STRUCTURAL QUALITY ASSURANCE OF WOOD LIGHT-FRAME CONSTRUCTION SUBJECT TO EXTREME WIND HAZARDS

by

Christopher B. Ackerman

B.S.E. Civil Engineering and Operations Research
Princeton University, 1994

Submitted to the Department of Architecture and the
Department of Civil and Environmental Engineering
in Partial Fulfillment of the Requirements for the Degrees of

Master of Science in Building Technology

and

Master of Science in Civil and Environmental Engineering

at the

Massachusetts Institute of Technology

September 1996

Signature of Author . . . .

Certified by . . . .
Leonard J. Morse-Fortier
Assistant Professor of Building Technology
Thesis Supervisor

Certified by . . . .
E. Sarah Slaughter
Assistant Professor of Civil and Environmental Engineering
Thesis Supervisor

Accepted by . . . .
Leon R. Glicksman
Professor of Building Technology and Mechanical Engineering
Chairman, Departmental Committee on Graduate Studies

Accepted by . . . .
Joseph M. Sussman
Professor of Civil and Environmental Engineering
Chairman, Departmental Committee on Graduate Studies

Thesis Reader: Gregory Chiu, Ph.D., P.E.

Title: Engineer, Insurance Institute for Property Loss Reduction

STRUCTURAL QUALITY ASSURANCE OF WOOD LIGHT-FRAME CONSTRUCTION SUBJECT TO EXTREME WIND HAZARDS

by

Christopher B. Ackerman

Submitted to the Department of Architecture and the
Department of Civil and Environmental Engineering on August 9, 1996
in Partial Fulfillment of the Requirements for the Degrees of

Master of Science in Building Technology

and

Master of Science in Civil and Environmental Engineering

ABSTRACT

The poor performance of wood light-frame construction (WLFC) during recent hurricanes disputed the efficacy of the wood light-frame building process in high wind environments. Deficiencies observed in the wake of Hurricane Andrew (1992) in particular suggest that solutions to the problem must consider human-based factors within the building process.

The objective of this thesis is to develop and demonstrate an analytical approach to assess the expected performance of WLFC subject to extreme wind hazards. The approach considers the constructed artifact to be the product of a complex societal process. Aspects of classical reliability theory are augmented with fuzzy mathematics to address uncertainties associated with human factors in the design and assembly of WLFC.

Analyses of a prototypical gable roof sheathing system is presented to illustrate the method. A number of expected system performance measures based on individual component *failure possibilities* are examined. The proposed approach demonstrates the suitability of fuzzy mathematics for the performance evaluation of WLFC in the face of uncertainty. Results suggest that fuzzy sets and systems can be used in more general models that explicitly consider the influence of human factors in the building process.

Thesis Supervisor: Leonard Morse-Fortier

Title: Assistant Professor of Building Technology

Thesis Supervisor: E. Sarah Slaughter

Title: Assistant Professor of Civil and Environmental Engineering

ACKNOWLEDGMENTS

Most sincere thanks and appreciation to all of my advisors for making this thesis such a tremendous learning experience. I thank Professor Morse-Fortier for his continuous encouragement and assistance. Thanks to Professor Slaughter for considering my ideas and for her helpful suggestions. I would also like to thank Greg Chiu for his generous devotion of time and technical expertise. I am also very grateful to Professors Morse-Fortier and Slaughter for partial support of this research.

Thanks also go to my housemates Ken, Tony, Manuel, and Scott for making our time in Boston together so enjoyable.

TABLE OF CONTENTS

CHAPTER ONE

Engineers and technologists in their quest to discover ways of organizing nature and flushed with their successes in the physical sciences have perhaps neglected their reliance on human infallibility. In engineering only the product, the hardware, is a physical system; the system which designs it, produces it and uses it, is human and, therefore, complex and vulnerable.

D.I. Blockley
The Nature of Structural Design and Safety

Introduction

Wood light-frame construction (WLFC) is a ubiquitous part of the built environment and a common type of construction for residential and light commercial buildings. Despite this pervasiveness, the popular perception is that WLFC is a fairly simple endeavor and does require much engineering attention. This perception seems to hold until a catastrophic event, such as a severe hurricane, causes significant damage to buildings and property. Such a condition reveals the degree of complexity of wood light-frame construction.

The vulnerability of existing low-rise buildings and significant shortcomings of the wood light-frame building process were made particularly clear by Hurricane Andrew in 1992. Based on observations made in the wake of Andrew, it seems many factors within the building process combine to influence the final state of the constructed artifact and the performance of WLF buildings subject to extreme wind loads. The complexities of structural behavior are compounded by the web of interacting social systems that design, construct and regulate the creation of wood light-frame buildings. Hence, if the performance of wood light-frame structural systems is to be examined from an engineering point of view, both structural behavior and construction process should be considered.

1.1 Objectives and Scope of Thesis

The main objective of this research is to develop an approach to assess the expected performance of WLFC subject to extreme wind hazards given the influence human factors in the building process. The approach proposed in this thesis based on the premise that the constructed artifact is as much a product of a sociological process as it is a physical system that can be analyzed with engineering principles. This research, while initiated primarily in response to poor performance of WLFC under wind loads, is as about *how* to examine the problem as it is the problem itself.

Given this approach, this thesis asks how can the response of a physical system be modeled in such a way as to consider the influence of human factors? More specifically, what type of structural performance model of WLFC can be developed when qualitative parameters such as "construction quality" are as important as parameters such as wind speed that are more easily quantified? Prior to addressing these questions, relevent existing methodologies and literature are reviewed and interpreted in light of WLFC.

The primary result of this endeavor is the development of a methodology for the performance evaluation of wood light-frame construction. The methodology uses fuzzy mathematics to incorporate the influence of human factors on the strength parameters of the system. The expected performance of a wood light-frame roof system under a wind load and subjected to hypothetical human factor scenarios is evaluated. The suitability of fuzzy measures based on failure possibility are examined and interpreted in light of the results.

1.2 Thesis Organization

In the development of an approach to consider human factors in the building process, this thesis covers a range of topics. Chapter Two establishes a context for the rest of the thesis by presenting alternate ways to interpret human factors in the building process. A sociological interpretation of the building process as a *socio-technical system* is presented and used to formalize some of the events surrounding Hurricane Andrew. A probabilistic model is then presented to illustrate fundamentals of the effects of adverse human action within the building process on the performance of the constructed artifact. A description of the events surrounding the catastrophic damage to a set of WLF buildings in South Florida is provided to illustrate a case of building process failure.

Chapter Three provides basic background of wind loads on low-rise buildings. Wind action on gable roof systems is described, as well as the simplified structural

Chapter Three provides basic background of wind loads on low-rise buildings. Wind action on gable roof systems is described, as well as the simplified structural behavior of wood light-frame gable roof systems under wind loads. Special provisions for high wind construction, including continuity of load path, are briefly given. The chapter concludes with a discussion of current research on wind loads on low-rise buildings and application of such knowledge to damage mitigation.

Chapter Four covers the modeling of structural system reliability and performance. The nature of uncertainty in structural engineering models is first described in some detail. The basic principles of reliability theory are presented, with particular attention to structural reliability theory. Alternate methods of analyzing structural reliability are also reviewed. The chapter closes with an extensive discussion of the theories of fuzzy sets and possibility and their application to address uncertainties in structural engineering models.

Chapter Five considers ways to model human factors and the building process. The state-of-the-art in human error modeling for structural reliability is reviewed as well as the application of process simulation to the construction process. The proposed methodology to assess the expected performance of wood light-frame construction based on concepts covered previously concludes the chapter.

Chapter Six develops and illustrates the proposed methodology to assess the expected performance of wood light-frame construction. The computational aspects of the methodology are first illustrated by an example of a simple pin-connected frame structure. The methodology is then applied to a prototypical gable roof sheathing system. Assumptions regarding uplift limit state, and the resistance and load effect models are described. Results of simple fuzzy expected performance analyses are also presented.

A summary and the conclusions of this research are presented in Chapter Seven. Limitations of the methodology and its application are discussed, followed by recommendations for further work.

CHAPTER TWO

The growth of engineering has been marked by the influence and successful application of the physical sciences to engineering problems. This, however, has not been matched by a similar application of the social sciences to the social aspects of engineering problems.

N.F. Pidgeon and B.A. Turner
"Human Error and Socio-Technical System Failure"
Human Error in Design and Construction

Performance of Wood Light-Frame Construction and Building Process Failure

The poor performance of wood light-frame (WLF) construction during Hurricane Andrew of 1992 prompted a reevaluation of the WLF building process. Based on the post-disaster studies, it seems a number factors within the building process are likely responsible for this damage. This chapter attempts to better understand the nature and possible influence of human factors in the building process.

To understand the nature and possible causes of these effects, we examine human factors in the building process from two points of view. The WLF building process can be viewed from a sociological perspective as a *socio-technical system* wherein general patterns of catastrophic events are formalized. A probabilistic interpretation is also presented to provide a more quantitative formulation of the effects of human factors on building performance. This chapter concludes with a case study that illustrates the concept of building process failure according to these constructs and that application of engineering principles to improve the performance of wood light-frame construction must consider the human element within the building process.

2.1 Assessment of Damage to Wood Light-Frame Construction

Hurricane Andrew struck south Florida, on August 24, 1992. Many aspects of the building process, including design, building codes and enforcement, and construction quality, were tested by Hurricane Andrew. The estimated at $20 to $25 billion in damage to a densely populated area in Dade County caused by Andrew makes it on costliest natural disasters in the history of the United States ("Assessment" 1993).

This extreme level of damage prompted a number of industry and government investigations (e.g. "Building" 1992; Douglas 1992; "Assessment" 1993; Keith and Rose 1994) to assess the aftermath of Andrew and provide recommendations for the mitigation of future hurricane damage. At a macro level, the damage seems to have been caused by several interconnected factors, which are summarized below:

Table 2.1

Factors Contributing to Damage of Hurricane Andrew
(adopted from "Assessment" 1993)

Factor	Examples
Construction	Workmanship, inspection, and building code requirements.
Design	Aesthetic and structural elements.
Building Products and Materials	Performance standards and building code requirements.
Preparedness	Home owner awareness, preparation, maintenance and, training.
Acceptable Risk Policy	Probable extreme wind speeds and storm surge in coastal areas.

These factors are all considered to be part of the wood light-frame building process, which can be broadly defined as the interconnected social systems that create, regulate, and maintain the WLF built environment. All of these factors have the potential to affect the level of damage form hurricanes, although some particular factors may have more of an influence than others.

Generally, structural failure of low-rise WLF buildings was the result of negative pressure overloading the building envelope combined with a lack of load path continuity within the framing system. Breaks in this load path such as inadequate (absent or improperly installed) framing connections, load transfer straps or partition-to-wall bracing were observed in south Florida following Andrew. It is believed such deficiencies, when present, significantly contributed to the structural failure ("Building" 1992).

Site observations of wood light-frame building performance were also conducted to assess the types of failure and speculate on their possible causes. It was generally observed that opening protection, roof coverings and roof sheathing attachment were the most significant parameters that determined the overall hurricane resistance a particular WLF building. Most structural damage observed was related to roof systems, and gable roofs in particular ("Assessment" 1993). One of the most commonly-observed failure modes of gable roof systems is the uplift of sheathing panels. The significance of this type of failure is that it constitutes a breach in the building envelope, which leads to building content damage and, to a lesser degree, progressive failure of other parts of the roof (Figure 2.1).

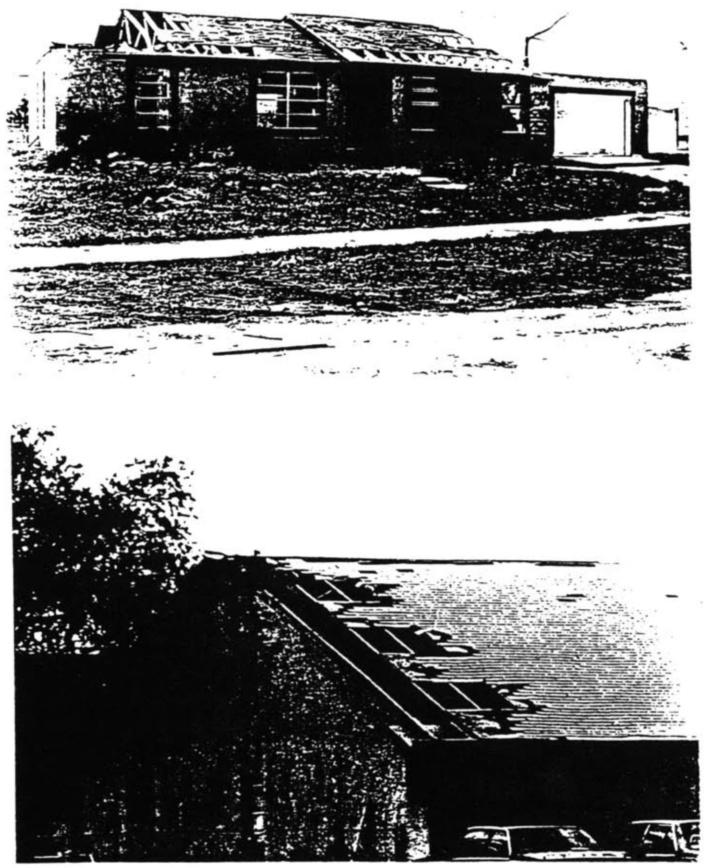


Figure 2.1
Typical modes of wind-induced damage to homes observed in the wake of Hurricane Andrew. Note individual panel loss at gable ends and roof perimeter (a) - (c) and subsequent gable roof end failure (d) (from "Assessment" 1993)

Failure of roof systems was due to one or more of the following construction weaknesses (Wolfe et al. 1993):

- Weaknesses in the attachment of roofing to roof sheathing
- Sheathing attachment to roof framing
- Rake overhang details

A multitude of recommendations have been made since Hurricane Andrew and cover specifics such as fastener spacing in prescriptive building code requirements to programs to inform home owners to prepare their homes for impending storms. Revisions of building codes, increased enforcement and education of builders and code inspectors with regard to load transfer mechanisms (Wolfe et al. 1993), certification of specialized inspectors, load path checklists during construction ("Building" 1992) and overall increased compliance with wind resistant construction practices ("Assessment" 1993).

It is evident from this brief review of Hurricane Andrew's damage to the built environment that it has significantly influenced on the current WLF building process. As a starting point to gain a better understanding of this influence, and how the damage from future hurricane events might be mitigated, a description of the wood light frame building process is presented in the next section.

2.2 The Wood Light-Frame Building Process

Wood light-frame construction (WLFC), the first uniquely American building system, was invented in Chicago the 1830's and spread quickly throughout the United States following the increased availability and reduced cost of small wood framing members and machine-made nails. It is one of the most flexible building systems and provides an extremely economical construction method for low-rise buildings. The platform frame is the most common type of construction for residential and light commercial buildings in North America today (Allen 1990). The construction sequence of a prototypical platform frame structure is depicted in Figure 2.2.

Despite pervasive industrialization since the 1830's, wood light-frame construction is still primarily assembled manually on site with common hand tools. Today the residential construction industry is a highly fragmented (i.e. diverse and localized) collection of small-size firms that rely on the manual skills of its labor force (Ventre 1979). Common residential low-rise construction is considered to be "nonengineered" by the model building codes. It is because extensive engineering analysis is not applied (or required) in the design or construction of typical wood-frame buildings that WLFC is regulated more closely than any other type of construction in the United States. This regulation is manifest in the documentation and enforcement of detailed *prescriptive requirements* intended to provide the necessary guidance to design and construct safe and durable buildings. The nonengineered building process of wood light-frame construction is schematically depicted in Figure 2.3.

Prescriptive requirements have traditionally been based on past experience and satisfactory performance and evolve over time to incorporate new knowledge and technological advances. The development and enforcement of such prescriptive requirements are the primary means of quality assurance, i.e. mechanisms that control design and construction quality, in the WLF building process. The term *quality assurance* is formally defined a number of ways in the literature and in this thesis refers to *the process by which the various components of the complete building process are coordinated with the aim of achieving the design objective* (Thoft-Christenson and Baker 1982). The "design objective" of the WLF building process is the cost-effective provision of safe, reliable and durable low-rise buildings.

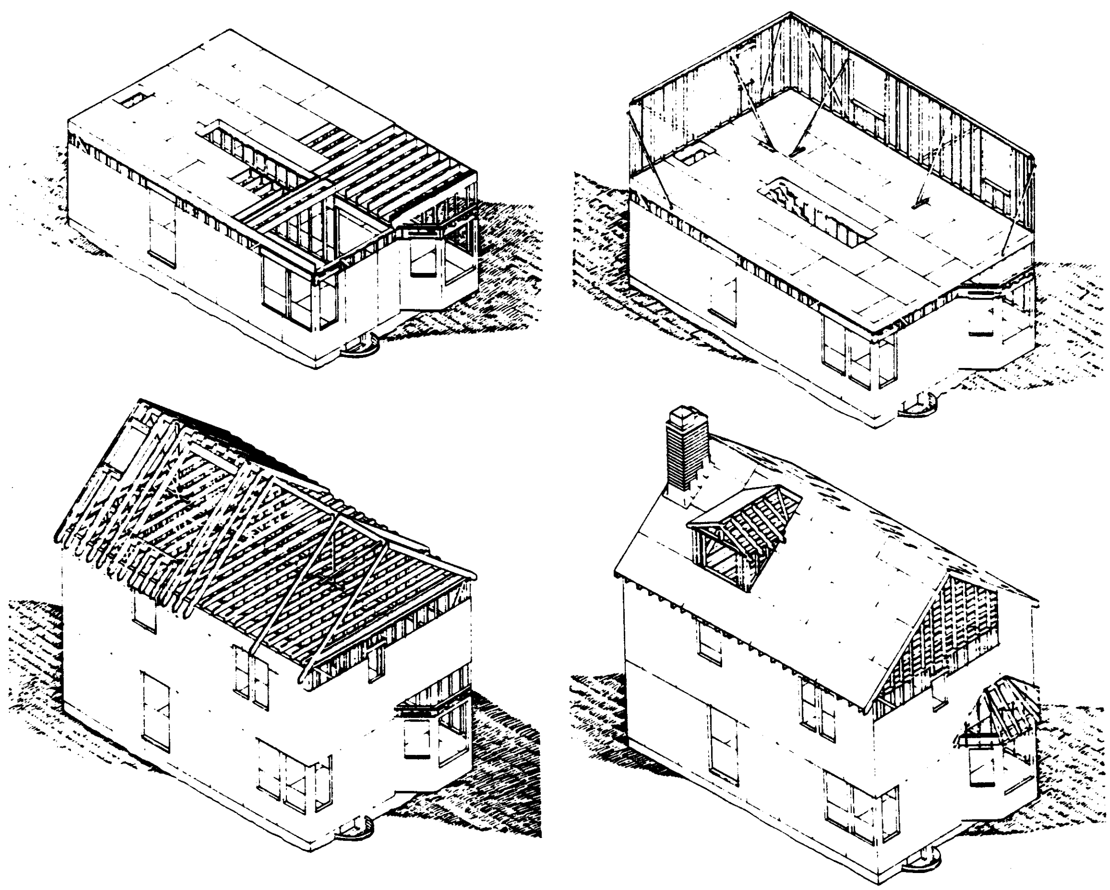


Figure 2.2
Sequential schematics of a typical platform residential wood light-frame structure (from Allen 1990)

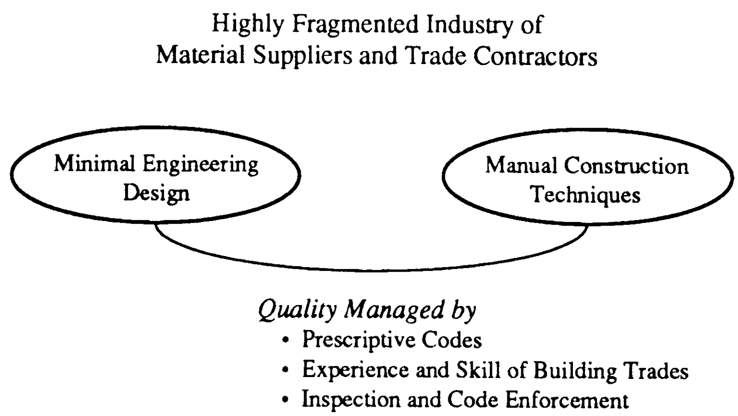


Figure 2.3
The nonengineered building process of wood light-frame construction

In its current form this type of quality assurance - the development and enforcement of prescriptive requirements - has its limitations. For the most part, societies of developed countries view housing construction as a relatively simple endeavor for which the sufficient knowledge and experience already exist to build reliable structures. But as Walker and Eaton (1983) point out, rapid changes in building practice during the past three or four decades can nullify past experience, especially with respect to hurricanes. The confluence of these developments with the regional scale of wind storms creates a condition such that not one building but rather a set of buildings is at risk to damage from extreme wind hazards. These points were made abundantly clear in south Florida by Hurricane Andrew.

Tools to analyze how and where things can go wrong in the building process of WLFC might provide some insight into the events surrounding Hurricane Andrew and perhaps aid in avoiding similar situations in the future. Although the building process as described in this section is complex and difficult to quantify, two models to formalize the influence of human action on the building process can provide a starting point.

2.2.1 Socio-Technical Systems Theory

Pidgeon and Turner (1986) propose a theory of the building process in response to their observation that large-scale failures (e.g. Hurricane Andrew) are rarely caused by any single factor but rather a confluence of events. Their theory is based on the premise that individuals, groups, and organizations influence the design, construction, monitoring, operation and maintenance of all technological systems. In the analysis of failure events, they posit that

> "If we learn to think of engineering systems in socio-technical terms we become critical of approaches to the causes of disasters which stress only the technical factors...A more thorough learning experience, and further reduction in the incidence of failures, can only arise from a consideration of the technical, human and organizational causes of disasters."

This approach suggests that analysis of the social context within which engineering occurs is requisite to a thorough understanding of such a complex system. Pidgeon and Turner also point out that in the analysis of catastrophic events it is incorrect to ascribe the term *human error* (which connotes primarily individual failings) to the variety of behavioral problems underlying the malfunction of a socio-technical system. They advocate the use of *human factors* to include the complexity associated with personnel management, the transfer of information, communication, personal misunderstandings and misapprehensions *as well as* individual fallibility. For this reason, human factors in this thesis refer to any and all societal influences that adversely affect the performance of the constructed artifact.

According to the socio-technical system model, a three stage pattern precedes any large-scale failure (Table 2.2). The first stage is characterized by an appropriate set of beliefs about accepted hazards and precautionary norms (e.g., laws, codes of practice and "engineering judgment"). The second stage, referred to as the hazard *incubation period*, is a series of events that collectively create a condition disparate from that assumed by beliefs and norms of the first stage. This leads to a *trigger event* of catastrophic consequences precipitating the placement of blame among the various parties involved in the complex socio-technical system and ultimately a reevaluation of norms and beliefs.

Table 2.2

The Sequence of Events Associated with Catastrophes
(after Pidgeon and Turner 1986)

Stage I	*Notionally Normal Starting Points* Initial culturally acceptable beliefs about the world and its hazards. Associated precautionary norms set out in laws, codes of practice and "engineering judgments."
Stage II	*Incubation Period* The accumulation of an unnoticed set of events which are at odds with the accepted beliefs about hazards and the norms for their avoidance.
Stage III	*Trigger Event* Final critical event or abnormal operating condition. Irrecoverable onset of disaster and ultimate transformation of general perceptions of Stage I.

Pidgeon and Turner identify two implications for the application of their model. First, the successful detection of a potential incubating hazard prior to a trigger event can provide as much insight as an actual trigger event into the avoidance of future catastrophes. Second, while it is unreasonable to expect a precise and highly accurate prediction of any particular combination of events that might lead to disaster, failures can nonetheless be prevented by the identification of an incubating hazard at a sufficiently early stage.

Finally, there are three principal routes for the introduction of misinformation into the building process that can contribute to the incubation of disasters: individual, small group and institutional. Individual factors refer to cases of human error such as an individual's failure to perform a well-defined function. Small group factors are problems associated with work role definitions, allocation of responsibility and communication between individuals. Institutional factors are related to the institutional bodies, codes of practice and regulatory frameworks within the building process.

The applicability of Pidgeon and Turner's model of the building process Hurricane Andrew is obvious. The events surrounding Hurricane Andrew as presented Section 2.1 are a classic example of socio-technical system failure triggered by a catastrophic event and involving a combination of social, technological and organizational factors. Indeed, the human influence at a number of levels in the building process - from the absence of engineering design to poor construction to inadequate code enforcement - seemed to be contributive factors to the extensive damage of Hurricane Andrew.

There is little value in the description of past events if no lessons are learned and no is action taken to avoid future catastrophes. A vulnerability problem still exists and its gravity and pervasiveness should not be overlooked. As Gaus et al. (1993) note, the existing housing stock along the coastal regions of the south east United States is, like many houses prior to the destruction of Hurricane Andrew, "like a time bomb waiting to go off." Unfortunately, the pattern suggested by Pidgeon and Turner has yet to come full circle and the "incubation period" continues for a significant amount of coastal low-rise construction in the United States.

The socio-technical model of the building process was presented in this section to provide a perspective wherein past events can be appropriately interpreted. Further, this model serves as a starting point from which we can begin to address the multitude of factors that apparently affect the performance of wood light-frame buildings subject to extreme wind hazards. Within this context a simple model to measure the possible effects of social factors is described in the next section.

2.2.2 Probabilistic Interpretation of the Building Process

The building process can also be interpreted as a sequence of events with various outcomes. In a probabilistic context, the event tree approach expresses a sequence of plausible Boolean events as branches unfolding from an existing "state." In such a framework, outcomes of events related to the influences of human factors can be delineated from other possible outcomes. Ellingwood (1987) represented the influence of human factors on the likelihood of structural failure in such a way. In the following model the term *error* is considered to be a discrete manifestation of human factors that contributes to a *failure event*, defined as "an unacceptable difference between expected and observed performance" (Carper 1989).

Consider the failure of a generic "structure" to be caused by either random overloading or understrength or by adverse effects of human factors in the building process. Let this failure be an event denoted by F which is the union of two independent events. Mathematically this is stated as

$$F = F_s \cup F_e \tag{2.1}$$

where

F_s = failure due to stochastic variability in loads and resistances

F_e = failure due to error in the building process

Now consider an event E to represent the occurrence of a generic "error" that represents the negative effects of human factors on the structure. If the events are assumed to be independent, the possible outcomes of the building process including the influence of human factors can be presented in an event tree (Figure 2.4). Following the total probability theorem, the probability of failure can be stated as

$$P(F) = \left[P(F_e|E) + P(F_s|E)\right]P(E) + P\left(F_s|\overline{E}\right)P\left(\overline{E}\right) \tag{2.2}$$

where

$P(F_e|E)$ = conditional probability of failure given error occurrence

$P(F_s|E)$ = conditional probability of failure given stochastic variability

E = event that an error occurs

$\overline{E}$ = event that an error does not occur

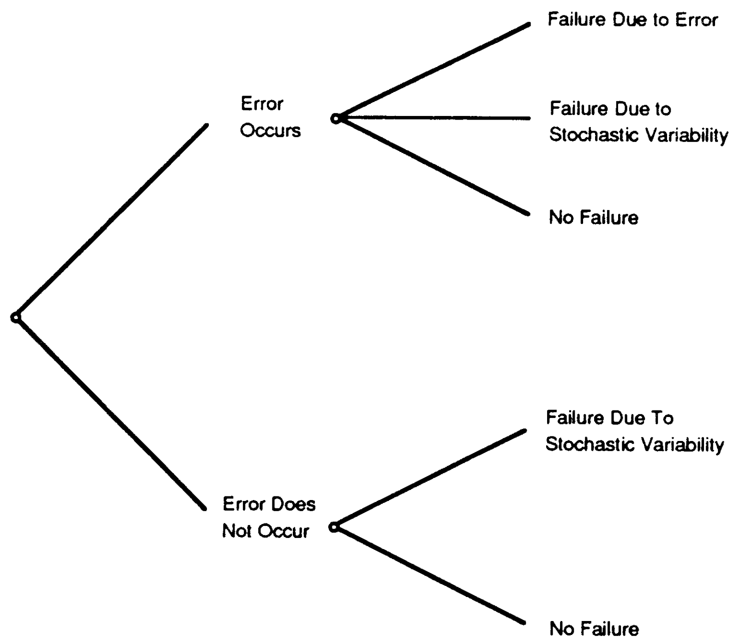


Figure 2.4
Event tree analysis of failure (after Ellingwood 1987)

This model includes the important notions of error consequence, detectability and correctability. As Ellingwood points out, $P(F)$ can be reduced by controlling either the incidence of errors by limiting $P(E)$, by controlling the impact and consequences of the errors on structural performance, i.e. limiting $P(F|E)$ by designing the structure to tolerate and absorb the effect of errors, or by a combination of these two strategies.

One scenario of interest is failure due to stochastic variability given an error has occurred, denoted by the term $P(F_s|E)$. Hurricane Andrew can be interpreted as an example of this possibility; the combination of an extreme load (e.g. a hurricane) and an error (e.g. imperfect design or construction). This is also consistent with Pidgeon and Turner's notion of the *incubation of failures* as outlined in the previous section and could be the confluence of increased building, technological changes in wood light-frame materials and construction technology, and a series of relatively mild hurricane seasons along the south Florida coast. This very plausible combination of events illustrates how the occurrence of error contributed to failure only in the presence of an extreme load event.

It should also be noted that the term $P(F_s|\overline{E})$, i.e. the probability of failure due to stochastic load given no error has occurred, represents an outcome normally controlled by factors of safety implicit in prescriptive codes. Indeed, prescriptive requirements are developed under the presumption that buildings are either designed and constructed in accordance with the provisions or amended to be through inspection. The efficacy of this control mechanism is obviously diminished if the inspection process is a less than effective error filter. Further, no matter how rigorously such provisions are developed and enforced, they cannot in and of themselves anticipate all necessary situations.

The term $P(F_e / E)$ represents an obvious outcome caused by errors due to human factors in the building process and can be controlled by making the structure more robust to the effects of errors. In principal this is analogous to a structure having multiple layers of redundancy whereby an occurrence of an error has a negligible impact on the probability of failure. As discussed briefly in Section 2.2, WLFC has evolved by experience to include many layers redundancy. But if modifications to the structural system sufficiently remove these redundancies or when past experience is no longer valid, the structural robustness of WLFC is reduced and hence increase $P(F_e | E)$.

2.4 Case Study of Building Process Failure

The complexities and contradictions associated with the greater than expected vulnerability to hurricane damage of WLFC are illustrated by the case of a residential development in south Florida. Post-disaster site investigations were conducted by one investigator that led to conclusions in contrast with those based on some prevailing interpretations of wood light-frame building performance (Morse-Fortier 1996). The case is intended to illustrate some of the failings of the current wood-light frame building process and how the WLF building process itself conflates the notion of a singular "human error" and necessitates the use *human factors*.

The Lakes by the Bay residential development, located in south Dade County, Florida, was comprised of two-story residences situated on raised earth berms along curving streets and cul-de-sacs. This site organization lead to a number of different building orientations, a series of pie-shaped lots around cul-de-sacs and irregularly-shaped spaces between individual buildings. As the name implies, residences within the development were located adjacent to and surrounded by small lakes. Although these created fairly unusual circumstances, the design and construction of these dwellings fell within the purview of the building codes by which traditional residential dwellings are designed and constructed.

The residences of Lake by the Bay were extensively damaged due to Hurricane Andrew and prompted a number of studies. One engineering assessment concluded the houses were not properly engineered and constructed to withstand the wind loads induced by Andrew. In contrast, Morse-Fortier (1996) posited that the residences were designed and specified in accordance with the prescriptive criteria of the building code. On-site investigation of nailing schedules and tie-downs verified that the residences were indeed largely constructed as specified. Why the residences sustained such extensive damage seems to be rooted in the coincidence of a number of unusual circumstances that were not (and perhaps could not have been) addressed by the prescriptive standards to which the residences were built.

