000 FORMAT(I5,F10.2,2X,3F7.2)
     RETURN
      END
      SUBROUTINE PFL(FFUN,TEMP,A,B,BETA,PF)
      DIMENSION FFUN(11),TEMP(11),A(410),B(410)
      DO 12 J=1,10
      IF(FFUN(J).GT.0.0.AND.FFUN(J+1).LT.0.0) GO TO 17
      IF(FFUN(J).LT.0.0.AND.FFUN(J+1).GT.0.0) GO TO 17
12    CONTINUE
17    BETA=TEMP(J)
      DO 1 I =1,403
      IF(A(I)-BETA) 1,3,2
1     CONTINUE
3     TEM=B(I)
      GO TO 14
2     TEM1=(B(I)-B(I-1))
      TEM2=(A(I)-A(I-1))
      TEM=(TEM1/TEM2)*(BETA-A(I-1))
      TEM=TEM+B(I-1)
14    PF=(1.0-TEM)
      RETURN
```

```
      END
       SUBROUTINE TRMATRIX (TA,TTA,N)
       DIMENSION TA(10,10),TTA(10,10)
       DO 1 I=1,N
       DO 2 J=1,N
       TA(I,J)=0.0
       IF(I-J)2,100,2
   100 TA(I,J)=1.0
     2 CONTINUE
     1 CONTINUE
       DO 3 I=1,N
       DO 4 J=1,N
4      TTA(I,J)=TA(J,I)
3      CONTINUE
       RETURN
       END
C      *************************
       SUBROUTINE TRANSP(T,TT,SY,D,NN,N)
       REAL T(10,10),TT(10,10),SY(10),D(4)
       DO 3 I =1,N
       DO 4 J =1,N
4      TT(I,J)=T(J,I)
3      CONTINUE
       DO 5 J =1,NN
5      SY(J)=SQRT(D(J))
       DO 6 J =NN+1,N
6      SY(J)=SQRT(1.0)
       RETURN
       END
```