Performance-Based Engineering of Wood Frame Housing: Fragility Analysis Methodology

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Abstract: Recent trends in building construction have highlighted the need for improved methodologies for engineering new light-frame structures for housing and techniques for condition assessment of existing structures. The aftermath of natural disasters during the past decade, the rapid evolution of design and construction methods, and heightened expectations on the part of the public and its scrutiny of perceived and actual deficiencies in codes and code enforcement have further underscored these needs. Among the high-priority areas identified at a 1997 ASCE workshop on wood engineering research needs were behavior and performance of wood structural systems; criteria for performance assessment; and methods for condition assessment of damaged systems following natural disasters. The development of appropriate and usable fragility models and system reliability analysis tools is necessary to meet these needs and to make meaningful advances in performance-based engineering of wood frame structures. This paper provides an overview of efforts to develop such models and tools, and suggests a possible methodology for assessing probable response of light-frame residential construction exposed to various levels of natural and man-made hazards. The implementation of performance-based engineering for residential construction will enhance durability and reduce maintenance costs of the nation's housing inventory, and will facilitate reductions in risk of death, injury, and property damage from extreme natural hazards such as earthquakes and hurricanes.

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Background

Developments in probability-based limit states design (or LRFD, as it is called in the United States) have occurred rapidly (Ellingwood 1994), beginning with the general load requirements in ANSI Standard A58 (now ASCE Standard 7-98) (Galambos et al. 1982; Ellingwood et al. 1982), followed by LRFD for steel buildings and bridges and, most recently, AF&PA/ASCE Standard 16-95 for engineered wood construction (Standard 1996). Most code and standard groups in the United States, Canada, Japan, and Europe have come to recognize the advantages of limit states design.

In LRFD of engineered wood structures (Standard 1996), the structural safety performance requirement is expressed by the set of equations,

$$\lambda\phi R' > \sum \gamma_i Q_i \qquad (1)$$

in which R' = adjusted resistance of a member, component, or connection; ϕ = resistance factor, taking into account uncertainty in short-term strength as well as the mode of failure; and λ is a time-effect factor. The resistance, R', includes factors that adjust the short-term strength for specified end use conditions (ASCE Standard 16-95, Chapter 2, Table 2.6-1). Factor λ accounts for the effect of temporal characteristics of the load on strength. On the right-hand side of Eq. (1), Q_i = structural action (axial force, moment, or shear) due to load i, and γ_i = load factor that accounts for uncertainty in load i. The partial factors in Eq. (1) are set to be consistent with the uncertainty in a variable and a specified target reliability level using structural reliability concepts (Galambos et al. 1982; Ellingwood et al. 1982; Ellingwood and Rosowsky 1991). Currently, the right-hand side of Eq. (1) is defined for all construction materials in Chapter 2 of ASCE Standard 7–98.

With its probabilistic basis and supporting statistical databases, Eq. (1) represents a vast improvement over traditional allowable stress design (National 1991). However, it is strictly valid only for safety checks of individual members, components, and connections that are performed as part of the design process for new buildings. Such checks provide only an approximate picture of how a system of such members and their connecting elements might perform in service or during an extreme natural hazard event such as a hurricane or earthquake.

The difference between member and system performance is built, only indirectly, into the reliability-based calibration of Eq. (1) to existing design practice (Ellingwood and Rosowsky 1991). Current safety checks for certain members are modified to account for system effects through the use of simple adjustment factors. Examples of this include the effective length factors (K factors) commonly used in column design to account indirectly for second-order effects, increases in nominal strength for repetitively used light-frame members, and the response modification factor, R, and deflection amplification factor, C_d, used in modern earthquake-resistant design. The performance of light-frame wood assemblies is affected by load sharing, partially composite action of members and sheathing, and connection behavior. These

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mechanisms affect system performance in two ways: (1) through the system interactions that occur when a number of components, each with inherently variable engineering properties, is used; and (2) through the modeling assumptions that are made in analyzing the behavior of the system or designing it. The importance of these system effects in wood floors, roofs, and other load-distributing elements is recognized in both the National Design Specification and ASCE Standard 16-95 by specifying repetitive-member load-sharing (or "system") factors. These factors currently are based on judgment and relatively simple modeling assumptions. Yet, system performance is becoming more important in a number of contexts. Unless a system fails immediately following failure of its weakest member (a rare occurrence in most practical structural systems), that system as a whole generally is substantially stronger than would be indicated by an assessment based on the strengths of the individual members in the system. Surveys of existing light-frame residential construction conducted by NAHB and other organizations following recent natural disasters (e.g., NAHB 1994; 1996; 1999a; b, c, d) highlight this discrepancy between estimated and observed strength.