It follows that the residences failed because they did not possess sufficient strength to resist the loads *despite* the fact that they were in conformance with the required code provisions. Wind loads were unusually high for residences for four separate reasons. The lakes provided an open upstream fetch, the earth berms represent escarpments over which velocities increase, the relationships among buildings allowed funneling, and finally, the houses themselves were taller than usual.

What then can be said? If the residences of Lakes by the Bay were designed and built to code, where then does the fault lie? The troubling conclusion is that the applicable code prescriptions did not ensure adequate resistance given such conditions and the structural engineering and design necessary to do so was not required. Regardless of where the fault lies, such a condition represents a failure in the building process of wood light-frame buildings.

The models described in this chapter were used to interpret the conditions surrounding Hurricane Andrew and to introduce a number of concepts central to this thesis. Pidgeon and Turner's socio-technical system model provides insight into social aspects of catastrophe in the building process. The broadly defined term human factors is considered to include individual, small-group and institutional levels of human influence that contribute to the incubation of structural vulnerability. Ellingwood (1987) provides an alternate model of possible outcomes in light of human factors in the building process and attempts to quantify the possibly adverse effects of human factors on structural performance. Before these concepts are further developed, a discussion of hurricane hazards and the nature of wind loads on low-rise buildings is presented in the next chapter.

CHAPTER THREE

Basically society has considered that housing does not warrant engineering analysis and design.

G.R. Walker and K.J. Eaton
"Application of Wind Engineering to Low Rise Housing"
Journal of Wind Engineering and Industrial Aerodynamics

Structural Behavior of Wood Light-Frame Construction Under Wind Loads

The built environment is subject to a number of natural loadings, including wind. The influence of near-surface winds on buildings is increasing as the built environment expands. Unfortunately, near-surface winds are one of the most variable of all meteorological phenomena and hence loads for design are difficult to predict. Major parts of the United States are vulnerable to extreme wind events, such as tornadoes and hurricanes (Wind 1993).

Simplified behavior of low-rise wood light-frame structures under wind loads is summarized in this chapter. The nature of wind loads due to extreme wind events and their effects on low-rise buildings is discussed, including a review of the currently accepted method for calculating wind loads (e.g. "Minimum" 1993). Approximate methods to account for the spatial variability of wind loads on low-rise buildings is also presented.

The adverse effects of localized loading and the importance of load path continuity on structural response to loads are discussed, with a particular emphasis given to roof systems. Examples of recommended construction methods and details are included to illustrate key concepts and strategies underlying hurricane-resistant construction. Finally, research related to low-rise buildings currently underway within the wind engineering community is reviewed, including strategies to mitigate losses to extreme wind events.

3.1 Wind Loads on Low-Rise Buildings

The violent nature of extreme wind events such as hurricanes makes it difficult to accurately measure maximum wind speeds *in situ*, necessitating the use of other methods, including post-disaster surveys of damage to structures. The Saffir-Simpson Scale (Table 3.1) is the currently accepted classification method for hurricanes. The Saffir-Simpson scale provides reasonably good assessments of storm intensities and expected damage. However, Conner et al. (1987) points out discrepancies between expected and actual levels of damage have been observed, suggesting that site-specific factors (e.g. topography, local design and construction techniques) might significantly influence actual damage.

Table 3.1

Saffir-Simpson Scale

Category	Damage Potential	Sustained Wind Speed (mph)	Gust Speed (mph)
Category One	Minimal	75 - 90	115
Category Two	Moderate	91 - 110	130
Category Three	Extensive	111 - 130	160
Category Four	Extreme	131 - 155	190
Category Five	Catastrophic	156 - 185	225

Wind loads on structures arise from the incidence of wind flow, the turbulent wake generated by the building and the internal forces induced by the dynamic response of the structure. The first two sources of wind-induced loads are the dominant factors with respect to low-rise buildings. These loads induce lateral and axial loads on vertical elements, uplift loads on non-vertical surfaces, as shown schematically below.

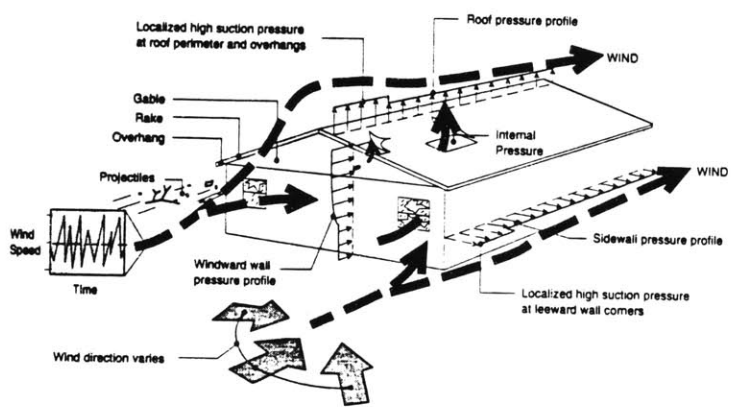


Figure 3.1
Basic wind effects on typical low rise buildings (from "Assessment" 1993)

Design wind speeds are determined from a basic wind speed map of contours based on the likelihood of occurrence and expected magnitude of extreme events (e.g. Figure 1 of ASCE 7-93, "Minimum" 1993). Wind loads are usually idealized as distributed static loads despite temporal and spatial fluctuations of pressures. The design wind pressure p_m to be used for the design of a structure's main wind-force resisting system (MWFRS) is normally calculated as ("Minimum" 1993):

$$p_m = qG_hC_p - q_h(GC_{pi}) \tag{3.1}$$

where

q = reference velocity pressure [psf]
 = q_h for leeward wall, side walls, and roof elevated at mean roof height
G_h = gust response factor
C_p = external pressure coefficient
GC_{pi} = internal pressure coefficent

Design wind pressures p_{cc} for components and cladding for building of height less than or equal to 60 feet are calculated according to

$$p_{cc} = q_h\left[(GC_p) - (GC_{pi})\right] \tag{3.2}$$

where GC_p is the external pressure coefficient for loads on building components and cladding and all other quantities are the same as defined in equation (3.1.1).

The reference velocity pressure q_z (in lb/ft²) at height z is calculated with

$$q_z = 0.00256K_z(IV)^2 \tag{3.3}$$

where

V = basic wind speed [mph]
I = importance factor
K_z = velocity pressure exposure coefficient

and the numerical constant accounts for air density and includes a unit conversion coefficient. For the calculation of wind loads on roofs the mean roof height (MRH) is normally used as the reference height to determine the various coefficients.

The product of external pressure coefficient and gust response factor GC_p is of particular interest to this thesis since the presence of local instantaneous peak negative pressures may adversely affect cladding elements and the immediate underlying supporting members. In the determination of design wind loads, ASCE 7 prescribes values of GC_p by location of the components with respect to the overall building. Such values are determined in accordance with the following figure:

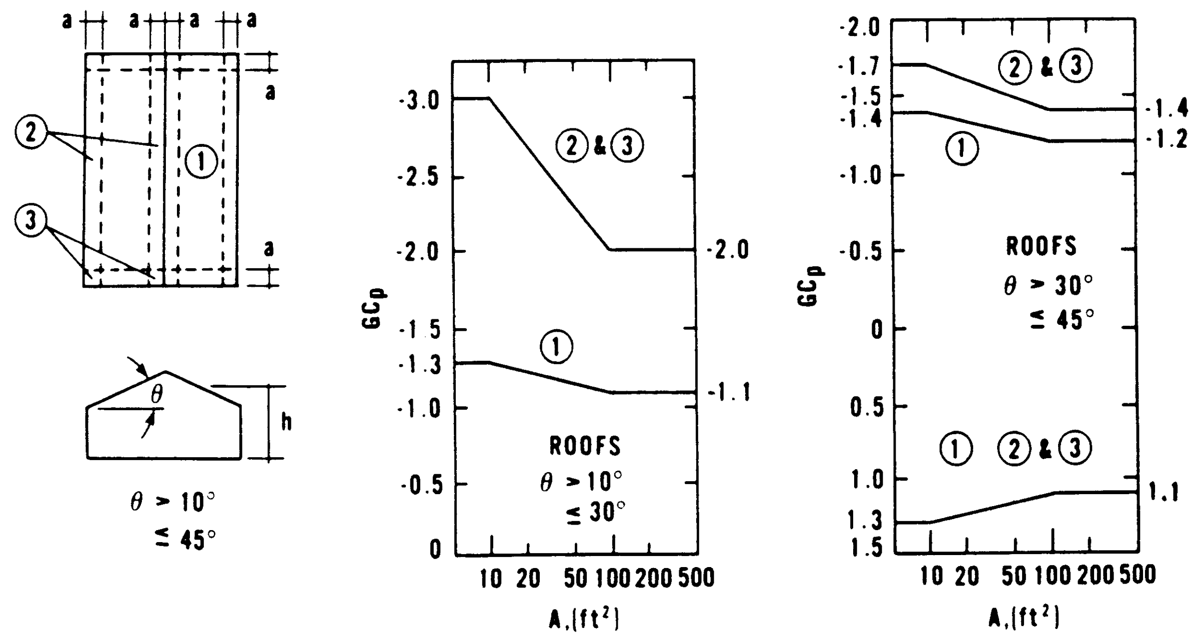


Figure 3.2
External pressure coefficients for loads on building components and cladding for building with mean roof height less than or equal to sixty feet. Coefficients for walls as a function of tributary area (a), and coefficients for roofs as a function of tributary area (b) (from "Minimum" 1993)

The nature of wind flow around low-rise roofs has been the subject of much recent research. Meecham (1988) conducted wind tunnel tests of gable and hip roof geometries and has observed the following with respect to wind flows around gable roofs of low to moderately sloped roofs (Figure 3.3):

> *Winds normal to the roof ridge.* Wind flow normal to the ridge separates from the leading edge and reattaches downstream on the windward slope. A secondary separation occurs at the ridge that induces a relatively uniform negative pressure on the leeward slope.
>
> *Quartering winds.* The prominent effect of wind incident at a 45° of a gable roof is a "hot spot" of very high negative pressure induced by conical vortices that emanate from the leading (windward) corner of the roof. The windward corner of the leeward slope experiences the maximum suction under such conditions.

Winds parallel to the roof ridge. Flow parallel with the roof ridge separates form the windward gable edge and reattaches around the middle of the building length. This flow induces high suction at the leading edge.

It is important to point out that while the above loading conditions correspond to the three fundamental angles of incidence of wind, a given building can experience a hybrid mix of these wind load scenarios and their complements (i.e. wind flow incident at the same angle but in the opposite direction) during the passage of a hurricane. For this reason loads and uplift pressures used for the purposes of design (Figure 3.2) and specification are actually an "envelope" of maximum loads corresponding to these fundamental directions.

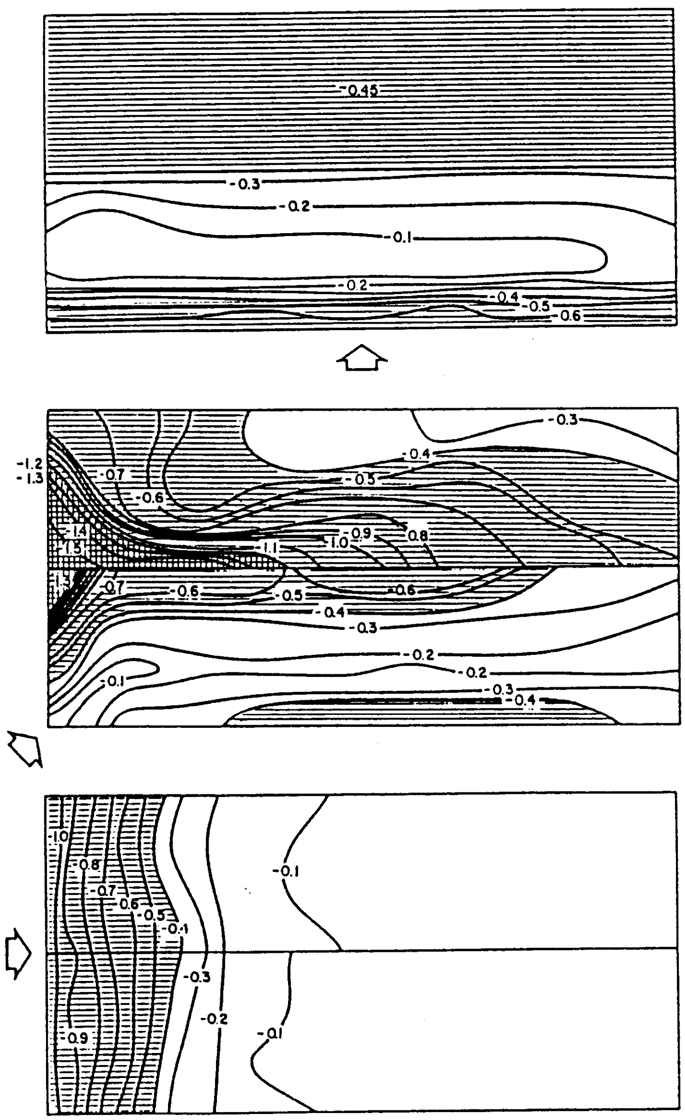


Figure 3.3
Contours of mean pressure coefficients measured from wind tunnel tests of simulated open country exposure for three basic wind directions: winds normal to the ridge (a); quartering winds (b); and winds parallel to the ridge (c) (from Meecham 1988)

3.2 Simplified Structural Behavior of Low-Rise WLFC

The nature of wood light-frame construction is that of a "skeletal" frame enclosed and stiffened by sheathing elements on both the interior (e.g. drywall) and exterior (e.g. plywood panel) wall faces. Sheathing acts to stiffen the frame elements and is an essential component to overall stability and strength; the structural response of wood light-frame buildings to wind loads is very dependent upon how well the sheathing is fastened to the underlying framing members. If the roof, wall and ceiling planes have adequate shear resistance, and are connected to one another, the overall structural response of a wood light-frame building should be adequate. The details of its response will be determined by the relative stiffness among numerous and competing load paths (Morse-Fortier 1993).

Loads act on wall and floor subsystems either in or out of plane. Lateral resistance to shear forces is provided by the diaphragm action of sheathed (and hence stiffened) frame elements. The structural behavior of wood-framed shear walls has been studied extensively (e.g. Tuomi and McCutcheon 1978; Easley et al. 1982; Gupta and Kuo 1985).

Parallel-member wood structural systems subjected to out of plane loading exhibit load sharing among members. The two primary load sharing mechanisms in WLFC are *two-way action* and *partial composite action*. Two-way action is due to sheathing elements acting as a wide and continuous beam in the direction perpendicular to the members. This tends to distribute the load to adjacent members and reduces differential deflection of adjacent members caused by stiffness variability or non-uniform loading. Partial composite action is a mechanism that causes behavior resembling a T-beam and is due to the semi-rigid connection between members and sheathing elements (Bulleit et al. 1993).

The structural behavior of low-rise wood-frame construction subject to wind loads is fairly complex but can be simplified for the purposes of this discussion. The response to lateral wind loads can be categorized into three primary modes and summarized as follows:

- Vertical wall elements ("studs") act as vertical beams that transmit transverse pressure and leeward suction to the top and bottom wall plates.
- The top plate reactions are transmitted by diaphragm action of the ceiling or roof to bracing walls parallel to the principle wind direction.
- Bracing walls transmit the racking forces to the foundation through shear action.

As described in section 3.1, wind loads induce uplift pressures normal to the roof surface as well. The response of typical low-rise wood-frame construction to uplift loads can be understood through a "chain" analogy; to resist uplift loads the building must provide a continuous load path from the roof to the foundation. This path depends upon connection strength of cladding (e.g. shingles) to secondary members (e.g. roof sheathing), secondary members to the roof framing (rafters or trusses), roof framing to top plate, top plate to bottom plate and bottom plate to foundation (slab or sill). This load path is graphically depicted in Figure 3.4.

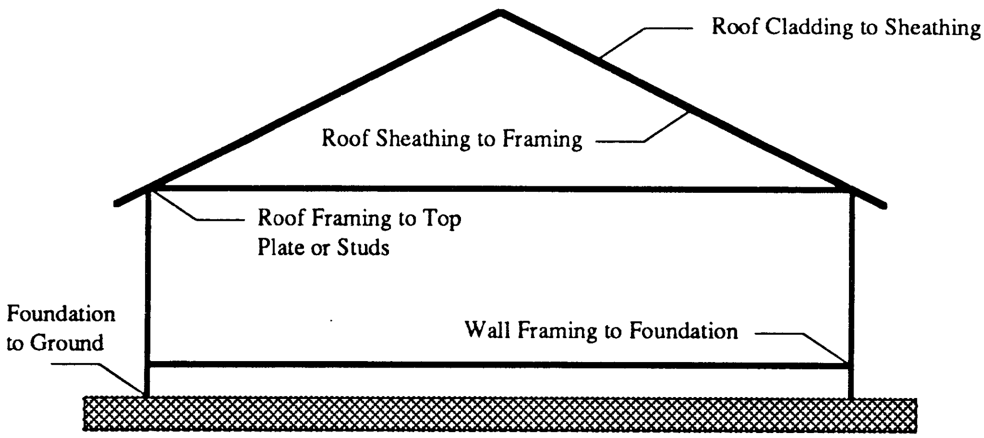


Figure 3.4
Uplift "chain" of typical wood light-frame structure subject to wind loads

Meecham's (1988) investigation of the aerodynamic properties of roof geometries was undertaken in response to observations that hip roofs perform better than gable roofs under extreme wind conditions. Possible causes of commonly observed failure modes (Section 2.1) for roof systems such as global overturning, failure of discrete end trusses and localized failure of cladding elements were inferred from the results of scale model wind tunnel tests. The study concluded that the different aerodynamic behavior of the two geometries, coupled with different framing systems, leads to significantly different internal distributions of the wind-induced loads. These results suggest that gable roof systems may be more vulnerable to wind damage than hip roof systems.

3.3 Design and Construction Provisions of WLFC Subject to Extreme Wind Hazards

The structural characteristics of WLF buildings derive from their design and the prescriptions offered in building codes and industry- or agency-published construction guides. Primarily for practical purposes, the structural behavior of low-rise WLFC is communicated to participants in the building process most often through code prescriptions and recommended construction details. Documents such as the *Coastal Construction*

Manual ("Coastal" 1986) published by the Federal Emergency Management Agency; *Wind- and Flood-Resistant Construction* ("Wind" 1995), published by the Building Officials and Code Administrators International, Inc.; *Standard for Hurricane Resistant Residential Construction* ("Standard" 1993), published by the Southern Building Code Congress International, Inc.; *Wood Frame Construction Manual for One- and Two-Family Dwellings* ("Wood" 1996), published by the American Forest and Paper Association, and *Surviving the Storm - Building Codes, Compliance and the Mitigation of Hurricane Damage* ("Surviving" 1989), published by the All-Industry Research Advisory Council, all provide guidelines for construction to reduce hurricane damage and losses.

The information contained in such documents embody basic tenets of hurricane resistant construction, such as the importance of an adequate and continuous load path, and strategies to maintain the integrity of the building envelope to minimize the loss of property. Although an exhaustive account of such recommendations is beyond the scope of this thesis, some example are given below of typical details to provide a continuous load path.

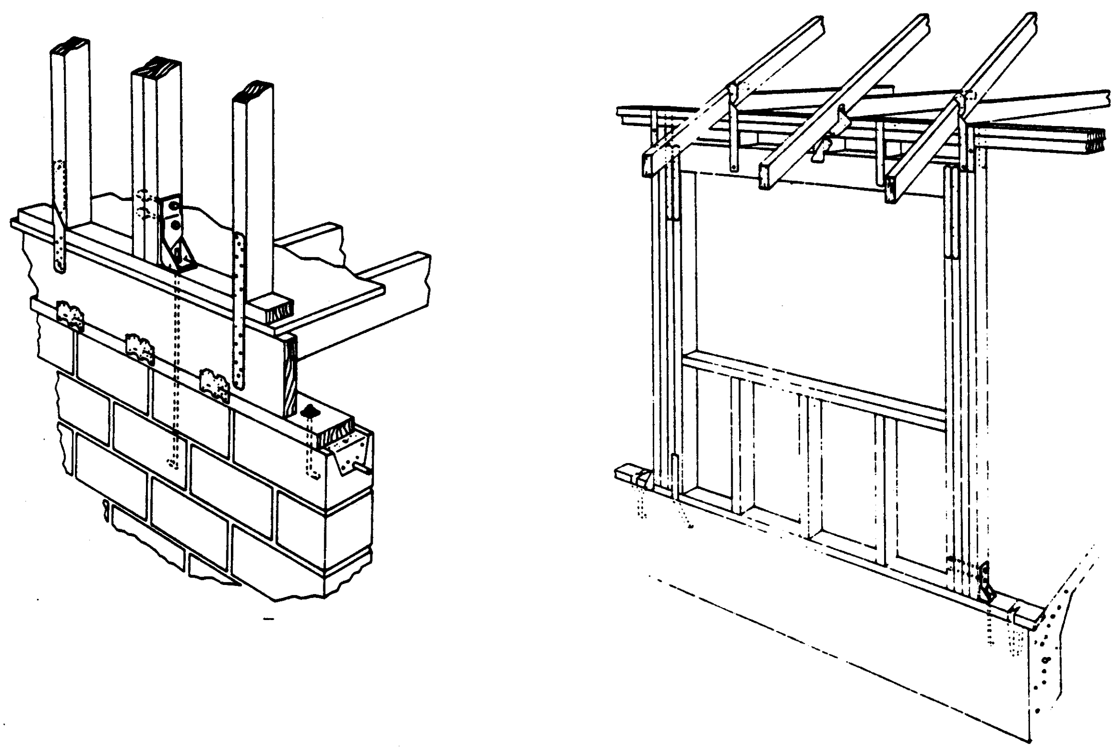


Figure 3.5
Typical wind-resistant details for residential wood light-frame construction (from "Standard" 1993)

3.4 Current Research on Wind Loads on Low-Rise Buildings

Active research hopes to improve methods to estimate actual storm speeds, from which more reliable predictions of design loads should be possible (e.g. Ho et al. 1992; Davenport 1983). Other work is devoted to developing more sophisticated loss estimation models for the purposes of risk allocation and predicting losses to insured property (Stubbs and Boissonnade 1993; Boissonade and Dong 1993).

Much experimental work on wind loads on low-rise structures has been conducted at the Boundary Layer Wind Tunnel Laboratory at the University of Western Ontario (Surry 1990). Retrofitting techniques that might improve roof cladding and sheathing performance are currently under investigation by researchers at Clemson University (e.g. Sutt et al. 1996) as well as work to better characterize the system behavior of uplift-resistant connections (Zaitz 1994). Other researchers (Lin and Surry 1992) suggest that modifications to roof geometry could improve the aerodynamics of low-rise roof systems and hence improve building performance.

Related research examines how newly acquired knowledge of wind loads can be effectively translated into practice. For example, Gaus et al. (1993) argue the effective reduction of hurricane losses is dependent on housing economics and public education and that practical retrofit strategies must developed with these issues mind. For these reasons Gaus et al. advocate a number of simple and relatively inexpensive retrofit methods, such as augmenting the sheathing-rafter connections in the attic interiors of existing homes with beads of construction adhesive. This type of research, i.e. the consideration of technical solutions in light of real world constraints, is an important field of study because it provides a necessary link between fundamental investigations and public policy.

The nature of wind loads due to extreme events and the simplified structural response of low-rise buildings to such loads was briefly presented in this chapter. The importance of a continuous load path to resist uplift loads was described as well as current research that emphasizes estimation of actual wind loads, statistical vulnerability assessments and the translation of this type of knowledge into practice. Although the response of WLF buildings to wind loads is fairly well understood qualitatively, more rigorous analytical techniques are being developed to evaluate the expected performance of wood light-frame structural systems. Traditional methods of reliability analysis and their application to structural systems are the subject of the next chapter.

CHAPTER FOUR

We must balance the needs of exactness and simplicity, and reduce complexity without oversimplification in order to match the level of detail at each step of the problem we face.

R.E. Bellman and M. Grentz
"Limitations in Decision Making and System Performance"
Information Sciences

Modeling Structural System Performance

Performance describes the behavior of a system and is closely associated with *reliability*. In engineering applications, a formal theory has evolved based on failure physics and the theories of probability and statistics to describe reliability of the component or system in question. Such models are simplified mathematical constructs that approximate reality for the study of failure mechanisms or making predictions (Dai and Wang 1992).

This chapter provides a review of some well-established methodologies to evaluate system safety and performance and the application of such methods to structural systems. General reliability characterizations and block diagram topology are discussed in some detail since these constructs will be employed in subsequent chapters. Traditional structural reliability theory is described and illustrated with an example of a simple structural system. The chapter concludes with a description of possibility theory, fuzzy uncertainty analysis, and a review of applications of fuzzy sets and logic reported in the literature.

In the following sections the terms "component" and "system" are defined as generic elements and collections of elements that might appear in engineering applications where an evaluation or prediction of performance is of interest.

4.1 Uncertainty in Structural Engineering Models

Uncertainty in engineering analysis can be divided into system uncertainty and parameter uncertainty. Structural engineering problems contain elements of both types of uncertainty, which can be further divided into four categories: (1) the analytical model, (2) the loads applied to the system, (3) the ability of the system to resist loads, and (4) the response of the structure due to the combination of these three (Wadia-Fascetti and Smith 1994).

Mathematical descriptions of the behavior of actual systems are typically based on analytical models, empirical models, or some combination of the two. Analytical models are deterministic mathematical descriptions based on mechanics and often require assumptions to make the model computationally tractable. Empirical models are descriptions based on historical data of past behavior. But these models are descriptions of particular systems and hence, represent only systems similar to the one used to develop the model. To overcome the limitations of each approach, analytical and empirical models are often combined.

Three primary ways to treat uncertainty in decision making are statistics, multi-subjective probability, and fuzzy sets and measures (Lind 1985). The relationship between these methods is schematically depicted in Figure 4.1. The application of these methods to engineering problems has been studied extensively (e.g. Lewis 1987; Dai and Wang 1992).

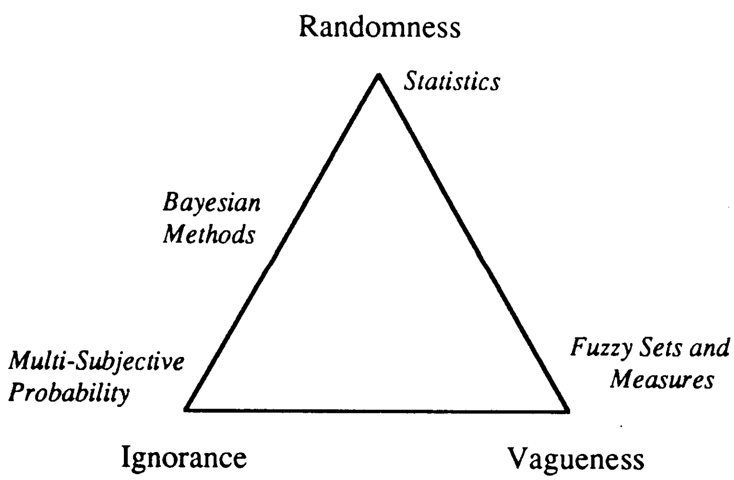


Figure 4.1
Symbolic elements of uncertainty in engineering decision problems (from Lind 1985)

In structural engineering analysis and design, methodologies that address uncertainty draw extensively from the theories of probability and statistics to characterized the stochastic nature of structural loads and resistances. But as will be illustrated in this chapter, such methods are not always the most appropriate in circumstances where parameter descriptions are vague and mixed with ignorance or lack of data. Nonetheless, the probability theory is well established and currently the most widely accepted methodology to treat uncertainty in engineering applications and hence warrants discussion.

4.2 Probabilistic Techniques of Reliability Analysis

Reliability engineering pervades many disciplines in both its body of knowledge and applicability to the analysis of random failures and the development of strategies to minimize the probability of their occurrence (Lewis 1987). As the theory of probability is central to many methods of reliability analyses, the prevailing probabilistic interpretation of reliability is presented below. A simple two variable formulation illustrates the fundamental concepts attendant to the theory.

4.2.1 Probabilistic Design Methodology

Probabilistic design methodology accounts for the stochastic nature of loads, material and geometric properties. The fundamental probabilistic model of the reliability of a system can be expressed as

$$\begin{aligned} R &= P(\text{strength}(r) \geq \text{stress}(s)) \\ &= P(g(r,s) \geq 0) \end{aligned} \tag{4.1}$$

where R is the reliability, $P(\bullet)$ is probability, r is a vector of design parameters affecting the design strength (load resistance), s is a vector of the design parameters affecting the applied stress (load effect), and $g(\bullet)$ is a limit state function defined below. A limit state function is a mathematical expression of This strength-stress inference model is graphically depicted in Figure 4.2.

The nature of inference region is described by the convolution integral

$$F = \int_{-\infty}^{\infty} f_s(s)\left[\int_{-\infty}^{s} f_r(r)dr\right]ds = \int_{-\infty}^{\infty} f^*(s)ds\,, \tag{4.2}$$

where F corresponds to the failure probability and $f^*(s)$ is the so-called failure function. F is by definition the cumulative distribution function of $f^*(s)$.

The limit state function $g(\bullet)$ is a mathematical description of the boundary between safe and failed regions described as:

$$\begin{aligned} g(r,s) &< 0 && \text{failed state} \\ g(r,s) &= 0 && \text{limit state surface.} \\ g(r,s) &> 0 && \text{safe state} \end{aligned} \tag{4.3}$$

The probability of failure is then defined as

$$P_f = P\{g(r,s) < 0\}, \tag{4.4}$$

which can also be stated as $P_f = 1 - R$ since the reliability R is defined as the probability the system *will not* fail.

The limit state function can be generalized to n random variable design parameters

$$g(X_1, X_2, X_3, \cdots, X_n) = 0 \tag{4.5}$$

and the corresponding generalized probability of failure is written as

$$P_f = \int_{\Omega} f_X(x_1, x_2, x_3, \cdots, x_n) dx_1, dx_2, dx_3, \cdots, dx_n \tag{4.6}$$

where $f_X(x_1, x_2, x_3, \cdots, x_n)$ is the joint probability density function of X and Ω is the domain of the failure surface represented by the limit state.

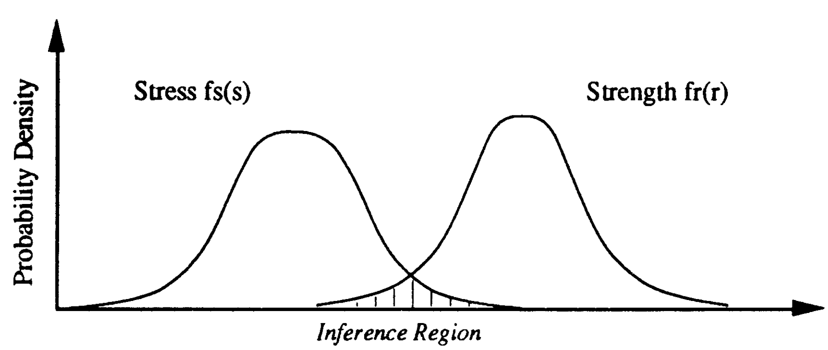


Figure 4.2
Stress and strength distributions and inference region (adapted from Dai and Wang 1992)

4.2.2 Reliability Analysis in Engineering

As mentioned previously, reliability connotes the likelihood that a system will not fail and can be formally defined as *the probability that a system will perform properly for a specified period of time under a given set of operating conditions* (Lewis 1987).

