Margins of Quality in Engineered Systems

Performance and reliability based engineering for design, requalification, construction, operation, maintenance, and decommissioning

by

Dr. Robert Bea

Engineering & Project Management Program

Department of Civil & Environmental Engineering

University of California at Berkeley

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Ocean Engineering Services

60 Shuey Drive

Moraga, California 94556

(925) 631-1587

bea@ce.berkeley.edu

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Chapter 1

Introduction

This development is about ways that engineers can address some categories of uncertainties in their work and works. The 50 years of background and experience on which this development is based has identified four general categories of uncertainties that engineers should understand and address in their work: 1) natural - inherent variability uncertainties, 2) engineering analytical model - parameter uncertainties (professional), 3) human and organizational performance uncertainties, and 4) information - knowledge understanding uncertainties.

In this development, the first two categories of uncertainties are addressed: natural and professional. The approach that is used to address these two categories of uncertainties are 'Margins of Quality' that are incorporated into engineered systems. The bases for development of these Margins of Quality are Reliability Assessment and Management (RAM) methods. These margins of quality are intended to provide defenses against the first two categories of uncertainties. Traditionally, these margins of quality often have been termed 'factors of safety'; and often joked about by engineers as 'factors of ignorance'. But, this is really not a joke. Engineers can not be certain about all of the 'demands' their systems must confront and sustain nor can they be certain about the 'capacities' that their systems will have to confront and sustain the demands that the systems can experience during their life cycles.

What about the other two categories of uncertainties? What should engineers do to address these uncertainties? The sequel to this development addresses these uncertainties: Human and Organizational Factors in Quality and Reliability of Engineered Systems. These two categories of uncertainties require different approaches and strategies. However, there are things that engineers can do to help address these categories of uncertainties as they configure and proportion their systems and determine their margins of quality. These things will be incorporated under the term engineering for 'robustness' - incorporating sufficient defect and damage tolerance into the engineered system. This course will further develop these important additional elements.

In this development, the terms 'engineered systems' will be defined as the professionally engineered composition of seven interactive and interdependent components: 1) structures - physical support elements, 2) hardware - physical operating elements (e.g. electrical, hydraulic, mechanical), 3) procedures (formal, informal, software), 4) human 'operators' or operating teams (perform the activities), 5) organizations (collections of operating teams, determine resources, means and methods), 6) environments (internal, external, social), and 7) interfaces between the foregoing. In this course, the primary focus is on the first two components; however, the interfaces between these two components and the other components can not be ignored because they can have important influences on the margins of safety that are achieved as a result of engineering.

One clear example of an important interface in an engineered system is Quality Assurance and Quality Control (QA/QC). Margins of safety are founded on specified QA/QC measures and procedures that are not only defined and conducted during the design phase, but also must be maintained during the other phases in the life-cycle of the engineered system. QA / QC are proactive and interactive approaches to help assure that what was intended is realized throughout the life-cycle of an engineered system.

The recent history of engineered systems, both recent and ancient, have given engineers some valuable insights into the factors that are operative in the failures of these systems during their life-cycles. The background work on which this course is based has identified two fundamental categories of factors that have been involved: 1) Intrinsic (belonging to the essential nature), and 2) Extrinsic (not belonging to the essential nature). The first category incorporates the first two categories of uncertainties previously cited. The second category incorporates the second two categories of uncertainties.

The experience indicates that about 20% of the failures are fundamentally involved with the first category - Intrinsic factors. The remaining 80% of the failures are fundamentally involved with the second category - Extrinsic factors. This experience is actually a tribute to what has been accomplished by engineering; the margins of safety traditionally used by engineers have been effective at helping manage two important categories of uncertainties. An objective of this course is to help continue this trend - to help engineers understand how to use margins of safety to address two important categories of uncertainties.

There are other important observations that can be made from the study of failures of engineered systems. Most of the failures (about 80%) occur during operations and maintenance of the engineered systems. This is to be expected because it is during the long-term exposure to hazards during the operations and maintenance activities that 'weak links' can develop in the engineered system. The other 20% of the failures develop during the design and construction - manufacture phases. The operations - maintenance phase failures are termed 'noisy' failures - they generally have very noticeable and often publicized effects. The design - construction phase failures are termed 'quiet' failures - they generally are not very noticeable and often show up in court or legal proceedings (there are exceptions - e.g. the design - construction of the San Francisco Bay Bridge). Margins of Quality can be used to address both noisy and quiet challenges to successful engineering work and works.

There is one very important observation that has developed from the study of engineered systems: the majority of the failures that develop during operations and maintenance can be traced directly to flawed engineering design. Engineered systems are designed that can not be built as intended; work-arounds are developed during construction and construction can introduce some additional flaws. The result is passed to the field in the form of a 'buggy' system that often has not properly addressed the operating and maintenance requirements of the system during its life-cycle. The system is designed for what people could do, not what they will do. Little or no detailed consideration is given to the inspection, maintenance, and repair activities that will have to be conducted during the operating phase if the desired quality and reliability of the system are to be realized.

Experience with failure of all types of engineered systems indicates that the single largest cause of failures that develop during the design phase are due to inappropriate or incorrect demands used to design the system. Errors in the characterization and definition of demands accounts for about 50 % of these system failures. Engineers generally understand and know the capacities of the elements that comprise the structure and hardware in their systems than they understand the demands. This is sensible because the system element capacities generally have much smaller uncertainties than those associated with the demands. These errors involve demands that are developed during design, construction, operation, maintenance, and decommissioning (primary life-cycle phases). These errors involve both local demands that are developed on parts of the system and on global demands that represent the integrated effects of the local demands.

But, not all engineering demand ‘problems’ are due to errors. An example is the definition of loadings for reassessment and requalification of existing structures. There are few generally accepted design codes or guidelines for reassessing and requalifying existing structures. Should these structures be required to carry or sustain the same loadings as an equivalent ‘new’ structure? Perhaps not. A good example of such a problem is the reassessment and requalification of the San Francisco Bay bridges. It is clear that engineers need to be better prepared to evaluate and characterize demands that will or can be imposed or induced on or in their systems during their lifetimes. This preparation is especially important for engineering reassessments of existing systems, for there are few generally accepted guidelines to determine the loadings or loading effects that are appropriate for reassessment and requalification of systems. Similar statements need to be made regarding the capacities of the system (structure, hardware) elements as they are affected by their exposure to the operating environments; corrosion and fatigue effects on the capacities of structure and hardware elements are examples.

This raises the questions concerning the use of 'design codes'. If we have design codes, then why do we need to consider the content of this course? This important question has several answers. First, if the design codes (generally specified as 'minimum') are 'proven' and 'acceptable' and are applied to the categories of systems for which they are intended, then their direct application is warranted. However, if they are not proven, have potentially unacceptable consequences, or the system lies outside or at the borders of applicability of the design code, then their direct application may not be warranted. It is for such systems that the content of this course becomes potentially useful to engineers. The author's reflection on his uses of engineering system design codes during the past 50 years (structures and hardware) is that they are a place to start, but seldom to stop - particularly for very important or 'new' types of systems. It is particularly important to recognize the limitations of design codes and to engineer accommodations to these limitations as is necessary to be able to develop engineered systems that will have desirable and acceptable 'quality' throughout their life-cycles.

This last paragraph has raised a term that has particular importance: 'quality'. In this course, that term will be defined as freedom from unanticipated defects. Quality is fitness for purpose. Quality is meeting the requirements of those that own, operate, design, construct, maintain, and regulated engineered systems. Quality includes four important requirements: 1) serviceability, 2) safety, 3) compatibility, and 4) durability. Serviceability is suitability for the proposed purposes, i.e. functionality. Serviceability is intended to guarantee the use of the system for the agreed purpose and under the agreed conditions of use. Safety is the freedom from excessive danger to human life, the environment, and property damage. Safety is the state of being free of undesirable and hazardous conditions. Compatibility assures that the system, does not have unnecessary or excessive negative impacts on the environment, business (private enterprise) or society (public enterprise). Compatibility is also the ability of the system to meet economic time, and aesthetic requirements. Durability assures that serviceability, safety, and compatibility are maintained during the life of the system. Durability is freedom from unanticipated maintenance problems and costs. Experience has adequately demonstrated that the most important requirement for an engineered system is durability.

With this definition of quality, the terms margins of quality take on more meaning. As engineers, we would like to define 'factors of safety' - really Margins of Quality - that provide safeguards against the uncertainties that we face in engineering systems - or at least the natural and professional uncertainties. We want these factors as safeguards to help us assure that we have desirable and acceptable quality and reliability in the engineered system. We want to be sure that the system will have desirable and acceptable serviceability, safety, compatibility, and durability during its life cycle.

The general framework that will be used to address margins of quality comes from the field of reliability engineering. Reliability methods will be used because they are able to address the key issues associated with the two previously defined categories of uncertainties. Reliability engineering provides some of the critical tools to address the uncertain demands and capacities that pervade all of the quality attributes. These issues regard the required or acceptable reliability of the system and the uncertainties that are associated with both the quality demands and the capacities of the system.

Applied to the definition of margins of quality in an engineered system, the Reliability Assessment and Management (RAM) process proceeds through the following steps:

Step #1 Based on an assessment of costs and benefits associated with a particular development and generic type of engineered system, and regulatory - legal requirements, define the target reliabilities for the quality elements of the system. These target reliabilities should address the four quality attributes of the system including serviceability, safety, durability, and compatibility. Allocate the target reliabilities to Intrinsic and Extrinsic uncertainties (hazards).

Step #2 Characterize the life-cycle demand 'conditions' that can affect and challenge the quality capacities of the system during its lifetime. These could be operating - serviceability conditions (e.g. weather, environmental). These could be construction - compatibility conditions (e.g. economic, schedule). These could be maintenance - durability conditions (e.g. corrosion, fatigue). These could be decommissioning - safety conditions (e.g. worker exposure to hazardous operations).

Step #3 Based on the system characteristics characterize the 'demands' associated with the life-cycle conditions. These demands and the associated conditions should address each of the four quality attributes of interest (serviceability, safety, durability, compatibility).

Step #4 Evaluate the variabilities, uncertainties, and 'biases' (differences between nominal and true values) associated with the demands. This evaluation must be consistent with the variabilities and uncertainties that were included in the decision process that determined the desirable and acceptable 'target' reliabilities for the system (Step #1) - Intrinsic uncertainties.

Step #5 For the generic system define how the elements will be engineered according to a proposed and specified processes (procedures, analyses, strategies used to determine the system configuration and proportions), how these elements will be configured into a system, how the system will be constructed, operated, maintained, and decommissioned (including Quality Assurance - QA, and Quality Control - QC processes).

Step #6 Evaluate the variabilities, uncertainties, and 'biases' (differences between nominal and true values) associated with the capacities of the system elements and the assemblies of elements for the specified conditions including design, construction, operations, maintenance, and decomissioning activities, and specified QA/QC activities). This evaluation must be consistent with the variabilities and uncertainties that were included in the decision process that determined the desirable and acceptable 'target' reliabilities for the system (Step #1).

Step #7 Based on the results from Steps #1, #4, and #6, and for a specified 'engineering format', determine the format factors - margins of quality.

It is important to note that several of these steps are highly interactive. For some systems, the loadings induced in the system are strongly dependent on the details of the design of the system. Thus, there is a potential coupling or interaction between Steps #3, #4, and #5. The assessment of variabilities and uncertainties in Steps #3 and #5 must be closely coordinated with the variabilities and uncertainties that are included in Step #1. The QA/QC processes that are to be used throughout the life-cycle of the system influence the characterizations of variabilities, uncertainties, and biases in the 'capacities' of the system elements and the system itself. This is particularly true for the proposed IMR (Inspection, Maintenance, Repair) programs that are to be implemented during the system's life cycle. Design criteria, QA/QC, and IMR programs are highly interactive and are very inter-related.

Chapter 2

Quality

2.1 Requirements

In this development, quality is defined as freedom from unanticipated defects in an engineered system. Quality is fitness for purpose (Matousek 1990). Quality is meeting the requirements of those who own, operate, design, construct, and regulate engineered systems. These requirements (attributes) include four primary attributes: 1) serviceability, 2) safety, 3) compatibility, and4) durability.

Serviceability is suitability for the proposed purposes, i.e. functionality. Serviceability is intended to guarantee the use of the engineered system for the agreed purpose and under the agreed conditions of use. Safety is the freedom from excessive danger to human life, the environment, and property damage. Safety is the state of being free of undesirable and hazardous situations.

Compatibility assures that the engineered system does not have unnecessary or excessive negative impacts on the environment and society during its life-cycle. Compatibility also is the ability of the engineered system to meet economic, time, and aesthetic requirements - largely business objectives - on time, on budget, happy customers.

Durability assures that serviceability, safety, and environmental compatibility are maintained during the intended life of the marine structure system. Durability is freedom from unanticipated maintenance problems and costs. Durability is perhaps the most important attribute of quality because if insufficient durability is incorporated into an engineered system, then there can be unanticipated and undesirable degradations in any or all of the other quality attributes.

2.2 Engineered Systems

In this work, Engineered Systems are defined as comprised of seven inter-related, interactive, and interdependent elements:

1) The operators and operating teams - the people that have their 'hands on the wheels' of the system activities
2) The organizations - the groups of operating teams and other groups that determine resources, means
3) The procedures (formal, informal, software) that the operators use to perform their activities,
4) The structures - support elements that are involved in these activities,
5) The hardware - electrical, hydraulic, mechanical elements that are intended to facilitate these activities,
6) The environments (external, internal, social) in which the operator activities are performed, and
7) The interfaces between the foregoing components.

These components are highly inter-related, interactive, and interdependent. Hence, unambiguous definitions with clear - crisp boundaries are not possible. A System is a set of two or more elements (components) whose behavior has an important effect on the behavior of the system, are interdependent, inter-related and interactive.

In this work, the term 'engineered system' is used: a system is a collection of elements (components) which interact with each other to function as a whole. These assemblies are brought into being through the engineering processes that include concept development, design, construction / manufacturing, operations, maintenance, and decommissioning. Management of the engineering processes is also included.

2.3 Analyses of Engineered Systems

Synthesis is the key to understanding engineered systems and includes identifying and describing a system of which the elements to be understood are a part, explaining and understanding the properties and functioning of the system, explaining and understanding the behaviors of the elements in terms of their roles in the functioning of the system. Engineering systems 'sense making' consists of placement of items into frameworks, comprehending, constructing meaning, anticipating, interacting in pursuit of understanding, patterning, and redressing surprise. We would like to better understand engineered systems to lessen their potentials for 'revenge' effects (unintended consequences) and to increase their potentials for important improvements in the quality of life.

Reductionism has been one of the hallmarks of the scientific method and the industrial revolution. If one wants to understand something, then we take it apart to see how it works. From an understanding of how the parts work, we try to extract an understanding of the whole. This is the essence of analysis: 1) take the thing to be understood apart (decomposition), 2) understand the performance of the parts, and 3) reassemble the understanding of the parts into an understanding of the whole.

But, this process has presented some important dilemmas particularly as the something to be understood has become very complex and interactive, more organic than mechanistic, and more probabilistic than deterministic.

From reductionism another process has begun to evolve; expansionism. Expansionism increases understanding by expanding the whole or system to be understood, not by reducing the system to its elements or parts. Understanding proceeds from the whole to its parts, not vice versa. This can be called synthesis, or putting things together. This is the equivalent of analysis in reductionism. Synthesis and analysis are complementary processes.

Expansionism reverses the sequence of analysis: 1) identify a whole (system) of which the element to be understood is a part, 2) explain the performance or properties of the whole, 3) explain the performance of the element in terms of its function/s within the system. In expansionism synthesis precedes analysis:

In synthetic thinking the thing to be explained is treated as a part of a containing system. In analytical thinking the thing to be explained is treated as a system to be taken apart. Analytical thinking reduces the focus; synthetic thinking expands the focus; expansionism. Expansionism is ‘systems thinking’.

2.4 Failures of Engineered Systems

The following is a summary of important observations that have resulted from a long-term study (1988 - 2005) of more than 600 'well documented' major failures and accidents involving civil and environmental engineered systems (Bea 2000a). Sufficient reliable documentation was available about these failures and accidents to understand the roles of the various components that comprised the systems during their life-cycle phases leading to the accident or failure; in many cases, personnel that had participated in the developments were interviewed to gain additional insights about how and why the accidents and failures had developed.

In this work, 'failure' has been defined as realizing undesirable and unanticipated compromises in the 'quality' of the engineered system during its life cycle. The work indicated that it was essential to identify how the system had been developed throughout its life-cycle to the point of failure including development of the concept/s, design, construction, operation, maintenance, and for some systems, decommissioning. The 'history' (heritage) of a system generally had much to do with development of failures.

Uncertainties that were major contributors to the accidents and failures were organized into four major categories: natural variability, modeling uncertainties, human and organizational uncertainties, and knowledge - understanding uncertainties. Often, it was not possible to develop unambiguous definitions and evaluations of these uncertainties. A fundamental purpose of this definition was to help direct efforts to better understand and manage the sources and effects of the different categories and sources of uncertainties.

The studies of the accidents clearly showed that the factors involved in causation of the major failures (direct cost more than 1988 U.S. $ 1 millions) most often (80 % or more) involved human, organizational and knowledge uncertainties. These were identified as Extrinsic Factors (not belonging to the essential nature). The remaining 20% of the causation factors involved natural and model related uncertainties. These were identified as Intrinsic Factors (belonging to the essential nature).

Of the extrinsic factors, about 80% of these developed and became evident during operations and maintenance activities; frequently, the maintenance activities interacted with the operations activities in an undesirable way. Of the failures that occurred during operations and maintenance, more than half of these failures could be traced back to seriously flawed engineering design; structures and their foundations may have been designed according to 'accepted standards' and yet were seriously flawed due to limitations and imperfections that were embedded in the standards and/or how they were used. Frequently, structures and foundations were designed that could not be built, operated, and maintained as originally intended. Changes (work-arounds) must be made during the construction process to allow the construction to proceed; flaws can be introduced by these changes or flaws can be introduced by the construction process itself. After the structure is placed in operation, modifications are made in an attempt to make the structure workable or to facilitate the operations, and in the process additional flaws can be introduced. Thus, during operations and maintenance phases, operations personnel are faced with a seriously deficient or a defective structure that can not be operated and maintained as intended.

Of the 20% of failures that did not occur during operations and maintenance of the structures, the percentages of failures developing during the design and construction phases are about equal. There are a large number of ‘quiet’ failures that develop during these phases that represent project failures and frequently end up in legal - court proceedings. The approaches described here have been used to help address these types of failures (compatibility attribute) during design and construction phases.

The failure development process was organized into three categories of events or stages: 1) initiating, 2) contributing, and 3) propagating. The dominant initiating events were developed by ‘operators’ (e.g. design engineers, construction, maintenance personnel) performing erroneous acts of commission or interfacing with the system components that have ‘embedded pathogens’ that are activated by such acts of commission (about 80%); what is carried out has unanticipated and undesirable outcomes. The other initiating events are acts or developments involving omissions (something important left out, often intentional short-cuts and violations). Communications breakdowns (withheld, incomplete, untrue, not timely) were a dominant category of the initiating events.

The dominant contributing events were organizational malfunctions (about 80%); these contributors acted directly to encourage or trigger the initiating events. Communication malfunctions, interface failures (organization to operations), 'culture' malfunctions (excessive cost cutting, down-sizing, outsourcing, and production pressures), unrealistic planning and preparations, and violations (intentional departures from acceptable practices) were dominant categories of these organizational malfunctions.

The dominant propagating events were found to be organizational malfunctions (about 80%); these propagators were responsible for allowing the initiating events to unfold into a failure or accident. With some important additions, the dominant types of malfunctions were found to be the same as for the contributing events. The important additions concerned inappropriate selection and training of operating personnel, failures in quality assurance and quality control (QA / QC), 'brittle structures and hardware' (damage and defect intolerant), and ineffective planning and preparations.

Most failures involve never to be exactly repeated sequences of events and multiple breakdowns or malfunctions in the components that comprise a system. Failures result from breaching multiple defenses that are put in place to prevent the failures. These events are frequently dubbed ‘incredible’ or ‘impossible.’ After many of these failures, it is observed that if only one of the ‘barriers’ had not been breached, then the accident or failure would not have occurred. Experience has adequately shown that it is extremely difficult, if not impossible to accurately recreate the time sequence of the events that actually took place during the period leading to the failure. Unknowable complexities generally pervade this process because detailed information on the failure development is not available, is withheld, or distorted by memory. Hindsight and confirmational bias are common as are distorted recollections. Stories told from a variety of viewpoints involved in the development of a failure seem to be the best way currently available to capture the richness of the factors, elements, and processes that unfold in the development of a failure.

Procedure and software (computer) related malfunctions frequently were found to be a primary player in failure causation. The procedures were found to be incorrect (faulty), inaccurate (untrue), incomplete (lacking important parts), excessively complex (unnecessary intricacy), poorly organized (dysfunctional structure), and poorly documented (ineffective information transmission. These malfunctions often were embedded in engineering design guidelines and computer programs, construction specifications, and operations manuals. They were also embedded in contracts (formal and informal) and subcontracts. They were embedded in how people were taught to do things. With the advent of computers and their integration into many aspects of the design, construction, and operation of engineered systems, software errors are of particular concern because the "computer is the ultimate fool" and it is easy to become "trapped in the net".

Software errors in which incorrect and inaccurate algorithms were coded into computer programs have been at the root cause of several recent failures of engineered system ('computer aided failures'). Guidelines have been developed to address the quality of computer software for the performance of engineering analyses. Extensive software testing is required to assure that the software performs as it should and that the documentation is sufficient. Of particular importance is the provision of independent checking procedures that can be used to validate the results from analyses. High quality procedures need to be verifiable based on first principles, results from testing, and field experience.

Given the rapid pace at which significant industrial and technical developments have been taking place, there has been a tendency to make design guidelines, construction specifications, and operating manuals more and more complex. Such a tendency can be seen in many current guidelines used for design of engineered systems. In many cases, poor organization and documentation of software and procedures has exacerbated the tendencies for humans to make errors. Simplicity, clarity, completeness, accuracy, and good organization are desirable attributes in procedures developed for the design, construction, maintenance, and operation of engineered systems.

Environmental influences can have important affects on the quality and reliability of engineered systems. Environmental influences include: a) External (e.g. wind, temperature, rain, fog, time of day), b) Internal (lighting, ventilation, noise, motions), and c) Sociological and cultural factors (e.g. values, beliefs, morays). These environmental influences can have extremely important effects on human, operating team and organizational malfunctions, the structures and hardware, and on the primary mediums that engineers must deal with - engineering materials.

One of the very sobering observations concerning many accidents and failures is that their occurrence is directly related to knowledge (information) access and development challenges. During this work, these challenges were organized into two general categories: unknown knowables, and unknown unknowables. The first category represents information access and understanding challenges. The information exists but is either ignored, not used, not accessed, or improperly used; other investigators have identified this category as 'predictable surprises.'

The second category represents limitations in knowability or knowledge. There are significant limitations in our abilities to project system developments or characteristics very far in space or time. Our abilities to know all of the things that are potentially important to the systems that we engineer is limited. Often, there are major limitations in knowledge concerning new or innovative systems and the environments in which these systems will be developed and exist. There is ample history of accidents and failures due to both of these categories of challenges to knowledge. They appear to be most important during the early phases of constructing and operating engineered systems; 'burn-in' failures. Things develop that one did not know or could not know in advance of the activities. They also appear to be most important during the late life-cycle phases; 'wear-out' failures. In this case, the quality characteristics of the system have degraded due to the inevitable effects of time and operations (frequently exacerbated by improper or ignored maintenance) and the hazards posed by unknown knowables and unknown unknowables interact in undesirable ways. This recognition poses a particularly important limitation on proactive reliability and risk analyses that are conducted before systems are constructed and put in service; in a predictive sense, one can only analyze what one understands or knows.

The studies indicated that there was an important discriminating difference between ‘major’ and ‘not-so-major’ failures that involved the ‘energy’ released by and / or expended during the accidents and failures. Not-so-major failures generally involve only a few people, only a few malfunctions or breakdowns, and only small amounts of energy that frequently is reflected in the not-so-major direct and indirect, short-term and long-term ‘costs’ associated with the failure. Major failures are characterized with the involvement of many people and their organizations, a multitude of malfunctions or breakdowns, and the release and / or expenditure of major amounts of energy; this seems to be because it is only through the organization that so many individuals become involved and the access is provided to the major sources of this energy. Frequently, the organization will construct ‘barriers’ to prevent the failure causation to be traced in this direction. In addition, until recently, the legal process has focused on the ‘proximate causes’ in failures; there have been some recent major exceptions to this focus, and the major roles of organizational malfunctions in accident causation have been recognized in court and in public. Not-so-major accidents, if repeated very frequently, can lead to major losses and it has become obvious that it is important for engineers to develop approaches and strategies to address both categories of accidents.

This study also indicated that to many engineers, the human and organizational factor part of the challenge of designing high quality and reliability systems is ‘not an engineering problem;’ frequently, this is believed to be a ‘management problem.’ Often, the discrimination has been posed as technical and non-technical. The case histories of recent major failures clearly indicates that engineers have a critical role to play if the splendid history of successes and achievements are to be maintained or improved. Through integration of technologies from the physical and social sciences, engineers can learn better how to to reach such a goal. The challenge is to wisely apply what is known. To continue to ignore the human and organizational issues as an explicit part of engineering is to continue to experience things that engineers do not want to happen and whose occurrence can be reduced. Engineers can exert important influences on the 'non-technical' parts of systems.

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Chapter 3

Uncertainties

3.1 The End of Certainty

Uncertainty is something that is uncertain, indefinite, problematical, not certain to occur, dubious, not clearly identified or defined. In general, engineers are not comfortable with two things: uncertainty and people. Unfortunately, engineering and engineered systems are dominated by both; there are very few things that are really certain and there is nothing that is not influenced or touched by humans. Most engineering is taught in a deterministic analytical framework where outcomes certain - right or wrong - yes or no. Real engineered systems rarely operate in such a framework - most important things are uncertain and the wonderful world of 'maybe' dominates - the end of certainty.

In this work, uncertainties have been organized into four major categories. This organization has developed because of the different means and methods that need to be used to assess and manage (engineer) to defend against the unanticipated and undesirable effects that can arise from these uncertainties.

The first category of uncertainty has been identified as natural or inherent randomness (Type I Uncertainty = "inherent randomness"). This category of uncertainty is essentially 'information insensitive' - gathering additional data and information has no important effect on our characterizations of the uncertainties. Frequently, this type of uncertainty has been termed 'aleatory'.

The second category of uncertainty is identified as analytical modeling or professional uncertainty (Type II Uncertainty = "modeling"). This type of uncertainty applies to deterministic, but unknown values of parameters (parameter uncertainty); to modeling uncertainty (imperfect understanding of problems, simplified analytical models used in practice); and to the actual state of the system (imprecise knowledge of properties and characteristics). This category of uncertainty is 'information sensitive' - gathering additional data and information can have an important effect on our characterizations of the uncertainties. Frequently, this type of uncertainty has been termed 'epistemic'. In some cases, it has proven to not be possible to develop unambiguous differentiations of Type I and Type II uncertainties - and the term amalgamatic has been applied in such cases.

The third category of uncertainty has been identified as related to human and organizational performance (Type III uncertainty - human and organizational). People and their organizations have important effects on all engineered systems from the time of development of a concept to the time the system is decommissioned. The actions and inactions of people can not always be anticipated and are not always desirable or have desirable outcomes. In one context, this category of uncertainty could also be termed natural or inherent. A primary reason for identifying this category is because of the different approaches and strategies that must be used to address and manage this source of uncertainties.

The fourth category of uncertainty has bee identified as related to development of knowledge and understanding (Type IV uncertainty - knowledge). This category has been divided into two sub-categories: unknown knowables and unknown unknowables. In the first case, the knowledge does exist, but it has not been accessed or not accessed properly. In the second case, the knowledge does not exist. Type IV uncertainty is differentiated from Type II uncertainty because in the case of Type II uncertainty, an analytical - professional model does exist, the physics and mechanics are understood and incorporated into an analytical model, but the model is not perfect or complete and it is not possible to define or characterize unambiguous parameters. Type IV uncertainty could be categorized as part of Type III or Type II uncertainties. Again, a primary reason for identifying this category is because of the different approaches and strategies that must be used to address and manage this source of uncertainties.

In the end, one could contend that uncertainty is uncertainty and that the differentiations are not necessary. In this work, differentiations have been used because of the different approaches and strategies that have proven to be useful in assessing and managing the uncertainties.

In this development, Margins of Quality will be used to address Type I and Type II uncertainties. Other 'human centered' approaches and strategies have proven to be useful in addressing Type III and Type IV uncertainties.

3.2 Developing Characterizations of Uncertainties

Given that an engineer would like to characterize Type I and II uncertainties, how can such characterizations be developed? There are four primary approaches that should be used in an integrated and complimentary way.

- **Judgment**,
- **Simulations**,
- **Experiments** (field, laboratory, office), and
- **Process reviews** (analysis of relevant past failures and successes)

All of these sources represent viable means of providing quantitative characterizations. It is rare to find a structured and consistent use of these four approaches in current assessments.

Simulations (analytical experiments) in the laboratory, office, or field can provide significant insights into how and when uncertainties are developed - and their characteristics. Simulations and simulators can not replicate all important variables and conditions; prototype or real situations generally have very important characteristics that can not be simulated.

Field and office experiments are an important way to gather information on uncertainties. They represent samplings of the more general situation being studied, and must be carefully designed to avoid bias in the results. Measurement equipment often provides important biases that make the realities of prototype behavior different from the behavior observed in the laboratory - and even from behavior observed in the field.

Studies of past successes and failures involving pertinent engineered systems also are an important source of information that if carefully and insightfully done can provide important data on uncertainties in prototype conditions. Qualified engineering assessors are a ‘must’ in performing such studies.

Judgment is perhaps the most important sources of quantitative information on uncertainties. Judgment should not be thought of as the opposite of rational thought. Qualified judgment based on adequate knowledge and experience is based upon both the accumulation of experience and a mental synthesis of factors which allow the evaluator to assess the situation and produce results. Judgment has a primary and rightful place in making quantitative evaluations because available data is always deficient for the evaluation of a particular situation.

Given the present situation regarding definitive quantitative information on which to base objective quantitative evaluations of uncertainties, one must rely primarily on qualified judgment. As adequately structured databases are developed and implemented for uncertainty evaluations, then in the future, more reliance can be placed on objective data and evaluations based on a combination of data and judgment. It is not likely in the near-term, that sole reliance can be placed on objective data sources to provide quantitative evaluations. Adequately qualified and unbiased judgment (expertise) will be essential to develop meaningful results.

This then poses an important problem about how to determine what constitutes adequately qualified and unbiased judgement, who might have such judgement, and how the characterizations might best be elicited or developed. Knowledge, experience, and judgement are inter-related and can be used to define expertise and experts. Klein (1999) has experts as those that see and understand patters, anomalies (cues), situations, workings, opportunities (for high leverage of expertise), improvisation (unique application of expertise), past and future events (extensive knowledge and an ability to develop mental simulations), small differences, own limitations, and thinking about thinking (metacognition). Even with expertise, a wide variety of biases that can distort judgment must be recognized and measures taken to neutralize these biases as much as possible. Vick (2002) has summarized a large number of recent studies of such biases and how they might best be recognized and accommodated in developing assessments of uncertainties.

One of the important issues addressed by Vick is that of estimating very small probabilities or likelihoods of occurrence. Even experts have difficulties estimating or assessing very small probabilities (those much smaller than about 0.01). There is a marked tendency to underestimate large uncertainties. The paths suggested to help overcome such limitations in thinking include anchoring (with specific examples based on extensive databases) and decomposition (breaking the problem into smaller parts that have sufficient information for estimation of uncertainties).

In the end, it has been contended that all expressions concerning uncertainties are expressions of "Degrees of Belief" (Vick, 2002). The author is in complete agreement with this statement. It is critical to identify the bases for these assessments and the potential biases involved in their development.

3.3 Mathematics of Uncertainties

In this development the mathematics of uncertainty have been adopted from those of statistics and probability. Statistics deals with analysis of data; thus, statistics deals with results from the past - outcomes or data gathered from 'experiments' or 'trials'. Probability deals with the likelihoods of outcomes from 'experiments' or 'trials' whose outcomes are not known or can not be known in advance; thus, probability deals with results in the future. Results from the past often do not provide a sufficient basis with which to project future results; the past is not always a guide to the future. In this work, we will use the mathematics of both fundamental statistics and probability theory.

3.3.1 Distribution Functions

There are a wide number of statistical models that can be used to characterize or describe the distributions of variables - continuous and non-continuous. There are more than 30 of these 'shaping functions' that have been used by engineers to define how variables are distributed in terms of their likelihoods. The choice of one or the other of these shaping functions is dependent on how well the shaping function can model the available data.

In this development, two types of distribution functions will be used: Normal and Lognormal. When the process underlying the uncertainty (variability) can be represented as the sum of a large number of independent random variables (independent of their forms), then a Normal (or Gaussian) distribution will be a good model (due to the Central Limit Theorem). When the process underlying the uncertainty can be represented as the product of a large number of independent random variables, then a Lognormal distribution will be a good model.

The assumption that will be made in the subsequent development of Margins of Quality is that the distributions of demands and capacities are Lognormal; i.e., the logarithms of the variables or parameters are Normally distributed. Experience has shown the Lognormal distribution to be an acceptable model for many types of processes of concern in system reliability problems.

When the processes are not well represented by either the Normal or Lognormal distributions, other analytical models will need to be investigated, and the model that provides the best fit to the data in the portions of the distribution of primary concern used in the calculations. In many instances, a Lognormal or Normal distribution can be fitted to the tails or extreme values of the distributions. This allows the reliability analysts to define an equivalent uncertainty and central tendency values that will adequately model the extreme values of the distributions and yet retain the simplicity of the formulations that will be developed - for engineering applications.

For some conditions, it is important to recognize limitations that can be placed on the probabilities of extreme conditions; there are physical limitations on the maximum or minimum values of a variable or parameter. Thus, a continuous probability distribution function needs to be truncated at the limiting condition, and the probability distribution normalized to unity (so the total probability of all outcomes is unity).

When truncated probability distributions and other types of continuous distributions must be utilized, then either numerical integration schemes that allow the direct use of a given type of distribution, or transformation of the continuous distribution into equivalent normal probability distributions must be used to determine the necessary characteristics.

The author has found that ‘graphical statistics’ methods can provide an extremely powerful way to examine how different distribution functions can be used. In this approach, the first challenge is to assemble ‘data’ on the demand or capacity issue of concern. For example, the engineer might have assembled data on the Bias associated with particular demand or capacity analytical processes (ratios of measured results to the predicted or analytical results for the set of conditions present when the measured results were developed). The 'N' different Biases would be rank ordered (largest n =1, to smallest n = N) and the plotting positions (PP) determined from (for 'small' data sets to develop unbiased estimates of the mean position of the distribution):

$$PP = n / (N+1) \qquad \text{Equation 3-1}$$

The next step is to plot these data on different types of graphical statistics plotting papers (e.g. Normal, Lognormal, Weibull, Extreme Value; books of such papers are published) and determine which distribution does the 'best job' of fitting the important parts of the distribution – generally the large values of demands or small values of capacities. It is here that fitting the tails of the distributions to a Lognormal distribution function can be developed. The result often is a median value and standard deviation that is not the same as the median and standard deviation values expressed by all of the data – when concerned with 'failures' often (not always) we are not concerned with all of the data, only the extreme values of the demands and capacities. This is why conventional 'best fit' tests (e.g. Student 't' test) are not particularly useful – they test over the entire range of relative likelihoods). Thus, the process of fitting the distributions must be backgrounded with an adequate understanding of what the fitting will be used for and how this 'approximation' can affect the final results.

Two forms of statistical shaping functions will be used: the density form and the cumulative form. The density form expresses the relative likelihood that the variable will fall between two limits (class interval). The area under the density distribution is constrained to unity. The probability density function for the variable X can be expressed as:

$$f_x (X) = p (x < X < x + \Delta x) \qquad \text{Equation 3-2}$$

where $f_X (x)$ is read as the likellihood that the variable X is a particular value, x, in the interval from x to $x + \Delta x$.

The cumulative form expresses the likelihood that the variable will have a value that is equal to or less than a specified amount. The values of the cumulative form are constrained between zero and unity; representing the integration of the density form from minus to plus infinity. The cumulative probability distribution function for the variable X can be expressed as:

$$F_X (s) = P (X < x) \qquad \text{Equation 3-3}$$

where $F_X (x)$ is read as the likelihood that the variable X is equal to or less than a given value of the variable, x. In this development, in the case of characterization of system capacities, this function is often called a ‘fragility function’ because it expresses the likelihood that the resistance or capacity of the system is equal to or less than a given value of the demand.

3.3.2 Characterizations of Uncertainties

In this development, Type I uncertainties in a variable, X, will be characterized with two parameters (and an assumed form of the distribution - generally Normal or Lognormal):

- central tendency measures of the distribution of X (e.g. median, X_{50}, and mean, $\overline{X}$), and
- dispersion measures of the distribution of X, (coefficient of variation, V_X, standard deviation σ_X)

Type II uncertainty in a variable also will be characterized with two parameters (and an assumed form of the distribution):

- central tendency measures of the Bias, B_X (median, B_{X50}, mean, $\overline{B}_X$) and,
- dispersion measures of the Bias, the coefficient of variation, V_{BX}, standard deviation σ_{BX}

The mean of a variable, $\overline{X}$, can be computed from N values of the variable, x, as follows:

$$\overline{X} = \sum_{1}^{N} \frac{x}{N} \qquad \text{Equation 3-4}$$

For Normally distributed variables, the mean, mode, and median are all the same values (symmetrical distribution). For Lognormally distributed variables, the mean, mode, and median generally are all different values. A Lognormal distribution is a Normal distribution of the logarithms of the variable. These central tendency measures are analogous to the center of gravity of a shape (same as in the case of the mean).

The standard deviation of a variable, σ_X, can be computed from the mean of the variable, $\overline{X}$, and N values of the variable x, as follows:

$$\sigma^2 = \sum \frac{(x - \overline{X})^2}{N} \qquad \text{Equation 3-5}$$

In the case of a Lognormal variable, σ_{lnx} is the standard deviation of the logarithms of the variable X. Further:

$$\sigma_X = X_{50}\,[\exp(\sigma^2_{lnX})\text{-}1]^{0.5} \qquad \text{Equation 3-6}$$

The standard deviation is analogous to the moment of inertia of a shape – the moment of inertia expresses how much of the area is distributed away from the shape's neutral axis.