Thus, the "apparent" or "notional" reliabilities associated with Eq. (1) are not indicative of the reliabilities (or, conversely, failure rates) that would be observed in a damage survey of light-frame residential construction. Indeed, calculations performed on an individual member basis may lead to a pessimistic appraisal of system reliability and performance. This lack of agreement has some important implications for the structural engineering profession as it strives to provide building structural systems that perform in accordance with the expectations of the facility owner and occupants for a range of natural and man-made hazards at a reasonable cost. Moreover, the use of such notional computed failure rates to estimate insurance premium underwriting costs or to determine postdisaster public policy is questionable. To achieve reasonable agreement (within the inherent limitations imposed by statistical sampling) between calculated and observed structural failure rates so as to be able to use such estimates as a basis for building regulation, properly validated system reliability analysis models are essential.

Current probability-based LRFD criteria are prescriptive in nature. While they are easily used and interpreted, they create a false sense of security and the illusion that meeting code minimums results in a satisfactory building. There is ample evidence to suggest that this is not the case (Ellingwood 1998), and recent natural disasters in the United States and Japan have highlighted the social, political, and economic ramifications of this traditional view of codes. Performance-based engineering is a new paradigm, gaining momentum in many countries, including the United States, in which the design process is structured to meet performance expectations of the building occupants, owner, and the public. While public safety considerations always are paramount, the building performance expected by the owner or occupants often exceeds what is implied by the code minimums. In particular, economic disruptions caused by building failures in recent natural disasters have been unacceptable, and have severely taxed the abilities of local governments to deal with them.

The aftermath of natural disasters during the past decade, the rapid evolution of design and construction methods, heightened expectations on the part of the public, and its scrutiny of perceived and actual deficiencies in codes and code enforcement, all have made it clear that an improved basis for designing new houses and assessing the condition of the existing residential building stock is required (Committee 1991). There also is a need for tools to evaluate new and existing building products designed and marketed in the international arena. Finally, as indicated previously, there is an urgent need for tools for interpreting building performance data following natural disasters (NAHB 1999b,d). Only with these tools can performance problems be fully identified and solutions be proposed. Among the high-priority areas identified at a recent ASCE workshop on wood engineering research needs (Fridley 1998) were behavior and performance of wood structural systems; criteria for performance assessment; and methods for condition assessment of damaged systems following natural disasters. The development of appropriate and usable system risk and reliability analysis tools is essential to meet these needs. Experiences in the SAC Joint Venture (SAC 1995), which was directed toward steel moment frames subjected to earthquakes, have suggested some of the ways that quantitative risk assessment methods might be utilized for this purpose in light-frame residential construction.

The purpose of this paper is to provide an overview of efforts to develop performance-based engineering concepts for light-frame wood structural systems common in residential construction, and to suggest approaches that have been found to be useful in other contexts for evaluating the probable response of such construction to natural hazards. Improved methods for evaluating the reliability of light-frame structural systems will pay dividends by facilitating the introduction of new technologies, achieving cost savings in light-frame construction without sacrificing performance, and supporting the new performance-based engineering paradigm for residential construction.

