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Impact of Structural Aging on Seismic Risk Assessment of Reinforced Concrete Structures in Nuclear Power Plants

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Abstract

The Structural Aging Program is addressing the potential for degradation of concrete structural components and systems in nuclear power plants over time due to aging and aggressive environmental stressors. Structures are passive under normal operating conditions but play a key role in mitigating design-basis events, particularly those arising from external challenges such as earthquakes, extreme winds, fires and floods. Structures are plant-specific and unique, often are difficult to inspect, and are virtually impossible to replace. The importance of structural failures in accident mitigation is amplified because such failures may lead to common-cause failures of other components. Structural condition assessment and service life prediction must focus on a few critical components and systems within the plant. Components and systems that are dominant contributors to risk and that require particular attention can be identified through the mathematical formalism of a probabilistic risk assessment, or PRA. To illustrate, the role of structural degradation due to aging on plant risk is examined through the framework of a Level 1 seismic PRA of a nuclear power plant. Plausible mechanisms of structural degradation are found to increase the core damage probability by approximately a factor of two.

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EXECUTIVE SUMMARY

Reinforced concrete components are subject to a phenomenon known as aging, leading to time-dependent changes in engineering properties that may impact their ability to withstand various challenges from service requirements in operation, the environment, and accidents. While some of these changes may be benign, aggressive environmental factors and influences can cause deleterious changes in material strengths and stiffnesses. The Structural Aging (SAG) Program, in progress since 1988, is addressing the potential for degradation over time of reinforced concrete structural components in nuclear power plants (NPPs) due to aging and an aggressive environment. The principal objectives of the SAG Program are to: (1) develop criteria to ensure that degradation of reinforced concrete structures in NPPs does not endanger public health and safety, and (2) provide a technical basis for criteria and guidelines to evaluate structures for continued service.

Structures have an important safety function in any NPP. Although they are essentially passive under normal operating conditions, they play a key role in mitigating the impact of extreme environmental events such as earthquakes, winds, fire, and floods on the response of essential plant safety systems. Moreover, the importance of structural components and systems in accident mitigation is amplified by the so-called common cause effect in which failure of a structure may lead to failure or loss of function of appurtenant mechanical or electrical components and systems. In contrast to mechanical and electrical components, which are routinely inspected and tested in-service and can be replaced if necessary, structural components are plant-specific and unique, often are difficult to inspect or are inaccessible to in-service inspection, and are virtually impossible to replace in-service if they become degraded. Thus, the impact of poor structural performance can be more serious than a cursory examination of plant safety systems might indicate.

Time-dependent changes in structural properties, as well as challenges to the system, are random in nature. Accordingly, safety evaluations of new and existing structures should be conducted within a probabilistic framework. As part of the SAG Program, a methodology has been developed that provides a reliability-based framework for condition assessment and probability-based life prediction of concrete structures in NPPs. This reliability-based methodology takes into account time-dependent stochastic changes in resistance due to aging effects as well as randomness in plant loads.

Because of their complexity, lack of access, and cost of in-service inspection and maintenance, structural condition assessment and service life predictions in support of plant risk management necessarily must focus on a few critical structures in the plant. An importance ranking methodology has been developed to identify a list of structural components that are most critical for plant safety. This list can be reduced further by considering the impact of structural degradation on overall plant risk in probabilistic terms. Those structures that are dominant contributors to plant risk should be the focus of any condition assessment supporting a decision regarding continued service. Conversely, those structures for which performance contributes little to risk need not be examined. Dominant contributors to risk can be identified through the mathematical formalism of a probabilistic risk assessment (PRA) or a probabilistic margin assessment.

A PRA provides a structured framework for analyzing uncertainty and for evaluating the consequences of various events to plant safety, particularly those events beyond the design basis that occur with small probability. A PRA requires identification of a set of initiating or design-basis events (hazard analysis), models of system operation to mitigate these events (plant logic), assignment of probabilities that components perform as designed (through test data, fragility, or theoretical structural reliability analysis), determination of the probability of core damage or offsite release, and finally, an assessment of the significance of the calculated risk. The U.S. Nuclear Regulatory Commission has recognized the benefits of probabilistic risk assessment for evaluating vulnerability of plant components and systems; the requirement for an Individual Plant Examination (IPE), and subsequently, an IPE for External Events for all operating plants in the United States reflects this recognition.

Much of the research focus on the impact of aging on plant risk has been on performance of mechanical and electrical components which play an active role in accident mitigation. The importance of aging structural components in plant safety has received less attention. PRAs conducted to date indicate that structural systems generally play a passive role in mitigating design-basis internal initiating events; one notable exception is the pressure-retaining function of the containment following a degraded core incident involving failure of the reactor pressure vessel. On the other hand, structures play an essential role in mitigating extreme external events initiated by earthquake, wind, and other external influences. Because of their importance for external initiators and the common-cause issue noted above, the effect of deterioration of structural components due to aging and the potential role this phenomenon might play in accident mitigation should be examined carefully.

The response of NPP systems to extreme earthquakes has received considerable attention during the past decade, and the importance of various structural components in mitigation of degraded core incidents is evident in a Level 1 seismic PRA. Studies have shown that up to 25% of the total core damage risk and 90% of offsite risk may be due to earthquake. Moreover, the plant logic for seismic events is well established. Accordingly, in this report, increases in risk due to structural aging and deterioration of reinforced concrete structures are examined in the context of a Level 1 seismic PRA of a paradigm plant.

The report is organized along the following lines:

Chapter 2 presents fundamentals of structural reliability analysis and summarizes the plant logic for the Zion Plant Unit No. 1. This plant was selected as the paradigm for the study because: (1) it has been widely reviewed and analyzed in previous studies; (2) its hazard analysis, fragility models and plant logic are scrutable; and (3) the plant logic (core damage and plant damage state) depends on a mix of structural, mechanical, and electrical components. The Zion plant has no apparent problems due to structural aging.

Chapter 3 summarizes the seismic hazard analysis, drawing from the results presented in the original PRA. Chapter 4 presents a critical appraisal of the structural component fragility models that are required by the plant logic, and examines the impact of potential degradation mechanisms on structural fragility.

Chapter 5 examines the sensitivity of core damage probability to aging and time-dependent limit state probabilities of key structural components. Of particular interest are the questions: (1) how much does one component contribute to plant risk? (2) to what extent does changing a component reliability or fragility impact the estimated risk or high-confidence-low-probability-of-failure value of acceleration (HCLPF)? and (3) which parameters have the most effect on risk and the uncertainty attached to the estimate of risk? Making plausible assumptions regarding the damage accumulation rate over a period of 40 years, it was found that substantial damage to major structural components led to an increase in estimated core damage probability by less than a factor of 5, while the HCLPF acceleration decreased by less than 20%. Assuming that such changes are within the epistemic uncertainty range associated with the risk analysis, investing in in-service inspection/maintenance for structures that show less sensitivity to plant risk would not be a cost-effective risk management strategy.

Chapter 6 elucidates how priorities for in-service condition assessment of concrete structures can be established in support of decisions concerning continued service. It is recommended that the results of the plant risk analyses being performed in the IPEEE Program be overlaid on the time-dependent reliability methodology developed in the SAG Program to identify those few structures with a potential to contribute to plant risk for further condition assessment.

1. Introduction

1.1 Background

Safety-related systems for accident mitigation in nuclear power plants include mechanical, electrical and structural components. Such components are subject to a phenomenon known as aging, leading to time-dependent changes in engineering properties that may impact their ability to withstand various challenges from plant operation, the environment and accidents. While some of these changes may be beneficial or benign, many aggressive environmental factors and influences can cause deleterious changes in materials and components.

Facility life extension is becoming increasingly important in many areas, including the nuclear industry, due to the current economic climate (CERF, 1991; ASME, 1992). By the end of the decade, a majority of the commercial nuclear power plants (NPPs) in the United States will be more than 20 years old. It is expected that many will be evaluated with a view toward continuing their service beyond the original 40-year license period. Such evaluations must ensure that the capacity of the plant safety-related systems to mitigate design-basis events has not deteriorated during their prior service history to an extent that endangers public safety. The nuclear plant aging research program is addressing these concerns for aging components and systems that may affect safe plant operation (Vora, et al, 1991).

Much of the research focus on the impact of aging and deterioration in NPP structures has been on mechanical and electrical components (Shah and McDonald, 1987). Mechanical and electrical systems play an active role in mitigating accident sequences that arise from internal or external challenges. Relative to structures, mechanical and electrical components are easily and often frequently tested in-service. They can, if necessary, be replaced during normal scheduled maintenance with a minimum disruption of facility operation. Moreover, many such components are nominally identical, and as a result test data often are available for modeling their failure rates.

In contrast, structures are passive under normal operating conditions, playing a role (in terms of structural demand and capacity) mainly in responding to "external" events such as earthquakes, extreme winds, fires and floods (Uncertainty, 1986). However, the importance of structural failures in accident mitigation is amplified because failure of a structural component may affect several other safety-related systems simultaneously. Unlike mechanical and electrical components, structural components and systems are plant-specific and tend to be unique, and only limited data are available to judge their performance in-service. Structures may be difficult to inspect with any regularity, key components may be inaccessible, and significant structural degradation may not be readily noticed. Moreover, structural components usually cannot be replaced, if at all, without taking the facility out of service for an extended period of time. Thus, the importance of structural systems is significant, if not evident, in maintaining plant safety.

The Structural Aging Program (SAG), in progress since 1988, (Naus, et al, 1993) is addressing the potential for degradation of concrete structural systems in NPPs over time due to aging and aggressive environmental stressors. The principal objectives of the Structural Aging Program are to: (1) develop criteria to ensure that degradation of reinforced concrete structures in NPPs does not endanger public health and safety; and (2) provide a technical basis for criteria and guidelines to evaluate structures for continued service. In support of these objectives, the following issues are being addressed:

Which structural components and systems are susceptible to detrimental aging effects?

What are the significant degradation mechanisms, and how can they be modeled mathematically?

What inspection/maintenance programs, if any, will be necessary to ensure acceptable in-service performance beyond the initial service period?

What criteria and supporting evidence are necessary to evaluate service life extension?

Time-dependent changes in structural properties, as well as challenges to the system, are random in nature. Safety evaluations of new and existing structures can be conducted rationally and systematically within a probabilistic framework (Melchers, 1987). During the past four years, a methodology has been developed that provides a reliability-based framework for condition assessment and probability-based life prediction of concrete structures in NPP (Ellingwood and Mori, 1993; Mori and Ellingwood, 1993; 1994a; 1994b). The reliability analysis takes into account time-dependent stochastic changes in resistance as well as randomness in structural loads. This methodology provides a basis for assessing the capability of new or existing concrete structures to withstand design-basis (or larger) events during a projected period of service without significant increase in risk to public safety. Some of the questions that already have been answered by this research include:

Which aging factors are significant for concrete structures in terms of their future reliability?

What is the remaining service life of a concrete component if reliability is to be maintained without any inspection/repair?

What is the significance of aging in a system of structural components in terms of aging in the individual components?

What nondestructive techniques are most useful for assessing reliability of an existing concrete structure? Which nondestructive evaluation parameters are most important for informative reliability analysis?

What inspection policies and data sampling plans should be implemented to maintain the desired level of reliability at a reasonable cost?

Reinforced concrete structural components, such as shear walls, slabs, beams and columns, that are found in the reactor enclosure building, control or auxiliary building, and other balance-of-plant facilities are designed and constructed to high standards. However, there have been several instances where the capacity of the containment or other safety-related structures has been challenged by sources of degradation. A questionnaire was sent to U.S. utilities concerning inspection, degradation and repair of reinforced concrete structures in light-water reactor plants (Naus, et al 1994). Responses provided by 29 utilities representing 41 units revealed that the majority of plants are inspected visually only in compliance with integrated-leak rate tests and surveillance of post-tensioning systems as required by RG 1.35 (1990). More thorough inspection/maintenance generally is performed only in response to degradation that already has occurred, not as a preventive measure. The most common manifestations of degradation observed for concrete structures are concrete cracking and reinforcement corrosion. Causes for concrete deterioration were drying shrinkage, acid/chemical attack, thermal movement or cycling, freeze-thaw damage, expansive aggregate reactions and seawater exposure. Locations of concrete deterioration noted included containment dome, walls, slabs and equipment supports within reactor buildings, and walls and slabs of auxiliary buildings. In addition to corrosion, cracking of tendon anchors and low or unbalanced prestressing forces have been observed. Subtasks in the SAG program deal with component assessment and repair technologies (Naus, et al 1994), nondestructive evaluation techniques for in situ concrete (Refai and Lim, 1991; Snyder, Clifton and Carino, 1992), and concrete repair techniques (Krauss, 1994).

Structural condition assessment and service life predictions must focus on a few critical structural components and systems within the plant because of their complexity, lack of access (one oft-stated concern relates to the inaccessibility of the reinforced concrete basemat which may be vulnerable to leaching or sulfate attack), and cost of in-service inspection and maintenance. An importance ranking methodology has been developed to identify that subset of structural components that are most significant for plant safety (Hookham, 1991). This ranking methodology establishes relative component importance on a simple ordinal scale, based on subjectively assigned numerical ratings. The list of critical structural components can be further reduced by considering the impact of their degradation on plant risk explicitly in probabilistic terms. Those structural components and systems that are dominant contributors to plant risk should receive the focus of attention in in-service condition assessment. The dominant contributors can be identified through the mathematical formalism of a probabilistic risk assessment (PRA) or a probabilistic margin assessment.

A PRA provides a structured framework for analyzing uncertainty and for evaluating the consequences of various events, particularly those beyond the design basis with small probability, to plant safety. The focus of a PRA is on major accidents that may lead to threats to public health and safety or extensive property damage. It can assist in identifying specific systems and parameters for regulatory attention and in improving existing deterministic safety requirements. It provides a comprehensive picture of the capability of the plant to mitigate accidents, throwing light on portions of the system that might otherwise not be examined due to variations in customary design procedures and analysis tools. PRA can be used to eliminate or modify rules where implementation or enforcement is not commensurate with their signficance to facility safety, allowing regulatory attention to be directed toward factors that are risk-significant. A PRA requires identification of a set of initiating or design-basis events (hazard analysis), development of models of safety system operation to mitigate these events (referred to as plant logic models), assignment of probabilities that components and systems in the plant perform as designed (through test data, fragility models, or theoretical structural reliability calculations), determination of the probability of core damage or offsite release, and finally an assessment of the significance of the calculated risk. Although some of the first applications of PRA were to NPPs (Reactor, 1975), they since have been applied in numerous other contexts, e.g., to bridges (Sexsmith, et al, 1994), chemical plants and LNG storage facilities (Ravindra, 1994), offshore structures, and other critical facilities.

When PRA is applied to NPPs, design-basis events are categorized as "internal" and "external" events (PRA, 1983). Internal events include loss of coolant accidents (LOCA) and anticipated transients without scram (ATWS) in BWRs, and other transients. External events include extreme winds (hurricanes and tornadoes), earthquakes, fire and flood. Early safety studies focussed exclusively on risk due to internal events. It later was realized that external events also had significant potential for initiating core damage. During the past 15 years, over two dozen seismic PRAs have been completed; only a few of these have been published or subjected to independent scrutiny by peer review (Sues, et al, 1990).

The U. S. Nuclear Regulatory Commission has recognized the benefits of probabilistic methods in identifying contributors to risk, evaluating accident sequences, and identifying oppportunities to improve safety (Review, 1994). The commission views PRA as one means for ensuring consistent, predictable and efficient regulation for the nuclear industry. The Independent Plant Examination (IPE) program, initiated in 1989, now is requiring an assessment of risk-significant contributors for all licensed plants in the United

States (Flack, 1994). Its objectives are to develop an appreciation of severe accident behavior and the most likely sequences, to identify vulnerabilities and dominant contributors to risk systematically, to quantify probabilities of core damage and fission product release, and to facilitate reduction of core damage probability through modifications of hardware and training. Benefits anticipated from the IPE include a better appreciation of various safety issues, impact of design changes, improvements in reliability-based maintenance programs, and insights useful to support license renewal. There is a follow-on program on Individual Plant Examination for External Events (IPEEE).

Most utilities are conducting their IPE/IPEEE risk assessments using a so-called "Level 1 PRA" (PRA, 1983), which focuses on determining probability of core damage; some have extended the analysis to consider containment failure modes (Level 2 PRA) or offsite consequences to improve quantitative understanding of risk. (Such an analysis that focuses on core damage without considering the source term or offsite consequences is sometimes referred to as a probabilistic safety analysis, or PSA.) The overall average core damage probabilities reported to date in the IPE for all PWR and BWR plants (internal events) are (Flack, 1994):

BWRs: 2.0×10^{-5}/yr
PWRs: 7.8×10^{-5}/yr

There is an order-of-magnitude variation among the plants reporting, however, demonstrating that risk estimates are sensitive to plant design, modeling assumptions, and level of effort in the PRA. Although across-plant comparisons of risk in absolute terms are not particularly useful at the current state of the art, variations in estimated risk for a particular plant that arise from changes in modeling assumptions, component participation, etc., are meaningful as they convey a sense of the relative importance of individual components in accident sequence mitigation. It should be emphasized that there is no correct answer in PRA; only an answer that is consistent with the available knowledge (and database).

PRAs conducted to date reveal that structural systems generally play a passive role in mitigating design-basis (or larger) internal initiating events; a notable exception to this is the pressure-retaining function of the containment following a degraded core incident involving failure of the reactor pressure vessel. Internal events are mitigated by plant operators and by mechanical and electrical systems in the plant. On the other hand, the structural components play an essential role in mitigating extreme events initiated by earthquake, wind and other external influences, and their failure probabilities due to external events can be higher. Moreover, failure of major structural components may impact the operation of a number of mechanical and electrical systems as well (so-called common cause failures). Thus, deterioration of structural components and systems due to aging and other aggressive environmental influences may be more serious in terms of overall plant risk than might be evident from a cursory examination of their role in accident mitigation.

Prevention or mitigation of degraded core incidents is a prime consideration in assurance of plant safety. The importance of various structural components and systems in meeting this goal becomes most evident in a level 1 PRA for external events. Thus, the significance of structural aging and deterioration to plant risk can be evaluated by considering the impact that such deterioration has on probability of core damage associated with external initiating events. The most important of these events from a risk viewpoint appears to be earthquakes; studies have shown that up to 25% of the total core damage risk and 90% of offsite consequences may be due to earthquakes (Uncertainty, 1986; Ravindra, 1994). It is in mitigating the effects of strong ground motion due to earthquakes that structural systems play a particularly significant role. Accordingly, increases in risk due to structural aging and deterioration are examined in this study within the framework of a seismic PRA.

1.2 Review of Previous Work

Previous work has considered the impact of aging mainly on mechanical and electrical systems. This work is reviewed briefly in this section, essentially in chronological order.

Davis, et al (1985) utilized existing internal events PRAs of three plants - Oconee, Calvert Cliffs, and Grand Gulf - to relate aging to increases in risk. The sensitivity of risk to component failure rates was evaluated simply as the derivative of core damage probability with respect to component failure rate. There was no attempt to determine the increase in component failure rates due to aging. Only active mechanical and electrical components were considered. Components most significant for risk were found in the auxiliary feedwater system (AFW), reactor protection system (RPS) and service water system (SWS). In these systems, pumps, valves, circuit breakers and actuating circuits had the most potential for impact on increasing risk.

Meale and Satterwhite (1988) reviewed data from the Nuclear Plant Reliability Data System (NPRDS) for 15 different safety and support systems. Only active mechanical and electrical systems were considered. Failure data were collected in five generic categories. Aging was the source of 32% of failures; 49% resulted from "other" (cause could not be determined or assigned to another category); 10% from design and installation; 7.5% from testing and maintenance; and 1.5% were human-related.

An expert panel, the "Technical Integration Review Group for Aging and Life Extension, or TIRGALEX" was convened to prioritize NPP structures and components with respect to further evaluation in the NPAR program (Levy, et al, 1988). A risk-based methodology was used to establish an ordinal ranking of some 30 components based on (1) perceived increase in plant risk from component aging, and (2) adequacy of current risk management practices. The risk-importance of these components was established from four internal event PRAs. Components in which aging contributed less than 10^{-7}/yr to core damage were eliminated from further consideration; others were considered likely candidates for improvements in aging management. The reactor pressure vessel (RPV), containment, and other Category I structures ranked highest from a "risk importance" point of view, but relatively low from "increase in failure rate due to aging" and "management practices" points of view. It was concluded that these structural components may require attention with regard to aging/risk management, but are not the most critical (Levy, et al, 1988, Table 2.8, p. 2.29). Of these the BWR Mk-I containment ranked 3rd; the others were ranked less than 10th.

In one of the first attempts to incorporate aging effects into a PRA, Vesely, et al (1990) considered internal events involving active components for two "NUREG-1150" plants: a PWR and a BWR. A linear failure rate model was assumed to describe aging, and component failure data were tied to the TIRGALEX database. The analysis was based on a Taylor series expansion of core melt frequency, C, in terms of component failure rates, p, i.e., Σ (dC/dp) dp; term dC/dp is determined from the PRA logic, while dp is determined from an analysis of degradation and data for the component in question by integrating the failure rate over the appropriate time interval. Thus, surveillance and maintenance (overhaul) intervals are incorporated in the determination of dp. The analysis predicts the average core melt frequency increase resulting from aging under a given maintenance program. It was found that different aging maintenance programs can have a large effect on core melt frequencies. However, relatively few components contribute significantly to increasing risk; thus, component prioritization for inspection and maintenance purposes can be effective. This approach subsequently was criticized (Donnell, 1992) for inadequacies of the linear aging model and limitations in the aging database; it was suggested that there is little hard evidence of significant age-related degradation in important components.