The ratio of the standard deviation to the mean is known as the coefficient of variation (V):

$$V_X = \frac{\sigma_X}{\overline{X}} \qquad \text{Equation 3-7}$$

The coefficient of variation can be thought of as a normalized measure of the variability of the characteristic *or parameter of concern. This is a very important and useful way to express uncertainty in a variable. It is nondimensional. It expresses the relative variability or dispersion of a distribution – the larger V is, the larger is the dispersion or variability.*

For Lognormal variables:

$$V_X = [\exp(\sigma^2_{lnX}) - 1]^{0.5} \qquad \text{Equation 3-8}$$

and:

$$\sigma_{lnX} = [\ln(1\ V^2_X)]^{0.5} \qquad \text{Equation 3-9}$$

Typical V's for the yield strengths of steel and the compressive strength of concrete are in the range of 8 to 10 %, and 8 to 15 %, respectively. If values are found that are outside this range, typically this is due to poor quality control in the manufacturing process.

Typical V's for the shear strengths of soils fall in the range of 20 to 30 % for relatively homogeneous soils to in excess of 50 % for very inhomogeneous soils (like calcareous formations). The coefficient of variation for prediction of maximum wave heights in a storm generally fall in the range of 20 to 30 %.

The coefficient of variation of the expected annual maximum wind speeds and wave heights in the Gulf of Mexico are about 30 %; in the North Sea they are about 15 %. The coefficient of variation of peak ground accelerations for large magnitude earthquakes at close distances to the epicenter generally fall in the range of 60 % to in excess of 80 %. The coefficient of variation of the expected annual maximum earthquake peak ground acceleration in the coastal areas of California are in the range of V = 100 %; along the East coast of Canada they are in the range of V = 200 %.

One could develop a general ‘scale’ to express variability that might look like this: Very Low (very high quality assurance and control, QA/QC), V = 5 – 10%; Low, V = 10 – 20%; Moderate, V = 20 – 30 %; High (very low QA/QC), V = 30 – 50%; Very High, V = 50 – 100 %; Extremely High, V = 100 – 200 %. The influence of QA/QC is to reduce the likelihood of ‘outliers’; they are detected and corrected / eliminated. Very stringent QA/QC results in very low V’s. Note that ‘natural’ things tend to have low or no QA/QC and thus tend to have much higher V’s.

For Lognormal variables, $\sigma_{\ln X}$ is related to V_X as follows:

$$\sigma_{\ln X} = \sqrt{\ln(1+V_X{}^2)} \qquad \text{Equation 3-10}$$

For $V_X \leq 0.3$ to 0.4, $\sigma_{\ln X} \cong V_X$. *This approximation provides an important check on your calculations.*

For Lognormally distributed parameters, the standard deviations of the variables (X) can be computed from the 90-th percentile (X_{90}, 90 percent of the values are equal to or less than this value) and 10-th percentile (X_{10}) values of the distribution of X as follows:

$$\sigma_{\ln X} = 0.39\,(\ln X_{90} - \ln X_{10}) \qquad \text{Equation 3-11}$$

If it were desirable to fit a Lognormal distribution to the 10,000 year and 100 year return period (the concept of return periods will be discussed later) values of a parameter, X, the uncertainty could be found from:

$$\sigma_{\ln X}. = 0.72 \ln (X_{10,000\ yr} / X_{100\ yr}) \qquad \text{Equation 3-12}$$

If it were desirable to base the characterization of the uncertainty on the 10,000 year and 1,000 year return period values of X, then:

$$\sigma_{\ln X} = 1.61 \ln (X_{10,000\ yr} / X_{1,000\ yr}) \qquad \text{Equation 3-13}$$

3.3.3 Bias - Type II Uncertainties

For Type II uncertainties, the Bias is defined as the ratio of the true or actual value of a variable, X, to the predicted (design, nominal) value of the parameter:

$$B_X = \frac{\text{True or Measured Value}}{\text{Predicted or Nominal Value}} \qquad \text{Equation 3-14}$$

Determination of Bias is one of the most critical parts of determining Margins of Quality. Typically biased values are used in traditional engineering processes, procedures, codes, and guidelines. This is because the engineer wants to be 'conservative.' Biases are frequently 'hidden' in design codes and guidelines, again in an attempt to be conservative. Problems develop due to the compounding of these drives to be conservative and a lack of knowledge of how conservative the results are. In addition, what is conservative for one set of conditions may or may not be conservative for another set of conditions.

The best way to identify the magnitude of the Bias is to compare the results from the analytical models that will be employed by the engineers in determining the demands and capacities associated with a given system during a given point in its life cycle with 'measured' or observed results. However, extreme care must be taken with measured and observed results. Laboratory results generally are different from those in the field – it is extremely difficult to simulate all of the realities of the field in the laboratory and laboratory experiments can introduce effects that are not present in the field. Small scale models generally differ from prototype systems in very important ways (e.g. defects per volume of material). Laboratory scaled models all involve important compromises in what is scaled and how it is scaled. Note that in this case the quantification of the Bias will depend on a particular engineering process (e.g. design code, guideline) because of the reference to a particular analytical process used to determine the 'nominal' or 'engineering' values.

Structural steel element and foundation element Type II Biases and Bias uncertainties are summarized in Table 3.1. These Biases are based on nominal characteristics for the structural elements cited in the American Petroleum Institute (API) guidelines for design of fixed and floating platforms and for dynamic storm and earthquake loadings (2000) and the American Institute of Steel Construction (AISC) (2000). Structural concrete element biases and uncertainties are summarized in Tables 3.2, 3.3, and 3.4. These biases are based on nominal characteristics for the concrete elements cited in the American Concrete Institute (ACI) guidelines (2000).

In many cases, biases are intentionally put in the design guideline in an attempt to be 'conservative'; lower-bounds to test data are utilized rather than mean or best estimate characterizations. For example, the steel yield and ultimate tensile strengths are stated on a 'nominal' value that is generally placed at minus two standard deviations below the mean value. For a steel that has a Coefficient of Variation in yield and tensile strengths of V = 10% and these strengths are Normally distributed, the mean bias $\overline{B}$= 1.2. Another frequent source of bias regards the loadings that are used to reference the structural element capacities. In many cases this loading is 'static.' If the loadings of interest are dynamic or have strain rates that substantially exceed those referenced as static, then there can be important increases in the strength and stiffness that is effective for these loadings. Cyclic dynamic loadings can have counteracting effects (low-cycle fatigue effects). The load engineer must assure that there is a proper matching of the bias with the loadings that are of primary concern.

Table 3.1 - Biases and uncertainties in structural and foundation element capacities for dynamic storm and earthquake loadings based on API and AISC design guidelines

Structure Element	Median Bias B_{50Ru}	Capacity Uncertainty COV %	Structure Element	Median Bias B_{50Ru}	Capacity Uncertainty COV %
Tubular					
Braces			**Plates**	1.05	7 - 8
tension	1.3	10 - 12			
compression	1.4	15 - 18	**Stiffened**	1.1	10 - 12
bending	1.5	10 - 12	**Panels**		
hydrostatic	1.4	10 - 12			
Tubular Joints			**Cylinders**		
T, Y			**Ring-**		
compression	1.2	20 - 22	**Stringer**	1.0	15 - 18
tension	2.7	15 - 16	**Stiffened**		
X, DT					
compression	1.1	10 - 12	**Ring**		
tension	1.7	20 - 22	**Stiffened**	1.0	10 - 12
K, YT					
compression	1.3	20 - 24	**Box Girders**	1.1	10 - 12
tension	1.7	20 - 24			
Piles			**Drag Anchors**		
Static axial			**clays**	1.5	40 - 50
clays	1.0	30 - 40	**sands**	1.2	50 - 60
sands	0.8	50 - 60			
Static lateral					
clays	1.0	20 - 30	**Cables**	1.5	10 - 15
sands	1.1	40 - 50			
Dyn. axial			**Tendons**		
clays	2.5	35 - 45	**(machined**	1.1	7 - 8
sands	0.9	50 - 60	**connections)**		
Dyn. lateral					
clays	1.0	25 - 35			
sands	1.1	40 - 50			

Table 3.2 - Uncertainties in concrete element properties

Property	Bias	COV
concrete compressive strength	1.1	0.10
concrete tensile strength	1.2	0.15
Reinforcement yield stress	1.12.	0.06
Prestressing wire strength	1.06	0.04
Concrete wall thickness		
- walls	1.0	0.05
- domes	1.0	0.11
Distance between tensile compressive longitudinal reinforcement	0.95	0.05
Location of shear reinforcement	1.00	0.10

Table 3.3- Model uncertainties for capacities of concrete beams

Loading	Material	Bias	COV
Bending	normal density agg.	1.13	0.10
	light weight aggregate	1.27	0.12
Shear			
w/o reinf.		1.48	0.41
w reinf.		1.23	0.35
w/axial comp & w/o reinf.		1.22	0.24

Table 3.4- Concrete element capacity characteristics

Elements	Loadings	R_{50} / Rn	COV
Reinforced Slabs	Flexure, Pressure	1.12 - 1.22	0.14 - 0.16
Reinforced Beams	Flexure	1.01- 1.09	0.11 - 0.12
Prestressed Precast Beams	Flexure	1.04 - 1.06	0.057-0.097
Post-tensioned Cast-in-Place Slabs, Domes	Flexure, Pressure	1.02-1.05	0.061-0.144
Short Columns	Axial and Flexure	0.95 - 1.05	0.14 - 0.16
Slender Columns		0.95 - 1.10	0.17-0.12

Analytical results derived from mathematical models are similarly different from those in the field. All analytical models have 'deficiencies' or 'flaws' due to the assumptions that underlie the analytical models; 'engineers have an awesome ability to build analytical models and a dangerous tendency to believe that their results represent reality.' Results from analytical models can introduce their own forms of Bias and can seriously distort the understanding of the true variability associated with some processes.

Even field results must be regarded carefully because of the inherent limitations in measurement technology and equipment and the necessary analyses that must be performed on the measured 'signals.' One of the most important reasons for utilizing the principles developed here is to determine and characterize all appropriate sources of Bias (Type II uncertainty) and variability (Type I uncertainty) and then to determine how to define demands and capacities and the associated engineering analytical procedures that will take proper account of these uncertainties. This must be done in the context of the analytical models that will be employed by the engineers in determining both demands and capacities of the systems. This is one of the primary reasons for this development – to develop a comprehensive and thorough understanding of demand and capacity processes (fundamental understanding of physics and mechanics required – adequate deterministic understanding first), then to understand the uncertainties and variabilities associated with these processes, and then to configure the demand and capacity processes and parameters so that a desirable and acceptable reliability can be developed during the life cycle of a system. As will be discussed subsequently, it will be important to assure that there is consistency between the uncertainties that are included in the analytical results and those that are included in the 'target' reliabilities.

One category of Type II uncertainty that is frequently ignored is that associated with the demand and capacity distributions based on small data sets. This uncertainty can be expressed with the statistical concept of 'confidence intervals.' Such intervals express the uncertainty that a particular distribution lies in a given range for a given number of data points that are used to characterize the distribution. The uncertainty is smallest for the central tendency areas of the distribution and largest for the tail areas and varies with the square root of the number of data points. This type of uncertainty is clearly associated with the statistical models that are used in the analyses. This type of uncertainty is clearly information sensitive in that additional data points can be developed and this can lead to a decrease in this uncertainty (but, it costs to gather such data). Thus, eventually the engineer can make an evaluation of the potential value of additional data in improving the reliability of a given system.

3.3.4 Correlations

The "correlation coefficient", ρ_{xy}, expresses how strongly two variables, X and Y, are related in magnitude to each other. It measures the strength of association between the magnitude of two variables. The correlation coefficient ranges between positive and negative unity (-1 ≤ _≤ +1). The correlation coefficient is the ratio of the covariance (COV) of the marginal variables to the product of their standard deviations:

$$\rho_{xy} = \frac{\mathrm{COV}[X,Y]}{\sigma_x \sigma_y} \qquad \text{Equation 3-15}$$

$$\mathrm{COV}[X,Y] = \overline{[XY]} - \overline{X}\,\overline{Y} \qquad \text{Equation 3-16}$$

If ρ = 1, the two variables are perfectly correlated, so that knowing X allows one to make perfect predictions of Y. If ρ = 0, they have no correlation, or are independent, so that the occurrence of X has no affect on the occurrence of Y and the magnitude of X is not related to the magnitude of Y. Independent random variables are uncorrelated, but uncorrelated random variables (magnitudes not related) are not in general independent (their occurrences can be related).

Independence - dependence refers to the potential effect that the occurrence of one variable has on the occurrence of another variable. This characteristic is addressed using conditional probabilities (to be discussed later). Correlation refers to the potential connection between the magnitude of one variable with the magnitude of another variable. This characteristic is addressed using the correlation coefficient.

Frequently, the correlation coefficient can be quickly and accurately estimated by plotting the variables on a scattergram that shows the results of measurements or analyses of the magnitudes of the two variables. Plot paired values of the magnitudes of the X and Y variables. Two strongly positively correlated variables will plot with data points that closely lie along a line that indicates as one variable increases the other variable increases. Two strongly negatively correlated variables will plot with data points that closely lie along a line that indicates as one variable increases, the other variable decreases. If the plot does not indicate any structure or systematic variation in the variables, the general conclusion is that the correlation is very low or close to zero.

3.3.5 Analyses of Component Uncertainties (Decomposition)

To evaluate the uncertainties of the system demands and capacities from the components of the demands and capacities that contribute uncertainties (decomposition), one can use the algebra of Normal Functions. This approach is equivalent to a first order - second moment (FOSM) method to propagate the central tendencies and uncertainties of multiple parameters. This approach is based on a first order Taylor Series expansion of the distribution characteristics and then retention of only the first two terms of the expansion.

Other 'advanced' methods are available to perform analyses of component uncertainties. These include the First Order Reliability Method (FORM), the Second Order Reliability Method (SORM), and the Monte Carlo Simulation Method (MCSM) (Melchers 1987, Ang and Tang 1975, Benjamin and Cornell 1970, Thoft-Christensen and Baker 1982). These advanced methods require much more extensive computational capabilities. The comparisons that have been performed indicate that for most engineering system uncertainty analysis problems, the FOSM approach can provide acceptable accuracies (results within 10% of those from the advanced methods).

3.3.5.1 Addition & Subtraction of Random Variables

For the addition or subtraction of two random variables, $(X \pm Y) = Z$, the mean of the resultant distribution can be calculated as follows:

$$\overline{Z} = \overline{X} \pm \overline{Y} \qquad \text{Equation 3-17}$$

Note that the mean values are not affected by correlation: the mean of the sum is the sum of the means (this is generally true). Thus if,

$$Z = \sum_{i=1}^{n} a_i X_i \qquad \text{Equation 3-18}$$

then,

$$\overline{Z} = \sum_{i=1}^{n} a_i \overline{X}_i \qquad \text{Equation 3-19}$$

The standard deviation of the resultant distribution can be determined as follows:

$$\sigma_Z = \sqrt{\sigma_X{}^2 + \sigma_Y{}^2 \pm 2\,\rho\,\sigma_X\,\sigma_Y} \qquad \text{Equation 3-20}$$

ρ is the correlation coefficient between the two variables X and Y. In this case, correlation has an important effect on the variance (square of the standard deviation) or the standard deviation. For the general case:

$$\sigma_Z^2 = \sum_{i}^{n} \sum_{j}^{n} a_i a_j \rho_{ij} \sigma_{Xi} \sigma_{Xj} \qquad \text{Equation 3-21}$$

which for uncorrelated random variables reduces to:

$$\sigma_Z^2 = \sum_{i=1}^{n} a_i^2 \sigma_{Xi}^2 \qquad \text{Equation 3-22}$$

Note that if one had three or more random variables, the foregoing formulations can be applied repeatedly to determine the resultant mean and standard deviation. If one had Lognormal distributions, it would be necessary to determine the equivalent mean value of the distributions (not the mean of the logarithms) and the standard deviations of the distributions (not the standard deviation of the logarithms).

Note that for the either addition or subtraction of two independent random variables, (X ± Y) = Z,

$$\sigma_Z = \sqrt{\sigma_X^2 + \sigma_Y^2} \qquad \text{Equation 3-23}$$

For the case of independent Logarithmic random variables and for Z = X Y, then log Z = log X + Log Y. Thus,

$$\overline{\log Z} = \overline{\log X} + \overline{\log Y} \qquad \text{Equation 3-24}$$

$$Z_{50} = X_{50}\, Y_{50} \qquad \text{Equation 3-25}$$

and

$$\sigma^2 \log Z = \sigma^2 \log X + \sigma^2 \log Y \qquad \text{Equation 3-26}$$

3.3.5.2 Multiplication of Random Variables

For the multiplication of two random variables, (X Y) = Z, the mean of the resultant distribution can be calculated as follows:

$$\overline{Z} = \overline{X}\overline{Y} + \rho\, \sigma_X \sigma_Y = \overline{X}\overline{Y} + COV_{XY} \qquad \text{Equation 3-27}$$

where COV_{XY} is the covariance of X and Y (note $\rho_{XY} = COV_{XY} / \sigma_X \sigma_Y$). In this case, the mean value of the resultant distribution is dependent on the correlation between the variables. Only if the variables are not correlated:

$$\overline{Z} = \overline{X}\overline{Y} \qquad \text{Equation 3-28}$$

The standard deviation of the resultant distribution can be calculated as follows:

$$\sigma_Z = \overline{X}\,\overline{Y}\sqrt{(1+\rho^2)[\frac{\sigma_X^2}{\overline{\mathbf{X}}^2} + \frac{\sigma_Y^2}{\overline{\mathbf{Y}}^2} + (\frac{\sigma_X^2}{\overline{\mathbf{X}}^2}\frac{\sigma_Y^2}{\overline{\mathbf{Y}}^2})]} \qquad \text{Equation 3-29}$$

or

$$\sigma_Z = \sqrt{(1+\rho^2)[(\overline{X}\sigma_Y)^2 + (\overline{Y}\sigma_X)^2]} \qquad \text{Equation 3-30}$$

When the random variables X and Y can be considered independent ($\rho = 0$), then:

$$\sigma_z^2 = \sigma_X^2 \overline{Y}^2 + \sigma_Y^2 \overline{X}^2 + \sigma_X^2 \sigma_Y^2 \qquad \text{Equation 3-31}$$

When the random variables X and Y can be perfectly dependent ($\rho = \pm 1$), then:

$$\sigma_z^2 = 2\sigma_X^2 \overline{Y}^2 + 2\sigma_Y^2 \overline{X}^2 + 2\sigma_X^2 \sigma_Y^2 \qquad \text{Equation 3-32}$$

When the random variables X and Y can be considered independent $\rho = 0$), then:

$$V_Z = \sqrt{V_X^2 + V_Y^2 + V_X^2 V_Y^2} \qquad \text{Equation 3-33}$$

and when the coefficients of variation are small:

$$V_Z = \sqrt{V_X^2 + V_Y^2} \qquad \text{Equation 3-34}$$

If the values of the variables Xi are independent, then:

$$\overline{Z} = \prod_{i=1}^{n} \overline{X_i} \qquad \text{Equation 3-35}$$

and

$$\sigma_Z^2 = \prod_{i=1}^{n} \overline{X_i^2} - (\prod_{i=1}^{n} \overline{X_i})^2 \qquad \text{Equation 3-36}$$

To determine the product of two variables when one of the variables is raised to a power ε ($Z = X\,Y^{\varepsilon}$):

$$\overline{Z} = \overline{X}(\overline{Y})^{\varepsilon} + \rho\, \sigma_X \sigma_\Psi \qquad \text{Equation 3-37}$$

and

$$\sigma_Z = \overline{X}(\overline{Y})^{\varepsilon} \sqrt{(1+\rho^2)[\frac{\sigma_X^2}{\overline{X}^2} + \frac{\sigma_Y^2}{\overline{Y}^2} + (\frac{\sigma_X^2}{\overline{X}^2}\frac{\sigma_Y^2}{\overline{Y}^2})]} \qquad \text{Equation 3-38}$$

When the random variables X and Y can be considered independent ($\rho = 0$), and V_X and V_Y are small ($V << 1$), then:

$$V_Z \cong \sqrt{V^2{}_X + (\varepsilon V_Y)^2} \qquad \text{Equation 3-39}$$

For all $Z = a\,X$:

$$\overline{Z} = a\overline{X} \qquad \text{Equation 3-40}$$

$$\sigma_Z^2 = a^2\sigma_X^2 \qquad \text{Equation 3-41}$$

Then for $Z = X^a$, log Z = a log X and:

$$\overline{\log Z} = a\overline{\log X} \qquad \text{Equation 3-42}$$

$$\sigma_{\log Z}^2 = a^2\sigma_{\log X}^2 \qquad \text{Equation 3-43}$$

3.3.5.3 Division of Random Variables

For the division of two random variables, (X / Y) = Z, the mean of the resultant distribution can be calculated based on FOSM as follows:

$$\overline{Z} = \frac{\overline{X}}{\overline{Y}} \qquad \text{Equation 3-44}$$

The standard deviation of the resultant distribution can be calculated as follows:

$$\sigma_Z = \frac{\overline{\mathbf{X}}}{\overline{\mathbf{Y}}}\sqrt{[\frac{\sigma_X^2}{\overline{\mathbf{X}}^2} + \frac{\sigma_Y^2}{\overline{\mathbf{Y}}^2} - 2\rho(\frac{\sigma_X}{\overline{\mathbf{X}}}\frac{\sigma_Y}{\overline{\mathbf{Y}}})]} \qquad \text{Equation 3-45}$$

When the random variables X and Y can be considered independent ($\rho = 0$), and V_X and V_Y are small ($V \ll 1$), then:

$$V_Z \cong \sqrt{V_X{}^2 + V_Y{}^2} \qquad \text{Equation 3-46}$$

3.3.5.4 Moments of a Quadratic Form and of a Root

For a function $Z = a X^2 + bX + c$, based on the FOSM the resulting first and second moments can be determined as follows:

$$\overline{Z} = a(\overline{X}^2 + \sigma_X^2) + b\overline{X} + c \qquad \text{Equation 3-47}$$

$$\sigma_Z^2 = \sigma_X^2(2a\overline{X} + b)^2 + 2a^2\sigma_X^4 \qquad \text{Equation 3-48}$$

For a function $Z = X^{1/2}$

$$\overline{Z} = [\overline{X}^2 - \frac{\sigma_X^2}{2}]^{1/4} \qquad \text{Equation 3-49}$$

$$\sigma_Z^2 = \overline{X} - (\overline{X}^2 - \frac{\sigma_X^2}{2})^{1/2} \qquad \text{Equation 3-50}$$

3.3.6 Probability

Probability is defined as a numerical measure of the likelihood that an event occurs relative to a set of alternative events that do not occur. The probability of an event E is denoted as P(E\X) where PO ius the probability operator and \X denotes whatever may be known or assumed in determining P(E|X) - all probabilities are conditional . In most cases, P)E|X) will be deonoted as P(E), the state of conditionality being understood.

The probability P(E) of an event E is a real positive number: $0 \leq P(E) \leq l$. Let E_1 and E_2 be two events. The *union* of E_1 *or* E_2 is an event denoted by $E_1 \cup E_2$ and it is the subset of sample points that belong to E_1 or E_2. The intersection of E_1 *and* E_2 is an event denoted by $E_1 \cap E_2$ and is the subset of sample points that belong to both E_1 and E_2. The two events E_1 and E_2 are said to be mutually exclusive if they have no sample points in common; in this case, $E_1 \cap E_2 = 0$ (an impossible event).

The probability that either or both of two events E_1 and E_2 occur is

$$P(E_1 \cup E_2) = P(E_1) + P(E_2) - P(E_1 \cap E_2) \qquad \text{Equation 3-51}$$

For two mutually exclusive events

$$P(E_1 \cup E_2) = P(E_1) + P(E_2) \qquad \text{Equation 3-52}$$

The probability that both events E_1 and E_2 occur is

$$P(E_1 \cap E_2) = P(E_1|E_2)\, P(E_2) = P(E_2|E_1)\, P(E_1) \qquad \text{Equation 3-53}$$

for the conditional probability

$$P(E_1|E_2) = P(E_1 \cap E_2) / P(E_2) \qquad \text{Equation 3-54}$$

If E_1 and E_2 are independent,

$$P(E_1|E_2) = P(E_1) \text{ and } P(E_2|E_1) = P(E_2) \qquad \text{Equation 3-55}$$

$$P(E_1\ P(E_1 \cup E_2) = P\ (E_1) + P(E_2)\ E_2) = P(E_1)\ P(E_2) \qquad \text{Equation 3-56}$$

Note that for non mutually exclusive events if small probabilities are assumed and the cross products discarded (known as small event approximation):

$$P(E_1 \cup E_2) = P\ (E_1) + P(E_2) \qquad \text{Equation 3-57}$$

If ~~E~~ is the event that E does not occur (complementary event),

$$P(\cancel{E} \cup E) = P(\cancel{E}) + P(E) = 1 \qquad \text{Equation 3-58}$$

and therefore

$$P(\cancel{E}) = 1 - P(E) \qquad \text{Equation 3-59}$$

For events E_i, i = 1, …, n, mutually exclusive and collectively exhaustive (covering all possibilities without intersections)

$$P(A) = P(A|E_1)\ P(E_1) + P(A|E_2)\ P(E_2) + \ldots + P(A|E_n)\ (P(E_n) \qquad \text{Equation 3-60}$$

These results lead to Bayes' Theorem - for a particular event E_i and the event A

$$P(E_i \mid A) = \frac{P(A \mid E_i)P(E_i)}{\sum P(A \mid E_i)P(E_i)} = \frac{P(A \mid E_i)P(E_i)}{P(A)} \qquad \text{Equation 3-61}$$

This expression can be interpreted as follows: E - cause, A - effect, P(E) - probability of cause, P(A) - probability of effect, P(A|E) - conditional probability of effect given the cause, P(E|A) - conditional probability of cause given effect.

For a binary state of failure and non failure of a element or system caused by some process, the observed effect of that process can be called 'evidence' (E, e.g. observed collapse or non-collapse). Then Bayes' Theorem can be expressed as follows

$$P[\text{failure} \mid E] = \frac{P[E \mid \text{failure}]P[\text{failure}]}{P[E \mid \text{failure}]P[\text{failure}] + P[E \mid \text{nofailure} + P[E \mid \text{nofailure}]P[\text{nofailure}]}$$

Equation 3-62

3.4 Elements and Systems: Series and Parallel Elements

In this development, the references are made to 'elements' and 'systems'. Systems (e.g. structures, hardware) are comprised of an assemblage of elements. Subsystems are sub-assemblies of elements frequently referred to as 'components.'

A system can be decomposed into sub-systems of series and parallel elements. A series sub-system is one in which the failure of one of the elements leads to the failure of the system. A parallel system is one in which the failure of the system only occurs when all of the elements have failed. *Any system can be equivalently represented by a parallel system of series subsystems and / or a series system of parallel subsystems.*

3.4.1 Parallel Element Systems

In probabilistic terms, the probability of failure of parallel element system can be expressed in terms of the probabilities of failure of its N elements as:

$$Pf_{system} = (\ Pf_1) \text{ and } (Pf_2) \text{ and } \ldots (\ Pf_{N)}$$ Equation 3-63

$$Pf_{system} = (\ Pf_1) \cap (Pf_2) \cap \ldots (\ Pf_{N)}$$ Equation 3-64

The strength characteristics of a parallel system are dependent on the ductility (deformation or strain capacity) and residual strength (load or stress capacity after the yield strength has been developed) characteristics of the elements that comprise the system. A ductile - high residual strength element is one which can continue to carry a major portion of its capacity when it reaches its yield-capacity strain (stress or load carrying ability). A ductile element is able to develop large plastic deformations without 'failure' (fracture, rupture). A perfectly brittle element ceases to carry any load as soon as its yield strain is reached.

The foregoing discussion can be visualized by examining the capacity of system comprised of perfectly elastic - plastic elements. The capacity of the system will be the sum of the capacities of the individual elements. This assessment is dependent on the ductility or strain capacity of the elements. They must be ductile enough to allow the loads to be redistributed to the other non-yielded elements after yield of the first element. In a similar way, given sufficient ductility, a system comprised of elements that have residual strengths that are less than the yield strength has a strength that is equal to the sum of the residual strengths. The residual strength and ductility of elements have major influences on the behavior of a system comprised of these elements.

A parallel system with N perfectly ductile elements, the expected capacity, $\overline{R}$, of this system is determined by the sum of the expected capacities of the elements (i = 1 to N):

$$\overline{R} = \sum_{i=1}^{N} Ri \qquad \text{Equation 3-65}$$

If the capacities of the elements are independent ($\rho = 0.0$) and Normally distributed, the standard deviation of the system capacity, σ_R, can be expressed in terms of the standard deviations of the capacities of the elements, σ_i as :

$$\sigma_R{}^2 = \sum_{i=1}^{N} \sigma_i{}^2 \qquad \text{Equation 3-66}$$

If the capacities of the elements are positively correlated, the standard deviation of the capacity of the system will increase, and the probability of failure of the system will increase.

An important conclusion that can be reached from these results is that if the degree of correlation between the capacities or the probabilities of failure of the parallel elements is high ($\rho \geq 0.8$), *then the probability of failure of the system is will be approximately the probability of failure of a single element.*

3.4.2 Element Performance Correlations

There can be a variety of ways in which correlations can be developed in elements and between the elements that comprise a system that have important ramifications on the performance characteristics of the elements, and consequently on the characteristics of the system itself. Important sources of ‘correlations’ include:

- **capacity and demand correlations**
- **element to element characteristics correlations, and**
- **failure mode correlations.**

Correlation or dependence of element performance characteristics (expressed by the correlation coefficient, ρ_{ij}) means that there is a (positive or negative) statistical relationship between paired element (i and j) characteristics (e.g. strength, ductility). For a positive correlation, if any one element is above its expected characteristic, the other paired element is also likely to be above its expected characteristic. Unity correlation, $\rho = 1.0$, implies perfect dependence. Negative correlation implies as the characteristic of one element increases, the characteristic of the other paired element decreases. Independence of the element strengths implies no statistical correlation of the element strengths; thus, $\rho = 0.0$. Correlation in element characteristics can be developed by construction (e.g. materials, fabrication), operations (e.g. corrosion protection), and natural causes (e.g. deposition of soils).

Correlations can also be developed between the failure modes of a system. A useful expression to determine the correlation coefficient between the probabilities of failure of a system's components (or correlation of failure modes) is:

$$\rho_{fm} = \frac{V^2{}_S}{V^2{}_R + V^2{}_S} \qquad \text{Equation 3-67}$$

where $V^2{}_S$ and $V^2{}_R$ are the squared coefficients of variation of the load (S) and capacity (R), respectively. It is often the case for structures that the coefficients of variation of the loadings are much larger than those of the capacity. Thus, the correlation of the probabilities of the failure of the system's components are generally very close to unity, and there is a high degree of correlation between the system's failure modes.

3.4.3 Series Element Systems

A series (weak-link) system fails when any single element fails. In probabilistic terms, the probability of failure of a series system can be expressed in terms of the probabilities of failure of its N elements as:

$$Pf_{system} = (Pf_1) \text{ or } (Pf_2) \text{ or } \ldots (Pf_N) \qquad \text{Equation 3-68}$$

For a series system comprised of N elements, if the elements have the same strengths and the failures of the elements are independent ($\rho = 0$), then the probability of failure of the system can be expressed as:

$$Pf_{system} = 1 - (1 - Pf_i)^N \qquad \text{Equation 3-69}$$

If Pfi is small, as is usual, then approximately:

$$Pfsystem \approx N\ Pfi \qquad \text{Equation 3-70}$$

If the elements (independent) have different failure probabilities:

$$Pfsystem = \sum_{i=1}^{N} Pf_i \qquad \text{Equation 3-71}$$

If the elements are perfectly correlated then:

$$Pf_{system} = \text{maximum}\ (Pf_i) \qquad \text{Equation 3-72}$$

These results indicate that even though most systems are highly redundant in the sense of indeterminacy, the effects of correlation (principally through the loadings), and the effects of less than ideal ductile element behavior erase the majority of significant effects on the system. These results indicate that if one is able to identify the Most Likely to Fail (MLTF) element within a system, and then characterize the probability of failure of that element, then one has a good estimate of the system probability of failure.

This development allows expression of useful 'bounds' on the reliability of systems comprised of 'i' elements that have probabilities of failure P(Fi) for the conditions of independence (zero correlation coefficient) and perfect dependence (unity correlation coefficient):

Parallel element systems

$$\prod_i P(Fi) \le P[\bigcap_i Fi] \le \min(P(Fi)) \qquad \text{Equation 3-73}$$

Series element systems

$$\max(PFi)) \le P[\bigcup_i Fi] \le 1 - \prod_i (1-(PFi)) \le \sum_i P(Fi) \qquad \text{Equation 3-74}$$

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Chapter 4

Reliability

4.1 Reliability and Quality

Reliability (Ps) is the likelihood or probability that the engineered system will perform acceptability. The probability of failure (Pf) is the likelihood that the system will not perform acceptably (Pf = 1 - Ps).

Reliability can be characterized with demands (S) and capacities (R). Demands and capacities must be defined with the same type of terms and units. When the demand exceeds the capacity, then system 'fails'. The demands and capacities can be variable and uncertain.

Quality is defined as freedom from unanticipated defects. Quality is fitness for purpose. Quality is meeting the requirements of those who own, operate, design, construct, and regulate structure systems. These requirements include those of serviceability, safety, compatibility, and durability.

Reliability is defined in this work as the probability (likelihood) that a given level of quality will be achieved during the design, construction, and operating life-cycle phases of an engineered system. Reliability is the likelihood that the system will perform in an acceptable manner. Acceptable performance means that the system has desirable serviceability, safety, compatibility, and durability. The compliment of reliability is the likelihood or probability of unacceptable performance; the probability of failure. Failure is an event that results in an undesirable compromise in quality of the system.

The probability or likelihood that the system will survive the demand is defined as the reliability

$$Ps = P (R \geq S) \qquad \text{Equation 4-1}$$

where P (.) is read as the probability that the capacity (R) equals or exceeds the demand (S). Ps is the probability of success, or reliability.

The probability of failure (Pf) is the compliment of the reliability

$$Pf = 1 - Ps \qquad \text{Equation 4-2}$$

Or,

$$Pf = P (R \leq S) \qquad \text{Equation 4-3}$$

The system Pf's can be expressed in a variety of ways to address a variety of considerations important to a given system. For example, serviceability demands could be expressed in terms of the number of days that the system should be available. The capacity would then be expressed as the number of days that the structure system would be available. Durability demands could be expressed as the number of years that the system would be in service. The durability capacity would then be the number of years to a fatigue or corrosion failure. Safety demands could be expressed as the loadings or displacements (pressures, strains) that could be imposed on or induced in a system. The safety capacity would then be the capacity of the system to withstand loadings or displacements (pressures, strains). Compatibility demands could be expressed in schedule, budgetary, or environmental pollution requirements; e.g. the funds that are required to construct a system. The compatibility capacities would be expressed as the maximum schedule, budgetary, or environmental pollution tolerance, e.g. the funds that are available to pay for the construction of the system.

The cumulative probability distribution function for the resistance can be expressed as

$$F_R (S) = P (R < s) \qquad \text{Equation 4-4}$$

where $F_R (S)$ is read as the probability that the resistance, R, is equal to or less than a given value of the demand, s.

The probability density function for the loading can be expressed as

$$f_S (S) = p (s < S < s + \Delta s) \qquad \text{Equation 4-5}$$

where p (S) is read as the probability that the loading is a particular value, S, in the interval from s to s + Δs.

The probability of failure Pf can be determined as the product of the probabilities of two independent (or dependent) events, P_1 and P_2 (P_1 = fs (S) ds and $P_2 = F_R$ (S), summed over all possible occurrences

$$Pf = Pf = \int_{-\infty}^{+\infty} F_R(S) f_S(S) ds \qquad \text{Equation 4-6}$$

or expressed in a numerical integration form

$$Pf = \sum F_R[s] f_S[S] \Delta s \qquad \text{Equation 4-7}$$

Note that if we know or can develop the probability functions for the demand and capacity, numerical integration can be used to perform the convolution of the demand and capacity to determine the probability of failure. This formulation will work for any and all forms of distribution functions for demands and capacities. Only for special forms of distributions can the closed form of this formulation be used, e.g. Normally distributed R and S or Lognormally distributed R and S. *This is a very important concept that should be carefully studied and remembered.*

If R and S are independent Normally distributed variables, Pf may be expressed as

$$Pf = P(G \leq 0) = P[G(R,S) \leq 0] = P[(R-S)] \leq 0] \qquad \text{Equation 4-8}$$

where G () is termed the 'limit state function'; the probability of failure is the probability of violation of the defined limit state. Thus

$$\overline{G} = \overline{R} - \overline{S} \qquad \text{Equation 4-9}$$

and

$$\sigma_G^2 = \sigma_R^2 + \sigma_S^2 \qquad \text{Equation 4-10}$$

Since R and S are Normally distributed, G, a linear function of R and S, is also Normally distributed and $(G-\overline{G})/\sigma_G$ is unit Standard Normal giving

$$Pf = \Phi\left(\frac{0-\overline{G}}{\sigma_G}\right) = \Phi\left(\frac{\overline{S}-\overline{R}}{\sqrt{\sigma_S^2+\sigma_R^2}}\right) \qquad \text{Equation 4-11}$$

The *Reliability Index*, β, may now be defined as the ratio and $(\overline{G}/\sigma_G)$ or the number of standard deviations by which and $\overline{G}$ exceeds zero. In analytical terms, the reliability can be computed from

$$Ps = \Phi(\beta) \qquad \text{Equation 4-12}$$

where $\Phi(\beta)$ is the standard Normal distribution cumulative probability of the variate, β, from – infinity to β.

Given Lognormally distributed (these terms refer to the analytical model that describe the probability distribution of the parameter) independent demands (S) and capacities (R), β is computed as follows:

$$\beta = \frac{\ln(R_{50}/S_{50})}{\sqrt{\sigma_{\ln R}^{2} + \sigma_{\ln S}^{2}}} \qquad \text{Equation 4-13}$$

where R_{50} and S_{50} refer to the median capacities and demands, respectively. Or,

$$\beta = \frac{\ln(FS_{50})}{\sigma_{\ln RS}} \qquad \text{Equation 4-14}$$

where FS_{50} is the median 'Factor of Safety'.

There can be correlations between demands and capacities. As the demands change in magnitude or intensity, the capacities can change in magnitude or intensity. Increasing demands resulting in decreasing capacities are an example of negative correlation in the demand and capacity.

For the case of Lognormally distributed correlated demands and capacities, β is computed as follows:

$$\beta = \frac{\ln(R_{50} / S_{50})}{\sqrt{\sigma_{\ln R}^{2} + \sigma_{\ln S}^{2} - 2\rho\sigma_{\ln R}\sigma_{\ln S}}} \qquad \text{Equation 4-15}$$

Given Normally distributed correlated demands and capacities, β is computed as follows:

$$\beta = \frac{R - S}{\sqrt{\sigma^{2}_{R} + \sigma^{2}_{S} - 2\rho\sigma_{R}\sigma_{S}}} \qquad \text{Equation 4-16}$$

Note that for systems whose demands and capacities are positively correlated (as magnitude of demand increases so does the magnitude of the capacity), there is a reduction in the denominator of the two foregoing expressions, a consequent increase in the Safety Index, and a decrease in Pf. In the same way, for systems whose demands and capacities are negatively correlated (as the magnitude of demand increases, the magnitude of the capacity decreases), there is an increase in the magnitude of the denominator of the foregoing expressions, a consequent decrease in the Safety Index, and an increase in Pf.