Performance-Based Design for Residential Construction

Performance-based engineering is not a new concept. In the United States, it dates back to the late 1960s, when the U.S. Department of Housing and Urban Development sponsored a large research program ("Operation Breakthrough") at the National Bureau of Standards to develop model criteria for the design and evaluation of innovative housing systems (Performance 1977). These model criteria were intended to be equivalent, in a general sense, to the contemporary HUD Minimum Property Standards, and accordingly were similar in scope to the provisions in a Model Building Code. Specific criteria addressed desired building attributes: safety, serviceability, the integrity of the building envelope, mechanical, electrical, and illumination systems, and fire safety; and for building components and systems foundations, structural systems, roofing and cladding, fenestration, and so forth. The basic format of the performance document) consisted of a nontechnical *requirement*, expressing qualitatively a fundamental goal (e.g., building structures shall remain stable under extreme loads), a set of *criteria* to ensure that the requirement is satisfied (e.g., design flexural strength shall exceed the maximum moment due to design loads), methods of *evaluation* to measure satisfaction of each criterion (based on analysis or test methods), and a *commentary* to explain the rationale of each provision. The proposed criteria were organized around attribute-system pairings, with a set of requirements, criteria, evaluation, and commentary provided for each pair. The differences between performance-based engineering and traditional engineering are more significant than they may appear upon first encounter (Ellingwood 1998). Perhaps most important, the level of performance to be provided by the code is not articulated beyond the general "life safety" objective. Other desirable attributes (e.g., serviceability, durability) receive haphazard treatment or may not even be addressed. Normally only one criterion/evaluation pair (approved method for checking whether objectives are achieved) is permitted. Commentaries are sparse or non-existent, making it difficult to apply the code to unusual design situations.

The performance concept was envisioned as a tool for clarifying the intent of code provisions, facilitating new technologies in building construction, and fostering innovation in building construction. The recommendations of a workshop on disaster mitigation in the early 1970s included action items related to the need for performance criteria for building design (Wright et al. 1973), indicating that the profession seemed to recognize the value of these ideas in the abstract. However, the performance concept never came to fruition in the 1970s for a number of reasons, some of which had to do with available technology for structural design, liability concerns, and attitudes in the building code community (Ellingwood 1998). More recently, however, there has been a resurgence of interest internationally, and a number of countries now are moving toward performance-based codes, particularly in the area of fire-resistant design of building structural systems (Buchanan and Barnett 1995) and in earthquake-resistant design with the SEAOC Vision 2000 activity (Poland 1995). The International Code Council (ICC 1998) includes a performance-based Code Technical Subcommittee, the role of which is to foster performance-based design in the International Building Code (IBC). Concurrently, the committee with responsibility for the National Building Code of Canada is working toward an "Objective-based Code," originally scheduled for completion in 2001, but now delayed. The motivation for all these activities is clear: to ensure that hazards are treated consistently, to couple the design provisions more closely to performance expectations with regard to safety and function than is possible in the traditional prescriptive code setting, and to move beyond the focus on occupant safety to encompass other losses arising from failure to perform according to expectations.

Accordingly, the time is ripe to develop tools for performance-based engineering and condition assessment of existing light-frame residential wood construction. Such tools currently are unavailable or are in a rudimentary stage of development. Some of the major issues that must be addressed in the development of a performance-based design methodology for residential construction are discussed in the following sections.

Performance Requirements and Limit States

Verification that a building performance requirement (e.g., immediate occupancy under moderate wind events; life safety under design-basis earthquakes) has been met and, requires a mapping between the qualitative stated performance goal and a response quantity (force, deformation) that can be computed using principles of structural analysis and mechanics. When design is based on member behavior, this mapping usually is relatively straightforward, e.g., the "life safety" requirement in AF&PA/ASCE Standard 16-95 is checked by Eq. (1). As noted previously, however, the performance of individual members within the system may not be indicative of system performance.

Most recent proposals for performance-based engineering are based on three or four generally stated goals for buildings of different occupancies:

1. Serviceability under ordinary occupancy conditions;
2. Immediate occupancy following moderate events;
3. Life safety under design-basis events; and
4. Collapse prevention under maximum considered events.

As noted previously, traditional codes and standards, including the first generation of probability-based LRFD standards, are concerned mainly with Eq. (3). As part of the move toward performance-based engineering and probability-based performance assessment, the profession will need to consider Eqs. (1), (2), and (4) as well and develop engineering tools to check compliance with these goals. This means that such goals as "serviceability," "immediate occupancy," and "collapse prevention" will need to be expressed in terms of structural responses that the structural engineer can evaluate with available analytical tools or supporting test methods.

