Proceedings of the International Workshop on

SEISMIC DESIGN AND REASSESSMENT OF OFFSHORE STRUCTURES

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OBJECTIVES AND SCOPE OF THE WORKSHOP

The objectives of this workshop were to define the state-of-the-art of seismic design and reassessment of offshore structures and facilities, and to determine directions for future research in this area. The issues covered by the workshop spanned a broad spectrum of scientific and engineering issues including: estimation of local seismicity; prediction of strong ground motion; analysis of the dynamic response of structure, foundation, and soil systems; structural modeling; analysis of the response of equipment and structural appendages; reassessment, requalification and retrofit strategies; and public policy issues.

PREFACE

There are presently more than 4,000 offshore structures in the coastal waters of the United States and many more in other parts of the world. These structures have been constructed in water depths ranging from 6 meters to more than 300 meters. Some are more than 30 years old and many were designed according to criteria that fall short of today's standards. In spite of this, there is great pressure to extend the useful life of these structures so as to be able to recover the valuable remaining reserves of oil and gas.

The aging of offshore structures poses a number of serious challenges to the oil and gas industry and to the agencies which oversee and regulate their use. Not the least of these challenges is to establish guidelines for the reassessment and requalification of existing offshore structures in light of presently available knowledge of environmental loads and structural performance. For structures located within the Pacific Rim earthquake zone, and in other seismic regions of the world, earthquakes are one of the primary environmental loads which must be considered in any reassessment.

The International Workshop on Seismic Design and Reassessment of Offshore Structures was held at the California Institute of Technology on December 7–9. 1992. The Workshop was attended by 123 individuals from government, industry, and academia. The Workshop was organized around the five topic areas: 1) site seismic hazard and ground motion, 2) design, reassessment and requalification, 3) structural performance, 4) operations, and 5) public policy. Invited state-of-the-art lectures were presented in each topic area. Working groups further developed each topic and identified needed future action.

These Proceedings contain the findings, recommendations, and conclusions of the Workshop participants. The Executive Summary was prepared by the Editor. All other portions of the Proceedings were prepared by the relevant lecturer or working group co-chairs with only minor editing.

EXECUTIVE SUMMARY

This Executive Summary presents the major conclusions and recommendations of the International Workshop on Seismic Design and Reassessment of Offshore Structures. Full versions of the text of all invited lecturers and complete working group reports are contained elsewhere in these Proceedings.

Site Seismic Hazard and Ground Motion

Continuous improvements have been made in the state-of-the-art of estimation of seismic ground motion for design and reassessment of offshore platforms. C. B. Crouse of Dames & Moore provides an overview of methods used to determine seismic design parameters for offshore structures. A site-specific analysis is generally performed to develop seismic design parameters for a particular site. The first step is to define the seismic sources in the site region, and develop a model of the attenuation of ground motion that is appropriate for the region. Next, probabilistic and/or deterministic analyses are performed to estimate the ground motion parameters. For new designs, both approaches are commonly used. For reassessment and requalification, a probabilistic approach is generally preferred. There are three basic inputs needed for a probabilistic approach; 1) identification of earthquake sources in the site region, 2) definition of the earthquake recurrence for each source, and 3) definition of the attenuation of earthquake ground motion, usually expressed as a function of distance from the source and earthquake magnitude.

An example is given of a typical offshore platform located in a subduction zone environment. This example is used to illustrate some of the more important issues in the determination of seismic parameters. These issues include: extrapolation from available data to larger design level events, inclusion of basin effects on soil amplification, the relationship between vertical and horizontal earthquake motions, and the effect of the water column on vertical acceleration.

It is concluded that estimation of ground motion for offshore structural design and reassessment is a challenging process involving several technical disciplines including seismology, geology, geotechnical and structural engineering, as well as political considerations. Unfortunately, many uncertainties still exist in the specification of seismic design parameters based on site-specific seismic hazard analyses.

The Working Group on Site Seismic Hazard and Ground Motion, co-chaired by Allin Cornell of Stanford University and Paul Somerville of Woodward Clyde Consultants, makes the following major conclusions and recommendations:

1. There is a critical need for more recorded earthquake seafloor motion data in regions where offshore facilities are proposed or currently constructed.

2. There is a need for the application of the latest state-of-the-art techniques in the prediction of ground motion, especially in the period range of 1–4 seconds.

3. Further research should be undertaken related to defining the subsurface input reference point for strong ground motion.

4. Deep geophysical data is needed to better characterize offshore seismic sources.

Design, Reassessment And Requalification

Robert Bea of the University of California observes that the primary objectives of a seismic design methodology are to assure that a new offshore structure will have sufficient strength and ductility to satisfy its intended purposes without undue expense or risk. He argues that the primary focus of the platform designer should be on the response that develops after first significant yielding occurs in the structural elements and components that comprise the structural system of the platform, and the damage states that can lead to collapse of the platform. In the design criteria, member resistance and factors should be chosen to provide acceptable reliability against significant damage or collapse. A strength level earthquake is not anticipated to induce significant damage or inelastic response in the structural elements.

Static push-over analyses have been used extensively to demonstrate the capacity and ductility of offshore platforms. The margin of safety beyond the elastic performance requirement is often expressed by a platform Reserve Strength Ratio (RSR) which is the ratio of the maximum lateral load capacity of the platform to a reference load induced by the strength level earthquake. Bea has found that platforms designed according to current American Petroleum Institute (API) guidelines generally have an RSR ≥ 2.

In contrast to a seismic design methodology, a seismic requalification methodology should provide a set of processes and parameters to insure that an existing platform will be able to develop acceptable performance during its proposed service period. Bea summarizes his "comprehensive" seismic requalification methodology. In this methodology, the judgment of fitness for purpose is based on the total probability of failure of the platform system, including topside equipment and operations, the potential costs, and the consequences associated with failure. If a platform is judged not fit-for-purpose, guidance is provided as to how to improve its performance characteristics.

Two different approaches may be used to define the fitness-for-purpose criteria in this methodology: "utility optimization" or "standard of practice." The utility optimization approach is based on an evaluation of the positive and negative utilities associated with different risks. Utility may be measured in a variety of ways. Costs and benefits are commonly measured in monetary terms. The objective is usually expressed in terms of minimization of total expected cost.

The standard-of-practice approach is based on current design and requalification decisions which have been made for other platforms. The underlying premise of this approach is that, over time, the profession, industry, and regulatory agencies have defined acceptable combinations of probabilities of failure and consequences associated with failure. In the standard-of-practice approach, the decision on whether a structure is fit-for-purpose is based upon its probability of failure and total cost of failure as compared to other new and existing platforms. Straight line "acceptable" and "marginal" boundaries have been proposed in a log-log total probability vs. total cost space.

Bea expresses the opinion that none of the available seismic requalification methodologies are as highly developed as seismic design methodologies. It is argued that the industry has the necessary technology to develop definitive engineering guidelines from the available requalification methodologies and that high priority should be given to developing such guidelines. It is further asserted that generally accepted fitness-for-purpose criteria that are dependent on consequences and likelihoods of failure need to be developed for both new and existing offshore structures.

David Wisch of Texaco Corporation presents the results of a 1992 API requalification project. A panel, consisting of Wilfred Iwan (Chairman), Allin Cornell, George Housner and Charles Thiel, was constituted by the API to recommend an approach to seismic requalification of offshore structures. The API Panel's strategy is based on the establishment of separate acceptable levels of risk for life safety and environmental hazards. The Panel reasoned that life safety and environment hazards are essentially different in character and need not, indeed cannot, be measured by a common scale such as dollars. The Panel's acceptable risk for life safety is based on the concept that the risk to a worker on an offshore platform should be comparable to that for a worker in a similar facility onshore. This was believed to be a publicly acceptable level of risk. Acceptable environmental risk is based upon the risk of spills that occur from other sources in the same region which are deemed publicly acceptable. The Panel uncovered no safety or environmental issues associated with the seismic safety reassessment process that indicated platforms should be subject to risk criteria more restrictive than those for land-based industrial facilities.

The Panel argued that the focus of seismic safety reassessment should be on limiting, to an acceptable level, the risk due to catastrophic impacts of earthquakes, where a catastrophic impact is defined as one that has unacceptable large life and/or environmental safety consequences. It was further argued that offshore facilities should have more rigorous site hazard and engineering behavior analysis than onshore facilities in order to achieve these goals, even though they have comparable quantitative risk limits. The Panel recommend that decisions related to cost and economic return be left to platform owners and not be included in reassessment or requalification requirements imposed by regulatory agencies.

The Panel concluded that use of the design and analysis quidelines presented in the API recommended practice document, RP 2A, 19th edition, will produce a structure with life safety comparable to that of well-engineered structures onshore provided that: 1) the hazard study and structural analyses are peer reviewed, 2) the seismic hazards are determined in accordance with strength and ductility level earthquakes with 200 and 1,000 year return periods, respectively, 3) a ductility level earthquake analysis is always performed, and 4) proper allowance is made for life-safety risks associated with platform appurtenances.

For structures which do not meet the guidelines of RP 2A, the Panel determined that an objective of an annual probability of failure (collapse) of less than 1×10^{-3} would be consistent with providing a level of life safety comparable to that of onshore structures. The Panel also concluded that median value results from hazard analyses should be used in any probabilistic analyses to verify satisfaction of this objective. For Southern California waters, the Panel recommended an environmental performance objective of a release of no more than 2,000 barrels from any possible source including wells, pipelines, and onboard storage.

Wisch notes that the API Panel study is only one of a series of steps being undertaken by the offshore industry. Some of the questions that are being and will be answered by these studies include: 1) whether platforms can be categorized as to their expected response based on type or age, 2) whether an effective screening procedure can be developed to avoid detailed case-by-case assessment of offshore structures, 3) how independent review and peer review should best be incorporated into the reassessment process, 4) the different issues that must be addressed in reassessment outside the U.S., and 5) how acceptability criteria should relate to recommended industry practice, codes, regulatory requirements, and economic considerations.

The Working Group on Design, Reassessment and Requalification, co-chaired by Kris Digre of Shell Oil Co. and William Ibbs of U.C. Berkeley, arrives at the following conclusions and recommendations:

1. The strength level and ductility level earthquakes employed in current API recommended practice need to be more precisely specified; perhaps tied to some other criteria.

2. A ductility level analysis should always be required.

3. Accelerographs should be installed in all offshore structures for the purpose of acquiring data on structural performance during actual earthquake excitation.

4. Technology should be further developed and shared regarding the following: joint and member capacities (including in-frames), mass coefficients for wave loading, effects of marine growth on structural response, and the relation between static and dynamic load analysis procedures.

5. A majority of the working group felt that there should be a separation of life safety, environmental consequences, and economic decisions for requalification but a consensus on this matter could not be reached.

6. A full probability-of-failure risk analysis should be performed rather than a mere determination of the survival of a structure for a given return period earthquake.

7. A strength level earthquake analysis need not be performed for requalification as long as a ductility level analysis is performed.

8. A careful peer review should be conducted of both the structural and seismic hazard elements of any design, reassessment, or requalification process. More work is needed to develop an appropriate peer review process and guidelines for reviewer qualification.

9. Research is needed to more precisely define manned and unmanned operations, and catastrophic consequences of environmental pollution.

10. There is a need to build greater consensus on the appropriate criteria for safety goals for design and requalification. Further research is needed on such issues as: specification of life safety and environmental safety objectives, the use engineering judgment, and how to properly incorporate the consequences of failure into any performance objective.

Structural Performance

Daniel Dolan of PMB Engineering uses case studies to demonstrate that methods traditionally used to determine the structural safety of offshore platforms in hurricane and storm environments may result in erroneous conclusions when applied to platforms in regions of high seismicity. In regions of low seismicity, reassessment analyses concentrate on evaluating structural response under wave loading conditions. Older platforms in nonseismic regions are in shallow to medium water depths and exhibit only limited dynamic response. Therefore, analysis procedures are usually based on calculation of the Reserve Strength Ratio. According to Dolan, use of the Reserve Strength Ratio to evaluate the performance of platforms subjected to extreme earthquake loading can result in potentially misleading conclusions.

Present seismic design practice for new offshore platforms focuses on two primary objectives: providing sufficient strength and stiffness to insure that no significant structural damage occurs for earthquakes having a reasonable likelihood of occurrence during the lifetime of the structure, and providing reserve strength and/or ductility to prevent collapse during rare earthquake events. These objectives are typically satisfied by examining the performance of the structure under the action of a strength level and a ductility level earthquake. Older offshore platforms may have three areas of deficiency: 1) inadequate design ground motion levels, 2) inadequately arranged or detailed structural framing, and 3) reduced capacity resulting from damage, corrosion or fatigue degradation. The reassessment of existing offshore structures is a relatively new process which has not yet been standardized to the same extent as the design of new structures. This lack of standardization creates problems in establishing performance requirements on other than a case-by-case basis.

There are two general forms of reassessment analysis: 1) static pushover analysis, and 2) time domain analysis. In both analyses, large deformation modelling must be included if the collapse mechanism is to be accurately modeled. A static pushover analysis may be used to establish the static ultimate strength of the platform and the failure sequence for the selected loading pattern. It may also be used to quantify the energy absorption of the structure for the selected loading pattern and to identify weak or critical elements in the structure. However, important aspects of structural response such as load reversal, soil and structure cyclic degradation, changes in vibrational characteristics due to nonlinear response, and soil and structure hysteresis may not be accounted for through a static pushover analysis. These features can be extremely important in determining the reliability of a platform under extreme earthquake loading.

Another fundamental limitation of the static pushover analysis is that it does not recognize the difference between brittle and ductile response behavior. Experience and analyses have proven that a platform with greater ductility will withstand much more severe ground shaking than one that responds in a brittle manner.

Time domain analysis is used to determine the reliability of a structure for a given loading which is assumed to be characteristic of the site ground motion. Such an analysis provides a basis for evaluating critical response behavior including local vibration of structural elements. Through time domain analysis, it may be found that a platform which fails the strength level earthquake requirement may pass the larger ductility level earthquake requirement due to the energy absorbed by hysteretic action of structural members and the soil. The peak base shear may even exceed that predicted from static pushover analysis for the same structural model without the structure collapsing.

A complete reassessment of an offshore platform must also examine the response of topside facilities in addition to determining the likelihood of structural collapse. Topside risk can be greatly reduced by cost effective mitigation measures such as fire protection and other safety precautions.

Dolan concludes that a significant amount of research and development work has been accomplished in the area of reserve strength assessment for new and existing platforms. However, inadequate progress has been made in the analysis of offshore platforms in seismic regions for the purpose of reassessment. Additional industry effort will be needed to provide adequate standardization of the procedures and performance objectives for seismic reassessment. Furthermore, additional testing should be undertaken since much of the current input to reassessment analysis is based on information which is unsubstantiated by either laboratory or field tests.