To very good accuracy, the reliability can be computed from ($1 < \beta < 3$)

$$Ps \approx 1.0 - 0.475 \exp -(\beta^{1.6}) \qquad \text{Equation 4-17}$$

The probability of failure can be computed as

$$Pf \approx 0.475 \exp (-\beta^{1.6}) \qquad \text{Equation 4-18}$$

Very approximately

$$Pf \approx \approx 10^{-\beta} \qquad \text{Equation 4-19}$$

Whenever possible, the exact form of the standard cumulative Normal distribution function Φ() should be used- it is an Excel Macro!.

4.2 Time & Encounter Considerations

If the 'average return period' (ARP), the average number of years (or other units of time) between occurrences of the demand events) associated with the demand that equals the system's capacity can be determined, then for conditions when the demand uncertainty is much larger than the capacity uncertainty:

$$Pf \approx ARP^{-1} \qquad \text{Equation 4-20}$$

The foregoing expression is an extremely important concept. If one can determine the return period of the loading event that will bring the system to its performance or 'limit' state of interest (e.g. serviceability), then the reciprocal of that return period is a good estimate of the probability of failure of the structure during the specified time period.

The time period that often is used to define the probability characteristics of the maximum demands is one year; thus, the demandss will need to be expressed as the likelihoods of expected annual maximum demand, S, being equal to or less than some value, s, or:

$$F_{Sa} = P (S < s) \qquad \text{Equation 4-21}$$

Other time periods can be used; for example the demand characterization could be based on the lifetime (exposure period) of the system, L (expressed in years). In this case, given that the demands were independent from year to year (this is another way of saying that the occurrence of a demand in one year does not influence the occurrence of another demand in another year), the lifetime likelihood would be related to the annual likelihood by:

$$F_{ST} = (F_{Sa})^{L} \qquad \text{Equation 4-22}$$

The effect of the exposure period, L, is to increase the median value of the loading and decrease the standard deviation of the loading.

If the capacity were changing as a function of time, for example, due to fatigue degradation of the strength, then Pf could be determined for discrete time intervals recognizing the change in the capacity, and the Pf's summed over the total exposure period (L).

Relating the annual risk, Pf_a, to the lifetime risk, Pf_L, is simple if each year is considered a statistically independent event (no correlation of trials from year to year). In this case, for a lifetime of L years:

$$Pf_L = 1 - (1 - Pf_a)^L \qquad \text{Equation 4-23}$$

For small Pfa, this gives:

$$Pf_L = L\ Pf_a \qquad \text{Equation 4-24}$$

There could be correlation of risk from year to year due to statistical dependence through several important variables in Pf including the system resistance, some of its demands, and some of the sources of uncertainty (e.g. methods of analysis). Many of the variables are independent of the natural randomness associated with such occurrences as storms or earthquakes, and may be considered constant during the lifetime. If one takes the other extreme assumption, and considers perfect dependence or correlation from year to year, then:

$$Pf_L = Pf_a \qquad \text{Equation 4-25}$$

In this development, the demand probabilities and probabilities of failure will be based on a period of one-year (annual), unless otherwise stated.

The average return period (ARP, expressed in years) associated with a given variable (expected annual maximum Ha) can be expressed as:

$$ARP = (1 - F_{Ha})^{-1} \qquad \text{Equation 4-26}$$

where F_{Ha} expresses the likelihood that the annual expected maximum demand variable at the system location, H_a, is equal to or less than a given value, h:

$$F_{Ha} = P\ (H < h) \qquad \text{Equation 4-27}$$

For some types of demands an understanding of the probabilities of encountering the demands can be developed from the results of a Poisson model. This probability model has no temporal or spatial memory. The model expresses the probability of experiencing n events in L years, $P_N(n)$, given that one experiences N demand events on the average of once every T years as:

$$P_N(n) = \frac{(L/T)^n \exp(-L/T)}{n!} \qquad \text{Equation 4-28}$$

The probability of experiencing at least one demand event during an exposure period L is:

$$E = 1 - \exp(-L/T) \qquad \text{Equation 4-29}$$

E is called the encounter probability. When the exposure period is equal to the average return period of the demand event, the encounter probability is 63.2 percent.

4.3 Intrinsic and Extrinsic Factors

Failures to achieve desirable quality in an engineered system can develop from Intrinsic (I, Type I and Type II uncertainties) or Extrinsic (E, Type III and Type IV uncertainties) factors (exclusive, exhaustive). The probability of failure of the system to develop quality attribute (i), P(Fi), is

$$P(F_i) = P(F_{iI} \cup F_{iE}) \qquad \text{Equation 4-30}$$

where ($\cup$) is the union of the failure events and i= 1 (serviceability), i=2 (safety), i=3 (compatibility), and i=4 (durability). The probability of failure of any one of the quality attributes (i) due to Intrinsic Factors is P(FiI). The probability of failure of any one of the quality attributes (i) due to Extrinsic Factors is P(FiE).

The probability of important Extrinsic Factors developing in a quality attribute (i) in the system is P(Ei); the probability of no important Extrinsic Factors is $P(\bar{E})$ and $P(\bar{E}) = 1 - P(E)$. Then

$$P(F_i) = P(F_{iI} \mid E_i)\, P(E_i) + P(F_{iI} \mid \bar{E}_i)\, P(\bar{E}_i) + P(F_{iE} \mid E_i)\, P(E_i) \qquad \text{Equation 4-31}$$

The first term addresses the likelihood of system failure due to Intrinsic Factors given Extrinsic Factors (e.g. system fails in a storm due to inadequately maintained drainage system). The second term addresses the same likelihood given no Extrinsic Factors. This is the term normally included in traditional reliability analyses (Bea 1999). The third term addresses the likelihood of system failure directly due to Extrinsic Factors (e.g. system fails in a fire due to arson). One would expect that normally the first and third term would comprise about 80% of the total probability of failure and that the traditional second term would comprise about 20% of the total probability of failure. These three terms contain important clues to how engineers can approach development of adequate and acceptable quality and reliability in their systems.

The first clue pertains to the two conditional probabilities of failure (one due to Intrinsic Factors, one due to Extrinsic Factors) given Extrinsic Factors: $P(F_I \mid E)$ and $P(F_E \mid E)$. These two probabilities of failure characterize the 'robustness' or defect and damage tolerance of the engineered system (structure, hardware); one is the probability of failure due to Type I and Type II uncertainties (Inherent) and the other is the probability of failure due to Type III and Type IV uncertainties with both probabilities conditional on the occurrence of a significant or major damage or defect due to Type III and Type IV uncertainties.

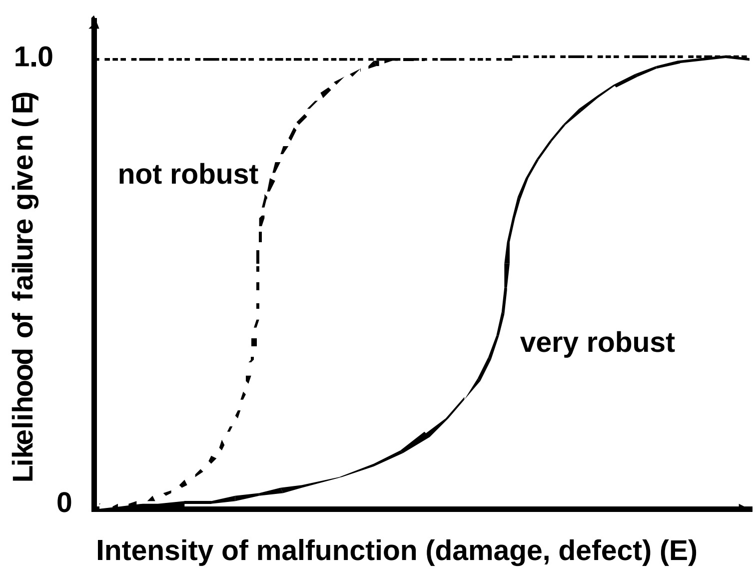

Figure 4.1- Robustness in Systems as function of extent of damage and / or defects

The shape of the system's 'fragility curve' (Fig. 2.1) can be controlled by engineering. *This is explicit design for robustness or defect (error) tolerance and fail-safe or intrinsically safe design.* For the intensities (magnitude) and types of intrinsic and extrinsic demands that can be expected, the system should be configured and designed so that it does not fail catastrophically (or have unacceptable quality) when these types and magnitudes of demands occur. The fragility curve for a particular system can be determined using a combination of results from analytical models and experimental (field, laboratory) observations. Development of desirable and acceptable robustness in engineered systems will be addressed in this development.

The second clue regards the probability of failure due to Intrinsic Factors given no Extrinsic Factors. This is the term traditionally addressed by reliability analyses. This is the term addressed by most design formats (e.g. Working Stress or Factor-of-Safety Design). 'Margins of safety' will used in this development to address natural and modeling sources of uncertainty (first two types noted earlier). Specified 'quality assurance and control' (QA / QC) measures and their associated procedures are used to exclude or properly manage Extrinsic Factors (unanticipated and undesirable compromises in 'specified' conditions and characteristics).

The third clue regards the probability of experiencing the Extrinsic Factors. It is here that direct engineering controls and management efforts are needed to help assure that things go properly. The associated measures are directed at limiting unintended and undesirable developments associated with human, organizational, and knowledge related factors (Type III and Type IV uncertainties). A subsequent course and development will address the Extrinsic Factors.

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Chapter 5

Quality Goals

5.1 Reliability, Consequences, & Risks

The ‘expected’ risk (E[Ct]) associated with an unexpected and undesirable (failure) event is conventionally defined as the product of the likelihood of the event and the consequences associated with the event. In the case of a failure (f) and the expected (average, best-estimate) consequences or ‘costs’ associated with that failure (E[Cf]):

$$E[Ct] = E[Pf]\,E[Cf] \qquad \text{Equation 5-1}$$

Risk has multidimensional properties and characteristics. These include the maximum possible losses and gains and the relationships of these maximum possible losses and gains to the investment capacity and to the economic ‘goals.’ Costs can be expressed with a wide variety of metrics. Multi-attribute decision analysis methods can be used to combine these metrics into a single ‘utility’ measure. These methods can reflect the decision makers relative preferences for various types of outcomes and for various magnitudes of these outcomes.

Based on results of a survey of more than 600 executives of major business and detailed interviews of another 50 top executives, Shapira (1995) advanced a model of risk taking that is summarized in Figure 5-1. The vertical axis describes risk taking in terms of the uncertainties associated with the proposed venture or alternative. The horizontal axis shows the risk taker’s cumulative resources.

Two reference points are considered: an ‘aspiration’ (desired) amount for resources that adapts to experience, and fixed survival amount at which resources are exhausted. Two risk taking focuses are identified: aspiration and survival.

Risk taking is driven by two decision rules. The first rule is used whenever cumulative resources are above the aspiration amount. Uncertainty is allowed so that the risk taken increases monotonically with distance above the aspiration amount. The probability of undertaking a venture that develops a result placing the decision maker below the aspiration amount is set to some relatively low number.

The second rule is used when cumulative resources are below the aspiration amount. Uncertainty is allowed so that the risk taken increases monotonically with negative distance from the aspiration amount. This rule provides for risk seeking for losses. The farther current resources are below the aspiration amount, the greater the risk required to make recovery likely.

Risk can be varied in two ways. The first is by choosing among alternatives with varying probabilities. The second is by altering the scale of the investment in the chosen alternative; this is equivalent to changing the bet amount. this second alternative depends on the resources available and there is a constraint on risk taking that can be quite severe as resources are exhausted.

The aspiration level can shift as a function of experience and other 'influences.' Consider an organization whose resource position is X_1 in Figure 5-1. Given that the organization is focused on the aspiration level AL_1 and takes a risk that leads to a successful outcome that had resulted in a new resource position marked X_2 . The organization is now above the original aspiration level, and if the aspiration level stays where it was, the next risk taking action is going to be much more modest. However, if based on the recent success, the organization would shift the aspiration level to a new point AL_2, the next attempt at risk taking would be more modest than had the aspiration level not changed.

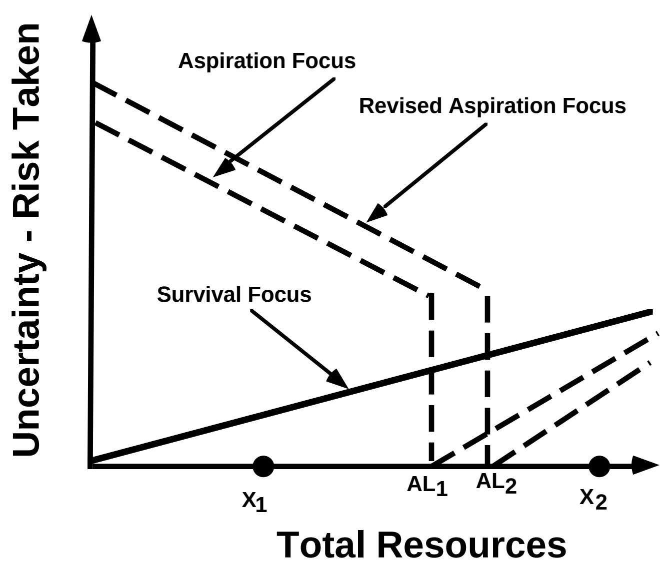

Figure 5.1 - Shapira's risk taking model

Target reliabilities or 'reliability goals' are those that can be used to engineer and manage the systems during their life. There are a variety of complimentary ways to help owners and operators (including regulators) arrive at decisions concerning these target reliabilities. Such decisions must be made by those that bear the primary responsibilities for the activities associated with the systems and for the consequences associated with those activities; generally, these people are not those that engineer a particular system during or for a particular part of its life cycle. The system engineers can provide information and insights concerning the tradeoffs between reliability and 'costs'. The system managers (corporate and public regulatory) must provide the decisions regarding what target reliabilities will be used to engineer the facilities (Wenk, 1986). Engineers can and should have important inputs to the decision process used to define target reliabilities; engineers can provide some of the analytical (thinking) models and guidelines that can be used in such decision processes.

The approaches that will be developed here include economics and utility based approaches, approaches based on historic or actuarial data for similar systems, and approaches based on present-day standards of practice. Each of these approaches has its advantages and disadvantages (limitations). Each of these approaches should be used in a complimentary and integrated manner to define the resultant target reliability or reliability goals for a particular engineered system.

5.2 Economics Approaches

The profit (Pr) associated with a system during its life-cycle (t = 0 to L) can be expressed as:

$$Pr = \Sigma\, Qi\, (Pi - Ci) \qquad \text{Equation 5-2}$$

where the summation is over the life-cycle (time i = 0 to L), Qi is the quantity or productivity units during a time period ' i', Pi is the price obtained for the units, and Ci is the costs required to produce the units. All price units must be present valued since they occur at different times during the life of the system.

Adequate profitability in an enterprise is essential to provide the resources required to achieve acceptable and desirable reliability and the associated levels of quality. There is an investment in reliability that can provide returns that lead to increases in profitability (Figure 5-2); this is 'good business'. However, when the investments in reliability no longer lead to increases in profitability (benefits do not offset the investments) this is 'bad business'. And, when the investments are made to the point that a negative profitability results, this is 'going out of business.'

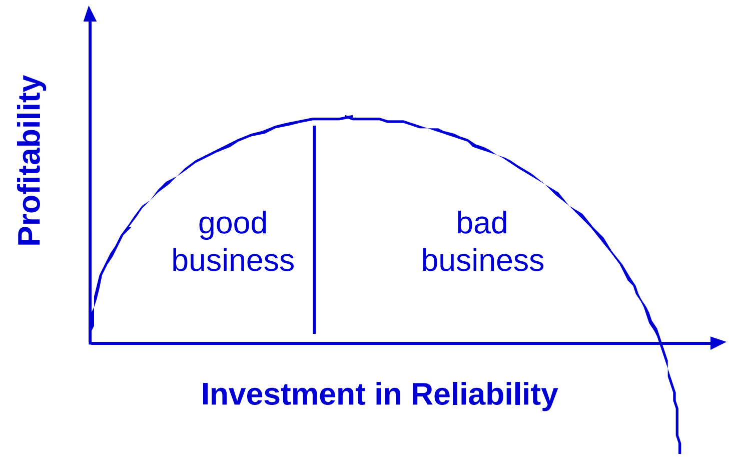

Figure 5.2- Profitability and the business of investments in reliability

Quality and reliability have influences on both the productivity units and on the costs of these units. Quality and reliability can also have influences on the price obtained for the units. The effects of compromises in quality and reliability can be translated to the costs required to produce units in a given time period and the number of units that are produced. These effects will be referred to as the 'costs of failure' associated with each of the quality attributes (CFj).

Providing quality in the design, construction, maintenance, and operation of a system can result in lower life-cycle costs, be safer, and minimize unrealized expectations. Quality can result in significant benefits. Achieving adequate levels of quality and reliability is not quick, easy, or free. It can be costly in terms of the initial investments of manpower, time and other resources required to achieve it (Fig. 5.3). But, if it is developed and maintained, it can result in significant savings in future costs. In addition, initial costs can be reduced by discarding ineffective and inefficient programs that are currently in use. A basic objective is to find ways to reduce both initial and future costs and thereby provide both a short-term and long-term financial incentive to implement improved quality and reliability programs. An objective is to find the level or degree of quality that

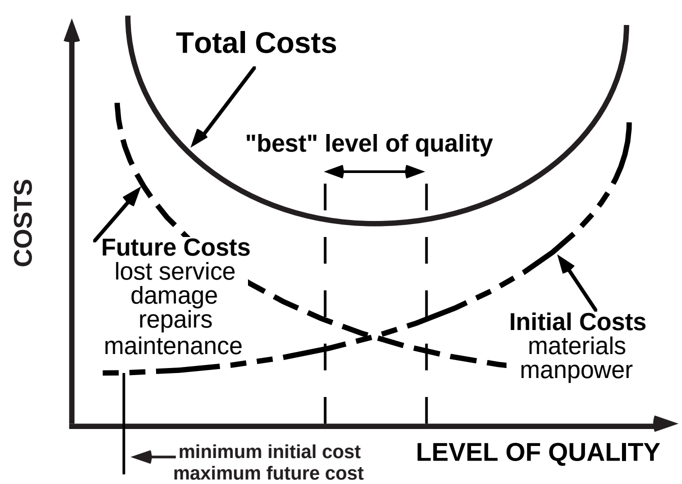

Figure 5.3 - Consideration of initial and future costs associated with various levels of quality

will minimize the total of initial and future costs.

A risk based approach could be based on a the total life cycle risks (Rt) associated with a system:

$$Rt = Rti + Rtf \qquad \text{Equation 5-3}$$

Rti are the initial risks associated with design, construction, and commissioning the system. Rtf are the future risks (present valued) associated with the operation of the system. These risks recognize both the likelihoods of realizing failures or compromises in quality (Pfj) and the consequences or 'costs' associated with these failures (CFj). The objective would be to define the system that could develop the minimum total risk during the life of the system.

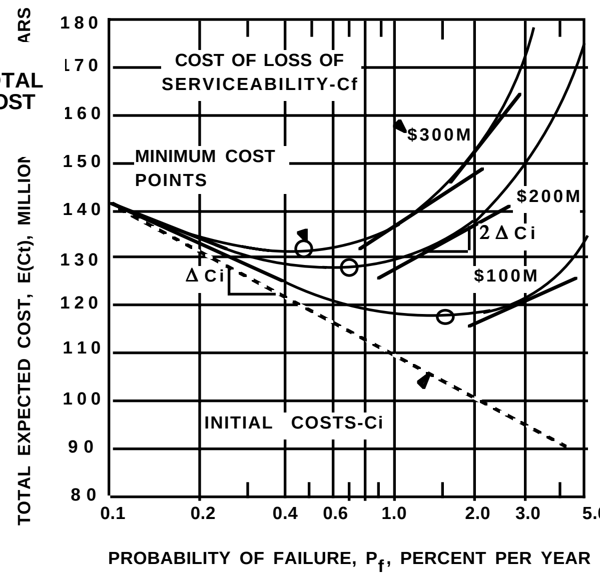

Figure 5.4- Example total cost optimization

An example application for the quality attribute of 'safety' could be developed as follows. Let the expected probability of failure to achieve adequate safety be $\overline{Pfj} = \overline{Pf2}$. This probability will be expressed on an annual basis (probability of failure per year of exposure).

Let the expected operating costs associated with the inadequate safety be $\overline{Cf2}$. Then $\overline{Rf2} = \overline{Pf2} \bullet \overline{CF2}$. Let the expected initial risks associated with varying Pf2 be a linear function of the logarithm of Pf2 that has a slope ΔCi (the cost required to change Pf2 by a factor of 10) (e.g. Figure. 5.4). The equation for the total risk (initial plus operating) could be differentiated and equated to zero to determine the minimum total risk and thus define the Pf2 that would develop the minimum total risk

$$Pf2o = 0.4348 / Rc\ (pvf) \qquad \text{Equation 5-4}$$

Rc is a ratio of the costs associated with the safety failure (CF2) to the cost required to reduce the Pf2 by a factor of 10 (ΔCi). This a non-dimensional measure of the initial and operating costs.

The pvf is a Present Value Function that is used to bring to present value terms the future costs associated with the compromises in safety. In this example, the pvf is based on replacement of the system in the case of the compromises in safety, a continuous discounting function, and a time-invariant annual Pf:

$$pvf = [1-(1+r)-L]/r \quad \text{Equation 5-5}$$

where r is the net discount rate (investment rate minus inflation rate) and L is the life or exposure of the system in years. As approximations, for long life systems (L ≥ 20 years), pvf ≈ r^{-1}; for short life systems (L≤ 5 years), pvf = L. The product of Rc times pvf is an expression of the 'effective life' of the system.

Another 'boundary' on the safety risk could be defined by equating the slope of the total cost expression to unity. It is at this point that there is a rapid rise in the total expected cost with the change in Pf2. In this case the expression becomes Pf2m = 2 Pf2o, or the 'marginal' Pf is twice the 'optimum' Pf. The results are shown in Figure 5.5.

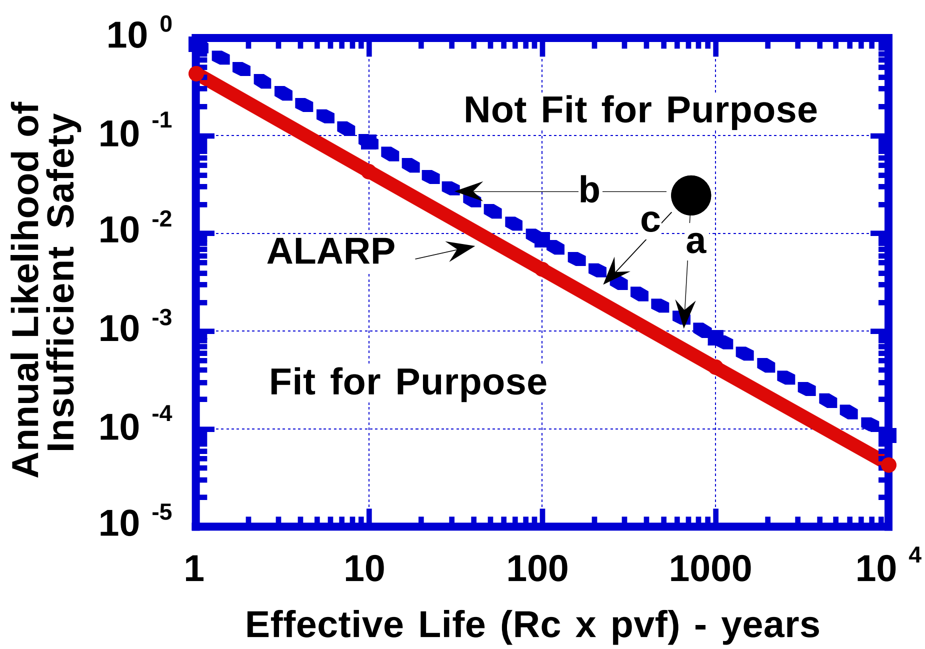

Figure 5.5– Fitness for purpose reliability evaluation guideline

The line indicated as ALARP is taken to be 'As Low As Reasonably Practicable' (resulting in the minimum total cost). The dashed line indicates the marginal Pf. The combination of Pf and the measure of the initial and operating costs that lies above the dashed line is indicated as 'not fit for purpose'. The combination of Pf and the measure of initial and operating costs that lies below the solid line is indicated as 'fit for purpose'. The area between the two lines can be interpreted as the area that is subject to economic cost-benefit analyses to define an acceptable combination of likelihoods of failure and the consequences associated with the failure.

In the economics or utility based approach considerations of the broad category of potential 'consequences' associated with a failure is important. It is at this point that the very sensitive and difficult issues associated with potential environmental impacts must be addressed – including potential human impacts (injuries, fatalities). There are two basic approaches to address such impacts. The first is to address the potential impacts as a separate issue, not integrating the potential environmental injury (including human life) impacts with the other potential impacts (e.g. property losses, production and productivity losses). The second is to address the potential environmental impacts by integrating them with the same metrics to evaluate the other potential consequences.

It is at this point that the question is often raised: what is the value of a human life that should be included in the economics based approach? The reasonable way to pose the question is how much should be invested to save a human life in association with the proposed system operations? Terms like ICAF (Incremental Cost of Averting a Fatality) have been developed to help answer such questions. Analyses of the ICAF implicitly integrated into current design guidelines for natural hazards (e.g. storms, earthquakes, North American, European) indicates ICAFs in the range of U.S.$1 to $25 millions (2000). Recent studies of societal ICAFs indicate values in the range of U.S. $1 millions for developed countries like the U.S. and Norway. For developing countries like Mexico, Brazil, China, and India the ICAFs are in the range of U.S. $0.1 to 0.3 millions.

It must be remembered that the objective of the entire Target Reliability process is an attempt to identify the 'best' alternative for design and operation of a system. 'Best' means to identify that alternative (or alternatives) that can provide the best chance to realize a system that has desirable and acceptable quality and reliability during its life cycle. Once such a system has been identified (together with the associated risk assessment and management processes), then the objective shifts to implementing the risk assessment and management process during the system life cycle that can result in a system with 'zero failures' – the objective is desirable and acceptable quality in the context of resources that should be expended to achieve such goals.

5.3 Historic 'Actuarial Data' Based Approaches

A second approach is based on experience with engineered systems in which actuarial or historic data is used to identify historic precedents associated with other engineered systems. The premise of this approach is that societies and the organizations that are parts of these societies through time and experience arrive at judgments concerning what is acceptable and what is not acceptable in terms of risks.

An expression of the historic approach is given in Fig. 5.6. This expression is based on historic annual probabilities of failure (high consequence events) and the consequences associated with the failures. The probabilities of failure are actuarial in the sense that they are based on the statistics associated with the failures of systems in the past. They are not notional in the sense that they are analytically derived or computed. The consequences are expressed in terms of U.S. dollars (1984) and deaths. Note that the expression indicates that a statistical death equates to about U.S. $1 millions.

Are there problems with the historic approach? Certainly there are! Risks associated with systems change with time. Statistically based risks involve 'mixed populations' of systems that are different in their details. The historic failures involve a wide variety of 'causes' that range from natural (caused by 'acts of God') to unnatural (caused by acts of 'man'). Many failures are never reported.

In Figure 5.6 note that fixed drilling rigs have a Pf ≈ 1 E-3 per year. This is due to all causes. Further examination of the causes indicates that about 20% can be attributed to 'natural' causes; the rest are due to 'accidents' (e.g. blowouts, collisions, fires, explosions). Thus, Pf for natural hazards would be about 2E-4 per year. If there were a balance between natural and accidental hazards, then the Pf for natural hazards would be about 5E-4 per year.

Another challenge associated with this approach regards the uncertainties and variabilities that are incorporated into this actuarial data based approach; there are no Type II uncertainties. Modeling variabilities are not present in actual tests (the median or central tendency values are!). Thus, when this approach is used, the information must first be carefully evaluated to eliminate Type III and Type IV uncertainties (those due to human and organizational factors and the associated information - understanding developments), and then it must be realized that the remainder represents fundamentally Type I uncertainties (due to natural variabilities). It is important there is a consistent treatment of the reliability targets with the analytical processes that are used to help demonstrate that these targets can be achieved. It is for this reason, that some reliability analysists favor not including the additional Type II uncertainties – but, they do favor including the necessary corrections to the predictive analytical models to assure that they develop realistic results; this means that the Bias central tendency corrections must be included.

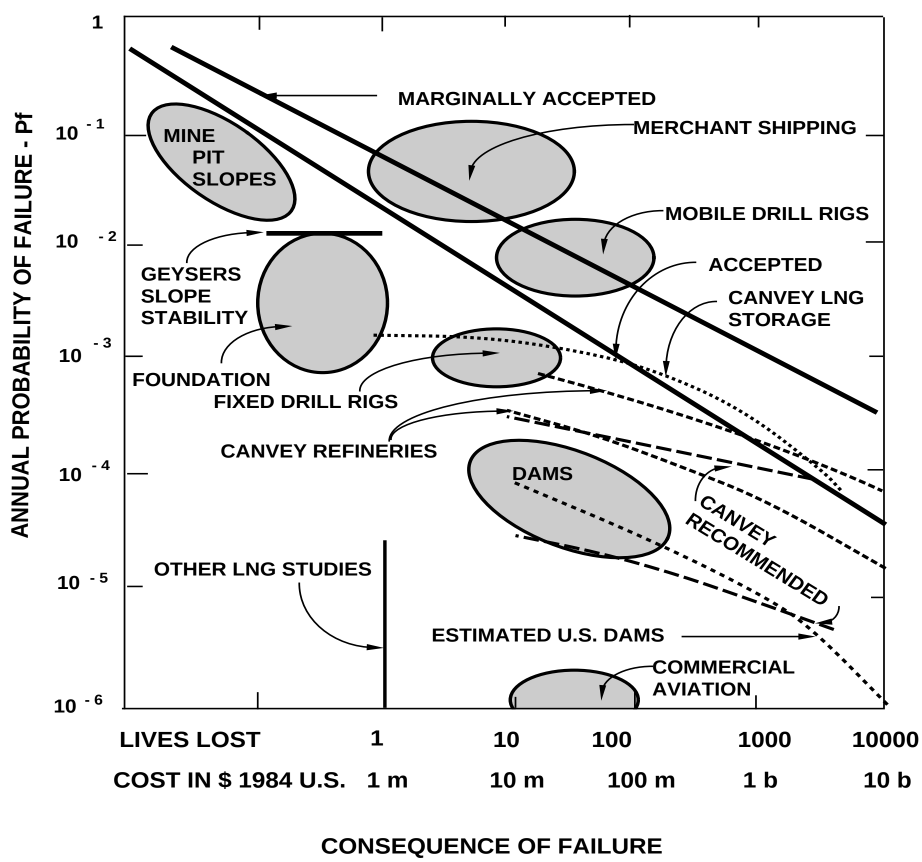

Figure 5.6- Historic risk tradeoffs

Typical collapse failure rates for engineered public structures like domestic housing, commercial office buildings, and bridges in the U.S., Canada, western Europe have been estimated to be in the range of 2E-5 per year.

Another expression of historic risk tradeoffs is that of the FAR (Fatal Accident Rate). The FAR is the number of fatalities per 10E8 hours of exposure to given activities in the U.K (1980-1990) (Fig. 5.7). Commercial industrial activity FARs range from 4 (chemical processing) to as high as 250 (commercial aviation). The commercial aviation FAR increases by factors of up to 10 at different locations around the world (Africa and China have the highest FARs).

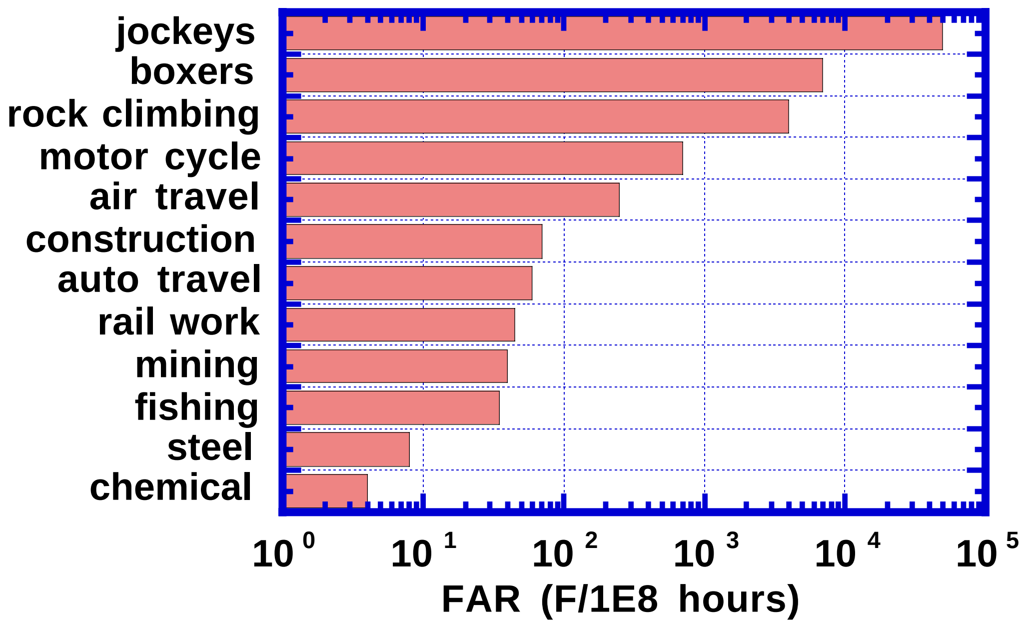

Figure 5.7 - General activity fatal accident rates

Figure 5.8 summarizes the FAR for onshore and offshore exploration and production (E&P) workers worldwide during the period 1988 – 1992 (about 75% of the hours were onshore). The E&P FAR range from a low of about 3 to a high of about 40. Operations in the North Sea area and U.S. have the lowest FAR (average of 5 to 6). Operations in South America and Africa have the highest FAR (average of 20 to 25).

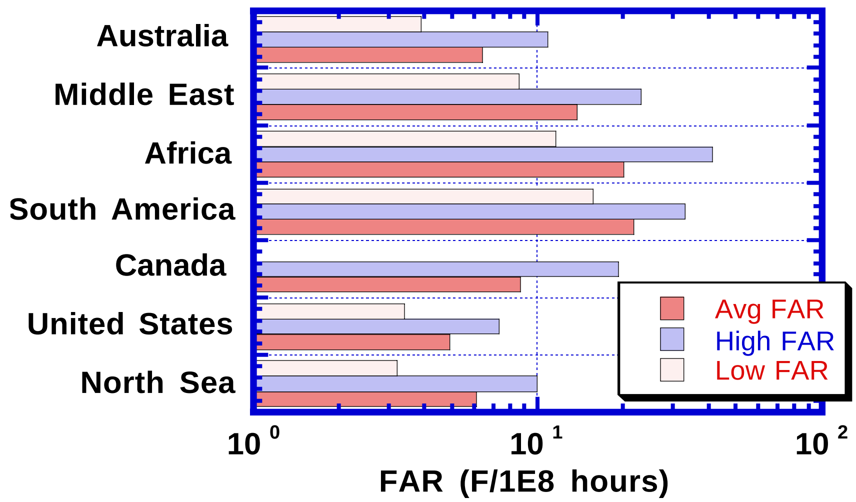

Figure 5.8 - Worldwide exploration and production fatal accident rates 1988 - 1992

5.4 Standards of Practice Approach

A third approach is that of the 'standards-of-practice.' Standards-of-practice are represented in the design codes and guidelines that are utilized by organizations and industries. Standards-of-practice are also included in the decisions that are made by system owners / operators and the associated regulators to design, requalify, and operate such systems.

Codes and guidelines can be analyzed using reliability based models to determine the reliabilities inherent in the codes and guidelines. The American Society of Civil Engineers (ASCE) Standard "Minimum Design Loads for Buildings and Other Structures" defines four categories of buildings and other structures that represent increasing hazards to human life in the event of failure (Table 5.1).

For example, for wind loadings, Category II facilities have an importance factor, I, of 1.0. The design wind loading are associated with an annual probability of 0.02 or an average return period of 50 years (98 %tile, 2.05 standard deviations from the median). Given an uncertainty in the annual maximum wind loadings of: σlnV. = 0.72 ln (V 10,000 yr / V 100 yr) = 0.72 ln (250 mph / 125 mph) = 0.50. The wind force varies as a function of the square of the wind speed, thus the uncertainty in the wind force could be estimated as: σlnD. = 2 (0.5) = 1.0. The ratio of the design wind force to the median annual maximum wind force would be: BD50 = exp (2.05 x 1.0) = 7.8. Given a design factor of safety of FS = 2.0, a capacity median bias of BC50 = 1.5, a design force median bias of BD50 = 0.67, and an uncertainty in the structure capacity of σlnC. = 0.25, the annual Safety Index could be determined from: $\beta = \ln (7.8 \times 2.0 \times 1.5 \times 1.5) / 1.03 = 3.5$, or an annual probability of failure of Pf ≈ 2.3 E-4. The average return period of the wind force that would bring the structure to its ultimate limit state would be approximately 5,000 years. Category III and IV structures would be designed to have greater capacities and reliabilities.

Table 5.1– Classification of buildings and other structures for definition of design loadings

Occupancy	Category
Low hazard to human life (agriculture facilities, temporary and storage facilities)	I
Structures except Categories I, III, and IV	II
Substantial hazard to human life (more than 300 occupancy buildings, more than 250 occupancy schools, more than 500 occupancy adult education facilities, more than 50 occupancy health care facilities)	III
Essential structures (hospitals, emergency facilities, power facilities)	IV

The ASCE Standard specifies a two-level design for important facilities. The strength level earthquake is specified to have an average return period of 475 years. The maximum capable earthquake is specified to have an average return period of 1000 years. The later specification would imply a probability of failure of the order of 1E-4 per year. These results are consistent with the guidelines issued by the U.S. Department of Energy for natural phenomena hazards and the onset of significant damage. For facilities in which there are concerns for occupant safety and continued operation, the annual probabilities of ‘failure’ (onset of significant damage) are in the range of 1E-4 to 5E-4 per year.

A standard-of-practice approach for design and requalification of offshore structures has been developed to avoid some of the problems associated with the economics and historic data based approaches. An expression of the standard-of-practice approach is given in Figure 5.9. The probabilities of failure are computed or notional and include only Type I or 'natural' (inherent randomness). The probabilities of failure are the total Pf and include both intrinsic (environmental, natural) and extrinsic (human, organizational) caused failures. The consequences of failure include the best estimates of economic costs associated with loss of property, injuries, restoration, productivity, and resources. The data points shown are for new and existing platforms that have been evaluated to determine their risks. The two lines labeled acceptable and marginal are those placed by the author based on the decision making processes that have accepted or rejected the assessed risks.