Phillips, et al (1991) performed one of the few evaluation of the significance of risk to aging in passive components. It was asserted that passive components have not been considered because they have low failure rates compared to active components and there are too many of them in the plant to consider systematically. A weld in the AFW system in a NUREG-1150 PWR plant was selected as the basis for the evaluation because that weld was identified in the TIRGALEX study as having risk significance, and a known aging mechanism (thermal fatigue) is known to be at work. An illustrative calculation reveals little impact on plant risk in 48 years of service for likely weld failure probabilities. However, an AFW weld failure probability of 1.0 increases the probability of core damage by nearly four orders of magnitude to 0.13.

Wolford, et al (1992) evaluated the risk impact of aging degradation in an AFW system in a "NUREG-1150" plant. Three failure rate models - linear, exponential, and Weibull - were considered using maximum likelihood estimation techniques. Statistical methods were used to evaluate the statistical significance of aging, as measured by increasing failure rates, from actual maintenance data and to determine confidence intervals on aging parameters. When aging of the AFW system was considered (internal events PRA), there was essentially no change in core damage frequency for all three failure rate models. However it was noted that aging cannot be detected without quality data extending over a long time period, and such data are difficult to obtain.

More recently, Vesely (1992) described a methodology for incorporating component aging into a standard PRA and discussed some of the issues that must be addressed in doing so. The methodology recommended was similar to that considered previously (Vesely, et al, 1990); however, three aging models with linear, exponential, and Weibull failure rates were considered. Again, only active components were considered, but there was a brief discussion of the potential importance of passive components (piping, containment). It was noted that truncation of minimum cutsets in the PRA can lead to underestimation of aging effects because cutsets that were unimportant in the original PRA may become important when aging is considered.

Two recent studies have investigated the feasibility of setting in-service inspection requirements based on risk ranking (Vo, et al, 1994a; 1994b). In the first study, internal event PRAs were used to identify the most significant pressurized systems for plant risk and therefore for special attention during inspection. Seven PWRs and two BWRs were considered. Although each plant differed, the low pressure injection system, high pressure injection system, RPV, and service water system were the most significant for all PWRs, while the RPV, emergency service water system, and high pressure coolant injection were most important for BWRs. In the second study, ASME classifications and in-service inspection requirements were found to be in quantitative agreement with rankings based on core damage. It was recommended that in-service inspection should ensure that the risk of core damage resulting from pressure boundary failures should be a small fraction of total core damage risk (5% or less). Risk should be allocated among components of a system in accordance with their relative importance. Parameter and modeling uncertainties were mentioned but not considered explicitly. Component aging was not considered in either of these studies.

None of the above studies has addressed aging or deterioration in structural components or systems. Such aging effects are the subject of this report.

1.3 Objectives of Study

The study reported herein examines the role played by structural degradation on plant risk through the vehicle of a seismic PRA of an operating PWR plant. We seek to determine whether plausible changes in certain critical structural component or system capacities due to reinforcement corrosion or concrete deterioration from aggressive environmental influences have a statistically significant impact on the probability of core damage or plant damage states.

The results of this study, while plant-specific, can be used to draw some general conclusions about those structural components that should receive special attention in any in-service condition assessment performed in support of consideration for service life extension. Structural components thus identified would be obvious candidates for the time-dependent reliability assessment and in-service inspection and maintenance programs mentioned previously (Mori and Ellingwood, 1993).

1.4 Organization of Report

Chapter 2 presents some fundamentals of structural reliability analysis, summarizes the plant selected as a paradigm for the study, and describes the plant logic models used in subsequent reliability studies.

Chapter 3 summarizes the seismic hazard analysis, drawing mainly upon the results presented in the original plant PRA.

Chapter 4 presents the structural, mechanical and electrical component fragility models that are required by the plant logic models. A discussion is presented of postulated degradation mechanisms and their potential impact on structural fragility parameters.

Chapter 5 validates the current analysis procedure by comparing core damage probabilities with those reported in previous studies. Sensitivities of core damage probability to variations in component failure probabilities and to fragility parameters and modeling assumptions are evaluated. The role of structural components identified as having aging significance with regard to plant risk then are evaluated further. The sensitivity of core damage probabilities to plausible degradations in structural capacity due to aging are examined.

Chapter 6 presents recommendations for establishing priorities for in-service condition assessment of concrete structures in support of decisions regarding continued service of nuclear plant facilities.

2. Probabilistic Risk Assessment

2.1 Analysis of Safety

In the classical analysis of safety involving system demand, S, and capacity, R, the probability of failure or limit state probability can be computed from the convolution integral (Shinozuka, 1982; Melchers, 1987),

$$P(R < S) = \int_0^\infty F_R(x)\, f_S(x)\, dx \tag{2.1}$$

in which $F_R(x)$ = cumulative distribution function (CDF) of R, $f_S(x)$ = probability density function (PDF) of S, and R and S are expressed in dimensionally consistent units. In proper structural design with the usual design criteria, factors of safety, and other conservatisms in the analysis, this limit state probability is acceptably low.

Reliability methods now have been developed to the point where design and evaluation of existing facilities can be based on rational probabilistic assessment of the uncertainties contributing to risk (Ellingwood, 1992). In particular, seismic risk analysis is a formal methodology whereby inherent randomness and modeling uncertainties are analyzed and propagated through a model of an engineered system in order to arrive at event probabilities that can be used as a basis for engineering decision. The precise nature of the risk analysis and model depends on the nature of the decision to be made. Here, we are concerned with its usage in evaluating an existing reinforced concrete structure in a NPP with a view toward extending its service life.

2.2 Seismic Risk Analysis of Nuclear Power Plants

Probabilistic risk analysis of NPPs can be categorized in a very general way as "internal events" and "external events" PRAs. Internal events PRAs involve initiating events that originate within the plant envelope (PRA, 1983). Structural components and systems play little role in the operation of engineered safety features required to mitigate such internal events, and any effects of structural aging likely would not be apparent. External events PRAs involve initiating events that arise from external sources; included in these would be earthquake, wind, tornado, or hurricanes, and floods. Among these, earthquakes are significant contributors to plant risk. They can initiate an accident and the resulting severe distortions or failures in structural components may cause attached mechanical and electrical components to fail simultaneously (referred to as common-cause failures), thus amplifying the importance of structural components and the significance of structural failures with regard to plant safety. Thus, the impact of structural aging on plant risk in this study is evaluated within the context of a seismic PRA.

A seismic PRA is divided conceptually into hazard, fragility (capacity), and plant logic components (PRA,1983). The following steps generally are followed:

1. Identify seismic hazard from potential seismogenic sources, historical seismicity in the vicinity of the plant, and attenuation of ground motion to the site.

2. Develop plant logic to explain the interrelation of various plant components and systems in mitigating the effect of initiating events.

3. Develop fragility models to determine capacity of plant components and systems probabilistically.

4. Measure risk by calculating core damage probability, plant damage states, and offsite consequences to public health and safety.

As suggested by the last item, risk in seismic PRA of nuclear plants can be measured in several ways: in terms of core damage probability, expected cost of failure, radiation exposure, prompt or latent health effects, or others. Here, we will use probability of seismically-induced core damage or plant damage state leading to release as a surrogate for risk. It also is important to assess uncertainty in or lack of knowledge about the estimate of risk. We are interested in in-service condition assessment, inspection and maintenance procedures that not only minimize risk but also reduce the uncertainty about risk.

Existing plant logic models will be used in the following assessment. No attempt has been made to study transient events (e.g., LOCA) in combination with earthquake in terms of combined dynamic response. Although strong ground motion may initiate a transient or LOCA, it is assumed that the probability of a coincidence dynamic effects of a LOCA with earthquake is negligible.

2.3 Selection of Paradigm Nuclear Plant

2.3.1 Zion Nuclear Power Plant

The Zion Nuclear Power Plant is located north of Chicago, IL and is operated by Commonwealth Edison. It is a two-unit station, each unit a Westinghouse four-loop PWR in a large dry containment. The containment building and balance of plant were designed by Sargent & Lundy Engineers in the late 1960's. The plant began operation in 1973. The selection of the Zion Plant as the paradigm should not be construed as meaning that it is suspected as having any problem caused by structural aging. Rather, it was selected because: (1) it has been widely reviewed and studied in previous studies; (2) its PRA (particularly the seismic hazard analysis, fragility analysis and plant logic is scrutable; and (3) perhaps most important, the core damage and dominant plant damage state depend on a mix of structural, mechanical and electrical components. In many other PRAs, risk is dominated by the behavior of a few nonstructural components (Sues, et al, 1990) and the impact of structural aging on risk would be less apparent. The seismic event is a significant contributor to overall risk of severe core damage and offsite release for the Zion Plant.

Virtually all safety-related plant systems are located within Seismic Category I structures, including the containment building (reactor building), auxiliary building, and crib house enclosing the service water pumps and essential piping. The turbine building at Zion is not a Category I structure, but it has a common wall with the auxiliary building which is considered Category I.

2.3.2 Plant Logic Models

The plant safety systems are designed to bring the reactor to safe shutdown and prevent radionuclide release in the event of an extreme environmental or accidental event (Zion, 1981). A large enough earthquake can cause one of several initiating events to occur. Accident sequences are constructed diagramatically from system fault trees and safety function event trees. The possible sequences of events involving the operation of safety-related plant systems that follow are described by event trees. The event trees are constructed in accordance with the functions provided by the various safety systems. Each path through an event tree is mutually exclusive and together the paths are collectively exhaustive.

Certain of the sequences identified may lead to various states of core degradation, ranging from damage to a fully molten core and meltdown. These, in turn, can be related to various plant damage states and radionuclide release categories. The precise definition and quanitification of these categories is beyond the scope of the current report. In PRA terminology, we will concentrate on Level 1 results (PRA, 1983) (sometimes denoted a PSA) and will focus on "core damage," meaning a sequence in which a combination of failures leads to uncovering the core with a prolonged loss of cooling without distinguishing between the various degrees of damage. Containment failure and offsite consequences will not be considered. It is assumed that the plant logic models developed from the fault and event trees are logically correct. Random (nonseismic) basic events involving equipment failures - those that do not depend on the level of response or on any other basic event - are not considered in this analysis, as they are not affected by structural aging.

Each plant damage state derived from (collections of) accident sequences is represented by a Boolean expression relating component failures in the form of unions of minimum cut sets. (A cut set is the intersection of basic component failure events required for system failure; a singleton cutset is one with only one component, a doubleton one with two components, and so forth. Higher order cutsets contribute little to accident sequence probability if the individual component failure probabilities are small.) The fault trees and sequences often are culled probabilistically, eliminating low-probability terms in multiple cutsets, in order to make the analysis of plant damage states manageable. Any functional dependence among components is taken into account in the Boolean construction of the fault trees and accident sequences. The importance of structural failures is magnified by the fact that they may be common to several sequences and may affect several plant safety-related systems simultaneously.

In seismically induced accident sequences at most plants studied, offsite power is lost almost immediately. Failure of ceramic insulators almost certainly occurs at ground accelerations large enough to endanger any other component. We assume that for any earthquake large enough to affect the plant, the turbine trips due to loss of load following the loss of offsite power. The Boolean expression for core damage resulting from a seismically-induced transient with loss of cooling can be simplified to:

$$T = 8+10+17+[(12+22)*(4+9+14+21)] \tag{2.2}$$

in which the numbers refer to components summarized in Table 2.1* and the symbols + and * denote union and intersection, respectively. (The fragility parameters m_R, β_R, and β_U in Table 2.1 are described in section 4.) The numbering corresponds to the original PRA (Zion, 1981), and only components that appear in the plant logic are presented. If the earthquake initiates a small LOCA followed by loss of safety injection or cooling, the core damage Boolean is,

$$SLOCA = 4+8+10+14+17+21+(9*26) \tag{2.3}$$

Analysis of plant logic shows that the probability of other core-damage sequences is negligible. Thus, combining sequences T and SL, the Boolean expression for severe core damage becomes:

* Tables and figures appear at the end of each chapter.

$$CD = TR + SLOCA \tag{2.4}$$

$$CD = 4+8+10+14+17+21+[9*(12+22+26)] \tag{2.5}$$

The system failure event is dominated by elements in series in both cases. A mixture of structures (shear walls, slabs), mechanical (piping and tanks) and electrical components are represented.

A review of plant logic shows that the only plant damage state with non-negligible probability that is induced by seismic action is damage state SE, involving a loss of AC power followed by RCP seal failure and a small LOCA with failure of safety injection, followed by failure of fan coolers and containment spray. The Boolean expression for damage state SE is

$$SE = 4+8+10+14+17+21+[9* 25*26] \tag{2.6}$$

Containment damage may occur as a result of impact between the reactor and auxiliary building. If the containment does not fail, damage state SE results in radionuclide release category 2R. Eqns 2.5 - 2.6 will be used in the sequel to assess impact of aging on plant risk.

It may come as some surprise to find that the structural performance of the containment does not appear in the Boolean for core damage at the Zion plant. This, in fact, is typical of Level 1 PRAs for seismic events. (Of course, subsequent failure of the containment at a later stage of accident progression following core damage leads to one of several release categories). The containment is designed with a higher factor of safety than other structural components in a NPP, and its final structural configuration may depend on its protective rather than on structural response functions. The estimated probability of a properly design (undegraded) containment failing due to overpressurization from a design-basis LOCA is on the order of 10^{-8} (Schueller, 1984; Rajashekhar and Ellingwood, 1995). The seismic capacity of the Zion containment is at least as great as the lowest capacity of the other structural components appearing in Eqns 2.5 and 2.6. Therefore, it is assumed that containment failure does not contribute to the probability of core damage due to earthquakes, nor to the increase in Level 1 risk that aging would contribute.

2.4 Analysis of Uncertainty

All sources of uncertainty (variability) must be included in any probabilistic risk assessment model (Uncertainty, 1986). Predictive mechanistic models of a phenomena (hazard, behavior) are necessary in analysis or decision-making involving rare events. Probabilistic descriptions of the important parameters in these mechanistic models also are required. These mechanistic and probablistic models can be used to predict likely value (measured, in a first-order sense, by the mean, median or mode of the variable), variability (measured by standard deviation, logarithmic standard deviation, or coefficient of variation), or probability distribution. It is this quantitative analysis of uncertainty that is the dimension of decision-making that is neglected when the decision process is deterministic.

It is convenient to think of variability predicted from a model as having two sources: inherent randomness and knowledge-based (or epistemic) uncertainty. Inherent randomness in a phenomenon is a basic attribute (at least at the customary level of engineering analysis), and is essentially irreducible at the current state of the art by acquisition of additional data. However, most models are based on idealizations and approximations of behavior. Such approximations involve additional modeling variables that also contribute uncertainty and must be included in risk analysis. Since these sources of uncertainty depend on the model of the phenomenon selected, they are reducible if a more accurate model is selected. Moreover, the quality of estimates of statistics is sample-dependent, larger samples yielding more consistent and stable values.

In risk analysis involving rare events, analysis of these uncertainties often must be based subjectively on professional engineering judgement and experience rather than on empirical data. Their characterization and analysis thus are thought of in a Bayesian rather than classical statistical sense. The probabilistic model still provides the framework for analyzing these sources of uncertainty. Subject to the general guidelines above, there is an element of subjectivity in the the identification and separation of randomness vs uncertainty. Once this is done, however, the analyst must keep them separate in the risk analysis, bearing in mind the need to separate reducible sources of variability from irreducible sources. This separation is key, as it guides subsequent efforts for additional data collection and improvements in modeling. Propagation of the various sources of uncertainty through the risk analysis leads to a measure of confidence in the risk estimates. This measure of confidence can be as useful in decision analysis as the estimated risk itself.

Table 2.1

Key structural, mechanical and electrical components at Zion NPP and their fragility parameters

Component*	$m_R(g)$	β_R	β_U
4. Service water pump	0.63	0.15	0.36
8. Auxiliary building shear wall	0.73	0.30	0.28
9. Refueling water storage tank	0.73	0.30	0.28
10. Interconnecting piping	0.73	0.28	0.33
12. Condensate storage tank	0.83	0.28	0.29
14. Pump enclosure cribhouse roof collapse	0.86	0.24	0.27
17. 125 V DC batteries, racks	1.01	0.28	0.63
21. Service water buried 48" pipe	1.40	0.20	0.57
22. CST 20" piping	1.40	0.20	0.57
25. Containment ventilation system fan coolers	1.74	0.49	0.23
26. Pressurizer encloser roof collapse	1.80	0.39	0.34

* Component numbering corresponds to number in the original Zion PRA (1981).

3. Seismic Hazard Analysis

The basis of modern seismic hazard analysis is summarized elsewhere (Reiter, 1990). At the most fundamental level, the seismic hazard at a plant site is displayed through a complimentary cumulative distribution function (CCDF) relating the annual frequency of ground shaking (or spectral acceleration) at or above a given intensity to that intensity. Elementary considerations suggest that the probability distribution of peak ground acceleration can be described by a Type II distribution of largest values (Cornell, 1968). The seismic hazard, defined as the complementary CDF of acceleration, $G_A(x)$, is thus,

$$G_A(x) = 1 - \exp[-(x/\mu)^{-\gamma}] \tag{3.1}$$

in which (μ, γ) are parameters of the distribution. The return period of an event with intensity, x, is defined as,

$$T(x) = 1/G_A(x) \tag{3.2}$$

The parameter, γ, describes the slope of the CCDF. The implication of this model of seismic hazard is that ground motions A_1 and A_2 at two different return periods T_1 and T_2 can be related by,

$$T_1/T_2 \approx (A_1/A_2)^{\gamma} \tag{3.3}$$

Since γ varies from about 2.3 for sites in the Eastern United States to about 5 for sites in the Western United States (Ellingwood, 1994b), we see that doubling the peak ground acceleration increases the return period by a factor of approximately 5 in the EUS and 32 in the WUS, provided that the ground accelerations do not saturate at large magnitudes. Although the physics of seismology must place some upper limit on acceleration sustainable by the earth's crust, and earlier seismic hazard analyses typically assumed an upper limit, more recent seismic hazard analyses have placed no upper limit on peak ground acceleration.

Seismic hazard curves for the Zion Plant are illustrated in Figure 3.1. The basis for these curves is described elsewhere (Zion, 1981). In generating these curves, only earthquakes with body-wave magnitude $m_b > 4.0$ were considered, assuming that earthquakes of smaller magnitude cannot cause structural damage to a NPP. Since the process by which the curves are constructed involves uncertainty (multiple seismic source zone hypotheses, recurrence rates and Richter b-values, upper bound magnitudes, attenuation relations, variation of ground response with depth, etc.), a family of hazard curves is developed, one for each set of hypotheses. The assignment of probabilities (uncertainties) to these parameters and the weighting of the relative likelihood (truth) of each hypothesis is done at present by expert opinion. The uncertainty in the seismic hazard displayed by the family of curves increases at increasing levels of ground motion intensity, as illustrated in Figure 3.1. The major contributor to this uncertainty is ground motion attenuation, where the coefficient of variation frequently exceeds 0.6, but the other factors mentioned above contribute as well. Parameters describing these hazard curves and their relative likelihood are summarized in Table 3.1. Mean and median curves also are presented.