The methodologies used for seismic reassessment of offshore platforms must necessarily differ from those used for platforms subjected to hurricanes and other types of storm loading. In most cases, the reassessment philosophy for nonseismic regions is based upon the premise that some warning will exist prior to the event allowing for the shutdown of operations and evacuation of the platform. In such cases, the consequences of failure are predominantly economic. On the other hand, earthquakes generally occur without warning and the peak response of the structure occurs very quickly making shutdown and evacuation difficult or impossible. Reassessment objectives for platforms in high seismic regions must therefore include the life loss and environmental consequences of failure.

Michael O'Neill of the University of Houston provides a general overview of issues related to the performance of pile foundations for offshore structures. He points out that the reassessment of pile foundations differs from design in a number of important aspects including: 1) indirect data on pile capacity exists, 2) foundation performance under past environmental loading can be inferred through system identification, 3) loads to be resisted by the piles through quasi-static loading and through superstructure feedback are not the same for reassessment and design, and 4) piles may have suffered damage or deterioration since installation.

One of the important issues related to the analysis of pile behavior is whether degrading and rate dependent models are required in determination of the Reserve Strength Ratio. For the large monotonic pile deflections used in Reserve Strength Ratio analyses, which may greatly exceed the deflections that occur in a seismic event, the cyclic loading has no effect on the lateral soil resistance. Therefore, if structural failure does not occur in any pile until such deflections are achieved throughout the system, consideration of degradation is unnecessary. However, rate effects may still need to be considered. On the other hand, if structural collapse or pullout occurs in any pile at small deflections, within the range of seismic loading, cyclic degradation criteria may control, and the Reserve Strength Ratio could be very small.

Another important issue is the effect of combined degradation, loading rate, and pore pressure generation. Seismic loadings can produce both degradation and rate effects in sands. Degradation can occur due to reduction in lateral effective stresses against the pile resulting from volume changes in the soil around the pile, and due to the generation of pore water pressure resulting from the passage of seismic waves and the induced motion in the pile. This behavior, coupled with structural loading, may cause pile failure. An area that needs further study is whether the vertical component of seismic waves produces vertical compressional waves in the ocean floor that can induce pore water pressures in the soil not accounted for by standard upward propagating wave analyses.

Still another important need is to arrive at unbiased (non-conservative) estimates of static lateral and axial pile capacities. A rational assessment of the suitability of pile foundations to withstand seismic loadings should be based upon the most likely static axial capacities of the piles as well as there most likely lateral behavior. In the design of a new structure, such an approach is not necessary as conservative estimates usually suffice. A similar observation may be made for stiffness computations, except that apparently conservative (soft) estimates of pile stiffness may result in unconservative predictions of structural response. It is virtually impossible to eliminate bias from estimates of pile capacity at a given site without direct capacity measurements at that site.

Other issues which must be considered in the design and assessment of pile foundations include: 1) determining whether shallow flow slides can occur during seismic events for sites on sloping seafloors, 2) assessing whether group action will have an effect on pile response, 3) assessing group effects for quasi-static overload analyses, 4) assessing the probability of the occurrence of significant storm loadings during significant seismic events and their combined effects on pile response, and 5) estimating the soil properties at the actual site of construction of the structure based upon soil borings at some distance from the site.

The Working Group on Structural Performance, co-chaired by Hugh Bannon of Exxon Corporation and Joseph Penzien of International Civil Engineering Consultants, makes the following recommendations for needed future work:

1. A study should be initiated to evaluate the lower bound joint capacity equations proposed by current design guidelines.

2. The industry should support experiments designed to better define the strength degradation and energy absorption capacity of joints subjected to cyclic loads.

3. Simple guidelines need to be developed for modeling and analysis of damaged members. Additional testing and analytical research may be needed.

4. Further analytical and experimental work is needed to reduce the uncertainty in pile capacity predictions which is generally greater than that for structural resistance evaluations. This includes: studies to evaluate the loss of strength due to local pore pressure buildup adjacent to a pile, studies to develop a better understanding of axial and lateral capacities of piles in clusters, studies of the effect of soil aging on pile capacity, studies of the reduction in axial capacity of a pile due to lateral motion, and studies of the applicability of the plasticity index as an indicator of cyclic response characteristics for offshore pile foundations.

5. Additional studies are needed in the following areas: guidelines for the necessary level of detail in structural analysis, review of load combination guidelines, studies of the adequacy of the static pushover method for seismic analysis, and establishment of failure criteria for members, joints, and systems for use in time history analysis.

6. There is a crucial ongoing need for communication and interaction between structural engineers, seismologists, and geotechnical engineers in order to avoid possible errors of omission or misapplication.

7. There is an urgent need for more seismic instrumentation and data-recording systems offshore and for the disclosure of existing data to be used on an industry-wide basis.

Operations

Robert Visser of Belmar Engineering reviews the potential earthquake hazards to process equipment, pipeline risers, drilling rigs and cranes, living quarters, and wells and conductors. He concludes that from both quantitative and qualitative analyses, the risk of topside earthquake damage is low provided all equipment and piping are adequately restrained and drilling rigs and other slender structures are designed to appropriate earthquake loading.

The seismic reassessment of a topside platform facility involves the analysis of a complex system which includes people, hazardous materials, equipment, and safety systems. The potential hazards to a platform operation from an earthquake are: 1) process equipment damage, 2) pipeline riser rupture, 3) drilling rig or crane collapse, 4) quarters building collapse, and 5) well or conductor failure. In the absence of statistical earthquake damage information, the earthquake risk assessment of an offshore platform facility must be based on a qualitative assessment.

Reducing the risk of earthquake damage to process equipment is easily accomplished on most platforms. As most equipment items are rigid bodies, they can be tied to the deck or to each other such that there is no movement even in the most severe earthquakes. Design and assessment guidelines have recently been developed by an API subcommittee to assist in this evaluation. Onshore experience indicates that most process equipment damage is related to storage tank failures. Such failures are readily preventable.

Pipeline risers are normally clamped to a platform structure. The primary sources of riser failure are falling objects, platform structural failure, or a mudslide. Mitigation of this hazard can be undertaken through the installation of automatic shut-off valves.

There are a number of slender structures on a platform that could collapse during an earthquake. These include drilling rigs, cranes, flare booms, and cantilevered structures. In addition, debris from these structures can cause damage to process equipment, piping, risers, and the platform itself. Mitigation may be undertaken by designing such slender structures for earthquake loadings. As drilling rigs on offshore platforms are usually on skids, they should be tied down to prevent sliding. Life safety hazards to crew quarters may also be prevented by design for earthquake loads. The appropriate design loads must be determined from a deck floor spectrum. Specification of such a spectrum is difficult and additional work is needed to simplify this process.

Well or conductor failure during an earthquake is most likely when there is a massive platform failure or foundation failure. The potential of a blowout or an oil spill can be mitigated by the installation of surface controlled subsurface safety valves on all wells that are capable of natural flow.

A safety reassessment of an existing topside facility can often be accomplished by a walk-through of the facility. It is equally important to have a preventative maintenance program in place to assure that all safety devices are operative at all times. A training program is also highly desirable.

Visser concludes that the risk of damage from an earthquake to platform topsides is low provided: 1) all equipment and critical piping is properly tied down, and 2) the drilling rig, crane booms, flare booms, etc. are designed for appropriate earthquake loading.

The Working Group on Operations, co-chaired by Michael Craig of UNOCAL Corporation and David Hopper of Hopper and Associates, reaches the following conclusions and recommendations:

1. Accurate and early specification of seismic design criteria for topside equipment is critically important for non-structural design team members and for equipment vendors and manufacturers.

2. Regular topsides structural inspections of earthquake-prone platforms should be formalized through industry recommended practice documents.

3. Guidelines for the design and sizing of deck equipment should be consistent with the most recent topsides seismic hazard mitigation guidance in API recommended practice documents.

4. Existing failure data on surface controlled subsurface well safety valves should be collated and evaluated within the context of potential catastrophic structural failure of the platform.

5. The development and use of reliable earthquake response data acquisition equipment should be strongly encouraged.

6. Personnel emergency evacuation and facility shut-down procedures should incorporate earthquake preparedness and earthquake response.

7. Consideration should be given to developing integrated early warning systems for earthquakes.

8. A detailed inspection according to a well-established procedural checklist should be triggered by any "significant" earthquake.

9. Numerical models for platform seismic strength and ductility analysis should include proper detailing in conductor framing areas.

Public Policy

Thomas Tobin, Executive Director of the California State Seismic Safety Commission, points out a number of crucial issues in public policy as it relates to the seismic performance of offshore structures. Tobin emphasizes that *any* decision regarding acceptable level of risk for offshore facilities is a public policy matter shared by industry, government agencies, and the public. Public policy needs to be based on existing law, fact, valid analytical arguments and solid rationale. Furthermore, in a democratic society, public perspective and values must also be considered. It is Tobin's opinion that these perspectives have not always been effectively considered in policy making for offshore structures and that this shortcoming is a root cause of many of the industry's problems in California, and perhaps elsewhere.

It is noted that standards are certainly needed to protect against risks to human life. However, far more conservative standards of performance come from the need to protect environmental values. Environmental protection includes a broad spectrum involving a variety of living, cultural, economic, visual, and natural resources, and a plethora of agencies and organizations. The issue is complicated by distrust and differing values. An acceptable level of risk must be based on consideration of all of these factors.

Unfortunately, an acceptable level of risk is not constant, as perceived by the public. Acceptability often changes after a disaster. Furthermore, the failure of standards in one sector of society will often affect the acceptability of standards in another sector.

Tobin emphasizes that "for a policy to be effective, it must be viewed as credible by regulators, those regulated and the public." All stakeholders must believe that the policy addresses their individual and collective concerns. There must be mutual trust.

It is argued that a separation of power is beneficial to the credibility of policies as well as the reliability of their execution. Separating the responsibilities of ownership, operations, and regulation is critical to successful seismic risk management. A number of guidelines are given for effective public policy related to offshore structures including: 1) involve the public in a meaningful way, 2) involve the government agencies who share responsibility for environmental protection, 3) adopt standards and procedures in an open process, 4) build the analytical capacity within the regulatory agencies and support this capacity, and 5) formally incorporate rigorous peer review at every level.

The Working Group on Public Policy, co-chaired by Sylvia Earl of the National Oceanographic and Atmospheric Administration and Richard McCarthy of the California Seismic Safety Commission, makes the following conclusions and recommendations:

1. It should be absolutely clear to the operator, regulator, and citizens of the coastal community permitting any offshore development that specific acceptable levels of risk have been developed and will be adhered to under any circumstances in the design or reassessment of an offshore platform.

2. Action should be taken to implement a peer review process in the seismic design and reassessment of offshore platforms and to develop effective procedures for obtaining public input throughout the process.

3. The applicability to other natural hazards of the reassessment procedures developed by the API Panel on Seismic Requalification of Offshore Platforms should be investigated.

4. Active and passive triggers for reassessment need to be more precisely defined. A more precise definition of what constitutes a "catastrophic" event particularly as it relates to environmental consequences needs to be developed.

Conclusions

The state-of-the-art of seismic design of offshore structures has improved greatly over the past 30 years in the areas of seismicity determination, strong ground motion estimation, and structural response evaluation for offshore structures. However, there are still many areas that need further research and development. Of equal importance to these technical issues, there is a need to determine acceptable levels of risk for offshore structures and to agree upon strategies to achieve these levels of risk in both old and new structures. It is believed that the results of the International Workshop on Seismic Design and Reassessment of Offshore Structures will help to guide the way toward realizing these ends.

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Policy Issues in Reviewing Offshore Structures

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Introduction

If policy issues do not dominate the reassessment of offshore structures, they certainly provide the perspective to determine what should be done and how. The decisions regarding the acceptable level of risk for these facilities is a policy matter shared by many players; the industry, agencies and the public.

Policies should be based on existing law, fact, valid analytical methods and solid rational. But in our democratic society, factual, scientific and engineering methodology will not always carry the day. The public's perspective, or their values, must also be considered. However, these perspectives are not considered effectively in policy making for offshore structures, and this shortcoming is a root cause of many of your industry's problems in California. I recommend serious introspection, policy research and analysis, and above all, a willingness to do your business, and the public's business, differently.

Policy and Public Policy

Seismic risk management is a public policy issue. As William Ruckelshaus wrote in Science nearly ten years ago:

> "To effectively manage...risk, we must seek new ways to involve the public in the decision making process.... They need to become involved early and they need to be informed if their participation is to be meaningful."

A policy is a prudent course of action designed to influence and determine decisions and actions. Policy issues include the sharing of power and expense, and setting performance expectations such as income and risk. When the act of adopting a policy affects a legitimate public interest, it becomes a matter of public policy. Since the public is the ultimate owner of our offshore oil reserves, and since the public shares the benefits of exploitation and the risk of a failure, risk reduction and management is an issue of public policy.

The reassessment of offshore structures raise the following policy questions:

- Who is responsible to determine the acceptable level of risk?
- What is acceptable?

- Who is responsible to see that risk management standards and commitments are carried out?
- Who pays for the cost of setting, enforcing and complying with the standards?
- Who accepts the physical, financial and environmental risks?
- Who is liable and for what?
- How is the public represented in these activities?

Acceptable Level of Risk

What loss can be tolerated without adverse consequences?

The following simple relationships to relate the acceptable level of risk (R_a), to other risk factors. First, the Residual Risk (R_r) is related to the Estimated Risk (R_e) as

$$R_r = R_e - \text{the reduction in risk due to mitigation and risk management.}$$

The Estimated Risk is the risk of the consequences of a failure of the structure and its supporting system before any risk reduction or risk management is considered. The Residual Risk is the risk remaining after the Estimated Risk is reduced. Risk can be reduced by strengthening a structure or removing some of its vulnerable elements, and by managing the consequences of a failure by means such as establishing an emergency response capability, preparing plans to speed recovery, and monitoring.

Once these risk reduction and management measures are implemented, it must be determined whether the remaining or, Residual Risk is acceptable. Thus, the question is whether

$$R_r \leq R_a \ ?$$

Time does now allow an exhaustive discussion of risk reduction and management policy issues, so this paper concentrates on risk reduction standards for an acceptable level of structural performance. However, arriving at an acceptable level of risk also must include risk management measures.

Standards must protect against risks to life. However, a far more conservative standard of performance comes from the need to protect environmental values. Environmental protection is a broad concept involving a variety of living, cultural, economic, visual, and natural resources, a plethora of agencies, and a battle field littered with long lasting disputes, distrust and differing values. Policy making is difficult regardless of issue. However, if common sense tells us that a good policy should result in a more or less predictable course of action, the acceptable level of risk must be based on consideration of all of these factors.