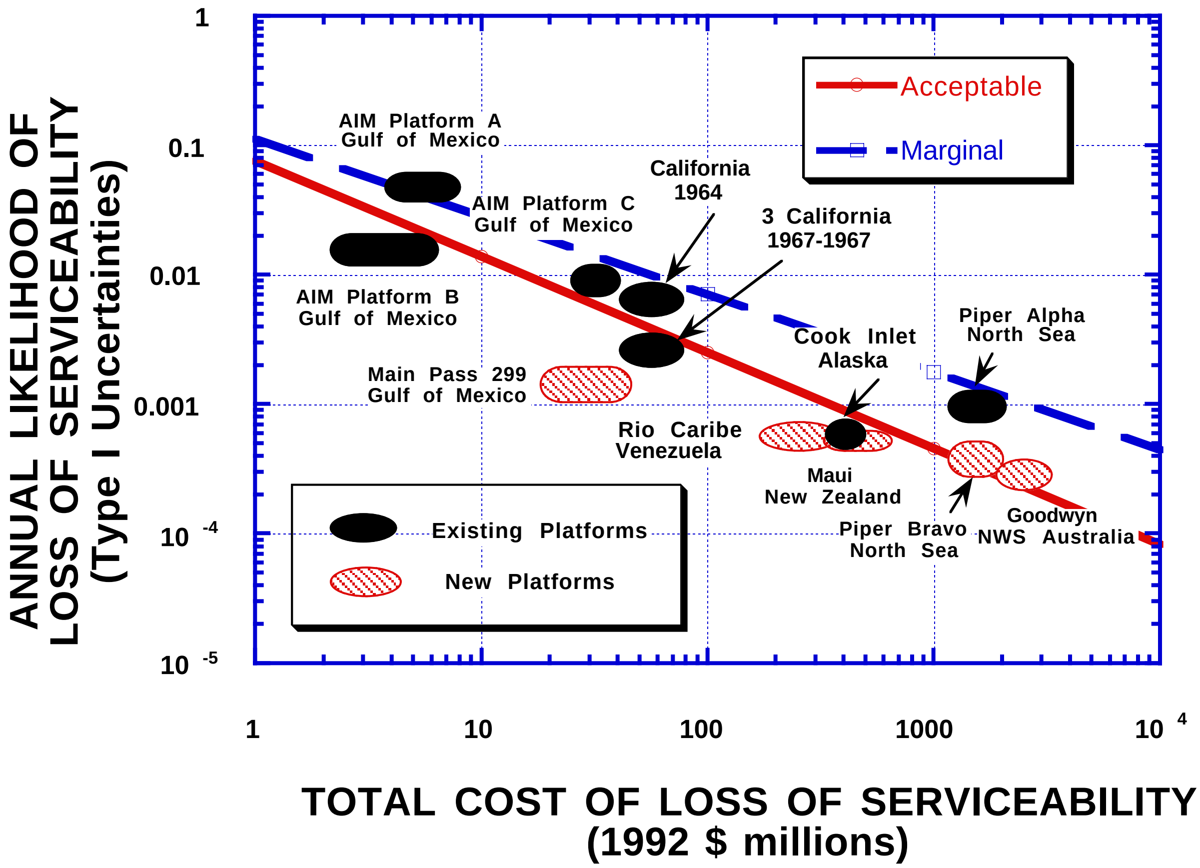

Figure 5.9 - Standard-of-Practice approach

5.5 Insights

A summary of the advantages and disadvantages of the three approaches that have been discussed are given in Table 5.2.

Table 5.2 - Advantages and disadvantages of different approaches to define target reliabilities

Approach	Advantages	Disadvantages
Economics	• explicit treatment of initial and future costs • explicit inclusion of probabilities of throughout life cycle initial and future costs • is able to include future profit effects (negative future costs) • explicit inclusion of the effects of inflation and value of investments • explicit inclusion of exposure time or expected life of the engineered system	• difficulties of developing realistic initial costs • difficulties of developing realistic future costs • difficulties of assessing all categories of future costs (operations, restoration, replacement, productivity, on-site and off-site, human and other environmental injuries and fatalities) in common economic terms • difficulties of developing realistic economic factors (e.g. present valuing functions) • difficulties of developing realistic probabilities associated with initial and future costs
Historic	• recorded experiences used to indicate acceptable & tolerable likelihood of failures / accidents • recorded experiences used to indicated ranges of consequences associated with failures / accidents • multiple categories of consequences included	• accuracy of recorded experiences - not all experiences • accuracy's of recorded experiences - not all consequences • mixed populations - different types of engineered systems with different design, construction, operations, and maintenance histories • time effects in recorded experiences: past reliabilities $\neq$ future reliabilities • Type 2 - model uncertainties not included

Standards of Practice	• represent present decisions concerning tradeoffs of reliabilities and consequences • public consensus based	• preferences of different societies not included • time lags between recorded decisions and applications of implications of recorded decisions • all types of uncertainties not included • not available for all types of engineered systems • not available for new and innovative engineered systems

There is no perfect approach to solve the difficult problem of determining what is an acceptable and desirable risk of associated with an engineered system. One of the most important elements of the question of "how safe is safe enough?" is the process of answering that question. The process of looking in a fundamental and deliberate manner at what constitutes an acceptable and desirable risk is perhaps the most valuable part of developing an answer to this question. The process must include effective communications of the risks that are to be taken so that understanding and agreement are developed.

A fundamental objective should be to optimize the use of economic resources while preserving 'reasonable' protection for life, property, and the environment. Consider three alternative systems. One is designed, constructed, operated, and decommissioned in a 'perfect' manner. This system results in the highest possible net life-cycle income for the venture. The second system is designed, constructed, operated, and decommissioned in a 'minimum compliance' manner. While this system has a lower initial cost, its future costs bring its life-cycle benefits to a very low value. The search is for the combination of design, construction, operation and decommissioning strategies and activities that will bring the system to have the highest possible benefits. Perfection is not possible. Something close to it is possible and this is the challenge faced by engineers and managers in today's business environment.

Systems that are indicated as not fit for purpose can be modified to be fit for purpose by using three basic strategies:

- Reduce Pf – reduce the likelihoods of failures,

- Reduce the consequences of failures - decrease CF, decrease pvf,
- Combinations of reducing Pf, CF, and pvf.

Additional strategies for risk management include:

- Avoiding the hazards
- Transferring all or a portion of the risks (e.g. insurance, project partners).

A risk management system should be practical, realistic, and be cost and benefit effective. Risk management systems need not be complicated. Excellent risk management results from a combination of common sense, qualified experience and judgment, knowledge, intuition, wisdom, and integrity. Mostly an excellent risk management system is a willingness to operate in a caring and disciplined manner in approaching the critical features of any activity in which risk can be generated. Risk management is largely a problem of doing what we know we should do and not doing what we know we should not do.

The purpose of a risk management system should be to enable and empower those that have direct and daily responsibilities for the quality and reliability of a system during its life cycle. The engineers can play vital roles in this empowerment. If technology is not used wisely, scarce resources and attention can be diverted from the true factors that determine quality and reliability. The purpose of a risk management system should be to assist the ‘front line operators’ to take the right (sensible, appropriate) risks and to achieve acceptable quality and reliability. To try to completely eliminate risk is futile. To help identify and manage risks and make appropriate use of technology to accomplish these objectives should be one of the key objectives of a risk management system.

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Chapter 6

Margins of Quality

6.1 Quality Formats

There are three formats presently in use in design and requalification of engineered systems: Working Stress Design (WSD), Load and Resistance Factor Design (LRFD), and Limit States Design (LSD) (Figure 6.1). In this illustration, the imposed or induced demand on or in the system is shown as 'load' (vertical axis). The maximum capacity that can be mobilized by the system to resist this demand is shown as the 'capacity'. The response of the system to that demand (or combination of demands) is shown as 'displacement' (horizontal axis). This illustration could be applied to either an element or component that comprises a system or to the entire assembly of elements and components; the system.

For many years, WSD has been the primary format used in the U.S. LRFD saw its initial development in Europe, but in the last few years this format has found its way into U.S. design codes and guidelines. LSD saw its initial development in Canada following the failure of the Quebec Bridge. However, in various forms this format has found its way into parts of the other two developments. So, today many engineering codes incorporate all three formats in one form or another.

At the outset, it is important to recognize that these are different ways that engineers can use to ‘design’ their systems. They can lead to exactly the same results if developed consistently. However, currently, often they do not, and that is because they have not been developed consistently. Generally, they have been ‘calibrated’ to produce results that are desired – generally consistent with favorable experience. Sometimes, there can be differences introduced that are intended to make one format more favorable than another – generally the differences are introduced to result in 'lighter' or 'less expensive' structures – this is done to provide an incentive for the engineer to use a certain format or the material associated with a particular format or design - requalification code or guideline.

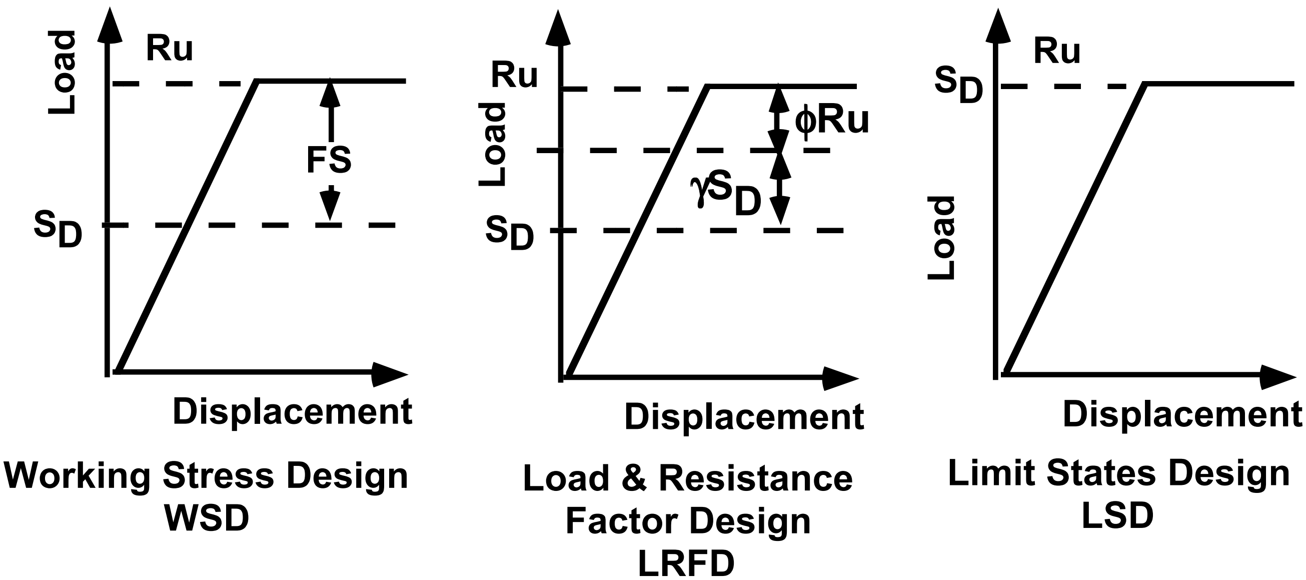

Figure 6.1: Three margins of quality alternative formats

In the case of WSD, a Factor of Safety is introduced so that the capacity of the system is higher than the designated design demand. Often, the design demand is designated at the '100-year' or similar 'extreme' value.

In the case of LRFD, there are two factors that are utilized; one the load factor is generally (not always) greater than unity and is used to factor up or increase the designated design loading; the other is a resistance factor that is used to factor down or decrease the designated capacity.

Both formats rely on linear elastic system analyses. This is a very important constraint. The reason for this constraint is because of the practicality and widespread use of linear elastic analyses by engineers. An alternative, nonlinear inelastic analyses are not widely used and are very complex to perform. Another alternative, limit equilibrium plastic analyses, although easier to perform, require a number of important assumptions that must be understood and accommodated by the user, and hence require engineers that have expertise in this type of analysis. These simplier analyses also do not directly provide details on the demand response (e.g. displacement) characteristics of the system.

For LSD, there is no factor of safety and there are no loading or resistance factors. The design loading is chosen to be at a probability level (average return period) that is close to the level at which the system is expected to fail – often in the range of 10,000 years (contrasted with 100 years as compared with loadings used in WSD and LRFD). Perhaps the most important aspect is that the system analyses must be performed at the ultimate limit state condition – nonlinear inelastic structural analyses or limit equilibrium plastic analyses. Because of the difficulties of performing such analyses, sometimes mixtures of LSD and WSD or LSD and LRFD are found so that the ease of linear elastic structural analyses can be preserved.

Most engineering codes and guidelines have been developed to give engineers guidance on how to proportion the elements that comprise systems. Most engineering codes and guidelines do not address the characteristics of the assembly of elements and components that comprise the system. However, some codes and guidelines have now begun to address the performance characteristics of the assembly of elements - the system. For example, the desired margins of quality of design (or requalification) of individual elements can be prescribed with factors of safety. The desired margins of quality of design of the assembly of the elements can be prescribed with a comparable index that has frequently been termed the Reserve Strength Ratio (RSR). The RSR is the ratio of the system demand maximum capacity to the system design (or reference) demand. Thus, the RSR is directly comparable to the factor of safety; one refers to the performance characteristics of the system, the other to the performance characteristics of elements that comprise the system.

Recently, Performance Based Design (PBD) has been proposed for design and requalification of some 'special' systems often located in 'special' environments. The objective of this format is to give the engineer guidance on what is expected in the way of performance of the system for all of the important performance states. These performance states frequently have been divided into two categories: 1) serviceability, and 2) ultimate limit states. The serviceability limit states (SLS) are intended to address normal performance characteristics (e.g. limiting displacements while in operation, no significant damage, etc.). The ultimate limit states (ULS) are intended to address abnormal or extreme performance characteristics (e.g. no collapse, repairable damage, etc.). The input demand requirements are generally specified as are guidelines for proportioning the elements that comprise the system. The engineer must then demonstrate that the particular system configuration that has been designed can meet the SLS and ULS. This requires that the engineer be able to perform the system analyses in both domains of performance.

6.2 Working Stress Design

WSD often utilizes nominal 'static' demands to define the serviceability response characteristics and performance of the structure. If dynamic loadings or dynamic loading effects are present, then dynamic load factors (DLF) can be introduced to help characterize these demand components. Linear elastic analyses are used to describe the system response characteristics for the given nominal design loadings. Based on characterization of the demands and capacities as being Lognormally distributed, the traditional factor-of-safety (FS) in working stress design can be developed directly from the fundamental equation for the Safety Index as:

$$FS = Fe_{50} \left(B_{S50} / B_{R50} \right) \exp \left[\left(\beta \, \sigma_{\ln SR} \right) - \left(2.33 \, \sigma_{\ln S} \right) \right] \qquad \text{Equation 6-1}$$

where Fe_{50} is a factor (median loading effect) that incorporates the interactive effects of dynamic - transient loadings and the nonlinear behavior of the system. B_{S50} is the median bias in the maximum demand (loading) on the element of concern, B_{R50} is the median bias in the capacity of the element, β is the 'target' (desired, acceptable) annual Safety Index for the element, $\sigma_{\ln SR}$ is the total uncertainty in the demands and capacities (standard deviation of the logarithms), and $\sigma_{\ln S}$ is the uncertainty in the annual expected maximum loadings. σ_{lnS} is the standard deviation of the logarithms in the expected annual maximum loadings imposed on and induced in the system.

The 2.33 refers to 2.33 standard deviations from the mean value, or the 99th percentile. This is equivalent to the reference of the design loading to an average annual return period of 100 years. In the case of seismic loadings, a 200-year return period is often used (99.5 percentile) and a value of 2.57 would be used in the foregoing expression.

The total uncertainty in the demands S and capacities R for independent demands and capacities can be determined from:

$$\sigma_{SR} = \sqrt{\sigma_S^2 + \sigma_R^2} \qquad \text{Equation 6-2}$$

where σ_S is the uncertainty (standard deviation of the logarithms) in the annual maximum demands and σ_R is the uncertainty in the capacities of the elements.

The FS is the ratio of the design capacity of a system element (RD) to the design or reference demand (S_D):

$$FS = R_D / S_D \qquad \text{Equation 6-3}$$

In the WSD format, the design equation is formulated as:

$$R_D \geq FS\ S_D \qquad \text{Equation 6-4}$$

or:

$$R_D / FS \geq S_D \qquad \text{Equation 6-5}$$

Alternatively, the factor of safety could be referenced to the load capacity of an entire system and thus reflect the aggregated effects of the elements that comprise the system. The Reserve Strength Ratio (RSR) is now generally used for this purpose. The reliability based expression for the RSR is:

$$RSR = Fe_{50}\ (B_{S50} / B_{R50})\ \exp\left[(\beta\ \sigma_{lnSR}) - (2.33\ \sigma_{lnS})\right] \qquad \text{Equation 6-6}$$

where all of the parameters refer to the global demands and capacities developed on and in the structural system. As before, in this expression the design loading S_D is defined at a 100-year return period.

The transient / dynamic loading - nonlinear performance factor, Fe, is dependent on the ductility (strain - deformation capacity), residual strength (load - stress capacity beyond yield), and hysteretic (cyclic load - deformation - damping behavior) characteristics of the system. It is also dependent on the transient / dynamic loading characteristics including the duration of the imposed or induced loadings, the periodicity, and the force-time characteristics of the loadings. Additional background on the loading factor will be developed later in this Chapter. The important thing is to recognize that there can be very important differences between loadings or demands that are determined on the basis of static analyses of idealized elastic systems subject to non-time varying demands or loadings. Dynamics (time changing demands) acting on nonlinear inelastic systems (operating at or close to their Ultimate Limit States) can change the picture considerably!

Generally, the 'true' ultimate capacity of the element (Ru) is not used in the design process, and another 'nominal' or design capacity (R_D) is used. The capacity bias is introduced to recognize this difference:

$$B_R = Ru / R_D \qquad \text{Equation 6-7}$$

In a similar manner, the loading bias is introduced to recognize the difference between the design or nominal demand (S_D) and the 'true' maximum demand (S_M):

$$B_S = S_M / S_D \qquad \text{Equation 6-8}$$

The results of the foregoing developments are summarized in Figure 6.2 for biases in the demands and capacities of unity and for a 100-year design loading condition. In developing these results it has been assumed that the total uncertainty is equal to the loading uncertainty. This is equivalent to assuming that the resistance or capacity uncertainty is negligible compared with the demand uncertainties (generally, this is a very good approximation). Some interesting trends are indicated in Figure 6.2. For low Safety Indices ($\beta \leq 2.5$), the FS and RSR are about unity and there is little significant variation in these parameters with uncertainty in the demands and capacities. For high Safety Indices ($\beta \geq 3.5$), the FS and RSR are very sensitive to the uncertainties. For low uncertainties ($\sigma \leq 0.2$), there is little change in the FS and RSR as a function of the target reliability expressed in the Safety Indices.

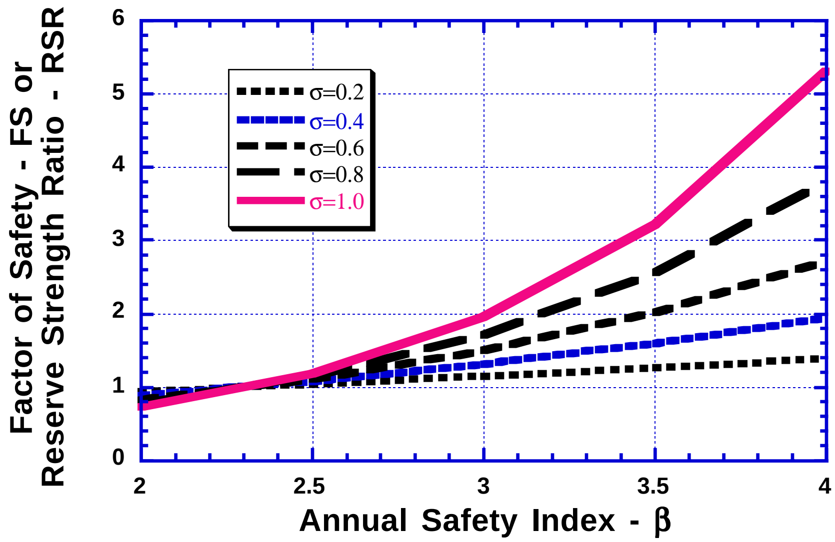

Figure 6.2- Element factor of safety or system Reserve Strength Ratio as function of total uncertainties in maximum loadings and capacities and biases of unity

6.3 Load and Resistance Factor Design

The LRFD format utilizes a load factor (γ, generally greater than unity), and a resistance factor (ϕ, generally less than unity) as follows:

$$\phi R_D > \gamma S_D \qquad \text{Equation 6-9}$$

Thus, generally (not always) the design demand S_D is factored up, and the design capacity R_D is factored down. Generally, the factoring is done such that the engineer is still able to use linear elastic analysis methods in design computations. This is a very important constraint that is placed on the formulation.

To allow the load and resistance factors to be proportioned according to the uncertainties in the loading and resistance, the following 'splitting coefficient' approximation can be used:

$$c = \frac{\sqrt{a^2 + b^2}}{a + b} \approx 0.7 \text{ to } 0.8 \qquad \text{Equation 6-10}$$

where a and b represent the uncertainties in the demands and capacities. Note that the splitting factor can be determined for individual cases (elements, systems) that depends on their respective uncertainties. A factor of c = 0.8 will be used in the subsequent developments.

Based on this approximation, the reliability approach can be used to determine the demand and capacity factors based on the Lognormal format:

$$\gamma = Fe_{50}\, B_{S50} \exp(0.8\, \beta\, \sigma_S - 2.33\, \sigma_S) \qquad \text{Equation 6-11}$$

$$\phi = B_{R50} \exp(-0.8\, \beta\, \sigma_R) \qquad \text{Equation 6-12}$$

If the design demand, SD, were composed of two components: Sd_D (for dead loading) and Ss_D (for storm loading), then:

$$\phi\, R_D > \gamma_d\, Sd_D + \gamma_s\, Ss_D \qquad \text{Equation 6-13}$$

$$\gamma_d = B_d 50 \exp(0.8\, \beta\, \varepsilon\, \sigma_d) \qquad \text{Equation 6-14}$$

$$\gamma_s = B_S 50 \exp(0.8\, \beta\, \varepsilon\, \sigma_s) \qquad \text{Equation 6-15}$$

where another 'splitting coefficient', ε, can be determined from:

$$\varepsilon = \frac{\sqrt{\sigma_d^2 + \sigma_s^2}}{(\sigma_d + \sigma_s)} \qquad \text{Equation 6-16}$$

The results of the foregoing developments are summarized in Figure 6.3 and Figure 6.4 for the demand and capacity factors, respectively. As for the WSD illustration, it has been assumed that the biases in the median demands and capacities are unity, and that a 100-year design demand has been referenced.

Some interesting observations can be developed from the results summarized in Figures 6.3 and 6.4. High uncertainties and high Safety Indices imply large loading factors and small resistance factors. The ‘switch-over’ in the loading factors for Safety Indices less than about 2.9 is due to the use of the 100-year return period design reference loading. For example, for a loading uncertainty of 0.8 and a Safety Index of 2.5, a loading factor of 0.77 results from this formulation. However, if a 50-

year return period design reference loading is used for the same loading uncertainty and Safety Index, a loading factor of 0.96 is developed. The reference return period used to define the design loading condition has an important influence on the load and resistance factors and the factors of safety.

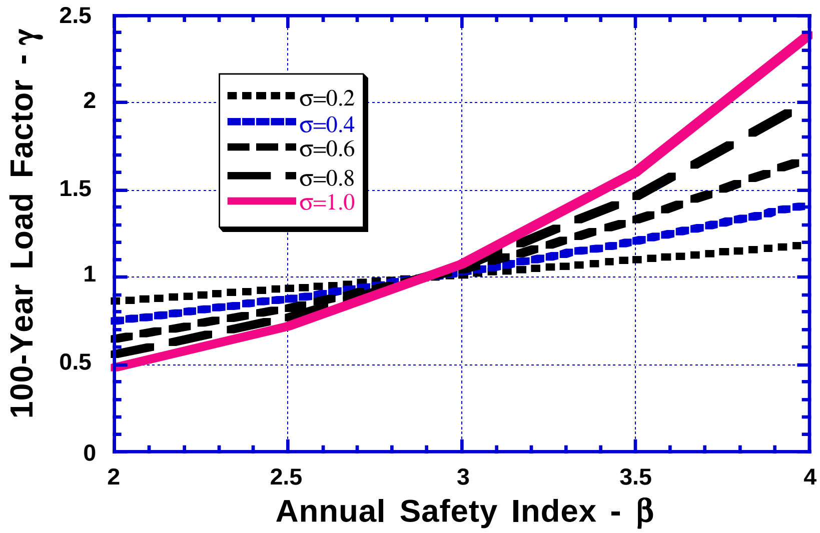

Figure 6.3- Loading factor for 100-year design loading as function of annual Safety Index and loading uncertainties

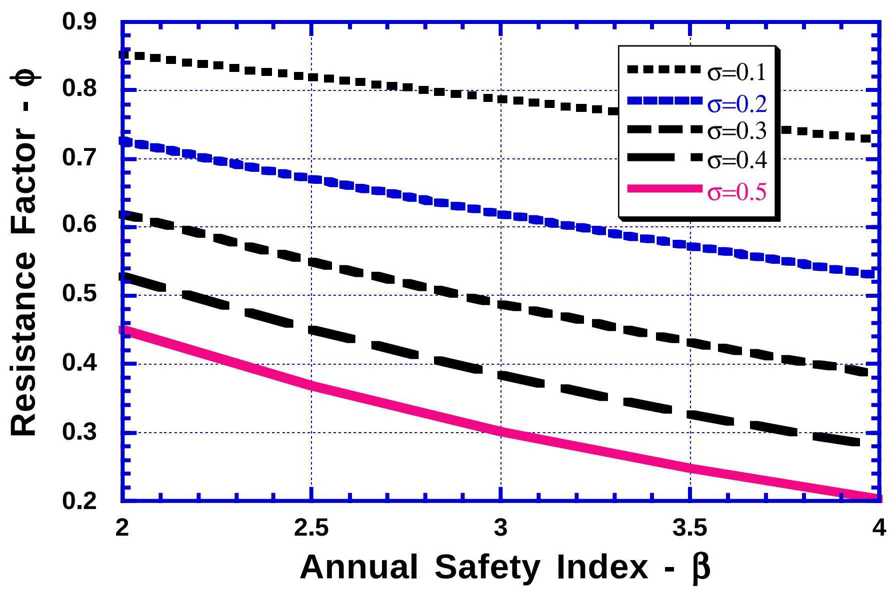

Figure 6.4 - Resistance factor as function of annual Safety Index and capacity uncertainties

These results could be verified as follows. For a Safety Index of 2.5, a 100-year return period design loading, and a loading uncertainty of 0.8, a loading factor of 0.77 is found. For the same Safety Index and an uncertainty in the resistance of 0.3, a resistance factor of 0.55 is found. This indicates a total uncertainty of $\sigma = 0.85$. The Factor of Safety is indicated to be 1.3 for these values.

For the same conditions (100-year design loading, biases of unity, annual Safety Index), the Factor of Safety is related to the Load and Resistance Factors as follows:

$$FS = \gamma / \phi \qquad \text{Equation 6-17}$$

The results from the foregoing example indicate FS = 1.3 versus $\gamma / \phi = 1.4$. The difference in result is due solely to the inaccuracies introduced by the splitting factor of 0.8. For these uncertainties, the use of a splitting factor of 0.777 would result in identical factors of safety.

Code 'calibrations' make very good use of the foregoing expression in development of LRFD codes. A 'reasonable' load factor is chosen based on judgment or 'precedent' or determined from analysis, then the resistance factor is determined based on the factor of safety contained or implied in the WSD. Alternatively, the Safety Indices implied in the WSD code are determined based on assessments of the biases and uncertainties in the WSD element resistance formulations. 'Sticky' problems develop when the factors of safety are not explicit or the biases are not the same in the two code formats and formulations.

6.4 Limit State Design

A third format for design guidelines has been the Limit State Design (LSD) format. In this format, the design demands are defined at the Ultimate Limit State (ULS) condition and the structure capacity is defined for this condition. Based on characterization of the demands and capacities as being independent and Lognormally distributed, the LSD condition can be expressed as:

$$1 = Fe\,(B_S / B_R)\,\exp\,[(\beta\,\sigma) - (K\,\sigma_S)] \qquad \text{Equation 6-18}$$

where K is the number of standard deviations of a standardized Normal distribution that are required to define the ULS loading condition. This is fundamentally the same expression used to define the WSD factors of safety, with the factor of safety set to unity. Solving for K for the condition of the median loading and capacity biases of unity and an effective loading factor of unity gives:

$$K = \beta\ (\sigma / \sigma_S) \qquad \text{Equation 6-19}$$

For the condition where the total uncertainty is dominated by the uncertainty in the demand ($\sigma \approx \sigma_S$), then

$$K \approx \beta \qquad \text{Equation 6-20}$$

Given that the demand – capacity characterizations have been based on an annual (1 year) time period basis, then the Average Return Period (ARP) for the ULS condition is:

$$ARP_{ULS} = [1-\Phi(K)]^{-1} \approx [1-\Phi(\beta)]^{-1} \qquad \text{Equation 6-21}$$

A K or β of 3.72 would define a LSD ULS loading ARP of 10,000 years. A K or β of 3.09 would define a LSD ULS loading ARP of 1,000 years. It was based on this development that lead the author to propose that the probability of failure for an offshore structure could be determined by defining the ARP associated with the loading that would bring the structure to its ULS condition:

$$Pf \approx ARP^{-1}_{ULS} \qquad \text{Equation 6-22}$$

It has been contended that the single greatest obstacle to use of LSD methods is the necessity to analyze the structure for the ULS loading conditions and requiring analysis of the structure performance in the nonlinear range (at the collapse condition). Plastic design and ULS analysis methods have been developed to help address this need. The analytical process developed by the author (identified as Ultimate Limit State Limit Equilibrium Analysis – ULSLEA) and embodied in the computer program identified as ULSLEA and TOPCAT (Template Offshore Platform Capacity Analysis Tools) was specifically developed to address this need. The TOPCAT computer program has been licensed for international distribution by Engineering Dynamics Inc. as part of the SACS platform design and analysis computer program system (identified as SACS TOPCAT).

6.5 Combinations of Demands

Elements in structure systems can be subjected to a variety of types of different demands at the same time. For example, for most structure systems dead loadings (gravity, buoyancy) are generally reasonably well known. However, ‘tolerances’ in elements and components of the system can result in substantial differences between the 'ideal' and the 'real' dead loadings. Similarly, there can be important differences between the specified 'maximum' live loading and the maximum live loadings that are actually imposed on a system at any given time. The live loadings imposed on structure systems are amenable to control or management. Weight control on structure systems is an important issue that can have important effects on the safety and serviceability of these systems.

As a general guideline, when the design engineer is faced with issues concerning combinations of loadings from multiple sources for a given return period event, the premise that should be used is to select the expected maximum value of the parameter that dominates the loading for the system, component, or element at the given return period (level of probability), and set the other parameters conditional on the occurrence, time, and direction of application of the dominant parameter. Often this implies setting the other values at their usual 'expected' levels. If for some reason, the design engineer decides to use 'conservative' values for the important parameters that determine the storm or earthquake maximum loadings, then the bias in the maximum design loadings should be introduced into the determination of the appropriate factors of safety for design of the given elements.

The extreme environmental condition loading uncertainties will be characterized in three categories: 1) dead loadings, SD, 2) live loadings, SL, and 3) environmental loadings, SH. The total loading is:

$$S_T = S_D + S_L + S_H \qquad \text{Equation 6-23}$$

Based on the First Order Second Moment (FOSM) characterizations developed earlier, the mean total loading can be determined from the mean values of the loading components distributions

$$\bar{S}_T = \bar{S}_D + \bar{S}_L + \bar{S}_H \qquad \text{Equation 6-24}$$

If Lognormal distributions are used to characterize the loading distributions, then the values indicated above as means need to be replaced by the median values for the Lognormal distributions. The uncertainties (σ = standard deviation of the annual distribution of the maximum loadings, or in the case of Lognornally distributed variables, σ = standard deviation of the logarithms of the annual distribution of maximum loadings) associated with these loadings can be determined from (assuming no correlations between the three components of loadings):

$$\sigma^2_{ST} = \sigma^2_{SD} + \sigma^2_{SL} + \sigma^2_{SH} \qquad \text{Equation 6-25}$$

If there are correlations between the different sources of loadings, the correlation between two components of the loadings (e.g. Live, L, and Environmental, H) can be expressed with the correlation coefficient ρ_{LH} ($-1 \leq \rho_{LH} \leq 1$) and the uncertainty in the combined loads (L + H) determined as:

$$\sigma^2_{L+H} = \sigma^2_L + \sigma^2_H + 2\rho_{LH}\,\sigma_L\sigma_H \qquad \text{Equation 6-26}$$

The foregoing can be used to characterize the uncertainties in the total loadings on a system and its components and elements. For system components that have very high values of dead and live loading that generally have much lower uncertainties than associated with the storm or earthquake annual expected maximum loadings, the combined loading uncertainties will be lower than associated with the environmental loadings.

6.6 Demand 'Dynamic' Effects

The demand 'dynamic' effects can be expressed as:

$$F = [Fe]\,[X]\,[Y]\,[Z] \qquad \text{Equation 6-27}$$

F is the total demand effect induced in the system. [X] represents the parameters used to describe the basic demand conditions. [Y] represents the kinematics or motions developed by the demand conditions. [Z] represents the imposed static demands exerted on the elastic system by the demand conditions and kinematics. [Fe] represents the demand effects correction factor intended account for the dynamic, transient loading effects and the nonlinear, hysteretic response effects not recognized by static elastic analyses.

Due to the transient and dynamic demand aspects of many categories of demands, it can be important to recognize the potential differences between demands and demand effects. The term 'demand' is taken to represent the demands that are imposed on a system that are fundamentally independent of how the system responds or reacts to the imposed demands. Such demands frequently are referred to as being 'static' (even though they can vary with time).

The term 'demand effects' is taken to represent the internal forces that are induced or generated within a system that are dependent on how the system responds or reacts to the imposed demands. Such demands frequently are referred to as being 'dynamic demand effects.

Demand effects induced in a system are determined by two classes of factors:

- *The first class is related to the **characteristics of the demands**.*
- *The second class is related to the **performance characteristics of the system**.*

Fe is similar to the Dynamic Amplification Factor (DAF) for a structural system. The DAF is the ratio of the dynamic response parameter, $X_{D,}$, to the static response parameter, X_S. Generally, the DAF is associated with elastic systems. Fe is associated with systems that have significant nonlinear inelasticity. Both the DAF and Fe depend on demand and structure system characteristics. It is worthy to note that the demand characteristics exert dominant effects on both Fe and the DAF.

For an undamped SDOF elastic system subjected to an instantaneously applied loading, Po: DAF = 2. For an undamped SDOF elastic system subjected to impulsive applied loadings having a maximum value of Po and applied in a time duration of td, for td / Tn $\leq$ 0.25: DAF = 2 π ζ td / Tn. ζ is an area coefficient to express the area under the Po - td diagram: for a rectangular pulse, ζ = 1.0; for a triangular pulse ζ = 0.5, for a half sine wave pulse ζ = 2 / π. It is important to note that for this impuslive loading relationship that the DAF $\leq$ 0.5.

For a damped SDOF elastic system with a natural period Tn, subjected to sinusoidal loading with a period, T, the steady state response characteristics can be determined from (R = Tn / T = f / fn = frequency ratio):

$$DAF = \{[1 - (Tn / T)^2]^2 + 4 D^2 (Tn / T)^2\}^{-0.5} = \{[1 - (R)^2]^2 + 4 D^2 (R)^2\}^{-0.5} \qquad \text{Equation 6-28}$$

The peak DAF is: DAFm = 1 / 2D. For: Tn / T $\leq$ 1.4, DAF $\leq$ 1.0 (peak dynamic loading is deamplified). For an undamped SDOF elastic system subjected to N cycles of sinusoidal loading, the peak DAF is: DAFm = N π.

The nonlinear response characteristics of the system can be envisioned as having two primary effects. The first is to effectively lengthen the period/s of the system. The second is to effectively increase the damping in the system.

It is apparent that the loading related factors are inter-related with the system response or performance related factors. For example, the degree periodicity of the demand can influence the amount of cyclic degradation or strain accumulation experienced by the system. Similarly, the rate of loading will influence strain-rate effects in the system.

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Chapter 7

Time Dependent Reliability

7.1 Effects of Operating Exposure

Engineered systems are subjected to a wide variety of operating conditions that are cyclic in nature – these are constantly changing and fluctuating demands. These cyclic demands develop cyclic strains in the elements of these systems. If the strains are large enough, then the strength, stiffness, and capacity of the system elements can be reduced; this is the process of fatigue degradation. Engineered systems can also be subjected to operating conditions that can result in removal of the material/s that comprise the system (e.g. corrosion, biological attack). There can be a wide number of different types of conditions that can lead to degradation in the performance characteristics of the elements of a system and hence a system. Thus, it is important to be able to understand such conditions, their effects on the elements of the system, and their quality and reliability implications.

Note also that time or aging exposure can sometimes be a beneficial factor that increases the reliability of a system; to a point - before other aging processes can become dominant. Examples include aging of concrete, aging of other composites like epoxy - glass - carbon fibers, aging of soils (consolidation, thixoropy) and the foundation elements founded in and on them, aging of wood (curing reducing the moisture content). The reliability characterizations of systems to define Margins of Quality must recognize both beneficial and detrimental effects of aging.

This chapter summarizes background that can be used to develop reliability based fatigue design criteria for structure systems. A similar approach can be used for other time dependent processes that can lead to degrations in the capacities of the system elements or of the entire system. The premise of these criteria is to design the elements so that there is a small likelihood of fatigue degradation in either the structure elements or in the structure system. Use of inspection, maintenance, and repair (IMR) techniques to provide quality assurance in fatigue performance of engineered systems is outlined; these are 'defensive' measures that can be used to help assess and manage time dependent effects on the reliability of a system.

Present experience with the majority of engineered systems indicates that engineers have designed adequately for fatigue effects. However, there are notable exceptions. These include systems in which certain types of cyclic loadings or demands were ignored, stress or strain raisers were ignored, ‘improved’ materials were used expecting that the fatigue strength would be adequate, or unanticipated stress raisers were added to the elements.

Many fatigue problems have been associated with flaws introduced into the system in the course of its design and construction (e.g. poor welding, misaligned members), or in the course of its operation and maintenance (e.g. corrosion damage, dropped objects damage). Thus, one of the primary aspects of design for fatigue reliability includes quality assurance and control (QA/QC) throughout the life-cycle of the system (inspection, maintenance, and repair: IMR).

In general, design for fatigue reliability is concentrated on details of elements, and in particular joints. This is the first line of fatigue 'defense.' For it is in the local details and joints that the significant or major stress-strain raisers are developed. However, given the very large uncertainties associated with predictions of the cyclic strain histories that will be experienced during the lifetime of a system, and with the fatigue strength of as-constructed and as-maintained elements, high fatigue reliability of elements is rarely achieved.