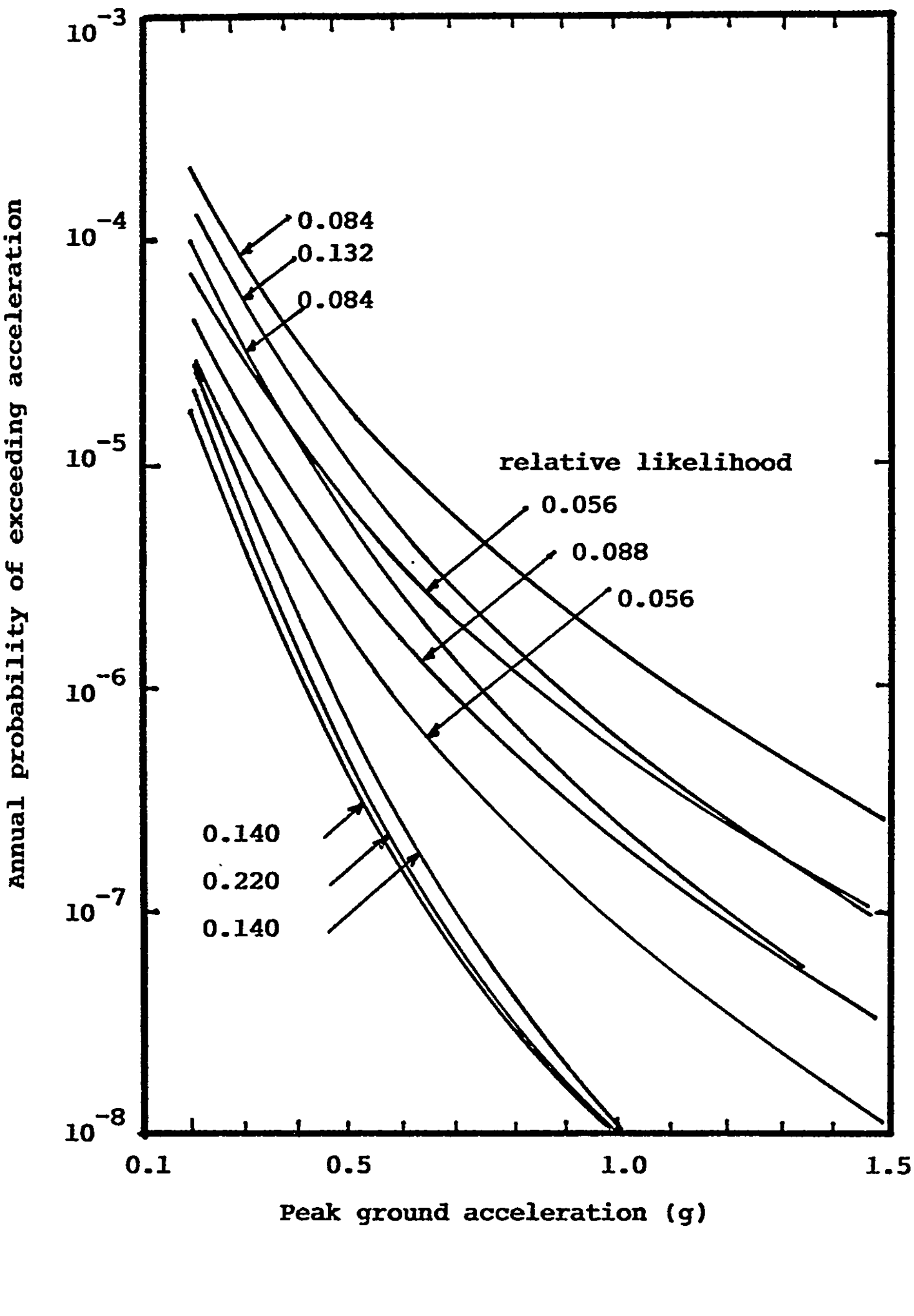
The acceleration on the horizontal axis in Figure 3.1 is referred to as effective peak acceleration, or EPA. The EPA reflects the damaging capability of the earthquake (Hall, 1982; Kennedy, 1984). For strong ground motion from large earthquakes at sites in the Western United States, the EPA is close to the instrumental peak acceleration. For other earthquakes, where the instrumental peaks are associated with high frequencies, the EPA is less than the instrumental peak. In constructing the hazard curves for Zion, it was assumed that the EPA = Instrumental Peak/1.23; the fragility models (discussed subsequently in Section 4) were developed with this assumption.

The uncertainty in seismic hazard modeling is the single largest contributor to uncertainty in core damage probability or plant damage states (Ravindra, et al, 1984; Ellingwood, 1994a). This uncertainty cannot be reduced at the current state-of-the-art of seismic hazard analysis (Reiter, 1990), and no attempt is made to deal with this issue in this report. Truncating the seismic hazard curves has little impact on risk because the dominant contributors to core damage probability come from the range 1 SSE to 5 SSE (Ravindra, et al, 1984). Limitations in current information on seismic hazard at sites in the Eastern United States do not allow the seismic hazard curves to be truncated at less than about 5 times the SSE for such sites.

Nuclear plant structures are designed for two levels of earthquake, an operating basis earthquake, or OBE, and a safe-shutdown earthquake, or SSE. The OBE now is required to be not less than one-half the SSE. Because the damping levels and response spectrum shapes may differ for these two earthquakes, either one may govern the design of a specific structural, mechanical or electrical component. Neither earthquake has a statutory probabilistic basis. Studies have found that the median probability of exceeding the SSE is on the order of 10^{-4}/yr. The median OBE exceedence probability may be on the order of 10^{-2}/yr. At the Zion Plant, the OBE was 0.08g, while the SSE (referred to at the time as the design-basis earthquake) was 0.17g. Using the mean hazard curve parameters in Table 3.1, the probability of exceeding the OBE annually is 1.48×10^{-3}; for the SSE, this probability is 1.15×10^{-4}. In an approximate sense, the OBE is a ground motion comparable in magnitude to the 475-year mean recurrence interval (annual probability of 2.1×10^{-3}) ground motion that is the basis for the NEHRP recommended provisions for seismic regulations for buildings (NEHRP, 1992).

Table 3.1 Seismic hazard curve parameters for the Zion NPP

No.	Prob.	μ	γ
1	0.056	0.0118	3.665
2	0.088	0.0099	3.335
3	0.056	0.0088	3.058
4	0.084	0.0168	3.723
5	0.132	0.0146	3.388
6	0.084	0.0127	3.070
7	0.140	0.0189	4.685
8	0.220	0.0215	4.832
9	0.140	0.0251	4.975
Median		0.0121	3.480
Mean		0.0117	3.394



3.1 Seismic hazard curves for the Zion NPP

4. Fragility Modeling

4.1 Fundmental Model

The fragility of a component or system is defined as its probability of failure, conditioned on a level of excitation (ground motion, spectral acceleration, spectral velocity, etc.) that is consistent with the specification of the hazard. In this study, the excitation is measured in terms of effective peak ground acceleration (EPA) to facilitate use of prior studies of the Zion NPP. The definition of failure depends on the performance requirement of the component or system considered. For structures, failure is assumed to occur when (generally inelastic) deformations are large enough to interfere with the operation of attached safety-related equipment. For mechanical and electrical components, failure involves loss of designated safety-related function, rupture of pressure boundary, and similar performance-related limits.

Component fragility

The most common way to model component fragility is through a lognormal distribution (Kennedy and Ravindra, 1984):

$$F_R(x) = \Phi\left(\frac{\ln(x/m_R)}{\beta_R}\right) \tag{4.1}$$

in which m_R = median capacity (expressed in terms of EPA) and β_R = SD(ln R) or logarithmic standard deviation in capacity. Parameter $\beta_R = \sqrt{(\ln(1 + COV(R)^2))}$, in which COV(R) = coefficient of variation; when COV (R) 0.30, $\beta_R \approx$ COV (R). Parameter β_R measures inherent randomness in the capacity of the component to withstand an earthquake with EPA, x.

Additional uncertainties in component capacity arise due to both mechanistic and statistical approximations in modeling. To first order, such modeling uncertainties can be assumed to be vested entirely in the estimated median, m_R; accordingly, the median, m_R, in Eqn 4.1 is replaced by a lognormal random variable, M_R, with median m_R and logarithmic standard deviation (coefficient of variation) β_U. The fragility function $F_R(x;M_R)$ thus becomes a random variable (more precisely, a random function of M_R) at any value of x. To display the overall variability in fragility, it is customary to describe the fragility of any component by a family of curves defined by the parameters (m_R, β_R, β_U). Parameters β_R and β_U denote the contributions of inherent randomness and uncertainty in the capacity of the component to withstand EPA, x. The fragility for failure of the Zion Unit No. 1 auxiliary building shear wall (Component 8 in Table 2.1, described subsequently in more detail) is displayed by the family of curves in Figure 4.1. This is analogous to the display of overall variability in the seismic hazard by the family of hazard curves in Figure 3.1.

The expected value (mean) of $F_R(x)$ can be obtained by substituting the median of M_R (m_R) into Eqn 4.1 and replacing β_R with $\sqrt{(\beta_R^2 + \beta_U^2)}$. To see this, observe that

$$F_R(x; M_R) = \Phi\left(\frac{\ln(x/M_R)}{\beta_R}\right) \tag{4.2}$$

and that M_R is lognormal with median m_R and logarithmic standard deviation, β_U:

$$F_{M_R}(m) = \Phi\left(\frac{\ln(m/m_R)}{\beta_U}\right) \tag{4.3}$$

The expected value of $F_R(x)$ is defined as,

$$E[F_R(x)] = \int_0^1 F_R(x|M_R = m)\, dF_{M_R}(m) \tag{4.4}$$

Performing the integration, it can be shown (Ellingwood, 1994) that,

$$E[F_R(x)] = \Phi\left(\frac{\ln(x/m_R)}{\sqrt{\beta_R^2 + \beta_U^2}}\right) \tag{4.5}$$

The mean fragility for the auxiliary building shear wall obtained from Eqn 4.5 is illustrated by the solid curve no. 6 in Figure 4.1. This mean fragility, when convolved with the best estimate of the seismic hazard curve using Eqn 2.1, yields a point estimate of core damage probability. We will utilize this point estimate in Section 5.

One can use a fragility such as that illustrated in Figure 4.1 to identify a level of acceleration at which there is high confidence that the component will survive. It is common in structural design and in checking safety to use a nominal or characteristic strength that has only a small probability, e.g., 0.05, of not being attained. This nominal strength occurs at the 5th percentile of the CDF of strength and sometimes is referred to as the 5% exclusion limit. Figure 4.1 shows that the 5% exclusion limit on acceleration has a frequency distribution arising from the uncertainty in the median, m_R. The lower α-fractile of this frequency distribution is a number, R_α; one could say that the probability of surviving acceleration R_α is 95% with confidence $(1 - \alpha)$. In a particular type of seismic PRA known as a "margins analysis," described subsequently in Section 5.1.3, this number is referred to as a "high-confidence, low-probability-of-failure" (or HCLPF) acceleration when $\alpha = 0.05$. Figure 4.1 shows that the HCLPF for the auxiliary building shear wall is 0.28g; in other words, the probability that the shear wall survives an earthquake with EPA of 0.28g is 0.95 with 95% confidence.

The 5% exclusion limit for the mean fragility (Eqn 4.5) is 0.37g; however, the information regarding uncertainty in the estimate that is displayed in the family of curves is lost, and the HCLPF of 0.28g occurs at a lower fractile of the mean fragility. An estimate of that fractile can be obtained by the following procedure. By definition, for any component,

$$\text{HCLPF} = m_R \exp[-1.645(\beta_R + \beta_U)] \tag{4.6}$$

where the constant -1.645 is simply the percent point function of the standard normal CDF at probability 0.05. On the other hand, using Eqn 4.5,

$$\text{HCLPF} = m_R \exp[z_p \sqrt{(\beta_R^2 + \beta_U^2)}] \tag{4.7}$$

in which z_p = percent point function at probability, p, to be determined. Equating 4.6 and 4.7, we have,

$$z_p = -1.645\,(\beta_R + \beta_U)/\sqrt{(\beta_R^2 + \beta_U^2)} \tag{4.8}$$

In the usual case where β_R and β_U are of comparable magnitude,

$$\sqrt{(\beta_R^2 + \beta_U^2)} \approx 0.7(\beta_R + \beta_U) \tag{4.9}$$

whence $z_p = -2.32$ and $p = 0.01$; in other words, the HCLPF can be obtained from the mean fragility in Eqn 4.5 at a cumulative probability of 0.01 rather than 0.05.

The lognormal model of fragility in Eqn 4.1 admits the possibility of near-zero values of component capacity. However, properly designed and installed components generally have some minimum capacity close to their nominal design capacity. Previous studies have shown that modifying the lower tail of the fragility to take this effect into account has little impact on the estimated core damage probability.

Fragility Parameters

The component fragility can be analyzed as the product of factors (Kennedy and Ravindra, 1984)

$$R = \Pi\, F_i\,(\text{SSE}) \tag{4.10}$$

in which F_i = factor of safety due to source i (defined below). If factors F_i are mutually statistically independent, the median and variability can be defined by,

$$m_R = \Pi\, m_i\,(\text{SSE}) \tag{4.11}$$

$$\beta_R = \sqrt{\sum \beta_i^2} \tag{4.12}$$

in which m_i = median of factor of safety due to source i, β_i = logarithmic standard deviation describing the inherent randomness or uncertainty associated with factor i, and SSE = EPA due to the safe-shutdown earthquake. Moreover, the component capacity, R, is lognormal by virtue of the central limit theorem of probability theory, lending some theoretical justification to the selection of the lognormal distribution in Eqn 4.1 to model fragility.

Factors F_i in Eqn 4.10 are derived from plant-specific design analyses, interpretation of equipment qualification tests, and (in the case of some mechanical and electrical equipment) fragility test data or equipment qualification testing from military test programs. Although mechanical and electrical component fragilities often are developed generically, fragilities for structural components usually are estimated on an individual basis because they are plant-specific. Parameter β_i (β_R or β_U, as appropriate) is inferred from empirical data, wherever possible, or from expert professional judgement.

Fragility parameters m_R, β_R and β_U often must be estimated when no empirical data exist from the estimated maximum and minimum values, r_{max} and r_{min}, of the capacity R and maximum and minimum values, m_{max} and m_{min}, of the median M_R. Frequently, an expert or expert panel can provide so-called maximum and minimum credible values of structural capacity, even if empirical data do not exist. If these extreme values are viewed as encompassing 95 percent of the possible values of R and M_R, respectively, then

$$m_R \approx \sqrt{r_{max}\, r_{min}} \tag{4.13}$$

$$\beta_R \approx \ln(r_{max}/r_{min})^{1/4} \tag{4.14}$$

$$\beta_U \approx \ln(m_{max}/m_{min})^{1/4} \tag{4.15}$$

Recent seismic margins assessments (Kennedy, 1985) have focussed on estimating the median and a conservative lower bound capacity. Those recommending this approach point out that conservative-thinking engineers find it easier to estimate a lower bound capacity based on experience than an uncertainty (measured by a logarithmic standard deviation). With these parameters determined, the logarithmic standard deviations necessary to perform the risk analysis can be inferred if the relative magnitude of β_R and β_U can be estimated.

Structural components appearing in plant logic expressions often are, in fact, structural systems and may require a system reliability analysis to determine their fragility (Casciati and Faravelli, 1991). Conceptually, this might be done using structural system reliability analysis procedures. This has not been attempted for any PRA or margin studies to date in the United States. The principal obstacles in using system reliability analysis in fragility modeling are the lack of calibration points and how to calibrate limited structural failure data to the parameters in the system reliability analysis (Reed and McCann, 1984). Moreover, as noted above, the focus of fragility of structural components is on their integrated effect in plant safety systems. Limit states of structural performance are short of overall collapse and relate more to large inelastic deformations that may impair operation of attached mechanical or electrical equipment. Structural system reliability modeling to date has not focussed on such limit states.

4.2 Fragilities of Key Structural Components at Zion

Fragility parameters used in the plant logic models for the Zion NPP PRA are summarized in columns 2 - 4 of Table 2.1 (Zion PRA, 1981). The following review (1) explains the basis for the original fragilities; (2) points out deficiencies and where additional data are available; and (3) indicates how aging might affect structural component fragility modeling. It is assumed that mechanical and electrical equipment fragilities are appropriate, as provided in Table 2.1.

The Zion NPP was designed in the late 1960's using structural codes, seismic regulations and other documents in effect at that time. The structural design criteria were quite conservative. Stresses resulting from the DBE (now SSE) of 0.17 g in combination with other loads were limited to below yield for the steel frames and prestressing tendons. Deformed bar reinforcment in the reactor building was permitted to reach yield strength, yielding being defined further as structural deformations giving rise to strain in the steel liner of 0.005 in/in (Wesley, et al, 1980, 1981), but no post-yielding was allowed. The design did not take advantage of the ultimate flexural strength assumptions in ACI Standard 318, which permit going beyond yield; moreover, the maximum strain in the concrete was limited to 0.002. All design analyses were elastic.

For fragility analysis, structural failure is assumed to occur when inelastic deformations are sufficient to interfere with the operation of attached or adjacent safety-related components (e.g., piping or electrical equipment). Thus, the failure criterion for the assessment of risk of an existing facility is essentially a deformation limit, and is associated with the onset of structural damage rather than with collapse. This is a conservative estimate of the point at which actual structural collapse occurs. An assessment of nonlinear behavior beyond the point of first damage would seem to be important in analyzing an existing structure in a NPP. While a full nonlinear dynamic analysis may not be feasible, a static (or pushover) analysis involves less effort and can assist in this assessment.

The primary structural systems of Category I buildings at Zion are constructed of reinforced (or prestressed) concrete or steel. The critical concrete structural components in the Zion Plant are the shear wall between the auxiliary and turbine buildings (Component 8 in Table 2.1) and the roof of the crib house (Component 14), both of which appear as singletons in the core damage and plant damage state Booleans Eqs 2.5 and 2.6). The pressurizer enclosure roof (Component 26) also appears in the core damage Boolean in a doubleton cutset, but has a much higher median fragility.

The capacity of each structural component is given as,

$$R = F_{sc} F_{br} \text{ (SSE)} \tag{4.16}$$

in which F_{sc} = seismic strength capacity safety factor and F_{br} = building response safety factor. This breakdown facilitates analysis by different individuals or panels of experts. For risk assessment of existing facilities, best estimates of in-situ capacity are required, as well as variabilities and uncertainties in the in-service values. The factors F_{sc} and F_{br} in Eqn 4.16 are analyzed as follows:

$$F_{sc} = F_s F_\mu \tag{4.17}$$

$$F_{br} = F_{rs} F_{mc} F_{cc} F_{ssi} \tag{4.18}$$

Estimates of median structural fragilities are obtained by scaling up from the original design analyses as defined in the Final Safety Analysis Report (FSAR) for a NPP and by expert judgement and familiarity with the manner in which similar buildings and other structures have responded in earthquakes ("Uncertainty," 1986). The components of F_{sc} can be visualized from the load-deformation relationships for a structural component illustrated in Figure 4.2. While design calculations usually are elastic, the structure has substantial reserve strength due to the force redistribution that accompanies nonlinear behavior before failure. This reserve strength is recognized in modern aseismic design (e.g., NEHRP, 1992; Riddell and Newmark, 1984; Uang, 1991) and in fragility modeling. Figure 4.2 shows the actual force-deformation (e.g., in-plane shear vs interstory drift) relationship and two idealizations: one elastic and the second elastic-perfectly plastic. Factor F_s describes the factor of safety, V_u/V_d, between general structural yielding, V_u, and design level, V_d, the latter of which generally is in the elastic range. The overstrength arises from design factors of safety, from material strengths being higher than the nominal values assumed for design or members being oversize, from stiffening effect of nonstructural attachments, and from nonlinear redistribution of forces. It sometimes is convenient to break F_s down into the product of two components: $F_s = F_y F_{nl}$, in which $F_y = V_y/V_d$ is the factor of safety on first yield or plastic hinge, and $F_{nl} = V_u/V_d$ is the factor of safety on general yield, reflecting inelastic redistribution of forces subsequent to first yield. In allowable stress design, F_y typically is about 1.4 - 1.5; in limit states design, which is based on first yield (NEHRP, 1992), $F_y = 1$. Typical values of F_{nl} for low-rise walls would be 2.0 - 2.6; for stiff buildings with short periods, F_{nl} is especially important because ductility is less effective in this region of the reponse spectrum.

The capability of a ductile structure to dissipate energy through inelastic cyclic deformations is reflected in factor, F_μ, defined as the ratio V_{eu}/V_u in Figure 4.2. Factor F_μ depends on the damping in the system and ductility capacity. For a lightly damped single-degree-of-freedom system, F_μ is approximately $\sqrt{2\mu\text{-}1}$ in the acceleration-amplified region of the response spectrum (greater than about 2 Hz). The total deflection at failure is $F_{nl}\,\mu$ times the elastic deflection computed at force V_y.

In the original fragility evaluation for the Zion Plant structures (Wesley, et al, 1980) to determine F_s, the strength of steel and of concrete were treated as random variables. The difference between specified yield strength of reinforcing steel and the yield strength of reinforcement provided by the vendor was considered. Strength gain in concrete beyond the specified compression strength due to aging was considered, but all strength statistics were based on standard-cure cylinder tests. Differences in strength of low walls with boundary elements predicted by the ACI Standard 318 equations and from test data were considered. To determine F_μ, ductility ratios were estimated from load-deflection envelopes for short walls tested with axial loads. The ductility-based approach does not account for time-dependent effects such as frequency content and duration of excitation and cumulative damage from sequences of large acceleration pulses. Since these factors cannot be reflected in a strictly elastic analysis, they increase the uncertainty in F_μ. Dimensional variabilities that may affect variability of load-carrying capacity in ordinary building structures were neglected.

Aging may have an impact on F_{sc}, mainly through F_s but to a limited extent through F_μ as well. The increase in f_c for protected and undamaged concrete with time should be taken into account. Deterioration of the concrete due to aggressive substances decreases the effective section and thus F_s; this may affect some structural behavioral limit states more than others (Mori and Ellingwood, 1994c). Corrosion of reinforcement reduces the effective reinforcement area and thus F_s. Cracking due to shrinkage and thermal effects may impact stiffness and building response in a minor way. The latter may affect the ability of the component to dissipate energy and thus cause F_μ to decrease.

Building response factor, F_{br}, in Eqn 4.18 is analyzed as the product of several factors: F_{rs} = dynamic response factor reflecting differences between design and median ground response spectra, damping, and dynamic modeling; F_{mc} = mode combination factor; F_{ec} = factor reflecting the manner in which the horizontal and vertical components of earthquake force effects are combined and F_{ssi} = soil-structural interaction factor. Sources of randomness or uncertainty in F_{br} include: (i) Damping - participation of nonstructural elements, changes at various amplitude and stress levels; (ii) Stiffness - construction tolerances; cracking in concrete; variation in material properties; (iii) Mass - unit weights of materials; construction tolerances; spatial distribution, including accidental eccentricities; contribution of superposed loads; modeling techniques; (iv) Modeling - distribution of loads; lumping of continuous masses; uncoupling primary and secondary systems; two-dimensional representations; equivalent linear analysis; and (v) Response analysis, including selection of response spectrum (site-dependent or probabilistic vs design) and combining responses. As noted previously, variabilities due to frequency content and duration of excitation must be included in F_{br}, since the earthquake demand parameter is in terms of EPA. The treatment of nonlinear behavior with the accompanying redistribution of forces and energy dissipation, which is incorporated primarily in F_{rs}, is without question the most difficult aspect of fragility analysis. Different ground motion characteristics (frequency content, duration) may be significant for nonlinear response; this tends to increase the level of uncertainty in fragility.