Unfortunately, life is unfair. Risk acceptability changes after a disaster. The level of risk considered as acceptable before a disaster is easily tossed aside by the public in lieu of demands for less risk. Furthermore, environmental damage caused by the failure of one company, can tar other companies and the regulatory agencies with the brush of political recriminations and overcautious reaction. The stakes are high. They include the public's acceptance of continued or expanded exploitation of offshore reserves. A disaster offshore California can influence other states as well. All companies' interests, as well as the public's interests are well served by defensible standards supported by the stake holders. These standards have a better chance of surviving a post disaster mood shift and political challenge than standards lacking broad support.

How can society agree upon an acceptable level of risk standard addressing environmental protection when the players involved (government owners of the resource, lessee companies, elected officials, insurers, design engineers, and regulators) all have differing views and capacities to accept losses? Who should be involved in policy decisions? In the offshore environment there are many state and federal agencies with some form of regulatory responsibility. Environmental laws have established procedures for considering complex policy and project matters. Precedents exist.

Agencies involved in environmental protection issues should be involved in establishing the acceptable level of risk. Agencies include the following federal agencies: National Marine Fisheries Service, the Fish and Wildlife Service, the National Ocean and Atmospheric Administration, the Environmental Protection Agency, the Minerals Management Service, the Coast Guard, the National Park Service, and the Corps of Engineers. Agencies at the state level in California include: The State Lands Commission, the Coastal Commission, the Department of Fish and Game and the California Environmental Protection Agency. Their sometimes overlapping and competing concerns create a complex, lengthy and rigorous regulatory process. Unless the affected local governments, public interest groups, applicants and the public are involved in the setting of environmental risk policies, controversy will surface over and over on each project. In other words, the policy won't work.

It is worth examining the policy structure of one agency, the California Coastal Commission, as an example of how environmental protection is addressed and to underscore the need for an open policy setting process involving the affected agencies. Coastal permits are required by state law for lease offerings, exploration activities and production facilities involving state lands. On the Outer Continental Shelf, the federal Coastal Zone Management Act requires that the Coastal Commission concur in plans of exploration and development. The Commission must find a proposed plan consistent with the National Oceanic Atmospheric Administration certified coastal management program before an activity may proceed.

The statutes define the basic policies. Section 30230 of the public Resources Code provides that:

> "Marine resources shall be maintained, enhanced, and where feasible, restored. Special protection shall be given to areas and species of special biological or economic significance. Uses of the marine environment shall be carried out in a manner that will sustain the biological productivity of coastal waters and that will maintain healthy populations of all species of marine organisms adequate for long-term commercial, recreational, scientific and education purposes."

Section 30232 provides that:

> "Protection against the spillage of crude oil, gas, petroleum products, or hazardous substances shall be provided in relation to any development or transportation of such materials. Effective containment and clean-up facilities and procedures shall be provided for accidental spills that do occur."

Offshore oil operations generally do not meet these statutory environmental requirements of the Coastal Act. Effective containment and cleanup for spills is not yet a reality. However, because offshore oil development is considered as "coastal dependent," Section 30260 provides an environmental override if three tests are met. The three tests are the following:

1. Alternate locations are not feasible or more environmentally damaging.

2. To do otherwise, would adversely affect the public welfare; and

3. Adverse environmental effects are mitigated to the maximum extent feasible.

Section 30262 provides one additional policy consideration:

> "Oil and gas development shall be permitted in accordance with Section 30260, if the following conditions are met: (a) The development is performed safety and consistent with the geologic conditions of the well site...."

The Commission's practice is to evaluate each project on a case-by-case basis. Feasible alternatives and mitigation measures and the public welfare balance are thought to depend on the unique circumstances of each proposal. The public welfare tests require that a judgment be made as to whether the risk remaining after considering all feasible structural and operational mitigation measures is acceptable. Permit applications have been denied, and concurrence has been withheld when, in the Commission's judgment, a proposal failed the "mitigation to the maximum extent feasible" and "public welfare tests." The "mitigation to the maximum extent feasible" test requires that, in balance, the feasible operational and structural mitigation measures are evaluated in the context of environmental protection and health and safety, and not only in the economic sense. This means that feasibility and the price of oil does not dictate the acceptable level of risk. There will be reserves that can not be exploited.

Public Involvement

Public policy making must involve the public, but how? The easy answer is that the government represents the public and that legal notices and the Federal Register provide adequate notice. Those of us who work in government are challenged to conduct the public's business as their civil servants. We know that in matters where the public has both a legitimate legal interest and intense concern, such as with offshore oil activities, this answer is not sufficient and that concerned citizens will sooner or later demand and get direct involvement in governmental decisions through their elected representatives, governmental bodies or law suits. Crafting a process that involves the public is a must.

Some will say that requalification of offshore platforms involves complex technical considerations and uncertainty: That it is difficult to technically train people to understand. They ask how can the public's views be valid? Others have a different view. Some sociologist believe that the public perception of risk is largely socially constructed and fears do not necessarily match the consequences. Knowledge and rational analysis don't always matter. Their views are based on life style attitudes and whether one trusts institutions. Some people simply fear technology. Some don't trust either you or me. As Professor Aaron Wildavsky points out, some simply believe in the four C's: Corporate Capitalism Causes Cancer! Some will argue that the media perpetuates this view. Complexity, seemingly irrational views, bias, distrust, media coverage, and conflicting agenda aside, these public values are important in the adoption of public policies.

Providing a mechanism to hear from the public is done by government every day. The California Environmental Quality Act process is one example. Listening to the public and responding to their views does not mean you must accept baseless and overly conservative criteria. We all recognize much of the risk in our society if false. For example, the risk from asbestos doesn't exist. Too much caution will sap the economic blood of society. Professor Paul Slovic advises that there is both wisdom and folly in public attitudes and perceptions. You must consider both. Both the experts and the public have a stake in decisions regarding reassessment, as well a something to offer. Industry, government and the public must respect each other's contributions. Providing for public participation in risk management policy decisions is a start.

Credibility of the Process

For a policy to be effective, it must be viewed as credible by the regulators, those regulated and the public. All stake holders must believe your policies address their concerns. There must be trust. However, does your present arrangement breed trust?

- There is a mismatch in power and wealth: Industry is seen as having the money, experts, data, and political clout.

- There are inherent conflict of interest. Industry wants and needs safe operations, but also will gain greater profits by extracting greater amounts of oil at lower costs. Our regulatory agencies have been told by the Congress and the state legislatures to both feed the government's coffers and protect the environment.

- Your standards are written by industry organizations without effective involvement of state and local agencies.

The mismatch in power, the dominance of the industry in writing technological standards, the dichotomous governmental mandates, and lack of public involvement, add up to a policy setting process that, in effect, excludes the public and other agencies, is hard to understand, and doesn't engender trust. Why should the public believe you? If you want policies that are both credible and appear credible, you need a policy making process that is credible.

These comments are not meant to criticize any standard, or the motives, character or competence of either the industry or government employees. They are meant to point out why the process lacks public credibility. Adopting public policies regarding an acceptable level of risk may not work unless the public's views are respected and an open and credible process to arrive at your policies is followed.

Separation of Powers

A separation of power effects the credibility of policies as well as the reliability of execution. Separating the responsibility for ownership and operations from safety regulation is a critical principle demonstrated by successful seismic risk management efforts. Even though many individuals having responsibility for potentially conflicting interests can and do make their decisions in the public interest, in the long term, given the changing pressures of the economy and politics a separation of these powers is healthy.

Earthquake risk management for buildings is somewhat independent of the regulated industry. The building codes for California are drafted with the full involvement of privately practicing engineers and architects, materials suppliers, development interest and government building officials, but the adoption is reserved to building officials who are charged by local governments to protect public health and safety. In California, these codes are reviewed and additional revisions are made by a state agency before they become "the law." The standards are enforced by local government. This system has its weaknesses, but is works fairly well and it enjoys a high degree of credibility.

Nuclear energy was once promoted and regulated by the Atomic Energy Commission. After a government reorganization, the Department of Energy assumed responsibility for promoting nuclear energy while responsibility for the public health and safety was assigned to an independent Nuclear Regulatory Commission. This arrangement has its warts, but it generally works and enjoys public credibility.

A separation of responsibilities assures that regulators not only have the expertise and adequate resources, but that they enjoy strong institutional support and are exposed to public accountability. None of us can change the existing division of powers regulating offshore oil at either the state or federal levels of government, but there are matters you can control.

First, involve the public, professional and environmental organizations, and all regulatory and environmental agencies in the development of your risk management policy.

Second, make it clear that the burden to prove that activities meet the risk management policy rests with the company proposing the project. Regulators should be in a position of reviewing materials explaining how the standard was met, not trying to prove inadequacy of a structure or its design.

Third, require expert independent peer review of site studies, analysis, and design materials, and that the reviews are conducted in open deliberations. Independent review of projects is not enough. You should provide for independent reviews of your standards and enforcement procedures as well. It's far better to suffer the sting of criticism from peers, than suffer a lack of credibility, or worse fail the ultimate test of a major earthquake.

Conclusion

We do not make public policy regarding the seismic safety of offshore facilities effectively, and this shortcoming is a cause of many of your industry's problems in California. Public attitudes and perspectives must be a consideration. You have an opportunity to improve your process as it applies to reassessment of existing facilities. Fair process will be improved if you:

- Seriously consider how you approach these issues and be willing to change your ways.
- Involve the public in a meaningful way regardless of the basis for their perception of risk or their hidden agenda.
- Involve the federal and state agencies who share responsibility for environmental protection in your seismic risk policy process.
- Draft and adopt your standards and procedures in an open process, explain what you have done clearly, and be prepared to live by the consequences.
- Build the analytical capacity within your regulatory agencies and support it.
- Formally incorporate rigorous review by independent experts.

Estimation of Ground Motion for Design Or Reassessment of Offshore Platforms

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Introduction

This paper provides an overview of the determination of seismic design parameters for offshore platforms and some of the important engineering and non-engineering issues that often need to be addressed during the development of these parameters. These issues will be discussed by participants attending this workshop. Recommendations for the resolution of some of the issues will be published in the workshop proceedings.

Overview

The objective of the seismic design process for offshore platforms is to produce a safe yet economical design. This philosophy has resulted in the development of a dual approach to seismic design that when properly implemented, achieves the following levels of performance: (1) the structure continues to operate during and after a probable earthquake or Strength Level Earthquake — SLE, and (2) the structure remains standing under the rare intense earthquake or Ductility Level Earthquake — DLE. These performance standards can be translated into design language as follows. For the SLE, in which the average return period of the design ground motion generally ranges anywhere from 50 to 500 years (the lower end of this range might apply to the requalification of a platform with a remaining lifetime of 5 to 10 years), the stresses in the tension members should be less than the yield stress and the stresses in the compression members should be less than the critical buckling stress. For the DLE, in which the average return period of the design ground motion is on the order of 1,000 to 10,000 years, the platform is designed with sufficient ductility so that the ductility demand is less than the platform's ultimate capacity.

The SLE and DLE ground-motion parameters used in design can vary depending on the structure or component. For offshore platforms the parameters are generally design spectra and/or accelerograms. However, for a pipeline running from the platform to a coastal terminal, the parameters are usually peak ground acceleration and peak ground velocity. In determining the design parameters, it is important to estimate the probability associated with the ground motion parameter and also the uncertainty in that probability.

A number of factors influence the development of seismic design parameters that are not all seismological or geotechnical related. For example, a variety of structural factors should be considered in the development of the design parameters: the type of structure, its importance, the cost, construction materials, dynamic properties (which the engineering seismologist should know before determining the design parameters), past earthquake performance, and consequences of failure. The consequences of failure not only involve repair costs, but also involve indirect costs resulting from (1) the shutdown of operations to make the repairs, and (2) potential indefinite closure due to adverse public reaction that also might prevent future development. There are also some non-engineering or political considerations, such as the local, state, and federal regulatory requirements, which are still evolving.

Generally, a site-specific analysis is performed to develop seismic design parameters for a particular site. The first step is to define the seismic sources in the site region, and develop (or select) a model of the attenuation of ground motion that is appropriate. Next, probabilistic and/or deterministic analyses are performed to estimate the ground-motion parameters. One of the questions to be addressed is which approach is appropriate. For new design, both approaches are usually used. For requalification, a probabilistic approach is generally used.

There are three basic inputs to a probabilistic approach. One is the identification of the earthquake sources in the site region. Some seismic zones are defined as areal sources while others are represented as line sources, such as discrete faults. However, in some cases it is not clear whether to define discrete faults within a region or whether to define the region as an areal source. Once the earthquake sources have been selected, the second input to the probabilistic model is a definition of the earthquake recurrence for each source. The usual relationship is a Guttenberg-Richter recurrence equation which expresses the logarithm of the expected annual number of earthquakes as a linear function of earthquake magnitude. Other types of earthquake recurrence models have been used, such as the characteristic model in which the seismic source (usually a fault) generates similar size earthquakes. One of the issues that the engineering seismologist often faces is which model is appropriate for a particular source. The third input is the attenuation of earthquake ground motion, which is usually expressed as a function of distance from the source to the site and earthquake magnitude. Other parameters can be included into the equation such as component type (horizontal or vertical), fault type (strike slip, reverse, and normal) and local geology.

The above three inputs are input into a standard probabilistic seismic hazard model. These models have been used for many years to produce a hazard curve which is a cumulative probabilistic distribution function of ground motion.

The deterministic approach has many elements in common with the probabilistic approach but the element of time is removed. In this approach, a probable or maximum earthquake is postulated for each seismic source, and attenuation equations or accelerograms representative of the postulated events are used to determine the ground motion at the site.

Example

To illustrate the approaches discussed above, a typical example for an offshore platform located in a subduction zone environment (Figure 1) is presented. Many seismic regions around the world are located in this type of tectonic environment, which comprises most of the Pacific rim. Figure 1 shows a subduction zone that is capable of generating earthquakes up to moment magnitude 9. The platform is located in the back arc region in the middle of a deep basin. Local fault sources capable of generating much smaller earthquakes are situated in the basin. In California the situation is not much different than that depicted in Figure 1 if the subduction zone is replaced with the San Andreas fault, also capable of generating great earthquakes. At many locations, the great subduction zone earthquake will have a recurrence rate that may be much higher than that of the local faults.