System robustness, or the ability of the system to tolerate defects without significant reductions in its quality characteristics is the second line of defense. Effective redundancy (configuration), ductility, ‘excess’ capacity must be mobilized, and appropriate correlation in the elements that comprise the system must be mobilized.

The third line of defense is inspection, maintenance, and repair (IMR). Inspections help disclose unanticipated flaws and defects, and confirm design objectives are met. Maintenance is intended to help preserve the system so that it can fulfill its intended purposes. Repair and design for repairability is intended to draw the engineer's attention to the necessity for restoring the system's capacity given future damage and defects.

Fatigue is the degradation in material, element, and system strength and stiffness as a result of cyclic straining-stressing (Figure 7.1). A fatigue failure occurs when the loading exceeds the fatigue capacity of the element. It is important to know how the fatigue capacity is defined. There can be substantial differences between the capacity associated with the occurrence of the first observable crack and that associated with complete failure of the joint by cracking. There can be a factor of 10 to 100 between the number of cycles of cyclic stress to cause the first observable crack and the number of cycles of that stress to cause complete failure of the element.

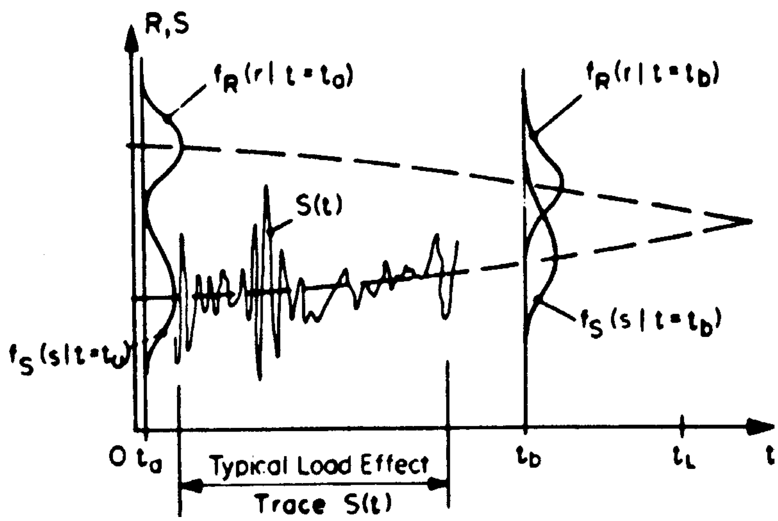

Figure 7.1 - Fatigue results in decreasing capacity while exposure results in increasing the likelihood of extreme cyclic demands

Note the increase in the mean value of the short-term distribution of the cyclic loadings in Figure 7.1. This is due to the increased likelihoods of experiencing extreme loadings as the period of exposure increases. Given that the engineer can develop a characterization of the time dependent decrease in the capacity of the elements that comprise a system during its lifetime and can also develop a characterization of changes in the demand characteristics imposed or induced in those elements during the system lifetime, then the probability of failure can be determined for each period of that lifetime to characterize the time dependent reliability (being careful to integrate time effects as necessary - because some are accumulative in nature).

7.2 Design for Fatigue

Design for fatigue reliability has four principal lines of defense:

- Minimize stress-strain risers - (stress concentrations) and cyclic straining-stressing through good engineering of the structural system and its details. This requires a high level of engineering quality assurance (QA) at the concept-development-design stage.

- Minimize flaws - (misalignments, poor materials, porosity-voids, etc.) through good, practical material and fabrication specifications and practices. This requires a high level of QA during the development of plans and specifications and during construction (involving materials selection, fabrication, transportation and installation). Further, there is a similar QA program required during operations to properly maintain the system.

- Minimize degradation at the local element through selection of good materials and fabrication practices, and good engineering designs (e.g. crack stoppers, damage localizers, and repairable elements). This requires a recognition that when (not if) fatigue degradation occurs, all reasonable precautions are taken to restrict its development and effects. Note, again QA plays a key role, particularly during operations to disclose the presence of fatigue degradation (early warning).

- Minimize degradation at system level so that when (not if) local fatigue degradation occurs, there are no significant effects on the system's ability to perform satisfactorily. Here good fatigue design requires system robustness (redundancy, ductility, capacity) and system QA. Inspections and monitoring to disclose global system degradation are another strategy to minimize potential fatigue effects.

The purpose of this discussion has been to outline the major factors and the complex interplay of these factors in determining fatigue reliability. Cyclic strains, material characteristics, engineering design, specifications, and life-cycle QA (inspections, monitoring) are all parts of the fatigue equation. This is the engineering equation of 'fail safe design' - fatigue may occur, but the system can continue to function until the fatigue symptoms are detected and repairs are made.

The alternative is 'safe life design' - the premise of this approach is that no significant degradation will occur and no repairs will be necessary. Safe life designs are difficult to realize in many long-life systems. This is because of the very large uncertainties that pervade fatigue design and analysis. Safe life design has been the traditional approach used in fatigue design for most systems. But, given the problems that have been experienced with fatigue cracking in some systems and the extreme difficulties associated with inspections of all types of systems, it is becoming clear that large factors of safety are need to truly accomplish safe life design and that fail safe design must be used whenever possible. Because of the extreme difficulties associated with inspections of complex systems and the high likelihoods of undetected fatigue damages, it is not normally reasonable to expect that inspections will provide the backup or defenses needed to assure fatigue durability.

Uncertainties and variabilities are present in each of the parts, and thus, reliability methods can play an important role in assisting the engineer to achieve fatigue reliable-durable structural systems.

7.3 Fatigue Analysis

A fatigue analysis can be organized into five basic components:

1) Characterize the life-cycle (short term and long term) cyclic conditions.

2) Determine the cyclic forces imposed on or induced in the structure (system).

3) Evaluate the cyclic strains-stresses developed in the element (detail) of concern.

4) Determine the degradation in strength and stiffness (damage of the element (detail) caused by the cyclic strains-stresses).

5) Given the fatigue damage, evaluate the acceptability of the element (detail) performance.

7.4 Fatigue Demands

In development of the following fatigue analysis procedures, it will be assumed that the intensity of the cyclic demands can be characterized with a long-term (e.g. T = 100 years) distribution of the intensities of the demand conditions (H) that can be represented by Weibull distribution (Figure 7.2). Two long-term cyclic demand conditions will be used; one that describes normal demand conditions and one that describes unusual demand conditions - two different types of demand conditions.

For extreme demand conditions, the cumulative distribution function [CDF = $F_x(X \leq x)$] of the cyclic demand range (h) is:

$$F_H(h) = 1 - \exp\left[-\left(\frac{h}{H_i}\right)^{\varepsilon_1} \ln N_1\right] \qquad \text{Equation 7-1}$$

For normal conditions, the CDF is:

$$F_H(h) = 1 - \exp\left[-\left(\frac{h}{H_o}\right)^{\varepsilon_o} \ln N_o\right] \qquad \text{Equation 7-2}$$

The Weibull distribution is characterized (shaped) with three key parameters: H, N, and ε; this makes it a very general distribution that can be used for many kinds of cyclic demand conditions. When the Weibull distribution shape parameter, ε, is unity, the distribution is equivalent to an Exponential distribution. The Weibull shape parameter can be larger or smaller than unity. Values greater than unity imply a greater number of cycles for a given cyclic demand range; thus more severe cyclic demand

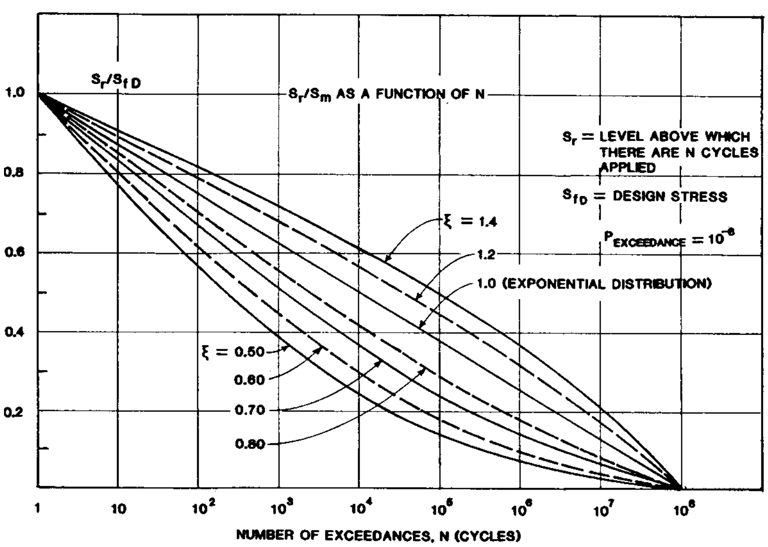

Figure 7.2 - Weibull long term distribution of stress ranges as a function of the shape parameter (ε)

conditions. Values less than unity imply fewer cycles for a given demand range; thus less severe cyclic demand conditions. The shape parameter is a function of the cyclic demand environment/s in which the system exists, and how the system responds to this environment (e.g. local loading effects, dynamic loading effects).

The fatigue stress range (peak to peak) (S_f) in the system 'detail' will be taken to be a function of the cyclic demand range:

$$S_f = C\,H^{\alpha} \qquad \text{Equation 7-3}$$

7.5 Fatigue Capacities

To characterize the fatigue capacities of the elements that comprise a system, a system composed of welded steel elements will be assumed. This assumption does not represent a substantial loss in generality for most system fatigue problems and materials used in such systems.

The number of cycles to 'failure' (N) of the system detail subjected to a cyclic stress range (S_f) will be taken as (Figure 7.3):

$$N = K\,S_f^{-m} \qquad \text{Equation 7-4}$$

This relationship is based on the log-linear relationship between the cyclic stress range and the number of cycles to fatigue failure:

$$\text{Log } N = \text{Log } K - m \text{ Log } S_f \qquad \text{Equation 7-5}$$

where m = negative slope of S-N curve, and Log K = life intercept of S-N curve (number of cycles axis).

The different curves in Figure 7.3 are for different types of weld details (e. g. X curve is for joints with weld profile control, X' curve is for joints without weld profile control; note it is here that specified QA/QC is defined). It is very important to understand that the fatigue design curves shown in Figure 7.3 are set at a conservative position that is generally minus two standard deviations of the test data (Figure 7.4). This substantial 'bias' (mean or median fatigue life / design or nominal fatigue life) has very important implications on the characterization of fatigue reliability.

Also note the two ordinates in Fig. 7.3 The left ordinate is the total 'nominal' cyclic stress range. This nominal stress does not include the stress concentration factor (SCF) effects of the joint details and welds. The right ordinate is the total 'hot spot' cyclic strain range. This hot spot strain range includes the strain - stress concentration effects of the joint details and welds. It is very important to understand which stress is referred to in S-N data and curves.

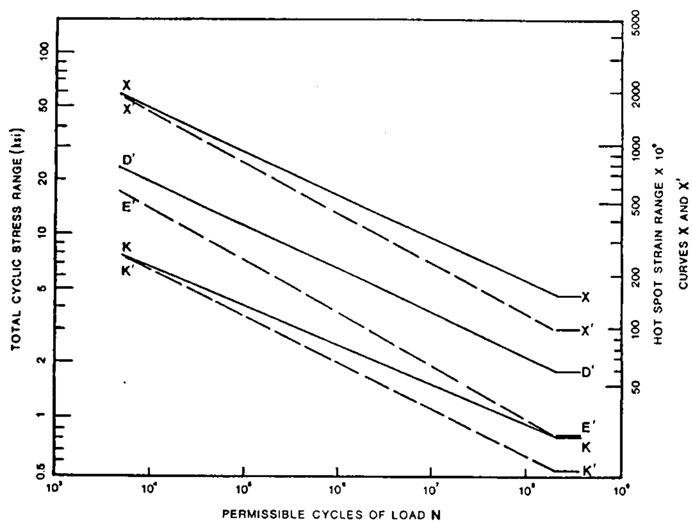

Figure 7.3 - AWS fatigue design stress range versus the number of cycles to failure (first observable crack)

Accumulation of fatigue damage (D) is assumed to be described by a linear damage accumulation rule (Palmgren-Miner):

$$D = \frac{n(S_{fi})}{{}_{i}N(S_{fi})} \qquad \text{Equation 7-6}$$

where $n(S_{fi})$ = number of stress cycles at stress fi, and $N(S_{fi})$ = number of cycles to failure at stress fi.

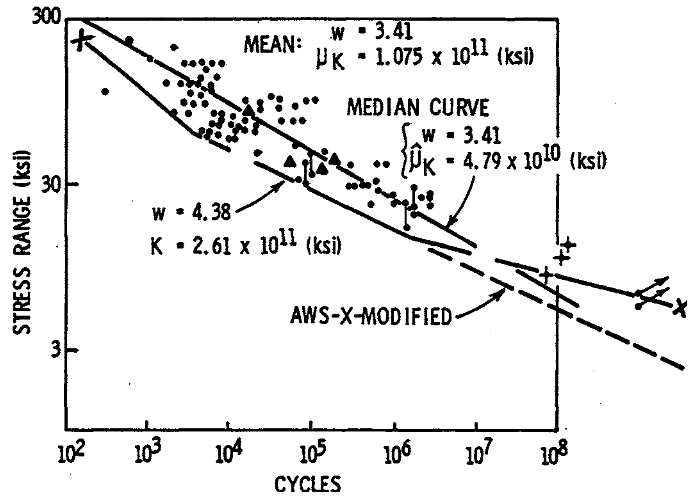

Figure 7.4 - American Welding Society (AWS) X-curve fatigue data

The summation is overall stresses, S_{fi}, experienced by the structural detail. This damage accumulative process will require integration or summation over the exposure period for the element; at time = 0, there is no damage and at time = exposure period the damage is all the damage that has accumulated between time = 0 and the defined exposure period.

When D = 1, failure is presumed to occur. Here again, there can be an important source of Bias. Fatigue test data clearly shows that fatigue failure does not always take place when D = 1. These data indicate that fatigue failure can occur between D = 0.5 and D = 4. The damage at failure is a function of the type of joint, the test conditions (e.g. corrosive environment), and the type of loadings (random cyclic stress ranges versus periodic stress ranges). This bias can only be determined by close examination of the test data, and given the normal limitations in fatigue tests of size of specimens, manufacture, and loadings, even the test data must be carefully interpreted.

Fatigue damage (D_L) accumulated over the life (T) of the detail can be computed by integrating over the long-term stress range distributions, the damage developed by each of the stress ranges summed with the linear damage accumulation rule is as follows:

$$D_L = \frac{T\, C^m}{K} (Y_0 + Y_1) \qquad \text{Equation 7-7}$$

where

$$Y_o = \frac{N_o}{T} \; H_o^{\alpha m} (\ln N_o)^{\frac{-\alpha m}{\varepsilon_o}} \; \Gamma\left(1 + \frac{\alpha m}{\varepsilon_o}\right) \qquad \text{Equation 7-8}$$

$$Y_1 = \frac{N1}{T} \; H_1^{\alpha m} (\ln N_1)^{\frac{-\alpha m}{\varepsilon_1}} \; \Gamma\left(1 + \frac{\alpha m}{\varepsilon_1}\right) \qquad \text{Equation 7-9}$$

and, $\Gamma(\cdot)$ = Gamma function (results from the integration analysis).

Now, let the design accumulated damage be limited to a fraction of the life damage:

$$D_D = \frac{D_L}{F_{sf}} \qquad \text{Equation 7-10}$$

or the design service life (T_S) be:

$$T_s = (F_{sf}) \cdot T \qquad \text{Equation 7-11}$$

where F_{sf} = fatigue life or damage Factor of Safety (commonly in the range of 2 to 3).

The fatigue design stress (S_{fD}) will be related to the fatigue design cyclic range intensity (H_{fD}) as before:

$$S_{fD} = C\, H_{fD}^{\alpha} \qquad \text{Equation 7-12}$$

Thus,

$$S_{fD} = \left[\frac{K\, H_{fD}^{\alpha m}}{T_s\,(Y_o + Y_1)} \right]^{\frac{1}{m}} \qquad \text{Equation 7-13}$$

7.6 Fatigue Reliability

Based on a fatigue analysis as developed to this point, the principal sources of uncertainty can be organized as follows:

- Characterization of the cyclic stress ranges (Sn) number of cycles experienced (Ns) at the 'boundaries' of a structural detail for a given 'life' or 'exposure' of the structural detail (frequently referred to as the long-term 'nominal' stress range - number of cycles characterization).
- Characterization of the 'hot-spot' (or potential location of crack initiation) cyclic stress ranges (Shs) (frequently accomplished using 'stress concentration factors' and results from finite element analyses).
- Characterization of the number of cycles to failure (Nf) for a given stress range (Sn, Shs) (frequently referred to as the S-N curve, often assumed to be a straight line on a log-log plot of S and N having an inverse slope m and life intercept K).
- Characterization of the 'damage' (D, Δ) accumulated as a result of the application of the various stress ranges and numbers of cycles of these stress range (frequently, a linear damage accumulation rule, Miners Rule, is used).

A reliability based approach can be based on a Lognormal format in which the random variables are assumed to have Lognormal distributions. The time for fatigue failure (Tf) is expressed as a function of the accumulated damage (D = Δ), the S-N curve parameters (K, m), a stress range model error parameter (B = actual/computed stress range), and a stress range parameter (Ω):

$$Tf = \frac{\Delta K}{B^m \Omega} \qquad \text{Equation 7-14}$$

The stress range parameter can be determined using a variety of approaches including a wave exceedance diagram (deterministic method), spectral method (probabilistic method), and a Weibull model of the stress ranges:

$$\Omega = \lambda(m)\, f_o\, s_m^m \left[1n\, N_T\right]^{-m/\xi}\, \Gamma\left(\frac{m}{\xi} + 1\right) \qquad \text{Equation 7-15}$$

where S_m = largest once in a lifetime stress range, ξ = stress range parameter, N_T = total number of stress ranges in design life, λ (m) = 1 unless the Rayleigh assumption was made in the analysis The probability of a fatigue failure (P_f) is expressed as:

$$P_{fF} = P\,[\,Tf \le T_S] \qquad \text{Equation 7-16}$$

where P [◊] is the cumulative distribution function of the time to failure Tf, and T_S is the service life.

$$P_{fF} = \Phi\,(-\beta_F) \qquad \text{Equation 7-17}$$

where Φ (·) = standard Normal distribution function and β_F is the fatigue Safety Index:

$$\beta_F = \frac{\ln(T_{50} / T_s)}{\sigma_{\ln T}} \qquad \text{Equation 7-18}$$

where T_{50}= median (50th percentile) value of T, and:

$$T_{50} = \frac{\Delta_{50} K_{50}}{B_{50}{}^{m}\Omega} \qquad \text{Equation 7-19}$$

$$\sigma_{\ln T} = \left[\ln(1 + C_\Delta^2)(1 + C_K^2)(1 + C_B^2)^{m2}\right]^{1/2} \qquad \text{Equation 7-20}$$

in which the C's are the coefficients of variation of each fatigue variable. Note that X_{50} is the median (50th percentile) value of the parameter X. It is here that the bias in design S-N curves must be removed. Generally, the -2σ level is used to define the ‘design’ S-N curve, then the bias in the expected S-N life intercept would be

$$B_K = \exp\,(2\,\sigma_{\ln K}) \qquad \text{Equation 7-21}$$

For $\sigma_{\ln K} = 0.4$, $B_K = 2.23$. If the life of the detail were based on the design S-N curve (e.g. a computed life of 10 years), the best estimate or expected life would be 22.3 years.

Uncertainty in the stress ranges (S) is expressed through the stress range model error parameter (B). The errors are attributed to:

8) Fabrication and assembly (B_M)

9) Cyclic state characterization (B_S)

10) Load predictions (B_F)

11) Determination of member loads (B_N)

12) Estimation of stress concentration factors (B_H)

Thus, the median bias is equal to the product of the median biases of the bias sources:

$$B_{50} = B_{M50} \bullet B_{S50} \bullet B_{F50} \bullet B_{N50} \bullet B_{H50} \qquad \text{Equation 7-22}$$

and:

$$C_B = \left[\prod_i (1 + C_i) - 1 \right]^{1/2} \qquad \text{Equation 7-23}$$

for i = M, S, F, N, H, K.

Recent studies have identified the resultant median bias associated with design of conventional tubular joints in offshore platforms to be in the range of B_{50} = 10 to 50, the resultant uncertainty in the bias to be in the range of C_B = 50 % to 60 % , and the total uncertainty in the estimated times to failure to be in the range of σ_{nTf} = 1.0 to 1.5. The implication of these findings are that while the computed life of a joint might be 10 years, the best estimate or expected life of the joint would be 100 years to 500 years.

It should be realized that in the foregoing development, the design reliability (Safety Index, β_d or probability of failure, Pf_F) has been targeted to a service life, Ts. For the Lognormal formulation, the Safety Index, β_t, for any exposure period, (t) can be expressed in terms of the design Safety Index, β_D for a service life as:

$$\beta_t = \beta_F - \frac{\ln (t / T_S)}{\sigma_{\ln T}} \qquad \text{Equation 7-24}$$

For t < T_S, the Safety Index is much larger than the design Safety Index, β_F (Figure 7.5). This explains why there is a very low probability of finding fatigue failures early in the life of a structure (if all has gone well). This equation also points out how inspections and repairs might be utilized to maintain the Safety Index above some value (Figure 7.6). Inspections can be used to reduce the uncertainties that contribute to $\sigma_{\ln T}$, and thus increase β_t.

It is important to recognize the limitations that are involved with most conventional inspection methods. The first line of inspections is 'visual'. Given the often marine-fouled, corroded, and painted surfaces found on most offshore structure connections, it is difficult to see small cracks. Even with cleaning and non-destructive testing (NDT), there are limitations because the entire structure can not reasonably be treated in this manner. Directing the cleaning and NDT to only 'low life' connections or areas also has limitations. As noted earlier, previous experience indicates that fatigue cracking often occurs at sites that are fundamentally unpredictable, particularly at the design stage. Thus, inspections should be based on a combination of 'deductive' analytical methods and 'inductive' methods. This approach will be developed in the next section of this paper.

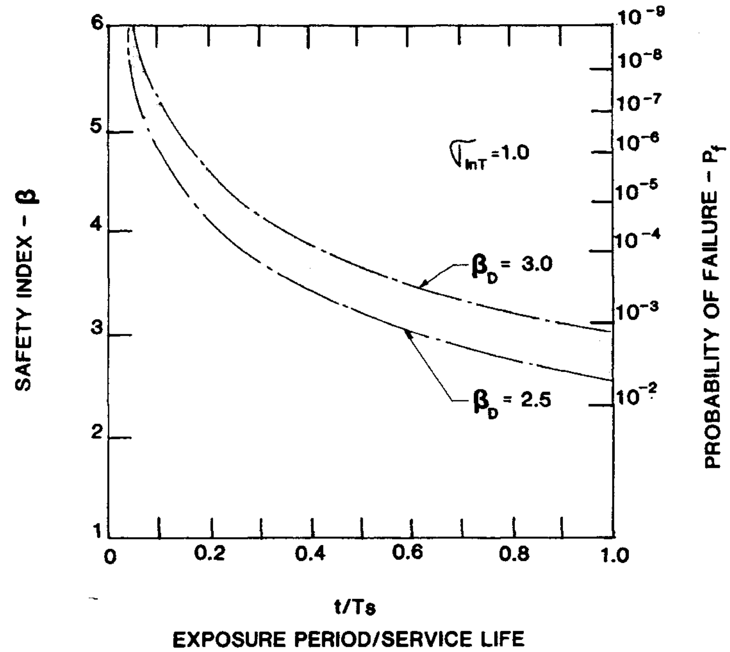

Figure 7.5- Fatigue Safety Index as function of exposure period

Repairs (if effective and well done) can increase β_t, by erasing all or large portion of the cyclic damage. Poorly designed and constructed repairs can have the opposite effect, resulting in a decrease in the reliability. It is important to realize that the repair operations involve hazards that can result in further damage to a structure system. In such cases the 'cure' (repairs) are worse than the 'disease' (fatigue cracks). All cracks found in structure elements do not need to be repaired. 'Judicious neglect' and closely monitoring the system and critical elements is often a preferable solution.

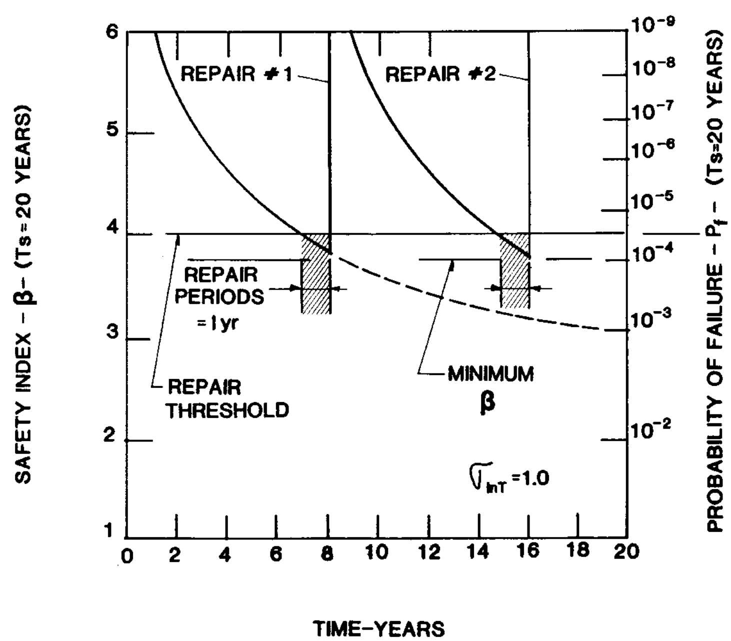

Figure 7.6 – Effects of IMR on fatigue reliability

7.7 Inspection, Maintenance, and Repair (IMR)

The objective of in-service inspections is to provide information and knowledge on the present, and future integrity of engineered systems. Inspections, data recording, management, and data analysis should all be a part of a comprehensive and integrated Inspection, Maintenance, and Repair (IMR) system. Records and thorough understanding of the information contained in these records are key aspects of IMR programs.

In-service inspections should be focused on:

- determination of condition of structural elements and structural system,
- disclosure of defects (design, construction, operation, and maintenance),
- assurance of conformance with plans and specifications, guidelines and rules, and quality requirements,
- disclosure of damage, and
- development of information to improve design, construction, operation, and maintenance procedures.

In-service inspections have three levels of intensity:

- general (global conditions),
- specific (basic aspects of defects and damage),
- detailed (precise descriptions of flaws and other items of operation and maintenance concern).

In-service inspections should be full-scope and include quality assurance and control measures in the structure, equipment, facilities, and personnel. Definition of the elements to be inspected should be based on two principal considerations:

1) consequences of defects and damage, and
2) likelihoods of defects and damage.

There are no general answers to the timing of inspections. The timing of inspections are dependent on:

- initial and long-term durability characteristics of the structure;
- margins that the operator wants in place over minimums so that there is sufficient time to plan and implement effective repairs;
- the quality of the inspections and repairs; and
- the basis for maintenance (on demand or when found - reactive, programmed - proactive, or combined).

Experience in inspections of engineered systems has adequately demonstrated that there are two distinct categories of defects that are present:

1) due to intrinsic (natural, inherent) causes - those that could have been or were anticipated (predictable), and

2) due to extrinsic (human error related) causes - those that could not have been or were not anticipated (unpredictable).

Current experience clearly indicates that a substantial amount (if not the majority) of the damage falls in the second category - unpredictable and due to the unanticipated 'erroneous' actions and inactions of people. IMR programs can not and should not be based solely on techniques that depend on the 'predictability' of damage and defects. This is one of the major reasons why methods that rely on structural, materials, and fatigue analyses / mechanics can not be depended on as the only basis to define IMR programs for engineered systems. In addition to this 'deductive' approach, an 'inductive' approach must be developed to help sense when things are 'not right' and then to develop information that can indicate how the damage or defect was developed and how it might best be repaired or prevented in the future.

In-service inspections and repairs are components in an IMR system that is intended to help disclose the presence of 'anticipated' and 'unanticipated' defects and damage to Critical System Details (CSD). Development of in-service inspection programs should address:

- elements to be inspected (where and how many),
- defects, degradation, and damaged to be detected (what),
- methods to be used to inspect, record, archive, and report results (how),
- timing and scheduling (when),
- organization, selection, training, verification, conflict resolution, and responsibilities (who), and
- objectives (why).

An IMR system is a critical part of the maintenance of in-service quality (serviceability, safety, durability, compatibility) of an engineered system. The IMR process must be in place, working, and being further developed during the entire lifetime of the structure. The IMR process is responsible for maintaining the quality of the structure during the useful lifetime of the structure. A fundamental and essential part of the IMR process is knowledge. The IMR process can be no more effective or efficient than the knowledge, data, and experience that forms the basis for the process.

The IMR process must be diligent and disciplined and have integrity. There must be a focus on the quality of the performance of the process; quality of the system will be a natural by-product. The IMR process should investigate a wide variety of alternatives to accomplish its fundamental objectives (maintenance of strength and serviceability). Inspections can range from general to detailed, visual to acoustic, periodic to continuous (monitoring). Maintenance can range from patching to complete replacement. Repairs can range from replacement as-was to re-design and replacement; temporary to permanent; from complete and comprehensive to judicious neglect.

The IMR process can be proactive (focused on prevention), or it can be reactive (focused on correction). The IMR process can be periodic (time based), or it can be condition oriented (occasion based). Combinations of proactive, reactive, periodic, and condition based approaches can be appropriate for different IMR programs. A major challenge is to find the combination that best fits a particular group of structures, their operations, and the organizations responsible for their integrity.

An IMR process should define the combinations and permutations of IMR that will produce the lowest total costs (initial and future) and optimize the use of resources without compromising minimum safety and reliability requirements.

Reliability Centered Maintenance (RCM, Figure 7.7) has been used to define IMR programs for complex structures such as airframes and nuclear power plants. RCM developed in the late 1960's by the commercial aviation industry is a method for developing and selecting maintenance alternatives based on safety, operational, and economic criteria. RCM employs a system perspective in its analyses of system functions, failures of the functions, and prevention of these failures.

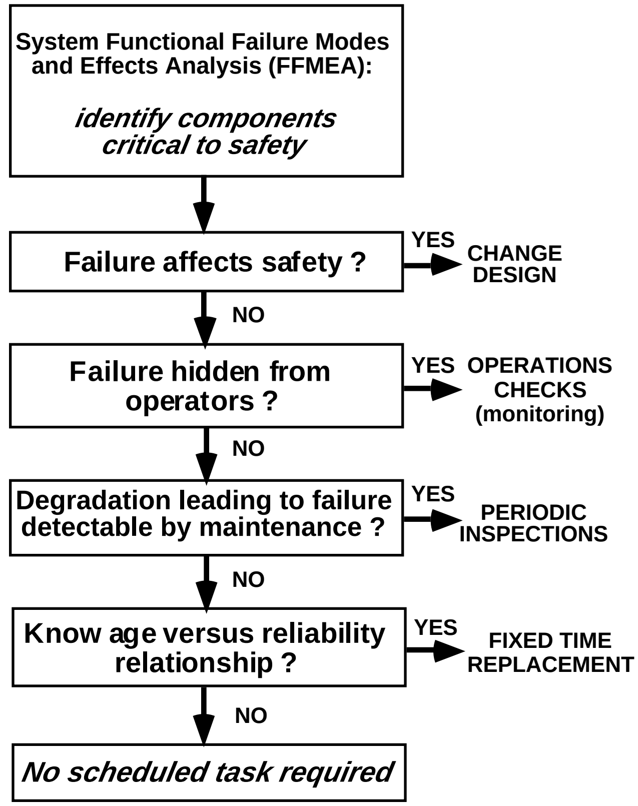

Figure 7.7 - Reliability Centered Maintenance strategy

The RCM methodology focuses on what should be done by only recommending IMR tasks on those component failure modes which are critical to maintaining important system functions. It provides a documented basis for the elimination of preventative maintenance tasks on components which do not support critical system functions. This approach not only eliminates the costs associated with the maintenance, but also reduces the risks of human errors developing during maintenance (maintenance errors often result in subsequent premature failures following restoration to service).

The airlines have found that this approach leads to greater effectiveness in providing a standardized justification process for doing and not doing maintenance. RCM has reportedly lead to a reduction in the amount of time an aircraft spends in the shop, more operational time, and fewer aircraft that have to be mobilized to provide the scheduled service.

There are five major steps in RCM: 1) definition of the system and subsystem boundaries, 2) definition of the subsystem interfaces, functions, and functional failures, 3) definition of failure modes for each functional failure, 4) categorize maintenance tasks, and 5) implement maintenance tasks.

In the first step, the system is divided into mutually exclusive subsystems. Elements that cross the subsystem interfaces are identified. Each subsystem has 'in-interfaces' that represent inputs to the subsystem and 'out-interfaces' that represent outputs from the subsystem.

In the second step, the inputs and outputs of each subsystem are linked with quantitative functional characterizations. Functional failures are then characterized for the subsystem (how the subsystem can fail to perform its functions termed a functional Failure Analysis, FFA).

In the third step, specific element failures that can cause each functional failure are identified. Generally, the dominant failure modes are developed from a failure modes and effects analysis (FMEA). The FMEA identifies conditions that must be prevented by maintenance actions.

In the fourth step, for each failure mode, a type of maintenance task is characterized: scheduled or unscheduled. Scheduled maintenance is generally assigned to all functions that can lead to safety related failures. If an effective scheduled maintenance task can not be defined, then the element / subsystem is either redesigned (to remove the failure mode or change the criticality) or the risk accepted. Unscheduled, condition based tasks are assigned to element / subsystem failures that affect operational capabilities

or have a significant influence on costs. Qualitative or quantitative evaluations of likelihoods and consequences can be utilized to assist the evaluations.

In the fifth step, the tasks are grouped and coordinated with available resources (equipment, personnel, time, etc.). If the current resource allocation is not sufficient or too sufficient, then cost/benefits can be estimated and decisions made on developing acceptable balances of costs/benefits of RCM.

Figure 7.8 summarizes the logical development of RCM IMR alternatives. The alternatives consist of Period Verification, Time Based Maintenance, Modification, Condition Based Maintenance, Break-down Maintenance.

In development of RCM the potential for 'secondary maintenance' has been recognized. Secondary maintenance addresses damage and defects that develop due to the occurrence of human and organizational errors. Experience reported by Jones (1995) indicates that 20% to in excess of 40% of the total maintenance effort is due to secondary

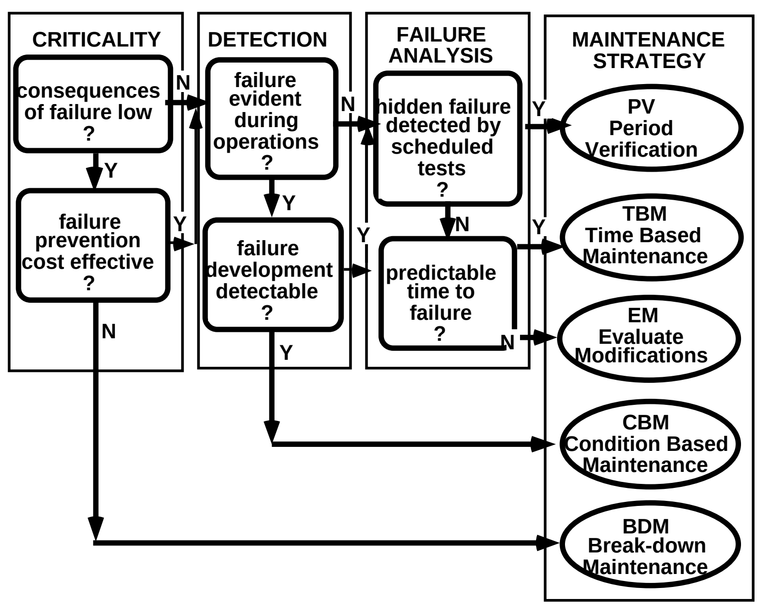

Figure 7.8 - Development of alternative IMR programs

maintenance. RCM methods address this category of maintenance. For example, the inspections of commercial aircraft include development of ‘ergonomic interventions’ such as socio-technical systems (management, team work), training (diagnosis, simulator training), information systems design (input, archiving, output), error controls, and improvements in inspection systems (lighting, access, etc.).

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Chapter 8

Robustness

8.1 Introduction

Two recent dramatic failures of engineered systems have demonstrated the potential effects of non-robust systems: the Hyatt Regency Skywalk and the World Trade Towers. The performance of both of these structures have important lessons to teach engineers about the importance of incorporating sufficient robustness - tolerance for damage and defects - into their systems.

Traditionally, engineers have used the term 'redundancy' for what many now interpret as robustness. Redundancy, or the degree of static indeterminacy can be a part of what is required to create a damage or defect tolerant system. But, we have learned that it takes much more than redundancy to create a robust system.

At this stage in development of engineering technology, it is fair to say that there are scant guidelines for engineers to understand how, where and how much robustness to incorporate into their systems. We think we understand the primary types of things that are required to create such systems. But, much more work is needed to translate this understanding into terms that engineers can readily understand and apply.

One thing we are beginning to realize is that often the engineering drive has been to create lighter and less costly systems (the drive for 'better, faster, cheaper' - or 'optimized' systems) frequently has resulted in systems that do not possess the damage and defect tolerance that needs to be there during operations of the system. These systems do not perform acceptably during their lifetimes. They are defect and damage intolerant and much effort is required to keep these systems in service; high IMR costs, high out-of-service costs, and other life-cycle costs come to dominate the picture. The low CapEx (capital expenditures) that were achieve with the 'optimized' system are sacrificed to the much higher OpEx (operating expenditures).

This is not to say that the drives for 'better, faster, cheaper' are bad; only that if they are carried to an extreme, that what is desired for the life cycle performance of a system may not be present. However, for this recognition to become a reality requires a focus on the long-term outcomes rather than on short-term outcomes. This requires attention not only to CapEx but also to Opex during the projected life of the system.

The fundamental reliability formulation that was developed in Chapter 4 clearly indicated the desirability of robustness in engineered systems; one for the case of the system performance in the face of Type I and Type II uncertainties; the other for the case of the system performance in the face of Type III and Type IV uncertainties. The different characteristics of these different categories of uncertainties can have important influences on how robustness might best be created in a given system. With these two contributors to the probability of failure of the system, and with a similar contribution to the probability of failure of the system without Type III and Type IV uncertainties, we want to be able to maintain the reliability of the system at the highest possible level during the lifetime of the facility - consistent with the other constraints that must be satisfied.

8.2 Engineering Robust Systems

No system is perfect (free from defects) and no system can remain perfectly intact during its lifetime; some defects and damage is inevitable. With significant defects and damage, the safety, durability, serviceability, and compatibility (Quality) of the system can be reduced. There is one key question associated with such defects and damage: How can the system be designed to have desirable damage and defect tolerance?

To this point, we have learned that it takes 4 fundamental things to create robust engineered systems:

- **Configuration** - the topology of the system elements, components, and system provides back-ups in the primary load carrying paths (frequently identified as redundancy).