Thus, a major task will involve identification of limit states (LS) or conditions in which the structural system as a whole ceases to perform its intended functions in some way. Such performance-based limit states must include serviceability limit states involving loss of function under conditions of ordinary usage and ultimate limit states related to the prevention of loss of life and structural system collapse. System limit states may be either strength or deformation related, and their identification requires a thorough understanding of the structural mechanics of system response and the role of components in ensuring acceptable system behavior. In residential construction, deformation limit states may relate to failure of windows/doors, separation of cladding from framing, or failure of roof-to-wall connections. Strength limit states may relate to structural failure of the roof/wall system, sheathing removal, structural collapse, or foundation failures. Since most light-frame structural systems are highly redundant, limit states based on reaching member strength may provide a misleading picture of the integrity of the system as a whole. One can envision, for example, strength limit states based on the failure of a certain percentage of members within a system or failure of two or three adjacent members (Rosowsky and Ellingwood 1991). Alternatively, structural system deformations (e.g. interstory drift) may provide a useful surrogate measure of system strength. One might, for example, envision a mapping between performance objective and structural limit state, measured in terms of deformation, δ, with respect to some characteristic dimension (e.g., clear span, story height) as follows (Ellingwood 1998):

1. Serviceability: $\delta < 0.005$;
2. Immediate occupancy: $\delta < 0.01$;
3. Life safety: $\delta < 0.05$; and
4. Collapse prevention: $\delta > 0.05$.

It should be noted that these limits were chosen for illustration purposes only, and are incomplete; for example, they do not address goals of limiting unacceptable vibrations of floors or the building as a whole. Suitable limits of structural system performance for performance-based engineering and probabilistic condition assessment can be determined from previous studies of performance of residential structural systems (both analytical and experimental) and interpretation of post-disaster building surveys [e.g., NAHB 1994; 1996; 1999a, 1999b, 1999c, 1999d,]. However, at the time many of these previous studies were performed, the thinking on performance-based engineering had not fully matured. A reexamination of this previous research may shed additional light on the relation between performance requirements and structural limit states.

The role of serviceability limit states vs safety-related limit states in ensuring various performance objectives for light-frame construction will require further consideration beyond what is provided in current codes of practice, where serviceability issues receive scant attention. The economic losses from serviceability failures exceed losses due to inadequate strength by a substantial margin on an annualized basis. A significant percentage (as high as 50%) of systems designed from engineering calculations are found to be governed by deflection rather than strength criteria. There is no reason to believe that the system effect in serviceability should be the same as for strength; for example, creep under sustained load may play an important role in serviceability of wood floors (Philpot et al. 1995) but may have little impact on ultimate strength of other components unless large second-order moments develop as a result. Differences in system effects for serviceability and safety limit states must be investigated.

System Reliability and Fragility Modeling

Most proposals for performance-based engineering make distinctions in the performance requirements for different building occupancy categories. Such distinctions are difficult to make without explicitly considering the risk levels associated with different building system categories and performance levels. Thus, this new paradigm will not reach its full potential without rational system reliability analysis procedures that are consistent with the performance goals articulated by building owners, occupants and regulatory authorities. The field of structural reliability provides the framework for evaluation of the role played by uncertainties in hazards, structural loads, strength and stiffness on building safety, serviceability, and durability. In this context, design for specific levels of performance requires not only the connection (or mapping) between the performance level and structural limit state, but also a relation between the limit state and annual probability of occurrence.

The limit state probability for a structure exposed to a hazard can be expressed as

$$P[LS]=\Sigma P[LS|D=x]P[D=x] \tag{2}$$

in which D is a random demand on the system (expressed, e.g., as a 3-sec gust wind speed, wind pressure, or a spectral acceleration at the fundamental period of the building, which are typical parameters used to characterize the natural hazard of interest) and $P[LS|D=x]$ is the conditional limit state probability, conditioned on $D=x$. The hazard is defined by the probability $P[D=x]$. The conditional probability $P[LS|D=x]$ is the fragility. If the hazard is expressed as a continuous function of state variable, x, then the summation in Eq. (2) is replaced by the usual convolution integral of structural reliability theory.