If a probabilistic seismic hazard analysis is performed for pseudovelocity (PSV) at each oscillator period over a wide period range from 0.04 to 4 sec, the result is typically that at very short periods, the PSV is governed by the local source (Figure 2), even though it has a much smaller rate of earthquake recurrence than the subduction zone or San Andreas fault. The primary reason for this result is that because the local fault is much closer to the platform, high frequency (short period) motions attenuate much less than those motions from a distant earthquake, regardless of the magnitude of the distant event. However, for longer period motions, the opposite may be the case. A large magnitude earthquake will tend to have much more energy concentrated in the longer periods, and thus the probabilistic hazard from the subduction zone, or San Andreas fault, may in fact be the dominant factor in determining long period ground motion as illustrated in Figure 3.

Uniform probability spectra are generated from figures similar to Figures 2 and 3. A spectrum with a 200-year average return period is shown in Figure 4; the local fault and subduction zone spectra for the same return period are also shown. Note the subduction zone spectrum is deficient at the short periods and enhanced at the longer periods. The local fault spectrum is just the opposite.

Some engineering seismologists have recommended the uniform hazard spectrum for the SLE or DLE, and in certain applications, this spectrum may be acceptable. However, in many situations, this spectrum represents a starting point in the development of the design spectra. The aforementioned geotechnical, structural and regulatory factors also enter into the developmental process. For example, with respect to geotechnical factors, many platform sites are located on local geologies which consist of an upper layer of soft soils that could range from several tens of feet to more than 100 feet thick. Also, the regional geology may consist of a large basin or a high Q (low anelastic attenuation) crust several kilometers thick. Furthermore, as indicated in the example problem, there may be an earthquake source capable of generating earthquakes much greater than magnitude 8, or the site may be in an intraplate environment, such as the central or eastern U.S. where the earthquake source characteristics may be significantly different from those of interplate earthquakes. Little ground motion data have been recorded under these situations. Consequently, existing empirical attenuation equations generally do not adequately represent some of these special circumstances that are very common in areas where offshore platforms are located.

With respect to local geologic soil conditions (Figure 5), a fairly large ground-motion data base exists for shallow and deep stiff soil deposits, but relatively few accelerograms have been recorded on soft soil deposits, such as soft clay.

Many offshore platforms are located in large basins where long period ground motions can be amplified significantly. For example, during the 1971 San Fernando, California earthquake, the long period motions in the middle of the San Fernando and Los Angeles basins, were much larger and had longer durations than those at sites closer to the edges of these basins as shown in Figure 6. Empirical attenuation equations have been developed from ground motions recorded within basins (e.g., Crouse, 1987) or from ground motions where the depth to basement rock is incorporated as a parameter within the equations (e.g., Campbell, 1990). However, the database used to develop the equations may not adequately represent the basin in which the platform is located.

Accelerograms have not been recorded during a great earthquake, either a magnitude 8 on the San Andreas fault or a magnitude 8.5–9+ in a subduction zone. Therefore, methods to estimate ground motions from these events invariably require some sort of an extrapolation. If the structural engineer wants design time histories corresponding to the great earthquake in the subduction zone or on the San Andreas fault, then the engineering seismologist can simulate them by superimposing the motions from smaller magnitude earthquakes. These smaller earthquake motions are called Greens functions. For example, using this technique, Kanamori (1979) estimated the ground motion in downtown Los Angeles for a magnitude 8+ earthquake on the San Andreas fault similar to the 1857 earthquake. One of his simulations is shown in Figure 7. Note that the duration is on the order of several minutes and that the oscillations have periods on the order of 5–10 sec, which could impact deep-water platforms with fundamental periods in that range.

Before determining the final design spectra, the engineering seismologist should be in communication with the structural engineer and the owner to obtain input regarding the potential influence of the aforementioned structural and regulatory factors. Ideally, there should be a three-way communication link between them to reduce the uncertainty in the seismic design parameters and to eliminate under- and over-design. For the example problem presented above, the final design spectrum for a long-period platform might look like the one shown in Figure 8. Because of basin effects and the potential for a great earthquake, both of which can generate significant long-period motion, the design spectrum was constructed higher than the uniform hazard spectrum at long periods. Also because the greater earthquake dominates the long-period motions, the short-period portion of the spectrum was de-emphasized as shown in the figure because this portion of the spectrum is influenced by smaller local earthquakes.

Vertical Motions

The methodology presented above applies to horizontal and vertical motions. However, there are additional studies that should be considered before constructing the design parameters for the vertical component. This component of motion can affect the design of cantilevered structures on the top sides of platforms. These structures can be sensitive to vertical motions. The API–RP2A publication (API, 1991) recommends a vertical design spectrum that is simply one-half (1/2) the horizontal design spectrum. However, many situations are encountered in which this rule of thumb is not appropriate. If attenuation equations are available for the vertical component of motion, then the hazard analysis can be repeated using these equations. An example of uniform hazard spectra for horizontal and vertical components is shown in Figure 9 (Crouse and Quilter, 1991). These spectra were developed using horizontal and vertical component pseudovelocity attenuation equations that were derived from the same accelerogram database (Crouse, 1987; Crouse et al, 1985). As shown in Figure 9, the ratio between the vertical and horizontal components is less than 1/2, and the shapes of the two spectra are quite different.

The effect of the water column also may be important for offshore platforms. Nearly all accelerograms have been recorded at terrestrial sites. The strong shaking portions of horizontal ground motions are generally composed of shear waves. Water cannot transmit shear waves, but can transmit compressional waves. Observations of many strong-motion accelerograms indicate that the vertical component is comprised mostly of vertically propagating P waves. If that condition is expected to hold for a given platform, then the terrestrial and the seafloor motions can be quite different, because for the offshore site, the P wave will be transmitted through the water column and be reflected back toward the seafloor. Wave transmission and reflection results in total destructive interference of waves at the seafloor at certain frequencies (i.e., the natural frequencies of the water column). With a knowledge of the P-wave velocities and densities of the water and underlying half space, transfer functions can be derived and used to convert a terrestrial vertical ground motion to a corresponding seafloor motion (Crouse and Quilter, 1991). In one particular application for a platform in about 400 feet of water, the 1971 Holiday Inn vertical component was modified by the appropriate transfer function. This result is shown in Figure 10 where the reduction in the higher frequencies is observed. However, for a deep water platform site (e.g., one in 1,000+ feet of water), a reduction in lower frequencies is observed because the fundamental frequency of the water column is around 1 Hz. Because the reduction in the corresponding response spectra can be on the order of a factor of two at these natural frequencies, the validity of the assumption that the vertical motions are composed of vertically propagating P waves must be critically examined before conducting a water column analysis.

Issues

The last part of this paper discusses several ground-motion issues that will be addressed during this workshop. The first set of issues relates to the definition of the inputs for seismic hazard analysis. One issue is the characterization of earthquake sources. For many hazard analyses, questions, such as when is it appropriate to use an areal source and when is it appropriate to use a line source, must be addressed. Other questions are what is the appropriate earthquake recurrence rate to use for each source and how should this rate be determined? With respect to attenuation equations, many of them have been published. For a particular site, is it appropriate to use an existing equation or should new ones be derived from ground-motion data representative of the tectonic and geologic environment? Although some of the questions do not seem particularly difficult, the answers vary depending on which engineering seismologist is performing the hazard analysis. Not surprisingly, the results of the hazard analysis at the same or nearby sites can vary significantly among these seismologists.

Regarding the characterization of seismic sources, the recent trend has been (especially in southern California) to define discrete faults and estimate a recurrence rate for each fault based on offsets of geologic strata that have been observed by geologists. The slip rate for the fault is defined as the amount of offset divided by the time over which it occurred. This slip rate is then used to estimate the earthquake recurrence rate (e.g., Anderson, 1979). However, there are many uncertainties associated with this procedure. There is an uncertainty associated with the offset itself, and in the age dating. The percentage of slip that is seismic (seismic efficiency) and the percentage that is creep are also uncertain. There are many situations in which the seismic efficiency of a fault is not 100%, and in fact that is probably the rule rather than the exception.

During the 1970's when many seismic studies were being funded by the nuclear power industry, many attenuation equations were available to estimate ground motion. These equations predicted peak ground accelerations for a magnitude 6.5 earthquake that varied by an order of magnitude for a given distance from the earthquake to the site. There were some reasons for this large discrepancy. For example, the distance definitions or the magnitude-scale definitions were not necessarily the same for each equation, and the local geology (rock or soil) to which the equations applied was different. Perhaps the biggest reason for the differences was the lack of strong-motion data or the variable amounts of strong-motion data available to researchers who derived the equations. However, as more data became more widely available, the range in the ground-motion estimates diminished. Attenuation equations published during the last five years predict peak ground accelerations that now vary by about a factor of two. Thus, the situation is improving, but a factor of two is still quite a big difference. Thus, the derivation or selection of the proper attenuation equation to use in hazard analysis is one of the more significant issues.

When conducting probabilistic hazard analysis, it is appropriate to incorporate the standard error (σ) of the attenuation equation into the probability calculation. This σ has generally been expressed as a constant or as some function of magnitude. It is important to realize that regardless of how σ is expressed, it is composed of errors related to earthquake-to-earthquake variability and site-to-site variability in ground motion. When a regression analysis of ground motions is conducted to develop an attenuation equation, many accelerograms recorded at many different sites from many different earthquakes are included in the regression. However, the resulting attenuation equation is applied to only one site. Consequently, there has been some suggestion that it may be appropriate to remove the site-to-site component of variability from σ. Although the resulting σ may be only reduced by 10–20%, the ground motions at long average return periods can be significantly reduced. Therefore, the value of σ to use in hazard analysis becomes an important issue.

For seismic sources capable of generating great earthquakes there is always a question of extrapolation. If empirical approaches are used to estimate ground motions from these events, then some sort of extrapolation is involved even when simulation procedures using Green's functions are employed. One question that must be addressed is which extrapolation method is most appropriate.

The above discussion deals only with some of the technical issues. Regulatory issues will also be addressed in this workshop. One issue is the role of local, state and federal government, and another is peer review. Some type of regulatory process is considered necessary to ensure that certain standards are met for the safety of the public and environment. However, an overabundance of regulations does not serve any useful purpose. The issue is the level of regulation acceptable to government and industry. Similar comments apply to peer review.

Conclusion

The estimation of ground motions for offshore platform design or requalification is a challenging process that involves several technical disciplines, including seismology, geology, geotechnical and structural engineering, as well as political considerations. Many uncertainties and issues still exist in the development of seismic design parameters. Workshops such as this one and additional research are needed to resolve them.