Structural aging is believed to have little effect on most building response factors. While some increase in damping may occur due to cracking and component deterioration, the increase in overall variability will be small due to the number of uncertainties involved in Eqn 4.18.

The basis for the structural fragilities (components 8, 14 and 26 in Table 2.1) is given in the following discussion; parameters are summarized in Table 4.1 for later comparison with fragility parameters when aging is considered. The median fragilities in Tables 2.1 and 4.1 are given in terms of effective peak ground acceleration (EPA), a value that reflects the damage potential of the ground motion and that is typically on the order of 80% of the instrumental peak ground acceleration. The instrumental peak generally is associated with high frequencies to which the structure is insensitive. The seismic hazard curves in Figure 3.1 likewise are presented in terms of EPA. In some more recent studies, the fragility has been expressed in terms of the spectral acceleration. Provided that the fragility and seismic hazard both are expressed in terms of the same physical measure, the estimated risk measures are the same in both cases. When the demand on the system is specified in terms of EPA, the strength of materials, energy dissipation and structural response calculation and associated uncertainties must be reflected in the fragility curve. The conservatism with respect to the SSE (median fragilities all are greater than four times the DBE of 0.17g) occurs in part because the criteria used to design the plant did not recognize the inelastic energy absorption capacity of structures and in part because other considerations (e.g., protection of equipment from tornado-borne missile impact) may lead the components to be more massive than would be required by seismic-resistant design alone.

Improved approaches to aseismic design (e.g. Damping, 1973; Design, 1973) and to determine in-situ strength of concrete from cores (Bartlett and MacGregor, 1994) and nondestructive evaluations (Snyder, et al, 1992) and in fragility modeling have occurred in the past decade since the Zion PRA was completed. These improvements are ignored in the current study in favor of focussing solely on changes that would occur as a result of aging. Otherwise, the baseline fragility models would have to be redone to incorporate the new information.

8. Auxiliary building shear wall

The auxiliary building shear wall is a common wall between the auxiliary building and the turbine building. It consists of a braced steel frame with in-fill reinforced concrete panels. It is 108 ft (33 m) in height, 266 ft (81 m) long ($h_w/l_w = 0.41$) and 2 ft (610 mm) thick. Shear studs are welded in pairs at 12-in (300 mm) along the steel column webs to ensure composite action between the steel frame and the concrete panels. Failure of the shear wall may fail the structure housing critical equipment, leading to common-cause failures; excessive inelastic deformations may cause all attached piping to fail. The failure mechanism is highly nonlinear, involving failure first of the shear studs followed by redistribution of shear forces to the concrete panels and finally flexure/shear failure of the concrete panels that are unable to withstand the redistributed forces. In the original fragility analysis, the median fragility was 0.73g, with $\beta_R = 0.30$ and $\beta_U = 0.28$ (summarized in Table 4.1). The original fragility analysis is judged to be conservative with respect to m_R and somewhat unconservative with respect to β_R and β_U based on more recent information (Gergely, 1984).

14. Roof of crib house pump enclosure

The crib house is a partially open structure that houses the service water pumps, circulating water pumps and other equipment. The crib house pump enclosure is a concrete box-like structure situated within the crib house on the operating floor. The pump enclosure roof is a reinforced concrete slab 18 inches (457 mm) thick. It contains numerous large openings. Failure is assumed to occur from a loss of diaphragm action in the roof due to horizontal shear forces due to the earthquake; once diaphragm action of the roof is lost, the free-standing walls fail in flexure. Collapse of the roof is assumed to cause loss of function of all six service water pumps - an illustration of a common-cause failure that is typical of structures where component failure may affect several systems and eliminate redundancies that otherwise would be present. In the original PRA, the fragility parameters were $m_R = 0.86g$, $\beta_R = 0.24$, and $\beta_U = 0.27$ (Table 4.1). The roof of the pump enclosure is elongated in the NS direction, and fails due to EW excitation. In view of the eccentricities in the structures, the uncertainties in its response evaluation seem rather small.

26. Pressurizer enclosure roof collapse

The pressurizer enclosure is a reinforced concrete structure that sits on the operating floor within the reactor building and encloses that portion of the pressurizer that is above the operating floor. Three walls are cast-in-place; the fourth consists of moveable concrete panels for access. The roof is a 1-ft (305 mm) thick removeable slab that is bolted to the two permanent parallel walls. In the event of an earthquake, there is little diaphragm action and significant torsional response. Damage to the pressurizer enclosure may lead to rupture of the reactor coolant pressure boundary. In the original PRA, the fragility parameters were $m_R = 1.80$ g, $\beta_R = 0.39$, and $\beta_U = 0.34$ (Table 4.1).

4.3 Aging impacts on fragility

Structural aging and strength degradation impact the fragility parameters of the affected component. The median fragility is the parameter that generally is most affected. Strength degradation caused by loss of section (corrosion, deterioration of concrete slabs and walls) causes the median fragility to decrease. However, this decrease may not be linearly related to the fragility, which takes into account (albeit in a rudimentary manner) dynamic response and nonlinear behavior of the structural component. Thus, a 20% loss in section may not correspond to a 20% decrease in median fragility. There usually is some impact on variability and uncertainty, measured by β_R and β_U, as well; because of uncertainties in the degradation mechanisms, these variabilities will tend to increase as a result of aging. Structural fragility parameters, modified to take aging into account, are described in the following paragraphs and are summarized in the last several lines of Table 4.1. The impact of aging on parameter F_{br} (Eqn 4.18) was judged to be negligible, and thus changes are presented only for $F_{sc} = F_s F_\mu$. For reasons noted above, we focus on changes from the baseline fragility analysis.

Plant walkdowns conducted as part of in-service inspection/maintenance programs may be used to develop information that can be used to update the fragility models used in the risk assessment. This updating can be done using a Bayesian procedure. Sensitivity studies are important in that they can reveal which components are particularly important for plant performance and where in-service inspection efforts should be concentrated.

8. Auxiliary Building Shear Wall

Failure of the wall is initiated by failure (yielding) of the shear studs necessary to achieve composite action of the steel frame and concrete in-fill panels. It is conceivable that some damage to these shear studs could have occurred during the previous service life of the plant due, for example, to a previous earthquake. If the studs were to become completely ineffective, F_s would decrease but F_μ would increase at the same time, as the more ductile flexure mode of failure becomes dominant. The net effect would be to decrease the median fragility from 0.73g to approximately 0.65g, a decrease of approximately 10 percent (e.g., Ravindra, et al, 1984). At the same time, additional uncertainties in strength and damping cause β_R and β_U to increase to 0.34 and 0.30, respectively.

Shear wall deterioration due to expansive aggregate reactions might in an extreme case cause the median fragility to be reduced 25% from 0.73 g to 0.55g. However, damage of this magnitude would likely be detected during a normal walk-through inspection.

14. Cribhouse pump enclosure roof

Shear capacity of the roof slab is a function of concrete strength, reinforcement strength and placement, and detailing of the wall-slab joint. Corrosion of the reinforcement in the roof slab would require the slab to resist shear forces mainly by diaphragm action of the concrete acting alone. If the reinforcement becomes completely ineffective and the concrete resists the entire shear force, the median fragility is reduced by approximately 35% from 0.86g to 0.46g (Ravindra, et al 1984). At the same time, the β_R and β_U increase to 0.32 and 0.29, respectively. The uncertainties have been increased to account for eccentricities in the inertial forces, the effects of which would increase due to nonuniform strength degradation of the roof.

26. Pressurizer Enclosure Roof

The pressurizer enclosure roof is protected from the elements, so corrosion of the reinforcement in the roof slab is not likely to be a problem If expansive aggregate reactions cause a decrease of 25% in median in-plane shear capacity, the median fragility would be reduced from from 1.8 g to about 1.35 g; $\beta_R = 0.41$ and $\beta_U = 0.29$. As with the shear wall above, however, damage of this magnitude likely would be detected during a normal walk-through inspection.

Table 4.1 Fragilities of structural components

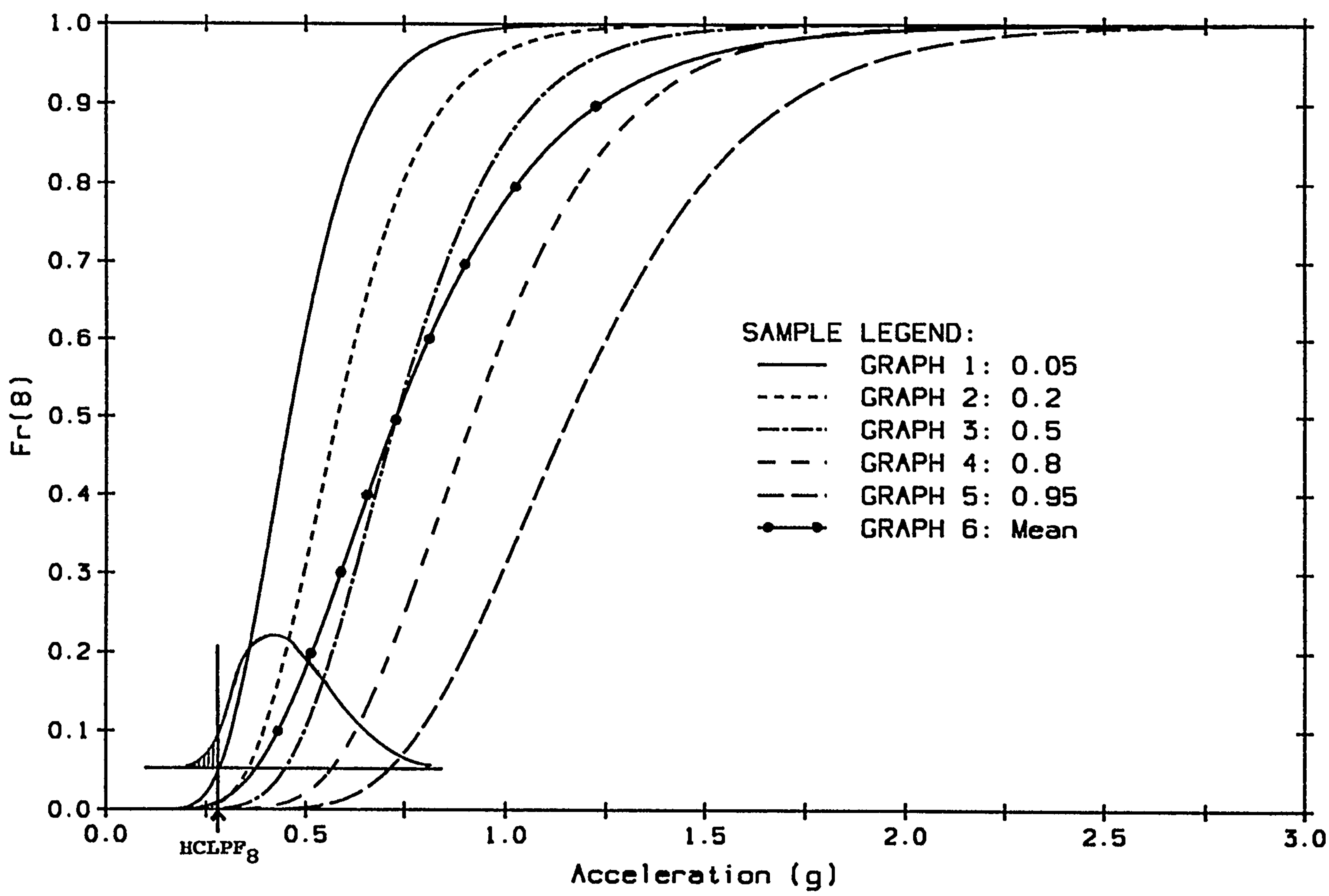
4.1a Original study, without aging

Structural component (cf Table 2.1)

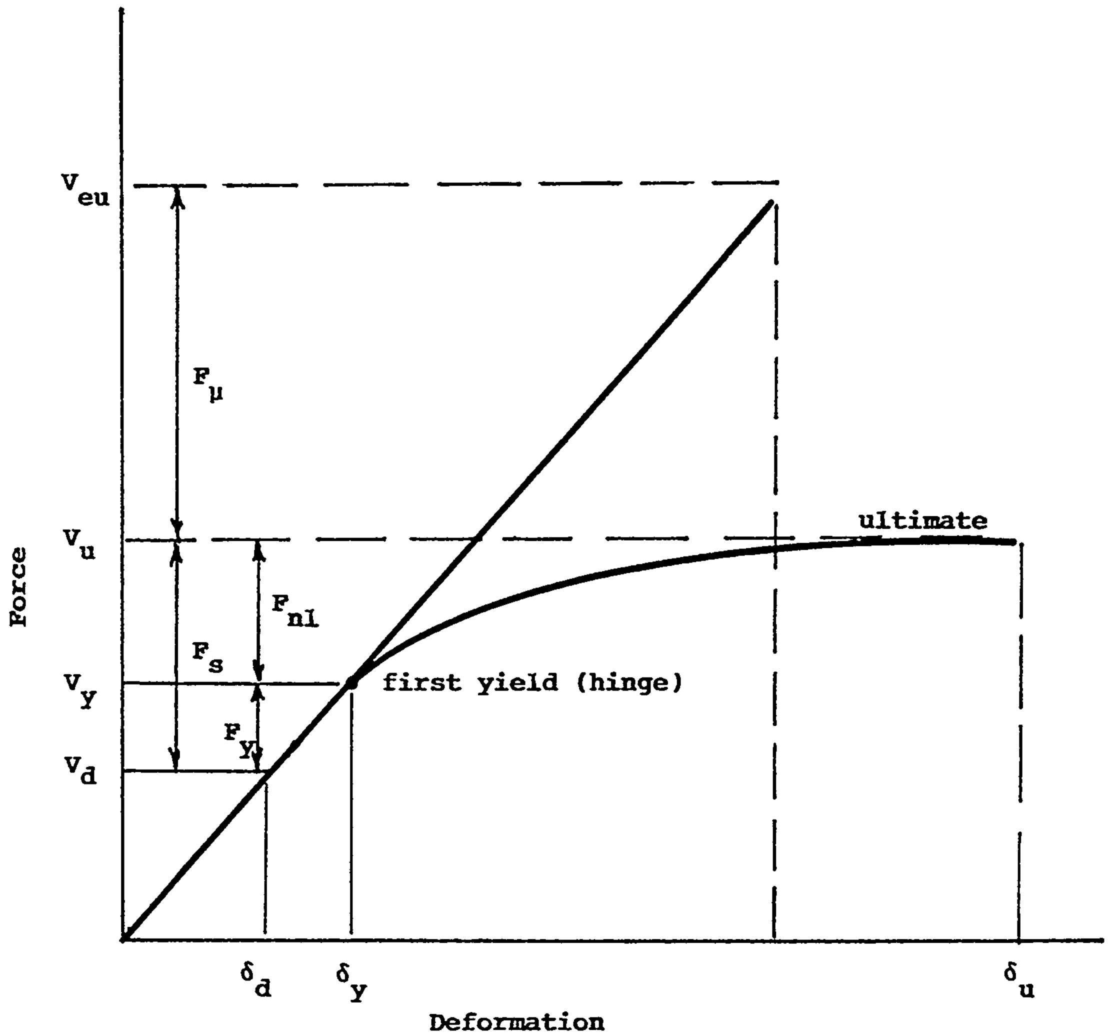
Parameter	8			14			26		
	m	β_R	β_U	m	β_R	β_U	m	β_R	β_U
F_{sc}	4.50	0.12	0.20	4.80	0.15	0.18	7.50	0.27	0.14
F_s	2.50	0.07	0.14	3.70	0.10	0.15	5.00	0.13	0.10
F_μ	1.80	0.10	0.14	1.30	0.11	0.10	1.50	0.24	0.10
F_{br}	0.95	0.28	0.20	1.06	0.19	0.22	1.43	0.28	0.31
F_{rs}	0.79	0.22	0.13	0.88	0.06	0.16	1.10	0.19	0.18
F_{mc}	1.00	0.17	0.00	1.00	0.18	0.00	1.00	0.17	0.00
F_{ec}	1.00	0.00	0.00	1.00	0.02	0.00	1.00	0.12	0.00
F_{ssi}	1.20	0.00	0.15	1.20	0.00	0.15	1.30	0.00	0.25
R	0.73g	0.30	0.28	0.86g	0.24	0.27	1.80g	0.39	0.34

4.1b After structural components have aged:

Parameter	8			14			26		
	m	β_R	β_U	m	β_R	β_U	m	β_R	β_U
F_{sc}	4.00	0.21	0.21	2.50	0.21	0.18	5.52	0.30	0.14
F_s	1.00	0.18	0.15	2.50	0.18	0.15	4.60	0.18	0.10
F_μ	4.00	0.10	0.14	1.00	0.11	0.10	1.20	0.24	0.10
R	0.65g	0.34	0.30	0.46g	0.32	0.29	1.35g	0.41	0.39



4.1 Fragility family for auxiliary building reinforced concrete shear wall failing in shear



4.2 Safety factors in fragility analysis

5. Risk Assessment

The Zion Plant has been the subject of several previous PRA studies, including a number that were conducted as part of the Seismic Safety Margins Research Program (e.g., Bohn, et al, 1984). The plant logic has been thoroughly reviewed and the dominant causes of core damage are believed to be well understood. Plant logic models for Zion Unit No. 1 core damage and plant damage state sequences CD and SE were summarized in Section 2.

If the fragility and hazard parameters, $\underline{m}_R$ and (μ,γ), are known, the probabilities of core damage, P(CD), (or damage state SE) can be obtained by

$$P(CD|\underline{m}_R,\mu,\gamma) = \int P(CD|A = x,\underline{m}_R)\, dG_A(x,\mu,\gamma) \tag{5.1}$$

in which $P(CD|A = x, \underline{m}_R)$ is obtained from Eqn 2.5 using the individual component fragilities in Section 4, and $dG_A(x,\mu,\gamma)$ is the seismic hazard in Eqn 3.2. However, the seismic hazard and component fragility models are uncertain (e.g., Figures 3.1 and 4.1), and thus the estimate of P(CD) computed from these plant logic models is a function of uncertain variables $(\underline{m}_R, \mu, \gamma)$ and has a distribution that reflects this uncertainty. The effect of these uncertain parameters can be displayed as a cumulative distribution function (CDF) of probability, P(CD) (sometimes termed cumulative frequency of probability), with an associated mean and variance. Moreover, even with $(\underline{m}_R, \mu,\gamma)$ fixed, P(CD) cannot be determined in closed form and must be determined by Monte Carlo simulation. Thus, the numerical procedure introduces an additional source of uncertainty in the form of a sampling distribution for P(CD).

When a structural component deteriorates due to aging, the fragility parameters for that component change, as described in Section 4. Such changes may cause P(CD) to increase. To determine whether such changes merit additional attention in evaluating continued service, one must determine whether they have a statistically significant impact on P(CD). Sampling distributions and the sensitivity of P(CD) to changes in fragility parameters for aging components must be evaluated in order to separate the effects of aging from sampling error in the risk estimation procedures.

Section 5 assesses the impact of aging of reinforced concrete structural components on plant risk, measured by core damage and plant damage state probabilities, through a series of sensitivity studies involving Zion sequences CD and SE. While the results obviously are plant-specific, the consistency of the general findings from the Zion PRA with findings from PRAs of other plants (e.g., Sues, et al, 1990) suggests some general guidelines and inferences for assessing service life extensions that can be drawn from such an assessment.

5.1 Review of Previous Work

Some results from previous work are summarized to validate the computations and results obtained in the present study. Note that none of the previous PRAs considered the the possibility of structural aging or its potential for impacting plant risk.

5.1.1 Seismic PRA

Two sets of previous results are presented; one was done as part of the original PRA for the Zion Unit 1. The second was performed as part of a later sensitivity study (Ravindra, et al, 1984). In the original PRA, the mean value of P(CD) was calculated as 5.6×10^{-6}/yr. The fractiles of the CDF were:

5%ile	3.0×10^{-8}/yr
50%ile (median)	2.0×10^{-6}/yr
95%ile	3.0×10^{-5}/yr

In the later sensitivity study of dominant contributors to seismic risk, the mean of P(CD) was not provided but the following fractiles of the CDF of P(CD) were obtained:

5%ile	4.7×10^{-7}/yr
50%ile (median)	3.9×10^{-6}/yr
95%ile	2.9×10^{-5}/yr

The difference of one order of magnitude in the 5%ile values is due to the truncation of the seismic hazard curves in the earlier study. The large dispersion (two to three orders of magnitude) in P(CD) is characteristic of what has been found in seismic PRAs of other plants. The uncertainty due to the seismic hazard (displayed in Figure 3.1) is the main contributor to this dispersion in P(CD). The integrand in Eqn 5.1 is significant mainly in the range 0.15g - 0.9g and so EPAs in the range of approximately 1 - 5 SSE contribute most to P(CD). The importance of structural failures is amplified in an external events PRA because many structures appear in the plant Booleans as singleton cutsets and structural failures impact many safety-related systems simultaneously (so-called common-cause failures).