Eq. (2) makes it clear that a structural system fragility analysis is an essential ingredient of a fully coupled risk assessment of a structural system. However, structural fragilities also can be used to determine probabilistic safety margins against specified hazard intensities that can be used for regulation and policy making. While the fragility provides a less informative measure of safety than a fully coupled risk analysis, it also has a number of advantages. For one, the system analysis is effectively uncoupled from the hazard analysis. Thus, a full probabilistic description of the hazard, which often is unavailable, is not essential. Absent credible data on hazards (often the case for extreme environmental events), one might simply inquire as to (a) the fragility were the design event to be exceeded by 50% (in the nuclear industry, where fragility analysis has been used for the safety margin analysis of existing facilities, such an event is denoted a "review-level event"), or (b) for what event is it 95% likely that the system in question will survive. Moreover, a fragility analysis is less complex than a fully coupled risk analysis, and there is less likelihood of miscommunication between the risk analysis team and the end users and decision makers.

A fragility analysis requires a thorough understanding of the mechanics of a structural system response to a range of challenges, ranging from those imposed by service conditions to those that may occur at levels well above the design basis. At the latter levels, the behavior of the structure is usually highly nonlinear in nature. Because of the complexity in behavior of structural components within the system at these levels of demand, closed-form models of behavior are not possible, and one usually must resort to finite element structural analysis. Nowadays, deterministic structural analysis is routinely performed by finite element analysis, and there are a number of commercial codes available for this purpose. In recent years, with advances in computation and in structural analysis software, it has become feasible to evaluate the system reliability by performing structural tests numerically, and evaluating the fragility directly. This has substantial advantages over the classical failure mode approach to system reliability analysis, which generally is difficult or impossible to implement in a complex system. On the other hand, it requires an accurate and computationally efficient finite element model of the system, one that can account for nonlinear structural action, as well as an efficient statistical sampling (variance reduction) plan to estimate the failure probabilities. Fig. 1 provides a conceptual illustration of an ensemble of load-deformation curves obtained from numerical experiments that might be performed by nonlinear finite element analysis. This figure illustrates the variability in performance of a building frame subjected to lateral forces at (a) a resultant lateral force that might be specified in a building code, and/or (b) a prescribed deformation or drift related to a specific building performance limit. The results of such analyses, with appropriate postprocessing, can be used to develop fragilities for different performance limit states, ranging from minor damage to incipient instability. Such a set of fragilities is illustrated in Fig. 2. For an assessment of building performance during extreme winds, for example, the demand variable on the horizontal axis of Fig. 2 might be wind velocity [which can be related to load (force) in Fig. 1 through available wind engineering procedures (ASCE 7-98)] or wind pressure, depending on the source of data and the needs of the analysis. For evaluations of behavior during earthquakes, the demand variable would be spectral acceleration, S_a, at the fundamental period of the building, which is consistent with the specification of seismic risk in many modern standards (e.g., ASCE 7-98).

A fragility analysis of a wood structural system must treat inherent variabilities in wood products used as construction materials, address the duration-of-load (DOL) phenomenon known to govern behavior under sustained loads (Ellingwood and Rosowsky 1991), if required, and model the behavior of connections. The latter is particularly important, as the connections in light-frame construction invariably are semirigid in nature, and a failure to treat their force-relative deformation characteristics

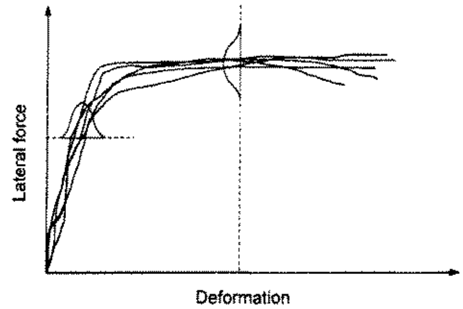

Fig. 1. Load-deformation curves (e.g., from nonlinear finite element analysis).