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EXAMPLE PROBLEM

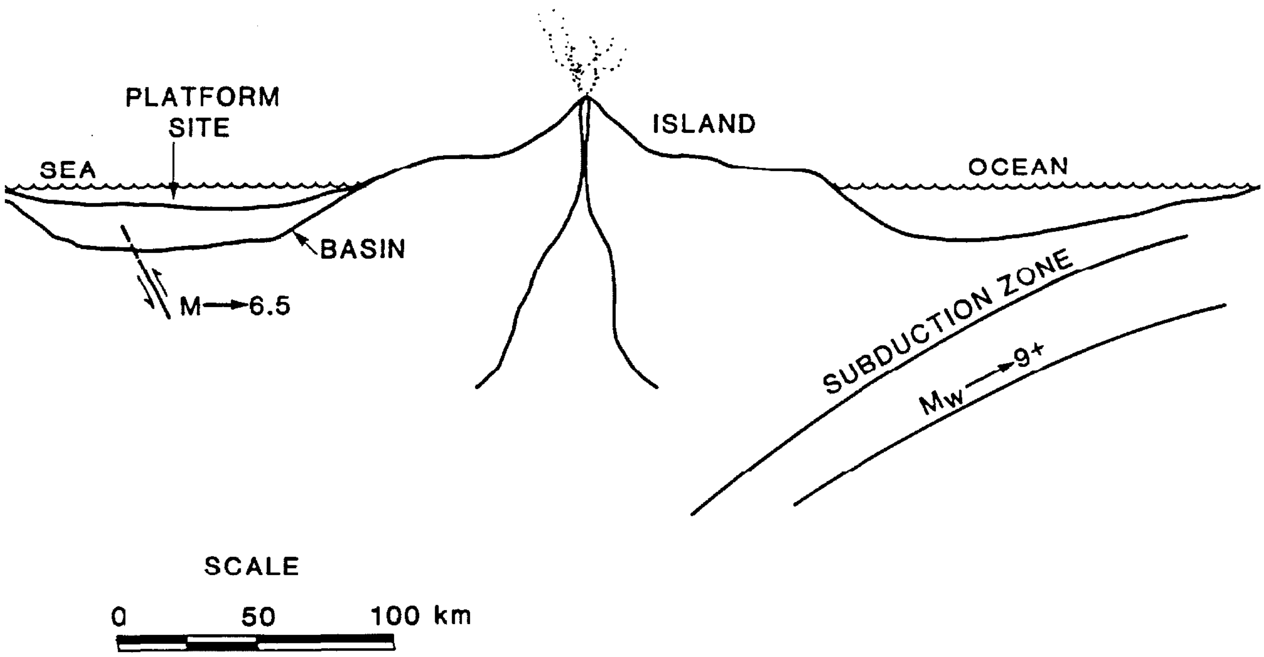


Figure 1

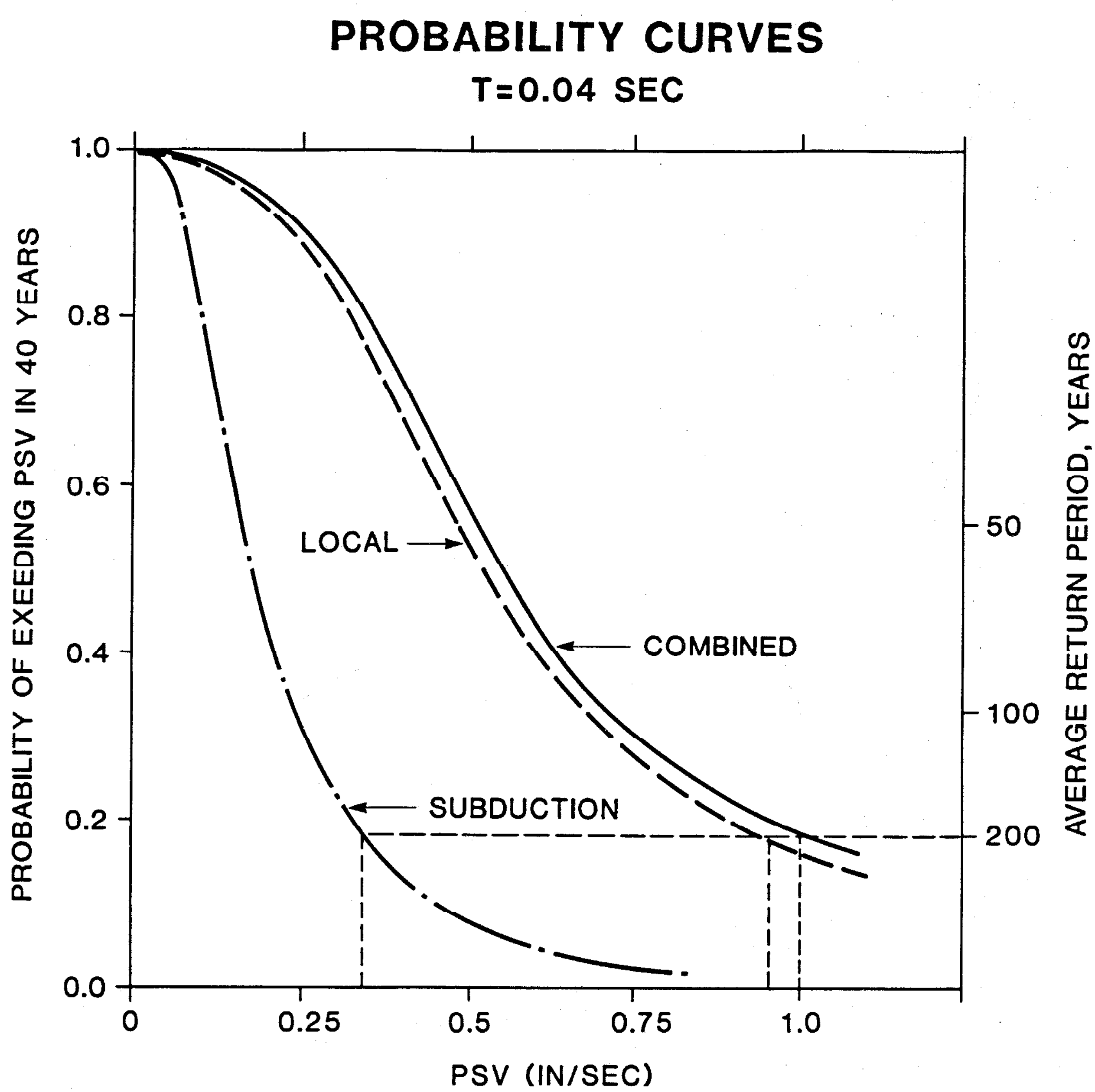

PROBABILITY CURVES
T=0.04 SEC
PROBABILITY OF EXEEDING PSV IN 40 YEARS
1.0
0.8
0.6
0.4
0.2
0.0
LOCAL
COMBINED
SUBDUCTION
AVERAGE RETURN PERIOD, YEARS
50
100
200
0
0.25
0.5
0.75
1.0
PSV (IN/SEC)

Figure 2

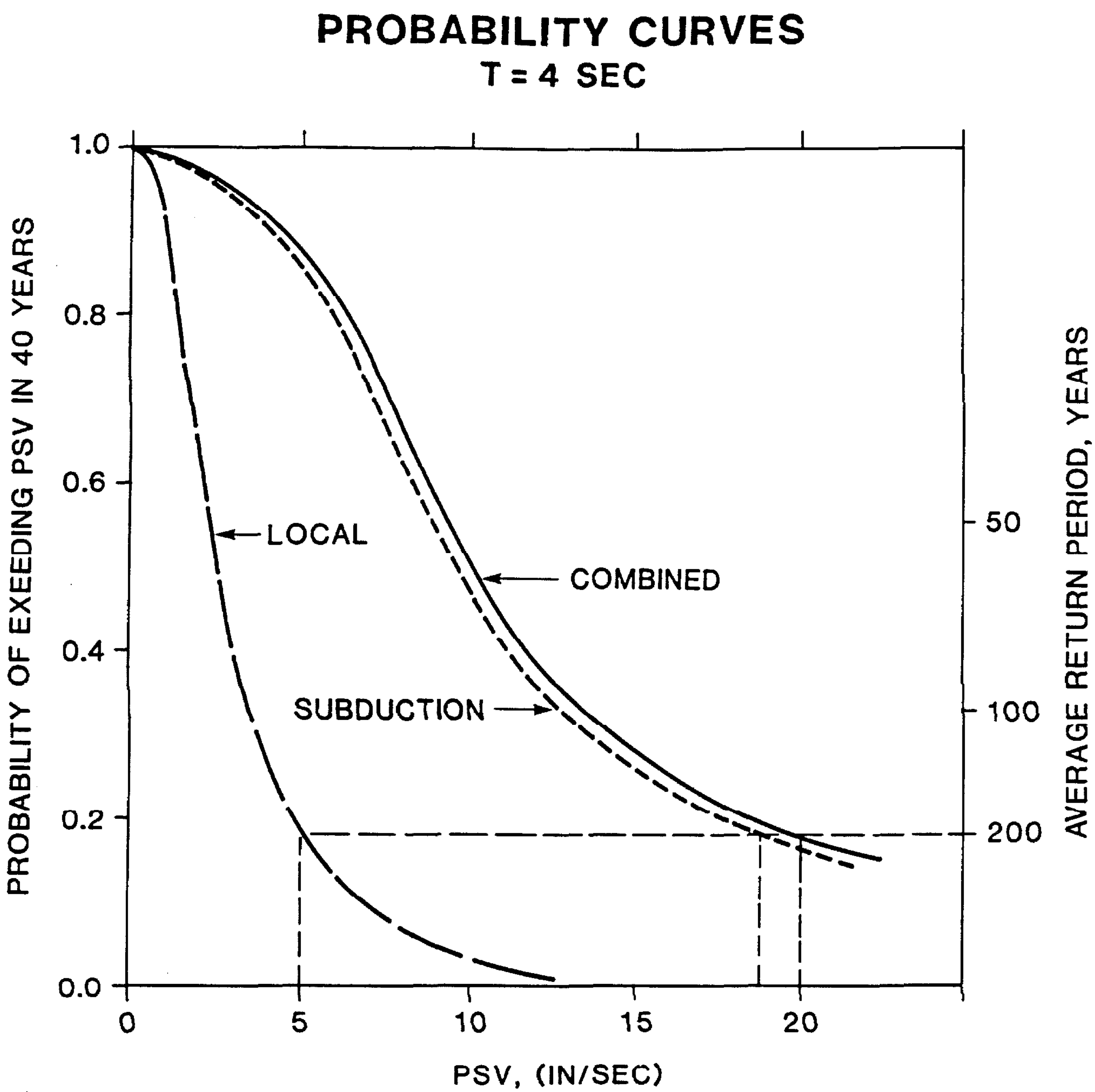

PROBABILITY CURVES
T = 4 SEC
PROBABILITY OF EXEEDING PSV IN 40 YEARS
1.0
0.8
0.6
0.4
0.2
0.0
LOCAL
COMBINED
SUBDUCTION
AVERAGE RETURN PERIOD, YEARS
50
100
200
0
5
10
15
20
PSV, (IN/SEC)

Figure 3

RESULTS OF PROBABILISTIC SEISMIC HAZARD ANALYSIS

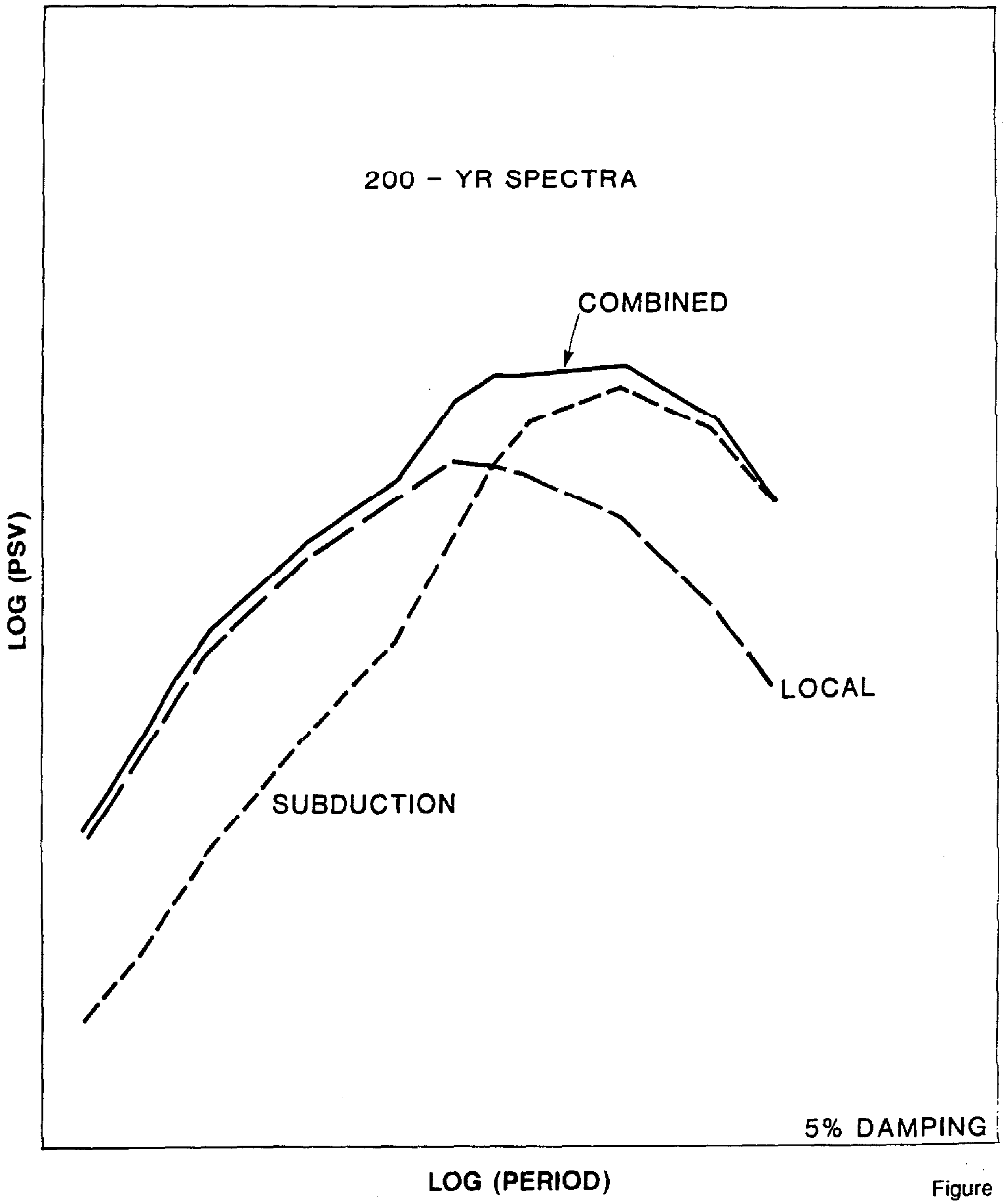


Figure 4

1991 UBC SITE CATEGORIES

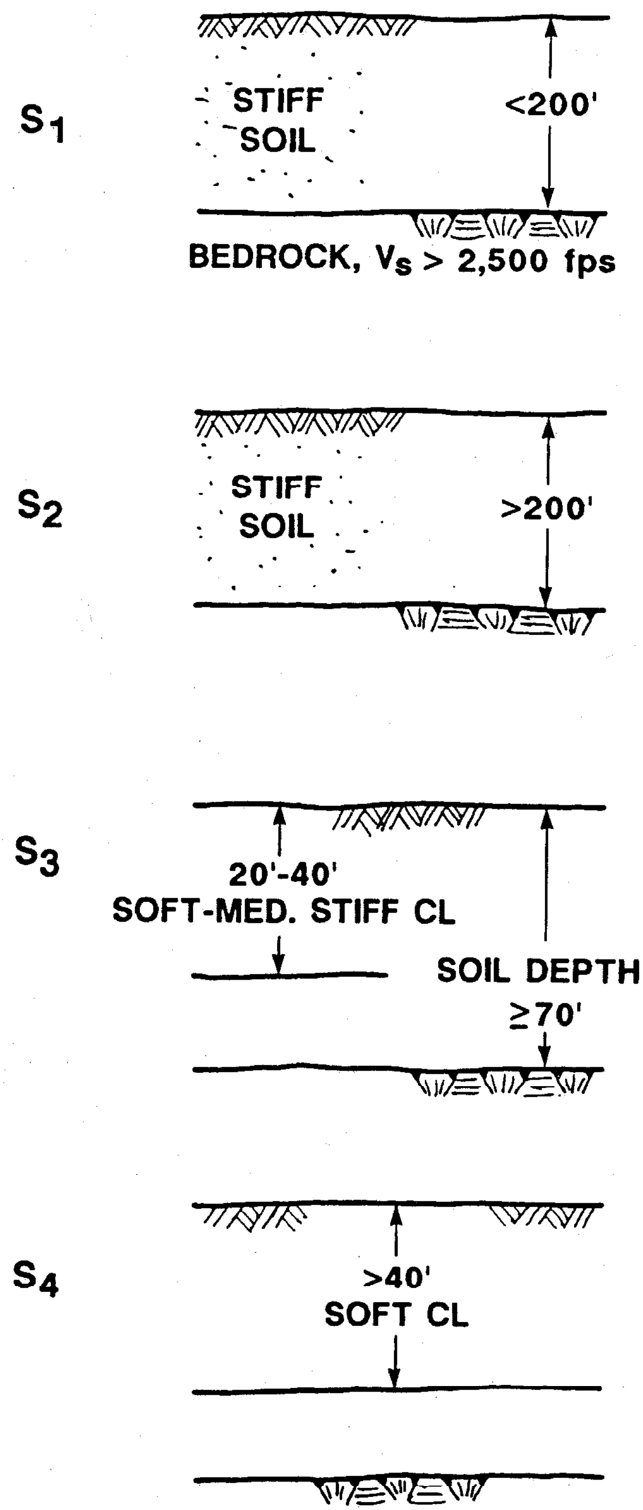
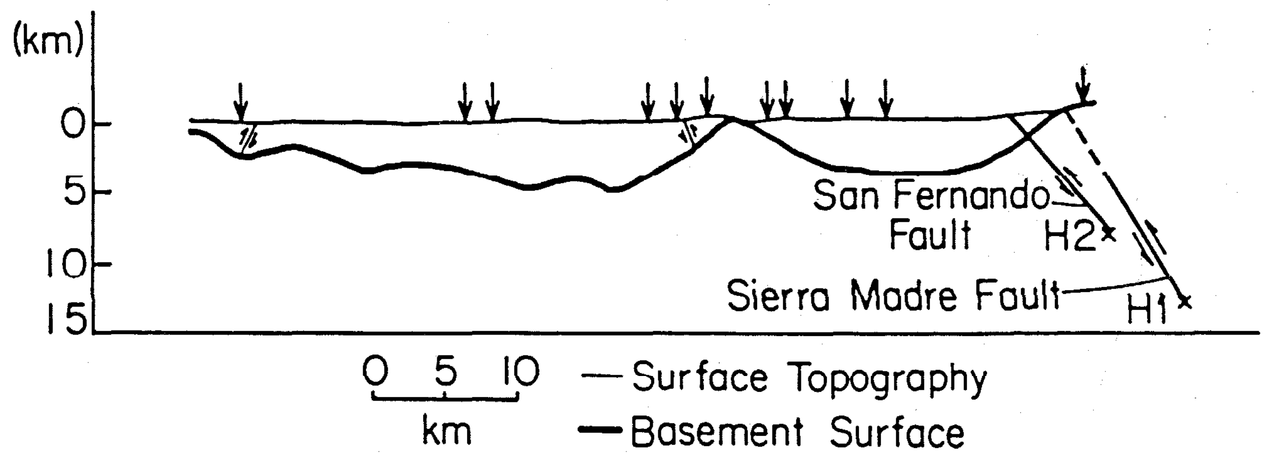
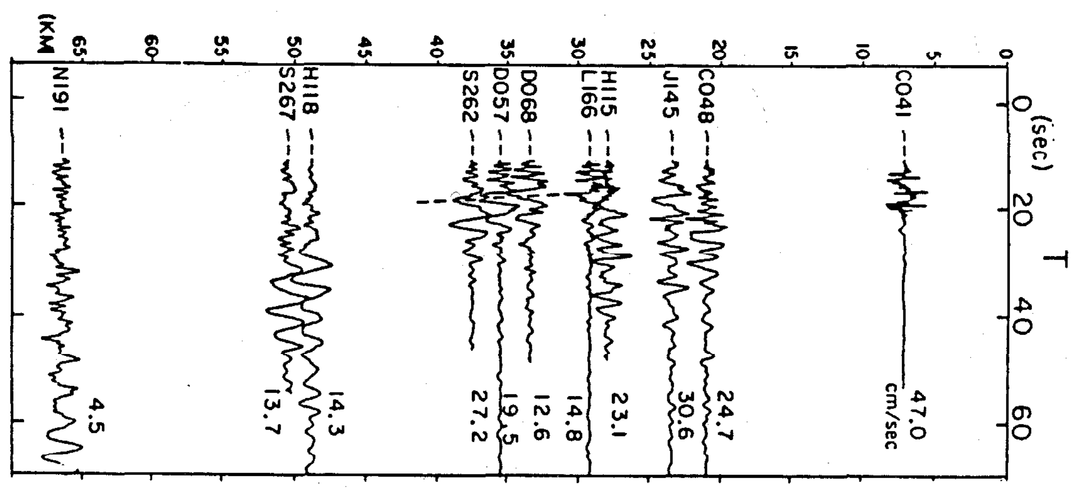