Classical reliability theory has its roots in the study of the failure behavior of electronic components and systems. The classical formulation defines reliability in terms of the probability density function (PDF) for the time to failure of a component or system. Mathematically, the probability that failure takes place on the interval between time t and $t+\Delta t$ is

$$f(t)\Delta t = P(t \le \mathbf{t} \le t + \Delta t) \tag{4.7}$$

where $\mathbf{t}$ is a random variable. The cumulative distribution function (CDF), which defines the probability that failure takes place at a time less than or equal to t is then expressed as

$$F(t) = P(\mathbf{t} \le t). \tag{4.8}$$

Reliability, or the probability that a component or system operates without failure for a length of time t is then defined as

$$R(t) = P(\mathbf{t} > \mathrm{t}) \tag{4.9}$$

from which it follows

$$R(t) = 1 - F(t) = 1 - \int_0^t f(t')dt'. \tag{4.10}$$

Consider a system (collection of components) subject to M independent events associated with the failure of different components or different failure mechanisms for the same component. Let X_i represent the event that the ith failure mode *does not* occur before time t. The reliability of such a system is then

$$R(t) = P(X_1 \cap \mathrm{X}_2 \cap \cdots \cap X_M). \tag{4.11}$$

Alternately, if F_i denotes the (time-independent) event that element i fails, the probability of failure P_f is

$$P_f = P(F_1 \cup F_2 \cup F_3 \cup \cdots \cup F_n). \tag{4.12}$$

For independent failure modes, the reliability of the system is

$$R(t) = P(X_1)P(X_2)\cdots P(X_M), \tag{4.13}$$

where the *mode reliability* is defined as

$$R_i(t) = P(X_i). \tag{4.14}$$

It then follow that the system reliability is

$$R(t) = \prod_i R_i(t). \tag{4.15}$$

The expression in (4.15) states mathematically the *series* model (or weakest-link model) system where system failure is defined as the failure of any one component. In principal a statically determinate structure can be considered a series system. Assuming independence among component failures and letting the reliability of the ith component to be denoted by R_i, it can be shown that (Lewis 1987) the reliability of a series system composed of n elements is

$$R_s = \prod_{i=1}^{n} R_i \tag{4.16}$$

where the time-dependence is neglected for simplicity.

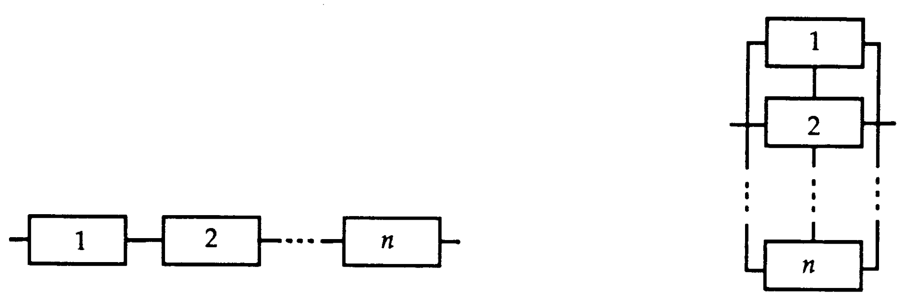
The *parallel* system is the second fundamental case whereby failure of *all* components must occur for the system to fail. If the reliability of the ith component is denoted by R_i, it can be shown that (Lewis 1987) the reliability of a parallel system of n elements is

$$R_s = 1 - \prod_{i=1}^{n} \left(1 - R_i\right). \tag{4.17}$$

The probability of failure P_f of this case can also be expressed in the context of the failure event F_i of element i as

$$P_f = P(F_1 \cap F_2 \cap F_3 \cap \cdots \cap F_n). \tag{4.18}$$

The configurations that correspond to the algebraic expressions in (4.16) and (4.17) can be represented graphically by block diagrams. The block diagram topology is commonly used in many engineering disciplines for its clarity of expression. Schematic representations of pure series and parallel systems are provided below:



$$R_s = \prod_{i=1}^{n} R_i$$

$$R_s = 1 - \prod_{i=1}^{n} (1 - R_i)$$

(a) **(b)**

Figure 4.3
Fundamental system configurations. Pure series system (a), and pure parallel system (b)

Although the preceding discussion illustrates some of the central tenets of reliability theory, the methods described can only be applied to the simplest systems. More advanced methodologies exist, such as Failure Mode and Effect Analysis (FMEA), and fault tree analysis (FTA), to analyze the safety and performance of complex systems. A detailed discussion of these methodologies is beyond the scope of this thesis, although such methods can been applied to evaluate the performance of structural systems.

4.2.3 Structural Systems Reliability Theory

Structural reliability theory is concerned with the rational treatment of uncertainties in structural engineering design. Broadly defined, the reliability of a *structure* is its ability to fulfill its design purpose for some specified time; narrowly speaking, it is the probability that a structure will not attain each specified limit state (ultimate or serviceability) during a specified reference period (Thoft-Christensen and Baker 1982).

The theory of structural reliability grew primarily out of a need for better methods of assessing risk to satisfy increased safety requirements of highly complex facilities such as offshore structures and nuclear power plants. To this end much theoretical work has been devoted to the development of stochastic models to describe loads due to extreme events (e.g. gust winds, waves and seismic events) as well as stochastic models for material strength. Formal methods exist to model structural loads that account for variations in time, duration and intensity (e.g. Madsen et al. 1986; Wen 1990).

Structural systems reliability theory makes use of such load and resistance models with varying degrees of complexity. A number of methods have emerged and are classified by the extent to which they use available information about the structural problem. Level I methods use specific loads and resistance factors (e.g. LRFD design formats); level II methods consider random variables to be represented by the means and variances and are used primarily to calibrate designs to satisfy "target" levels of reliability. Level III methods require the joint distribution of the uncertain parameters to calculate a probability of failure as a measure of reliability (Madsen et al. 1986). Since the majority of structural reliability studies makes use of level II methods, the following section describes such methods by examining a simple structure.

Consider a simple pin-connected planer frame structure:

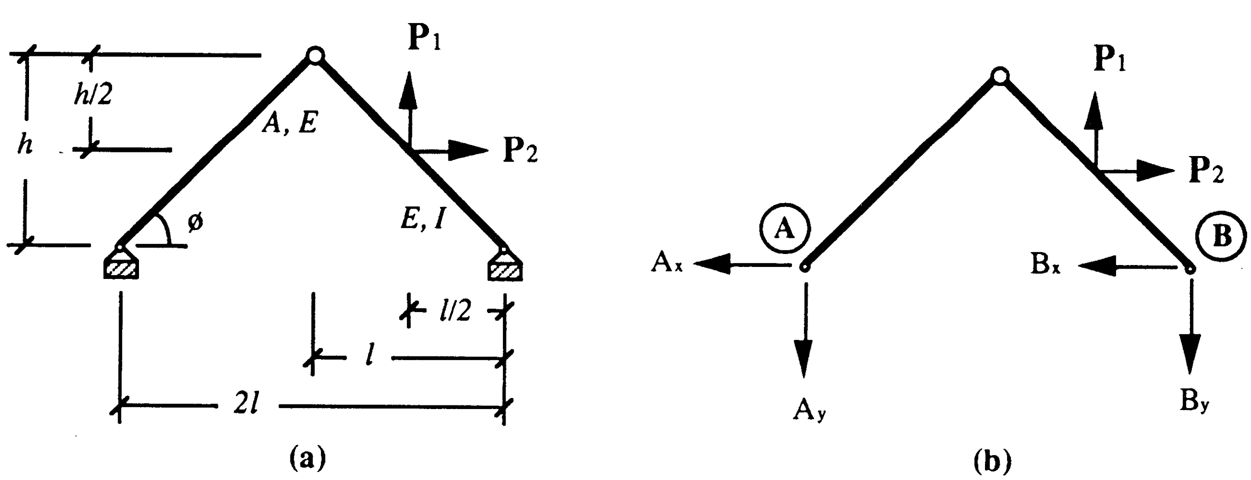


Figure 4.4
Simple planar pin-connected frame (a) and overall free-body diagram (b)

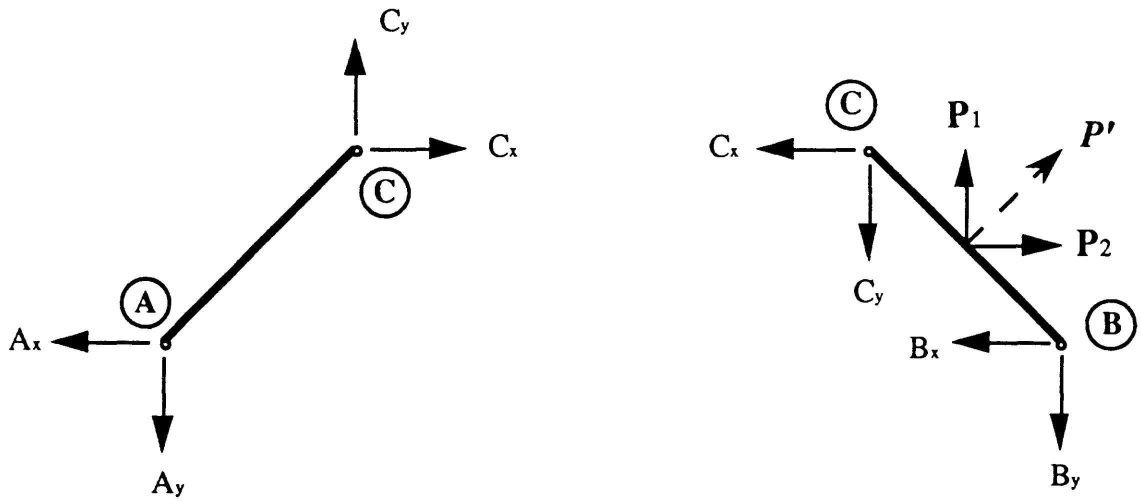


Figure 4.5
Free-body diagrams of members AC and BC

From the equations of static equilibrium, the reactions are

$$A_x = \frac{P_1 l + P_2 h}{4h} \qquad A_y = \frac{P_1 l + P_2 h}{4l}$$
$$B_x = \frac{3P_2 h - P l_1}{4h} \qquad B_y = \frac{3P_1 l - P_2 h}{4l} \qquad (4.19a) - (4.19d)$$

If, for simplicity we assume $P_1 = P_2 = P$, the reactions are:

$$A_x = \frac{P(l+h)}{4h} \qquad A_y = \frac{P(l+h)}{4l}$$
$$B_x = \frac{P(3h-l)}{4h} \qquad B_y = \frac{P(3l-h)}{4l} \qquad (4.20a) - (4.20d)$$

The possible ways in which the structure can fail must first be identified to assess the expected performance of the system. Broadly speaking, either the members or the connections can fail. The nature of, and relationship between, the critical performance factors of the frame with respect to the loads applied can be represented in the form of an event tree:

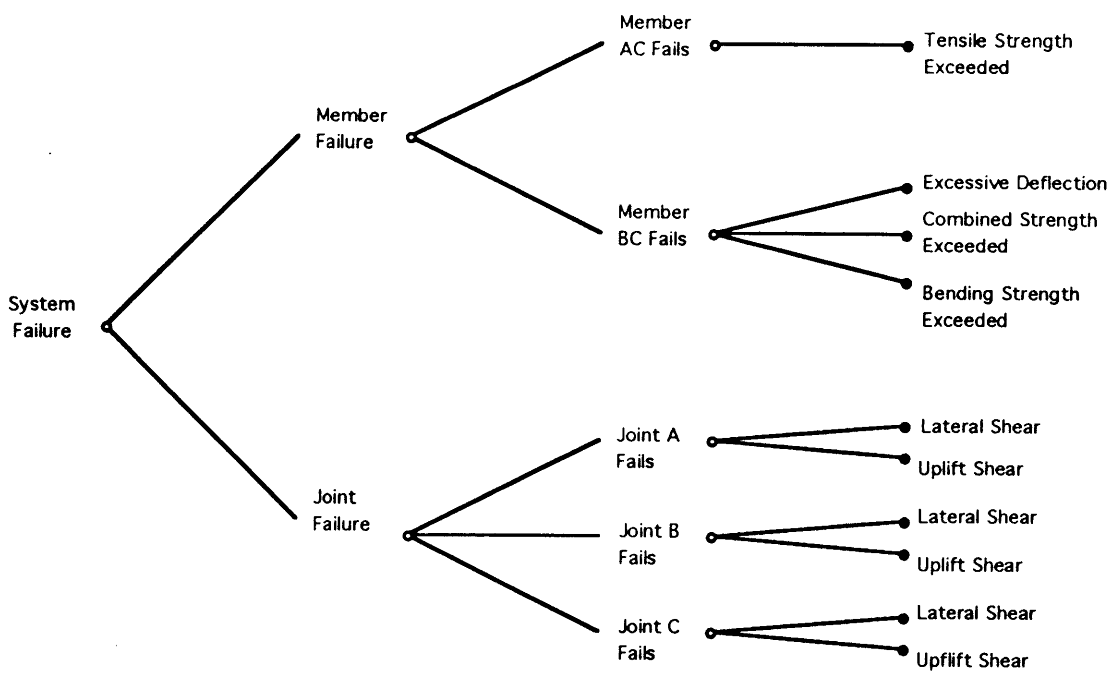


Figure 4.6
Event tree representation of failure events for simple pin-connected frame

Alternately, the failure events depicted in the event tree can be combined into an equivalent Boolean expression for the top "failure event"

$$\begin{aligned} P(F) &= P\left(F_m \cup F_j\right) \\ &= \underbrace{P\left(F^t_{AC} \cup F^d_{BC} \cup F^b_{BC} \cup F^c_{BC}\right)}_{\text{member failure}} \cup \underbrace{P\left(F^{ls}_A \cup F^u_A \cup F^{ls}_B \cup F^u_B \cup F^{ls}_C \cup F^u_C\right)}_{\text{joint failure}} \end{aligned} \tag{4.21}$$

where

F = system failure event

F^t_{AC} = tensile strength of member AC exceeded

F^d_{BC} = excessive deflection of member BC

F^b_{BC} = bending strength of member BC exceeded

F^c_{BC} = combined (axial and bending) strength of member BC exceeded

$F^{ls}_{A,B,C}$ = lateral shear capacity of joint exceeded

$F^u_{A,B,C}$ = uplift resistance of joint exceeded

As described earlier, level II methods express all quantities relevant to the reliability problem solely in terms of expected values and covariances of the basic structural parameters. The methods require a finite number of basic parameters, usually denoted Z_i such as loading, strength, geometrical, statistical and model uncertainty variables.

A failure function $g(z_i)$ of the type defined in (4.3) divides z-space into the safe set S and the failed set F. This can be expressed mathematically as

$$\begin{array}{ll} g(z_i) > 0 & z_i \in S \\ g(z_i) = 0 & z_i \in L_Z, \\ g(z_i) < 0 & z_i \in F \end{array} \tag{4.22}$$

where the boundary L_Z is referred to as the limit state surface. An inference variable can be obtained by replacing the parameters z_i in the failure function with the corresponding random variables Z_i. Let a "safety margin" M be an inference variable such that

$$M = g(Z_i). \tag{4.23}$$

The quantity of interest is the probability that M is equal to or less than zero,

$$P_f = P(M \leq 0). \tag{4.24}$$

A number of non dimensional indices based on () have been developed to measure structural reliability. Traditionally, structural reliability has been defined in terms of a reliability (or safety) index β. In two-dimensional space β is a measure of the distance from the location of the expectation of the safety margin to the limit state surface. Cornell (1969) defined a reliability index as

$$\beta_C = \frac{E[M]}{D[M]} \tag{4.25}$$

where

$E[M]$ = expected value of the safety margin M

$D[M]$ = uncertainty scale parameter

This is represented graphically below:

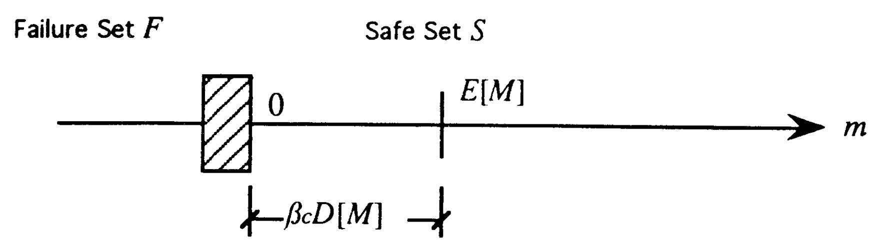


Figure 4.7
Graphical definition of reliability index for two parameter case

Cornell considered the fundamental failure function of the difference between resistance and load effect

$$g(r,s) = r - s \tag{4.26}$$

and defined the corresponding safety margin as

$$M = R - S \tag{4.27}$$

If the resistance and load effect are independent random variables, the reliability index is

$$\beta_c = \frac{E[R] - E[S]}{\sqrt{\mathrm{Var}[R] - \mathrm{Var}[S]}} = \frac{\mu_R - \mu_S}{\sqrt{\sigma_R^2 + \sigma_S^2}}. \tag{4.28}$$

If means and variances of the load and strength random variables were available, the reliability of the frame structure could be calculated according to (4.28).

The above application of the first-order second moment (FOSM) reliability method to a planar pin-connected frame is intended to illustrate the most common formulation of classical structural reliability theory. Application to more complex structures requires a much greater degree of mathematical sophistication and extensive probabilistic data. Further, structural systems reliability theory in its present form often requires very restrictive mechanical idealizations if one wishes to consider events beyond first element failure (Ditlevesen and Bjerager 1986). A less obvious (but more restrictive) limitation of classical structural reliability is the inability to address sources of uncertainty that are vague or imprecise (e.g. Blockley 1980; Ayyub 1991).

4.3 Wood Light-Frame Structural Reliability

Structural systems reliability theory has been applied to light-frame wood construction primarily in response to the wood industry's efforts to develop a load and resistance factor design (LRFD) specifications for both nonengineered light frame and engineered wood construction. Such efforts have been motivated primarily by the desire to better quantify the uncertainties in the overload capacity of repetitive member assemblies. However, it is evident that strict application of the many classical methods of structural reliability to WLF systems is problematic. It has been stated that such methods require either assumptions that oversimplify system behavior, the use of nonexistent probabilistic data, or lead to computationally intractable models (Bulleit et al. 1993).

Application of structural reliability theory to wood light-frame members and systems for code development purposes has been reported by a number of researchers (e.g. Rosowsky and Fridley 1993; Philpot et al. 1995; Bulleit 1985). In addition, Maamouri (1988) used the Monte Carlo method to determine the reliability of individual six-on-twelve wood Fink trusses as a function of member and connection strength variabilities. More general reliability studies have accounted for duration of load effects in wood members (Rosowsky et al. 1994) as well as load sharing mechanismss of wood light-frame floor assemblies (Rosowsky and Ellingwood 1991). In the latter study, system factors were developed from a reliability model that considered both duration of load and load sharing among members.

Results of such studies are used in the development of safety factors for reliability-based design of wood structures. Based on the concepts described in Subsection 4.2.1, design equations for wood systems have the form ("Load" 1986)

$$\phi R_n \geq \gamma_i Q_{ni}, \tag{4.29}$$

where R_n and Q_n are nominal code specified resistance and load, respectively, ϕ is a resistance factor and γ_i is a load factor. Research supporting the trend towards reliability-based codified design is very important for at least two reasons: first, reliability analyses are needed to verify factor values for use in LRFD design formats, which are design methodologies perceived to be both more rational and easier to use than traditional allowable stress design (ASD); and second, the development of systems models provides much needed insight into the system behavior of WLF structural systems.

As pointed out by Bulliet et al. (1993), characterization of wood systems and system behavior, definition of limit states, and the development of simplified models for the purposes of reliability analysis are prerequisites for effective application of existing reliability methods to WLFC. However, there does seem to be a paucity of analytical methods for the purposes of performance evaluation of as-built WLFC in the wood systems reliability literature. Other methodologies which do exist and have been applied to WLFC, are described in the next section.

4.4 Macroscopic System Performance Approach

Hybrid methods that combine both analytical and empirically-based models can be developed to assess the safety and reliability of existing structures (e.g. Yao 1985). This is often done so that distinctly different types of information such as natural hazard risk, expected structural response, and financial risks associated with the costs of failure, can be combined for the purposes of decision making. A recently conducted cost-benefit analysis of a new building code for windstorm resistant construction along the gulf coast of Texas was based on such a procedure. The methodology underlying this vulnerability assessment study is described in more detail below.

Stubbs et al. (1995) considered the implementation costs (Lombard et al. 1995), wind speed risk (Perry and Stubbs 1995) and the overall cost effectiveness associated with a proposed prescriptive building code developed by the Texas Department of Insurance. Hierarchical structures of expert systems were used to determine the building code break-even cost, defined as *the present value of the additional cost to implement the code when the reduction in losses resulting from the use of the new code balances the cost needed to implement the new code* (Stubbs et al. 1995). The objective of the study was to produce estimates of the expected damage ratio of a generic building as a function of wind speed. The calculations were based on a structural damageability algorithm that evaluates the vulnerability of a structure based on the damageability of the individual components.

As noted by Stubbs et al. (1995), the component damageability formulation is analogous to a failure mode in the structural reliability theory. A component damageability curve is defined by a linear relationship between the damage ratio and wind speed and resembles the form of an indicator variable. For a given wind speed, the damage ratio takes on values in the continuous interval [0,1] up to a threshold wind speed, where the ratio attains unity. Such a relationship is given by

$$DR_i(v) = \begin{cases} 0 \\ \dfrac{v - a_{i1}}{a_{i2} - a_{i1}}, \\ 1 \end{cases} \tag{4.30}$$

such that

$$\begin{aligned} v &\le a_{i1} \\ a_{i1} < v &\le a_{i2} \\ v &> a_{i2} \end{aligned} \tag{4.31}$$

where

$DR_i(v)$ = damage ratio for the ith damage mode

v = wind speed

a_{i1} = expert - supplied lower damage bound

a_{i2} = expert - supplied upper damage bound

The damage ratio for an entire structure is a weighted sum of the component damage ratios over all n failure modes

$$DR_s(v) = \frac{\sum_{i=1}^{n} I_i \, DR_i(v)}{\sum_{i=1}^{n} I_i}, \tag{4.32}$$

where

$DR_s(v)$ = the damage ratio of the entire structure

I_i = expert - supplied relative importance of the ith failure mode

Similar models were developed to predict the damageability of the contents of the structure. The study concluded that the implementation of increased building code provisions along the Texas gulf coast would be cost effective.

Although the study covered many facets of structural and nonstructural vulnerability to hurricane damage, some specific data presented are of particular interest to this thesis. Based on aggregate expert opinion, Stubbs et al. (1995) report damageability parameters of the roof system components. These are summarized below for the purposes of illustration.

Table 4.1

Roof Component Threshold Resistances For Texas Cost-Effectiveness Study
(adopted from Stubbs et al. 1995)

Damage Mode	Resistance Thresholds[1,2]			Resistance Thresholds[1,3]		
	Low		*High*	*Low*		*High*
Roof Decking Damage	80	-	120	94	-	142
Roof Framing Damage	80	-	120	88	-	132
Roof-Wall Anchorage Damage via Suction	90	-	120	113	-	150
Roof-wall Anchorage Damage via Suction & Int. Pressure	80	-	100	96	-	120

[1] in miles per hour fastest mile; exposure category C
[2] resistances for current inland code
[3] resistances for new inland code

The weighted-average-component algorithm is one accepted way to take advantage of expert opinion or disparate but related data in the characterization of structural system performance (Pandey and Barai 1995). The general form of (4.32) and assessments of wind speed-performance relationships described in Table 4.1 will serve as a benchmark for a similar type of analysis according to a proposed methodology described in Chapter Six.

4.5 Fuzzy Set Representation and Possibility Theory

Reliability has traditionally been defined in a probabilistic context although alternate ways to describe uncertainty have appeared in the literature. It is evident that probabilistic methods are limited by the large amount of data required to establish parameter values, and the sometimes sophisticated mathematics used to describe the probabilistic models. If neither sufficient statistical data nor suitably accurate models of the system are available, can uncertainties of the type described earlier be addressed?

The theories of fuzzy sets, fuzzy logic and possibility theory are potentially useful tools to address these uncertainties and these methods have attracted increasing amounts of attention in the structural engineering research community. It will be illustrated later in this thesis that the nature of vaguely defined or unique events, the testing of hypotheses, and the nature of subjectively estimating likelihoods associated the expected performance of WLF systems make fuzzy sets a powerful tool that augments traditional probability and statistics methods. This section describes the basics of fuzzy sets and logic and how they can be applied to uncertainties associated with structural engineering models. However, since the vocabulary and calculus of fuzzy sets are not yet widely used in the structural engineering community, the reader is referred to more detailed discussions (Bardossy and Duckstein 1995; Brown and Yao 1983) of fuzzy sets and systems.

4.5.1 Fuzzy Logic and Possibility Theory

The concept of the fuzzy set was proposed by Lotfi Zadeh in 1965 (Zadeh 1965) and possibility theory was introduced as a field of study by Zadeh in 1978 (Zadeh 1978). The theory of possibility, which is very much related to the fuzzy logic and the theory of fuzzy sets, is suitable for application to problems with limited or subjective information and where approximate but nonetheless accurate results are of interest. In contrast to the theory of probability, possibility theory is well suited to the analyses of unique situations. As mentioned previously, the theory of probability is primarily intended to characterize the outcomes of repeatable situations. An extensive philosophical and mathematical debate surrounds the relationship between the possibility and probability theories (e.g. Blockley 1980; Andersson 1988) and a more thorough account of this debate is beyond the scope of this thesis. However, a brief contrast should be made to clarify the fundamental concepts of fuzzy sets and possibility theory.

The contemporary school of thought assigns probability two primary interpretations; a *frequentist* interpretation related to the outcomes of repeatable experiments (e.g. the rolls of a die) and a more *subjective* "degree of belief" interpretation that represents an expression of an individual's measure of the relative likelihood of an outcome (e.g. a forty percent chance of rain). It has been argued that the probability of unique events is really a "fuzzy" measure (Lind 1985) and that probability theory should be used to estimate the chances of some event occurring but not to estimate the degree of belief in the truth of some hypothesis or theory (Blockley 1980). Further, while both Bayesian (subjective) probabilities and fuzzy sets quantify degrees of belief, there is an important distinction. In the Bayesian approach, probability measures the likelihood that an event is a member of a set, whereas fuzzy quantities express the degree of membership to a set.

In addition to the debate regarding subjective probabilities as an expression of a degree of belief regarding the outcome of unique events, there is a question of either theory being appropriate for a particular problem. As Andersson (1988) pointed out, "For systems in which man and human actions have a significant effect on the reliability properties of the system, i.e. for humanistic systems, traditional methods have so far proved insufficient. The reason for this is that these methods are intended, and have been produced, for the analysis of mechanical systems ... for humanistic systems, with their inherent limited and fuzzy information content, it has been found that the use of fuzzy sets may be appropriate."

A central element of fuzzy logic and possibility theory is the fuzzy set, which is a quantity defined by a membership function along a discrete or continuous interval. The membership function in fuzzy set theory is a measure of belonging to an imprecisely defined set, a concept perhaps best illustrated by means of an example, such as the one presented in the next section.

4.5.2 Fuzzy Sets and Membership Functions

The aspects of fuzzy sets relevant to this thesis are perhaps best described by example. Consider the example provided by Brown and Yao (1983). Let U represent a universe of consideration of suspension bridges comprised of two sets, well-constructed bridges D and less well constructed bridges E. It follows that

$$U = D + E. \tag{4.33}$$

Intuitively it seems the boundaries between D and E are not well defined, for there inevitably exist suspension bridges with varying degrees of construction quality which may be determined by professional judgment. Classical (Boolean) set theory, which is the basis for probability theory, requires a "crisp" division between well constructed and not so well constructed bridges and thus would provide a measure of the likelihood of a given bridge being a member of either D or E.

Now consider a continuum of suspension bridge construction quality, from absolutely poor to exceptional, to reside along the interval [0,1]. For simplicity, assume the range is divided into discrete intervals of 0.1, where 1 signifies clear membership to well constructed bridges and 0 signifies clearly not belonging to the set of well constructed bridges. According to the above logic, the universe of discourse U can be defined as

$$\begin{aligned} U = 1|1 \ + \ 1|0.9 \ + \ 1|0.8 \ + \ 1|0.7 \ + \ 1|0.6 \ + \\ + \ 1|0.5 \ + \ 1|0.4 \ + \ 1|0.3 \ + \ 1|0.2 \ + \ 1|0.1 \ + \ 1|0 \end{aligned} \tag{4.34}$$

or

$$U = \sum_{i=1}^{n} 1|x_i \tag{4.35}$$

where the " | " is referred to as a *delimiter* and acts to pair the first membership with the second value are discrete values x_i along the so-called universe of discourse U.

Fuzzy sets are defined along the universe of discourse. For example, "strong" support that a particular bridge is well constructed might be represented as

$$\text{strong} = 1|1 \; + \; 0.7|0.9 \; + \; 0.4|0.8 \; + \; 0.1|0.7 \qquad (4.36)$$

and "weak" might be defined as

$$\text{weak} = 0.1|0.3 \; + \; 0.4|0.2 \; + \; 0.7|0.1 \; + \; 1|0, \qquad (4.37)$$

where zero membership exists for all other x_i not listed in (4.36) and (4.37). This information can also be expressed in tabular form (below) or graphically as in Figure 4.5.1.

Table 4.2

Degrees of Membership for "Strong" and "Weak" Construction Quality
(adapted from Brown and Yao 1983)

Level of Construction Quality	Strong	Weak
1	1	0
0.9	0.7	0
0.8	0.4	0
0.7	0.1	0
0.6	0	0
0.5	0	0
0.4	0	0
0.3	0	0.1
0.2	0	0.4
0.1	0	0.7
0	0	1

It should be noted that fuzzy set theory can be considered a generalization of classical "crisp" set theory (Brown and Yao 1983; Bardossy and Duckstein 1995). To illustrate, consider the discrete fuzzy set F defined as

$$F = \sum_{i=1}^{n} \mu_i | x_i \qquad 0 \le \mu_i \le 1. \qquad (4.38)$$

This fuzzy set becomes a crisp set if there is clear and unambiguous support of x_1,

$$F = 1|x_1 + \sum_{i=2}^{n} 0|x_i. \tag{4.39}$$

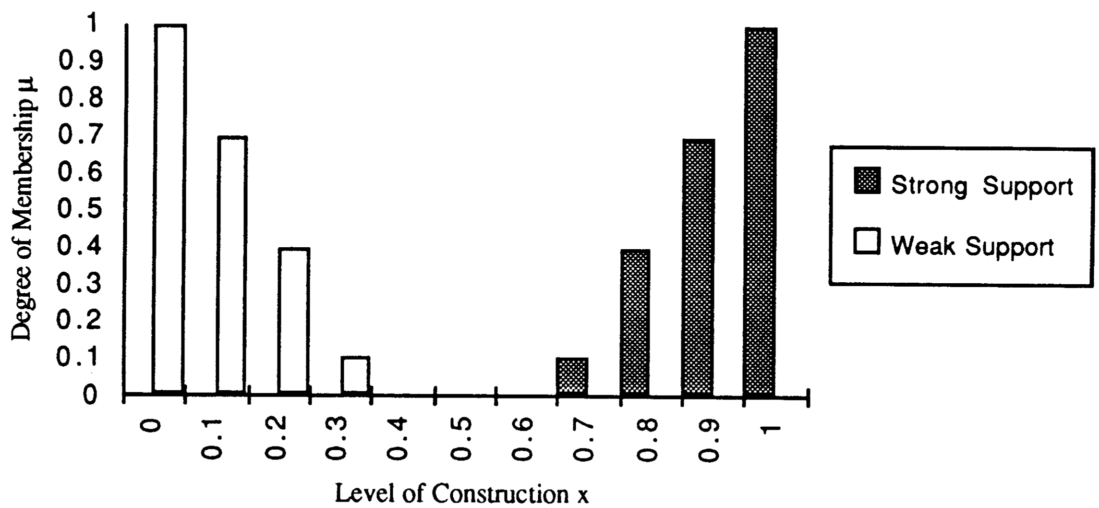


Figure 4.8
Graphical representation of the fuzzy sets "weak" and "strong" support for level of construction quality (adopted from Brown and Yao 1983)

It should also be pointed out that the sets corresponding to "weak" and "strong" as defined in (4.37) and (4.36) are special cases of fuzzy sets known as *fuzzy numbers*. If the fuzzy universe of discourse U is transformed from its discrete form given by (4.35) to the continuous form

$$U = \int_u 1|x, \tag{4.40}$$

the triangular shapes of the graphs of the degrees of support depicted in Figure 4.5.1 would approach so-called *triangular fuzzy numbers*. Triangular fuzzy numbers are the simplest fuzzy numbers because the membership function consists of an increasing and decreasing linear function that forms a triangle. The membership function of a triangular fuzzy number $\tilde{A}$ is formally defined as (Bardossy and Duckstein 1995)

$$
\mu_A(x) = \begin{cases} 0 & \text{if } x \le a_1 \\ \dfrac{x - a_1}{a_2 - a_1} & \text{if } a_1 < x \le a_2 \\ \dfrac{a_3 - x}{a_3 - a_2} & \text{if } a_2 < x \le a_3 \\ 0 & \text{if } a_3 < x \end{cases} \tag{4.41}
$$

where the $\tilde{}$ is used to dennote a fuzzy quantity and $a_1 \le a_2 \le a_3$. Note that $\tilde{A}$ reduces to a crisp value A_c if $a_1 = a_2 = a_3 = a_c$. The fuzzy number $\tilde{A}$ defined by (4.41) is sometimes denoted $\tilde{A} = (a_1, a_2, a_3)_T$ and referred to as a Triangular Fuzzy Number (TFN). This notation is less cumbersome than the form of (4.38) and is used in the discussion of fuzzy numbers in subsequent sections of this thesis. The fuzzy number $\tilde{A}$ is schematically depicted in Figure 4.5.2.