- **Ductility** - the strain or deformation characteristics of the system elements, components and system are such that large inelastic deformations can be sustained without substantial losses in demand, load, or stress carrying capacities.
- **Excess capacity** - the demand, load or stress characteristics of the system elements, components, and system are such that when excessive demands are experienced due to unanticipated over-loadings or redistribution of demands from other elements and components in the system, the system is able to sustain these loadings and demands without undue distress. Capacity is related to appropriate capabilities to satisfy anticipated and unanticipated demands.
- **Correlation** - in systems comprised of both series and parallel elements, if the correlation of the individual elements capacities is high (correlation coefficient greater than about 80%), then the likelihood of failure of the systems is determined by the probability of failure of the most likely to fail element in the system. In a series system, if the correlation of the capacities of the elements is very low - independent - then adding elements increases the probability of failure. In a parallel system, if the correlation of the capacities of the elements is very low - independent - then adding elements decreases the probability of failure. Appropriate management of correlation is important in creation of robust systems. Correlation can develop in different ways including demand - capacity, between the capacities of elements, and failure modes determined by the magnitude of the demand uncertainties relative to those of capacity.

There are several different ways to visualize robust systems. One way is to characterize the performance of the system in its post-yield state (Figure 8.1) without introduction of damage or defects. In this illustration, all three systems shown have been 'normalized' so that they have the same maximum demand carrying capacity; they are configured differently and have elements that are made of different materials. After the first nonlinear behavior is evident in the system, we would like to

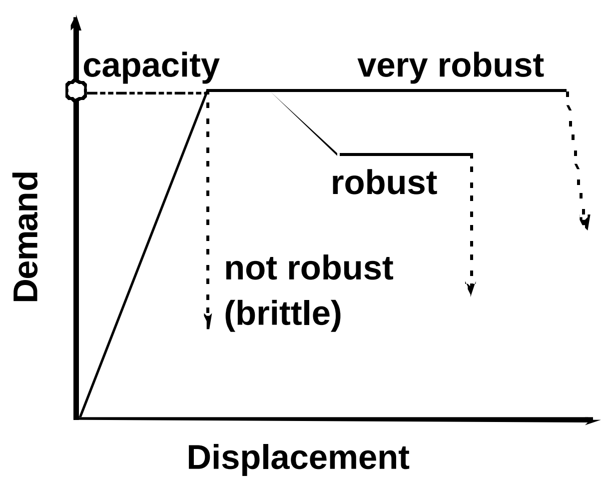

Figure 8.1 - Demand - displacement characteristics of different systems

have the system be able to carry the demands and to be able to develop large displacements (strains) without inducing undesirable performance in the system. 'Brittle' - non robust - systems are not desirable. There is no post-yield increase in demand carrying capacity and the system collapses immediately after reaching its yield displacement. Brittle elements and systems can result from a combination of brittle materials (brittle fracture) and / or ductile materials that are configured so that there are brittle modes of failure such as buckling (local or general).

Several important things are interacting in this characterization. The first is the demand carrying capacity of the system. The second is the displacement capacity of the system. The third is the area under the demand - displacement relationship. Given a specified demand capacity, we would like to have as much area under the demand - displacement relationship as is possible because this represents a capacity to dissipate energy or deform without a significant loss in demand capacity.

Figure 8.2 shows an idealized system comprised of three components that have identical yield and maximum (same) demand capacities, but very different post yield or residual capacities. Even though the elements all have the same yield - maximum demand capacities, the three different assemblies have three very different maximum and residual demand capacities. The system comprised of the very ductile (elastic - perfectly plastic) elements develops the highest demand capacity, the highest residual capacity, and hence the highest energy / work dissipation capabilities. Given that all other things are equal, this system can be expected to have a much greater reliability than the other systems. The numbers of elements, their configuration (to provide desirable element performance characteristics), and their resultant ductilities (post yield behavior) have very important effects on the behavior of the system.

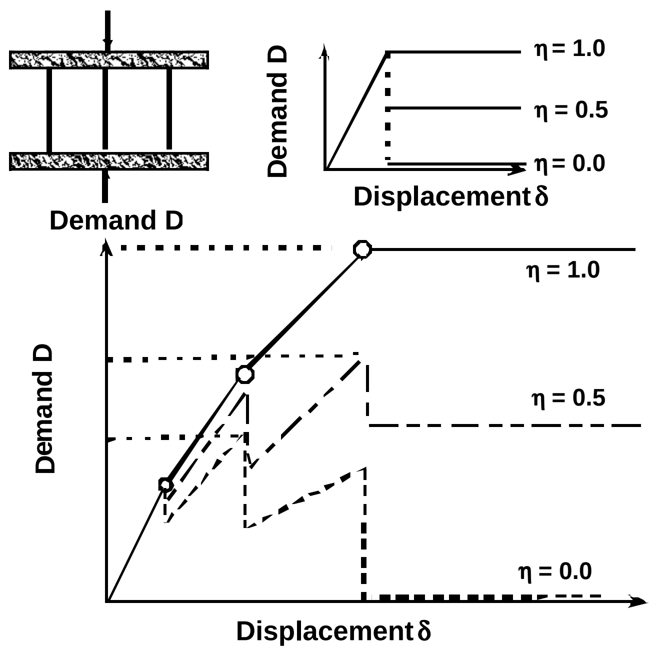

Figure 8.2- System performance dependent on post yield performance characteristics of elements

The next way to visualize a robust system is with damage or defects induced in the system (Figure 8.3). This visualization is portrayed as the percentage of undamaged capacity versus the percentage of damage induced in the system. In this visualization, the capacity could be expressed in either demand carrying, displacement, or energy absorption terms. The very robust system is able to sustain very large amounts of damage without showing any significant degradation in the capacity of the system while the non-robust system looses a very large percentage of its capacity with only minor or insignificant defects (hemophilic system). As elements in the robust system are damaged or defects are developed, they are able to shift their demands to other 'under demanded' elements (have excess capacity for the pattern of demands) and they are able to shift these demands because they have the ductility or ability to deform or displace and carry demands until the other under-demanded elements are able to pick up the demands.

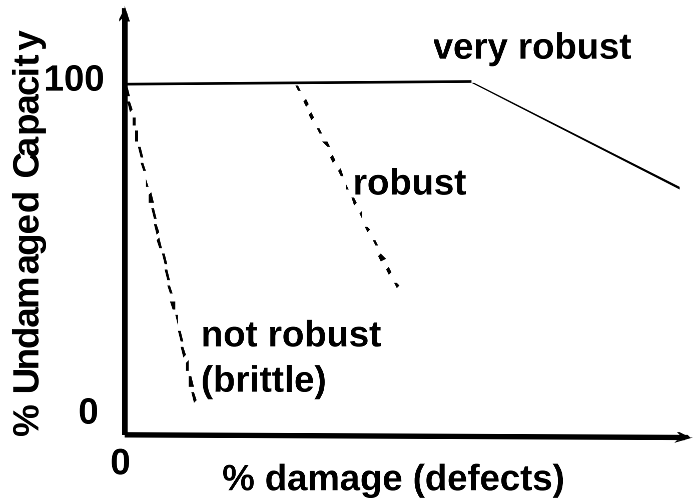

Figure 8.3 - Damage and defects effects on system capacity

The third way to visualize a robust system is in a probabilistic framework (Figure 8.4). This framework introduces the additional element of uncertainties and with these uncertainties the probabilistic characteristics of its elements; namely the potential effects of uncertainties in demands and capacities and 'correlations' (demand-capacity, element to element capacities, and failure modes). The 'probabilistic effects' can be as or in some cases more important than the deterministic effects previously discussed.

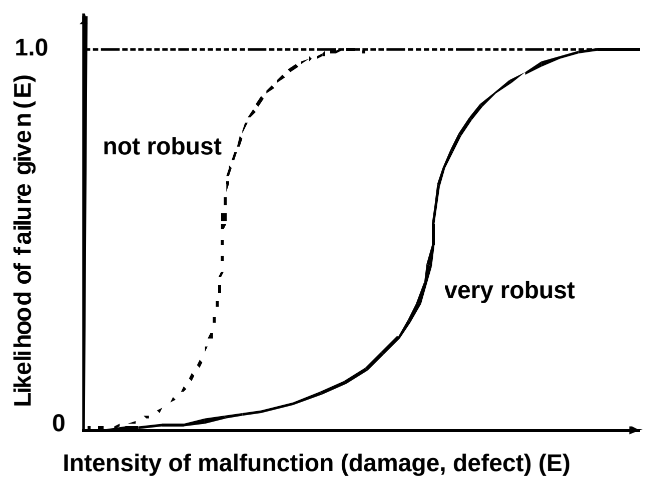

Figure 8.4 - Probabilistic effects of robustness

Studies have been performed on 'idealized' series systems and parallel systems with ductile elements with the assumptions that the strength of the elements can be modeled by Normally distributed random variables which are equally correlated with a common positive correlation coefficient ρ, and that the loads are deterministic and constant in time and all elements are designed so that they have the same Safety Index βe. The results are summarized in Figure 8.5 for series and parallel systems (Grigoriu and Turkstra 1979) comprised of different numbers of identical elements. Element to element correlations play an important role in determining the reliability of these systems. For series systems, high degrees of correlation are desirable to minimize the likelihoods of developing 'rogue' elements that have low capacities. For parallel systems (note the scale is reversed), low degrees of correlation (e.g. independence) are desirable to be able to enjoy the benefits from the system elements.

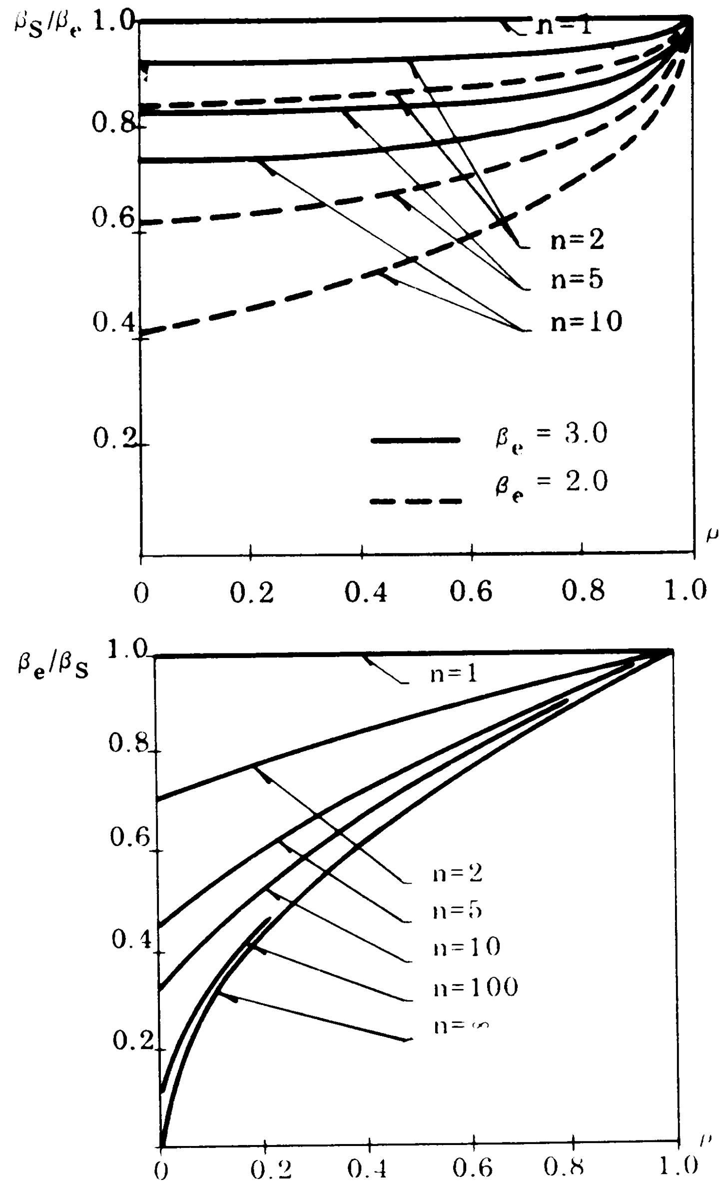

Figure 8.5 - Relationship of system Safety Index (β_S) to the element Safety Index (βe) for series (top) and parallel (bottom) systems (Thoft-Christensen and Bake, 1982)

Using numerical analyses, Guenard (1984) developed analytical results for idealized parallel systems that were comprised of elements that had different residual strengths (Figure 8.2). Results for two degrees of element to element correlation and element factor of safety of

1.5 (deterministic loading) are shown in Figure 8.6 (left zero correlation coefficient, right 0.5 correlation coefficient). For equity in the comparisons, as the number of members was increased, their common mean resistance value was reduced proportionally to insure that the system is deterministically equivalent from to case. Ductility (expressed with η) and 'redundancy' expressed by the number of elements have obvious benefits for the case of zero correlation. For brittle systems, there is little or no benefit in adding elements to the system. However, when the element to element correlation is increased, there is a dramatic reduction in the benefits of ductility and redundancy. Brittle systems are not affected very much.

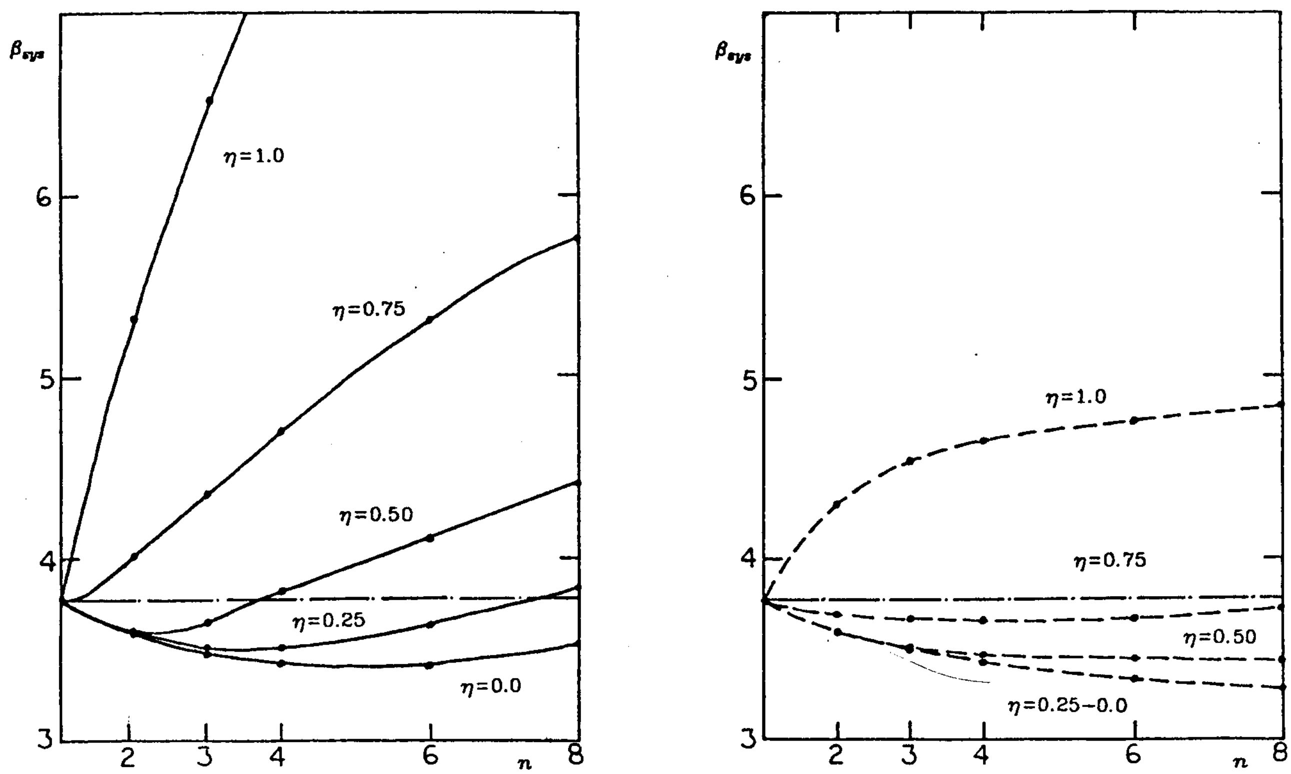

Figure 8.6 - Parallel element system Safety Index as function of numbers of Eeements, ductility, and correlation (left zero, right 0.5) (Guenard 1984)

8.3 Evaluating System Capacities

The starting point for evaluating the reserve strength of a defective or damaged system (structure) (Figure 8.7) is to develop an understanding of the nature and extent of damage to the elements and components that comprise such systems. In the case of an existing system, this understanding must be based on thorough inspections of the structural system. In the case of new structural systems, the definition of extent and location of damage is generally based on experience, evaluations of the criticality of the elements, and data bases that indicate the locations, extents, and likelihoods of damage.

The evaluation proceeds to a characterization of how the damage has affected the load and deformation performance characteristics of the damaged element or elements. Damage can also affect the "life" or durability characteristics of the element/s. Given multiple damage sites, the next problem is to define how the damaged elements affect the performance characteristics of the major components (assemblies of elements) that comprise the structural system.

At this point, the engineer is ready to characterize the structural system performance states of interest. Structural performance

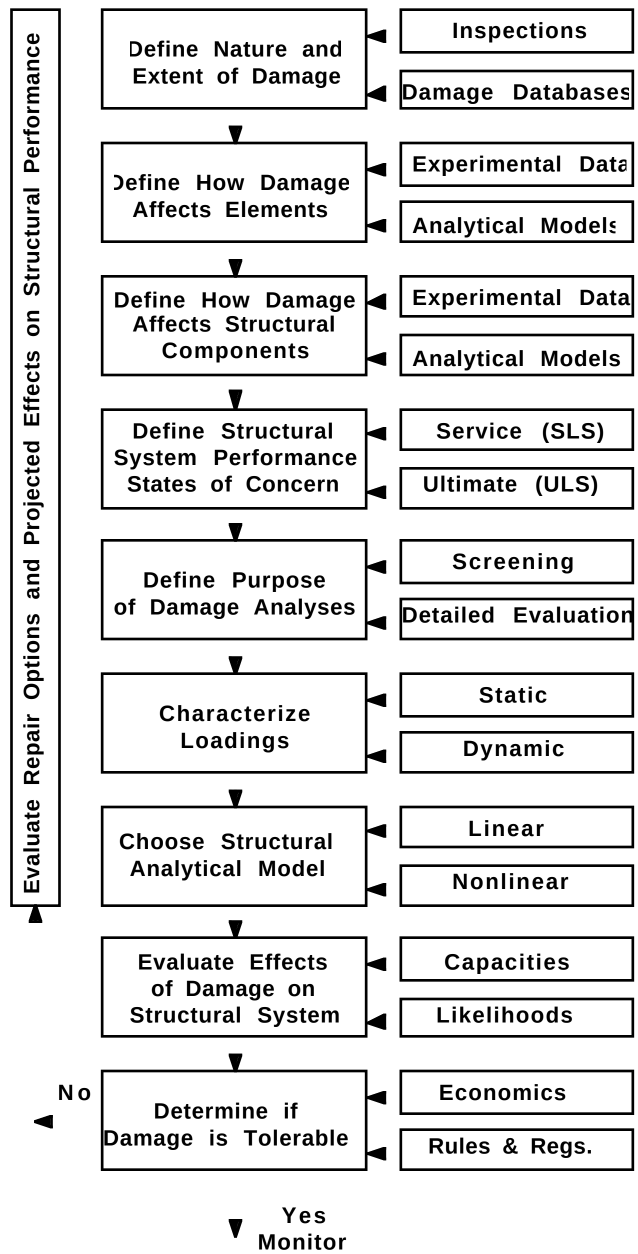

Figure 8.7 - System damage - defect assessment process

states are generally characterized in two general categories: 1) serviceability limit states (SLS), and 2) ultimate limit states (ULS). SLS are those structural performance states that influence the ability of the system to perform its intended purposes without undue interruptions or loss of efficiency. ULS are those structural performance states that define the conditions under which the system is rendered totally unserviceable or that result in catastrophic collapse (loss of stability) of the system.

Damage can affect both SLS and ULS. Damage important to SLS is generally durability and availability related. Damage important to ULS is generally related to reduced load resistance or deformation capacity. The loading conditions appropriate for SLS and ULS damage assessments are generally radically different. ULS damage assessments are concerned with very rare, extreme events while SLS damage assessments are concerned with frequently occurring events. It is for this reason, that the definition of the loading conditions must be based on the definition of the structural performance state of interest. The time, magnitude, direction, and combinations of loadings are dramatically influenced by the structural performance state of interest. Dynamic and transient loading effects will be different for SLS and ULS.

The definition of the structural performance state of interest will also have dramatic effects on the type of structural analysis that is appropriate for the damage evaluation. SLS analyses generally will be linear and elastic. ULS analyses generally will be nonlinear and inelastic (or "equivalent linear"). While linear, elastic analyses of structural systems have been highly developed, nonlinear, inelastic analyses are not so highly developed.

As noted earlier, the structural analyses can either be static or dynamic. Again, the type of analysis must be chosen appropriate for the structural performance state of interest and the loading characteristics that are appropriate for that performance state.

Once the type of structural analysis has been chosen and the loading characteristics defined, then the effects of damage on the performance characteristics of the structural system can be determined. If the damage is indicated to have negligible effects, then unless there are other extenuating circumstances or considerations, the system does not need to be repaired. If the assessment results in an uncertain condition ("maybe"), then additional monitoring, inspection, and analysis is indicated. If the damage is indicated to result in unacceptable performance characteristics, then the implication is that the damage should be repaired and the effects of various repair alternatives on the integrity of the structural system evaluated.

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Chapter 9

Criteria for Design and Requalification of Platforms In the Bay of Campeche, Mexico

9.1 INTRODUCTION

At the present time, there are more than 250 platforms located in the Bay of Campeche, Mexico. These platforms produce in excess of 2 million barrels of oil and 1.5 billion cubic feet of gas per day. The majority of the platforms are located in water depths between 30 m and 50 m. Most of these platforms were installed in the 1980's and 1990's, with some platforms installed in the late 1970's.

In October 1995, hurricane Roxanne formed in the western Caribbean Sea, crossed the Yucatan Peninsula, and entered the Bay of Campeche. Due to a southward moving front, the hurricane did not follow the normal northerly path of most hurricanes. It was forced back into the Bay of Campeche and the eastern coast of Mexico where it did considerable damage. Roxanne was the most severe hurricane to affect the Bay of Campeche during this century. It generated environmental conditions which approximated those of 100-year return period hurricanes (Oceanweather 1996a; 1996b; 1996c; Glenn, 1996).

Offshore, the majority of damage was confined to pipelines (Valdes, et al, 1997). There was some damage to platforms, but in general, these structures performed very well (Bayazitoglu, 1998). Following hurricane Roxanne, PEMEX initiated an extensive inspection of the platforms in the Bay of Campeche. Some damage to the platform structures were found. Most of the damage was confined to the underwater portions of the structures (cracked joints). PEMEX also initiated a program to reassess and requalify the platforms according to the API guidelines (API, 1993; 1997). The results from this program indicated that the majority of the platforms would not qualify for continued services without very extensive and expensive remedial work (Valdes, Ortega, 1998; Rey, et al, 1998).

Given the results from the platform and pipeline inspections and fitness for purpose studies, PEMEX and IMP initiated development of Risk Assessment and Management (RAM) based criteria for design and requalification of the platforms and pipelines in the Bay of Campeche (Bea, 1997a). The RAM based criteria was to take advantage of the results from the oceanographic, earthquake, and fitness for purpose studies, recent results from studies conducted by the API to develop advanced criteria for design of new platforms and assessment of existing platforms (API, 1993; 1997), and recent results from studies conducted by the International Standards Organization (ISO, 1997) for design of offshore platforms.

RAM processes to define structure design and reassessment criteria have been in use by the offshore industry since the 1970's (Marshall, Bea; 1975; Bea, 1975). The RAM processes address the following:

- environmental and operational loadings (demands) imposed on and induced in the platforms,
- capacities of the structures to resist these loadings,
- reliabilities of the structures given the evaluations of demands and capacities and assessment of the associated uncertainties,
- acceptable and desirable levels of reliability, and
- definition of loading and capacity criteria that will result in the defined levels of reliability.

This chapter documents how RAM processes were applied to development of criteria for design and requalification of PEMEX platforms in the Bay of Campeche. The criteria will be based on use of the API RP 2A guidelines (1993, 1997) that determine the platform hurricane and earthquake loadings and the capacities of the structural elements that comprise the platforms.

First, the platforms in the Bay of Campeche will be classified according to their functions and the acceptable levels of reliability will be determined. Second, the oceanographic conditions in the Bay of Campeche will be evaluated and the forces associated with these conditions characterized, including evaluations of biases (actual values / predicted or nominal values) and uncertainties. Third, the earthquake conditions in the Bay of Campeche will be evaluated and the forces induced in platforms characterized. Structure reliability analysis methods will be used to define environmental loading and structure capacity design and requalification parameters for the hurricane and earthquake conditions.

9.2 PLATFORM CLASSIFICATIONS

Guidelines were developed for platform Safety and Serviceability Classification (SSC) by PEMEX and IMP. These classifications are summarized in Table 9.1. The platform SSC is gauged either by the local productivity of a platform or by the production that it serves to support. This SSC is based on the provision of adequate measures by PEMEX to prevent injuries to personnel (life saving equipment, procedures, training), to resources (down-hole safety equipment, emergency shut-in equipment and controls), and to the environment (pollution prevention, control, and clean-up measures). These SSC are focused on the ability of the platform to adequately support extreme environmental loading conditions developed by hurricanes and earthquakes.

Table 9.1- Platform Safety and Serviceability Classifications (SSC)

SSC	Consequences	Local production or production handled (barrels per day)
1	Very High	> 100,000
2	High	50,000 - 100,000
3	Moderate	20,000 - 50,000
4	Low	1,000 - 20,000

Table 9.2 summarizes the target annual Safety Indices (β) and annual probabilities of failure (Pf) for hurricanes or earthquakes that were associated with

Table 9.2- Safety and Serviceability Classifications and Annual Safety Indices

SSC	Pf (new)	β (new)	Pf (exist)	β (exist)
1	2.0 E-4	3.60	4.0 E-4	3.40
2	3.5 E-4	3.44	7.0 E-4	3.23
3	5.0 E-4	3.33	1.0 E-3	3.12
4	1.0 E-3	3.12	2.0 E-3	2.87

each of the platform SSC (Pf ≈ $10^{-\beta}$). These target annual Safety Indices were based on the results from three approaches: 1) economics - utility optimization, 2) historic precedents (e.g. included in accident databases, previous codes and guidelines), and 3) current standards-of-practice (e.g. implied in codes, guidelines, present decisions concerning fitness for purpose) (Bea, 1991a). These target reliabilities were based on inclusion of Type I, natural (inherent, aleatory) uncertainties. Type II, modeling (parametric, analytical, epistemic) uncertainties were not included. These target reliabilities were approved for application to development of the criteria for design and requalification by the management of PEMEX (Valdes and Ortega, 1998).

9.3 HURRICANE CONDITIONS & CRITERIA

9.3.1 Wave Characteristics

There have been extensive studies of oceanographic conditions in the Bay of Campeche performed by A. H. Glenn and Associates (Glenn 1977; 1996) and Oceanweather Inc.; (Oceanweather 1996a - 1996c). Figure 9.1 summarizes the expected maximum hurricane wave heights as a function of the water depth and average return period (Oceanweather, 1996a). The wave heights in shallow water have been based on conventional shallow water shoaling processes that assume a non-deformable or rigid sea floor. Note that there are two 10,000 year characteristics shown. The upper 10,000 year results (deep water wave height of 32 m) is based on an extrapolation of the Oceanweather results. This extrapolation was based on Extremal or Lognormal distribution functions (they gave the same results). The lower 10,000 year results (deep water wave height of 27 m) is based on an assessment of the characteristics of a 'maximum credible hurricane' in the Bay of Campeche.

On the basis of meterological considerations, the 'maximum credible' hurricane was assessed to have a maximum central differential pressure of 110 millibars, a radius to the maximum winds of 50 km, and an average forward speed of 28 km / hr. Fully developed wave conditions were assessed based on wave generation by the stable wind field for more than 6 hours. A 'second generation' wind field - wave field analytical model was used (Suhayda, 1997). A maximum significant wave height of

16 m and an expected maximum wave height of 27 m resulted from this model. This maximum credible storm was evaluated to have an average return period of 10,000 years.

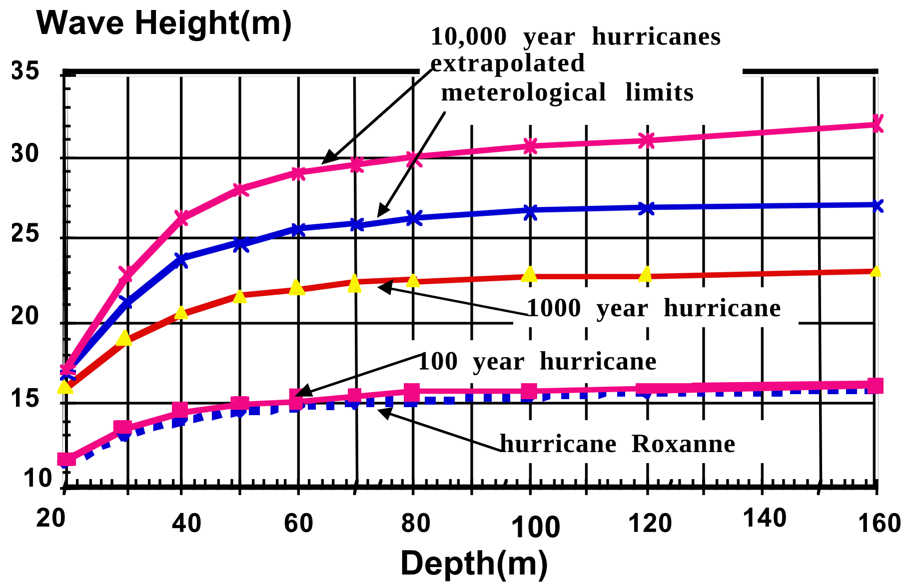

Figure 9.1- Rigid sea floor expected maximum wave heights based on Oceanweather Analyses and maximum credible storm analyses

Comparison of the two 10,000 year characteristics, one from extrapolation of the Oceanweather hindcasts (1996b) and the other from the meterological considerations, indicates that it is likely that there is a physical limit to the maximum wave heights that can be generated by hurricanes in the Bay of Campeche. Similar results have been developed for hurricanes in the Northeastern Gulf of Mexico. For the Northeastern Gulf of Mexico, the 'maximum credible' hurricane expected maximum wave height in deep water is in the range of 28 m to 29 m.

9.3.2 Effects of Deformable Sea Floor

Figure 9.2 summarizes the results of application of the Sea Wave Bottom Interactions (SWBI) analytical model to the Oceanweather hurricane Roxanne conditions (Suhayda, 1997; Zhaohui Jin, Bea, 1997). This analytical model is based on more than three decades of experimental and analytical work associated with evaluations of sea floor movements and wave conditions in the delta of the Mississippi River (Bea, 1975; 1996a; Clukey, et al, 1990; Forristall, et al, 1980; 1985; Gu, Thompson 1995; Kraft, et al, 1990; Suhayda, 1977; 1996; Shapery, Dunlap 1977; 1978). SWBI is able to account for the wave energy dissipated by hysteretic energy losses in the deformable sea floor. SWBI has been subjected to extensive verifications involving measured and predicted hurricane wave heights in the Mississippi River Delta (Forristall, et al, 1980; 1985).

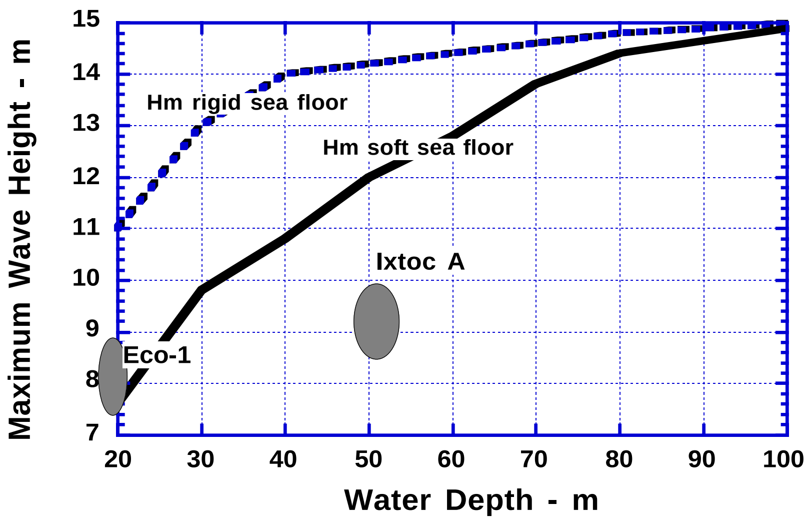

Figure 9.2 - Hurricane Roxanne maximum wave heights (Hm) for rigid sea floor and soft sea floor conditions

Best estimate soil and bathymetry characteristics in the Bay of Campeche were used to produce these results. There is reasonably good agreement between the observed and hindcast maximum wave heights. Note that the primary effects of the soft sea floor soils in modifying the maximum wave heights are in water depths less than about 80 m.

The differences between the observed and hindcast maximum wave heights could be attributed to several sources. First, there is an uncertainty in the observed wave heights (this uncertainty is reflected in the observed ranges shown in Figure 9.2. Second, there are uncertainties in the Oceanweather wave heights in both deep and shallow water. And third, there are uncertainties in the SWBI analyses (e. g. wave height, period, travel paths, soil characteristics).

Thus, based on this information, the SWBI results develop a maximum wave height that is ‘conservative’ compared with the observed maximum wave heights. There is a ‘bias’ (observed maximum wave height / predicted maximum wave height) of B_{Hm} = 0.83; the predicted results are about 20 % greater than the observed maximum wave heights. While only two reliable observations / reports were available in the Bay of Campeche, because of the extensive body of data and theory cited earlier, results for this stage of development of the wave force criteria were based on the conservative assessment of the shallow water wave heights. A future hurricane wave measurement program planned for the Bay of Campeche should help shed further light on this important issue.

Figure 9.3 summarizes the best estimate expected maximum wave heights for the soft sea floor conditions in the Bay of Campeche as a function of return periods in the range of 100 years to 10,000 years (Suhayda, 1997; Zhaohui Jin, Bea, 1997). In Figure 9.3, note that for water depths less than about 70 m, the 1,000 year return period maximum wave heights and 10,0000 year maximum wave heights (based on meteorological limits) are about the same. The effect of the soft sea floor soils is to effectively 'truncate' the wave heights for hurricane conditions having return periods in excess of 1,000 years. For the soft sea floor shallow water areas of the Bay of Campeche, the 1,000 year return period wave heights could be regarded as the maximum possible wave height conditions for all platform SSC.

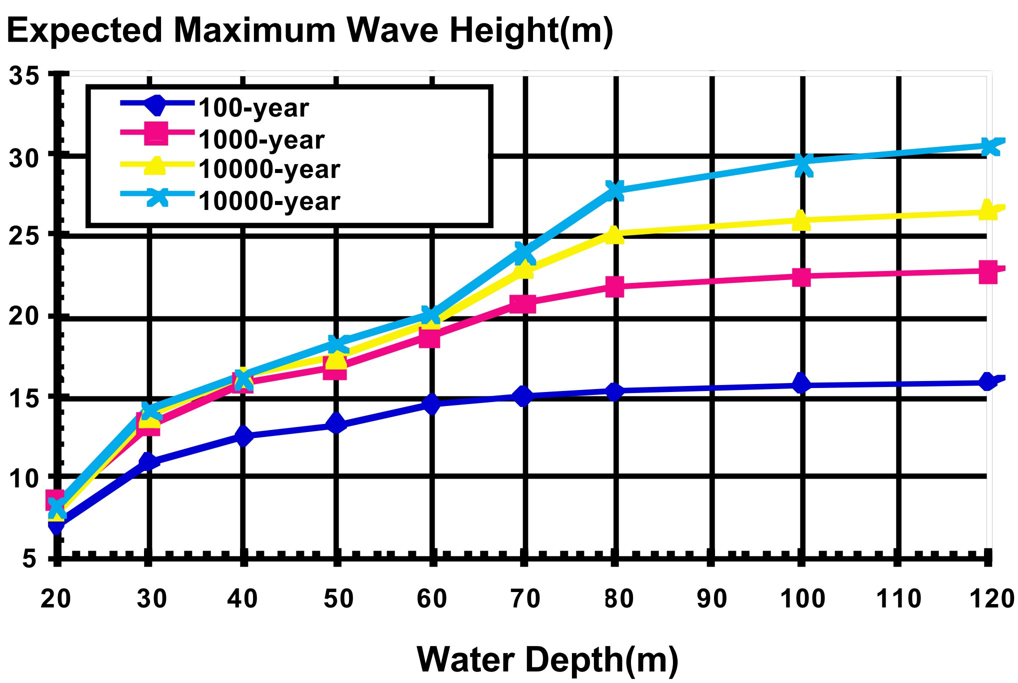

Figure 9.3 - Deformable sea floor expected maximum wave heights

9.3.3 Probability Characterizations

The foregoing results were used to determine Type I uncertainties associated with the expected annual maximum wave heights in the Bay of Campeche. Results for a deep water (water depths greater than 70 m) and two shallow water locations (30 m and 50 m) are summarized in Figure 9.4. Lognornal distributions were used for the probability function characterizations. The results indicated that for water depths less than about 70 m, the standard deviation of the logarithms of the expected annual maximum wave heights was in the range of $\sigma_{\ln Hm}$ = 0.25 to 0.30. This can be compared with values of $\sigma_{\ln Hm}$ = 0.38 to 0.47 determined during the first phase study for the rigid sea floor conditions. For deep water conditions (water depths greater than about 100 m) values in the range of $\sigma_{\ln Hm}$ = 0.40 to 0.45 were determined during this study. Note that these values and characterizations have not taken advantage of the effective 'truncation' of the wave heights at very long return periods.