5.1.2 Internal Events PRA

The mean core damage frequency from internally initiated events was estimated to be 5.7×10^{-5}. Thus, the mean core damage probability due to earthquake is an order of magnitude less. In contrast to the seismic PRA, however, no structural components appear in the core damage Booleans for internal events. The dominant contributors to risk from internal events were the reactor pressure vessel, service water system, and reactor coolant systems. The mean core damage frequency due to earthquake (5.6×10^{-6}) is only 11% of the overall mean core damage frequency (5.2×10^{-5}). However, earthquake accounts for 98% of the latent fatality risk at Zion (Ravindra, 1988). Comparisons for other plants show a similar impact of earthquake (and, by inference, earthquake-resisting structural components and systems) on overall plant risk.

5.1.3 Seismic Margins Analysis

Plant-level fragilities for CD or SE can be determined from the individual component fragilities in Table 2.1, once these are related through the plant logic expressions (Eqns 2.3 - 2.6). The family of plant fragilities displays the cumulative effect of uncertainties in the assumptions (reflected in the component median fragilities) underlying the component fragility determination, given that the EPA is equal to a particular value. Using the plant fragility analysis, the plant HCLPF can be identified as the 95% lower confidence interval on the 5% exclusion limit of acceleration; in other words, that value of acceleration below which it can be said (with 95% confidence) that the probability of core damage is less than 5%. In a seismic margins study, this plant-level HCLPF is compared to a review or screening level of the seismic hazard, specified in consistent terms (e.g., 0.3g EPA). This approach effectively uncouples the plant logic and fragility modeling from the seismic hazard analysis, which is the largest source of uncertainty. In a previous margins study for Zion Unit No. 1 (Budnitz, et al, 1985), the HCLPF reported for plant damage state SE was found to be approximately 0.27g based on instrumental peak ground acceleration.

5.2 Analysis of Risk and Uncertainty

5.2.1 Baseline Case

The impact of structural component aging on plant risk and seismic margins will be displayed by considering the changes in: (1) CDF of P(CD); (2) HCLPF; and (3) a point estimate of risk. The sampling distributions necessary to determine the CDF and HCLPF are determined by Monte Carlo simulation. A sufficient number of samples must be analyzed so that estimates of the mean, median or 5%ile probability are relatively stable. Otherwise, differences in estimated probability due to aging, degradation and other factors, may be indistinguishable from sampling error. In the cases that follow, the CDFs of P(CD) and P(SE) are based on 270 probability estimates (30 vectors of fragility medians, $\underline{m}_R$, convolved at random with 9 seismic hazard curves, appropriately weighted - see Figure 3.1). Estimates of the HCLPF are based on the 30 system fragilities.

The CDFs of P(CD) are illustrated in Figure 5.1 for 5 repetitions of the above experiment, displaying the sampling error associated with this particular Monte Carlo procedure. As in the studies summarized in section 5.1.1, these CDFs span three orders of magnitude. The sampling distributions in the estimate of the mean and median of P(CD) (based on 20 repetitions of the experiment) are illustrated in Figures 5.2a and 5.2b by cumulative distribution functions plotted on normal probability paper. The means and standard deviations of the estimated means and medians are summarized below, along with the 5th and 95%iles of P(CD):

Estimated Parameter		Mean	Standard deviation
5% ile	P(CD)	2.1×10^{-7}	-
Median	P(CD)	1.9×10^{-6}	0.34×10^{-6}
	P(SE)	1.8×10^{-6}	0.33×10^{-6}
Mean	P(CD)	5.3×10^{-6}	0.75×10^{-6}
	P(SE)	5.3×10^{-6}	0.74×10^{-6}
95% ile	P(CD)	2.2×10^{-5}	-

These means, medians and fractiles are consistent with the values obtained in previous seismic PRA studies (section 5.1.1), and the dispersions are comparable. Estimates of P(CD) and P(SE) are very close because the Boolean Eqs 2.5 and 2.6 contain the same components as singleton cutsets. The mean of P(CD) corresponds approximately to the 70th percentile of the CDF of P(CD) in Figure 5.1, indicating that the CDF is strongly skewed in the positive direction. This also implies that the more conservative of the seismic hazard curves in Figure 3.1 dominate the estimates of the upper fractiles of P(CD). If the sampling distribution for the estimated mean of P(CD) is assumed to be normal, the 90% "confidence interval" on the mean is $(4 \times 10^{-6}, 6.6 \times 10^{-6})$.

The plant level fragility for core damage, CD, is illustrated in Figure 5.3a. The HCLPF(CD), computed as the 95% lower confidence limit on the 5% exclusion limit, is approximately 0.24g (EPA). Multiplying by the factor 1.23 relating the EPA to instrumental peak acceleration (Section 3), one would obtain a HCLPF of approximately 0.29g (instrumental peak). The mean fragility for a component is defined by Eqn 4.5. Performing the plant logic calculations using the mean component fragilities collapses the plant level fragility for CD from the family of curves in Figure 5.3a to the single curve, $F_R(x)$, indicated in Figure 5.3b. While this $F_R(x)$ is not the mean fragility for the plant-level sequence, CD, it is a reasonable approximation to it. The 5% exclusion limit acceleration from Figure 5.3b is 0.26g. It should be noted that the mean fragility in Figure 5.3b is not described by a lognormal distribution.

The results obtained in the present numerical study are consistent with those obtained previously, both for P(CD) and for the HCLPF. Subsequent sensitivity studies will illustrate the deviations from this so-called baseline case due to various parametric variations and assumptions. Consistent with the scatter evidenced in the results in section 5.1.1 and in Figures 5.2a and 5.2b, it will be assumed that differences of a factor of 25 percent or less in point estimates of P(CD) or P(SE) are not statistically significant.

5.2.2 Seismic Risk due to Uncertainty in Fragility

The seismic hazard is the dominant contributor to uncertainty in the estimate of core damage probability, evidenced by the CDF in P(CD) in Figure 5.1 (Ellingwood, 1994a). The uncertainty in seismic hazard cannot be reduced at the current state of the art. The contribution of uncertainty in the fragility analysis to uncertainty in P(CD) is actually relatively small in comparison. To see this, we replace the family of hazard curves in Figure 3.1 with their weighted mean, computed as,

$$\overline{G_A}(x) = \sum w_i \, P\,[A>x|\ i] \tag{5.2}$$

in which w_i = weight associated with seismic hazard curve $G_A(x|\mu_i,\gamma_i)$ (viz Figure 3.1 and/or Table 3.1). This hazard curve can be described, in approximation, by a Type II distribution of largest values, with parameters $\mu = 0.0117$ and $\gamma = 3.394$. The CDF of P(CD) under this assumption is illustrated in Figure 5.4 for 5 repetitions of the Monte Carlo analysis involving 30 random vectors $\underline{m}_R$. The uncertainty in P(CD) exhibited in this figure is due solely to uncertainty in the (median) component fragilities, and has been reduced from three orders of magnitude in the previous case to less than one order of magnitude. The following fractiles are obtained for P(CD):

5%ile	1.6×10^{-6}
50%ile	4.6×10^{-6}
95%ile	1.2×10^{-5}

The difference between Figures 5.1 and 5.4 is most pronounced below the 70th percentile of the CDF, providing further support to the assertion that conservative and heavily weighted seismic hazard hypotheses dominate the upper fractiles of P(CD).

5.2.3 Point Estimate of Risk

There are sampling errors associated with the Monte Carlo algorithm, as illustrated in Figures 5.1, 5.2 and 5.4. In performing a parametric sensitivity study, one would like to assert that differences observed in P(CD) or HCLPF are (or are not) attributable to variations in the parameters being studied. In a complex problem such as the one under consideration, such differences can be masked by the sampling error inherent in the numerical algorithm. One can, of course, perform statistical tests to assess whether the differences are statistically significant. Such a procedure leaves open the possibility of Type I or Type II errors, which can be attacked by: (1) increasing the number of samples (computations), at considerable expense, or (2) using a variance reduction procedure to reduce the sampling error (Rubenstein, 1981). An alternative and simple approach is to base any comparisons on an appropriate point estimate of probability. It often has been suggested that the mean probability should furnish the point estimate in support of decision-making (e.g., Lewis, 1985). The mean of P(CD) or P(SE) can be estimated by convolving the mean, $F_R(x)$, in Figure 5.3b with the mean seismic hazard in Eqn 5.2.

For the Zion Unit No. 1 sequences above, these point estimates are:

$$P(CD) = 9.5 \times 10^{-6}$$
$$P(SE) = 9.5 \times 10^{-6}$$

They correspond approximately to the 80%ile of the cumulative frequency distribution for P(CD) (or P(SE)), as illustrated in Figure 5.1, and are approximately 1.8 times the mean values of P(CD) and P(SE) obtained from the fully coupled analysis in Section 5.2.1. Since such differences are not normally considered significant in seismic PRA, these point estimates will be used in subsequent sections to assess the impact of structural aging on plant risk. The results are nearly identical for P(CD) and P(SE).

5.3 Role of Aging on Plant Risk

In this section, the impact of aging on fragility of structural components is examined and the sensitivity of P(CD), P(SE), and HCLPF to aging effects is determined. First, however, we present some general information about importance and sensitivity analysis of accident sequence probabilities.

5.3.1 Importance/Sensitivity Analysis

Importance and sensitivity analysis are directed at providing answers to the following questions:

How much does one component contribute to plant risk?

To what extent does changing a component reliability or component fragility parameter impact the estimated risk or HCLPF?

Which parameters have the most effect on accident sequence probabilities, on risk, and on the uncertainty attached to the risk estimate?

By answering such questions one can determine the extent to which risk is reduced by changes to any parameter, where to invest in condition assessment so as to maximize the benefits in reducing risk, and how to manage risk optimally.

Two measures that have been widely used for ranking and selecting components are the Vesely-Fussell and Birnbaum Importance measures (Barlow and Proschan, 1976). The Vesely-Fussell (V-F) measure ranks the importance of a single component by the ratio of the probability of the cutsets containing that component divided by the probability of the top event (here, P(CD) or P(SE)). In a sense, then, it measures how much of the current risk is generated by the component in question. The Birnbaum Importance measure uses the derivative of core damage probability with respect to the failure probability of a component to evaluate the impact of a change in component risk on system risk.

As a simple illustration of these concepts, suppose that a system depends on three components and that failure of the system is given by the Boolean expression,

$$F = F_1 + F_2 * F_3 \tag{5.3}$$

Assuming that component failures are statistically independent events, the probability of system failure is,

$$P_F = P_1 + P_2 P_3 - P_1 P_2 P_3 \tag{5.4}$$

in which $p_i = P(F_i)$. The V-F measures for components 1, 2 and 3 are

$$I_{vf_1} = P_1 / P_F \tag{5.5a}$$

$$II_{vf_2} = I_{vf_3} = P_2 P_3 / P_F \tag{5.5b}$$

The V-F measure does not distinguish between the importance of components 2 and 3, since failure of both components 2 and 3 must occur for the system to fail. In contrast, the Birnbaum measures for components 1, 2 and 3 are,

$$\partial P_F / \partial P_1 = 1 - P_2 P_3 \tag{5.6a}$$

$$\partial P_F / \partial P_2 = P_3 - P_1 P_3 \tag{5.6b}$$

$$\partial P_F / \partial P_3 = P_2 - P_1 P_2 \tag{5.6c}$$

Here, components 2 and 3 are differentiated, as intuition would suggest.

A third measure of component importance can be obtained by comparing the conditional probability of system failure, given that component failure has occurred, to the unconditional failure probability. For the three-component example above,

$$P(F|F_1) = 1 \tag{5.7a}$$

$$P(F|F_2) = P_1 + P_3 - P_1 P_3 \tag{5.7b}$$

$$P(F|F_3) = P_1 + P_2 - P_1 P_2 \tag{5.7c}$$

Note that these probabilities are similar to, but not exactly the same as, the Birnbaum measures, provided that the individual probabilities are small. In particular, Eqn 5.7a indicates that the conditional probability of failure, given failure of component 1, is 1.0; this might be expected from the series form of the Boolean expression. The conditional probability formulation emphasizes the importance of those structural component failures that appear in plant Booleans as singleton cutsets.

If failure is conditioned on the occurrence of a specific ground (or spectral) acceleration, x, as in the component and system fragility analysis portion of a PRA, then all probabilities are conditioned on EPA = x and the factors computed by Eqns 5.5 - 5.7 are functions of x. Thus, components that are important for accident mitigation for moderate earthquakes (EPA << SSE) may be supplanted by other components for great earthquakes (EPA larger than the SSE). The unconditional system or sequence probability cannot be expressed by simple relations such as that in Eqn 5.4 because the component failures are not statistically independent but are correlated through the structural actions due to the common earthquake ground motion. The system failure probability must be determined by convolving the system fragility with the seismic hazard (Eqn 5.1), and the component failure probabilities do not appear explicitly in this convolution.

An overall sense of the importance of the various components to plant risk can be obtained by convolving the importance measures in Eqns 5.5 - 5.7 with the mean seismic hazard, denoted by Eqn 5.2. This removes the conditioning of the factor on the level, EPA = x, of the seismic hazard. However, such an approach is not informative with regard to differences that might exist in the relative importance of components at different levels of earthquake ground motion. Such information must be gleaned from an examination of the behavior of the importance measures as increasing functions of x.

5.3.2 Application to Zion Unit 1

The above concepts are applied to Zion Unit 1 sequences CD and SE. The V-F measures (conditioned on A = x) with respect to core damage CD and plant damage state SE for the three structural components are:

Shear wall:	$P_8/P(CD)$	(5.8a)
	$P_8/P(SE)$	(5.8b)
Crib house pump enclosure roof:	$P_{14}/P(CD)$	(5.8c)
	$P_{14}/P(SE)$	(5.8d)
Pressurizer enclosure roof:	$P_9P_{26}/P(CD)$	(5.8e)
	$P_9P_{25}P_{26}/P(SE)$	(5.8f)

The sensitivity measures of P(CD) with respect to these components (conditional on A = x) are,

$$\partial P(CD)/\partial P_8 = 1 - P(A_1) \tag{5.9a}$$

$$\partial P(CD)/\partial P_{14} = 1 - P(A_2) \tag{5.9b}$$

$$\partial P(CD)/\partial P_{26} = P_9 * (1 - P(A_3)) \tag{5.9c}$$

in which A_1, A_2, A_3 = collections of component failures not including those components for which sensitivities are computed (cf Eqn 2.5). A similar set of sensitivities is obtained for P(SE).

The component fragilities are functions of the three parameters (m_R, β_R, β_U). The sensitivity of P(CD) or HCLPF to any fragility parameter,e.g., median m_8, can be established using the chain rule of differentiation:

$$\partial P(CD)/\partial m_8 = (\partial P(CD)/\partial P_8)(\partial P_8/\partial m_8) \tag{5.10}$$

The first term on the right hand side of Eqn. 5.10 can be computed from Eqn 5.9a. The second term, $\partial P_8/\partial m_8$, can be computed directly from the fragility model. If the component fragility R is modeled by a lognormal distribution, this derivative is,

$$\partial P_8/\partial m_8 = -(m_8 \beta_8)^{-1} \ \phi\left(\frac{\ln(x/m_8)}{\beta_8}\right) \tag{5.11}$$

in which $\phi(\ )$ = standard normal probability density function. The change in P(CD) as a function of change in m_8 is,

$$\Delta P(CD) = (\partial P(CD)/\partial m_8)\, \Delta m_8 \tag{5.12}$$

and similarly for parameter β_S.

The dependence of the sensitivity factors in Eqs 5.9 - 5.12 on acceleration, x, can be removed by integrating over the relative frequencies of acceleration. The change in P(CD) as a function of change in m_8 thus is:

$$\partial P(CD)/\partial m_8 = \int_0^1 \frac{\partial P(CD)}{\partial P_8} \left| \frac{\partial P_8}{\partial m_8} \right| dG_A(x) \tag{5.13}$$

Figure 5.5 illustrates the variation in I_{vf} for sequence CD while Figure 5.6 illustrates the variation in the Birnbaum measure, $\partial P(CD)/\partial P_i$, both with respect to increasing ground acceleration, x. Figures 5.7 and 5.8 illustrate the sensitivities of P(CD) to variations in the component medians and overall variabilities $\sqrt{(\beta_R^2 + \beta_U^2)}$. As the effective peak ground acceleration increases, the sensitivity of P(CD) to failure of either component 8 or 14 decreases. Sensitivities for components 9 and 26, which appear in a doubleton cutset, reach a maximum at approx 0.5g, but are substantially less at all values of x than those for components 8 and 14. The results for sequence SE presented in Figures 5.9 - 5.12 are similar; the sensitivity factors for the RWST (9) and pressurizer enclosure (26) are not perceptible because these components appear in a triple cutset (cf Eqn 2.6). Figures 5.5 -5.12 confirm what common sense would dictate: that core damage probabilities are governed by those few components in a sequence with low (or highly uncertain) seismic capacity (measured by small median m_R or large β_R or β_U) that appear in the accident sequence Booleans as singletons.

When the V-F and Birnbaum measures for CD are convolved with the mean seismic hazard curve, the following values are obtained:

	Component		
Measure	8	14	26
V-F x 10^{+6}	15.5	1.76	0.39
Birnbaum	1.06	1.06	0.73 x 10^{-6}

Similar results are obtained for SE. The auxiliary building shear wall (8) and cribhouse roof slab (14) are the most significant structural components insofar as plant risk is concerned. Therefore, the impact of deterioration due to aging on plant risk will be illustrated in the next section by focussing on the shear wall (8) and cribhouse roof (14). The effects of aging on other structural components at Zion Unit No. 1 have a negligible impact on seismic risk.

5.3.3 Impact of Aging on Plant Risk

<u>8. Auxiliary building shear wall</u>.

Failure of the wall is initiated by failure of the shear studs necessary to achieve composite action of the steel frame and concrete in-fill panels. It is conceivable that damage to these shear studs could have occurred during the previous service life of the plant due, for example, to a previous earthquake. If the median fragility is reduced from 0.73g to 0.65g and the uncertainties increase, as indicated in Table 4.1, P(CD) increases from 9.5 x 10^{-6}/yr to 1.1 x 10^{-5}/yr. The HCLPF decreases from 0.24g to 0.22g.

<u>14. Cribhouse pump enclosure roof</u>.

Excessive corrosion of the reinforcement in the roof slab may reduce or virtually eliminate its contribution to resisting shear force. If the reinforcement is completely ineffective and the median fragility is reduced from 0.86g to 0.46g, the mean core damage probability increases to 1.7 x 10^{-5}/yr while the HCLPF decreases from 0.24g to 0.18g. If the auxiliary building shear wall and roof both have degraded the full amount, P(CD) increases to 1.9 x 10^{-5} while the HCLPF decreases to 0.17g. The assumption of simultaneous degradation in these two structures is very conservative.

26. Pressurizer Enclosure Roof

With median fragility decreasing to 1.35g, P(CD) remains at 1.9×10^{-5} and HCLPF(CD) = 0.17g, as in the previous section, and it may be concluded that failure of the pressurizer enclosure roof has no measurable impact on P(CD) or the HCLPF(CD) .

Summary

The results of the aging analysis are summarized in the table below:

Component aging	P(CD)	HCLPF
None	9.5×10^{-6}	0.24g
8	10.9×10^{-6}	0.22g
14	17.5×10^{-6}	0.18g
8 and 14	18.6×10^{-6}	0.17g
8, 14 and 26	18.6×10^{-6}	0.17g

This analysis indicates that aging of structural components 8, 14 and 26 has a minimal impact on seismic core damage probability (less than a factor of 2), but a more significant impact on HCLPF (approximately 20%). Thus, the importance of structural aging may seem more apparent if plant evaluations are based on a margins-type of analysis rather than on a fully coupled Level 1 PSA.

5.3.4 Time-dependent changes in risk

The rate of increase in component failure or risk of core damage can be investigated systematically using the sensitivity factors in Eqns 5.9 - 5.14. We consider two cases. In the first case, EPA = x is fixed, as if it were selected as a screening level event in a seismic margins analysis, where it is desired to demonstrate that the component or plant HCLPF is greater than x. In the second, the conditioning on EPA is removed, as in Eqn 5.1. In both cases, it will be assumed that only the auxiliary building shear wall (component 8) ages, and that aging is manifested by a reduction in the median, m_8.

Let us suppose that EPA = 0.3g, a reasonable level for a screening level earthquake when SSE = 0.17g. The rate of change in P(CD) with time is,

$$\partial P(CD)/\partial t = [\partial P(CD)/\partial P_8 \; \partial P_8/\partial m_8] \; \partial m_8/\partial t \qquad (5.14a)$$

in which (See Figures 5.6 and 5.7),

$$\partial P(CD)/\partial P_8 \approx 0.91 \qquad (5.15a)$$

$$\partial P_8/\partial m_8 = -0.13 \qquad (5.15b)$$

If x = 0.3g, P(CD) increases at a rate,

$$\partial P(CD)/\partial t \approx -0.12\, \partial m_8/\partial t \tag{5.14b}$$

If $\partial m_8/\partial t = -0.025$ (degradation to 90% of initial strength in a service life of 40 years), $\partial P(CD)/\partial t = 0.003$, a rate of change that is quite large; however, the probability that EPA = 0.3g is very small (see Figure 3.1). The impact of aging on P(CD) when considering all possible hazards is much smaller.