Fuzzy sets and numbers can be used to quantitatively express linguistic assessments and heuristic knowledge in a formal and consistent manner. The bridge example provided by Brown and Yao (1983) presented in this section illustrates how fuzzy set theory can be used to translate verbal expressions of support for membership when the data are subjective and the boundaries between sets are imprecisely defined. This section and Appendix A provide only a brief introduction to fuzzy sets and possibility theory and only concepts strictly relevant to this thesis. For more thorough accounts of fuzzy set theory and some applications to civil engineering problems, see Blockley (1980), Brown and Yao. (1983), Dong (1986) and Ayyub (1991).

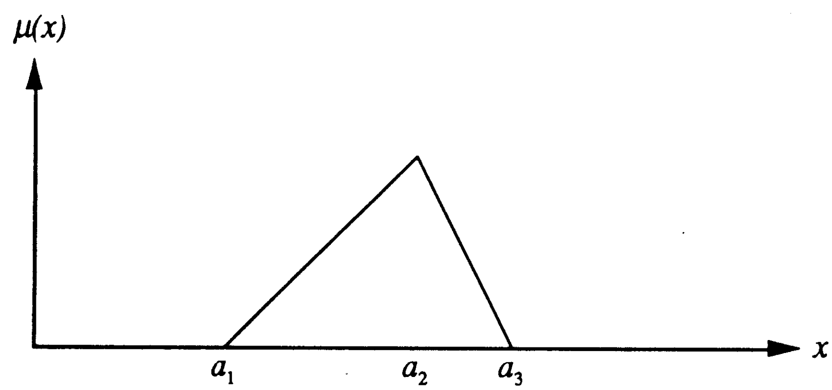


Figure 4.9

Membership function of the triangular fuzzy number (TFN) $\tilde{A} = (a_1, a_2, a_3)_T$

4.5.3 Applications of Fuzzy Sets in Structural Engineering

Fuzzy sets have been applied to a wide variety of structural engineering problems, from the quantification of dynamic uncertainties throughout the lifetime of a structure (Wadia-Fascetti and Smith 1994) to the assessment and classification of structural failures (Blockley 1980) to the selection of safe and cost effective construction strategies (Eldukair and Ayyub 1992). A significant number of fuzzy set applications reported in the literature use fuzzy sets primarily as a tool to make use of subjective and imprecise data for the safety and reliability assessment of existing structures. A more detailed description of the safety and damage assessment methodology and how it relates to this thesis is provided in Section 5.3. Some examples of fuzzy set applications reported in the literature that employ fuzzy sets and possibility theory is provided below.

Furuta et al. (1989) proposed an approach to incorporate subjective data in the reliability analysis of damaged structures. The effect of damage on structural integrity was introduced as a fuzzy reduction factor $\tilde{\phi}$, defined along [0,1] by a fuzzy set that corresponds to an assumed linguistic variable such as "severe," "moderate," or "slight" damage. A modified inference variable Z that corresponds to a safety margin is defined as

$$\tilde{Z} = \tilde{\phi}R - S, \tag{4.42}$$

where the ˜ denotes a fuzzy quantity and the member resistance R and load S are considered to be random variables according to classical structural reliability.

Furuta et al. assume that R and S are statistically independent and normally distributed random variables and define a fuzzy safety index $\tilde{\beta}$ as

$$\tilde{\beta} = \frac{\tilde{\phi}\mu_R - \mu_S}{\sqrt{\tilde{\phi}^2\sigma_R^2 + \sigma_S^2}}, \tag{4.43}$$

where μ_R and σ_R^2 and the resistance mean and variance and μ_S and σ_S^2 are the load mean and variance. A *fuzzy failure probability* is obtained from (4.43) by

$$\tilde{P}_f = \Phi(-\tilde{\beta}) \tag{4.44}$$

where $\Phi(\cdot)$ is the normal distribution function. Furuta et al. calculate fuzzy system failure probabilities with the PNET method (Ang and Ma 1981) based on load and resistance moments as well as the correlation between failure mechanisms.

Pandey and Barai (1994) proposed a sensitivity-based weighted-average method of structural damage assessment using fuzzy damage condition ratings. The approach is based on the premise that a member more sensitive to a damage parameter should receive a relatively higher weight for purposes of global damage assessment. The authors propose a fuzzy weighted average to combine subjective judgments of element-level condition ratings and a corresponding element importance factors. Mathematically, this is expressed as

$$\tilde{y} = \frac{\sum_{i=1}^{n} \tilde{w}_i \tilde{x}_i}{\sum_{i=1}^{n} \tilde{w}_i} \qquad (4.45)$$

where

$\tilde{y}$ = fuzzy weighted average

$\tilde{w}_i$ = fuzzy importance factor of the ith element

$\tilde{x}_i$ = fuzzy condition rating of the ith element

n = number of elements

and the ˜ is used again to denote a fuzzy quantity.

Pandey and Barai maintain that traditional methods of obtaining the member weights w_i that are based on heuristics, expert opinion or statistics are overly subjective and often produce inconsistent results. A more rigorous method based on gradient computations of static finite element equations is used to assign higher relative weights to structural members more vulnerable to damage. A condition rating is assumed from a collection of predefined reference fuzzy sets (such as "very good, "medium," and "poor") and combined with the member weights into (4.45) and mapped into linguistic terms for overall damage assessment. The authors argue such an approach produces a more realistic integrity index and is particularly useful when members are inaccessible to assess individual weights.

Dong et al. (1987) have proposed a reliability measure in terms of a *failure possibility*. The authors maintain a conception of failure based on possibility theory can complement the existing probabilistic and empirical methods of safety assessment. The method proposed by Dong et al. also combines subjective judgment and objective data to estimate the failure possibility of a structural system.

Three failure possibility criteria are described by Dong et al. The first is based on continuous-value (i.e. fuzzy) logic truth value of the statement $\tilde{S} > \tilde{R}$; he second is based on the fuzzy safety margin $\tilde{M} = \tilde{R} - \tilde{S}$; and the third is based on a fuzzy event $\tilde{M} = \tilde{S} \subseteq \tilde{R}$. The safety margin criterion is most relevant to this thesis. According to this criterion, the failure possibility *FP* is given by

$$FP = \frac{\int_{-\infty}^{0} \mu_{\tilde{M}}(x)dx}{\int_{-\infty}^{\infty} \mu_{\tilde{M}}(x)dx}. \tag{4.46}$$

where $\mu_M(x)$ is the membership function of the safety margin M and it is understood that the *safety possibility SP* is 1 - *FP*. The authors point out that calculation of the fuzzy failure possibility is highly sensitive to the evaluation criterion and illustrate that the safety margin criterion given by (4.46) provides the most reasonable estimate of failure possibility among the three criteria.

A number of methodologies to evaluate the reliability of structural systems have been reviewed. The theoretical background, mathematical complexity and inherent limitations of classical structural reliability theory have been outlined. The macroscopic performance evaluation methodology as applied to low-rise wood-frame buildings proposed by Stubbs et al. (1995) was also described. Fuzzy sets and the concept of failure possibility have been described and a number of applications of fuzzy sets to the problem of structural safety and performance have been reviewed.

But as argued in Chapter Two, effective application of these tools to assess the performance of buildings as *physical systems* should be augmented with methods that are able to consider the humanistic and social aspects of reality that inevitably have an influence on the performance of actual structures. To this end, the approach suggested by Dong et al. (1987), in conjunction with aspects of some methods described in the following chapter, are combined to provide a new methodology to measure the expected performance of wood light-frame structural systems subject to extreme wind hazards.

CHAPTER FIVE

For systems in which man and human actions have a significant effect on the reliability properties of the system, i.e. for humanistic systems, traditional methods have so far proved insufficient...for humanistic systems, with their inherent limited and fuzzy information content, it has been found that the use of fuzzy sets may be appropriate.

L. Andersson
The Theory of Possibility and Fuzzy Sets

Modeling Human Factors in the Building Process

It was shown in Chapter Two how the building process can be interpreted in both a sociological and a probabilistic context. It was also argued in the abstract that the broadly defined *human factors* inherent in the building process affect the performance of the constructed artifact. This chapter attempts to link these seemingly disparate interpretations of the building process towards the development of a methodology for the assessment of structural performance of wood light-frame structural systems.

This chapter begins with a discussion of the discrepancy between predicted rates of failure according to the structural reliability theory and observed rates of failure and that most failures are due to human factors. A review of some error modeling approaches for use in reliability analysis that have been reported in the literature is also provided. However, many of these models consider errors to be independent of the building process, a significant oversimplification and drawback of these models. For this reason, aspects of process flow methodology used to simulate construction activities are included and developed later in this thesis. A brief discussion of fuzzy-rule based modeling is also included to suggest an alternate approach to the consideration of effects of human factors on structural reliability.

5.1 Human Factors in the Building Process

Chapter Two presented the building process as a complex *socio-technical* system comprised of human, physical and procedural parameters that can influence the end state of the constructed artifact. Also recall from Chapter Two that term *human factors* refers to the humanistic parameters in the building process that might adversely affect structural performance. This term is used in lieu of human error, as advocated by Pidgeon and Turner (1986), in order to avoid precluding levels of organization higher than those connoted by error. In the literature, human error has been defined *as a significant deviation from standard practice* ("Report" 1986). Human factors as used in this thesis is the societal elements that contribute to the occurrence or amplify the effects of human error.

Investigations into the nature and effects of human error were initiated within the structural reliability community in response to the discrepancy between predicted rates of failure according to the structural reliability theory and recorded rates of actual failures. Building failure surveys (e.g. Blockley 1980; Ellingwood 1987) have confirmed that the majority of structural failures and concomitant financial damages in ordinary construction are consequences of human errors in planning, design, construction and utilization of structures (Nessim and Jordaan 1985; Ellingwood 1987; El-Shahhat et al. 1995).

Research into the nature of human error in the building process has proceeded from two directions: fundamental studies and frameworks for application. Fundamental studies focus on the application of probability and statistics in the development of phenomenological models and frameworks for application provide guidelines for quality assurance programs intended to minimize the effects of errors (El-Shahhat et al. 1996). Although human factors in the building process are difficult to quantify, a number of approaches to systematically address human error in engineering design and construction have emerged. Probabilistic models, scenario analysis and management strategies form the kernel of the growing body of knowledge regarding human error effects on structural reliability. Fundamental concepts are illustrated and select applications as reported in the literature are described in the following section.

5.2 Modeling Error Effects on Structural Reliability

The need to integrate human error in structural reliability models has been recognized. It is presumed that the modeling of human errors in structural reliability will provide a better understanding of error characteristics and mechanisms of occurrence and detection as well as provide more realistic estimates of the probabilities of failure.

Human errors in the building process lead to a chain of events that can be described in the following way: a human error causes a structural error (i.e. change in a basic design parameter) which causes a reduction in structural reliability, or alternately, an increase in the probability of failure (Arafah 1986). Reliability analyses incorporating the effects of

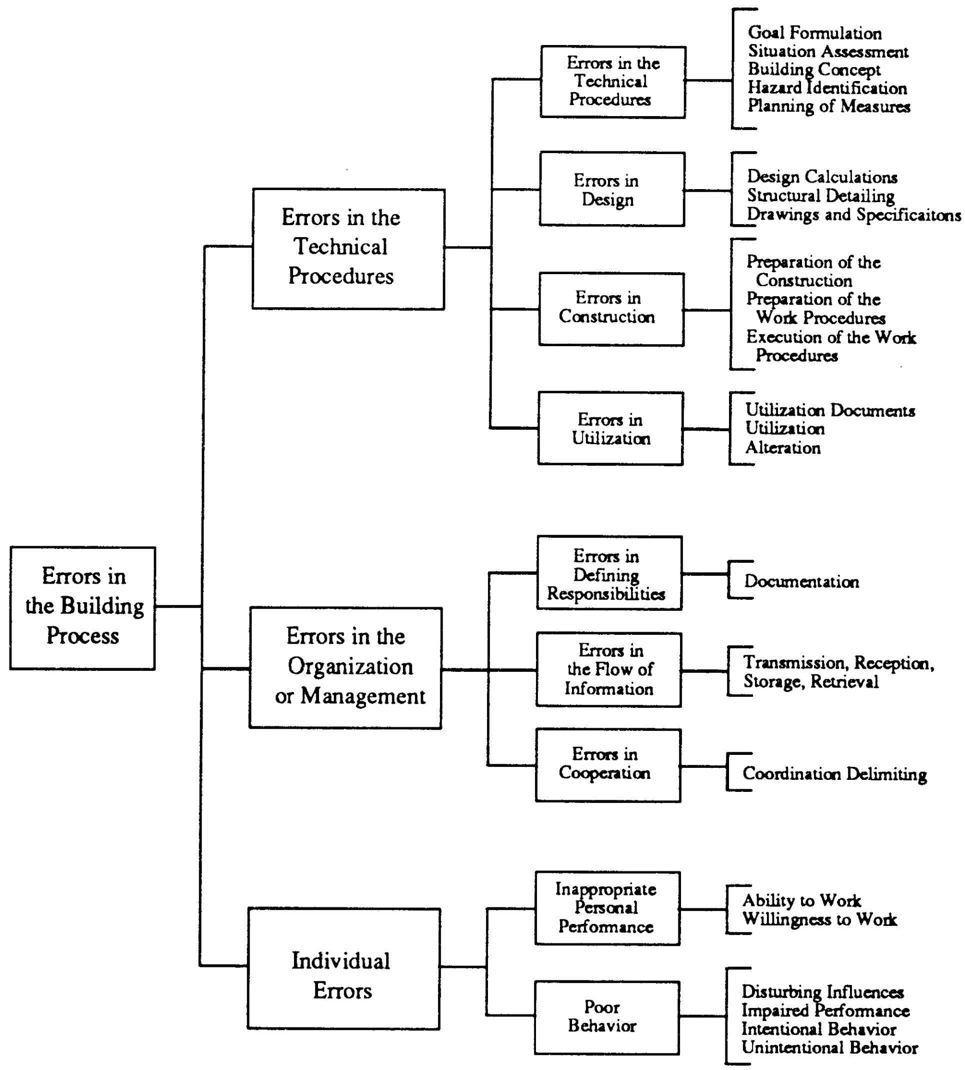


Figure 5.1
General classification of errors in the building process (adapted from Arafah 1986)

error should reflect (1) instances where the effect of the error is to modify the distribution functions of resistances or loads and (2), instances where the presence of error causes new or additional variables to be added to the limit state function or causes the function itself to be changed (Ellingwood 1987). Further, Arafah (1986) claims three fundamental relationships are critical in the integration of human factors on structural performance:

- The effect of error occurrence on the basic structural parameters.
- The relationship between these parameters and the structural reliability.
- The relationship between structural reliability and the expected cost of failure.

Mathematical models that consider these issues are reviewed below.

Consider again the probabilistic interpretation of the building process described in Section 2.2. According to the event tree of failure (Figure 2.2), the probability of failure is given by (2.2)

$$P(F) = \left[P(F_e|E) + P(F_s|E)\right]P(E) + P\left(F_s|\overline{E}\right)P\left(\overline{E}\right). \tag{2.2}$$

As Ellingwood (1987) points out, since $P(F/E)P(E) = P(E/F)P(F)$, this can be rewritten as

$$P(F)\left[1 - P(E|F)\right] = P\left(F|\overline{E}\right)P\left(\overline{E}\right), \tag{5.1}$$

which then reduces to

$$P(F) = P\left(F_s|\overline{E}\right)H_e, \tag{5.2}$$

where $H_e = P\left(\overline{E}\right)\left[1 - P\left(E|F_e\right)\right]^{-1}$ can be thought of as a "human error multiplier" that increases the classical failure probability conditioned on the event no error occurs.

Another way to introduce the effect of error occurrence on structural reliability has been provided by El-Shahaat et al. (1995). In the context of traditional FOSM reliability (Subsection 4.2.3), a factor λ is introduced as a modification of the strength variable such that the mean value is reduced by some fraction of the coefficient of variation. This modifies the classical reliability index β according to

$$\beta = \frac{\lambda\mu_R - \mu_S}{\sqrt{\sigma_R^2 + \sigma_S^2}} \tag{5.3}$$

where

$$\left.\begin{aligned}\lambda &= 1 \pm rV_R \\ &= 1 \pm r\left(\frac{\sigma_R}{\mu_R}\right)\end{aligned}\right\} \qquad r > 0$$

The effect of λ on the strength variable R and its relationship with the load effect variable S is depicted below:

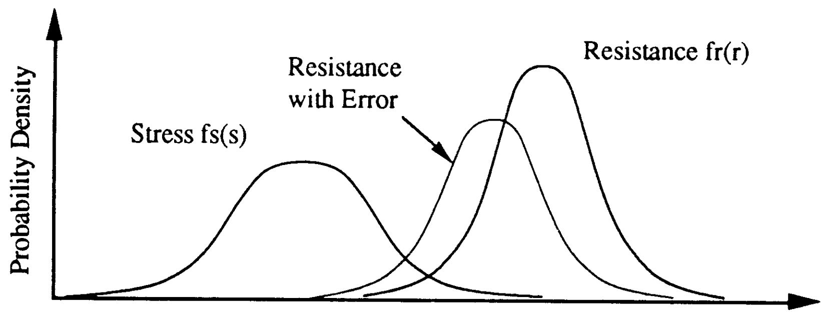


Figure 5.2
Schematic representation of error effect on strength variable R (adopted from El-Shahhat et al. 1996)

The above diagram illustrates a fundamental point regarding error effects on structural reliability, i.e. that errors (are assumed to) reduce the strength variable in a structural reliability model. In a probabilistic context, the effect of error may change the mean value, the variance, both mean and variance, and even the type of probability distribution function. Unfortunately the type of data required to establish such relationships are scarce.

Other methods can complement mathematical abstractions for the purposes of investigation. A scenario, defined as an abstract exploration of one path of an event tree, can form the starting point for more meaningful investigations into the effects of error on structural reliability. This method has been suggested by a number of researchers to improve quality assurance programs (Lind 1986; El-Shahhat et al. 1995). Arafah (1986) used the scenario approach to conduct sensitivity analyses of structural reliability of bridge structures to errors in order to identify priorities of inspection and quality control.

Although such studies are important for theoretical exploration the idealizations at times prevent the models from yielding any results of practical value. The primary reasons for this are that the models are often based on assumptions that:

- Errors are stochastic in nature
- Errors can be modeled separate from the building process

The validity of these two assumptions are questionable when one considers the true nature of human error. Hence, models based on the above seem to oversimplify the problem from a modeling point of view. According to Knoll (1982):

> If one adheres to the classic distinction of gross and random deviations, then this is in direct contradiction to the fact that errors originate with human action and are perpetuated and eventually caught by the same human agent, and that they are therefore, by definition, not random or stochastic events.
>
> In the light of what appears to be quite typical to real life cases, this is a gross oversimplification since errors are tied in manifold networks of relationships to the various parameters describing the building process.

In light of Knoll's second point, it seems that alternate approaches to investigating the influence of human factors on structural reliability would be useful. Methodologies outside of the traditional structural engineering might provide useful tools to examine the building process and how it can influence the performance of buildings. Two approaches that seem to have potential are briefly described in the following sections.

5.3 Process Simulation

The simulation of the construction process is currently an increasing field within civil engineering and has primarily been used to increase the efficiency of building activities (e.g Tommelein et al. 1993). Computer models of construction activities use properties of resources involved in their execution and the interactions between those resources (e.g. Vanegas et al. 1993). Although the objective of this type of analysis is usuaally to improve the coordination and execution of the building process in terms of resources, the modeling of construction activities as a process of interacting elements is a prerequisite for consideration of more qualitative social parameters.

Application of process simulation requires the division of the construction process into a set of discrete activities or tasks. The breaking down of the construction process into steps is fairly simple for common construction methods and can provide insight into how the tasks are completed and what factors influence the successful completion of the task. Although simulation of the process from this point on emphasizes the use of and relationships between resources necessary to do the work, certain attributes of the type of construction can be identified at the same time. For instance, the assembly of a wood light-frame roof system can be broken down into the basic tasks, and building attributes, such as roof geometry, complexity of framing and on-site supervision. The factors likely combine to determine the end state of the constructed artifact. It is this end state and the underlying process that should be considered in an assessment of expected structural performance.

Hence the parameters of the construction process - whatever they may be - combine to determine, along with other factors such as specifications and design intent, the actual parameters of the constructed artifact.

5.4 Fuzzy-Rule Based Modeling

Fuzzy rules can be used to provide a different approach to the modeling of the building process. While the simulation of the construction process can be an extremely valuable tool, being more of a classical engineering tool it is difficult to consider parameters that are not easily quantified in physical or temporal parameters. The modeling of the social aspects of the building process, like builder or inspector education, and their integration into a structural model seems to pose an interesting but ill-defined problem. Fuzzy rule-based modeling may be a useful tool in this endeavor.

Fuzzy rule-based modeling has many applications in other disciplines such as control theory and expert systems. Besides the increasing application in these areas, fuzzy rules can also be used for general descriptive purposes, and have proven to be very useful in cases where an explicit function is impractical or difficult to derive or calibrate (Bardossy and Duckstein 1995). In addition, fuzzy rules are not limited to the domain of fuzzy sets and logic, but can be used in conjunction with physically-based models (e.g. fuzzy control). This framework can be used to translate verbal rules into linguistic variables. For instance, a general example of a fuzzy rule might be (Bardossy and Duckstein 1995):

If events A and B or A and C occur, then the consequences may be either E or F and G

Such a statement, although fairly abstract, illustrates how mappings between fuzzy variables can be made based on heuristic or qualitative knowledge that seems to embody primary relationships between parameters. These mappings reflect the relationships between variables described by fuzzy sets and relationships in the form of rules. Although the above statement is rather vague, it illustrates how general the approach truly is. Fairly crude and verbal rules can provide a starting point wherein more sophisticated fuzzy rule-based models may be developed. For example, fuzzy rule-based modeling applied to the WLF building process might begin with such relationships as, "If building code effectiveness is low and construction activity is high, specification or construction error is somewhat high. Fuzzy mathematics can be used to define parameters such "building code effectiveness" and "specification error," as well as the numerical mappings between the fuzzy variables. This type of flexibility is difficult to achieve in non fuzzy environments. However, one of the challenges in effectively implementing fuzzy rules is the identification of appropriate variables to be used.

Although the use of fuzzy mathematics is not yet common currency in structural engineering science, it is argued that the fuzzy approach provides a high degree of flexibility. Exploratory modeling devoid of the constraints of classical probability theory with minimal computational effort to analyze the response of a model of a structural system subject to a high degree of human factor-influenced parameter and system uncertainties.

The human factor within the building process is a significant factor in the performance of structural systems. The current trend in structural reliability is to consider this influence, but the probabilistic context upon which traditional reliability theory is based imposes significant constraints and oftentimes unrealistic assumptions. Ways to examine the process underlying the design and construction of buildings seem to provide valuable insight into the effects of the human element on the building process. It is evident this influence is a complex phenomenon and is difficult to model within the context of probability theory. For these reasons a different approach is proposed in the next chapter to consider the influence of human factors on the expected performance of WLFC.

CHAPTER SIX

...Engineering practice, and thus the performance of civil engineering systems, can be improved by providing decision makers with tools that systematically remind the engineer of all the facts to be considered, give easy access to state of the art knowledge, stimulate the use of best judgment, and help make allowance for the nonquantitative parameters.

J.C. Santamarina and J.L Chameau
"Limitations in Decision Making and System Performance"
Journal of Performance of Constructed Facilities

Structural Quality Assurance of Wood Light-Frame Construction

Structural quality assurance is an analytical approach that considers the influence of various elements of the building process in the reliability analysis of structural systems (Madsen et al. 1986). This chapter develops and illustrates a methodology based on the tenets of structural quality assurance to assess the expected performance of wood light-frame structural systems. The computational aspects of the proposed methodology are illustrated by the analysis of a simple pin-connected frame structure. The proposed methodology is then applied to a prototypical gable roof sheathing system to examine the influence of various parameters on panel uplift failure due to a quartering wind.

The approach utilizes both subjective and objective data in the form of fuzzy sets to represent uncertain parameters in the load and resistance models. Two different fuzzy measures based on the component failure possibilities are used to assess expected system performance. Sensitivity analyses are then performed to investigate the influence of various parameters on the performance measures.

6.1 Structural Quality Assurance Methodology

The primary objective of this thesis is to develop a methodology that considers the effects of human factors in the building process of WLFC on the expected structural performance. The approach borrows extensively from established structural damage assessment methodologies. Since the adverse effects of human factors are assumed to reduce the load resisting capacity of the system, the treatment of error effects in structural engineering models is, in principal, identical to that of externally-induced damage (Yao 1985).

Assessing the reliability and safety of existing structures requires three types of data: (1) the physical properties of the structure, such as material properties; (2) the loads to which the structure is subjected; and (3) its failure mechanisms under these loadings or limit states. Fuzzy mathematics provide a flexible and computationally attractive tool for use in models of WLFC to consider uncertainties related to the influence of human factors in the building process. This general approach - the consideration of human influence on the reliability of structural systems - is referred to as structural quality assurance. A methodology of structural quality assurance that makes use of fuzzy sets and mathematics is proposed here and depicted schematically below. This methodology is illustrated in the following sections.

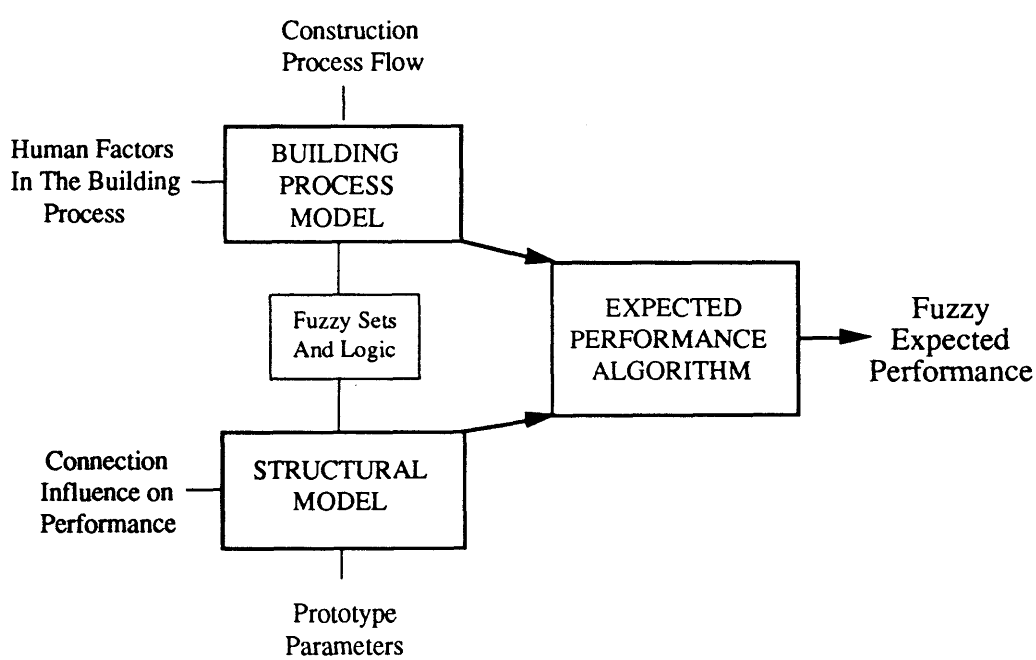


Figure 6.1
Proposed methodology for expected performance assessment of wood light-frame structural systems

6.2 Application of Proposed Methodology

To illustrate the computational mechanics of the proposed method to analyze the expected performance of structural systems, consider the simple pin-connected frame introduced in Subsection 4.2.3, as presented again below:

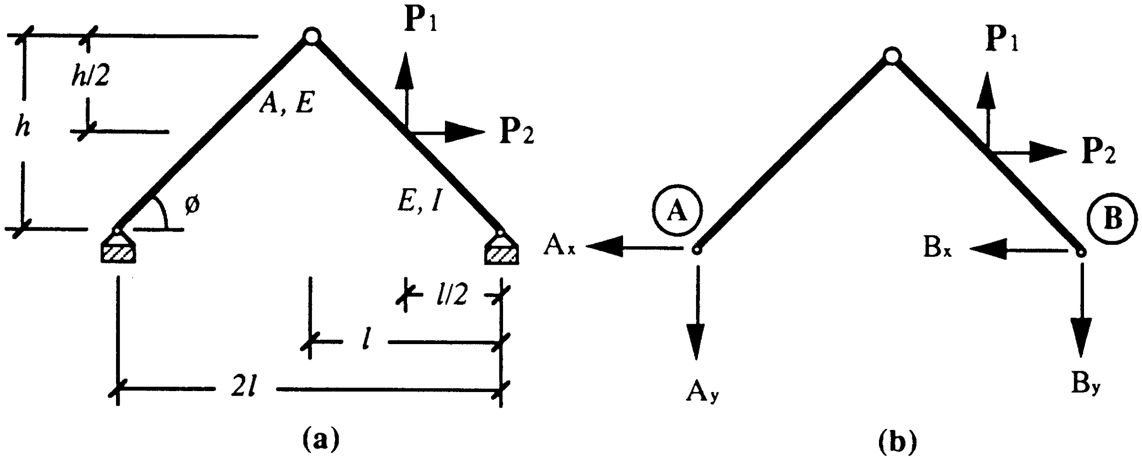


Figure 6.2
(a) *simple pin-connected frame subjected to lateral and uplift loads* P_1 *and* P_2; (b) *free-body diagram showing reactions at A and B*

Again, the reactions are given by equations (4.19a) - (4.19d),

$$A_x = \frac{P_1 l + P_2 h}{4h} \qquad A_y = \frac{P_1 l + P_2 h}{4l}$$

$$B_x = \frac{3P_2 h - P_1 l}{4h} \qquad B_y = \frac{3P_1 l - P_2 h}{4l} \cdot \qquad \text{(4.19a) - (4.19d)}$$

These joint reactions can also be expressed in matrix form for implementation in the computer. Let **P** represent a column-vector containing the loads, **U** a column-vector containing the joint reactions and **L** a matrix of coefficients determined by the geometry of the loaded frame. The above system of equations can then be restated as

$$\{P_1 \quad P_2\}\begin{bmatrix} \frac{l}{4h} & \frac{1}{4} & \frac{-l}{4h} & \frac{3}{4} \\ \frac{1}{4} & \frac{h}{4l} & \frac{3}{4} & \frac{-h}{4l} \end{bmatrix} = \{A_x \quad A_y \quad B_x \quad B_y\} \text{ , or} \qquad (6.1a)$$

$$\mathbf{P}^T\mathbf{L} = \mathbf{U}^T. \tag{6.1b}$$

For the purposes of illustration, assume the joint strengths have some capacity in excess of the forces required for stability given by the reaction vector **U**. This assumption corresponds to the factors of safety present in the strengths of actual structures. Also assume for simplicity that such excess capacity is constant for all joint components and is determined by a constant multiplier γ. The component strengths R_i are then

$$\begin{matrix} R_1 = \gamma A_x & R_3 = \gamma B_x \\ R_2 = \gamma A_y & R_4 = \gamma B_y \end{matrix}, \qquad \text{(6.2a) - (6.2d)}$$

such that $\gamma > 1$. Let **R** represent the vector of these joint strengths;

$$\mathbf{R} = \begin{Bmatrix} R_1 \\ R_2 \\ R_3 \\ R_4 \end{Bmatrix} = \begin{Bmatrix} \gamma A_x \\ \gamma A_y \\ \gamma B_x \\ \gamma B_y \end{Bmatrix} = \gamma \cdot \mathbf{U} . \tag{6.3}$$

Recall the so-called safety margin M in classical structural reliability analysis (as described in Subsection 4.2.2), which is a function of the random variables Z_i. For the fundamental case where M is defined as the difference between the strength R and load effect S,

$$M = g(Z_i) = R - S. \tag{6.4}$$

This quantity is used to evaluate the likelihood that the load effect exceeds the strength of a given component or system. In the present case, a vector **M** can be defined as

$$\begin{aligned} \mathbf{M} &= \mathbf{R} - \mathbf{S} \\ &= \mathbf{R} - \mathbf{U} \end{aligned} \tag{6.5}$$

where it is understood that **S** = **U**, $\mathbf{M} = \{M_1 \quad M_2 \quad M_3 \quad M_4\}^T$ and $M_i = R_i - U_i$.