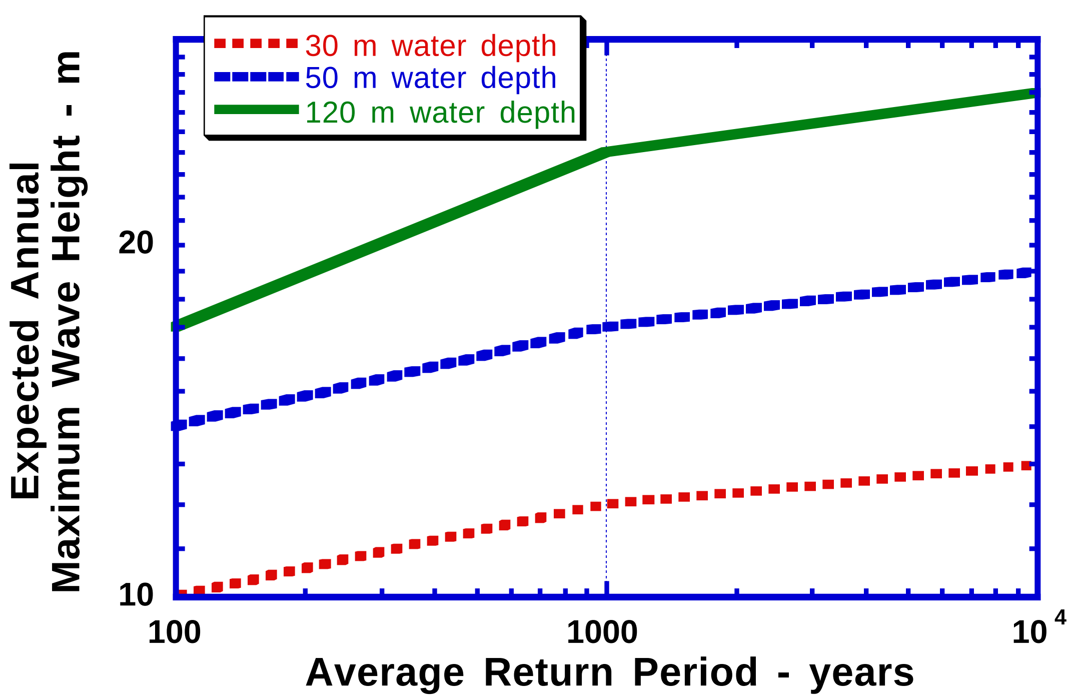

Figure 9.4 - Expected annual maximum wave heights

For the maximum wave heights influenced by the soft sea floor soils, there is an additional Type I uncertainty introduced by the natural variabilities associated with bathymetry (bottom slopes and geometry) and soil characteristics (strengths, shear modulus, cyclic strain behavior). The potential effects of these variabilities were studied by systematically analyzing the effects of changes in the sea floor characteristics. Plus and minus one standard deviation characterizations were analyzed (upper and lower 'bounds'). The results indicated that the variability decreased with increasing water depth. In these analyses, the variability in 50 m water depth was used. This was a value of $\sigma_{lnHm} = 0.10$.

In the subsequent RAM based design and requalification criteria analyses, the foregoing results were used to determine the total Type I uncertainties associated with the expected annual maximum wave heights in the Bay of Campeche. Lognornal distributions were used for these characterizations. The results indicated that for water depths less than about 80 m, the standard deviation of the logarithms of the expected annual maximum wave heights was in the range of σ_{lnHm} = 0.10 to 0.30 (Suhayda, 1997; Zhaohui Jin, Bea, 1997). A value of $\sigma_{lnHm} = 0.30$ was used in the criteria development for shallow water platforms. For deep water conditions (water depths greater than about 100 m) a value of $\sigma_{lnHm} = 0.45$ was used in the criteria development for deep water platforms.

Given these results, the next step was to re-evaluate the uncertainties and biases in the design and requalification hurricane force analysis procedures. This re-evaluation was based on the premise that the design forces would be determined using essentially 'unbiased' or 'best estimate' characterizations as defined by the API RP 2A guidelines (API, 1993, 1997; Bea, 1991b, 1997; Heideman, Weaver, 1992; Kriebel, et al., 1997). Most important is the incorporation of appropriate drag coefficients for rough members (Cd = 1.0), recognition of directional spreading at the center of intense hurricanes (kinematics factor = 0.85 to 0.88), recognition of shielding and blockage in the structures, and recognition of the directional characteristics of the extreme conditions waves and currents (essentially at right angles in shallow water) (Bea, 1990; Bea et al, 1991). Based on the use of the 'unbiased' procedure to determine aerodynamic and hydrodynamic forces, a force median bias, $\mathbf{B}_{\mathbf{Sm}} = 1.0$ was used in this development

In addition, a Type I variability in the determination of forces given the hurricane characteristics of $\sigma_{\ln Sm|Hm}$ = 0.15 was used (Bea, 1990, 1991; Heideman, Weaver. 1992). This variability included the natural randomness inherent in the wave - current kinematics and the forces. This variability does not include the unnatural randomness introduced by the force computation processes (Type II modeling uncertainty) (Bea, 1990, 1991; Kriebel, et al, 1997).

Given that the total maximum horizontal force (Sm) on the platforms varied with the square of the wave heights (Bea, 1977, 1990, 1997), the total uncertainty in the in the annual maximum horizontal hurricane forces was evaluated to be $\sigma_{\ln Sm}$ = 0.62 ($\sigma^2_{\ln Sm}$ = (2 x 0. 30)^2 + 0. 15^2) for shallow water (less than 80 m water depth) and to be $\sigma_{\ln Sm}$ = 0. 91 ($\sigma^2_{\ln Sm}$ = (2 x 0.45)^2 + 0.15^2) for deep water (greater than 80 m water depth).

9.3.4 Platform Loading Capacities

The next step was to evaluate the uncertainties and biases inherent in the evaluation of the platform system (combined structure and foundation capacities). For design of new platforms, a median bias of $\mathbf{B_{Ru}}$ = 1.4 was evaluated. This bias included the nominal steel yield strength effect (= 1.2), and the bias in the diagonal brace API RP 2A buckling formulation (= 1.2). The Type I uncertainty in the new platform lateral loading capacity was evaluated to be $\sigma_{\ln Ru}$ = 0. 15 (Bea, 1991b; Ekngesvik, et al, 1987; Nikolaidis, Kaplan, 1991). Based on the storm wave time-history loading characteristics and the platform nonlinear performance characteristics (ductility, residual strength), the transient loading - nonlinear platform response effective loading factor (Fe = dynamic nonlinear loading / static linear loading) was taken as Fe = 0.9 (Bea, Young, 1993; Bea, 1996b).

For reassessment of existing platforms, a median bias of $\mathbf{B_{Ru}}$ = 1.2 was evaluated. This bias included the nominal steel yield strength effect (= 1.0), and the bias in the diagonal brace buckling formulation (= 1.2). The transient loading - nonlinear response factor was again taken as Fe = 0.9. Note that this evaluation is based on the API Supplement 1 guideline that recommends that the expected values of the steel yield strength and other elements (e. g. joints, piles) be used in the reassessment analyses. The Type I uncertainty in the existing platform lateral loading capacity was evaluated to be $\sigma_{\ln Ru}$ = 0.25. This greater uncertainty for the existing platform capacity compared with the new platform capacity was attributed to the additional variabilities contributed by aging effects (corrosion, cracks, damage) (Ekngesvik, et al, 1987; Bea, et al, 1997). The effects of aging on the platform mean / median capacity would be included in the methods used to characterize the effects of aging on the structural elements in the platform.

9.3.5 Reserve Strength Ratios

The RAM based Reserve Strength Ratios (RSR = platform median lateral loading capacity / 100-year maximum lateral loading, Figure 9.5) were determined from the following expression (Bea, 1992; 1997b):

$$RSR = (\mathbf{B_{Sm}}\ Fe\ /\ \mathbf{B_{Ru}})\ \exp\ (\beta\ \sigma_{lnSmRu} - 2.33\ \sigma_{lnSm}) \qquad \text{Equation 9-1}$$

where σ_{lnSmRu} is the total uncertainty (standard deviation of the Logarithms) in the platform lateral loading capacity (σ_{lnRu}) and loading (σ_{lnSm}): ($\sigma^2 = \sigma^2_{lnRu} + \sigma^2_{lnSm}$). $\mathbf{B_{Sm}}$ and $\mathbf{B_{Ru}}$ are the median biases in the maximum lateral loading and maximum lateral capacity, respectively. Fe is the effective loading factor that incorporates dynamic – transient loading effects and the nonlinear, cyclic loading capacity of the structure (Bea, 1996b; Bea, Young, 1993). The 2.33 results from the use of the 100-year return period as the design loading (2.33 standard deviations from the mean or the 99th percentile of the annual maximum loadings). β is the annual Safety Index. This formulation is based on Lognormal distributions for both the expected annual maximum lateral loadings and the structure maximum lateral loading capacities (Bea, 1975).

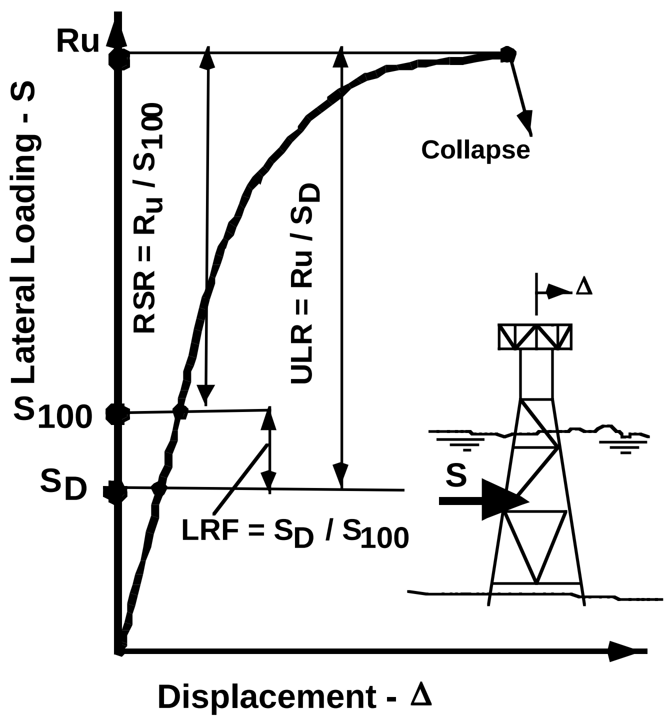

Figure 9.5- Reserve Strength Ratio (RSR), Load Reduction Factors (LRF) and Ultimate to Linear Ratios (ULR)

The total uncertainty is σ_{lnSmRu} = 0.64 and σ = 0.67 for new and existing platforms in shallow water, respectively. The total uncertainty is σ_{lnSmRu} = 0.92 and σ = 0.94 for new and existing platforms in deep water, respectively.

Table 9.3 - Hurricane loading Reserve Strength Ratios

SSC	RSR shallow water (new)	RSR deep water (new)	RSR shallow water (existing)
1	1.5	2.0	1.8
2	1.4	1.8	1.6
3	1.3	1.6	1.5
4	1.1	1.4	1.3

The RSR for existing platforms in shallow water and new platforms in deep (water depths greater than 70 m) and shallow water (water depths less than 70 m) are summarized in Table 9.3 for the four SSC.

9.3.6 Lower Deck Elevations

General guidelines for platform lower deck elevations of platforms not designed to sustain wave loadings in the decks are summarized in Figure 9.6 for platforms in water depths of 20 m to 100m. These results are based on a total (astronomical and hurricane tides) storm surge of 2 m and a wave crest elevation to wave height ratio of 0.7 for the Ultimate Limit State expected maximum wave heights. The 0.7 is a function of the wave steepness and water depth (Oceanweather, 1996a; 1996b). The Ultimate Limit State condition refers to the condition for which the platform would be brought to its maximum lateral loading capacity. These results are based on the Best Estimate and Upper Bound wave heights for the soft sea floor conditions.

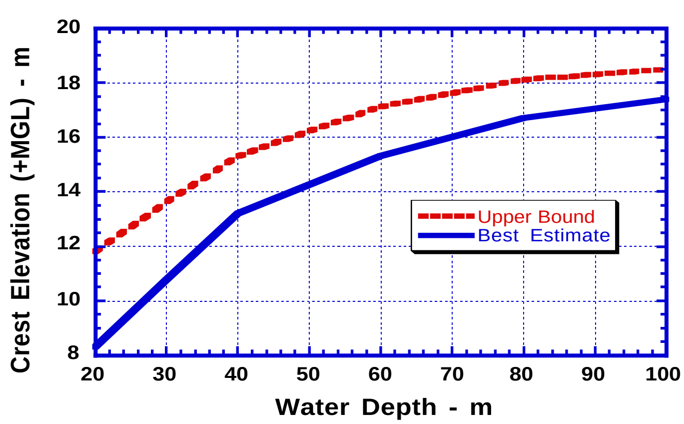

Figure 9.6 - Hurricane wave crest elevations / lower deck elevations

The average return periods of the Ultimate Limit State wave heights (ARP_{ULS}) can be determined from:

$$ARP_{ULS} = Pf^{-1} \qquad \text{Equation 9-2}$$

where Pf is the annual probability of failure that is identified with a particular SSC. This formulation presumes that the uncertainty associated with the expected annual maximum lateral loading is large compared with the uncertainty associated with the maximum lateral loading capacity of the structure (Bea, Smith, 1987). This is clearly the case given the foregoing evaluations of loading and capacity uncertainties.

For a given location (water depth), the expected maximum wave height can be determined from Figure 9.3 and Figure 9.4 for a given ARP. For example, for a new platform in a water depth of 50 m, for SSC 1, $ARP_{ULS} = (2.0\ E\text{-}4)^{-1}$ = 5,000 years and H_{MULS} = 17.5 m. The crest elevation associated with this condition would be 14.3 m. The general guidelines summarized in Figure 9.6 indicate that in water depths less than 50 m, the crest elevations and required deck clearances would be less than + 14.5 m (MGL - Mean Gulf Level). Note that for the RSR in Table 9.3 to be valid, the lower deck elevations that are not designed to sustain wave crest forces must be greater than the elevations shown in Figure 9.6.

9.3.7 RSR Criteria for Platforms with Lower Deck Wave Forces

Guidelines to determine wave-in-the-deck forces were developed for Bay of Campeche platform and hurricane conditions (Bea, et al, 1998). These guidelines are a modification of the API Supplement 1 guidelines that were intended to remove conservative bias in the API guideline based wave-in-deck forces. Based on the Guideline Procedure to determine wave forces acting on the lower decks of platforms located in shallow water (less than about 80 m), the natural variability in the annual expected total maximum lateral hurricane loading was determined to be σ_{lnsm} = 0.7. This uncertainty suggests that there is no significant increase in the rate of increase of total lateral force with inundation of the platform lower decks. This is in conformance with experimental data (Finnigan, Petrauskas, 1997). An additional variability of σ_{lnsm} = 0.4 was included to recognize the additional natural variability due to the deck wave loadings (Finnigan, Petrauskas, 1997). A total uncertainty in the wave-in-deck forces of σ_{lnsm} = 0. 81 results from this assessment.

Table 9.4- RSR criteria for existing platforms with lower deck wave forces

SSC	RSR wave crests in decks	RSR wave crests not in decks
1	2.0	1.8
2	1.8	1.6
3	1.6	1.5
4	1.5	1.4

The median bias (true or measured value / predicted or nominal value) in the computed annual maximum hurricane loadings (including the deck wave loadings) was evaluated to be $\mathbf{B_{Sm}}$ = 1.0, the effective loading factor Fe = 0.9, and the median bias in the computed lateral loading capacity was evaluated to be B_{Ru} = 1.2. The uncertainty in the platform lateral loading capacity was evaluated to be σ_{lnRu} = 0.25. The resultant uncertainty in platform loadings and capacities is σ_{lnSnRu} = 0.84.

Given this development, the RSR for wave-in-deck force conditions were determined from:

$$RSR = (1.0 \times 0.9 / 1.2) \exp (0.84 \beta - 2.33 \times 0.81) = 0.75 \exp (0.84 \beta - 1.89) \quad \text{Equation 9-3}$$

The annual Safety Indices, β, identified by PEMEX and IMP for SSC of the existing platforms in the Bay of Campeche and the associated RSR's for platforms whose lower decks experience wave crest forces, and for platforms whose lower decks do not experience wave crest forces are summarized in Table 9.4.

9.3.8 Load Reduction Factor Criteria

RAM based WSD guidelines were based on Load Reduction Factors (LRF) (Figure 9.5). The LRF is the ratio of the total lateral loading which develops design utilization ratios of 1.0 to the 100-year hurricane global design or reference total lateral maximum loading. The Ultimate to Linear Ratio (ULR) is the ratio of the ultimate lateral loading capacity (Ru) to that causing a unity check of 1.0 in the original design. The RSR, LRF, and ULR are related as follows:

$$RSR = ULR \bullet LRF \quad \text{Equation 9-4}$$

The ULR can be further defined as follows:

$$ULR = FSCE \bullet BCE \bullet SRF \bullet DM \qquad \text{Equation 9-5}$$

where FSCE is the nominal or code specified factor of safety of the most likely to fail (MLTF) member in hurricane loadings, BCE is the mean bias (mean true capacity / nominal or code capacity), SRF is the system redundancy factor that expresses the ratio of the lateral load that causes the failure of the first member and the ultimate lateral loading capacity (Ru), and DM is a designer margin that may be incorporated into a design.

In the development of LRF, based on results from studies of the ultimate limit state capacity characteristics of conventional steel, template-type platforms, the following ULR were defined (DM = 1.0) (Bea, Mortazavi, 1996, Bea, et al, 1997):

- Existing 8-leg platforms ULR = 1.25 x 1.1 x 1.2 = 1.7
- New 8-leg platforms ULR = 1.25 x 1.2 x 1.3 = 2.0
- Existing 4-leg platforms ULR = 1.2 x 1.1 x 1.2 = 1.6
- New 4-leg platforms ULR = 1.2 x 1.2 x 1.25 = 1.8
- Existing 'minimum' platforms (3 or less legs) ULR = 1.2 x 1.1 x 1.1 = 1.4
- New 'minimum' platforms (3 or less legs) ULR = 1.2 x 1.2 x 1.1 = 1.6

Based on these developments and the RSR summarized in Table 9.3, LRF were developed for the various SSC and types of offshore platforms. Two types of platforms resulted from this evaluation: 1) those with eight or more legs, and 2) those with 6 or fewer legs. The LRF for these two types of platforms are summarized in Tables 9.5 and 9.6. These LRF are applied to the 100-year storm force conditions and element WSD

Table 9.5 - Load Reduction Factors for new platforms in deep and shallow water

SSC	8 or more leg platforms (ULR = 2.0)		6 or fewer leg platforms (ULR = 1.8)	
	shallow water	deep water	shallow water	deep water
1	0. 75	1.00	0. 83	1.11
2	0. 70	0. 90	0. 78	1.00
3	0. 65	0. 80	0. 72	0. 89
4	0.55	0.70	0.61	0.78

interaction ratios determined using API guidelines.

Table 9.6 - Load Reduction Factors for existing platforms in shallow water

SSC	8 or more leg platforms (ULR = 1.7)	6 or fewer leg platforms (ULR = 1.6)
1	1.10	1.13
2	0. 94	1.00
3	0. 88	0. 94
4	0.76	0.81

9.4 Earthquake Conditions and Criteria

Based on the information provided by Chavez (1987, 1997a-1997d), Guerra and Esteva (1978), and Guzman (1997, 1982), a RAM approach was used to define earthquake response spectra based criteria that would be used for design and requalification of platforms in the Bay of Campeche.

The seismic environment in the Bay of Campeche is influenced by three primary types of earthquake sources. The Zone 1 source is associated with the subduction zone on the western Pacific coast of Mexico. The earthquakes in this zone occur at depths of 15 km to 20 Km and have moment magnitudes up to M = 8.2. The Zone 2 source is associated with the lithospheric slab within the central portion of Mexico. Earthquakes occur in this zone at depths of 60 km to 250 km and have magnitudes up to M = 7.5. The Zone 3 source occur in the Trans-Mexican volcanic belt located primarily along the east coast of Mexico, have depths up to 20 km, and magnitudes up to M = 6.7 (Chavez, 1997c).

The historic catalog of earthquake occurrences during the period 1970 - 1996 were used by Chavez to determine the activity rates in each of the three sources and the occurrences of varying earthquake intensities and magnitudes in each of the three sources. The majority of records contained in this catalog date from the 1930's when reasonably comprehensive teleseismic instrumentation was in place. The largest earthquakes that have occurred in the vicinity of the Bay of Campeche are in the range of M = 3.0 to M = 4.0. The largest earthquake that has occurred during the past 100 years within a 100 km radius of the Bay of Campeche is M = 6.0. The vast majority of the seismic activity is concentrated along the Pacific coast of Mexico. The source of nearby large earthquakes are associated with the Motagua Fault Zone and the fault system (Sierra Madre Oriental-Chipas Peten province) that generally parallels the east coast of Mexico.

Attenuation relationships were developed for Mexico geologic and seismotectonic conditions by Chavez (1987). These relationships were based on available strong ground motion recordings made in Mexico. These relationships are appropriate for on-land ground surface conditions that can be generally characterized as 'firm alluvium.' The relationships were tested for events that occurred on each of the three types of earthquake sources. Good agreements between recorded and predicted earthquake peak ground accelerations were reported by Chavez (1987).

The pile foundations supporting the platforms receive their input energy from earthquakes from two distinctly different points in the soil column (Bea, 1977; 1992; 1993; Bea, et al, 1979; 1987). The input of lateral energy occurs below the sea floor where the maximum lateral earth pressures can be generated on the piles. The input of vertical energy occurs at the point along the pile where the soil - pile shaft load transfer is maximized (Smith, 1994). Based on analyses of pile supported platforms subjected to earthquake excitations in Bay of Campeche soils, a depth below the sea floor of -12 m was chosen to reference the lateral accelerations and a depth of -115 m was chosen to reference the vertical accelerations from the seismic exposure analyses.

Figure 9.7 summarizes the results for the Bay of Campeche for each of the three seismic sources. A mean or average value for the four representative soil columns studied by Chavez (1997c; 1997c) are shown. The Zone 2 and Zone 3 sources develops ground motions that dominate the seismic exposure in the Bay of Campeche. At an average return period of 10,000 years, the indicated peak horizontal ground acceleration (PGA) is indicated to be PGA $\approx$ 0.25 g. Table 7 summarizes the ground motions at return periods of 200 and 1000 years and the uncertainties (Type 1) associated with the ground motions (expressed as the standard deviation of the Logarithms of expected annual PGA, $\sigma_{\ln PGA}$).

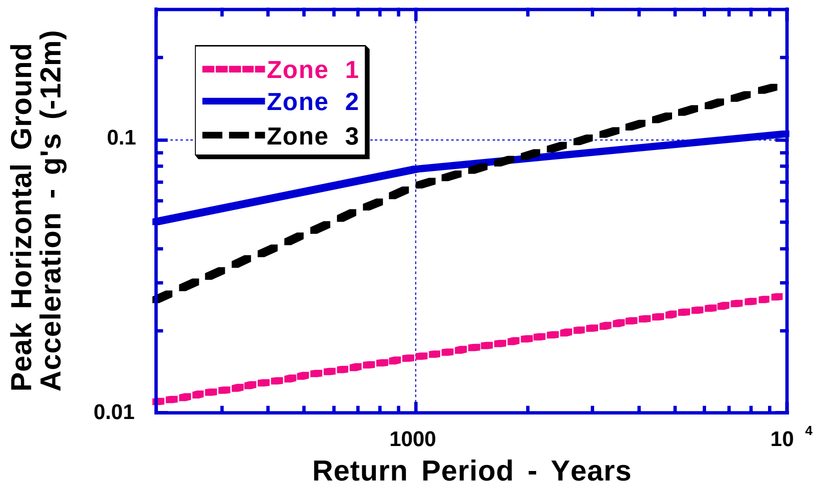

Figure 9.7 - Peak horizontal ground accelerations

The Zone 1 and Zone 2 sources have uncertainties in the expected annual maximum PGA that are comparable with that of offshore California (indicating the potential applicability of the platform seismic criteria contained in API RP 2A and its supplement) (Bea, et al, 1987; Bea, 1996c; 1997b). The Zone 3 (coastal - local) sources have an uncertainty that is comparable with 'intra-plate' seismic zones.

Chavez performed a number of site response studies for the Bay of Campeche using the computer program SHAKE (1997a - 1997d). Results from recently performed high quality soil borings and laboratory static and dynamic testing programs were used to characterize the strength, stiffness, damping, and hysteretic performance characteristics of the soils. Chavez introduced the deconvolved input motions from the three types of earthquakes at the base of the soil columns and determined the output motions close to the sea floor (appropriate for lateral motions characterizations) and at significant penetrations below the sea floor (appropriate for vertical motions characterizations.

Based on these characterizations of the seismic exposure in the Bay of Campeche, the next step in development of seismic criteria was to define the shape of the spectra appropriate for the seismic sources, local geology and soil conditions that can be used in the engineering analyses and evaluations. The site specific seismic exposure results developed by Chavez (1987, 1997a-1997d) were used together with the recently developed recommendations for design spectra of the

International Standards Organization (Craig, 1996; Bea, 1996c; 1997b). The soil condition SC-C (sands, silts, and stiff clays) was used for the sites in the Bay of Campeche. This indicates a spectra shaping soil parameter (ψ) of $\psi = 2.0$ (Crouse, McGuire, 1997). The seismotectonic conditions for the three zones were taken to be a mixed shallow crustal and deep subduction zone. This indicates a spectra shaping tectonics parameter (ε) of $\varepsilon = 0.\ 9$. Thus, the horizontal acceleration response spectra ordinates (Sa) in the long period range ($T \geq 1$ sec) as a function of the PGA is:

$$Sa / PGA = 1.8 / T \qquad \text{Equation 9-6}$$

A 200-year average return period was used for definition of the strength level earthquake (SLE) criteria. The PGA's for this return period are summarized in Table 9.7. The largest PGA (Type 2 earthquakes) is PGA = 5 % g. This is the same value defined in the draft ISO guidelines for the Bay of Campeche (Craig, 1996).

The smoothed Chavez (1997c) and draft ISO (1997) SLE horizontal acceleration response spectra are summarized in Figure 9.8. The peak in the Chavez spectra for Type 2 earthquakes would coincide exactly with the draft ISO spectra for a soil column damping ratio of 20 %. For an implied general soil damping ratio of 10 %, the draft ISO spectra 'clip-off' the Chavez spectra between periods of about 0.8 and 1.5 seconds. The draft ISO spectra imply more energy in the longer period range than indicated by the (Chavez 1997c) spectra, but are consistent with the Chavez 1987 spectra for sea floor response (rather than -12 m). The recent results developed by Chavez (1997d) for increased soil damping and for the second earthquake time history used to evaluate the Type II (or Zone 2) earthquake soil response characteristics would be generally 'enveloped' by the ISO-2 spectrum shown in Figure 9.8.

Table 9.7- Mean horizontal PGA at Bay of Campeche sites (At Elevation - 12 m) and uncertainties of peak horizontal ground accelerations for three seismic source zones

Seismic Zone	200 years %g	1,000 years %g	Uncertainty $\sigma_{\ln Am}$
1	1.2	1.6	0.58
2	5.0	7.8	0.84
3	2.7	6.8	1.75

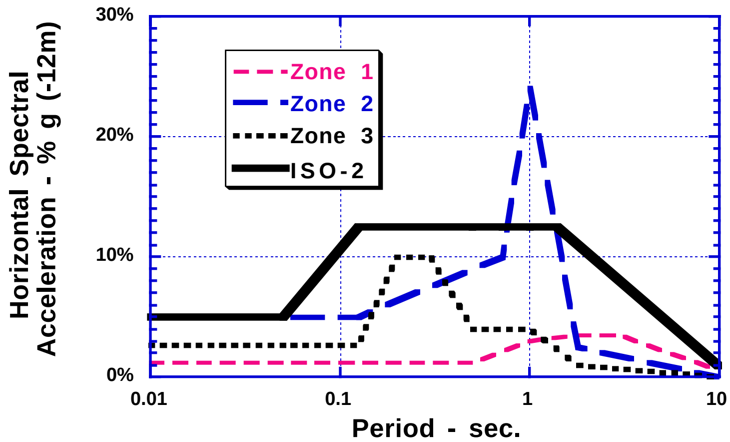

Figure 9.8 - Comparison of smoothed Chavez (1997c) horizontal acceleration response spectra and draft ISO (1997) response spectra

9.4.1 Probability Characterizations

In this development, the uncertainty in the seismic environment was evaluated to be $\sigma_{\ln PGA}$ = 1.1. The uncertainty was determined based on an envelope of the PGA in the return period range of 200 years to 10,000 years for both Zone 2 and Zone 3 earthquakes. The uncertainties contributed by the local geology and soil conditions was taken to be $\sigma_{\ln GS} = 0.5$ (Bea, 1992; 1996c; 1997b). This high uncertainty is to recognize the high uncertainties in the effects of the local soils on the response spectra.

The uncertainties contributed by the platform performance characteristics (mass, stiffness, damping) was taken to be $\sigma_{\ln Ru} = 0.3$. These values are consistent with those developed for the ISO earthquake guidelines (Bea, 1996c, 1997b). The total uncertainty in the induced seismic forces is thus $\sigma_{\ln E} = 1.24$ (square root of sum of squares of 1.1, 0.5, and 0.3).

The effective loading factor, Fe, reflects the ductility and residual strength characteristics of the platform system when subjected to transient loadings from earthquakes (Sucuoglu, et al, 1993; Miranda, 1994; 1997; Bea, Young, 1993). For the platforms in the Bay of Campeche, the ductility and residual strength characteristics of the platforms are assessed to be controlled by the vertical truss bracing patterns. Vertical or horizontal K bracing would have Fe ≈ 0.95 and vertical K with full horizontal framing would have Fe ≈ 0.63 (compared with 0.9 for wave loadings).

The median bias in the maximum earthquake forces was evaluated to be $\mathbf{B_E}$ = 1.0 in development of these criteria. This evaluation is based on the use of 'unbiased' best estimates for all of the critical parameters involved in development of these criteria.

The median bias in the platform capacity when it is subjected to earthquake loadings was evaluated to be $\mathbf{B_{Ru}}$ = 1.4 with an uncertainty of σ_{lnRu} = 0.15. This bias and uncertainty is evaluated to be associated with performance of the vertical diagonal bracing in the jacket structures loaded by the earthquake in compression (Ekngesvik, et al, 1987).

9.4.2 Earthquake Reserve Strength Ratios

The earthquake Reserve Strength Ratios (Bea, 1992; Krawinkler, 1995; Lawson, et al, 1994; Smith, 1994) can be determined from:

$$RSR = (1.0 / 1.4)\ (0.95 \text{ to } 0.63)\ \exp\ (1.25\ \beta - 2.57 \times 1.1) \qquad \text{Equation 9-7}$$

The 2.57 in the foregoing equation is based on the use of a 200-year return period for the SLE (rather than 2.33 associated with a 100-year return period hurricane reference value force). A 200-year value was chosen to be consistent with the SLE earthquakes defined by API (1993; 1997) and ISO (1997) guidelines.

Table 9.8 summarizes the RSR for new and existing platforms. The same annual Safety Indices used in development of the hurricane criteria were used for the earthquake criteria. It was been assumed that all new platforms will incorporate robust diagonal bracing patterns (Fe = 0.63). RSR's for existing platforms have been defined for both the early generation horizontal K bracing patterns (Fe = 0. 95) and the later more robust vertical K bracing patterns (Fe = 0. 63) (Bea, 1997b). Compared with the RSR for hurricanes, the RSR for earthquakes

Table 9.8 - Reserve Strength Ratios (RSR) for ductility earthquake evaluations

SSC	RSR (new)	RSR (existing, vertical K framing)	RSR (existing, horiz. K framing)
1	2.4	1.9	2.8
2	2.0	1.5	2.3

are larger. These more stringent RSR are the result of the greater uncertainties associated with the earthquakes.

9.5 CONCLUSIONS

A full-scope, life-cycle RAM approach has been used to define hurricane and earthquake criteria for design and requalification of platforms in the Bay of Campeche. The RAM approach proved to be very practical and workable (Rey, et al, 1998; Bayazitoglu, 1998)). The RAM approach facilitated re-examination of key premises and procedures to help assure that previously unrecognized biases were evaluated and their influences incorporated into the criteria in an explicit way. Very important sources of bias that were discovered included wave attenuation due to the soft sea floor soils, extreme condition wave - current directionality, and 'enveloping' earthquake spectra. The RAM approach facilitated interactions and communications between the management and platform operations personnel of PEMEX, the engineering and research personnel of IMP, and the contractors and consultants serving PEMEX and IMP in definition of design and requalification criteria that represented a consensus of backgrounds and viewpoints from these various groups (Valdes, Ortega, 1998).

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Chapter 10

Criteria for Design and Requalification of Pipelines in the Bay of Campeche, Mexico

10.1 Introduction

At the present time, there are more than 2,000 km of pipelines located in the Bay of Campeche. These pipelines transport in excess of 2 million barrels of oil and 1.5 billion cubic feet of gas per day. The majority of the pipelines are located in water depths between 30 m and 50 m. Most of these pipelines were installed in the 1980's and 1990's, with some pipelines installed in the late 1970's.

In October 1995, hurricane Roxanne formed in the western Caribbean Sea, crossed the Yucatan Peninsula, and entered the Bay of Campeche. Due to a southward moving front, the hurricane did not follow the normal northerly path of most hurricanes. It was forced back into the Bay of Campeche and the eastern coast of Mexico where it did considerable damage. Roxanne was the most severe hurricane to affect the Bay of Campeche during this century. It generated environmental conditions which approximated those of 100-year return period hurricanes (Oceanweather, 1996). The majority of damage was confined to pipelines (Valdes, et al, 1997). Pipeline damage consisted of broken and leaking connections (above and under water), and damaged weight coatings.

Given the results from the pipeline inspections and fitness for purpose studies, PEMEX (Petroleos Mexicano) and IMP (Instituto Mexicano del Petroleo) initiated development of Risk Assessment and Management (RAM) based criteria for design and requalification of the pipelines in the Bay of Campeche. The RAM based criteria was to take advantage of the results from the oceanographic, pipeline design and inspections, and fitness for purpose studies, recent results from studies conducted by DNV (1996), AGA (1993), ISO (1996), BSI (1993), and API (1993) to develop advanced criteria for design of new pipelines and assessment of existing pipelines. Probability based reliability methods were used to evaluate the full-scope (environmental and operating hazards), life-cycle (design, construction, operations, maintenance, decommissioning) risk characteristics associated with the Bay of Campeche pipelines (*Risk Assessment*) (Sotberg, 1990; Sotberg, Leira, 1994; Sotberg, et al, 1996; Jiao, et al, 1997; 1996; Bai, et al, 1994; Bai, Damslet, 1997; Bai, Xu, Bea, 1997). PEMEX and IMP utilized advanced risk management and decision analysis methods to define how the criteria should be defined to develop acceptable risks (*Management*) (Jiao, et al, 1997; Sotberg et al, 1997; Bea, 1997).

10.2 Serviceability and Safety Classes

Based on information provided by PEMEX and IMP (1997), pipeline security requirements to provide acceptable and desirable safety and serviceability were defined according to three pipeline characteristics: 1) category of contents, 2) location, and 3) importance to production.

Three categories of pipeline contents were defined: 1) **Category 1** - non-flammable fluids or gases, 2) **Category 2** - flammable and/or toxic fluids, and 3) **Category 3** - flammable and/or toxic gases.

Two types of pipeline locations were defined: 1) **Type A** - location pipelines not close to general human activity or major structures, and 2) **Type B** - pipelines close to general human activity, major structures, or shore crossings.

Three categories of serviceability were defined: 1) **Service 1** - has very high implications relative to production associated with failure (loss of containment) of the pipeline or riser - e.g. handles or is a critical part of a pipeline system that handles production in excess of 100,000 barrels equivalent per day, 2) **Service 2** - has normal implications relative to production associated with failure - e. g. handles or is a critical part of a pipeline system that handles production of 50,000 to 100,000 barrels equivalent per day, and 3) **Service 3** - has low implications relative to production associated with failure - e.g. handles production or is a critical part of a pipeline system that handles production of less than 50,000 barrels equivalent per day.

Definition of the Safety and Serviceability Classification (SSC) for a given pipeline or riser are dependent on consequences of failure associated with a pipeline or riser are associated with the potential implications of loss of containment. Three SSC were defined: 1) **SSC 1** - Very High, 2) **SSC 2** - High, and 3) **SSC 3** - Moderate. Table 1 summarizes the SSC based on the combinations of category of pipeline contents, location, and service.

Table 10.1- Serviceability & Safety Classifications (SSC)

Type of Service	Location	Contents Category 1	Contents Category 2	Contents Category 3
1	Type A	**SSC 3**	**SSC 1**	**SSC 2**
	Type B	**SSC 3**	**SSC 1**	**SSC 3**
2	Type A	**SSC 3**	**SSC 2**	**SSC 2**
	Type B	**SSC 3**	**SSC 2**	**SSC 2**
3	Type A	**SSC3**	**SSC 3**	**SSC 3**
	Type B	**SSC 3**	**SSC 2**	**SSC 2**

10.3 Probabilities of Failure and Safety Indices

Three approaches were used to define the probabilities of failure associated with design and requalification criteria for Bay of Campeche pipelines and risers: 1) economics, 2) historic performance, and 3) current standards-of-practice. Application of the economic approach resulted in identification of the total (all causes) probabilities of failure summarized in Table 10.2.

Table 10.2 SSC Probabilities of Failure for Design (Pfo) and Requalification (Pfm)

SSC	Pfo (annual)	Pfm (annual)
1	8.7 E-4 - 2.2 E-4	1.7 E-3 - 4.4 E-4
2	2.9 E-3 - 8.7 E-4	5.8 E-3 - 1.7 E-3
3	1.1 E-2 - 2.9 E-3	2.2 E-2 - 5.8 E-3

The approaches used to develop the SSC probabilities of failure did not include the variabilities or uncertainties associated with modeling or analyzing pipeline or riser demands and capacities. To maintain compatibility between the target reliabilities and the reliability analyses, Type II (model, analytical, epistemic) uncertainties were not included in the reliability analyses. However, the central tendency measure (medians) of the Type II 'biases' (actual value / nominal value) were included to develop realistic evaluations of the central tendency measures of the pipeline and riser demands and capacities.

Two primary hazards to the Ultimate Limit Strength (ULS) of a pipeline or riser were considered in development of these criteria: 1) **Accidents** - unanticipated defects and damage to the pipeline or riser caused by human and organizational errors, and 2) **Operating** - anticipated challenges to the strength (loss of containment) of a pipeline or riser due to internal and external pressures (burst capacity) and on-bottom stability.

The bust capacity of the pipeline or riser could be influenced by internal corrosion, external corrosion, or both (Bai, Xu, Bea, 1997; ASME, 1991; Bjorney, et al, 1997; Kvernvold, et al, 1992; Norland, et al, 1997). The corrosion would be dependent on the protective measures provided to ameliorate corrosion (e.g. inhibitors, coatings) and the projected life of the pipeline or riser.

The total probability of failure (Pft) could be expressed as:

$$Pft = Pfa + Pfo \qquad \text{Equation 10-1}$$

where Pfa is the probability of failure due to accidents and Pfo is the probability of failure due to operating conditions.

In development of these criteria, based on current analyses of the causes of pipeline failures in the Gulf of Mexico (Woodson, Bea, 1990; Mandke, 1990; Mandke, et al, 1995; Marine Board, 1994; Elsayed, Bea, 1997) and in the North Sea (AME, 1991; 1993; 1995; Cannon, Lewis, 1987; E&P Forum 1984; Simpson, 1983; SINTEF, 1989), it was evaluated that Pfa ≈ 0.25 Pft and Pfo ≈ 0.75 Pft.

The probability of failure due to operating conditions could be expressed as:

$$Pfo = Pfs + Pfob \qquad \text{Equation 10-2}$$

where Pfs is the probability of failure due to burst or loss of containment in the pipeline and Pfob is the probability of failure due to on-bottom conditions.