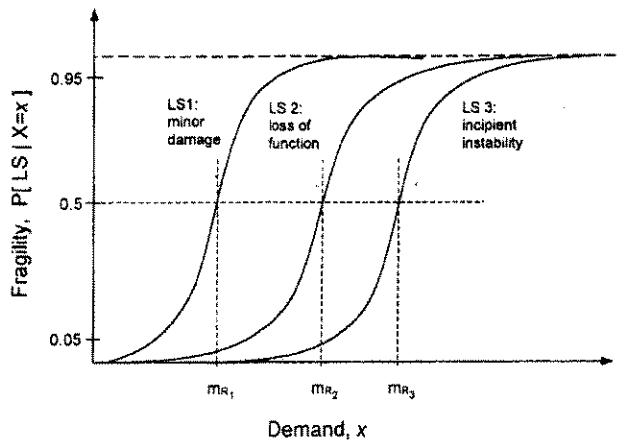

Fig. 2. Suite of fragility curves

properly in the analysis of the system may lead to gross error. All sources of uncertainty must be included in the reliability analysis. These include inherent randomness in component strengths and stiffnesses, as well as epistemic (or knowledge-based) uncertainties that arise from limitations in the supporting databases and approximations of reality made in the structural analysis models.

The fragility of a structural system often has been modeled by a lognormal cumulative distribution function,

$$FR(x) = \Phi[\ln(x/m_R)/\xi_R] \tag{3}$$

in which $\Phi[\cdot]$ = standard normal probability integral, m_R = median capacity (in units that are dimensionally consistent with demand, x, and ξ_R = logarithmic standard deviation, approximately equal to the coefficient of variation (COV)) V_R when V_R is less than 0.3. The parameters m_R and ξ_R can be determined by rank ordering the results of the numerical experiments described above and plotting them on lognormal probability paper. It may also be possible to augment the fragility curves with information on the vulnerability of different structural types in the existing building stock. The resulting curves can then be used to forecast the levels of damage caused by various magnitude events or for postdisaster condition assessment. A hypothetical suite of such curves (structural damage as a function of wind speed for different types of construction) is shown in Fig. 3. In addition to the applications in the nuclear industry mentioned above, a similar approach has been used to assess bridges damaged by the 1995 Hyogo-ken Nanbu (Kobe) earthquake (Shinozuka et al. 2000).

Wood is a natural material, with growth defects and large variabilities in its mechanical properties (Ellingwood 1981; Foschi 1984; Green and Evans 1987; Bodig et al. 1995). Moreover, the strength of wood is sensitive to the rate and duration of structural loads (DOL). In-grade testing programs of the late 1970s and early 1980s at Forintek Canada and the U.S. Forest Products Laboratory (e.g., Gerhards and Link 1986) provide a substantial statistical database needed to develop improved probability-based design procedures that take DOL (as well as species, size, and grade) effects into account. It is unclear at the present time whether the DOL effect must be considered in evaluating the reliability of structural systems subjected to wind or earthquake forces, the available literature being inconclusive on this point. On the other hand, such effects should be considered when performing a probability-based condition assessment of an existing structure that has been damaged by a natural disaster. Consideration should be given to the cumulative duration of structural loading (and consequent damage) prior to the occurrence of the natural hazard event.

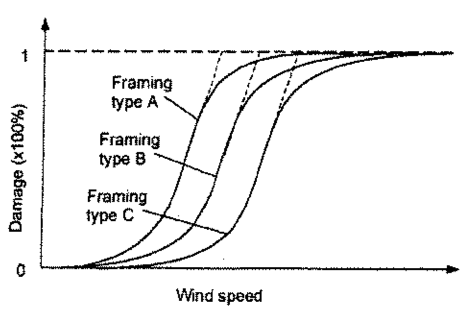

Fig. 3. Fragility-type curves for wind damage to residential structures

Fragility Modeling Tools for Condition Assessment

The fragility modeling procedure described above, while feasible with current computational resources, is relatively complex to perform. Thus, it would be desirable to have available approximate closed-form methods for building system fragility assessment. Such methods would have two purposes: (1) to provide a check of the validity of reliability assessments performed using more complex methods, in much the same way that mean-value FOSM (first-order second-moment) methods provide a check on more advanced FORM/SORM (first-order or second-order reliability methods) or full-distribution reliability analysis methods, and (2) to allow for a relatively quick means for assessing condition of a damaged system when the time available for decision-making does not allow for a more comprehensive examination.