As described in the previous chapter, the primary objective of the proposed methodology is to explicitly consider the influence of human factors in the building process on the expected performance assessment of a structural system. Assume at this point that human factors influence only the resistance side of (6.5), the effects of which are accounted for by a fuzzy reduction factor $\tilde{\phi}$. The difference between component strength and load-effect of the ith component is restated as

$$\tilde{M}_i = \tilde{\phi}_{r_i} R_i - S_i \qquad 0 \le \tilde{\phi}_{r_i} \le 1 \ , \tag{6.6}$$

where

$\tilde{M}_i$ = fuzzy margin

R_i = nominal component resistance

S_i = nominal component load effect

$\tilde{\phi}_{r_i}$ = unit fuzzy strength reduction factor

This formulation then accounts for some reserve strength capacity, due to factors of safety as determined by γ, and adverse effects of human factors that tend to counteract the reserve capacity. Note also that the margin is a fuzzy number because in the present case it is the difference between a fuzzy number and a crisp number. For simplicity the fuzzy reduction factor is assumed to be a triangular fuzzy number (TFN) defined as $\tilde{\phi}_i = (a_{i1},\ a_{i2},\ a_{i3})_T$. Therefore $\tilde{M}_i$ is a triangular fuzzy number defined by the triangular membership function $\mu_{M_i}(x)$, as depicted in Figure 6.3.

Recall the membership function $\mu_A(x)$ is a measure of the possibility or "likeliness" that a parameter whose value is not well known and described by the fuzzy number A takes on value x. Therefore the membership $\mu_{M_i}(x)$ of the fuzzy number M_i (the y axis) is a measure of the possibility that M_i takes on the value x. From the formal definition given in subsection 4.5.3, the safety margin *failure possibility* (*FP*) criterion for the ith component is defined as

$$\mathrm{FP}_i = \frac{\int_{-\infty}^{0} \mu_{M_i}(x)dx}{\int_{-\infty}^{\infty} \mu_{M_i}(x)dx}, \tag{6.7}$$

where again $\mu_{M_i}(x)$ is the membership function of M defined along x. The failures possibility defined in (6.7) is simply the ratio of the cumulative support for failure normalized by the entire area enclosed by the fuzzy number. It follows that in the limits the *FP* is zero if there is no interaction with zero and is unity if the entire fuzzy safety margin is less than zero, which is consistent with intuition (Figure 6.2).

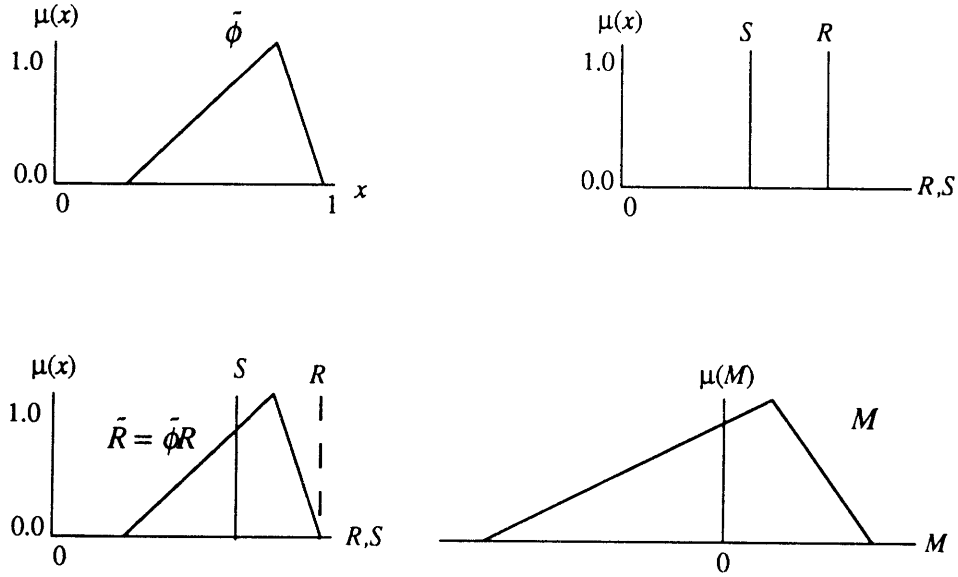


Figure 6.3
Relationship between reduction factor, resistance, load effect and fuzzy margin

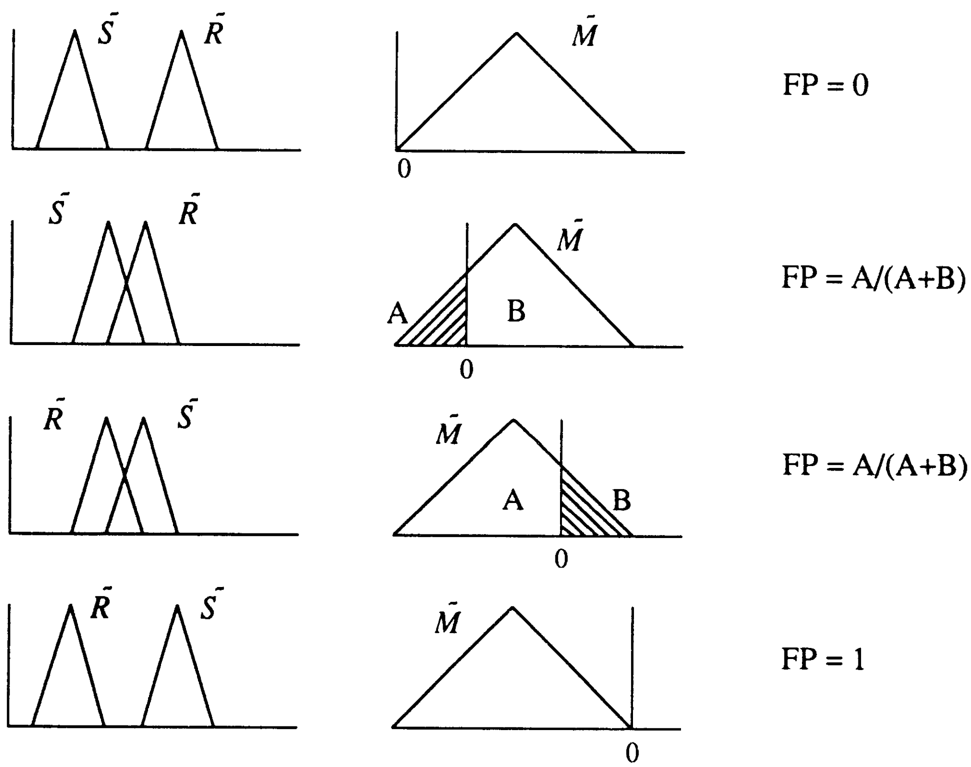


Figure 6.4
Failure possibility FP for the fundamental cases of the fuzzy margin (adapted from Dong et al. 1986)

The component fuzzy margins can be used in a number of ways to estimate the failure possibility of the system. In this chapter two fuzzy-based performance measures are considered based on a serial and hybrid models of system failure. A serial, or "weakest link" model, considers system failure to be defined by first element failure. Given a set of component failure possibilites, the system failure possibility *SFP* is simply (Dong et al. 1987)

$$SFP = \max_i (FP_i) \qquad i = 1,2,3,\ldots,n \tag{6.8}$$

where FP_i is the failure possibility of the ith component for a system of n compoents. This criterion of system failure provides an upper bound since the greatest component failure is used to describe system failure.

Alternately, a hybrid performance measure can be defined in terms of a nondimensional index I. If the relative importance of the individual components is known (or can be assigned), a fuzzy weighted average of the failure possibilities of all components i = 1,2,3 .... n. can be defined as

$$\tilde{I} = \frac{\sum_{i=1}^{n} \tilde{w}_i FP_i}{\sum_{i=1}^{n} \tilde{w}_i} \tag{6.9}$$

where

$\tilde{I}$ = fuzzy expected performance index

$\tilde{w}_i$ = fuzzy weight or importance factor of the ith component

FP_i = failure possibility of the ith component

Prior to calculating $\tilde{I}$, it is necessary to assign the weights $\tilde{w}_i$, which reflect the importance or contribution of the ith component to the failure of the structure. Traditional methods to obtain the weights, which are assumed to be fuzzy quantities in this analysis, include heuristics, statistical data, or based on expert opinion (Pandey and Barai 1994). For the statically determinate pin-connected frame described in this section, all components are equally important, in which case each $\tilde{w}_i$ is assigned a value of "absolutely important" according to an assumed set of reference triangular fuzzy number (TFN) weights shown in Figure 6.5. The expected performance index given by (6.9) can be mapped to a set of linguistic TFNs defined along [0,1] for easier evaluation (Figure 6.8).

An algorithm to evaluate the expected performance of the pin-connected frame using this method was implemented in the MATLAB® programming environment. A flow chart of the algorithm is presented in Figure 6.6, and a complete program listing is provided in Appendix C. Additional parameters assumed for the analysis are also listed in Table 6.1.

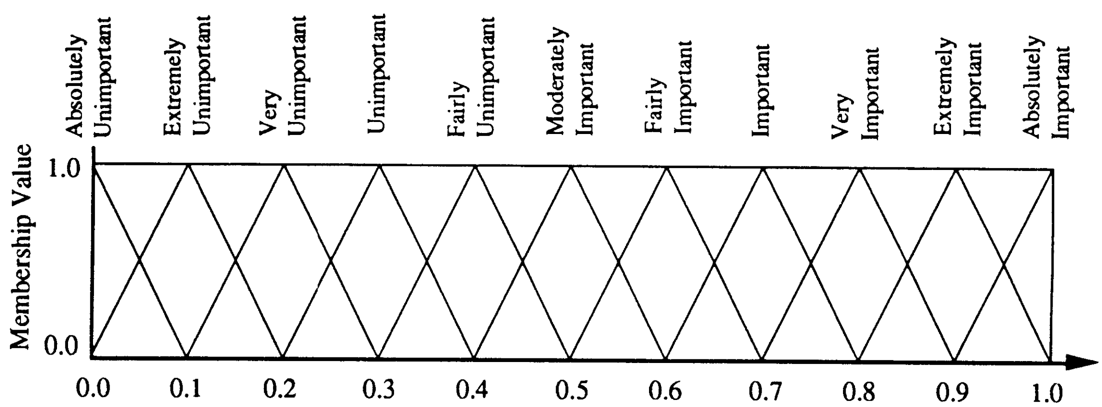


Figure 6.5
Reference fuzzy sets defined for structural importance weights (adapted from Pandey and Barai 1994)

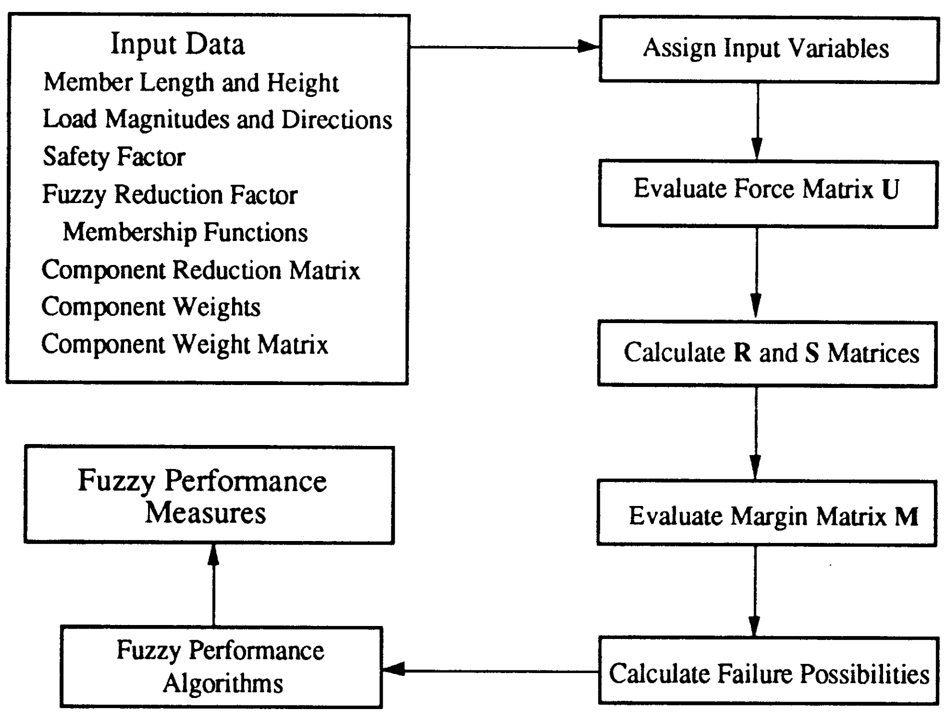


Figure 6.6
Algorithm for evaluating the expected performance of a simple pin-connected frame

Fuzzy set representation of uncertainty in this analytical approach affords the model significant flexibility, and the incorporation of human factor effects in the form of TFNs can support hypothesis testing by modifying the fuzzy parameter membership functions. More specifically, hypotheses can be characterized by sets of expected central values and the dispersion about that value. The parameters of the fuzzy reduction factors can be modified subjectively and assigned different parameters (that might correspond to various scenarios) for the purposes of sensitivity analyses. These modifications of the fuzzy parameters incur little computational cost since each scenario is a deterministic iteration.

Table 6.1

Assumed Pin-Connected Frame Parameters

Parameter	Variable	Value
height	h	10
length	l	10
vertical point load	P_1	10
horizontal point load	P_2	10
safety factor	γ	1.5
component weights	all w_i	$(0.90, 1.00, 1.00)_T$

Table 6.2

Assumed Human Factor Scenarios for Pin-Connected Frame

Scenario 1		TFN
horizontal fuzzy reduction factor	ϕ_1	$(1.00, 1.00, 1.00)_T$
vertical fuzzy reduction factor	ϕ_2	$(1.00, 1.00, 1.00)_T$
horizontal fuzzy reduction factor	ϕ_3	$(1.00, 1.00, 1.00)_T$
vertical fuzzy reduction factor	ϕ_4	$(1.00, 1.00, 1.00)_T$
Scenario 2		
horizontal fuzzy reduction factor	ϕ_1	$(0.40, 0.55, 1.00)_T$
vertical fuzzy reduction factor	ϕ_2	$(0.30, 0.90, 1.00)_T$
horizontal fuzzy reduction factor	ϕ_3	$(0.40, 0.55, 1.00)_T$
vertical fuzzy reduction factor	ϕ_4	$(0.30, 0.90, 1.00)_T$
Scenario 3		
horizontal fuzzy reduction factor	ϕ_1	$(0.40, 0.60, 0.80)_T$
vertical fuzzy reduction factor	ϕ_2	$(0.60, 0.80, 1.00)_T$
horizontal fuzzy reduction factor	ϕ_3	$(0.40, 0.60, 0.80)_T$
vertical fuzzy reduction factor	ϕ_4	$(0.60, 0.80, 1.00)_T$

For the purposes of demonstration, assume that three human factor scenarios can be defined by different sets of values of the fuzzy reduction factors (Table 6.2). Analysis of the pin-connected frame is conducted for each of the assumed scenarios, with all other parameters kept constant and as given in Table 6.1. The fuzzy set computations are performed according to the vertex method (Appendix A). The complete results are presented in Appendix B.

The results of the analyses are summarized in Table 6.3. It is evident from the table that scenario 1, which is assumed to represent an "ideal" case of no strength reduction, corresponds to an "absolutely perfect" ($\bar{I}$ = $(0.00,\ 0.00,\ 0.00)_T$) value of the expected performance and zero failure possibility, which is intuitively appealing. The results of scenario 2 (Figure 6.8) shows that two joint components have a non zero margin and lead to a maximum failure possibility (*SFP*) of 0.61.

The lower plot shows the resulting TFN of the expected performance index TFN mapped into linguistic performance space, where it can be observed that the expected peformance is nearest to "good." Scenario 3 represents an instance of siginifcant strength reduction and hence corresponds to a maximum failure possibility of and an expected performance of "medium." (In Figure 6.7, the x values in the fuzzy margin plots correspond to the margin values and the y-axis is the degree of support for those values. The axes in the bottom graph are non-dimensional values and supports for those values, respectively, in linguistic space arbitrarily defined along [0,1] for convenience.)

Table 6.3

Pin-Connected Frame Fuzzy-Based Expected Performance Measures

Scenario	SFP	Expected Performance Index
1	0.00	$(0.00, 0.00, 0.00)_T$
2	0.61	$(0.01, 0.02, 0.08)_T$
3	0.57	$(0.01, 0.02, 0.08)_T$

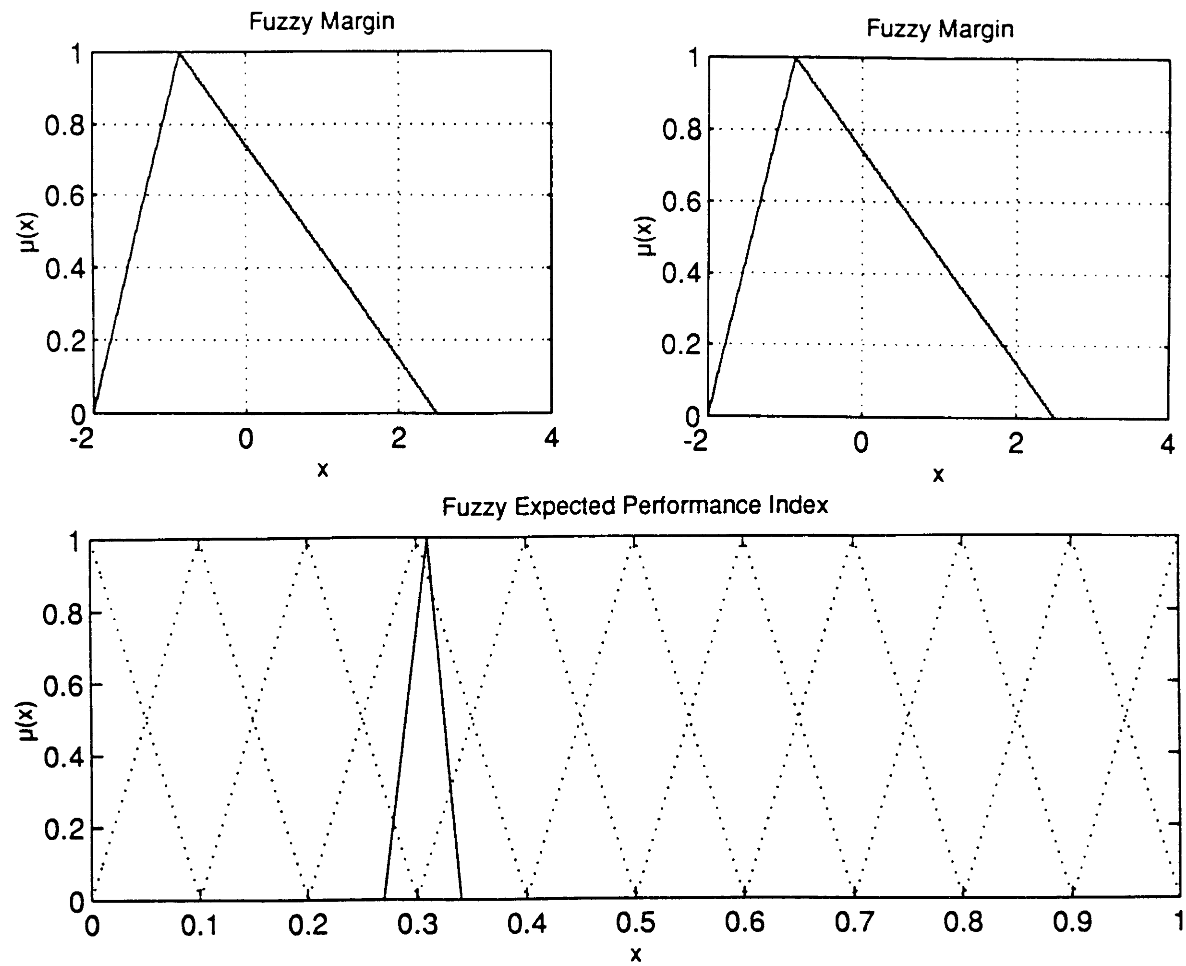


Figure 6.7
Graphical output of frame expected performance index for scenario 2 with SFP = 0.61

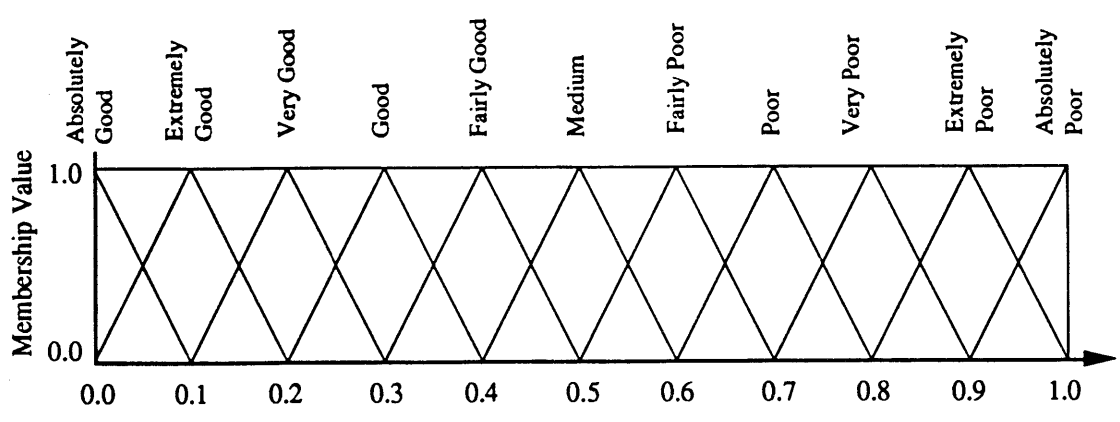


Figure 6.8
Reference linguistic variables used to map the expected peformance index

6.3 Analyses of Prototype Roof System

The expected performance of a prototypical gable roof sheathing system is presented in this section to illustrate an application of the proposed methodology to WLFC. The analysis proceeds according to the previous section with assumed parameters as listed in Table 6.4, considered representative of typical wood light-frame gable roof sheathing systems. A plan view of the gable roof sheathing system as depicted in Figure 6.9.

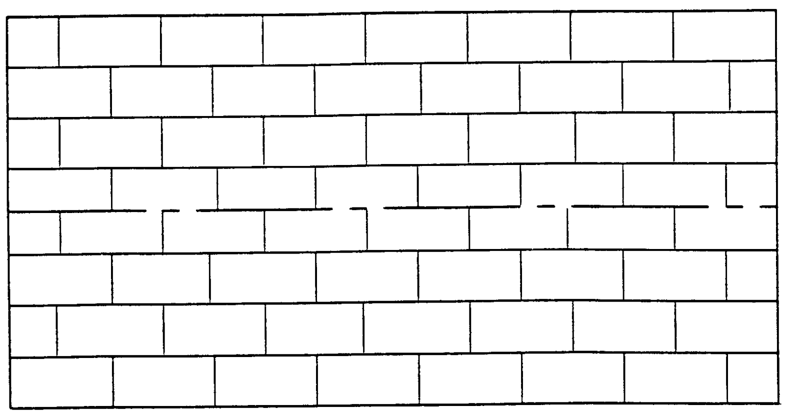


Figure 6.9
Schematic plan view of prototype roof system

Table 6.4
Assumed Roof System Prototype Model Parameters[†]

	Prototype Roof System Attributes			
Width	30 feet		Roof Slope	4 on 12 (18.4°)
Length	60 feet		Design Wind Speed	100 mph
Mean Roof Height	11.5 feet		Exposure Category	C
	Panel Field Fastener Spacing	Type[††]	Panel Edge Fastener Spacing	Type[††]
Zone 1[‡]	$\varepsilon_f = 12$ in o.c.	6d	$\varepsilon_e = 6$ in o.c.	6d
Zone 2[‡]	$\varepsilon_f = 6$ in o.c.	6d	$\varepsilon_e = 6$ in o.c.	6d
Zone 3[‡]	$\varepsilon_g = 6$ in o.c.	6d	$\varepsilon_e = 4$ in o.c.	6d

† In accordance with the South Florida Building Code (Douglas 1992).
†† Ultimate withdrawal capacity of 6d common nail is 262 lb (Douglas 1992).
‡ Zones as defined by "Standard" 1993, as given in Figure 6.12 of this Chapter.

6.3.1 Failure Mode and Limit State Considered

Sheathing panel uplift was a very common failure modes of gable roof systems oberved in the wake of Hurricane Andrew and other Hurricane events and is the only failure mode considered in this section. A wood light-frame gable roof sheathing system is idealized as a set of connection (or fastener) components, where failure is assumed to occur only at these fastener connections. This simple idealization seems reasonable given the observed performance of WFLC under extreme wind loads as described in Section 2.1. Uplift failure is assumed to occur when the net individual panel uplift resistance is exceeded by uplift force normal to the roof plane. This limit state is expressed as a margin in the typical form of $M = R - S$.

The definition of failure in the context of fuzzy representation differs from that of the probabilistic formulations in traditional structural reliabilty theory. In the present case, the support for membership in the fuzzy set "failed" is first dependent on how the failed set is defined. If the structural system is modeled as a series (weakest-link) system, then first component failure constitutes system failure, as described above. In this case the failure possibility of a system of n sheathing components would be the maximum failure possibility over all components, evaluated according to

$$SFP = \max_i (FP_i) \qquad i = 1,2,3,\ldots,n. \tag{6.8}$$

Such a definition of failure provides an upper bound for the failure possibility of the system, or conversely, a lower bound to the expected performance of the system.

In addition, the hybrid measure of failure based on the individual component failure possibilities weighted by the component's relative importance is also considered. If different components are weighted differently, these weights must be assigned in a consistent manner. The sheathing panels of the prototype roof system used in this study are weighted according to the scheme presented in Figure 6.10.

The importance of individual sheathing panel resistances are assigned subjectively based on the premise that sheathing panels at the outer edges of the roof are more important to overall system performance than panels within the field. For instance, although all panels collectively provide enclosure and structural integrity via diaphragm action, the gable end panels also provide lateral stiffness to gable end rafters or trusses. In addition, panels along the roof perimeter parallel to the ridge also import protection to the roof-to-top plate connection, an essential element in the prevention of progressive failure of the roof.

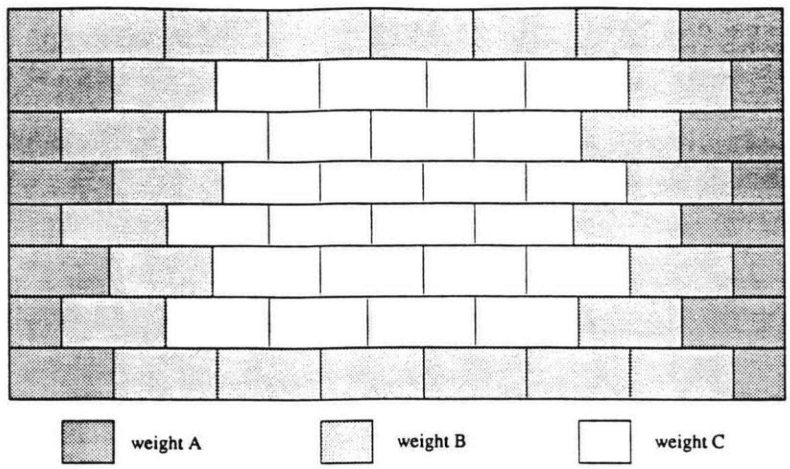


Figure 6.10
Sheathing panel weighting scheme

The weights are taken from reference fuzzy sets based on a model suggested by Pandey and Barai (1994) presented earlier in this chapter. The reference fuzzy sets, provided again in Figure 6.11 for convenience, are assumed to be triangular fuzzy numbers, as shown. Three weights are assigned to three different classes of panels according to the above argument. Let class A represent "extremely important" components, class B "very important" components and class C "important components."

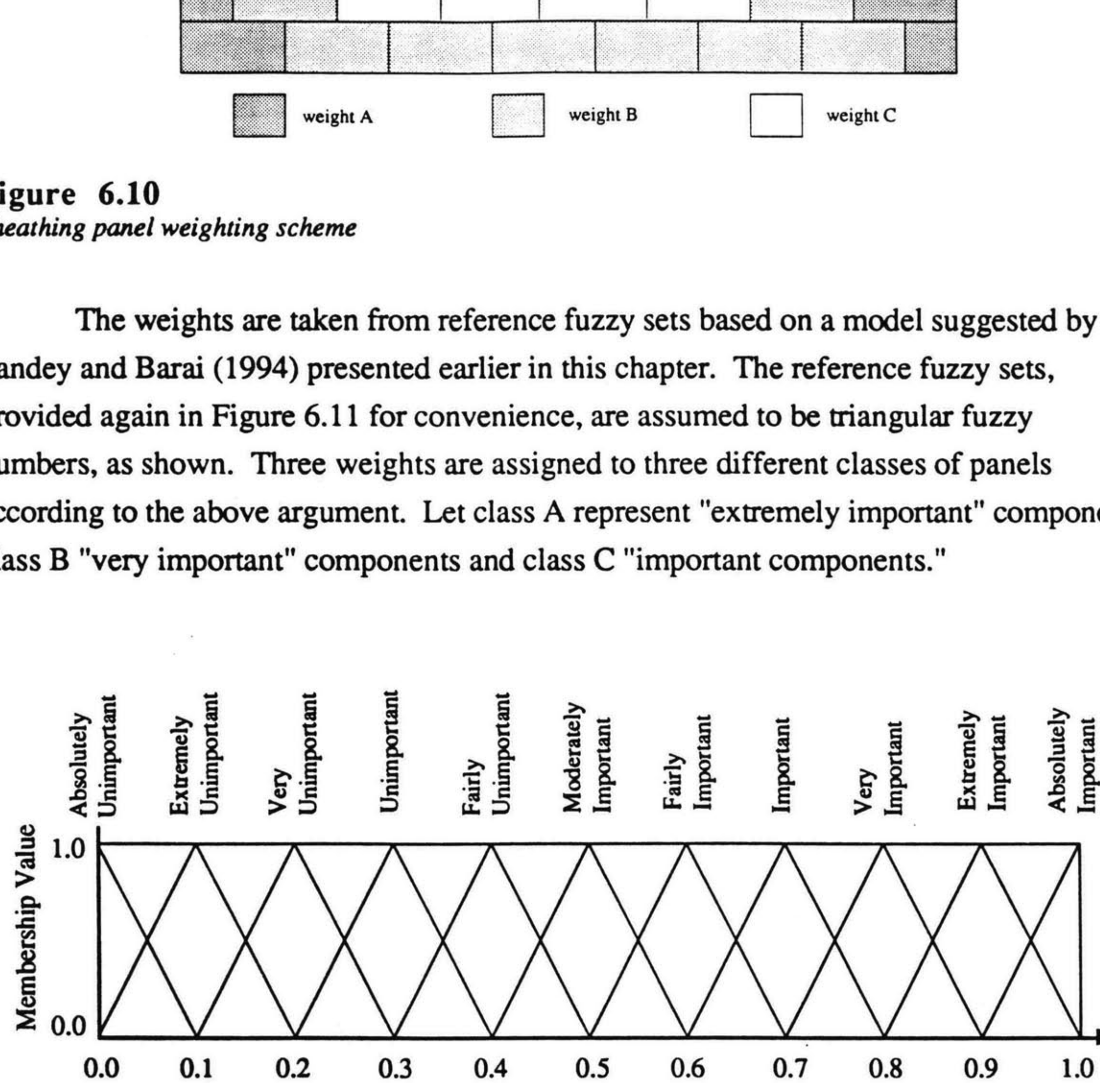


Figure 6.11
Reference fuzzy sets defined for structural importance weights (adapted from Pandey and Barai 1994)

The set reference triangular fuzzy sets that correspond to these linguistic assessments are

$$\tilde{w}_A = (0.80,\ 0.90,\ 1.00)_T$$
$$\tilde{w}_B = (0.70,\ 0.80,\ 0.90)_T. \qquad (6.10a) - (6.10c)$$
$$\tilde{w}_C = (0.60,\ 0.70,\ 0.80)_T$$

According to Pandey and Barai (1994), when weights are directly assigned they should be checked for consistency. A pairwise comparison of relative weight ratios r_{ij}, defined as

$$r_{ij} = \frac{w_i}{w_j} \qquad (6.11)$$

where w_{ij} is the relative importance of joint i relative to joint j. An importance matrix $\Re$ can then be constructed from the ratios of weights

$$\Re = \begin{bmatrix} r_{11} & r_{12} & \cdot & \cdot & r_{1m} \\ r_{21} & r_{11} & \cdot & \cdot & r_{2m} \\ \cdot & \cdot & \cdot & \cdot & \cdot \\ \cdot & \cdot & \cdot & \cdot & \cdot \\ r_{m1} & \cdot & \cdot & \cdot & r_{mm} \end{bmatrix} \qquad (6.12)$$

The above matrix is known as consistent if the following conditions are satisfied:

$$r_{jk} = r_{kj}^{-1} \qquad (6.13)$$

and

$$r_{jl} = r_{jk} \cdot r_{kl} \qquad \text{for all } j,k,l \le m \qquad (6.14)$$

It can be shown that the weighting scheme presented earlier in Figure 6.10 is consistent.