The failure probability of the auxiliary building shear wall itself is,

$$P_8 = -\int_0^1 \Phi \left(\frac{\ln(x/m_8)}{\beta_8} \right) dG_A(x) \tag{5.16}$$

It can be shown (Ellingwood, 1994) that P_8 can be approximated by,

$$P_8 \approx \tilde{G}_A(m_8) \exp((\beta_8 \gamma)^2/2) \tag{5.17a}$$

in which $\tilde{G}_A(m_8) = (m_8/\mu)^{-\gamma} \approx$ seismic hazard evaluated at median m_8. The increase in P_8 with t then becomes,

$$\partial P_8/\partial t = -(\gamma P_8/m_8)\partial m_8/\partial t \tag{5.17b}$$

With $\mu = 0.0117$g, $\gamma = 3.394$, $m_8 = 0.73$g and $\beta_8 = 0.41$:

$$\partial P_8/\partial t = -9.91 \times 10^{-6}\, \partial m_8/\partial t \tag{5.17c}$$

This rate is four orders of magnitude smaller than the rate indicated in Eqn 5.14b when EPA = 0.3g. The rate of increase in P_8 due to the same 10-percent degradation in strength in 40 years as before is of the order 10^{-7}.

Finally, the increase in P(CD) in time is,

$$\partial P(CD)/\partial t = -\frac{1}{m_8 \beta_8} \int_0^1 \frac{\partial P(CD)}{\partial P_8} \varphi(\frac{\ln x/m_8}{\beta_8})\, dG_A(x)]\, \partial m_8/\partial t \tag{5.18a}$$

Performing the integration numerically, we find that,

$$\partial P(CD)/\partial t = [\,5.94 \times 10^{-6}]\,\partial m_8/\partial t \quad (m_8 = 0.73g) \tag{5.18b}$$

For the same aging effect on m_8, the rate of increase in P(CD) with time is less than the rate of increase in P_8.

Thus, one arrives at different conclusions regarding the role and importance of structural component aging, depending on the level at which the reliability analysis is performed and whether the risk analysis is fully coupled. It is apparent that while a margins-type of analysis might be useful in identifying dominant contributors to risk in a qualitative sense, it may lead to an overly pessimistic numerical measure of the role of aging on plant risk.

Risk analysis performed using the plant logic models in Section 2, hazard and fragility models in Sections 3 and 4, and the methods in Section 5 provides a snapshot of risk, measured by P(CD), for a given window of time (here, one year, since the hazard curves in Figure 3.1 are expressed in terms of annual probability). As the median fragility, m_8, of the auxiliary building shear wall decreases from 0.73g to 0.65 g over a 40-year service life, P(CD) increases from 9.5×10^{-6} in year 1, when the component is undamaged) to 1.08×10^{-5} in year 40, when the capacity of the wall to withstand earthquake has degraded by 11 percent (cf Section 5.3.3). For predicting the useful service life or for scheduling in-service inspection and maintenance activities, one might be more interested in the probability of satisfactory performance over some period of time, say (t_1,t_2), than in the snapshot of reliability at time t.

The time-dependent reliability of a system can be analyzed once the hazard function for that system is known. The hazard function, h(t), describes the failure rate or conditional probability of failure as a function of elapsed time (Ellingwood and Mori, 1993). If T_f is a random variable denoting time to system failure (failure being defined generically), the hazard function is defined as,

$$h(t)dt = P(t < T_f \le t + dt | T_f > t) \tag{5.19}$$

The probability of failure in interval (t_1,t_2), given survival to t_1, can be obtained from integrating h(t);

$$\begin{aligned} P_f(t_1,t_2) &= P(t_1 < T_f \le t_2 | T_f > t_1) \\ &= 1 - \exp\left[-\int_{t_1}^{t_2} h(x)dx\right] \end{aligned} \tag{5.20}$$

Typical hazard functions for different failure processes are illustrated in Figure 5.13. If failures occur randomly in time, as would be the case if the earthquake process were statistically stationary and no structural deterioration occurred, h(t) is constant. When aging and an aggressive environment causes structural deterioration, h(t) increases in time. Hazard functions associated with common modes of aggressive environmental or chemical attack of reinforced concrete have been determined (Mori and Ellingwood, 1993).

Generally, one might envision a failure process consisting of a combination of the two effects;

$$h(t) = \lambda + h_a(t) \tag{5.21}$$

in which λ = constant (random) failure rate and $h_a(t)$ represents the additional contribution due to aging. Substituting Eqn 5.21 into 5.20, we obtain,

$$P_f(t_1,t_2) = 1 - \exp[-\lambda(t_2-t_1)] \exp[-\int_{t_1}^{t_2} h_a(x) lx] \tag{5.22a}$$

$$\approx \lambda(t_2-t_1) + \int_{t_1}^{t_2} h_a(x) dx \tag{5.22b}$$

for the usual case when the failure rates are very small (less than 0.01/yr). Eqn 5.23 breaks the overall failure probability into random and aging components.

For small probabilities and failure rates, the probability of core damage, expressed as a function of time, can be approximated by Eqn 5.21, in which the time increment is taken as one year. The probability of core damage during a 40-year service life is approximately,

$$P(CD,40) = 40\lambda + \sum_i \Delta\ P(CD)_i \tag{5.23}$$

in which λ = P(CD) computed under the assumption that no structural aging has occurred (Section 5.2), $\Delta P(CD)_i$ = increment in core damage probability due to aging in year i, computed using the revised fragility parameters m(i) and β(i) for any structural component that has deteriorated due to aging (Section 5.3), and the integration in Eqn 5.22b has been replaced by a summation.

Using the results of the analyses performed in sections 5.2 and 5.3, aging of the auxiliary building shear wall leads to an increase in the (integrated) probability of core damage during a 40-year service life of less than 7 percent. Simultaneous aging of the shear wall and pump enclosure roof increases the 40-year probability of core damage by 47 percent. The latter scenario is based on very conservative assumptions regarding the combined impact of aging on median fragilities m_8 and m_{14}. Accordingly, it seems unlikely that even severe structural aging of the auxiliary building shear wall or pump enclosure roof will have a significant impact on the probability of core damage from an earthquake at Zion Unit 1.

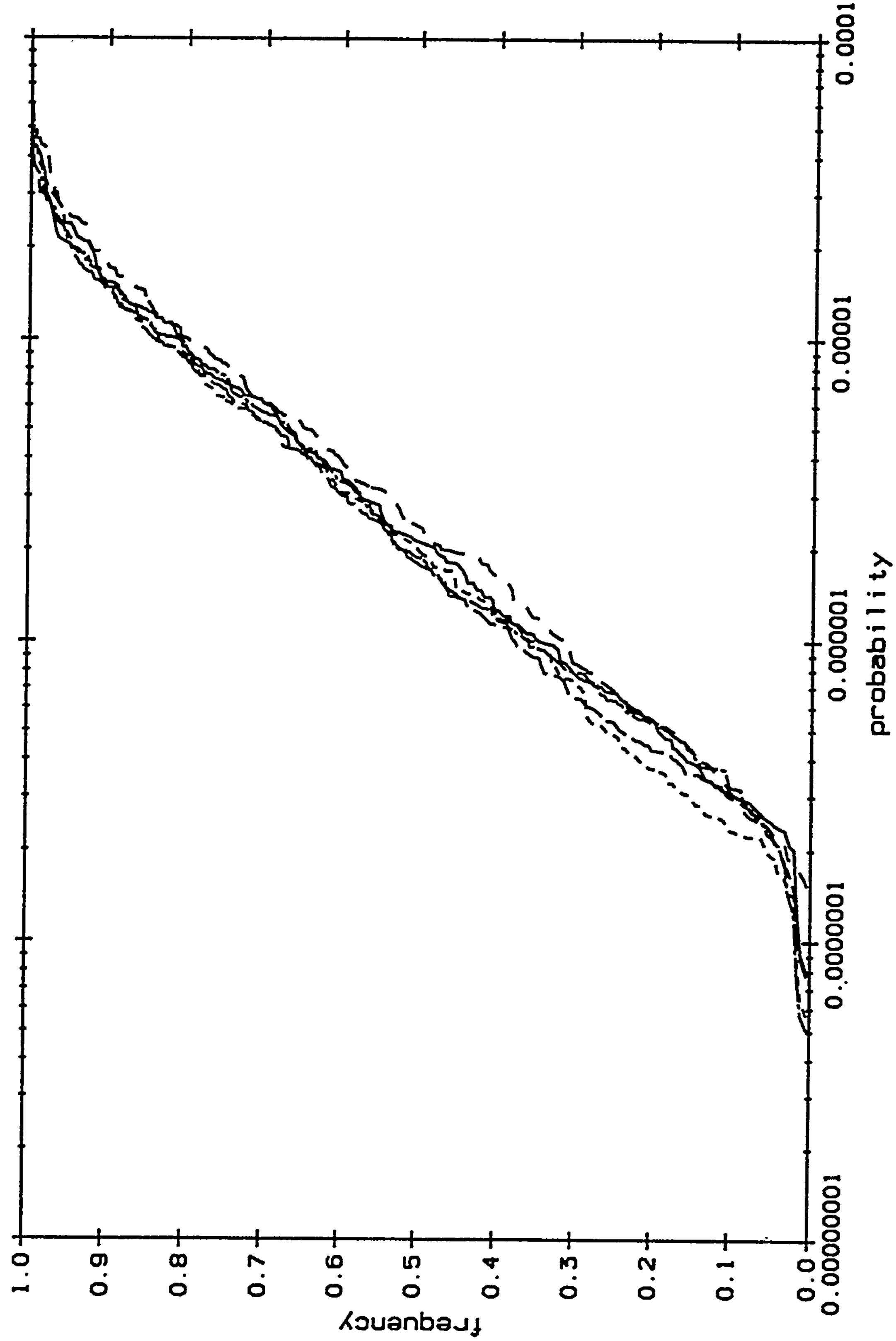
5.3.5 Other Considerations

An earthquake affects many systems in a plant simultaneously. Moreover, structures are constructed using common design and construction practices. Thus, structural actions (forces and deformations) due to the earthquake are correlated at different points in a plant. This introduces dependence in building response and structural behavior at different locations in a structure and may lead to so-called "common-cause" failures. Previous seismic PRA studies (Ravindra, et al, 1984; Ellingwood, 1994a) have concluded that such stochastic dependence has little impact on estimates of plant risk. The uncertainty in the seismic hazard (cf Figure 3.1) is so high in comparison to uncertainties due to fragility modeling that the influence of any stochastic dependence in component failures is diluted. In the case of fragility/margins analysis, where the structural behavior is uncoupled from the hazard, such effects become more apparent but still have a minor impact in comparison with other risk contributors.

Quality assurance/quality control in design and construction of nuclear power plant structures is stringent. While human errors in design and construction are known to have occurred, research has indicated that the role of design/construction error in plant risk has a minimal impact on estimates of mean (or median) frequency of core damage (Ravindra and Ellingwood, 1989).

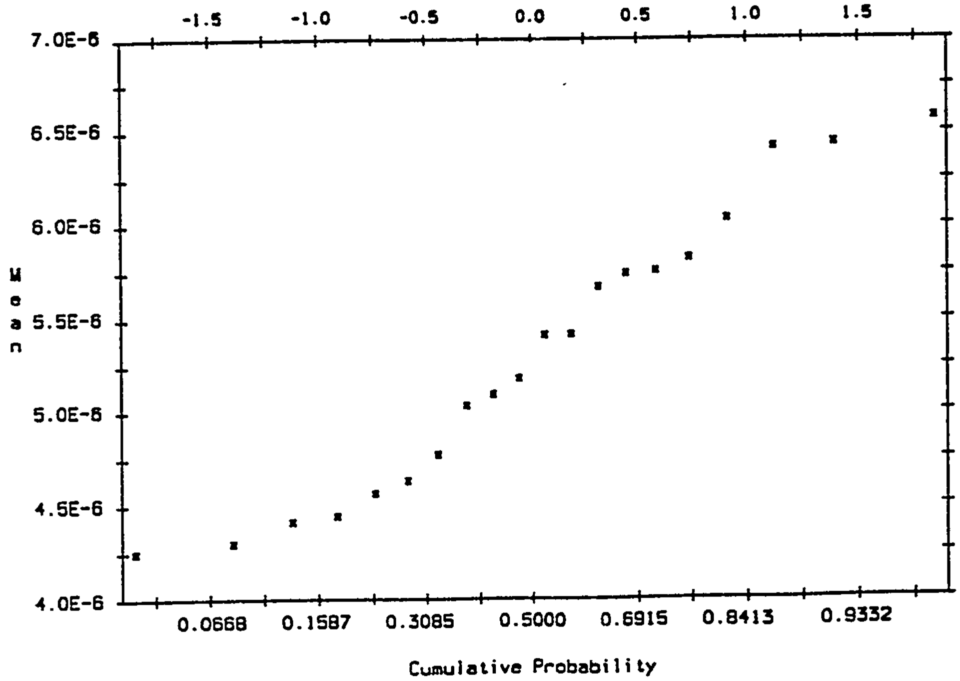
5.4 Summary

The analyses above indicate that substantial damage to structural components due to aging lead to less than an order-of-magnitude increase core damage probability. On the other hand, the impact on the HCLPF can be higher. Accordingly, if a margin analysis is used to assess suitability for continued service, the impact of structural aging on fragility models becomes more important and more care in applying these models is required. It should be straightforward to determine whether structurally significant damage has occurred during a walk-through inspection in support of a decision regarding continued service, provided that the component to be inspected is accessible. Sensitivity analyses of the type presented above can help to identify those structural components warranting particular attention.

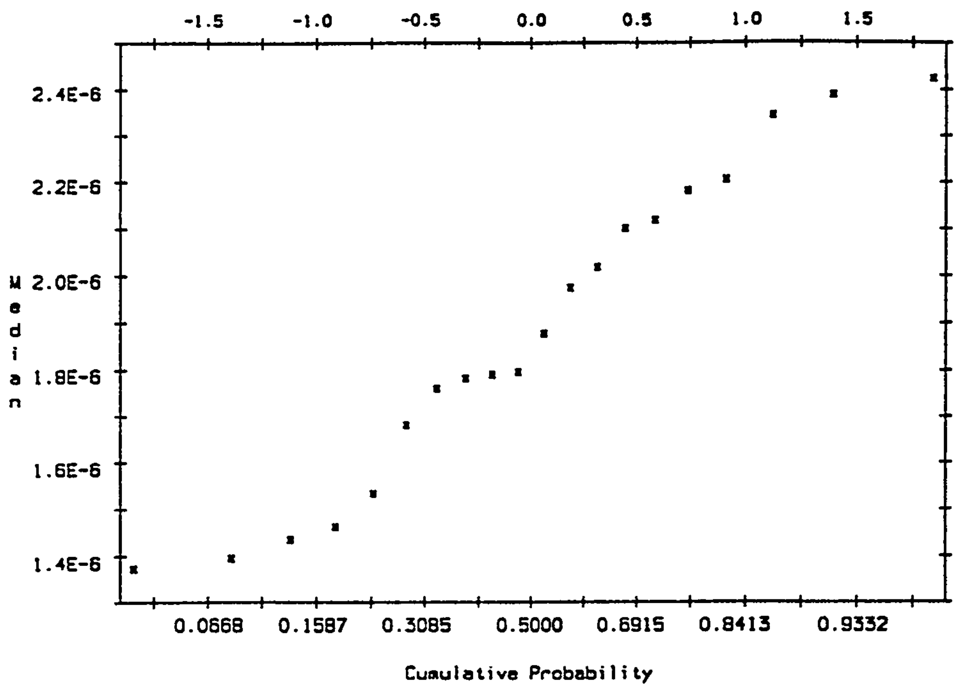


5.1 Cumulative frequency distribution for P(CD) - random seismic hazard

5.2a Mean of P(CD)

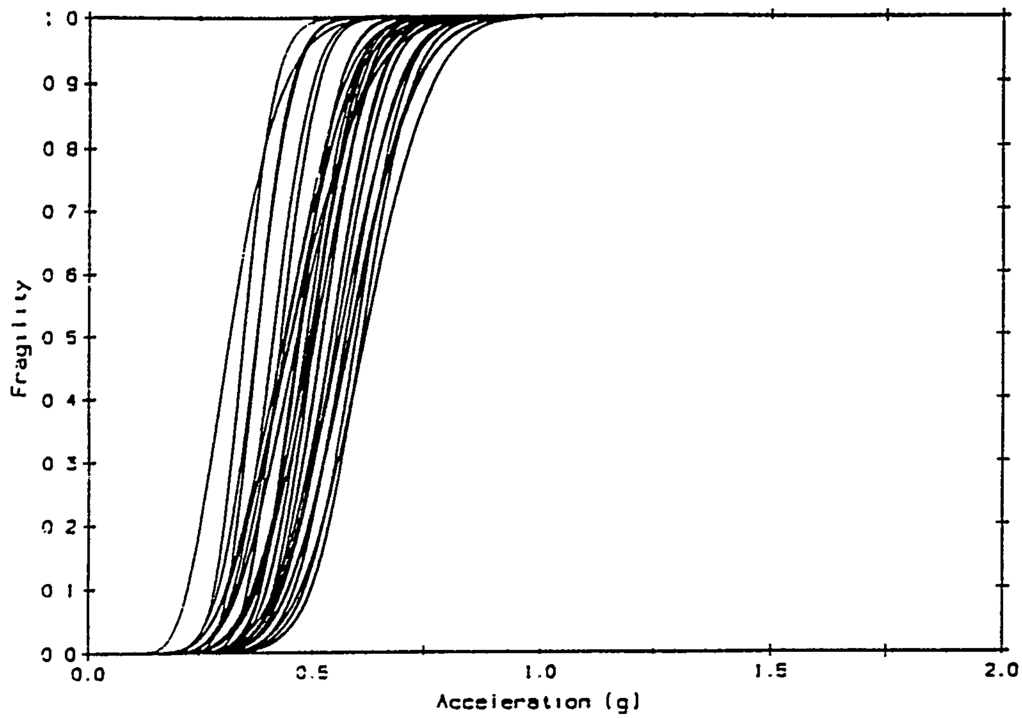


5.2b Median of P(CD)