Figure 5

BASIN EFFECTS

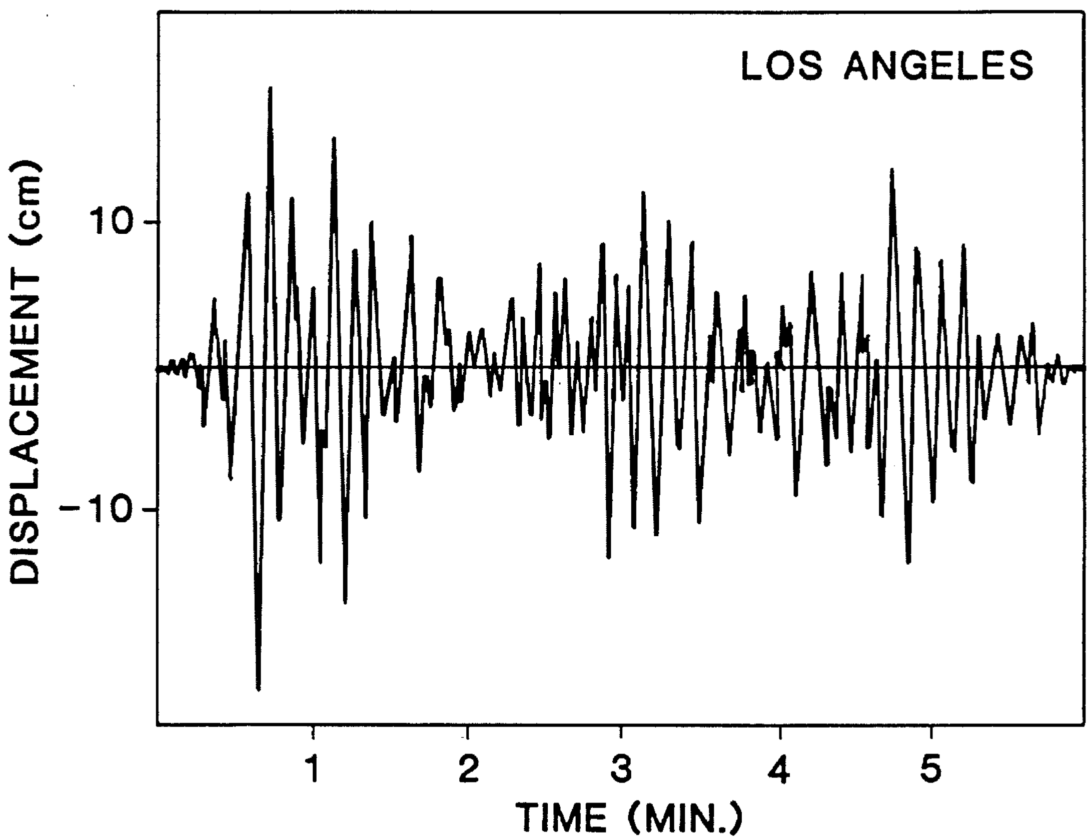




(LIU & HEATON, 1984)

Figure 6

1857 SAN ANDREAS EARTHQUAKE SIMULATED GROUND MOTION



(Kanamori, 1979)

Figure 7

RESULTS OF PROBALISTIC SEISMIC HAZARD ANALYSIS

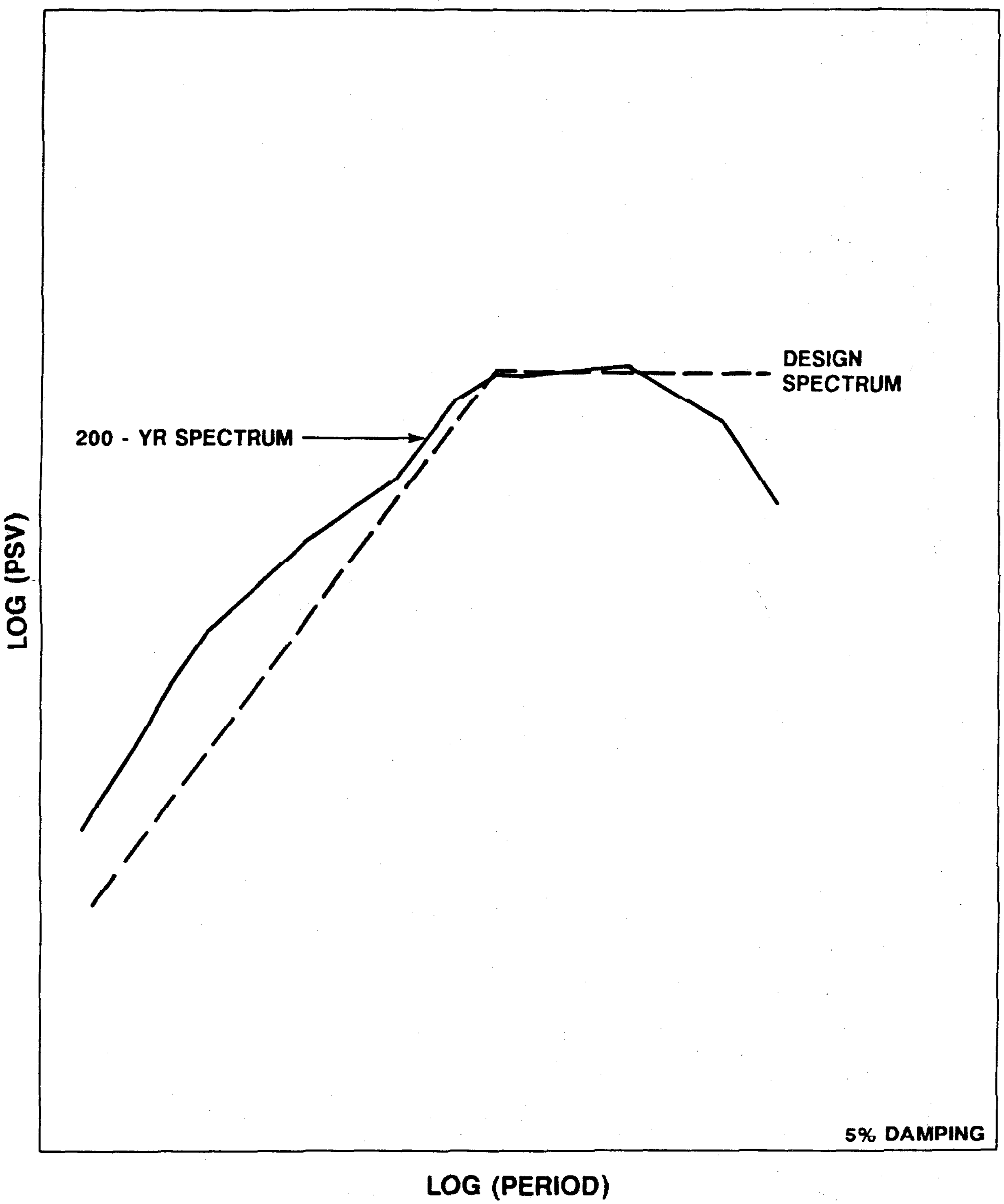
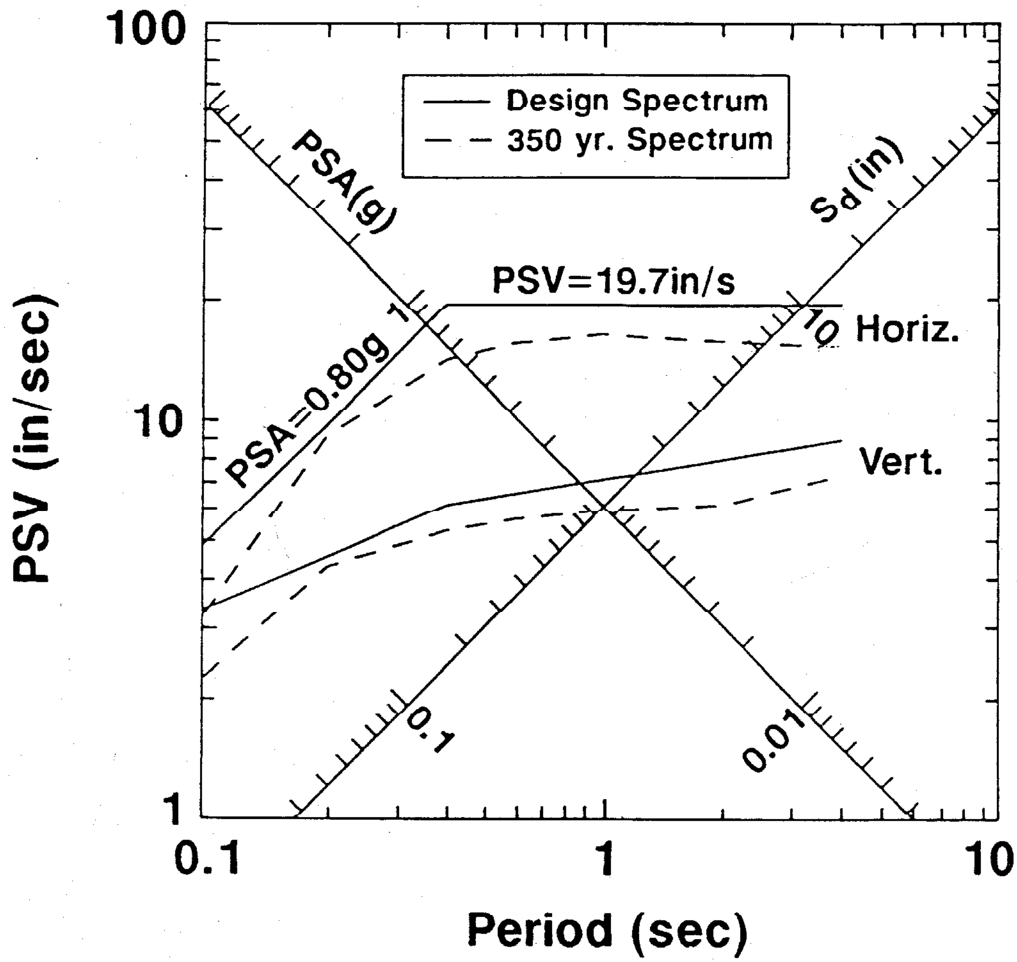


Figure 8

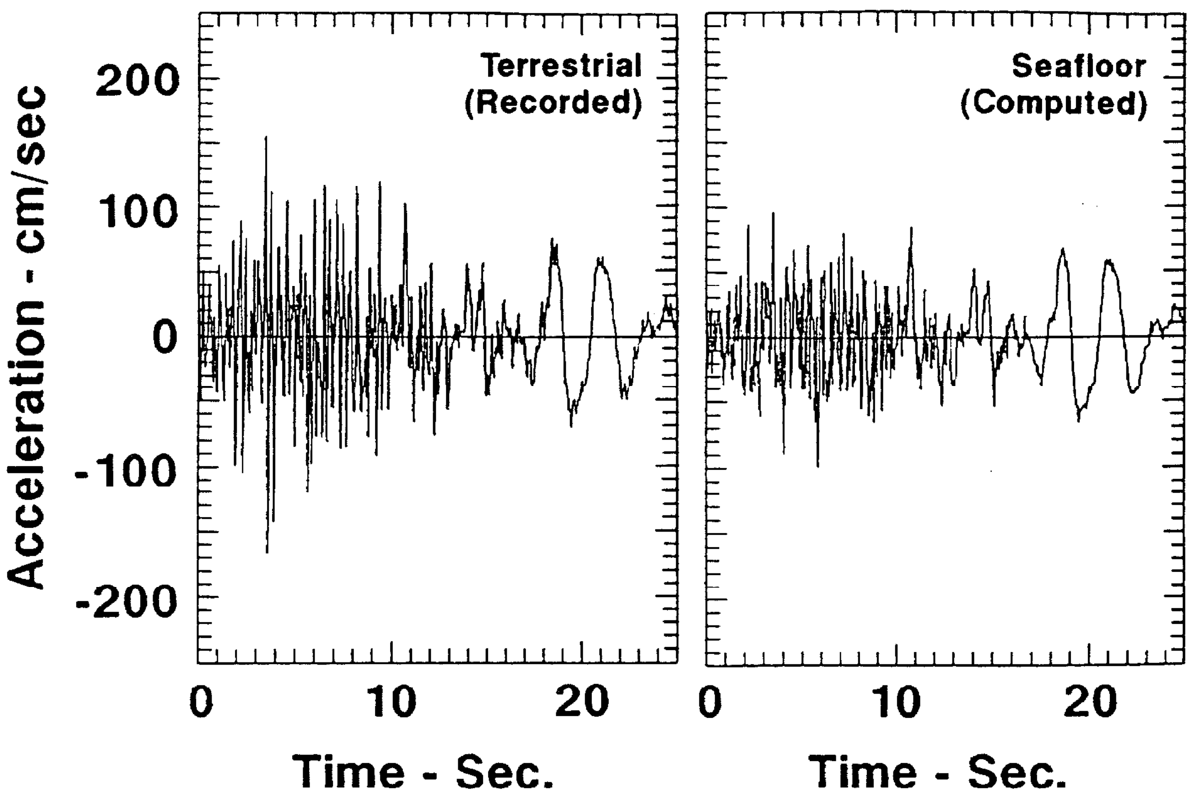
HORIZONTAL AND VERTICAL SPECTRA
MAUI A PLATFORM