To illustrate the fragility assessment, consider a light-frame floor system. The reliability of any one primary framing member in flexure is relatively easy to determine. However, because of the load-sharing capabilities of wood structural members and components, failure of the floor system may not correspond to failure of the first primary framing member (or connection) unless the failures are brittle in nature and there is little redundancy in the system. If the performance goal is "collapse prevention" or even "life safety," a system analysis based on first-failure (e.g., Folz and Foschi 1989) may lead to a pessimistic view of system capacity. Other studies (Rosowsky and Ellingwood 1991) have suggested that failure in assemblies of typical size often occur when two adjacent members fail, and the remaining structure cannot bridge over the damaged zone. On the other hand, if "prevention of local damage" is the performance goal, first-failure may be an appropriate limit state for the structure.

Fig. 4 presents the results from a fragility analysis of a floor system, with specific dimensions and member properties taken from (Foschi 1985). The fragility curves represent different system limit state criteria (first member, any two members, and all members) and were determined using a floor system model developed for reliability analysis purposes. The points shown with solid and open square markers correspond to results from actual floor system tests (Foschi 1985). The 13 floors tested were built with 12 joists of nominal 2×8 No. 2 or better Hem-Fir. The joints spanned approximately 12 ft (3.7 m) and were spaced 16 in. (406 mm) o.c. The sheathing was 5/8-in. (16 mm) Douglas Fir plywood and was fastened to the joints by $8d$ common nails spaced at 6 in. (152 mm) in all but two of the floors, where the

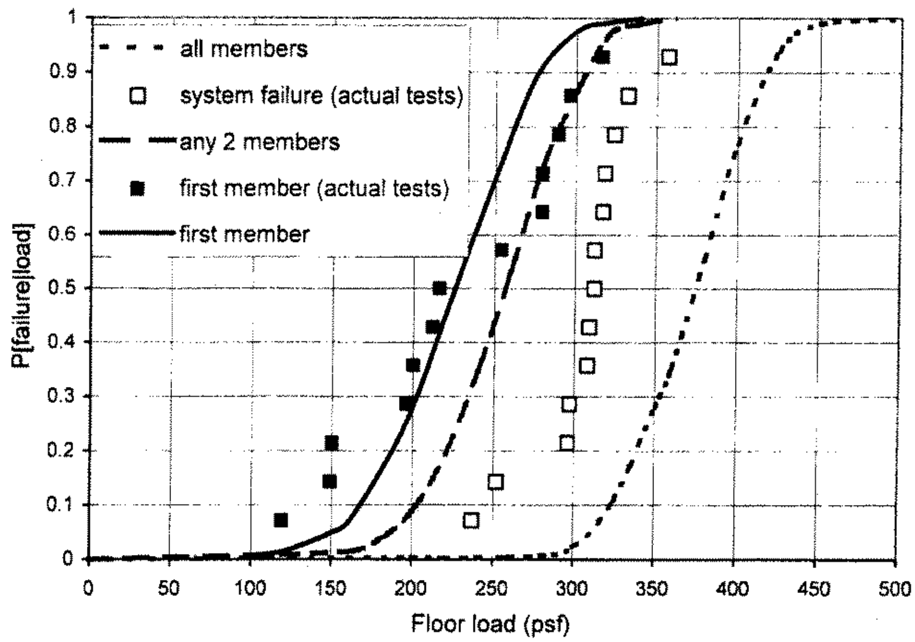

Fig. 4. Wood joist floor system fragility curve example

spacing was 12 in. (305 mm). All tests were conducted under uniformly distributed load. The "first member" failure cases compare well with the analytical fragility curves. In Foschi's study, "system failure" was defined by the peak load of the system rather than in terms of member failures. However, the system failure results fall between the two limiting cases of "any two" members and "all members" failing. This seems reasonable as these two criteria serve as bounds on the system failure limit state. While this is a relatively simple example, and involves only one floor geometry, it illustrates the concept of a fragility analysis for a light-frame system.