6.3.2 Resistance Model

The resistance of a sheathing panel to uplift load is assumed to be function of the withdrawal capacity of the fasteners used to connect the panels to the rafters/trusses and the spacing between individual fasteners. The nominal resistances of panels to uplift loads are discretized by panel and idealized first as deterministic forces. The magnitudes of the sheathing panel resistance are determined by code-specified fastener schedules and ultimate withdrawal capacities. The general relation for determining the net fastener strength is given by

$$R_{uplift} = ca\varepsilon\left[1+\left(\frac{s}{d}\right)\right] \tag{6.15}$$

where

c = fastener ultimate withdrawal capacity [lb / fastener]

ε = specified fastener spacing [fasteners / ft]

a = panel width [ft]

s = panel length [ft]

d = truss or rafter spacing [ft]

Since there are different fastener spacing schedules specified by code, the uplift resistance for sheathing panels varies by zone (Figure 6.12). For a typical panel in zone 1, the nominal uplift resistance, denoted R_1 (in pounds) is calculated as

$$R_1 = ca\left[2\varepsilon_e + \varepsilon_f\left(\frac{s-d}{d}\right)\right] \tag{6.16}$$

where ε_e and ε_f are the panel edge and field fastener spacings, respectively, and all other quantities are as defined in (6.15). For a typical panel in zone 2, the nominal panel uplift resistance R_2, again in pounds, is calculated by

$$R_2 = ca\varepsilon_e\left(2+\frac{s-d}{d}\right). \tag{6.17}$$

The nominal panel uplift for a typical panel in zone 3 is given by

$$R_3 = ac\left[\varepsilon_g + \varepsilon_e\left(1+\frac{s-d}{d}\right)\right]. \tag{6.18}$$

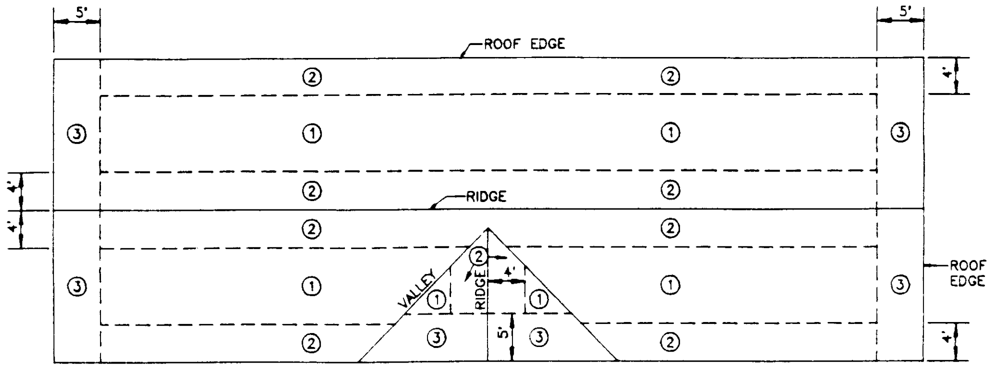


Figure 6.12
Gable roof sheathing fastener zones (after "Standard" 1993)

6.3.3 Load-Effect Model

The load-effect of an extreme wind load on a sheathing panel is idealized as discrete fuzzy uplift forces normal to the roof plane. The (deterministic) magnitude of the uplift pressure p normal to the roof plane is approximated by

$$p = qC_pC_g \tag{6.19}$$

where q is the reference velocity pressure corresponding to an assumed wind of speed V at mean roof height and the product C_pC_g is a pressure coefficient. For a wind speed of 120 miles per hour and a mean roof height of 11.5 feet, the reference velocity pressure is equal to 37 pounds per square foot. For use here, values for C_pC_g are based on data obtained from wind tunnel tests of gable roof scale models as provided by Meecham (1988). These data were selected in order to consider in more detail localized wind effects on gable roofs that might have a significant influence on expected performance.

For illustration, the contours of nominal peak-negative-pressure coefficients for a gable roof subjected to a quartering wind load as reported by Meecham are overlayed on the assumed sheathing schedule of the prototype roof system (Figure 6.13). It is evident from the contour profiles in the figure that any given sheathing panel is subjected to varying magnitudes of negative pressure. For this reason the peak negative pressure coefficients are first discretized by panel and then "fuzzified" to account for load variability. For simplicity, let C denote the product C_pC_g and assume the varying magnitude can be accounted for by a triangular fuzzy number. A "fuzzified" peak negative pressure coefficient for the ith panel is then

$$\tilde{C}_i = \left(c_{i,1},\ c_{i,2},\ c_{i,3}\right)_T \tag{6.20}$$

where the break points $c_{i,j}$ of the triangular fuzzy pressure coefficients are assigned subjectively based on Figure 6.13. In this way the triangular fuzzy number $\tilde{C}_i$ accounts for the dispersion of the load about a median value c_2 such that c_1 and c_3 are the minimum and maximum coefficient values, respectively, that fall within the area of a particular panel. (A complete listing of the pressure coefficient parameters is provided in Appendix C.) It then follows from (6.19) and (6.20), that the fuzzy load effect on the *i*th sheathing panel is

$$\tilde{S}_i = q\tilde{C}_i A_i \tag{6.21}$$

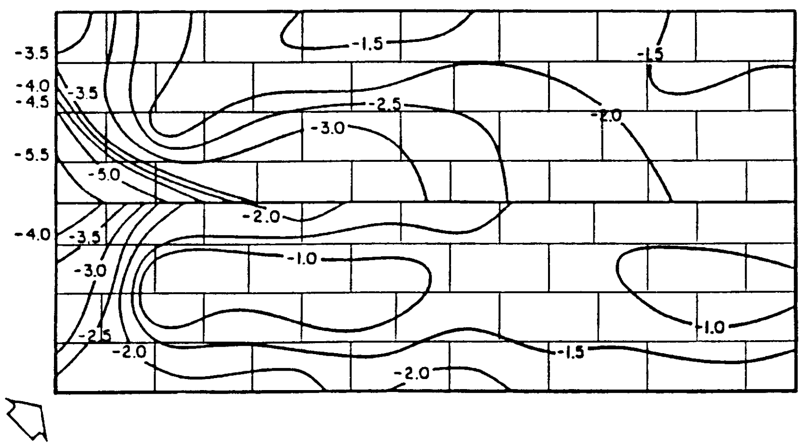


Figure 6.13
Contours of peak negative pressure ceofficients for gable roof subjected to quatering winds overlayed on roof sheathing schedule (modified from Meecham 1988)

6.3.4 Design and Construction Process Error Scenarios

Construction process flow methodology used to model construction operations was described in Section 5.4. It was argued in the Chapter Five that the nature of the process underlying the design and construction of WLF buildings can significantly affect the constructed artifact, i.e. the *in situ* structural parameter values. The various phases of the construction process can be broken down into discrete tasks. Understanding the nature of those tasks which comprise the roof assembly process, and how imperfection (in the form of specificaton and construction errors) affect the resulting structure, is the topic of this section.

The steps that comprise the assembly of a roof system (given a design specification) can be examined to gain insight into the possible instances where imperfect specification or installation might occur. For example, the construction process of a typical roof system involves placing and connecting roof framing, followed by installation of sheathing panels. (This is usually initated at one corner upon which subsequent panels are placed and fastened.) Although a more rigorous method may be developed through the use of process simulation or fuzzy rule-based modeling of the construction process, this section explains how subjectively defined hypothetical scenarios of design and construction imperfection can be translated into fuzzy reduction factors. The fuzzy reduction factors are then used as inputs into the performance algorithm. The primary objective is to conduct sensitivity analyses to either investiage the potential vulnerability of existing sheathing systems or to inform the development of prescriptive code requirements.

Assume a correlation exists between the installed quality of panels within the same row and the set of gable end panels. This grouping roughly corresponds to the roof zones as defined in Figure 6.11. Although this assumption is made primarily to simplify the analysis, it does seem reasonable given the fact that panels are often installed by rows running parallel to the length of the building and that panels at the gable ends are in general more difficult to install and inspect.

Now consider five different hypothetical scenarios in the specification and construction process of the prototypical roof presented earlier. Such scenarios might include the following:

(1) One entire row of fasteners along a rafter/truss in a panel are missing;
(2) One half of all fasteners are missing;
(3) Misuse of fastener schedule prescriptions such that the field spacing is used throughout the roof;
(4) Substandard fastener installation along perimeter and gable ends; and
(5) Substandard fastener installation throughout.

These scenarios might translate into the following sets of fuzzy reduction factor parameters:

Table 6.5

Assumed Human Factor Scenarios for Gable Roof Sheathing System

Scenario	$\tilde{\phi}_1$	$\tilde{\phi}_2$	$\tilde{\phi}_3$
1	$(0.90, 0.90, 1.00)_T$	$(0.90, 0.90, 1.00)_T$	$(0.90, 0.90, 1.00)_T$
2	$(0.50, 0.50, 1.00)_T$	$(0.50, 0.50, 1.00)_T$	$(0.50, 0.50, 1.00)_T$
3		*(parameter values adjusted in code)*	
4	$(0.20, 0.50, 0.80)_T$	$(0.50, 0.50, 1.00)_T$	$(0.55, 0.75, 1.00)_T$
5	$(0.20, 0.60, 0.80)_T$	$(0.20, 0.60, 0.80)_T$	$(0.20, 0.60, 0.80)_T$

6.3.5 Fuzzy Expected Performance Measures

The above scenarios are used as input parameters in the series of fuzzy performance algorithms described in this section. The performance of the prototype roof system under extreme loads is assumed to be a function of the *likeliness* of failure and the relative importance of the individual connection components. As mentioned previously, two performance criteria are used to characterized the expected performance of the system. The algorithms used to evaluate failure according to these criteria are described in this section.

The failure possibility *FP* corresponding to the support for failure of a triangular fuzzy margin $M = (m_1, m_2, m_3)_T$ is effectively the ratio of the area under the membership function and to the left of zero to the total area enclosed by the triangular fuzzy number M. Given a fuzzy resistance $\tilde{R}_i$ and load effect $\tilde{S}_i$ of the ith sheathing panel, the fuzzy margin is given by

$$\begin{aligned}\tilde{M}_i &= \tilde{\phi}_i R_i - q\tilde{C}_i A_i \\ &= \tilde{R}_i - \tilde{S}_i\end{aligned} \qquad (6.22)$$

where $\tilde{\phi}_i$ is a fuzzy reduction factor and all other quantities as defined earlier. The above combines the resistance load model $\tilde{R}$, which is influenced by the "quality" of the constructed artifact in terms of uplift resistance, with the load model $\tilde{S}$, which is influenced by the magnitude and dispersion of the wind-induced negative pressure.

The vector of component margins given by (6.22) evaluated in each scenario is used to characterize system performance according to the system failure possibility and fuzzy weighted-average index presented in (6.8) and (6.9), respectively.

6.4 Interpretation of Results

Results from the scenario analysis are presented in Appendix B and summarized in the Table 6.5 below. It appears that scenario 1, which is assumed to represent a fairly minor discrepancy between the (imperfect) actual and ideal conditions, does not correspond to any significant reduction in strength. No joint components fail and hence performance is not compromised. Scenario 2 corresponds to a fairly minor overall effect of human factors, although it induces a maximum failure possibility of 0.22. A plot of panel failure possibilities under scenario 2 (Appendix B) is summarized in Figure 6.14.

The values given in Figure 6.14 correspond to sufficiently non zero (i.e. values rounded to the nearest tenth) panel *FP*s and are overlaid on the panel schedule to compare the location of panels that have non zero *FP* with their relative importance to the overall system (Figure 6.19). There appears to be a noticeable support for failure of panel 34 and significant support for failure of panel 41. Reference to the panel numbering scheme and Figure 6.13 shows that these three panels, which are on the gable end of the leeward slope and nearest the ridge, are also located in localized areas of fairly high negative pressure (i.e. relatively high values of C_pC_g). Hence the reduction in scenario 2 signifies, in accordance with observations, the particular vulnerability of the leeward gable end sheathing panels to local concentrations of negative pressure due to a quartering wind.

Table 6.6

Gable Roof System Fuzzy-Based Performance Measures

Scenario	*SFP*	Expected Performance Index
1	0.00	$(0.00, 0.00, 0.00)_T$
2	0.22	$(0.00, 0.00, 0.00)_T$
3	0.57	$(0.00, 0.00, 0.00)_T$
4	0.38	$(0.00, 0.00, 0.00)_T$
5	0.42	$(0.00, 0.01, 0.05)_T$

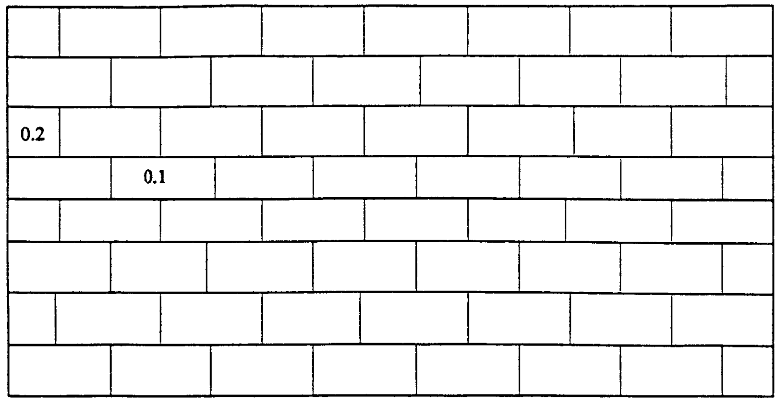


Figure 6.14
Schematic representation of supports for panel failure under scenario 2 (wind incident at lower left corner)

It is evident from the assumed reduction factors that scenario 3 is considered to represent a fairly significant reduction in resistance, which as expected, yields a set of fairly high panel failure possibilities. Although the *SFP* is 0.57, there are a number of panels that have sufficiently non zero *FP*s, particularly panels 25, 33, 34, 41 and 57, as shown in Figure 6.16. Scenario 4 yields similar results, where panels 25, 33 and 41 and 49 have significant support for failure (Figure 6.16). Again, these panels fall within areas of localized concentrations of negative pressure.

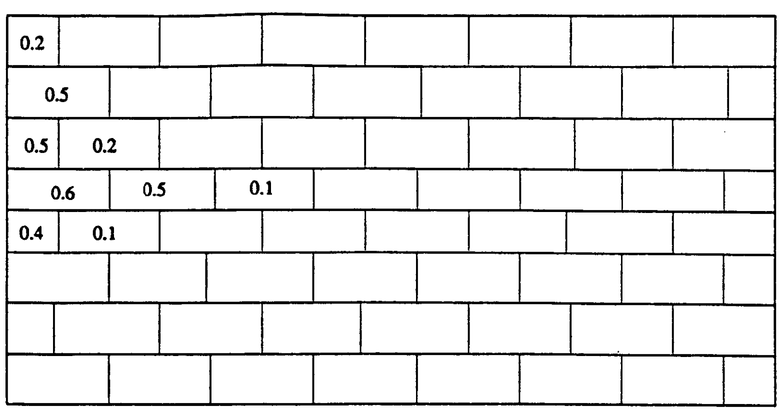


Figure 6.15
Schematic representation of supports for panel failure under scenario 3 (wind incident at lower left corner)

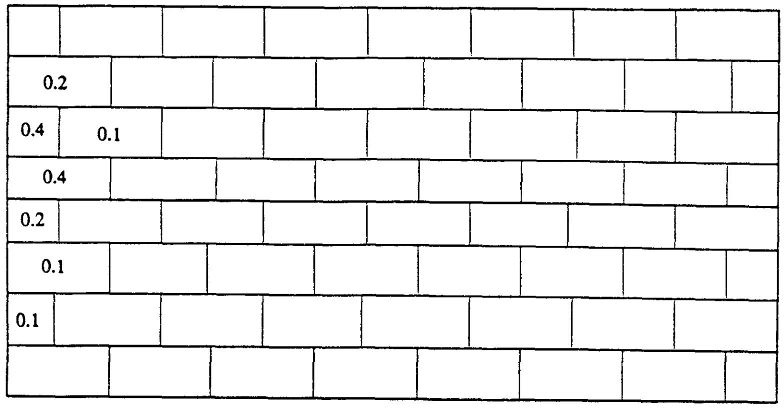


Figure 6.16
Schematic representation of supports for panel failure under scenario 4 (wind incident at lower left corner)

Scenario 5 produces a greater number of non zero *SFP*s, and examination of the component failure possibilities plots (Appendix B) suggests that scenario 5 corresponds to a greater overall vulnerability than the previous scenarios (Figure 6.17). These results are expected since scenario 5 represents a pervasive deficiency in assembly and hence lower *in situ* panel resistance. The same correlation between relatively high panel *FP*s and localized pressure concentrations For all of these scenarios, the fuzzy weighted average suggests that the systems are expected to perform in an ideal manner.

The preceding discussion has focused on individual panel *FP*s or the overall (maximum) system failure possibility. Reference to Table 6.6 shows that the two measures of expected performance yield disparate measures of system performance. The general trend of increasing frequency and magnitude of panel component *FP*s in scenarios 1 to 6 is congruent with the general increase in reduction factors (Table 6.5). Despite this and the above scenario observations, the expected performance index does not suggest substandard performance in any of the first four scenarios, and a negligible reduction in expected performance for scenarios 5.

The source of this disparity is the way that the index measure aggregates all panels *FP*s into one composite TFN. Even though each panel *FP* is weighted according to an assigned importance factor, the additive "system" importance (i.e. the denominator of the weighted-average index) grows sufficiently large for the gable roof example to significantly minimize the contribution of a few but nonetheless important panels to system failure. It seems this consequence of fuzzy normalization, which did not seem to affect the results in the pin-connected frame example, makes the index a poor measure of expected performance for systems of many components. This is especially true in instances where the failure of a few but nonetheless significant components corresponds to system failure.

It therefore seems the weakest-link system failure criterion is a more accurate measure of expected system performance when augmented by the entire vector of panel *FP*s. This is especially true if single component failure is of primary concern. The disadvantage of the weakest-link failure criterion is that possible influence of nearly-poor performing components and relative importance to the system are not explicitly considered in the *SFP* measure. In sum, the fuzzy performance index places greater emphasis on panels that are presumed to be more important to overall system performance, but the resolution of the measure is skewed because it is normalized by the sum of all of the component weights. While mathematically correct and attractive in its simplicity, the fuzzy weighted average does not seem to be an appropriate measure of gable roof sheathing performance due to extreme wind loads.

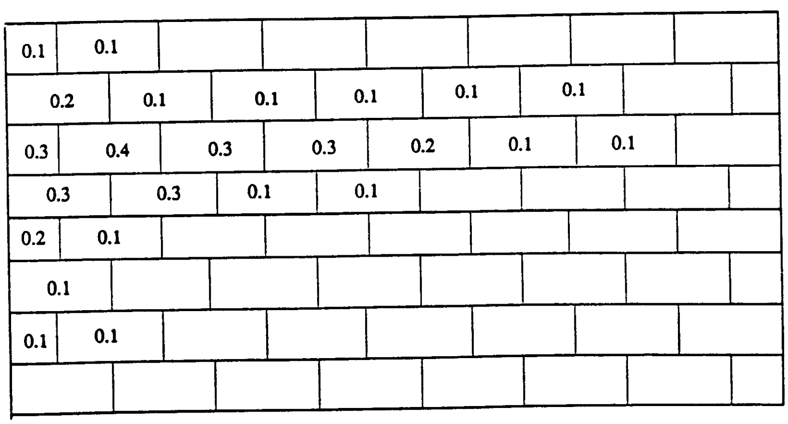


Figure 6.17
Schematic representation of supports for panel failure under scenario 5 (wind incident at lower left corner)

CHAPTER SEVEN

As the complexity of a system increases, out ability to make precise and yet significant statements about its behavior diminishes until a threshold is reached beyond which precision and significance (or relevance) become almost mutually exclusive characteristics.

N.F. Pidgeon and B.A. Turner
"Human Error and Socio-Technical System Failure"
Human Error in Design and Construction

Summary and Conclusions

The poor performance of wood light-frame construction (WLFC) in the southern United States during recent hurricane storms, particularly Hurricane Andrew of 1992, brought into question key parts of the WLFC building process. Low-rise, wood light-frame buildings comprise a significant portion of the built environment, and there is a clear need to mitigate damage to WLF buildings and building contents from future hurricanes. It is hoped that, at least in part, this can be achieved by learning lessons from the past and considering alternate ways to approach the problem.

This research was initiated in response to this problem of poor performance, and places significant emphasis on how to *approach* the problem. The approach presented in this thesis is based on the premise that the constructed artifact is the end result of a complex *socio-technical* process of design, construction and regulation. It seems that sociological patterns exist with regard to large-scale disasters of the type produced by Hurricane Andrew, and understanding such patterns can point out instances where more improvements can be made or are necessary. In addition, the potentially adverse influence of human action on the performance of the end product of the building process can be understood in a probabilistic context. Fundamental strategies to minimize the occurrence of failure, such as reducing errors or designing more robust structural systems to absorb this influence, were also introduced within this framework.

The basics of wind loads on low-rise buildings and how WLF structural systems respond to those loads were provided as background for the rest of the thesis in Chapter Three. The use of engineering concepts in the development of wind-resistant construction prescriptions, and recent work within this field, were also addressed. In Chapter Four, the general problem of performance according to traditional reliability theory was reviewed, including a discussion of the nature of uncertainty in structural engineering models. The application of reliability analysis to structural systems was also presented, where it was argued that the data requirements and mathematical complexity of probability-based structural reliability precludes a strict application of the theory to WLFC. Alternate methods of assessing expected structural performance in the face of uncertainty, such as fuzzy sets, seem to provide more suitable methods for application to the performance assessment of wood light-frame structural systems.

Ways to explicitly consider the adverse influence of human action on the performance of the constructed artifact were presented in Chapter Five. The state-of-the-art in integrating the effects of adverse human action in structural reliability models was reviewed. The probabilistic models presented rely heavily on traditional reliability theory and often make unrealistic assumptions regarding the sources and nature of human error in the building process. Process simulation and fuzzy rule-based modeling are methodologies that might provide a more flexible framework for such studies, although no such applications were found in the literature.

In Chapter Six, structural quality assurance was presented as the basis of an approach to assess the expected performance of WLFC subject to extreme wind hazards. A methodology was proposed that uses fuzzy sets to account for human-based uncertainties in the building process. The effect of human factors on the resistance parameters of a simple structural model leads to fuzzy margins that are used to calculate component *failure possibilities*. The methodology was applied to assess the expected performance of a typical gable roof sheathing system subjected to an extreme wind load. The performance of the sheathing system under five hypothetical design and construction error scenarios was evaluated. The results of the analyses generally conform with expectations and are in line with observed instances of hurricane-induced sheathing panel failure.

There were, however, discrepancies between the two fuzzy-set based performance measures considered in the analysis. This suggests that effective application of the methodology is dependent upon the types of fuzzy-based measures used to define system performance. The weakest-link model of system performance seems to provide more accurate results than the proposed weighted-average index. Nonetheless, the gable roof example illustrates a specific application of the proposed methodology, the results of which suggest the suitability of fuzzy mathematics for use in structural models that consider the influence of human factors on the performance of WLFC subject to extreme wind hazards.

Further Work

The development of the proposed methodology identifies several areas where more research is needed. In particular, increased use of social science tools to better understand the building process seems to be necessary if such issues are to be considered in an engineering study. In addition, research to increase current understanding of wind loads on low-rise buildings is necessary and should continue. The effective application of this knowledge to reduce the vulnerability of both new and existing WLFC is multidisciplinary in nature as it requires a consideration of issues related to preparedness policy and acceptable risk, cost-effective construction and retrofitting strategies, and general popular perception of the hurricane hazard. Dialogue between such fields of study is necessary and should continue.

An increased understanding of combined wind load effects on wood light-frame structural systems is needed to develop more sophisticated and realistic models of structural response. The simple roof sheathing uplift resistance model presented in Chapter Six does not consider combined effects, e.g. decreased withdrawal capacity of fasteners subjected to racking loads. Consideration of such issues would lead to improved response models and hence more realistic representations of overall structural performance.

While much work has been done to develop reliability models of repetitive member systems and subassemblies (e.g. walls and roofs), there is a lack of understanding how these subassemblies interact as a total building system. Improved methods to evaluate system reliability therefore seem necessary. This is a general problem within structural reliability theory, which continues to be an active field of study. The recent trend to consider the influence of human factors in structural reliability has prompted researchers to consider different approaches and should therefore be encouraged. It seems fuzzy sets and rule-based modeling might provide alternate ways to address uncertainty in structural reliability models.

Most of all, the use of fuzzy sets for structural performance evaluation of wood light-frame structural systems should be developed further. Fuzzy sets and fuzzy rule-based modeling is a burgeoning field of study in structural engineering science, and it seems further work would provide needed insight in places where traditional structural reliability theory falls short. This seems to be especially true for the consideration of adverse human action in structural performance models.

APPENDIX A

The Vertex Method

Calculations with fuzzy numbers adhere to the calculus of fuzzy sets and may involve mathematical operations different from standard "crisp" algebra. Fuzzy mathematical operations are mappings typically based on fuzzy interval mathematics in order to produce a membership function for the resulting parameter. The vertex method developed by Dong et al. (Dong and Shah 1987; Dong and Wong 1987) is one such technique that applies traditional function mapping to fuzzy parameters in such a way that the analysis can proceed in determinisitic iterations. The resolution of the calculated parameter membership function is determined by the number of α-cuts used in the mapping (Wadia-Fascetti and Smith 1994).

As this thesis uses only triangular fuzzy numbers (TFNs), this appendix treats the implementation of the vertex method for only two α-cuts since, by definition, TFNs are defined by values at α-cuts 0^+ and 1.0. A simple example of subtracting two TFNs is provided below to illustrate how the vertex method is used in this thesis.