(Crouse and Quilter, 1991)

Figure 9

1971 HOLIDAY INN VERTICAL COMPONENT



(Crouse and Quilter, 1991)

Figure 10

Seismic Design and Requalification Methodologies for Offshore Platforms

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Abstract: This paper summarizes current methodologies that are being used to design and requalify offshore platforms that are located in active seismic regions. While there are many common aspects between design and requalification methodologies, there are some significant differences that are discussed in this paper. Reliability, platform capacity characterizations, and fitness for purpose considerations are discussed as they influence the definition of some of the important aspects of seismic design and requalification methodologies. It is concluded that for the class of offshore platforms considered in this paper, seismic design methodologies are highly developed and there are definitive engineering guidelines. However, requalification methodologies need additional development. There is a need for comprehensive and definitive engineering guidelines for the performance of requalification analyses.

INTRODUCTION

Design of offshore structures to resist earthquakes involves many of the same problems and issues as their onshore counterparts. However, there are some very important differences; the most important of which is the presence of a water column over the earth's surface. This water column has important effects on the location and characterization of earthquake sources, identification of important travel path geology characteristics, and determination of site soil characteristics. The water column also influences the response of the sea floor soils, the types of foundations and structures that comprise offshore platforms, and the mass, stiffness, damping and strength characteristics of the platforms.

One of the most important effects of the water column is the storm waves and currents, and in the case of platforms located in the Arctic, the winter ice that can frequent its surface. These storm waves and currents and winter ice generally pose a very important source of loading for these structures. Because of these conditions, these structures are inherently designed for very large lateral loading in addition to very large vertical operating loading. In many seismic offshore areas, the loading developed by storms and intense ice conditions exceed those associated with very intense earthquakes. These loadings have important influences on the strength, stiffness, and other characteristics of the platforms. Storms and ice can also have some very important effects by acting directly on the foundation soils (e.g., cyclic shear straining, scour).

Steel, pile supported, tubular member spaced framed drilling and production platforms (Fig. 1) are discussed in this paper. Such structures are frequently referred to as "template-type" platforms. Approximately 100 such major platforms have been installed in potentially intense earthquake regions such as offshore California, Alaska, New Zealand, Japan, China, and Indonesia.

These platforms are composed of three primary components: a deck (superstructure) which supports the hydrocarbon drilling and producing facilities; a jacket (substructure) which spans the distance between the deck and the sea floor; and foundation piles which provide lateral and vertical support for the other components.

Fixed, bottom-supported offshore platforms tend to be long period structures that have natural periods (lateral, flexure, torsion) in the range of 1 to 5 seconds. The first vertical mode periods frequently are in the range of 0.3 to 0.5 seconds. Vertical motions can be important to these structures because of long span and cantilevered heavily loaded decks and the flexible facilities (e.g., piping) mounted on these decks.

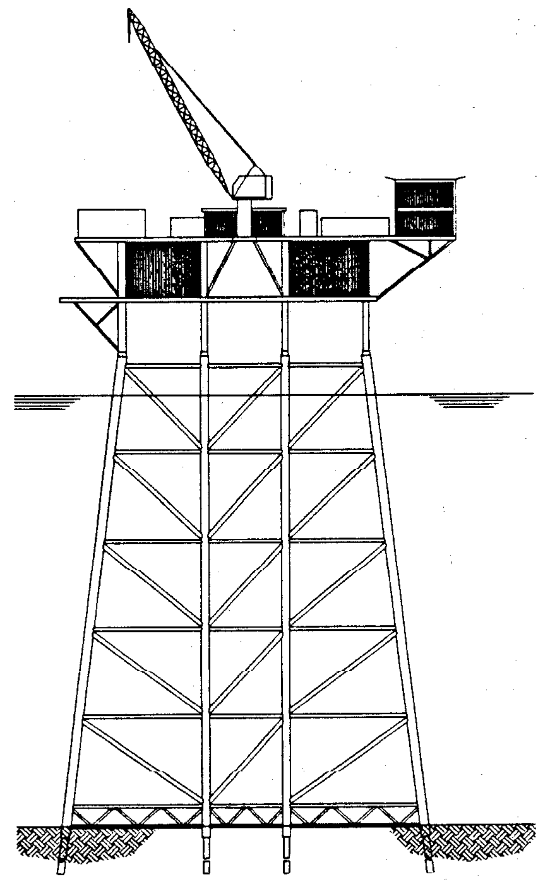

FIG. 1. Template-Type Offshore Platform

The foundation piles and soils, and in some cases the well conductors, can have very important effects on the stiffness and damping characteristics of the platforms (Bea 1991a). Similarly, the horizontal braces at the sea floor and the mud mats which are installation aids can have important effects on the structure response characteristics. The entrained water inside the platform elements and the water that is accelerated by the motions of the structure, frequently referred to as added mass, have very important effects on the mass and damping characteristics of the platform. One-third to three-quarters of the total mass of the platform can be attributed to hydrodynamic mass.

For low to moderate levels of excitations, these structures tend to be moderately damped, exhibiting system viscous damping ratios in the range of 3 % to 5 % (Mason et al. 1989). This energy dissipation is derived from energy losses in the steel elements, and in their interactions with the water and foundation soils. For high levels of excitations, damping can be significantly greater depending primarily on the hysteretic energy dissipation characteristics of the structure and foundation elements (Bea 1991a).

SEISMIC DESIGN

A Seismic Design Methodology (SDM) should represent a readily applied process and set of parameters that will guide the engineering of a platform system to have acceptable performance characteristics during its intended lifetime. The primary objectives of a SDM are to assure that the platform system will have sufficient strength and ductility to satisfy its intended purposes without undue expense or risk.

The primary concern of the platform designer is with the response that develops after first significant yielding occurs in the structural elements and components that comprise the platform structural system, and the damage states that can lead to collapse of the platform. The engineer's objective is to provide an acceptable degree of safety against undesirable performance during intense seismic events.

During the past 20 years, advanced earthquake design guidelines have been developed for steel, template-type offshore platforms (API 1991). This development has had a substantial element of its background founded in the framework of probabilistic seismic exposure and reliability based design and decision analysis methods (Bea 1977; Bea 1979; Bea et al. 1987; Moses 1990).

The American Petroleum Institute (API) guidelines (Fig. 2) are organized into four major parts: 1) characterization of the seismic environment, 2) design for strength, 3) design for ductility, and 4) other considerations.

Seismic Environment

The API seismic environment characterization guidelines address seismotectonic and site descriptions, source to site attenuation characterizations, deterministic and probabilistic seismic exposure evaluations, response spectra and time history ground motion characterizations, and specification of design ground motions.

Earthquake intensities used in the design for strength are referred to as Strength Level Earthquakes (SLE) and have average return periods that are in the range of a few hundred years. The SLE events are generally characterized with three component smoothed elastic response spectra that reflect the source, transmission, and local geology effects. Sometimes, SLE time histories are used.

Earthquake intensities used in design for ductility are referred to as Ductility Level Earthquakes (DLE) and have average return periods that are in the range of a few thousand years. The DLE events are generally characterized with three component ground motion time histories that are based on scaled recorded time histories or that are synthetic ground motion time histories.

These time histories are frequently chosen so that they contain the energies indicated by smoothed elastic DLE response spectra.

Strength Design

In the design for strength, linear, elastic response spectrum analyses provide the basic building block used to develop a detailed understanding of the loading *induced in the platform elements by SLE.* SLE elastic response spectra analyses and the associated member proportioning criteria are a matter of tractability and present analytical capabilities. Definitive guidelines are given for structure modeling to determine modal periods, masses, and damping. Response spectra mode combination guidelines are also given.

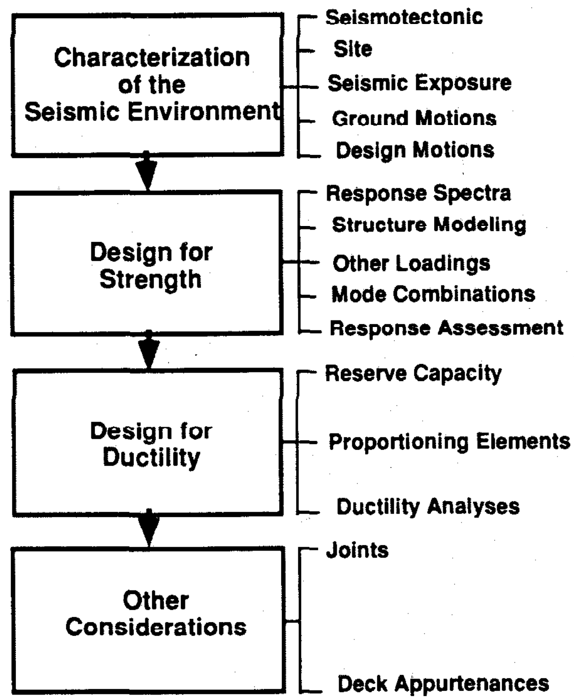

FIG. 2. API Seismic Design Guidelines

The design criteria that determine the SLE loading, member resistance's and factors of safety are chosen to provide acceptable reliability against significant damage or collapse. Earthquakes having intensities equal to that of the SLE are not anticipated to induce any significant damage or inelastic response in the structure elements.

The SLE provisions contain detailed guidance on design of the platform legs, braces, joints, and piles. These guidelines have been based on a substantial body of laboratory and field experimental results involving not only tests on individual elements but as well tests on assemblages of elements (components).

Provisions have been developed for explicit dynamic analyses of the pile foundations (API 1991, Bea 1992a). The pile foundation guidelines provide for characterization of the soils based on field and laboratory tests, recognition of strain rate and cyclic straining effects on the soils, pile-soil interaction analyses, descriptions of the operating and design loadings, and interpretation of the results based on the deformation characteristics of the pile foundation system.

At the present time, the SLE design provisions are stated in both Working Stress Design (WSD) and Load and Resistance Factor Design *(LRFD) formats. Studies were performed to* determine seismic exposure uncertainties and platform system capacity uncertainties (Bea et al. 1987, Moses 1990) and the reliability implied in the design of current platform elements (Moses 1990). In general, the LRFD guidelines were calibrated to produce on the average the same member sizes as derived from the WSD. In an attempt to develop more uniform reliability amongst platform elements, several important exceptions were developed for some platform elements and loading conditions (Moses 1990).

Ductility Design

In the current API guidelines, two approaches can be used in the design for ductility. If the structure is a conventional platform configuration that has 8 or more legs, the ratio of the DLE to SLE intensity is 2 or less, and the foundation soils are stable under the DLE ground motions, then there is no requirement for explicit ductility analyses. The ductility analyses required in previous editions of the API guidelines have been replaced by general good practice guidelines that address the configuration of the legs and braces, joint strength, and member compactness (to allow development of plastic moment capacities). The justification cited in the API guidelines is that analytical experience with this class of platforms has demonstrated that the explicit ductility analyses are not necessary.

If the platform configuration and site soils do not meet the foregoing conditions, then the guidelines suggest ductility analyses be performed. The analyses should demonstrate that the platform is capable of withstanding the DLE without collapsing. General guidelines have been developed by API for scaling DLE ground motion time histories from less intense time histories and for developing synthetic DLE time histories. The guidelines suggest that a minimum of three time histories should be studied.

Static push-over analyses (or their equivalent, ramp acceleration analyses) have been extensively used to demonstrate the capacity and ductility of the platform (Bea et al. 1979; Bea 1991a; Dolan, Crouse, Quilter 1992). In these analyses, an inertial loading pattern is established for the platform nodes (based on linear response spectra analysis results) and this loading pattern is monotonically increased until the platform is no longer able to support its vertical gravity loading (or in many cases, lateral displacements at which the numerical analysis becomes unstable). Alternatively, the base of the structure can be progressively accelerated until the structure becomes unstable ("ramp acceleration" analyses). Such analyses are less difficult and expensive to perform than time history analyses, and these analyses provide important insights into the ultimate limit state characteristics of the platform.

The margin of safety beyond the elastic performance requirement has been expressed with a platform Reserve Strength Ratio (RSR) (Bea et al. 1987; Bea 1992c). The RSR is the ratio of maximum lateral load capacity of the entire platform system to a reference lateral loading induced by the SLE. Platforms designed according to current API guidelines have been found to have RSR ≥ 2 (Bea et al. 1979; Bea et al. 1987).

Other Considerations

The other considerations addressed by the API SDM include structural joints and deck appurtenances. The joint provisions are intended to assure that these critical components are stronger than the braces. The deck appurtenances' provisions are intended to assure that piping, vessels, and other similar important equipment onboard the platforms are properly tied down and designed to have sufficient strength. In recognition of the importance of achieving satisfactory performance of appurtenances and facilities in past severe earthquakes involving onshore industrial structures, the deck appurtenances' provisions recently have been revised with the addition of significantly more guidance on design of the platform topsides and associated equipment (API 1992).

SEISMIC REQUALIFICATION

A Seismic Requalification Methodology (SRM) should provide the engineer with a set of processes and parameters to help assure that an existing platform will be able to develop acceptable performance during its proposed service period. The basic objectives of SRM are to determine if a given platform will have acceptable performance characteristics, and if not, what can be done to make it have such characteristics.