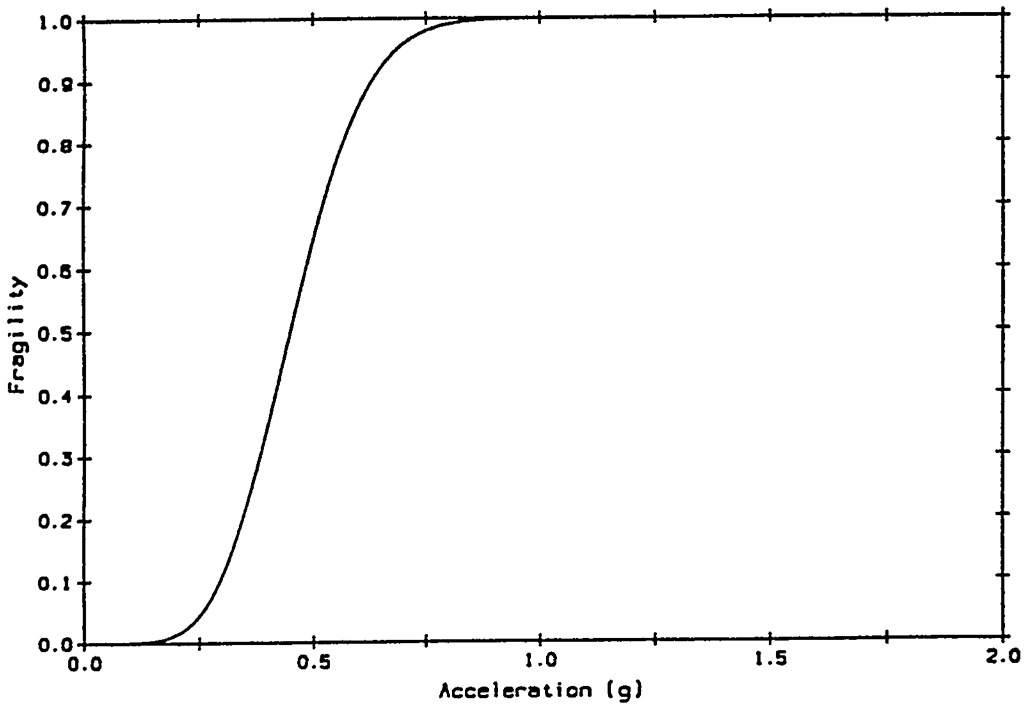


5.2 Sampling distributions for mean and median of P(CD)

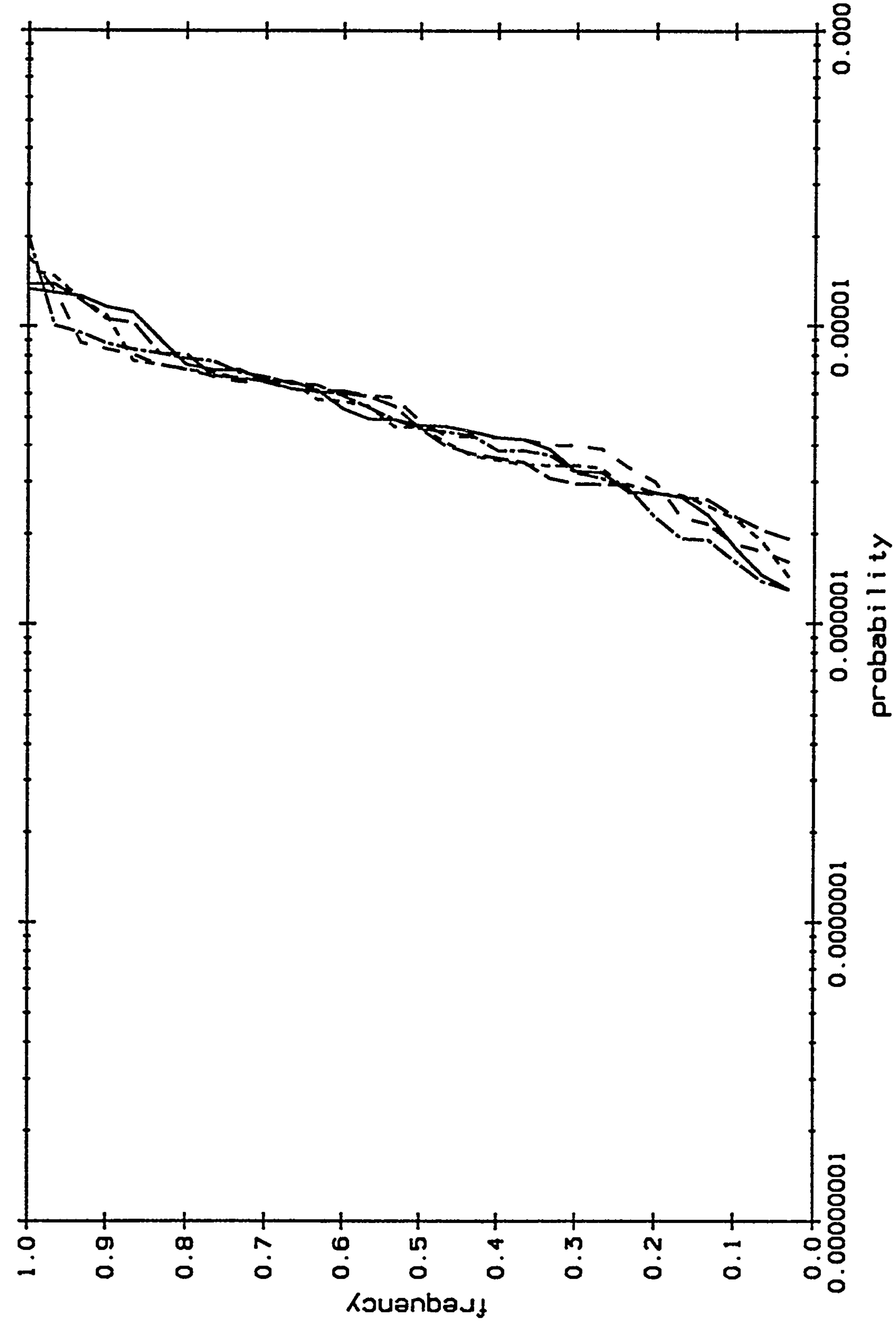
5.3a Uncertainty in fragility



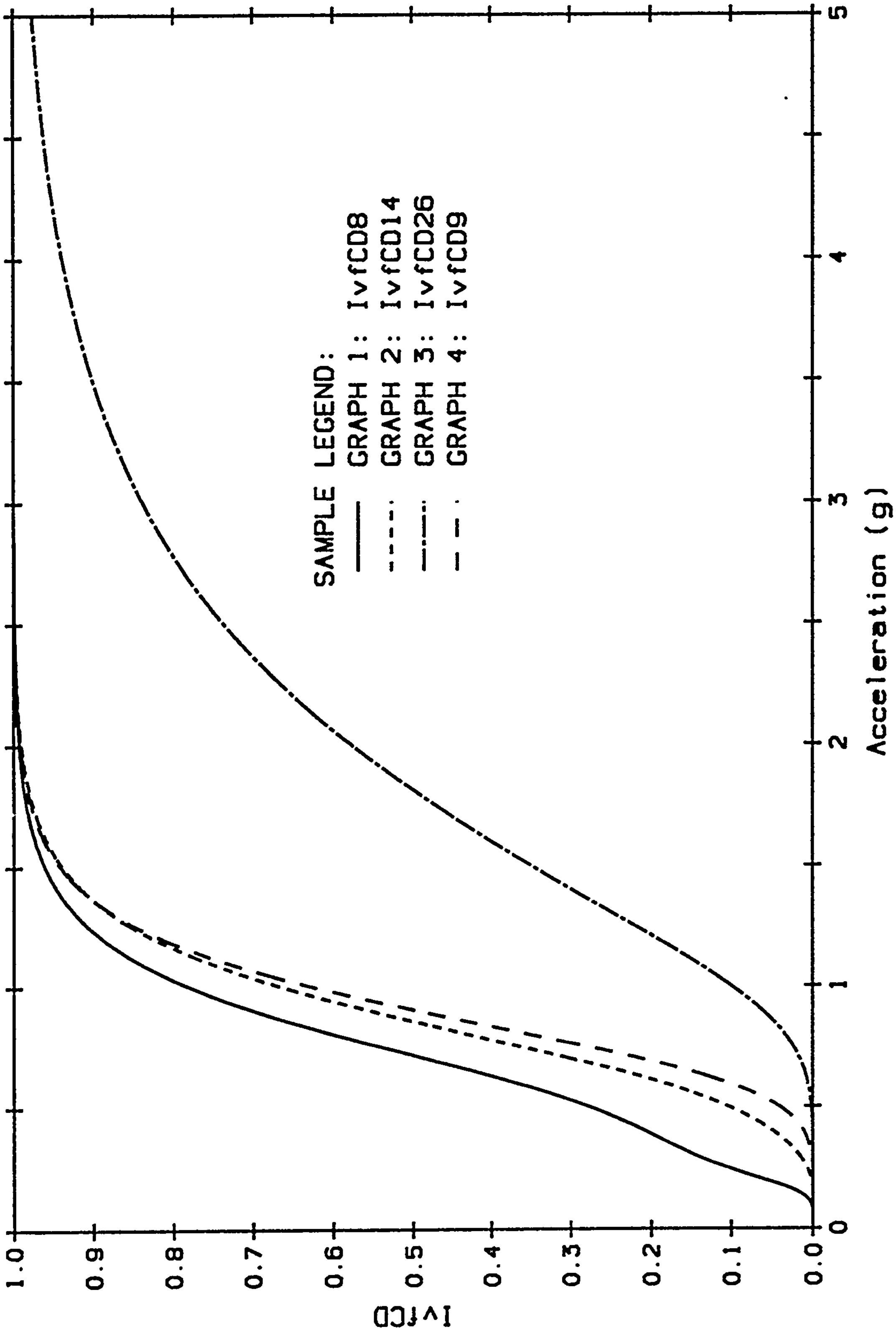
5.3b Plant level fragility from mean component fragilities



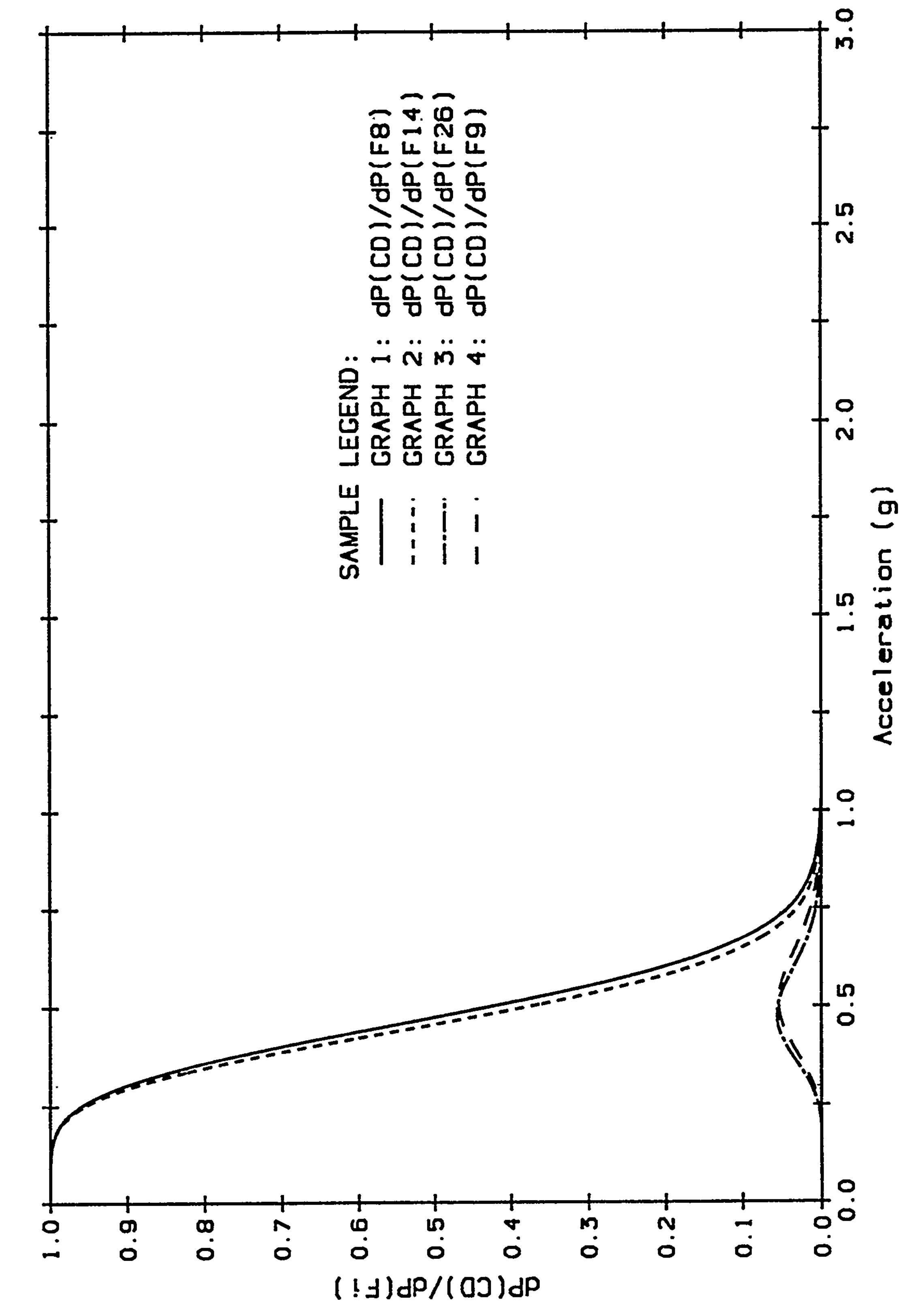
5.3 Plant level fragility for sequence CD



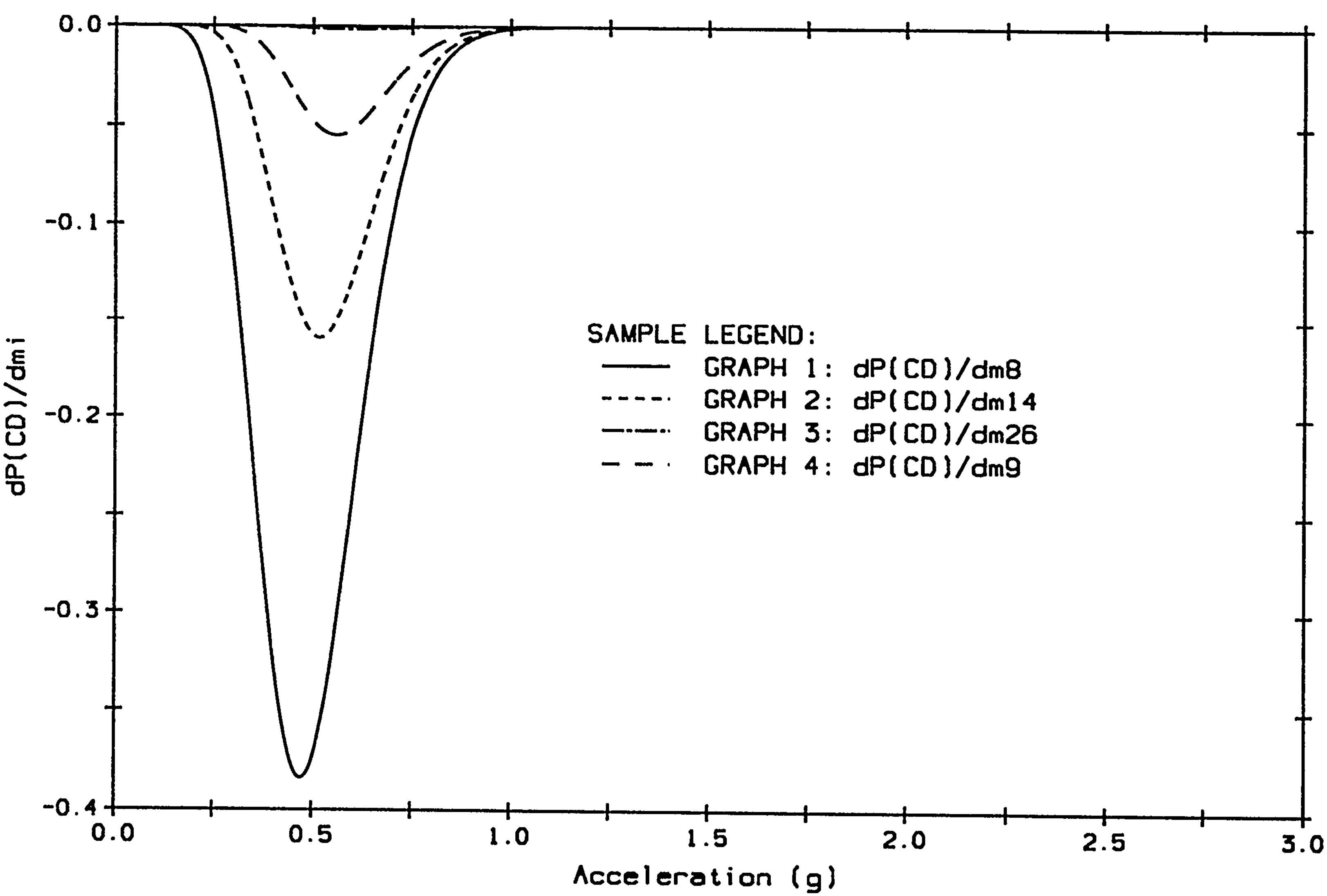
5.4 Cumulative frequency distribution for P(CD) -Mean seismic hazard