Consider a fuzzy parameter $\tilde{M}$ that is the difference between quantities $\tilde{R}$ and $\tilde{S}$. Let $\tilde{R}$ and $\tilde{S}$ be defined as TFNs such that $\tilde{R} = (r_1, r_2, r_3)_T$ and $\tilde{S} = (s_1, s_2, s_3)_T$. Assume these TFNs have parameters $\tilde{R} = (0, 3, 4)_T$ and $\tilde{S} = (1, 2, 5)_T$, as listed below:

Table A.1

Fuzzy Paramters of $\tilde{R}$ and $\tilde{S}$

α-cut	Lower Bound $\tilde{R}$	Lower Bound $\tilde{S}$	Upper Bound $\tilde{R}$	Upper Bound $\tilde{S}$
0^+	0	1	4	5
1.0	3	2	3	2

Using the vertex method at the α-cuts 0^+ and 1.0, the function that defines the resulting fuzzy number $\tilde{M} = \tilde{R} - \tilde{S}$ is performed five times (Table A.2). The calculated values are compared at α-cuts 0^+ and 1.0. The minimum and maximum values provide the lower and upper bounds, respectively. (Other measures need to be taken if the mapping function is nonlinear.) The results are summarized below and depicted graphically in Figure A.1

Table A.2

Calculation Summary for $\tilde{M} = \tilde{R} - \tilde{S}$ According to the Vertex Method

α-cut	$\tilde{R}$	$\tilde{S}$	$\tilde{M}$	Resulting Bound
0^+	0	1	- 1	-
0^+	0	5	- 5	Lower (min)
0^+	4	1	3	Upper (max)
0^+	4	5	-1	-
1.0	3	2	1	Singular Bound

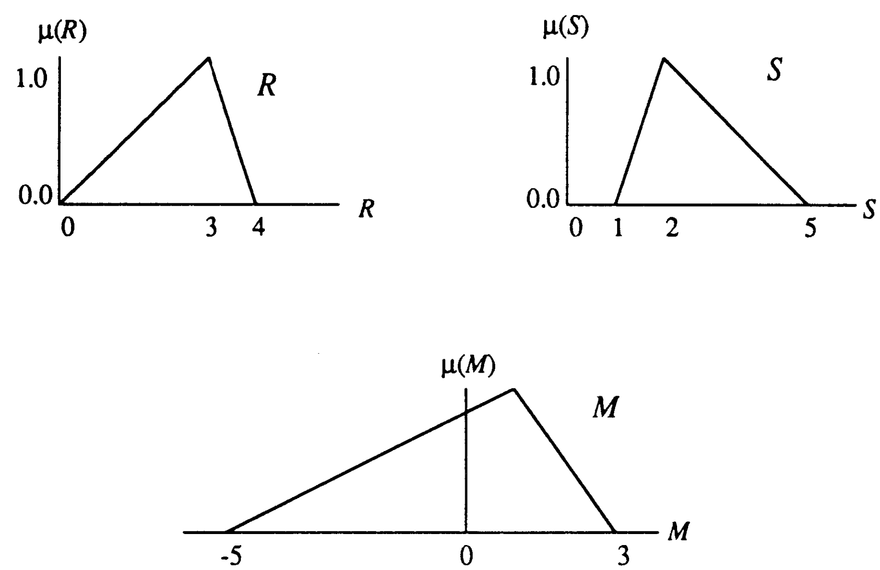
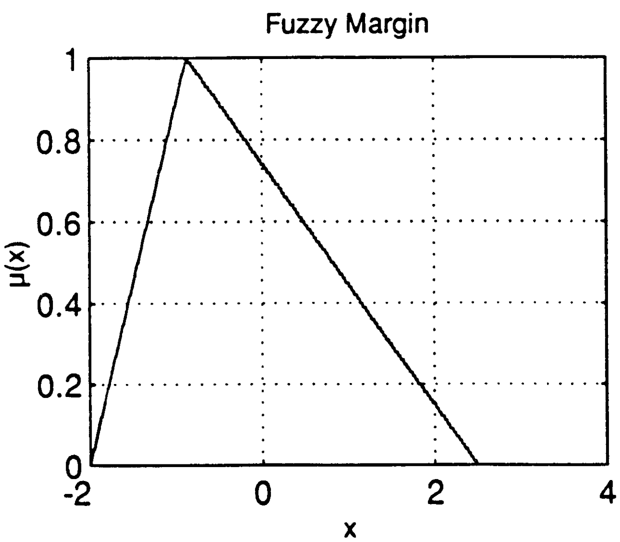


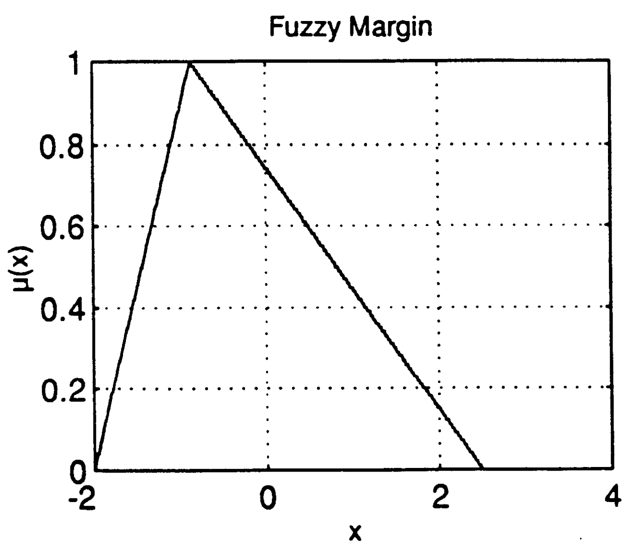
Figure A.1
Schematic representation of TFN vertex subtraction

APPENDIX B

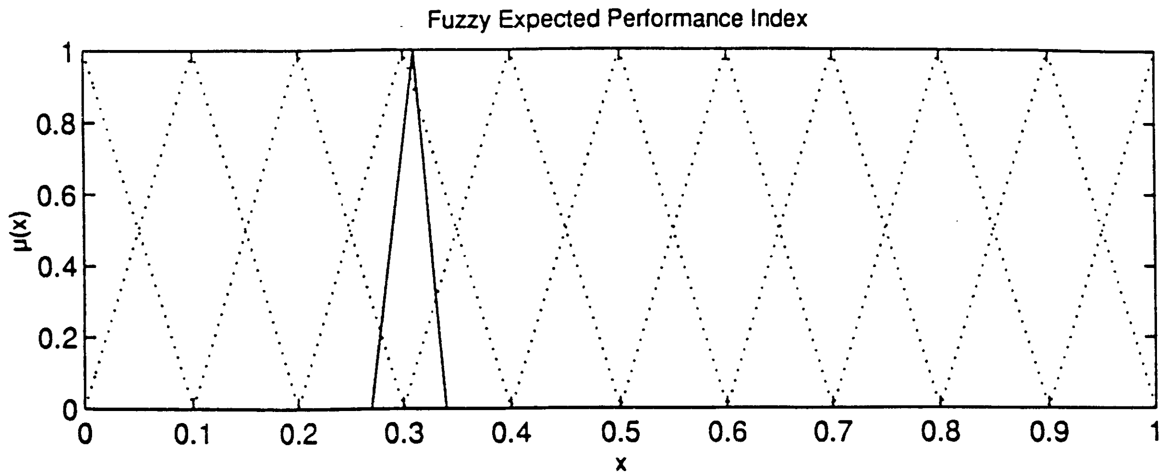
Sample MATLAB Input and Output

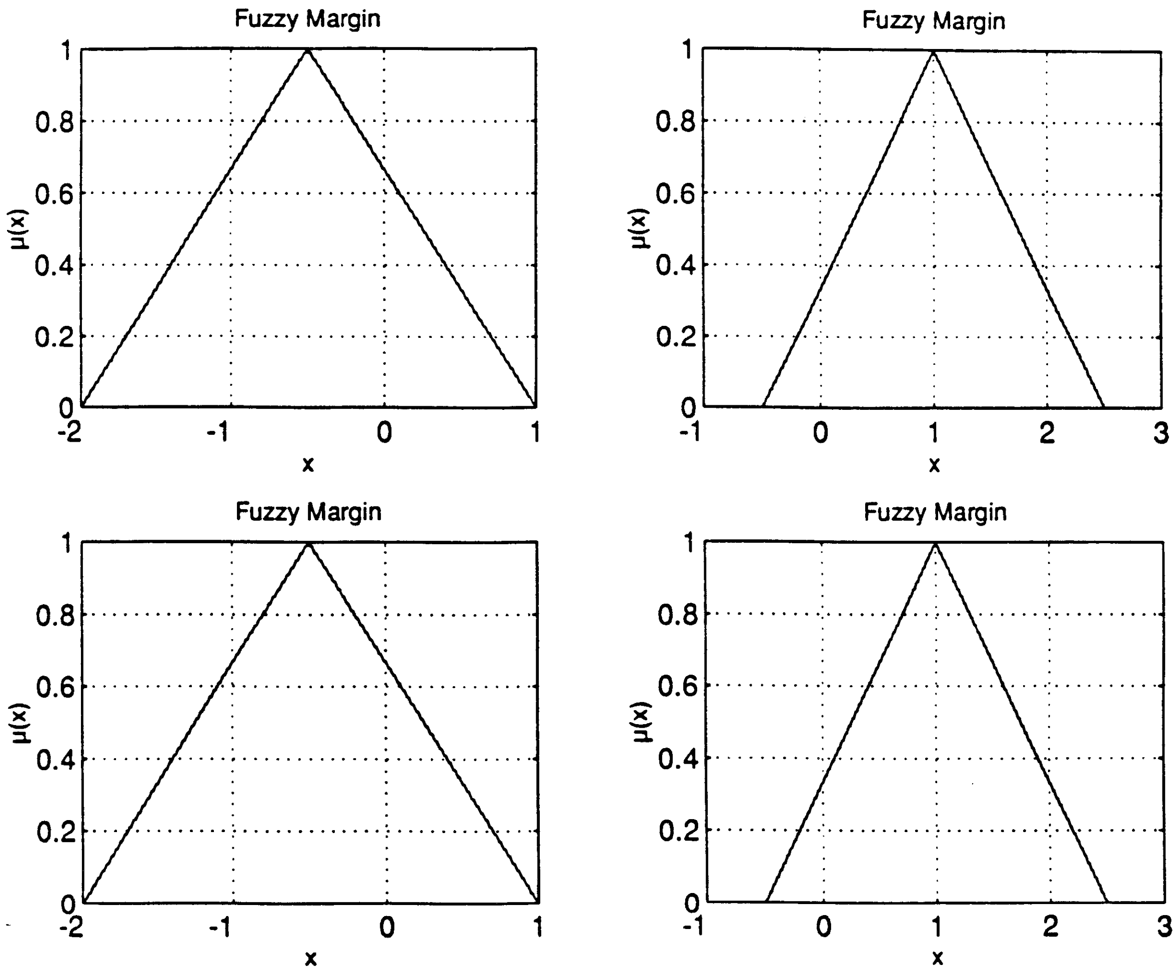


Fuzzy Margin
1
0.8
0.6
0.4
0.2
0
μ(x)
-2
0
2
4
x

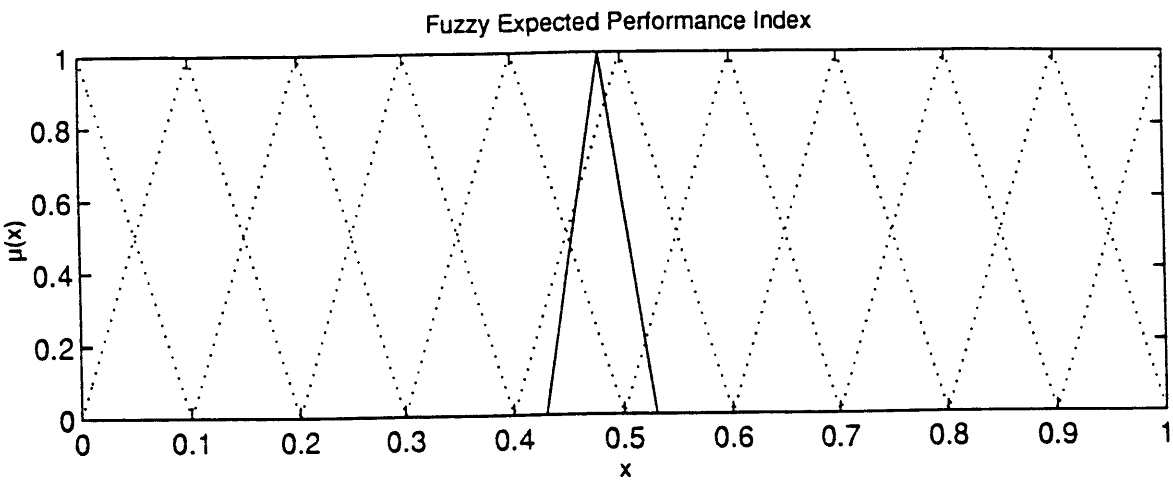


Fuzzy Margin
1
0.8
0.6
0.4
0.2
0
μ(x)
-2
0
2
4
x


Fuzzy Expected Performance Index
μ(x)
x
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
0.2
0.4
0.6
0.8



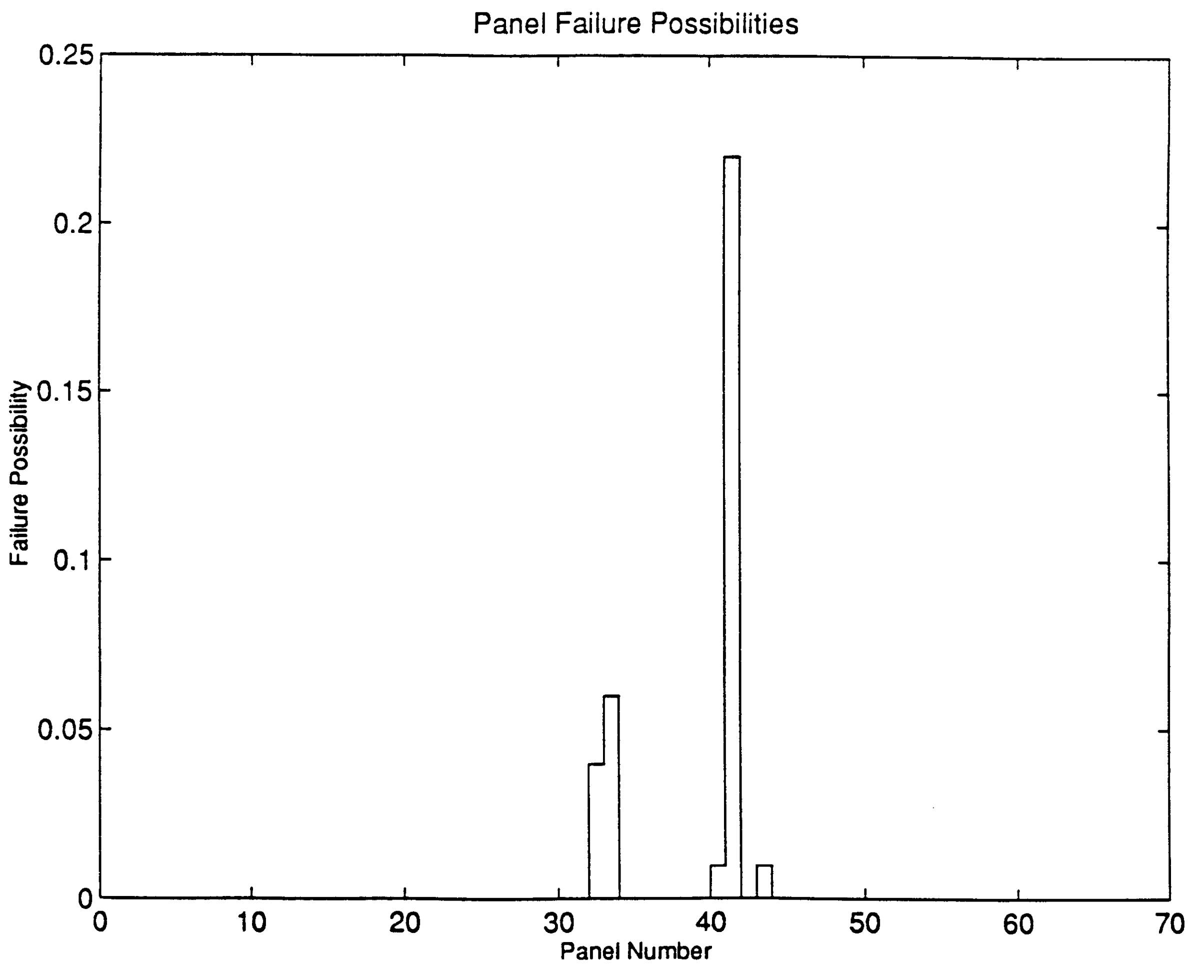
Fuzzy Margin
μ(x)
x
1
0.8
0.6
0.4
0.2
0
-2
-1
0
1
2
3

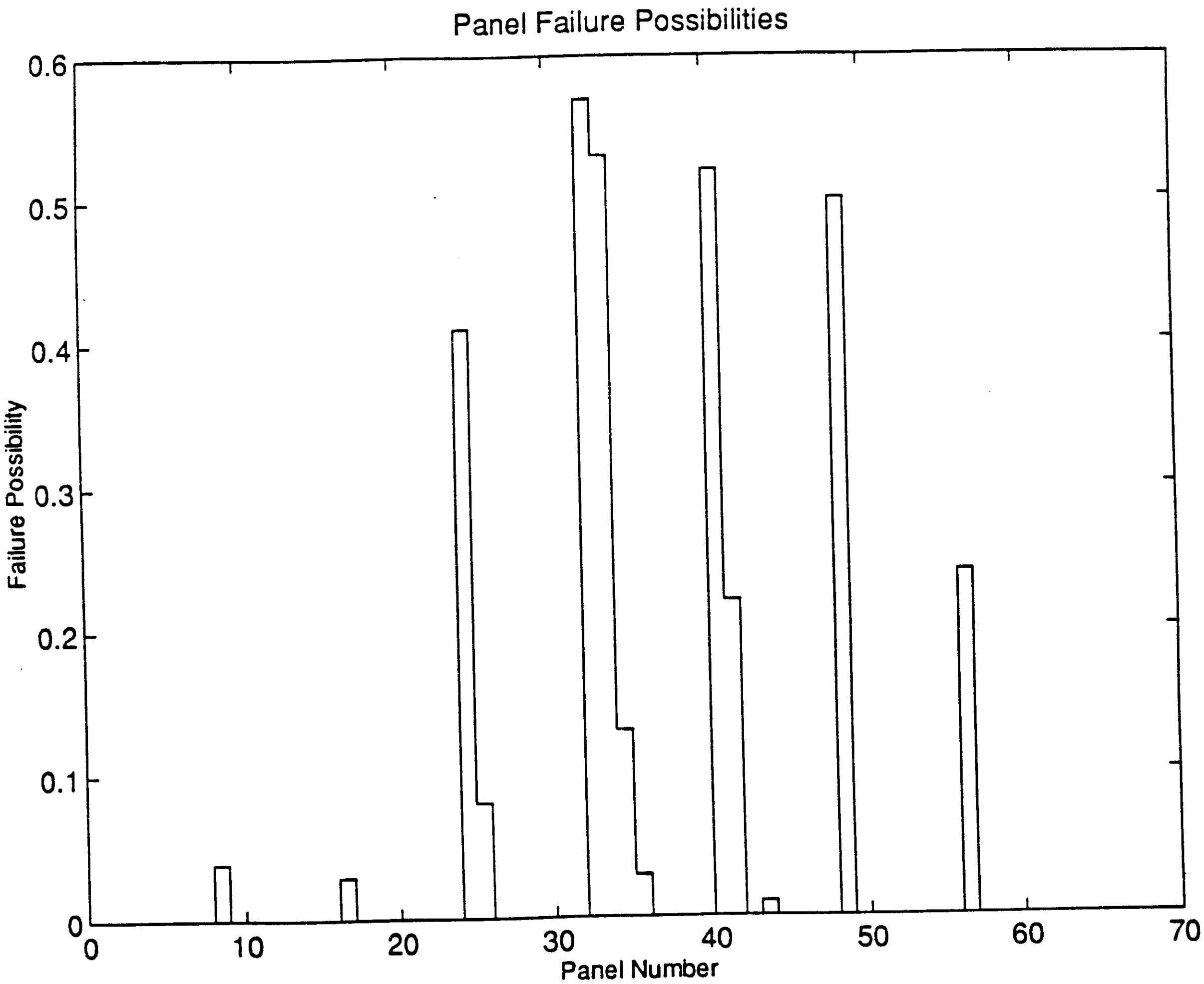

Fuzzy Expected Performance Index
μ(x)
1
0.8
0.6
0.4
0.2
0
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
x

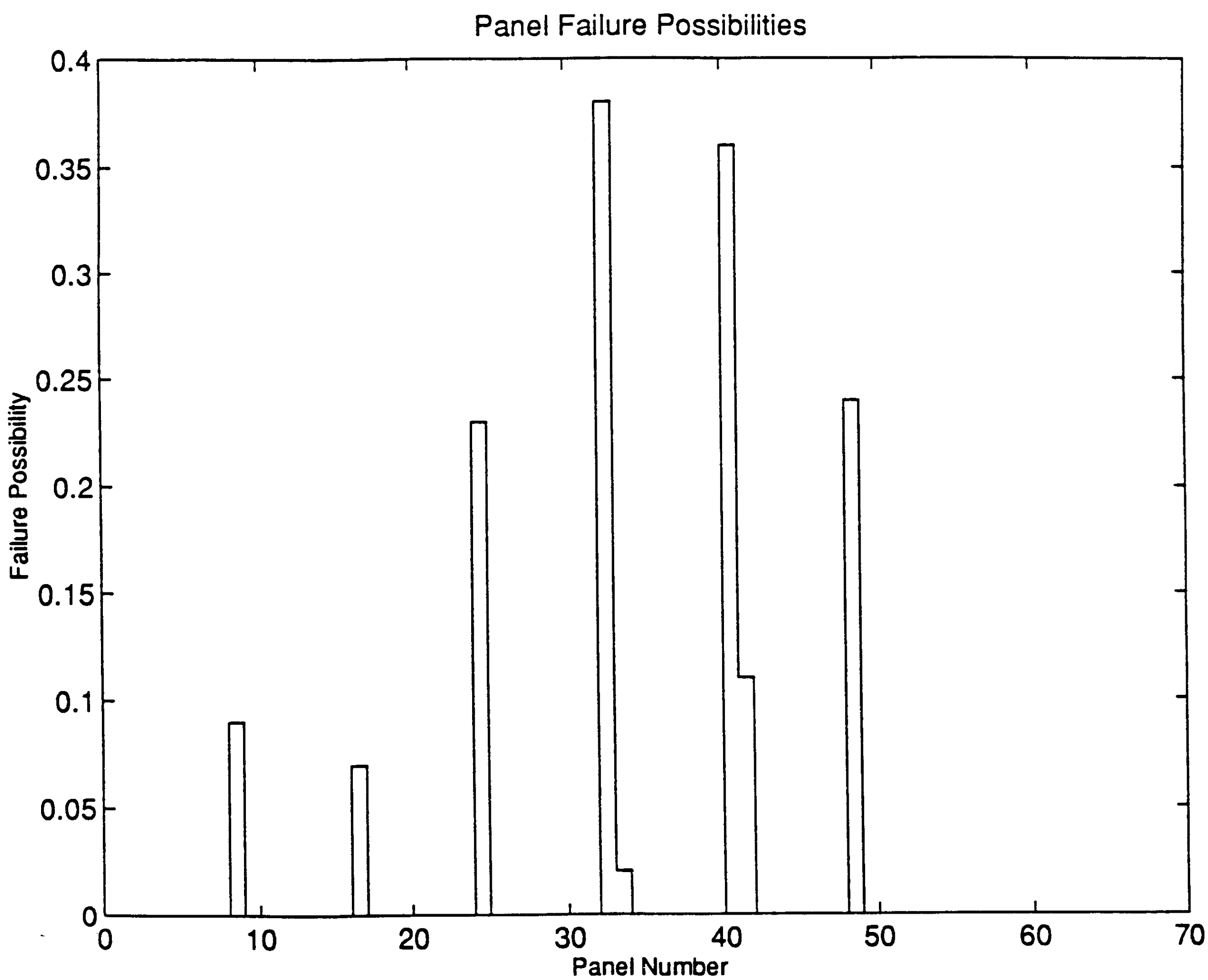
Assigned TFN Parameters of Panel Pressure Coefficients

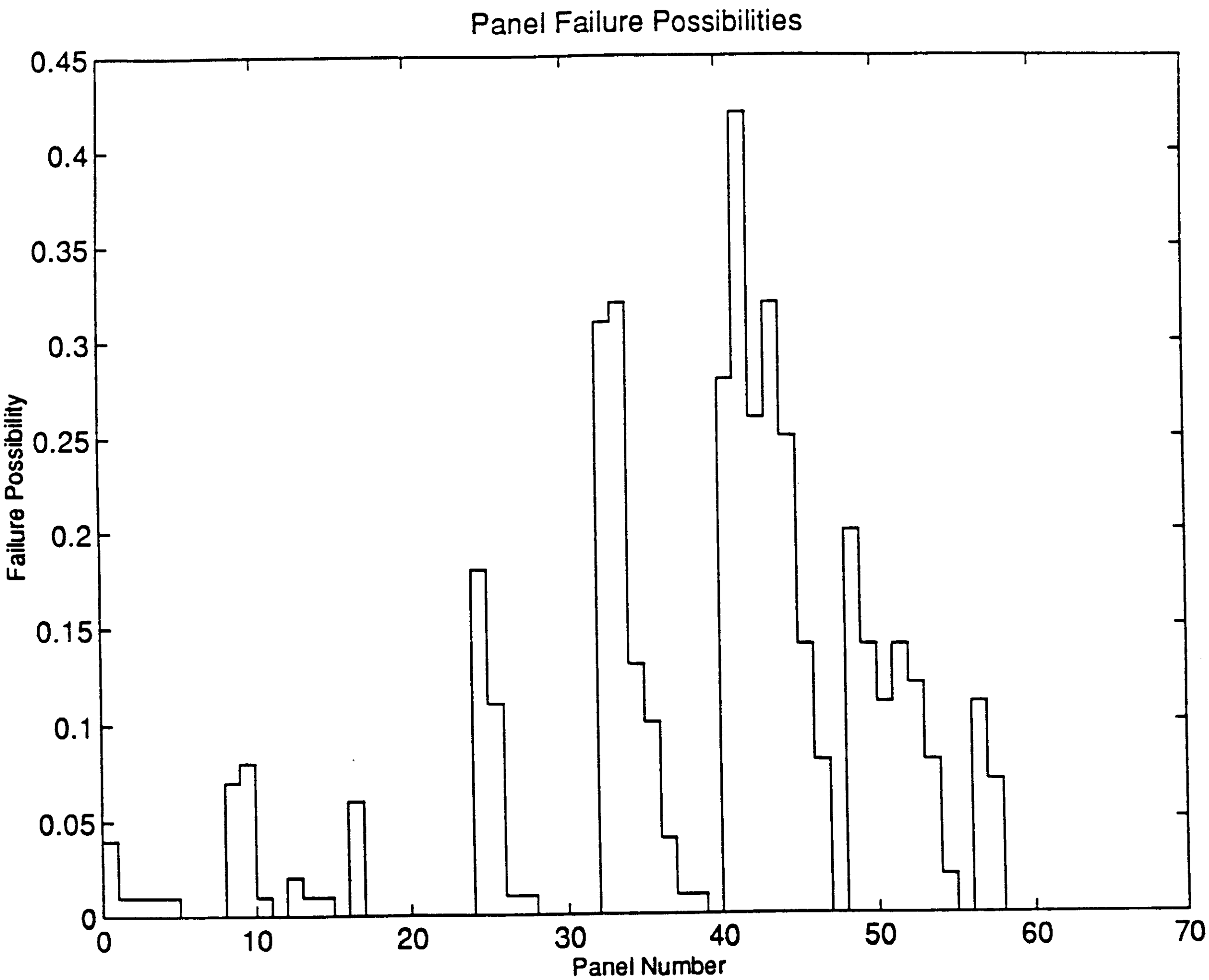
(by sign convention, all values below are negative)

Panel	C1	C2	C3	Panel	C1	C2	C3	Panel	C1	C2	C3	Panel	C1	C2	C3
1	1.80	2.20	3.10	17	0.80	2.20	3.60	33	3.10	4.50	5.40	49	2.50	3.30	5.00
2	1.50	2.00	2.20	18	0.70	0.80	1.40	34	2.90	4.00	5.10	50	1.70	1.90	2.70
3	1.40	2.00	2.20	19	0.70	0.80	1.30	35	3.20	3.30	3.40	51	1.60	2.00	2.50
4	1.30	1.80	2.20	20	0.70	0.90	1.30	36	2.70	2.90	3.30	52	1.70	2.10	2.60
5	1.40	1.80	2.30	21	1.10	1.20	1.30	37	2.30	2.50	2.70	53	1.80	2.20	2.40
6	1.40	1.70	1.80	22	1.10	1.30	1.30	38	2.00	2.20	2.30	54	1.50	1.90	2.20
7	1.30	1.60	1.80	23	0.70	0.80	1.10	39	1.80	1.90	2.20	55	1.30	1.50	1.70
8	1.30	1.60	1.80	24	0.80	0.90	1.10	40	1.70	1.70	1.80	56	1.40	1.60	1.70
9	2.40	2.90	3.40	25	3.00	3.70	4.40	41	3.20	4.50	5.00	57	3.10	3.40	3.70
10	0.80	1.50	2.50	26	1.00	2.50	3.90	42	1.80	3.00	4.50	58	1.80	2.40	3.20
11	0.80	1.70	1.50	27	1.00	1.80	2.40	43	1.90	2.70	3.20	59	1.40	1.60	1.80
12	0.80	1.10	1.40	28	1.20	1.70	2.30	44	2.50	3.00	3.20	60	1.30	1.40	1.60
13	0.90	1.30	1.70	29	1.30	1.50	1.70	45	2.30	2.70	3.00	61	1.30	1.50	1.90
14	1.20	1.40	1.60	30	1.20	1.40	1.60	46	2.10	2.20	2.50	62	1.60	1.70	1.80
15	0.80	1.20	1.50	31	1.00	1.20	1.40	47	1.70	1.80	2.20	63	1.60	1.70	1.80
16	0.70	1.00	1.30	32	1.00	1.20	1.40	48	1.70	1.70	1.80	64	1.30	1.30	1.40

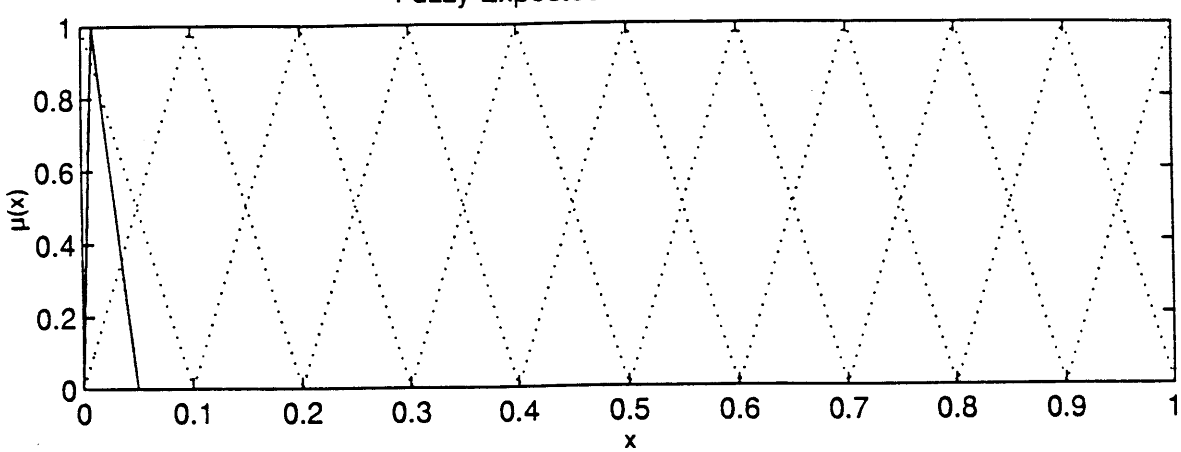

Panel Failure Possibilities
Failure Possibility
0.25
0.2
0.15
0.1
0.05
0
0
10
20
30
40
50
60
70
Panel Number


Panel Failure Possibilities
0.6
0.5
0.4
0.3
0.2
0.1
0
Failure Possibility
0
10
20
30
40
50
60
70
Panel Number


Panel Failure Possibilities
Failure Possibility
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0
10
20
30
40
50
60
70
Panel Number


Panel Failure Possibilities
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
Failure Possibility
0
10
20
30
40
50
60
70
Panel Number

Fuzzy Expected Performance Index



APPENDIX C

Sample MATLAB Program Listings

The programs listed in this appendix were written in MATLAB for the Macintosh, version 4.2c1 ("MATLAB" 1992) on a Machintosh IIci.