In development of these criteria, based on current analyses of the causes of pipeline failures in the Gulf of Mexico and in the North Sea, it was assumed that Pfs ≈ 0.50 Pfo and Pfob ≈ 0.50 Pfo. This equates to a 'balanced' design of the pipelines for the operating conditions and the environmental conditions.

Based on the foregoing developments, Table 10.3 summarizes the annual probabilities of failure and Safety Indices (β) associated with each of the four SSC for design (new) and requalification of pipelines and risers for strength (burst capacity) and stability (environmental, hydrodynamic, geotechnical).

Table 10.3 SSC and annual probabilities of failure and Safety Indices

SSC	Pf (new)	β (new)	Pf (existing)	β (existing)
1	1 E-4	3.72	2 E-4	3.54
2	5 E-4	3.29	1 E-3	3.10
3	1 E-3	3.10	2 E-3	2.87

10.4 Pipeline and Riser Strength

Pipeline and riser strength was formulated in terms of the capacity of the pipeline or riser to withstand the imposed pressures (internal, external) without loss of containment (rupture). The strength was formulated as (DNV, 1996; ISO, 1996; BSI, 1993; API, 1993; Bai, et al, 1994; 1997; Sotberg et al, 1997; ASME 1991):

$$t / D = p / 2 S \qquad \text{Equation 10-3}$$

where t is the design or existing thickness of the pipeline or riser, D is the diameter of the pipeline or riser, p is the maximum net pressure (internal - external) that the pipeline or riser must be capable of containing, and S is the specified nominal minimum yield strength of the steel used to design or existing in the pipeline or riser.

The API guidelines (1993) specify burst strength as:

$$t / D = (p / 2 S f) \qquad \text{Equation 10-4}$$

where the term 'f' represents the product of three terms: fd (design factor), fe (weld joint factor), and ft (temperature de-rating factor). The design factor is 0.72 for liquid and gas pipelines, 0.60 for liquid pipelines and risers on platforms, and 0.50 for gas pipelines and risers on platforms. The weld joint factor is specified as generally being 1.0 (when welding is conducted according to the specified codes and guidelines). The temperature de-rating factor is used for high temperature pipelines and the de-rating is specified by ASME guidelines (ASME B31.4 and ASME B31.8) (ASME, 1991). The 'f' factor can be interpreted as a factor-of-safety ($FS = f^{-1}$).

In the API guidelines, t is the nominal or design wall thickness of the pipeline or riser. The guidelines specify a number of measures that should be used to prevent corrosion or loss of wall thickness both inside and outside the pipeline or riser. This guideline is based on the assumption that the pipeline or riser operator will provide and maintain the pipeline so that no significant corrosion takes place. A corrosion allowance or thickness could be provided to recognize the need to allow some corrosion to take place without having to de-rate or replace the pipeline. This corrosion allowance is not specified in the API guidelines.

The DNV guidelines (DNV, 1996; Sotberg et al, 1996; Jiao, et al, 1997; Bai, et al, 1994; 1997) specify burst strength as:

$$t / (D-t) = p / (2 \cdot 1.1 \cdot S \cdot \eta_u) \qquad \text{Equation 10-5}$$

where η_u is a usage factor that depends on the safety class of the pipeline or riser. For a High Safety Class, η_u = 0.67. For a Normal Safety Class, η_u = 0.70. For a Low Safety Class, η_u = 0.74. Given the 1.1 that is multiplied times S, these values are very close to those of API.

However, in this case, the 't' that is referenced is the net wall thickness after corrosion has taken place. Corrosion protection can be provided to make this 't' the same as is referenced in API. However, if no protection is provided, a corrosion allowance must be estimated and added to the nominal wall thickness of the pipeline or riser to define the design wall thickness.

10.5 Corrosion

Experience with Gulf of Mexico pipelines and risers (oil and gas) has clearly shown that the primary operating hazard to the integrity of pipelines and risers is corrosion; primarily internal corrosion for pipelines, and external corrosion for risers (generally in the vicinity of the mean water level) (Elsayed, Bea, 1997).

Corrosion of steel in pipelines and risers is a function of what is transported in the pipeline or riser, what surrounds the exterior of the pipeline or riser, and how the corrosion is 'managed' (NACE, 1992). A variety of techniques can be used to reduce the rates of corrosion including internal or external coatings, cathodic protection (for continuously submerged segments of pipelines), dehydration of the gas or oil, and the use of inhibitors. Marine growth tends to inhibit or reduce corrosion of risers (Kvernvold, et al, 1992; NACE, 1992).

For these criteria, the loss of pipeline or riser wall thickness due to corrosion (t_c) was formulated as follows

$$t_c = t_{ci} + t_{ce} \qquad \text{Equation 10-6}$$

where t_{ci} is the loss of wall thickness due to internal corrosion and t_{ce} is the loss of wall thickness due to external corrosion.

The loss of wall thickness due to internal and/or external corrosion ($t_{ci/e}$) was formulated as follows

$$t_{ci/e} = \alpha_{i/e}\, \nu_{i/e}\, (Ls - Lp_{i/e}) \qquad \text{Equation 10-7}$$

where $\nu_{i/e}$ is the average (mean during service life) corrosion rate, $\alpha_{i/e}$ is the effectiveness of the inhibitor or protection (1.0 is perfect protection, and 10.0 is little effective protection), Ls is the service life of the pipeline or riser (in years), and $Lp_{i/e}$ is the 'life' of the initial protection provided to the pipeline.

This model assumes that there are no inspections and repairs performed during the service life of the pipeline or riser to maintain the strength integrity of the pipeline to carry pressure. Maintenance is required to preserve the protective management measures employed (e.g. renew coatings, cathodic protection, and inhibitors). The corrosion management is 'built-in' to the pipeline or riser at the start of the service period. Inspections and maintenance are performed to disclose unanticipated or unknowable defects and damage (due to accidents).

Stated another way, when an existing pipeline is requalified for service, inspections should be performed to disclose the condition of the pipeline and riser, and then an assessment performed to determined if under the then 'present' condition of the pipeline that it is fit for the proposed service. Alternative management of the pipeline could be to de-rate it (reduce allowable operating pressures), protect it (inhibitors, cathodic protection), repair it (doublers, wraps), or replace it.

For design and requalification , the corrosion rate is based on the owner/operators evaluation of the corrosivity of the fluids and/or gases transported inside the pipeline or riser, and of the corrosivity of the external environment conditional on the application of a certain protection or 'inhibition' program. Table 10.4 summarizes suggested median corrosion rates, their variabilities (standard deviations of the logarithms of the corrosion rates, approximately the coefficient of variation of the corrosion rates) and the linguistic variables used to describe these corrosion rates (NACE, 1992).

Table 10.4 - Corrosion rates and viariabilities

Descriptor	Corrosion Rate mm/year	Corrosion Rate Variability - %
Very Low	0.001	10
Low	0.01	20
Moderate	0.1	30
High	1.0	40
Very High	10.0	50

10.5.1 Corrosion Management

In this development, the effectiveness of corrosion management is expressed with two parameters, the inhibitor efficiency ($\alpha_{i/e}$) and the life of the protection ($Lp_{i/e}$). If the inhibitor (e.g. coating, dehydration, chemical inhibitor, cathodic protection) were 'perfect', then $Lp_{i/e}$ would equal 1.0. If experience had indicated otherwise, then the inhibitor efficiency could be introduced as summarized in Table 10.5.

Table 10.5 Inhibitor efficiency

Descriptor	Inhibitor Efficiency
Very Low	10.0
Low	8.0
Moderate	5.0
High	2.0
Very High	1.0

The life of the protection reflects the operator's decision regarding how long the protection that will be provided will be effective at preventing steel corrosion. For example, the life of high quality external coatings in the absence of mechanical damage can be 10 years, where the life of low quality external coatings with mechanical damage can be 1 year or less. Another example would be cathodic protection that could be reasonably provided to protect the pipeline for a period of 10 years, but the expected life of the pipeline was 20 years. Thus, there would be 10 years of life in which the cathodic protection was not provided and the steel would be 'freely' corroding. Table 10.6 defines the general categories of the life of protective systems. This same Table can be used to specify the expected service life of the pipeline or riser (Ls).

Table 10.6 - Expected life of protective system (Lp) or the service life (L_S)

Descriptor	$Lp_{i/e}$ or Ls (years)
Very Short	1
Short	5
Moderate	10
Long	15
Very Long	≥ 20

This formulation could be expressed in terms of 'effective' corrosion rates ($\nu e_{i/e}$) and 'exposed life' ($Le_{i/e}$) as follows:

$$t_{ci/e} = \nu e_{i/e} \, (Le_{i/e}) \qquad \text{Equation 10-8}$$

Figure 10.1 shows the corrosion allowance ($t_{ci/e}$) as a function of the 'effective' corrosion rate (internal and external, $\nu e_{i/e}$) and 'exposed life' ($Le_{i/e}$). The linguistic variables identified in Table 10.4 were used in this illustration. Corrosion thickness allowances for moderate corrosion and long lives are 1 mm to 2 mm. For highly effective corrosion management programs, the corrosion thickness allowances for low effective corrosion rates are 0.1 to 0.2 mm.

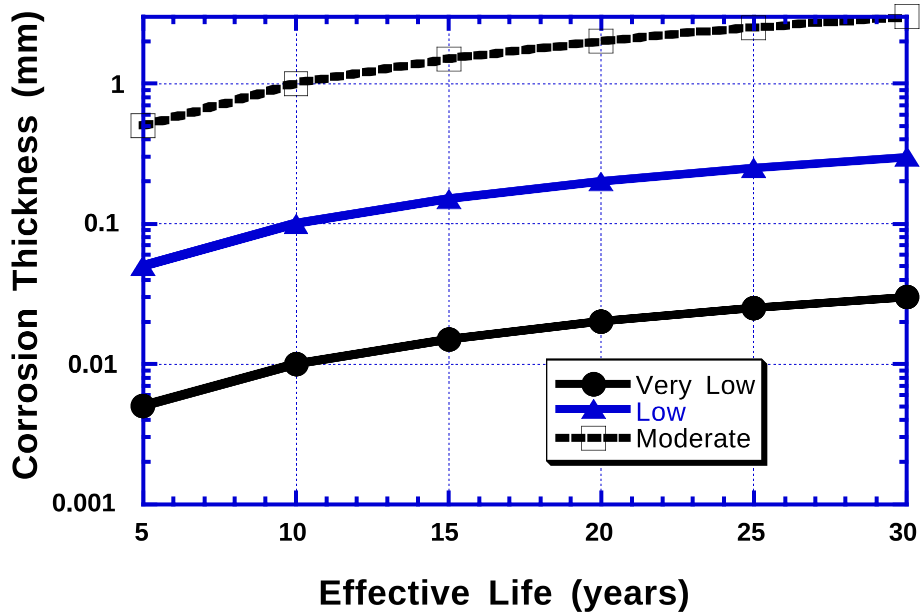

Figure 10.1 - Corrosion thickness allowance for design

Instrumentation or 'smart pigs' can be used to help develop evaluations of corrosion rates and remaining wall thicknesses (Rosen, 1997; Bal, Rosenmoeller, 1997; Shell, 1996). It is important recognize that making evaluations of corrosion rates and wall thicknesses from the recordings have significant uncertainties (Bal, Rosenmoeller, 1997). The measurements can give both 'false positives' and 'false negatives.' The pigs can miss significant defects and indicate the presence of defects that are not present. There are significant uncertainties in the depths and areas of corrosion indicated by the pigs due to such factors as variable temperatures and degrees of magnetism, and the speed of movements of the pig. Specifications for intelligent pig inspections of pipelines need to be developed if consistent and repeatable results are to be realized (Shell, 1996). Corrosion rates are naturally very variable in both space and time. Thus, if instrumentation is used to determine the wall thicknesses and corrosion rates, the uncertainties in these characteristics needs to be determined and integrated into the evaluations of the fitness for purpose of the pipeline.

10.6 RAM Based Formulation

A RAM based formulation of the foregoing developments can be developed as follows presuming that the demand (operating pressure) and capacity (pipeline burst pressure) are Lognormally distributed variables:

$$p_B = 2\ S\ t\ /\ D \qquad \text{Equation 10-9}$$

where p_B is the pipeline burst pressure, S is the stress associated with the burst strength of the pipeline, t is the pipeline wall thickness, and D is the pipeline diameter. The probability of failure is expressed as:

$$Pf = P\ (\ p_O \geq p_B\ ) \qquad \text{Equation 10-10}$$

where Pf is the probability of failure, p_O is the maximum operating pressure, and P (X) is read as the probability of (X). Based on Lognormally distributed pressures and burst capacities, the probability of failure is computed from:

$$Pf = 1 - \mathbf{\Phi}\ (\ \ln\ (p_{B50}\ /\ p_{O50}\ )\ )\ /\ (\ \sigma^2_{pB} + \sigma^2_{pO}\ )^{\ 0.5} \qquad \text{Equation 10-11}$$

where $\mathbf{\Phi}$ is the standard cumulative Normal distribution, p_{B50} is the 50th percentile (median) burst pressure, p_{O50} is the 50th percentile maximum operating pressure, σ_{pB} is the standard deviation of the logarithms of the burst pressure, and σ_{pO} is the standard deviation of the logarithms of the maximum operating pressures. The safety of the pipeline can be expressed as:

$$\beta = \ln\ (p_{B50}\ /\ p_{O50})\ /\ \boldsymbol{\sigma} = \ln FS_{pB/O\ 50}\ /\ \boldsymbol{\sigma} \qquad \text{Equation 10-12}$$

where β is the Safety Index, $FS_{pB/O\ 50}$ is the central or median Factor of Safety between the pipeline burst pressure and the maximum operating pressure, and $\boldsymbol{\sigma}$ is the total uncertainty in the pipeline burst pressure and operating pressure. Thus,

$$p_B = p_O\ (\mathbf{B}_{pO}\ /\ \mathbf{B}_{pB}\ )\ \exp\ (\ \boldsymbol{\beta}\ \sigma\ ) = p_O\ \mathbf{B}\ \exp\ (\ \boldsymbol{\beta}\ \sigma\ ) \qquad \text{Equation 10-13}$$

p_B is the ‘nominal’ burst pressure, p_O is the ‘nominal’ maximum operating pressure, $\mathbf{B}_{pO}$ is the median ‘bias’ in the nominal burst pressure, $\mathbf{B}_{pB}$ is the median bias in the nominal maximum operating pressure, and **B** is the resultant median bias in the nominal burst and operating pressures. Bias is defined as the ratio of the true value to the nominal (predicted, calculated) value.

A nondimensional pipeline wall thickness to diameter ratio can be expressed as:

$$t / D = (p_O / 2 S) (\mathbf{B} \exp (\beta \sigma_{lnp/R}) \qquad \text{Equation 10-14}$$

In this development, the Bias in the demand was taken as $B_p = 1.0$. This bias presumes that on the average that the pipeline will be operated at the design maximum operating pressure.

The Bias in the capacity was taken as $B_R = 2.0$. This bias is based on comparisons of pipeline burst strength tests compared with the burst strengths predicted by the hoop stress formulation used in this development (Figure 10.2, $B_R = 1.7$) (Bai, et al, 1994; Bai, Xu, Bea, 1997), and the strength of the steel at which the pipeline ruptures or looses containment ($B_R = 1.2$) (Jiao, et al, 1997). σ is the total Type 1 (natural, inherent) uncertainty in the demand and capacity elements:

$$\sigma^2_{lnp/R} = \sigma^2_{lnp} + \sigma^2_{lnR} \qquad \text{Equation 10-15}$$

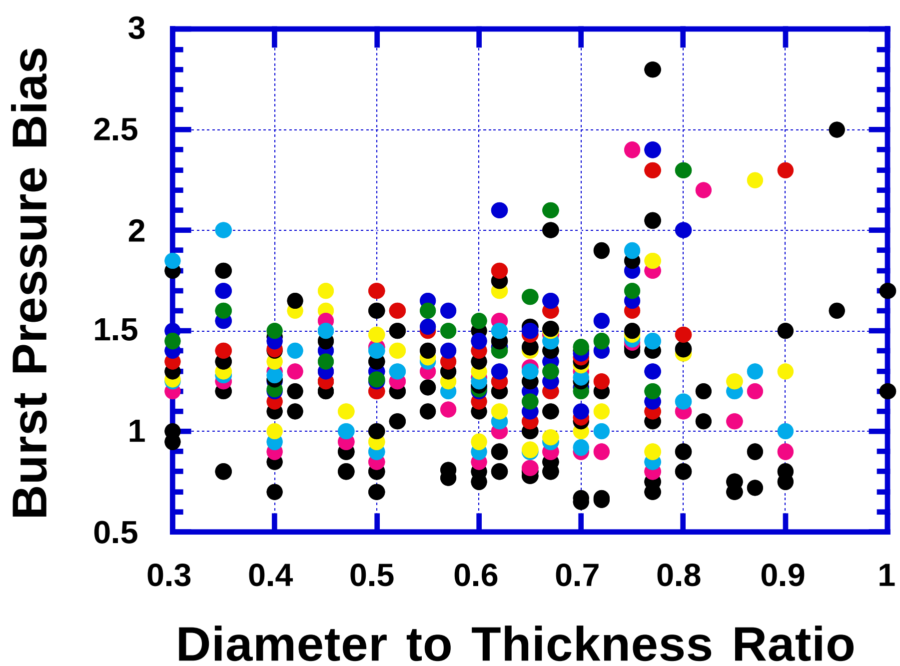

Figure 10.2 - Comparison between measured and predicted burst pressures

The uncertainty in the demand was evaluated to be $\sigma_{lnp} = 0.10$. This variability represents the natural or inherent variability in the pipeline or riser operating pressures (Bai, Xu, Bea, 1997; Sotberg, Leira, 1994). The uncertainty in the capacity was evaluated to be $\sigma_{lnR} = 0.20$. This uncertainty represents the natural or inherent variability in the pipeline or riser burst capacity as influenced by the variability in steel and welding strength (pipeline strength), steel thickness, pipeline or riser diameter, and corrosion thickness. The total or resultant uncertainty was thus evaluated to be $\sigma_{lnp/R} = 0.22$.

Note that this resultant uncertainty has not taken into account the variability added by corrosion damage or defects in the pipeline. Because corrosion has a very high natural variability and the effect of this variability on the burst capacity is also high, the total or resultant uncertainty for a moderately corroded pipeline based on the burst capacity formulation used here could increase to $\sigma_{lnp/R} = 0.40$ to 0.50. For severely corroded pipelines, $\sigma_{lnp/R} = 0.60$ to 0.80.

The Safety Index could be expressed as:

$$\beta = \ln\left((2\,\mathbf{B}\,S / p_O\,D)\,(t - t_{ci/e})\right) / \sigma_{lnp/R} \qquad \text{Equation 10-16}$$

The reliability based dimensionless ratio of pipeline or riser wall thickness to diameter (t / D) exclusive of corrosion thickness allowances can thus be expressed as:

$$t / D = (p_O / 2\,S)\,(\mathbf{B} \exp(\beta\,\sigma_{lnp/R})) \qquad \text{Equation 10-17}$$

$$t / D = (p_O / S)\,((1.0 / 2.0 \cdot 2) \exp(\beta\,0.22) \qquad \text{Equation 10-18}$$

$$t / D = p_O / S\,(0.25 \exp 0.22\,\beta) \qquad \text{Equation 10-19}$$

This formulation allows the dimensionless thickness to diameter ratio of the pipeline or riser (t / D) to be expressed as a function of the dimensionless ratio of the expected maximum operating pressure to specified minimum steel yield strength (p / S) times the exponential of 0.22 times the annual Safety Index (β). The t / D ratio is graphed as functions of p_O / S and β in Figure 10.3 for a total uncertainty (coefficient of variation) of 22 %.

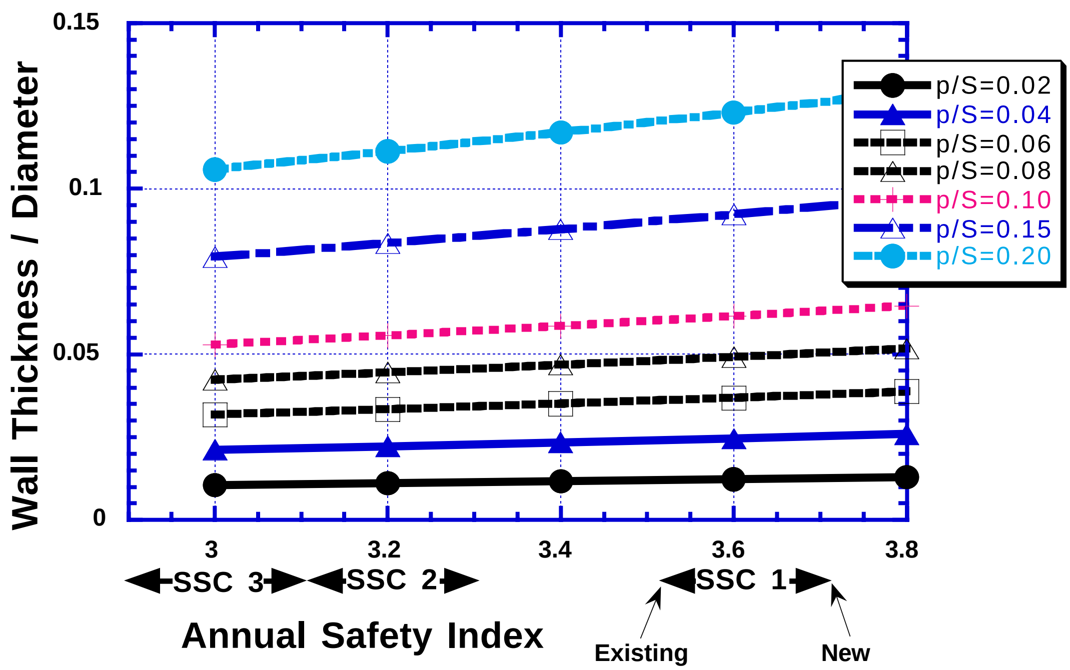

Figure 10.3 - Thickness to diameter ratio as function of ratio of maximum operating pressure to steel nominal yield strength and annual Safety Index

For a given maximum operating pressure to specified minimum yield strength ratio (p / S), for the lower p / S ranges there are small differences between the t / D ratios for new and existing pipelines. There are relatively insignificant differences between the different pipeline Serviceability and Safety Classes. Significant differences show up only for the higher operating pressure to yield strength ratios. Note that the differences between new and existing pipelines shown in Fig. 3 do not incorporate the larger uncertainties associated with existing corroded pipelines.

Fig. 10.4 shows the results for a total uncertainty of 40%. There is a dramatic increase in the required t / D ratios for given ratios of operating pressure to yield strength. In this case, the wall thickness that are referenced are those after corrosion; i.e. they are the minimum wall thickness in a given segment of a pipeline. The increase in required t / D ratios is one of the prices of allowing significant corrosion to develop inside or outside of a pipeline or riser.

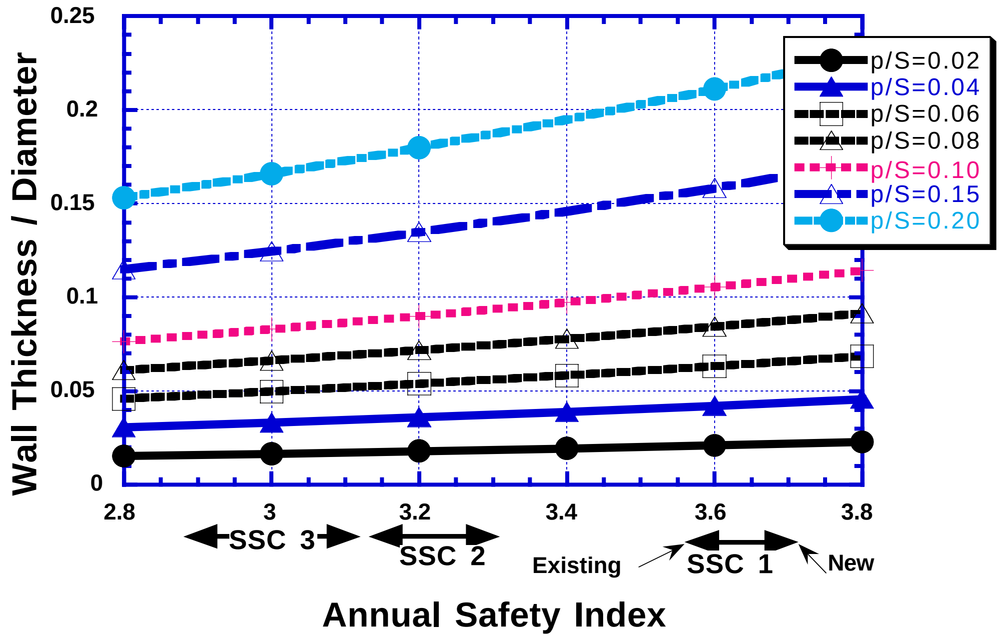

Figure 10.4 - Thickness to diameter ratio as function of ratio of maximum operating pressure to steel yield strength and annual Safety Index for total uncertainty of 40%

10.7 API Guideline Based Design Factors

The foregoing could be cast in the same form as the API guidelines as follows. Based on the RAM formulation:

$$t / D = (p_O / 2\ S)\ (\mathbf{B} \exp(\beta\ \sigma_{lnp/R})) \qquad \text{Equation 10-20}$$

$$p_O = (2\ S\ t / D)\ (\mathbf{B} \exp(\beta\ \sigma_{lnp/R}))^{-1} \qquad \text{Equation 10-21}$$

Based on the API guidelines:

$$t / D = (p_O / 2\ S\ f) \qquad \text{Equation 10-22}$$

$$p_O = (2\ S\ t / D)\ (f) \qquad \text{Equation 10-23}$$

Thus,

$$f = (\mathbf{B} \exp(\beta\ \sigma_{lnp/R}))^{-1} \qquad \text{Equation 10-24}$$

The API based risk assessment and management formulation for the design factor 'f' is summarized in Fig. 10.5. Also shown are the API design factor guidelines for liquid and gas pipelines and platform risers. For the uncertainties associated with new or uncorroded pipelines, the API guidelines result in very high reliability pipelines. However, the performance history of pipelines in the Gulf of Mexico for corrosion failures indicates corrosion failures of 'typical' pipelines at the rate of 2 E-2 to 5 E-2 per year (Mandke, 1990; Mandke, et al, 1995; marine Board, 1994; Elsayed, Bea, 1997). This is equivalent to annual Safety Indices in the range of β = 1.5 to 2. This Safety Index range is commensurate with the uncertainties associated with corroded pipelines. The analytical models indicate probabilities of failure that agree well with the performance history.

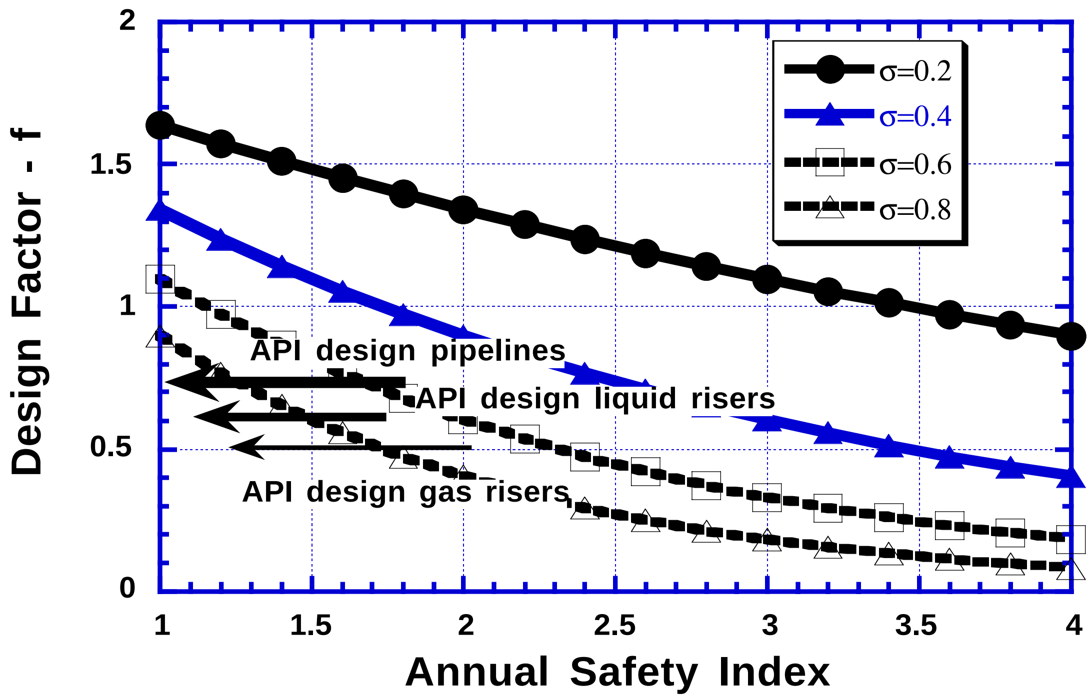

Figure 10.5 - RAM based API burst pressure design factors

Table 10.7 and Table 10.8 summarize the API based RAM design factors for burst pressure for each of the three SSC, for new and existing pipelines, and the uncertainties and biases evaluated during this study.

The new pipelines were based on total uncertainties of 24 % (very low corrosion), 30 % (low corrosion), and 37 % (moderate corrosion). The API 'f' design factors for gas risers (highest importance), liquid risers (next importance), and subsea oil and gas pipelines are 0.5, 0.6, and 0.72, respectively.

The annual Safety Indices associated with the API design factors could be determined from:

Table 10.7 - API burst pressure factors for new pipelines

SSC	β new	f very low corrosion	f low corrosion	f moderate corrosion
1	3.72	0.82	0.66	0.51
2	3.29	0.91	0.75	0.59
3	3.10	0.95	0.79	0.64

Table 10.8 - API burst pressure factors for existing pipelines

SSC	β existing	f very low corrosion	f low corrosion	f moderate corrosion
1	3.54	0.86	0.69	0.54
2	3.10	0.95	0.79	0.64
3	2.87	1.00	0.85	0.69

$$\beta = \ln (l/f\,B) / \_lnp/R \qquad \text{Equation 10-25}$$

Given this development, and an assessment of a total uncertainty of 50% for API pipeline design (allowing for corrosion uncertainties), a median bias in the pipeline demand and capacity of B = 2.0, one could determine the annual Safety Index for pipeline design implied by the API guidelines as β = 2.0 (Pf ≈ 1 E-2 per year) for subsea oil and gas pipelines, β = 2.4 for oil risers, and β = 2.8 for gas risers. These values are in excellent agreement with the performance characteristics of pipelines and risers in the Gulf of Mexico (Pf ≈ 1 E-2 per year).

10.8 DNV Guideline Based Usage Factors

The API 'f' design factors are comparable with the DNV 'ηu' usage factors (Table C2, p 28 in DNV guidelines)

$$f = 1.1\, \eta u \qquad \text{Equation 10-26}$$

The DNV usage factors for High, Normal, and Low Safety Classes are 0.67, 0.70, and 0.74, respectively.3 This would be equivalent to API ‘f’ factors of 0.73, 0.77, and 0.81. The ‘f’ factors developed here would be 0.79, 0.88, and 0.92.

The lower DNV usage factors reflects the lower Pf’s or higher β’s utilized in development of the DNV guidelines. The DNV usage factors do not include any corrosion degradation or corrosion thickness allowances incorporated into the pipeline design. However, the DNV guidelines were based on much lower uncertainties than utilized in this development. The background literature indicates total uncertainties associated with determination of the DNV usage factors that are of the order of 10 % to 20 %.

Given a total uncertainty of 25%, the DNV usage factors could be used to determine implied annual Safety Indices of β = 4.4 for High, β = 4.2 for Normal, and β = 4.0 for Low Safety Classes. For a total uncertainty of 50%, the DNV usage factors could be used to determine implied annual Safety Indices of β = 2.2 for High, β = 2.1 for Normal, and β = 2.0 for Low Safety Classes. The implied levels of reliability are very strongly influenced by the assessment of uncertainties associated with the pipeline and riser life-cycle (design, construction, maintenance, operation). This suggests an important need to couple the specifications of target reliabilities and uncertainties associated with determination of the design factors (Sotberg, et al, 1997; Bea, 1997).

10.9 Time Dependent Pipeline Reliability

The pipeline reliability is a time dependent function that is dependent on the corroded thickness of the pipeline (tci/e). The corroded thickness is dependent on the average rate of corrosion and the time that the pipeline or riser is exposed to corrosion. This time dependency can be clarified with the following

$$\beta = \ln (Kp\ t - Kp\ tci/e)) / \sigma_{lnp/R} \qquad \text{Equation 10-27}$$

where

$$Kp = (2\ B\ S / p\ D) \qquad \text{Equation 10-28}$$

If one defines

$$Kp\ t = FS_{50} \quad \text{Equation 10-29}$$

where FS_{50} is the median factor of safety in the burst capacity of the pipeline or riser. Then:

$$\beta = \ln\left(FS_{50} - FS_{50}\,(t_{ci/e} / t)\right) / \sigma_{lnp/R} \quad \text{Equation 10-30}$$

As the pipeline corrodes, the reduction in the pipeline or riser wall thickness leads to a reduction in the median factor of safety that in turn leads to a reduction in the Safety Index (or an increase in the probability of failure). In addition, as the pipeline corrodes, there is an increase in the total uncertainty due to the additional uncertainties associated with the corrosion rates and their effects on the burst capacity of a pipeline or riser.

An analytical model for the increase in total uncertainty as a function of the corrosion could be expressed as:

$$\sigma_{lnp/R}|t = \sigma_{lnp/R}|to\ (1 - t_{ci/e} / t)^{-1} \quad \text{Equation 10-31}$$

where $\sigma_{lnp/R}|t$ is the uncertainty at any given time 't', $\sigma_{lnp/R}|to$ is the uncertainty at time t = 0, $t_{ci/e}$ is the corroded thickness and t is the initial thickness. When $t_{ci/e} / t = 0.5$ the initial uncertainty would be increased by a factor of 2. Results for $\sigma_{lnp/R}|to = 0.2$ and = 0.30 and $FS_{50} = 2.0$ (same as median bias used previously) are summarized in Figure 10.6.

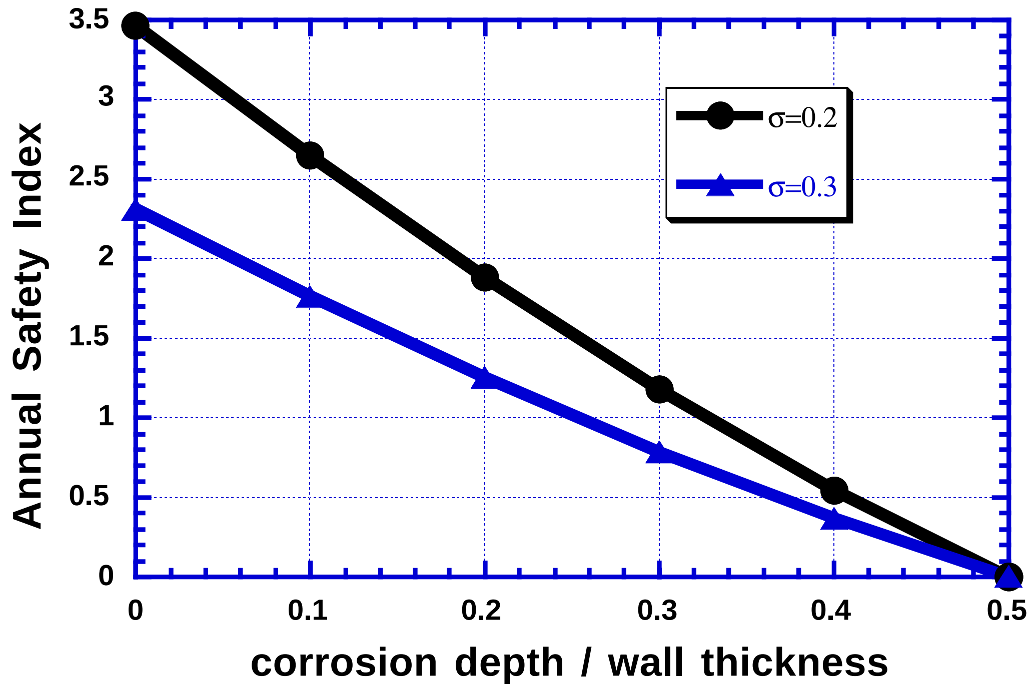

Figure 10.6 - Influence of corrosion depth and uncertainty on annual Safety Index

High quality assurance and control in the pipeline reliability management leads to lower uncertainty and higher reliability. Given corrosion, there is a decrease in the reliability of the pipeline as a function of time reflected in the depth of the corrosion normalized by the wall thickness. If the target reliabilities are defined as those that the pipeline should not be lower than during its life, then either corrosion protection must be provided to preserve the initial thickness of the pipeline or riser, or corrosion allowance must be added to the pipeline or riser initial thickness, or a combination of these two measures. For example, if an annual Safety Index of 2 during the pipeline life were desired, and the initial uncertainty associated with the pipeline demands and capacity were 20%, then the corrosion allowance would need to be 20% of the pipeline thickness. This would result in an initial annual Safety Index of $\beta = 3.5$. Given the projected corrosion rate for the life time of the pipeline or riser, the annual Safety Index would decrease to 2.0 by the end of the projected life.

Pipeline Inspection, Maintenance, and Repair (IMR) programs should be an essential part of the design and life-cycle management of pipelines (Bai, Damsleth, 1997; Nordland, et al, 1997; Bea, Xu, 1997). However, many pipelines are simply designed and it is ‘assumed’ that the pipeline will be ‘adequately’ maintained. In these criteria, it is apparent that a pipeline IMR program is an explicit part of the design or requalification of a pipeline.

For development of pipeline IMR programs, PEMEX and IMP are developing an RCM (Reliability Centered Maintenance) approach (Bea, Xu, 1997; Jones, 1995). This approach was developed initially for the commercial airframe industry. The RCM approach has found applications offshore in developing maintenance programs for equipment, piping, and production control systems (Jones, 1995).

10.10 Conclusions

Background for design and requalification of pipelines and risers to assure leak integrity have been developed during this study. A critical element in these RAM based criteria is the evaluation of biases and uncertainties associated with analyses of pipelines and risers in corrosion conditions. This evaluation must be closely coupled with the definition of the target reliabilities so that there is a consistency between the types of uncertainties included in the target reliabilities and those included in the reliability based determinations of design and requalification factors.

Design and requalification of pipelines and risers to assure leak integrity is intimately related to the Inspection, Maintenance, and Repair (IMR) program that is developed and implemented for a particular pipeline or riser. Once IMR programs are identified, then they can be associated with the design and requalification of specific pipelines and corrosion allowables determined. RCM (Reliability Centered Maintenance) approaches can be used to develop pipeline and riser IMR programs.

Criteria to assure pipeline and riser leak integrity have been based on traditional load and stress based procedures. Deformation / strain based procedures hold the promise for providing more realistic and less conservative results.

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