Curves such as those shown in Fig. 4, properly validated by experimental data, could be used in developing new building products or in assessing an existing floor system for a proposed change in occupancy, revised load, or suitability for continued service following an extreme event. First-generation performance documents for light-frame construction are likely to include at least two evaluation procedures (see previous discussion): the first based on the traditional safety check typified by Eq. (1) and the second based on product testing in lieu of calculations. For example, one might envision a performance requirement that "local damage shall not occur with 95% confidence at a uniform load intensity equal to the design (factored) load $1.6L$." For $L=40$ psf (1.9 kPa), this load intensity would be 64 psf (3.1 kPa). The curves in Fig. 4 show that local damage, measured by the failure of the first member, is 5% probable at a load intensity of approximately 130 psf (6.2 kPa) to 150 psf (7.2 kPa), meaning that such a floor clearly exceeds the performance objective.

The basis for the approximate fragility modeling procedures in Eq. (3) (or a similar cumulative distribution function modeling the system fragility). Under the assumption that the lognormal cumulative distribution function provides a suitable model of system fragility in most instances, the system fragility is a function of the median, m_R, and logarithmic standard deviation, ξ_R (approximately equal to the coefficient of variation). Research on methods for condition assessment of concrete and steel structural systems in critical facilities (Ellingwood et al. 1999) has suggested that m_R for a structural system often can be estimated, to first order, by performing one (generally nonlinear) finite element analysis in which all parameters are set at their median (or mean) values. It also has been found that ξ_R may be relatively insensitive to minor variations in design parameters within one class of general structural systems. In other words, the COV in resistance to wind uplift of a light-frame gable roof structural system may be approximately the same for minor variations in roof geometry (slope, aspect ratio) within this general roof type. If this can be demonstrated for light-frame construction, then it might be possible to obtain a set of fragilities for a range of performance goals, ranging from immediate occupancy to collapse prevention, by performing one nonlinear finite element analysis of the system. This analysis would be similar to the "static pushover" analysis, which is a common tool in modern earthquake engineering (Krawinkler and Seveviratna 1998). The potential benefits of this approach are significant enough to warrant further investigation as an evaluation tool. Such an approach, if it can be validated, would be invaluable in conducting a postdisaster condition assessment of a damaged building in situ using commercially available structural analysis software.

Summary

Structural engineers continually search for tools that facilitate innovative design and building products and enable them to be competitive in the marketplace. This paper provides an overview of concepts of one such tool, performance-based design, as it applies to residential construction. A performance-based design methodology for residential construction can provide a general framework for assessing the probable response of such construction exposed to various levels of natural and man-made hazards. This methodology could support enhancements in durability and reduction in maintenance costs, facilitate reductions in risk of death, injury, and property damage from extreme natural hazards, and provide a technical basis for the new paradigm of performance-based engineering of residential construction. Furthermore, it could facilitate innovative uses of wood materials in home construction, improve the competitive position of existing and emerging wood-based products in the construction industry, and lead to safer and more affordable housing.

Despite the current focus on natural hazard mitigation in the engineering community and advances in computational tools, a number of challenges to this approach to design remain. Perhaps foremost is establishing the relations between qualitative performance goals and quantitative measures of structural response and behavior. Research also is needed to provide reliability-based methods for assessing the limit state probabilities and performance of residential structural systems constructed with either traditional materials or innovative products not addressed specifically in current codes. Such research will provide essential technical support for performance-based engineering guidelines. The methods also will be useful for evaluating existing construction—to determine whether structures that have been overloaded or "overstressed" according to the original design specifications, or that have been damaged due to a natural or man-made disaster, are safe for continued occupancy. A properly validated system reliability methodology has numerous applications and implications for building regulation and insurance premium underwriting.

Advances in performance-based engineering for residential construction will directly address the objectives of (1) improving the durability of the housing stock and (2) reducing losses due to natural hazards. The long-term benefits of the development of a performance-based design methodology for housing include: risk-consistent and technically sound performance-based engineering criteria; improved building stock in which structures are better able to resist loads due to natural hazards; and reduced economic losses due to damage to the structure and building envelope in high hazard (hurricane or seismic) regions.

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