Program Listings:

External (Custom) Function Listings:

frame.m PROGRAM LISTING

```
% FRAME - Fuzzy reliability assessment of simple pin-connected frame
% using fuzzy triangular numbers.  FRAME outputs membership plots of
% the fuzzy reduction factors, fuzzy margins that interact with zero
% and a graphical mapping of the fuzzy expected performance index
% to reference fuzzy sets the linguistic domain defined along [0,1].
% This program uses the following input parameters:
%
%                  h = member height
%                  l = member length
%                 P1 = vertical point load
%                 P2 = horizontal point load
%              gamma = safety factor
%                inc = domain increment
%
%      [a11 a12 a13] = tfn phi1
%      [a21 a22 a23] = tfn phi2
%      [a31 a32 a33] = tfn phi3
%      [a41 a42 a43] = tfn phi4
%
%      [w11 w12 w13] = tfn weight1
%      [w21 w22 w23] = tfn weight2
%      [w31 w32 w33] = tfn weight3
%      [w41 w42 w43] = tfn weight4
%
% The program also calls the following external (custom) functions:
%
%        tfn.m - Triangular Fuzzy Number
%                Evaluates the membership of a triangular fuzzy number
%
%       rdtn.m - Round to Tens
%                Rounds calculations to the tens decimal place
%
%   tvertadd.m - Triangular Vertex Addition
%                Adds triangular fuzzy numbers via the Vertex method
%
%   tvertdiv.m - Triangular Vertex Division
%                Divides triangular fuzzy numbers via the vertex method

% Assign input variables

h = 10;
l = 10;
P1 = 10;
P2 = 10;
inc = 0.01;
gamma = 1.5;
a11 = 0.40; a12 = 0.55; a13 = 1.00;
a21 = 0.85; a22 = 0.90; a23 = 1.00;
a31 = 0.40; a32 = 0.55; a33 = 1.00;
a41 = 0.85; a42 = 0.90; a43 = 1.00;
w11 = 0.80; w12 = 0.90; w13 = 1.00;
w21 = 0.70; w22 = 0.80; w23 = 0.90;
w31 = 0.80; w32 = 0.90; w33 = 1.00;
w41 = 0.70; w42 = 0.80; w43 = 0.90;
PHI = [a11 a12 a13; a21 a22 a23; a31 a32 a33; a41 a42 a43];
  W = [w11 w12 w13; w21 w22 w23; w31 w32 w33; w41 w42 w43];
```

```
% Calculate joint reactions

P = [P1 P2];
L = [l/(4*h) 1/4 -l/(4*h) 3/4; 1/4 h/(4*l) 3/4 -h/(4*l)];
U = P*L;

% Set nominal resistances Ri's equal to gamma times the joint reactions
and nominal
% loads to the joint reactions

R = zeros(size(U));
S = zeros(size(U));

for k = 1: length(U)
      R(k) = gamma*U(k);
      S(k) = U(k);
end

% Calculate fuzzy resistances FRi's and fuzzy margins FMi's

FR = zeros(size(PHI));
FM = zeros(size(PHI));

for k = 1: length(U)
      for j = 1: length(PHI(1,:))
            FR(k,j) = rdtn(R(k)*PHI(k,j));
            FM(k,j) = rdtn(FR(k,j) - S(k));
      end
end

% Establish domains for fuzzy margins that interact with zero (i.e.
% elements i that have a nonzero failure possibility).

FP = zeros(size(U));

for k = 1: length(U)
      if (FM(k,1)) < 0 & (FM(k,2)) > 0
            X(k,:) = (FM(k,1):inc:FM(k,3));
           fm(k,:) = rdtn(tfn(X(k,:), [FM(k,1) FM(k,2) FM(k,3)]));
            subplot(2,2,k), plot(X(k,:), fm(k,:)), grid on;
            xlabel('x', 'Fontsize',10);
            ylabel('µ(x)', 'Fontsize', 10);
            title('Fuzzy Margin', 'Fontsize', 10);
               temp = find(X(k,:) == 0);
                  l = X(k,temp)-FM(k,1);
                  h = fm(k,temp);
              FP(k) = rdtn((l*h)/((FM(k,2)-FM(k,1))+(FM(k,3)-FM(k,2))));
      elseif (FM(k,1)) < 0 & (FM(k,2)) < 0
            X(k,:) = (FM(k,1):inc:FM(k,3));
           fm(k,:) = rdtn(tfn(X(k,:), [FM(k,1) FM(k,2) FM(k,3)]));
            subplot(2,2,k), plot(X(k,:), fm(k,:)), grid on;
            xlabel('x', 'Fontsize',10);
            ylabel('µ(x)', 'Fontsize', 10);
            title('Fuzzy Margin', 'Fontsize', 10);
               temp1 = find(max(fm(k,:)));
               temp2 = find(X(k,:) == 0);
                  l1 = X(k,temp2) - FM(k,1);
                  l2 = X(k,temp2) - X(k,temp1);
```

```
                h = fm(k,temp2);
            area1 = l1/2;
            area2 = (l2*h)+(((1-h)*l2)/2);
        FP(k) = rdtn((area1+area2)/(FM(k,2)-FM(k,1)+FM(k,3)-FM(k,2)));
    else
          FP(k) = 0;
         X(k,:) = zeros(size(X(1,:)));
        fm(k,:) = zeros(size(X(1,:)));
    end
    set (gcf, 'name', 'Fuzzy Margins', 'numbertitle', 'off');
end

% Evaluate system failure possibility SFP

SFP = max(FP)

% Evaluate fuzzy expected performance index

WFP = zeros(size(PHI));

for k = 1: length(U)
    for j = 1: 3
        WFP(k,j) = W(k,j)*FP(k);
    end
end

I = rdtn(tvertdiv(tvertadd(WFP), tvertadd(W)));

% Establish domain of fuzzy expected performance index in [0,1]

x = rdtn((0:inc:1));

% Define reference triangular fuzzy sets

AG = tfn(x, [0.0 0.0 0.1]);          % "ABSOLUTELY GOOD"
EG = tfn(x, [0.0 0.1 0.2]);          % "EXTREMELY GOOD"
VG = tfn(x, [0.1 0.2 0.3]);          % "VERY GOOD"
 G = tfn(x, [0.2 0.3 0.4]);          % "GOOD"
FG = tfn(x, [0.3 0.4 0.5]);          % "FAIRLY GOOD"
 M = tfn(x, [0.4 0.5 0.6]);          % "MEDIUM"
Fp = tfn(x, [0.5 0.6 0.7]);          % "FAIRLY POOR"
 P = tfn(x, [0.6 0.7 0.8]);          % "POOR"
VP = tfn(x, [0.7 0.8 0.9]);          % "VERY POOR"
EP = tfn(x, [0.8 0.9 1.0]);          % "EXTREMELY POOR"
AP = tfn(x, [0.9 1.0 1.0]);          % "ABSOLUTELY POOR"

REF = zeros(11,3);
REF = [AG; EG; VG; G; FG; M; Fp; P; VP; EP; AP];

% Map to linguistic domain
figure;

lingI = rdtn(tfn(x,I));
subplot(2,1,1), plot(x, [REF], 'y:', x, lingI, 'c-'), grid off;
xlabel('x', 'Fontsize',10);
ylabel('µ(x)', 'Fontsize', 10);
title('Fuzzy Expected Performance Index', 'Fontsize', 10);
set (gcf, 'name', 'Peformance Index', 'numbertitle', 'off');
```

gable.m PROGRAM LISTING

```
% GABLE - Fuzzy reliability assessment of gable roof sheathing system
% using fuzzy triangular numbers.  GABLE outputs membership plots of
% the fuzzy margins that interact with zero and a graphical mapping
% of the fuzzy expected performance index to reference fuzzy sets in
% the linguistic domain defined along [0,1].  The required input
% parameters are defined as follows:
%
%  Prototype Geometric Parameters:
%
%                d = truss or rafter spacing [ft]
%                n = number of sheathing panels
%
%  Wind Load Parameters:
%
%                q = reference velocity wind pressure [lb/sq.ft]
%              [C] = matrix of tfn panel pressure coefficients
%
%  Code-Specified Strength Parameters:
%
%                c = sheathing fastener withdrawal capacity [lb]
%        epsilon_e = specified edge fastener spacing  [fasteners/ft]
%        epsilon_f = specified field fastener spacing [fasteners/ft]
%        epsilon_g = specified gable fastener spacing [fasteners/ft]
%
%  Performance Parameters:
%
%     [wa1 wa2 wa3] = tfn weight A; "extremely important"
%     [wb1 wb2 wb3] = tfn weight B; "very important"
%     [wc1 wc2 wc3] = tfn weight C; "important"
%
%     [a11 a12 a13] = triangular fuzzy reduction factor phi1
%     [a21 a22 a23] = triangular fuzzy reduction factor phi2
%     [a31 a32 a33] = triangular fuzzy reduction factor phi3
%     [a41 a42 a43] = triangular fuzzy reduction factor phi4
%
%               inc = domain increment for interval [0,1]
%
% The program also calls the following external (custom) functions:
%
%       tfn.m - Triangular Fuzzy Number
%               Evaluates the membership of a triangular fuzzy number
%
%      rdtn.m - Round to Tens
%               Rounds calculations to the tens decimal place
%
%  tvertadd.m - Triangular Vertex Addition
%               Adds triangular fuzzy numbers via the Vertex method
%
%  tvertdiv.m - Triangular Vertex Division
%               Divides triangular fuzzy numbers via the Vertex method
%
% The script (MATLAB .m) files of these functions are provided at the
% end of the program listings.
```

```
% Assign input variables

d = 2;
n = 64;
q = 37;
a = 4;                        % standard panel width of 4 feet
s = 8;                        % standard panel length of 8 feet

% read pressure coefficients from file 'coefficients.dat'

fid = fopen('coefficients.dat');
  C = fscanf(fid, '%f %f %f', [3 64]);
fclose(fid);

c = 262;
epsilon_e = 2;
epsilon_f = 1;
epsilon_g = 3;

wa1 = 0.80; wa2 = 0.90; wa3 = 1.00; WA = [wa1 wa2 wa3];
wb1 = 0.70; wb2 = 0.80; wb3 = 0.90; WB = [wb1 wb2 wb3];
wc1 = 0.60; wc2 = 0.70; wc3 = 0.80; WC = [wc1 wc2 wc3];

a11 = 0.20; a12 = 0.50; a13 = 0.50; phi1 = [a11 a12 a13];
a21 = 0.20; a22 = 0.50; a23 = 0.50; phi2 = [a21 a22 a23];
a31 = 0.20; a32 = 0.50; a33 = 0.50; phi3 = [a31 a32 a33];

inc = 1;

% Define reduction scenarios

s1 = zeros(3,8);
s2 = zeros(3,8);

for j = 1: 3
      s1(j,:) = [phi1(j) phi2(j) phi2(j) phi2(j) phi2(j) phi2(j) phi2(j)
phi1(j)];
      for k = 1: 8
            s2(j,k) = phi3(j);
      end
end

% Assign weights and fuzzy reduction factors

we = zeros(3,8);
wf = zeros(3,8);
  W = zeros(3,n);
PHI = zeros(3,n);

for k = 1: 3
        we(k,:) = [WA(k) WB(k) WB(k) WB(k) WB(k) WB(k) WB(k) WA(k)];
        wf(k,:) = [WA(k) WB(k) WC(k) WC(k) WC(k) WC(k) WB(k) WA(k)];
end

  W = [we wf wf wf wf wf wf we];
PHI = [s2 s1 s1 s1 s1 s1 s1 s2];
```

```
% Calculate nominal Ris

zone1R = a*c*(2*epsilon_e+epsilon_f*((s-d)/d));
zone2R = a*c*epsilon_e*(2+((s-d)/d));
zone3R = a*c*(epsilon_g+(epsilon_e)*(1+((s-d)/d)));

R = zeros(1,n);

% Gable end panels are in zone 3

 R(1)  = zone3R;    R(8)  = zone3R; R(9)  = zone3R;  R(16) = zone3R;
R(17) = zone3R;   R(24) = zone3R; R(25) = zone3R; R(32) = zone3R;
R(33) = zone3R;   R(40) = zone3R; R(41) = zone3R; R(48) = zone3R;
R(49) = zone3R;   R(56) = zone3R; R(57) = zone3R; R(64) = zone3R;

% Perimeter panels are in zone 2

for k = 2: 7
      R(k) = zone2R;
end

for k = 26: 31
      R(k) = zone2R;
end

for k = 34: 39
      R(k) = zone2R;
end

for k = 58: 63
      R(k) = zone2R;
end

% Remaining field are in zone 1

for k = 10: 15
      R(k) = zone1R;
end

for k = 18: 23
      R(k) = zone1R;
end

for k = 42: 47
      R(k) = zone1R;
end

for k = 50: 55
      R(k) = zone1R;
end

% Calculate fuzzy resistances and fuzzy load effects

FR = zeros(3,n);
FS = zeros(3,n);
FM = zeros(3,n);
```

```
for j = 1: 3
      for k = 1: n
            FR(j,k) = PHI(j,k)*R(k);
            FS(j,k) = q*s*a*C(j,k);
      end
end

% Evaluate fuzzy margins

for k = 1: n
      FM(:,k) = tvertsub(FR(:,k), FS(:,k));
end

% Establish domains for and calculate failure possibilities of fuzzy
% margins that interact with zero

for k = 1: n
      if (FM(1,k)) < 0 & (FM(2,k)) > 0
                  X = round((FM(1,k):inc:FM(3,k)));
                 fm = round(100*(tfn(X, [FM(1,k) FM(2,k) FM(3,k)])))/100;
               temp = find(X == 0);
                  l = X(temp) - FM(1,k);
                  h = fm(temp);
              FP(k) = rdtn((l*h)/((FM(2,k)-FM(1,k))+(FM(3,k)-FM(2,k))));
      elseif (FM(1,k)) < 0 & (FM(2,k)) < 0
                  X = round((FM(1,k):inc:FM(3,k)));
                 fm = round(100*(tfn(X, [FM(1,k) FM(2,k) FM(3,k)])))/100;
              templ = find(max(fm));
              temp2 = find(X == 0);
                 l1 = X(temp2) - FM(1,k);
                 l2 = X(temp2) - X(templ);
                  h = fm(temp2);
              areal = l1/2;
              area2 = (l2*h)+(((1-h)*l2)/2);
            FP(k) = rdtn((areal+area2)/(FM(2,k)-FM(1,k)+FM(3,k)-FM(2,k)));
      else
            FP(k) = 0;
      end
end

% Stairstep plot of panel failure possibilities

stairs(FP);
xlabel('Panel Number', 'Fontsize',10);
ylabel('Failure Possibility', 'Fontsize', 10);
title('Panel Failure Possibilities', 'Fontsize', 12);

% Evaluate and return system failure possibility SFP

SFP = max(FP)

% Evaluate fuzzy expected performance index

WFP = zeros(3,n);

for j = 1: 3
      for k = 1: n
            WFP(j,k) = W(j,k)*FP(k);
```

```
      end
end

I = rdtn(tvertdiv(tvertadd(WFP), tvertadd(W)))

% Establish domain of fuzzy expected performance index in [0,1]

x = rdtn((0:0.01:1));

% Define reference expected peformance tfns

AG = tfn(x, [0.0 0.0 0.1]);          % "ABSOLUTELY GOOD"
EG = tfn(x, [0.0 0.1 0.2]);          % "EXTREMELY GOOD"
VG = tfn(x, [0.1 0.2 0.3]);          % "VERY GOOD"
 G = tfn(x, [0.2 0.3 0.4]);          % "GOOD"
FG = tfn(x, [0.3 0.4 0.5]);          % "FAIRLY GOOD"
 M = tfn(x, [0.4 0.5 0.6]);    % "MEDIUM"
Fp = tfn(x, [0.5 0.6 0.7]);          % "FAIRLY POOR"
 P = tfn(x, [0.6 0.7 0.8]);          % "POOR"
VP = tfn(x, [0.7 0.8 0.9]);          % "VERY POOR"
EP = tfn(x, [0.8 0.9 1.0]);          % "EXTREMELY POOR"
AP = tfn(x, [0.9 1.0 1.0]);          % "ABSOLUTELY POOR"

REF = zeros(11,3);
REF = [AG; EG; VG; G; FG; M; Fp; P; VP; EP; AP];

% Map to linguistic domain

figure;

lingI = rdtn(tfn(x,I));
subplot(2,1,1), plot(x, [REF], 'y:', x, lingI, 'c-'), grid off;
xlabel('x', 'Fontsize',10);
ylabel('μ(x)', 'Fontsize', 10);
title('Fuzzy Expected Performance Index', 'Fontsize', 12);
set (gcf, 'name', 'Peformance Index', 'numbertitle', 'off');
```

FUNCTIONS LISTING

```
function mu = tfn(x, params)
% TFN - Triangular Fuzzy Number
% TFN(X, PARAMS) returns a vector of memberships of a triangular
% fuzzy number.  PARAMS = [A1 A2 A3] is a 3-element vector that
% determines the membership function of the triangular fuzzy number.
% TFN(X, PARAMS) requires that a1 < a2 < a3.
%
% This function is partially based on the TRIMF function of the
% MATLAB Fuzzy Logic Toolbox by Roger Jang, (c) 1994-95 by the
% MathWorks, Inc.

a1 = params(1); a2 = params(2); a3 = params(3);
mu = zeros(size(x));

% Zero membership for x < a1 or x > a3

temp = find(x <= a1 | a3 <= x);
mu(temp) = zeros(size(temp));

% L-R membership values

if (a1 ~= a2)
      temp = find(a1 < x & x <a2);
      mu(temp) = (x(temp)-a1)/(a2-a1);
end

if (a2 ~= a3)
      temp = find(a2 < x & x < a3);
      mu(temp) = (a3-x(temp))/(a3-a2);
end

% The center value a2 always has membership of unity

temp = find(x == a2);
mu(temp) = ones(size(temp));


function y = tvertadd(tfns)
%  TVERTADD(TFNS)  returns a 3-element vector that defines a triangular
%  fuzzy number corresponding to the sum of m ordered tfns.
%  PARAMS = [a11 a12 a13; ... ; am1 am2  am3] is a matrix of m
%  triangular fuzzy numbers.  TVERTADD requires at least one aij to be
%  nonzero and for tfn A < tfnb B, a1 < b1, a2 < b2, and a3 < b3.

m = size(tfns,1); % m rows corresponds to m tfns
n = size(tfns,2); % n columns of a tfn is always 3

% Sum lower, middle and upper bounds

temp = sum(tfns);

for k = 1: n
      y(k) = temp(k);
end
```

```
function y = tvertdiv(tfn1, tfn2)
% TVERTDIV Triangular Vertex Division
% TVERTDIV(TFN1, TFN2) returns a vector of memberships of a
% triangular fuzzy number.  TFN1 = [a11 a12 a13] is a 3-element
% vector that defines the numerator tfn and TFN2 = [a21 a22 a23]
% is a 3-element vector that defines the denominator tfn.
% TVERTDIV(TFN1, TFN2) requires that a1 < a2 < a3.

for k = 1: 3
      a1(k) = tfn1(k);
      a2(k) = tfn2(k);
end

temp = zeros(1,4);

temp(1) = a1(1)/a2(1);
temp(2) = a1(1)/a2(3);
temp(3) = a1(3)/a2(1);
temp(4) = a1(3)/a2(3);

y(1) = min(temp);
y(2) = a1(2)/a2(2);
y(3) = max(temp);


function y = tvertsub(tfn1, tfn2)
%  TVERTSUB(TFNS) returns a 3-element row vector that defines a
%  triangular fuzzy number corresponding to the difference between tfn1
%  and tfn2.

tfn1 = tfn1';
tfn2 = tfn2';

a11 = tfn1(1); a12 = tfn1(2); a13 = tfn1(3);
a21 = tfn2(1); a22 = tfn2(2); a23 = tfn2(3);

% Evaluate difference at alpha-cut 0+

t(1) = a11 - a21;
t(2) = a11 - a23;
t(3) = a13 - a21;
t(4) = a13 - a23;

% Determine bounds for a1 and a3

y(1) = min(t);
y(3) = max(t);

% The bound for a2 at alpha-cut 1.0 is simply the difference between a12
and a22

y(2) = a12 - a22;

y = y';
```

```
function y = rdtn(x)
% RDTN - Round to Tens Decimal Place
% RDTN(X) returns a matrix of floating point numbers rounded to the
% tens decimal place (i.e. 0.xx).  This function is called to ensure
% consistent precision for the purposes of graphing output.

m = size(x,1);
n = size(x,2);

for j = 1: m
      for k = 1: n
            y(j,k) = round(x(j,k)*100)/100;
      end
end
```

BIBLIOGRAPHY

Allen, Edward. (1990). *Fundamentals of building construction.* 2nd Ed. John Wiley and Sons, New York, N.Y.

Andersson, Leif. (1988). *The theory of possibility and fuzzy sets.* Swedish Council for Building Research, Stockholm, Sweden.

Arafah, Abdulrahim M.A. (1986). *Integration of human error in structural reliability models.* PhD thesis, Dept. of Civil Engrg., University of Michigan, Ann Arbor, M.I.

Assessment of damage to single-family home caused by Hurricanes Andrew and Iniki. (1993). NAHB Research Center for the U.S. Department of Housing and Urban Development, Office of Policy Development and Research, Washington, D.C.

Ayyub, Bilal M. (1991). "Systems framework for fuzzy sets in civil engineering." *Fuzzy Sets and Systems*, 40 (1991), 491-508.

Bardossy, Andras, and Lucien Duckstein. (1995). *Fuzzy rule-based modeling with applications to geophysical, biological and engineering systems.* CRC Press, New York, N.Y.

Blockley, D.I. (1980). *The nature of structural design and safety.* John Wiley and Sons, New York, N.Y.

Boissonade, Auguste, and Wei-min Dong. (1993). "Hurricane loss estimation and forecasting: applications to risk management." *Proc., 7th U.S. Nat. Conf. on Wind Engrg.*, WERC and NSF, June 27-30, UCLA, Los Angeles, C.A. 115-124.

Brown, Colin B., and James T.P Yao. (1983). "Fuzzy sets and structural engineering." *J. Struct. Engrg.*, ASCE, 109(5), 1211-1225.

Building performance: Hurricane Andrew in Florida. (1992). FEMA, Washington, D.C.

Bulleit, William M., David V. Rosowsky, Kenneth J. Fridley, and Marvin E. Criswell. (1993). "Reliability of wood structural systems." *J. Struct. Engrg.*, ASCE, 119(9), 2629-2641.

Bulliet, William M. (1985). "Relative reliability of dimension lumber in bending." *J. Struct. Engrg.*, ASCE 111(9), 1948-1963.

Carper, Kenneth L. (1989). "Learning from failures." *Forensic Engineering*, K.L. Carper, ed., Elsevier, New York, N.Y. 14-31.

Coastal construction manual. (1986). FEMA, Washington, D.C.

Conner, Harold W., David S. Gromala and Donald W. Burgess. (1987). "Roof connections in houses: key to wind resistance." *J. Struct. Engrg.*, ASCE, 113(12), 2459-2474.

Cornell, C.A. (1969). "A probability-based structural code." *J. American Conc. Inst.*, 66(12), 974-985.

Dai, Shu-Ho, and Ming-O Wang. (1992). *Reliability analysis in engineering applications*. Van Nostrand Reinhold, New York, N.Y.

Davenport, A.G. (1983). "On the assessment of the reliability of wind loading on low buildings." *J. Wind Engrg. and Ind. Aero.*, Elsevier, (11), 21-37.

Ditlevesen, O., and P. Bjerager. (1986). "Methods of structural systems reliability." *Structural Safety,* 3 (1986), 195-229.

Dong, Wei-min, Wei-Ling Chiang, Haresh C. Shah, and Felix S. Wong. (1987). "Failure possibility of buildings." paper presented at ISFK Symposium, July 1987, Guangzhou, China.

Dong, Wei-min. (1986). *Applications of fuzzy set theory in structural and earthquake engineering*. PhD thesis, Department of Civil Engrg., Stanford University, Stanford, C.A., August 1986.

Douglas, Bradford K. (1992). "Wood building analysis and recommendations - Hurricane Andrew Part 2." Washington, DC: National Forest Products Association.

Easley, J.T., M. Foomani, and R.H. Dodds. (1982). "Formulas for wood shear walls." *J. Struct. Div.*, ASCE, 108(11), 2460-2478.

Edward, Keith L., and John D. Rose. (1994). "Hurricane Andrew - structural performance of buildings in south Florida. *J. Perf. of Const. Fac.*, ASCE, 8(3), 178-191.

El-Shahhat, A.M., D.V. Rosowsky, and W.F. Chen. (1996). "Accounting for human error during design and construction." *J. Arch. Engrg.*, ASCE, 1(2), 84-92.

Eldukair, Ziad A., and Bilal M. Ayyub. (1992). "Multi-attribute fuzzy decisions in construction strategies." *Fuzzy Sets and Systems*, 46 (1992), 155-165.

Ellingwood, Bruce. (1987). "Design and construction error effects on structural reliability." *J. Struct. Engrg.*, 113(2), 409-422.

Furuta, Hitoshi, Masaaki Ohshima and Naruhito Shiraishi. (1989). "Reliability analysis of damaged redundant structures." *Recent studies on structural safety and reliability, current Japanese materials research,* 5, T. Nakagawa, H. Ishikawa and A. Tsuri, eds. Elsevier Applied Science, New York, N.Y.

Gaus, Michael P., Edgar Small and Seong H. Kim. (1993). "Reduction of wind-storm damage to existing low-rise structures through self-help activities." *Proc., 7th U.S. Nat. Conf. on Wind Engrg.*, WERC and NSF, June 27-30, UCLA, Los Angeles, C.A. 273-282.

Gupta, Ajaya K. and George P. Kuo. (1985). "Behavior of wood-framed shear walls." *J. Struct. Engrg.*, ASCE, 113(2), 1722-1733.

Ho, T.C. Eric, David Surry and Alan G. Davenport. (1992). "Low building wind load variability with application to codes." *J. Wind Engrg. and Ind. Aero.*, Elsevier, (42), 1878-1798.

Knoll, Franz. (1982). "Modeling gross errors - some practical considerations." *Proc., Risk, Struct. Engrg. and Human Error*, M. Grigoriu, ed., University of Waterloo Press, Waterloo, Ontario, Canada, 139-161.

Lewis, E.E. (1987). *Introduction to reliability engineering*. John Wiley and Sons, New York, N.Y.

Lin, J.X., and D. Surry. (1993). "Suppressing extreme suction on low buildings by modifying the roof corner geometry." *Proc., 7th U.S. Nat. Conf. on Wind Engrg.*, WERC and NSF, June 27-30, UCLA, Los Angeles, C.A. 413-422.

Lind, N.C. (1985). "Reconciling fuzzy concepts and probability in structural engineering." *Proc. NSF Workshop on Civ. Engrg. Apps. of Fuzzy Sets,* Colin B. Brown, Jean-Lou Chameau, Richard Palmer and James T.P. Yao, eds. October 1985, West Lafayette, I.N. 257-270.

Lind, Niels C. (1986). "Control of human error in structures." *Proc., Modeling Human Error in Structural Design and Construction.*, A.S. Nowak, ed., ASCE, New York, N.Y. 122-127.

Load and resistance factor design specification for steel buildings. (1986). American Inst. of Steel Const., Chicago, I.L.

Lombard, P., N. Stubbs, and D. Perry. (1995). *Estimate of costs to implement the new building code for windstorm resistant construction along the Texas coast.* Report submitted to the Texas Dept. of Insurance. Mechanics and Materials Center, Texas A&M University, College Statin, T.X.

Maamouri, Mahmoud. (1988). *Reliability of wodd trusses.* Master's of science thesis, Dept. of Civil and Env. Engrg., University of Wisconsin-Madison, Madision, W.I.

Madsen, H.O., S. Krenk and N.C. Lind. (1986). *Methods of structural safety.* Prentice-Hall, Englewood Cliffs, N.J.

MATLAB® *User's Guide for Macintosh Computers.* (1993). The MathWorks, Inc., Natick, M.A.

Meecham, Douglas. (1988). *Wind action on hip and gable roofs.* Master of engineering science thesis, University of Western Ontario, London, Ontario, August, 1988.

Minimum design loads for buildings and other structures. ANSI/ASCE 7-93. (1993). ASCE, New York, N.Y.

Morse-Fortier, Leonard J. (1993). "Complexity and contradiction in structural performance of wood-frame houses in Hurricane Andrew." *Proc., 7th U.S. Nat. Conf. on Wind Engrg.*, WERC and NSF, June 27-30, UCLA, Los Angeles, C.A. 473-482.

Morse-Fortier, Leonard J. (1996). "An analysis of damage from Hurricane Andrew: a dissenting view." paper to be presented at the *ASCE Conference on Natural Disaster Reduction*, December, 1996, Washington, D.C.

Nessim, Maher A., and Ian J. Jordaan. (1985). "Models for human error in structural reliability." *J. Struct. Engrg.*, ASCE, 111(6).

Pandey, P.C., and S.V. Barai. (1994). "Sensitivity-based weighted-average structural damage assessment." *J. Perf. of Const. Fac.*, ASCE, 8(4), 243-263.

Perry, D., and N. Stubbs. (1995). *Development of wind speed contour map for Texas coastline.* Report submitted to the Texas Dept. of Insurance. Mechanics and Materials Center, Texas A&M University, College Statin, T.X.

Philpot, Timothy A., David V.Rosowsky and Kenneth J. Fridley. (1994). "Reliability of wood joist floor systems with creep." *J. Struct. Engrg.*, ASCE, 121(6), 946-954.

Pidgeon, N.F. and B.A. Turner. (1986). "H.E. and socio-technical system failure." *Proc., Modeling Human Error in Structural Design and Construction.*, A.S. Nowak, ed., ASCE, New York, N.Y. 193-203.

Report of the working group on occurrence of errors. Robert E. Melchers, chair. *Proc., Modeling Human Error in Structural Design and Construction.*, A.S. Nowak, ed., ASCE, New York, N.Y. 204-208.

Rosowsky, D.V., and K.J. Fridley. (1993). "Reliability-based design of wood members subject to ponding." *J. Struct. Engrg.*, ASCE 119(11), 3326-3343.

Rosowsky, David V., Bruce R. Ellingwood and Timothy A. Philpot. (1994). "Duration of load and reliability of wood structures." *Structural Safety and Reliability.* Schueller, Shinozuka eds. Balkema, Rotterdam, the Netherlands. 599-605.

Rosowsky, David, and Bruce Ellingwood. (1991). "System reliability and load-sharing effects in light-frame wood contruction." *J. Struct. Engrg.*, ASCE 117(4), 1096-1114.

Santamarina, J.C., and J.L. Chameau. (1989). "Limitations in decision making and system performance." *J. Perf. of Const. Fac.*, ASCE, 3(2), 78-86.

Stubbs, N., D. Perry, and P. Lombard. (1995). *Cost effectiveness of the new building code for windstorm resistant construction along the Texas coast.* Report submitted to the Texas Dept. of Insurance. Mechanics and Materials Center, Texas A&M University, College Statin, T.X.

Stubbs, Norris, and A. Boissonnade. (1993). "Damage simulation model for building contents in a hurricane environment." *Proc., 7th U.S. Nat. Conf. on Wind Engrg.*, WERC and NSF, June 27-30, UCLA, Los Angeles, C.A. 759-771.

Surry, David. (1990). "Recent and current research into wind loading of low buildings at the University of Western Ontario." *J. Wind Engrg. and Ind. Aero.*, Elsevier, (36), 1319-1329.

Surviving the storm - building codes, compliance and the mitigation of hurricane damage. (1989). All-Industry Research Council, Oak, Brook, I.L.

Sutt, Edward, Kailem Muralidhar and Timothy Reinhold. (1996). "Roof sheathing uplift resistance for hurricanes." paper presented at the *ASCE Structures Congress*, April 15-18, 1996, Chicago, I.L.

Thoft-Christensen, Palle, and Michael J. Baker. (1982). *Structural reliability theory and its applications*. Springer-Verlan, New York, N.Y.

Tommelein, I.D., R.I. Carr, and A.M. Odeh. (1993). "Assembly of simulation networks using designs, plans and methods." *J. Const. Engrg. and Mgmt.*, ASCE 120(4), 796-815.

Toumi, R.L., and W.J. McCutheon. (1978). "Racking strength of light-frame nailed walls." *J. Struct. Div.*, ASCE, 104(7), 1131-1140.

Vanegas, Jorge A., Edmundo B. Bravo, and Daniel W. Halpin. (1993). "Simulation technologies for planning heavy construction processes." *J. Const. Engrg. and Mgmt.*, ASCE, 119(2), 336-354.

Ventre, Francis T. (1979). "Innovation in residential construction." *Technology Review*, November 1979. 51-59.

Wadia-Fascetti, Sara, and H. Allison Smith. (1994). *Adaptive dynamic analysis considering structural lifespan: an approach based on fuzzy mathematics*. Report no. 113, The John A. Blume Earthquake Engineering Center, Dept. of Civil Engrg., Stanford Univ., Stanford, C.A., August 1994.

Walker, George R., and Keith J. Eaton. (1983). "Application of wind engineering to low rise housing." *J. Wind Engrg. and Ind. Aero.*, Elsevier, (14), 91-102.

Wen, Yi-Kwei. (1990). *Structural load modeling and combination for performance and safety evaluation*. Elsevier, New York, N.Y.

Wind and the built environment - U.S. needs in wind enginering and hazard mitigation. (1993). Committee on Natural Disasters, National Research Council. National Academy Press, Washington, D.C.

Wind- and flood-resistant construction. (1995). BOCA, Country Club Hills, I.L.

Wolfe, Ronald W., Ramon M. Riba, and Mike Triche. (1993). "Wind resistance of conventional buildings." *Proc., Hurricanes of 1992 - Andrew and Iniki One Year Later.* ASCE, December 1-3, 1993, Miami, Florida. DIV 1-DIV 11.

Wood frame construction manual for one- and two-family dwellings. (1996). American Forest and Paper Association, Washington, D.C.

Yao, James T.P. (1985). *Safety and reliability of existing structures*. Pitman, Boston.

Zadeh, Lotfi A. (1965). "Fuzzy sets." *Information and Control,* 8, 338-353.

Zadeh, Lotfi A. (1978). "Fuzzy sets as a basis for a theory of possibility." *Fuzzy Sets and Systems*, 1, 3-28.

Zaitz, M.D. (1994). "Roof sheathing racking effect on fastener withdrawal capacities." Master's of science thesis, Dept. of Civil Engrg., Clemson University, S.C.