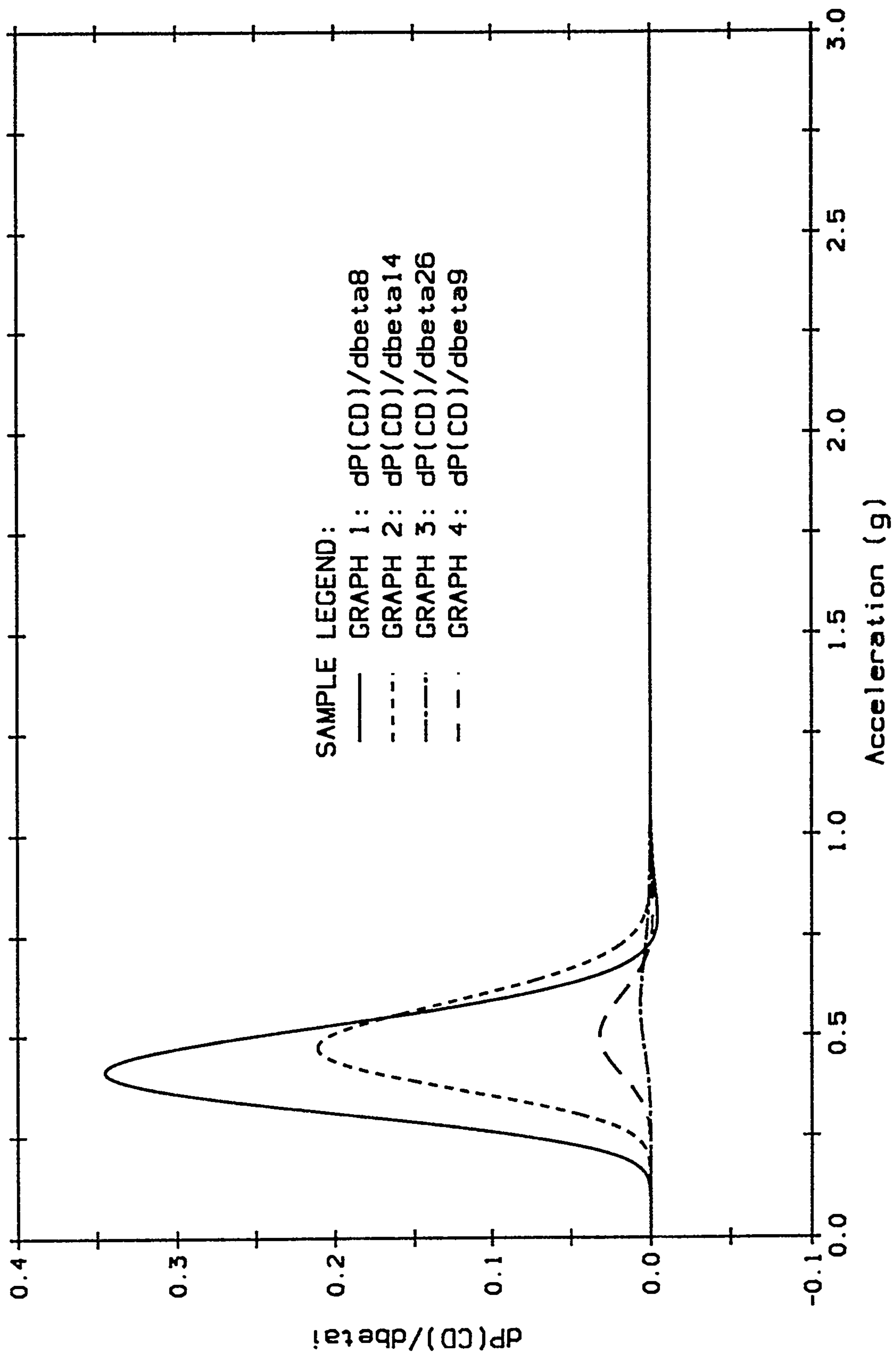
5.5 Vesely-Fussell measure for CD vs x



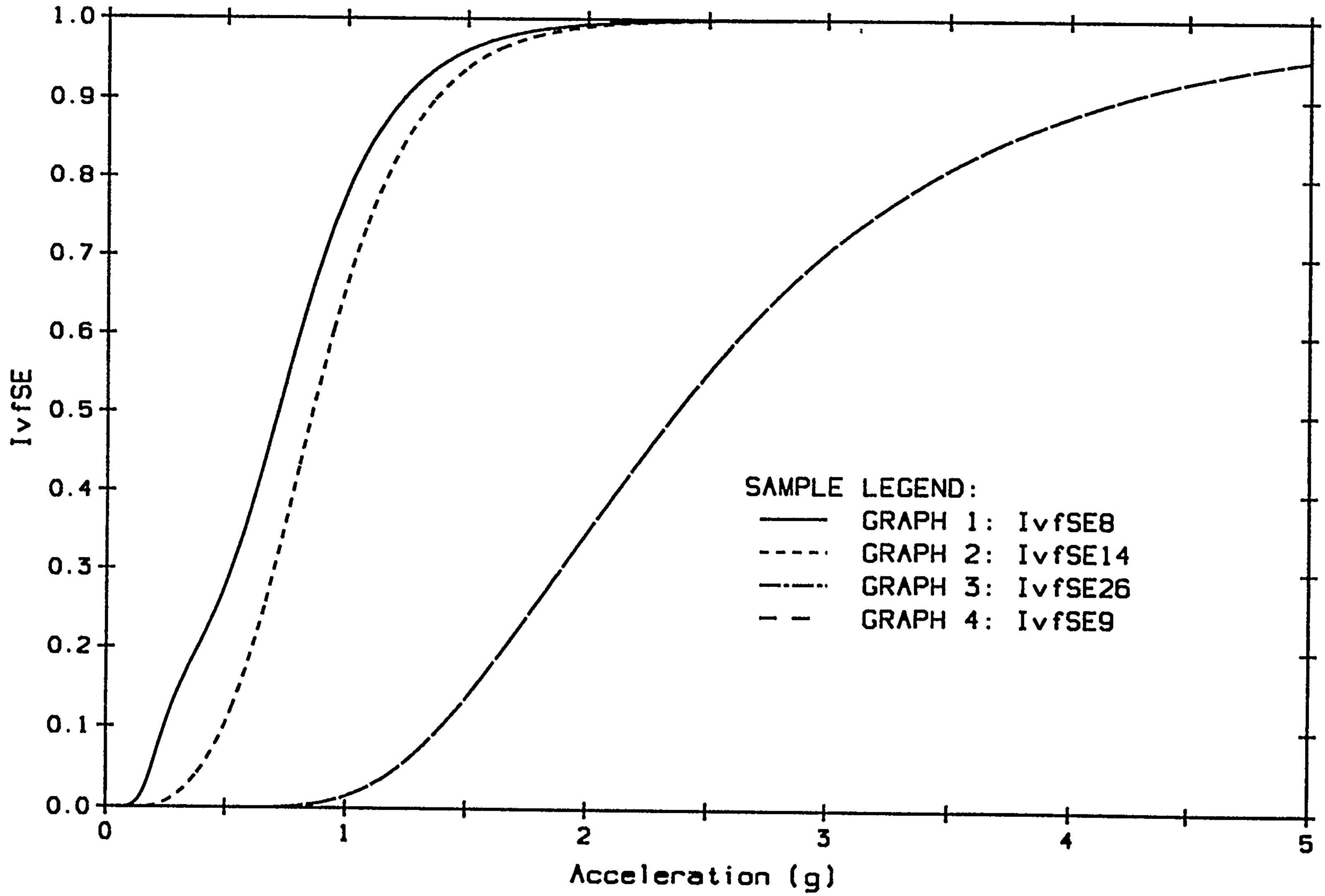
5.6 $\partial P(CD)/\partial P(i)$ vs x



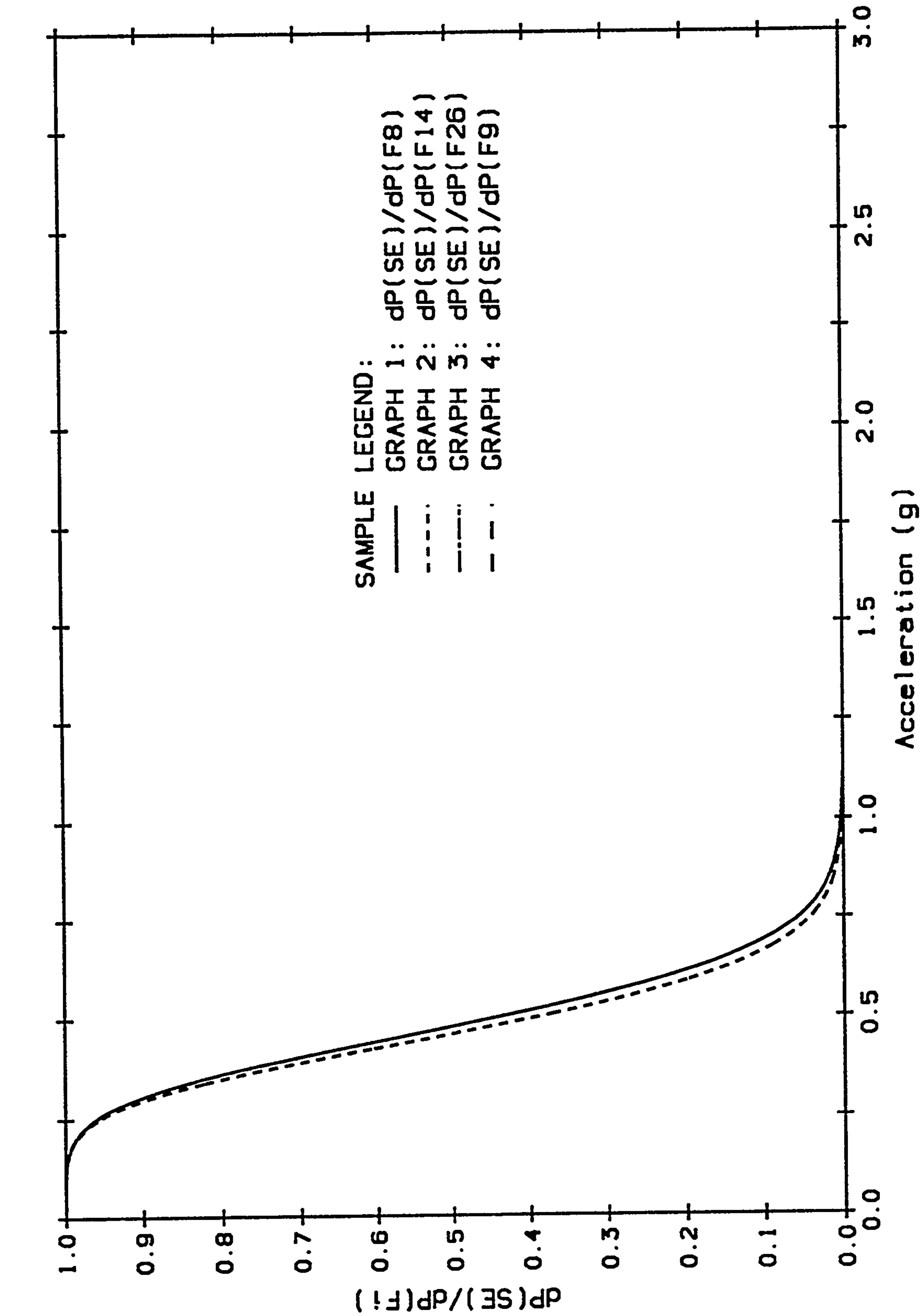
5.7 $\partial P(CD)/\partial m(i)$ vs x



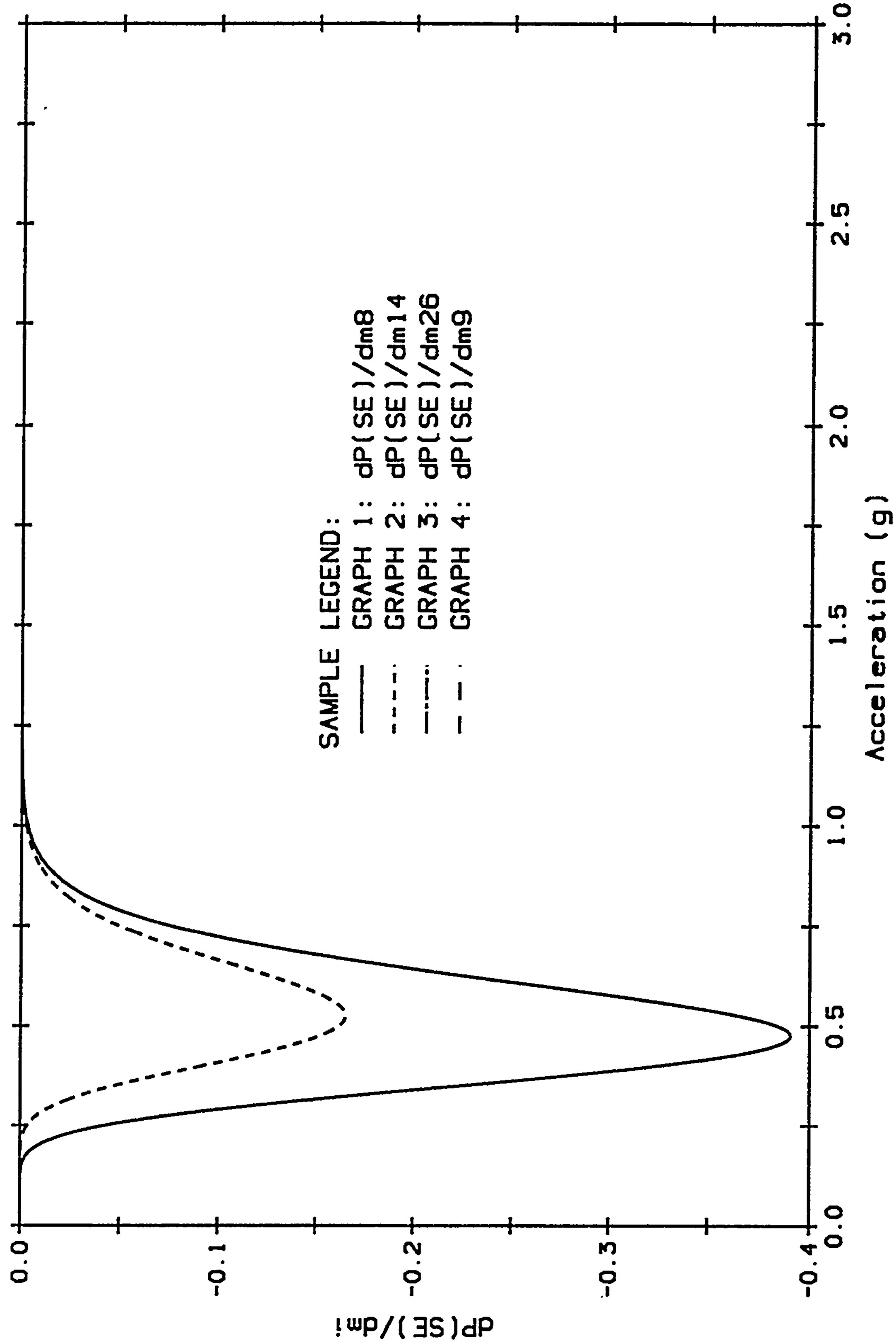
5.8 $\partial P(CD)/\partial\beta(i)$ vs x



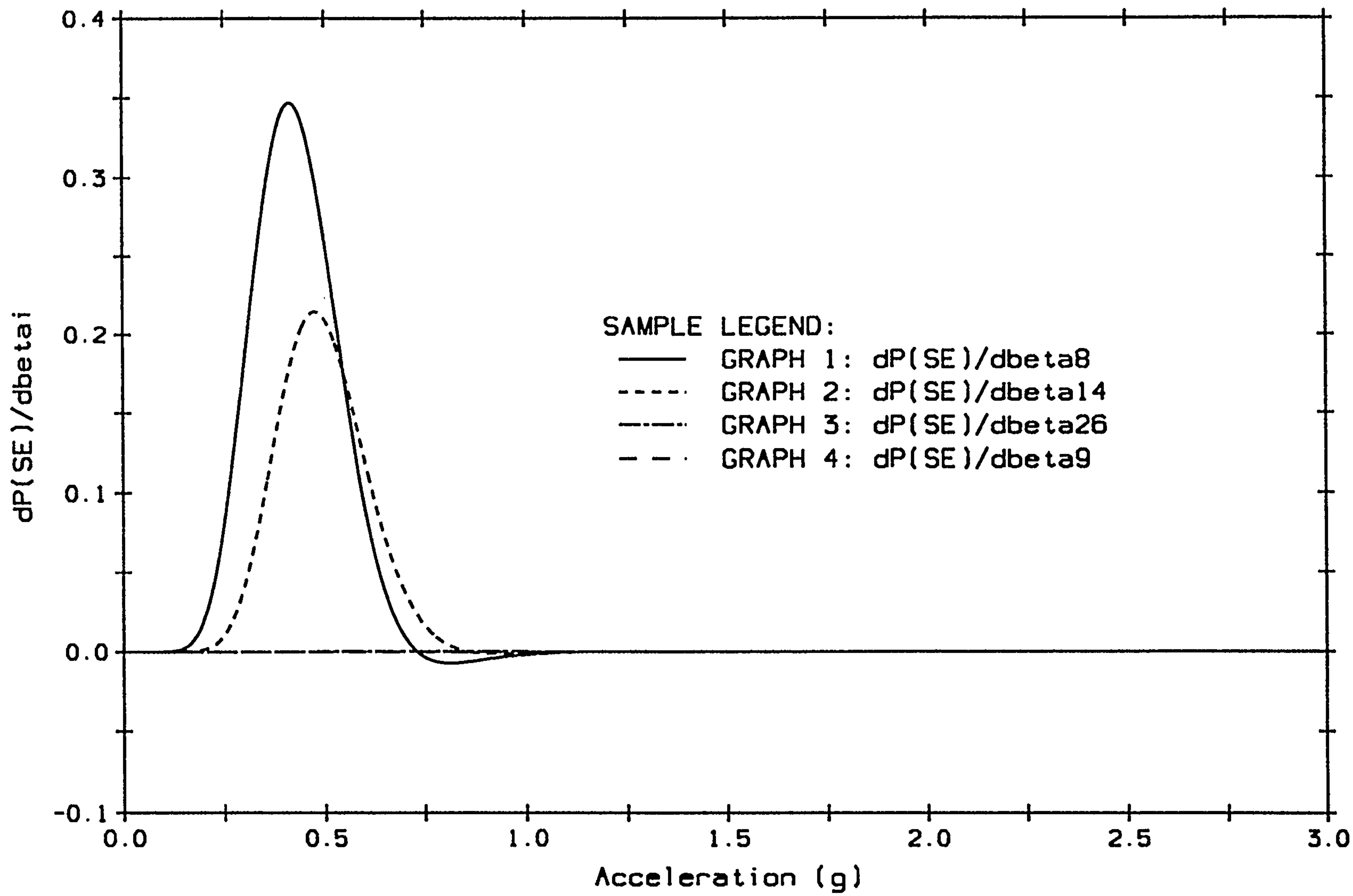
5.9 Vesely-Fussell measure for SE vs x



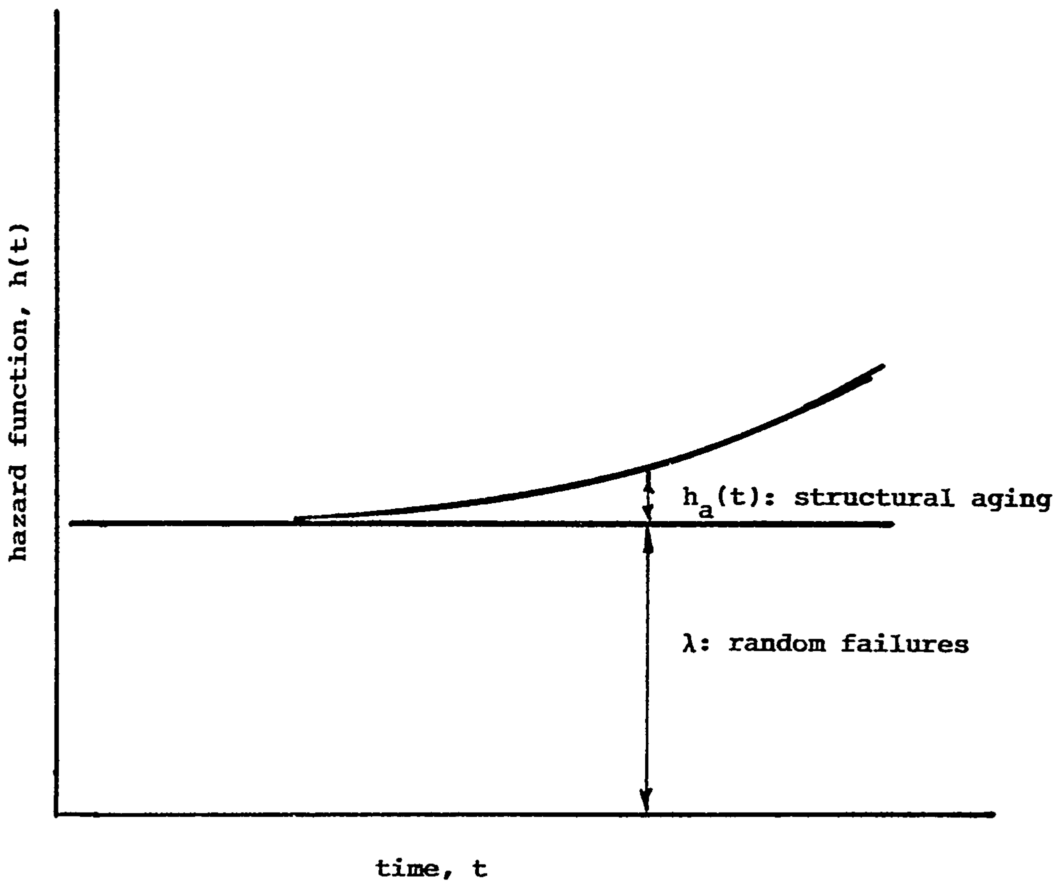
5.10 $\partial P(SE)/\partial P(i)$ vs x



5.11 $\partial P(SE)/\partial m(i)$ vs x



5.12 $\partial P(SE)/\partial \beta(i)$ vs x



5.13 Hazard functions

6. Risk Implications of Aging for Service Life Extension

Evaluation of the impact on plant risk of structural deterioration due to aging must consider a number of factors. First and foremost is the requirement to ensure that structural aging has not affected the ability of the plant to respond to challenges at or beyond the design envelope in such a way that public safety and health are compromised. Beyond this fundamental requirement, however, there are substantial economic and social issues. The utility must be able to meet the regulatory objectives for safe plant operation practically and economically; otherwise, the only alternative is to decommission the plant. Nuclear plants are responsible for over 20% of the power currently generated in the United States, and for some utilities the percentage is much higher. The costs and other impacts of decommissioning and power replacement to meet continuing consumer demand may cause economic dislocations in parts of the United States if this alternative is widely adopted.

In view of the above, it seems essential to focus any in-service condition assessments or periodic maintenance actions for structures that may be required to continue service on a selected subset of critical structural components that have the potential to impact plant safety. Other structural components might be visually inspected during routine operation or maintenance, but would require no detailed or invasive inspection/evaluations unless demonstrable problems became apparent. Such critical structural components might be identified in three steps.

First, the component screening procedure developed earlier in the Structural Aging Program should be used to rank structures with respect to their safety significance (Hookham, 1991). Such rankings are plant specific, of course. However, an illustrative application of this methodology to three LWR plants - (1) PWR Large Dry Metal Containment, (2) BWR Reinforced Concrete Mark II Containment, and (3) PWR Large Dry Prestressed Concrete Containment - revealed some common features for all plants. Structural components that ranked highly in all three included concrete internal structures, auxiliary building walls and floor slabs, reactor building foundation and shear walls, and control building foundation and shear walls. As components, foundations were generally most important, followed by slabs and walls (approximately equal in importance); beams and columns were of somewhat lesser importance. Most PRAs show that only a few components play a significant role in plant risk. Accordingly, a significant effort to identify those components for a particular plant and focus subsequent risk analysis on those should have substantial payoff, both in risk mitigation and economic terms. While both internal and external events should be considered in this evaluation, it is likely that structures will be found to play a significant role mainly in external events mitigation.

Second, the Individual Plant Examination for External Events (IPEEE) program currently nearing completion ultimately will result in a PRA for each operating nuclear plant in the United States (Flack, 1994). Plant logic Booleans such as those described in Section 2 should be available to model the impact of major external influences, including earthquake, wind, fire and flooding, on plant risk. As noted previously, it is in responding to challenges from such external influences that structural components and systems play a major role in mitigating core damage. The probability of containment failure due to a LOCA is more than an order of magnitude less than the probability of structural component failure from an earthquake. The plant logic models developed from the IPEEE should be overlaid on the structural component safety rankings to identify those components with a potential to contribute to risk.

Finally, a sensitivity analysis of the type conducted in Section 5.3 should be performed to identify those components that may be critical in terms of aging. Not all components identified in steps 1 and 2 may be easily accessible for inspection. One of the goals of this sensitivity study should be to establish whether aging of any inaccessible structural component or system is likely to have a significant impact on plant risk. Those that do not can be eliminated from further consideration; those that do present a dilemma with regard to in-service inspection and maintenance. While it is outside the scope of the present study to make recommendations as to what actions should be taken with regard to such structural components, the current methodology can minimize implementation of regulatory requirements that are indiscriminate and non-productive from a risk prevention point of view. For those structural components identified as being potentially risk-significant, regulatory positions and in-service inspection/maintenance policies should be developed using the time-dependent reliability analysis methodology developed previously in Task S4 (Mori and Ellingwood, 1993). It is important to use best estimates of strength - not code values of strength - in making these assessments.

The analyses of Zion Unit No. 1 in Section 5 indicate that substantial damage to structural components due to aging causes less than an order-of-magnitude increase in core damage probability. On the other hand, the impact on the HCLPF can be higher. Accordingly, if a margin analysis rather than a PSA is used to assess suitability for continued service, the impact of structural aging on fragility models will appear to be more important, and more care in applying these models is required. It should be easy to determine whether any damage has occurred during an inspection in support of a decision to continue service, provided that the component to be inspected is accessible.

The decreases in median structural fragility considered in Section 5.3 are conservative estimates of what might be expected when the degradation mechanisms considered act over a period of 30 - 40 years. Such conservative but plausible degradations in structural component capacity due to aging appear to increase either core damage or plant damage state probability by less than a factor of 2 and to decrease the HCLPF by 20% or less. These results, of course, are specific for Zion Unit 1; however, other seismic PRAs have indicated similar general levels of sensitivity of core damage probability or HCLPF to comparable variations in fragility parameters (Ellingwood, 1994a). Accordingly, a guideline for in-service inspection and repair might be devised to focus ISI/M only on those structural components in which degradation due to aggressive attack might lead to more than a factor of two increase in P(CD) or a 20% decrease in HCLPF. Investing resources on ISI/M for other components would not be cost-effective in managing risk. The fundamental question of whether an increase in core damage probability of a factor of 2 or a decrease in HCLPF of 20% is significant from a risk management point of view requires further examination (e.g., Pate-Cornell, 1994).

There are only limited data on aging of structural materials and components, and little for the durations of interest in service life extension. Accordingly, such considerations should include a sensitivity and uncertainty analysis to determine whether the conclusions are robust. Ideally, such conclusions and decisions therefrom can be based on results that are not sensitive to uncertainties in the aging process.

7. References

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10. SUPPLEMENTARY NOTES
H. Graves, NRC Project Manager

11. ABSTRACT *(200 words or less)*

The Structural Aging Program is addressing the potential for degradation of concrete structural components and systems in nuclear power plants over time due to aging and aggressive environmental stressors. Structures are passive under normal operating conditions but play a key role in mitigating design-basis events, particularly those arising from external challenges such as earthquakes, extreme winds, fire, and floods. Structures are plant-specific and unique, often are difficult to inspect, and are virtually impossible to replace. The importance of structural failures in accident mitigation is amplified because such failures may lead to common-cause failures of other components. Structural condition assessment and service life prediction must focus on a few critical components and systems within the plant. Components and systems that are dominant contributors to risk and that require particular attention can be identified through the mathematical formalism of a probabilistic risk assessment, or PRA. To illustrate, the role of structural degradation due to aging on plant risk is examined through the framework of a Level 1 seismic PRA of a nuclear power plant. Plausible mechanisms of structural degradation are found to increase the core damage probability by approximately a factor of two.

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