Panel on Seismic Requalification

Recently, a SRM has been developed by a Panel on Seismic Requalification of Offshore Platforms (1992). This SRM is based on a modified API seismic design process (Fig. 3). The SRM defines the requirements for condition surveys, probabilistic seismic hazard studies, strength

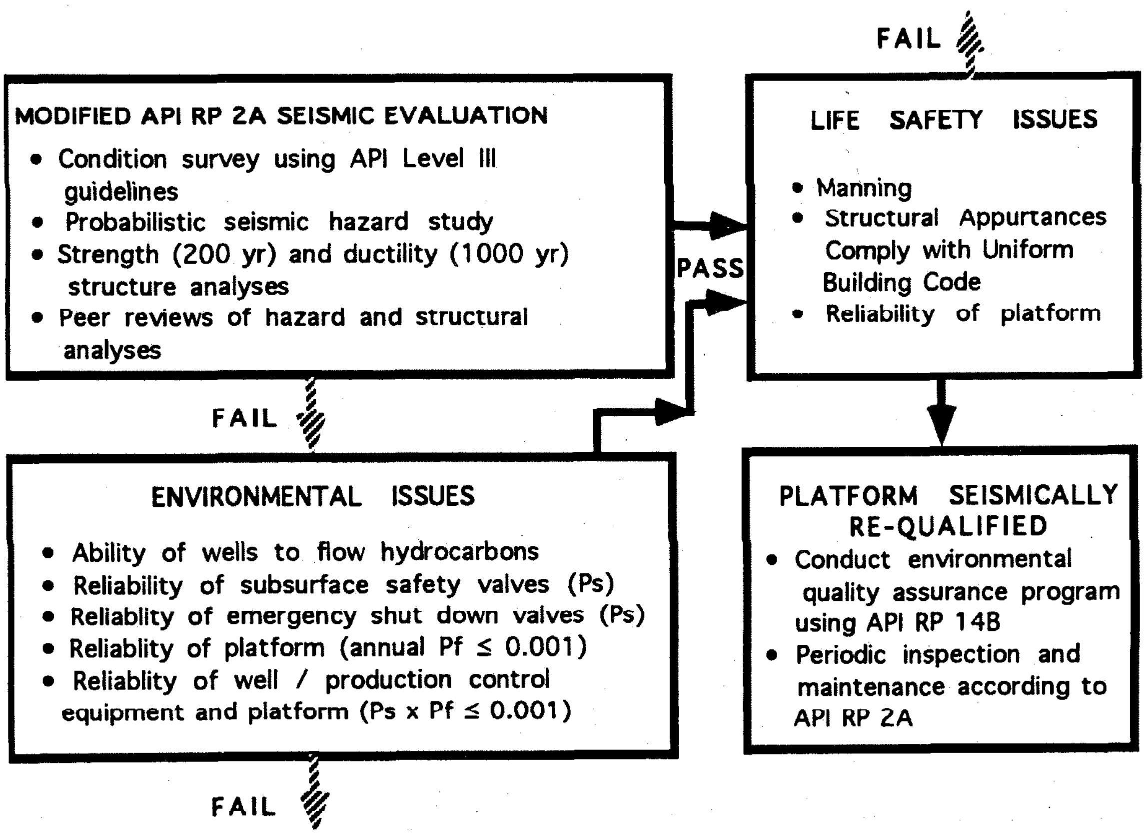

FIG. 3. API Panel seismic requalification procedure

and ductility structure analyses, and peer reviews of the seismic hazard and structure analyses. The SLE return period is defined as 200 years and the DLE return period as 1000 years. The platform must remain elastic in the SLE and not collapse in the DLE.

For a structure that is able to demonstrate such performance, the API Panel SRM guidelines then addresses life safety issues. Manning and the performances of structural appurtenances are addressed. The fundamental performance requirement is that the structure should have an annual probability of failure due to earthquakes (Pfq) which is ≤ 1 E-3.

For platforms that are unable to demonstrate adequate SLE and DLE performance characteristics, then the API Panel SRM addresses environmental issues associated with the release of hydrocarbons from the wells, pipelines, and storage vessels. Given that the size of any potential earthquake caused spill from one source is more than 2,000 barrels, the reliability of subsurface safety valves and emergency shut down valves are addressed and provisions given for demonstrating the reliability of this equipment. The performance requirements are that the platform must have an annual $Pf_Q \leq 1$ E–3 and that the reliability of the environmental safeguard equipment, P_{es}, must be such that $P_{es} \times Pf_Q \leq 1$ E–3 per year.

If at this juncture a platform is able to demonstrate adequate performance characteristics, then if the platform is "manned" life safety issues must be addressed with the considerations and requirements previously discussed. The Panel defines a manned platform as one that is continuously occupied by at least 5 persons.

If at this juncture a platform is able to demonstrate adequate performance characteristics, then if the platform is "manned" life safety issues must be addressed with the considerations and requirements previously discussed. The Panel defines a manned platform as one that is continuously occupied by at least 5 persons.

Once the platform has passed the structural, environmental, and life safety requirements, the platform is seismically requalified given that an environmental quality assurance program and periodic inspection and maintenance plan are developed and implemented.

No specific guidance is given by the API Panel for platforms that fail to pass either the environmental issue provisions or the life safety issues provisions. However, the implication is that the reliability of the environmental safeguard equipment must be increased, the life safety provisions increased, or the reliability of the platform itself increased.

Comprehensive SRM

An alternative SRM has been developed as a result of a long-term effort to develop comprehensive guidelines for the requalification of offshore platforms (Bea, Litton, and Vaish 1985; Bea and Smith 1987; Aggarwal, et al. 1990; McCarthy, Bea, Slosson 1991; Bea, Landeis, Craig 1992; Bea 1992c; 1992e; Sharp, Supple, Smith 1992). This SRM (Fig. 4) is similar in many respects to that developed by the API Panel. However, there are some very important differences. This SRM is analogous to requalification procedures presently being used to extend the useful lives of commercial airframes (Bea 1991f).

The SRM is initiated with a condition survey of the platform and with a characterization of the environment in which the platform is situated. This environment includes not only the seismic environment, but as well the oceanographic and geotechnical environments. Loading, response, and platform performance uncertainties are explicitly addressed including both inherent or natural variability (Type I) and modeling or parametric uncertainty (Type II).

As in the API Panel SRM, the probability of failure of the platform is explicitly addressed. However, in this approach it is the total probability of failure due to all environmental hazards (Pfe) that is addressed. This allows recognition of the potentially important contributions to the probability of platform failure due to storms and other important environmental hazards (e.g., ice loading, sea floor instabilities). In addition, the probability of failure of the operating system (Pfo) is explicitly addressed. This requires a detailed parallel study of the performance characteristics of the topside's equipment and operations not only for earthquakes, but as well for storms, drilling, workovers, and production operations.

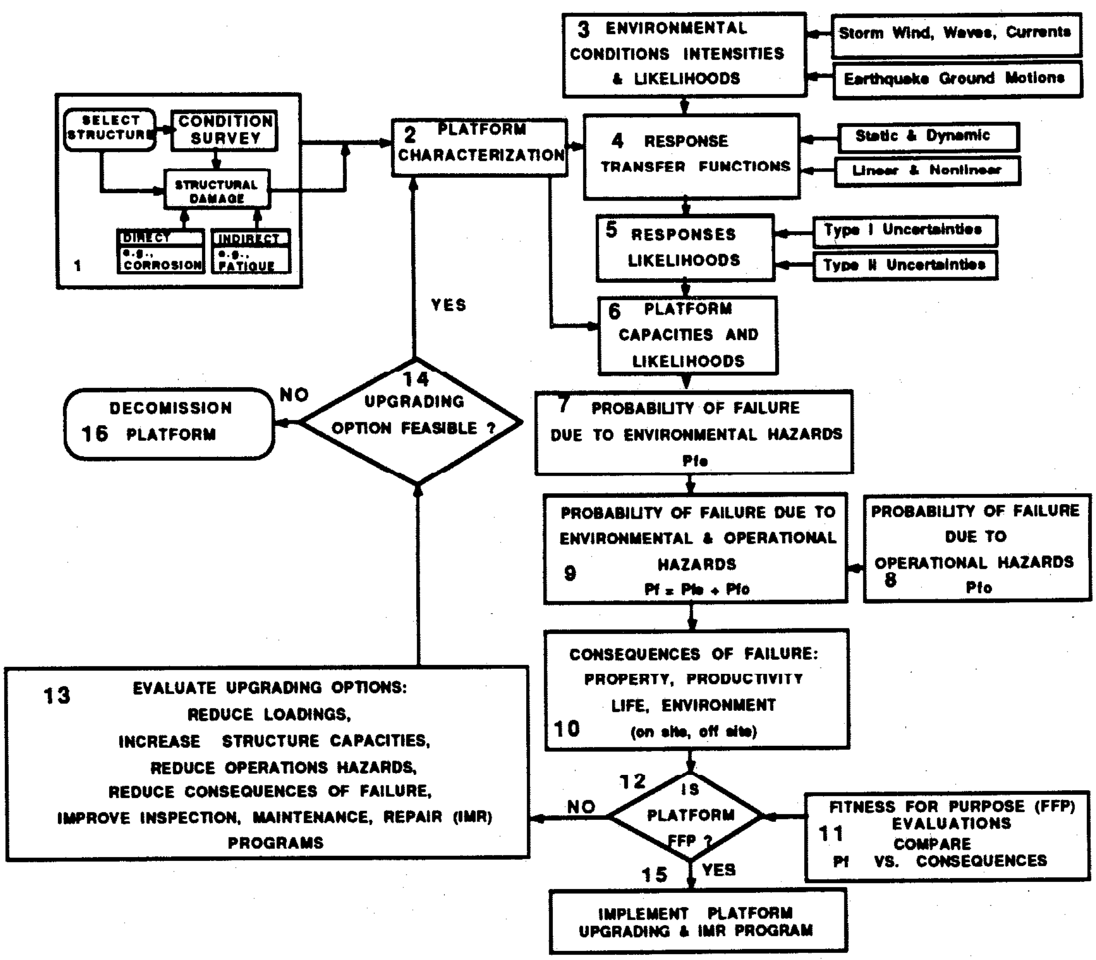

FIG. 4. Comprehensive seismic requalification methodology

In this approach, the judgment of Fitness For Purpose (FFP) is based on the total probability of failure of the platform system, including topside's equipment and operations, and the potential costs and consequences associated with the failure of the platform (Bea 1990c, 1992e).

If a platform is not FFP, then guidance is provided for how to improve the performance characteristics. These improvements include load reduction measures, capacity increasing measures, information development to reduce uncertainties and identify implicit sources of bias, consequence reduction measures (e.g., well and pipeline shut-in equipment, life saving equipment and training), and operating hazards reductions.

The process is continued until a feasible alternative is identified or the platform identified for decommissioning. The feasibility is determined by the projected profitability associated with the platform operations. The costs associated with the Inspection, Maintenance, and Repair (IMR) program (Bea 1991f), must be able to be borne by the platform profitability. Profitability is necessary if the resources to achieve reliability are to be available.

Both economic based FFP guidelines and standard-of-practice guidelines have been developed (Bea, Landeis, Craig 1992; Bea 1991c). The economics based guidelines are based on the premises of expected value cost optimization and differences in economic acceptability of new and old systems. The standard-of-practice guidelines have been based on FFP decisions and judgments for both new and existing platforms. As such these are engineering estimates of the likelihood's of failure and the costs associated with such failure, and design/requalification decisions based on these estimates. These FFP guidelines will be discussed in the last part of this paper.

Although somewhat more complex than the API Panel approach, this approach has several important ramifications. The first is that many structures located in seismic regions have significant contributions to their probabilities of failure from other than earthquakes. This approach recognizes these hazards and addresses mitigation of these hazards.

A second ramification regards the explicit treatment of the operating probabilities of failure. This reflects a SRM that is a part of a total reliability management of a given platform. This is also the derivative of an understanding that as platform's age, they do not tend to get stronger; they tend to get weaker. An older structure has a probability of failure that is greater than its new counterpart. Strengthening and load reduction measures included, frequently the greatest and most cost effective reductions in platform risks (likelihoods and consequences) can be obtained by improving the operating equipment and procedures onboard the platform.

The remaining part of this paper will address reliability, platform capacity, and fitness for purpose considerations in SDM and SRM.

STRUCTURE RELIABILITY

The likelihood (P) that the lateral loading effect (S) equals or exceeds the ULS capacity (R) is

$$Pf = P\,[S \leq R] \qquad (1)$$

Pf is termed the "probability of failure." Its compliment (1−Pf) is the reliability (Ps).

In this development, it will be assumed that the probability distributions that characterize the likelihood of future annual maximum lateral loading effects and the platform capacities are Lognormal. When properly fitted to the extreme condition loading effects and the platform capacities, Lognormal distributions provide acceptable probability characterizations (Bea 1991e; Bea 1992d).

In this development, all probabilities of failure are characterized on an annual basis (probability of failure per year of exposure to the environment). This helps avoid some of the correlation problems associated with life-time loading and capacity characterizations (Bea 1990a).

It is also assumed that the annual maximum loadings and the platform capacities are independent of each other. Although this assumption is not strictly true, it is sufficiently accurate for most SDM or SRM purposes. If the dependency or correlation between the annual maximum loadings and platform capacities can be determined (generally it is negative; as the intensity of the loadings goes up, the platform capacities reduce), then it is relatively easy to include this influence (Bea, 1990a; 1992c; Bea et al. 1992).

Given these assumptions, the annual Safety Index (β), can be determined as follows

$$\beta = \frac{\mu_R - \mu_S}{\sigma} \qquad (2)$$

where

$$\mu_R = \ln R_{50} \qquad (3)$$

$$\mu_S = \ln S_{50} \qquad (4)$$

R_{50} and S_{50} are the 50-th percentile or median values for the ULS capacity and expected annual maximum load effect, respectively. The uncertainties in the expected annual maximum load effect and the ULS capacity are determined as follows

$$\sigma = [\, \sigma_R{}^2 + \sigma_S{}^2]^{0.5} \qquad (5)$$

σ is a measure of the variability or dispersion of the probability distributions. It is computed from the square root of the sums of the squares of the Standard Deviations of the logarithms of R and S (σ_R, σ_S). It reflects the uncertainties associated with the ULS capacity and expected annual maximum loading effects.

An alternative measure of variability is the coefficient of variation (V). V is the ratio of the standard deviation to the mean value of the loading effect or capacity. V is a non-dimensional measure of variability. V is related to σ as follows

$$V = [\exp(\sigma) - 1]^{0.5} \quad (6)$$

The probability of failure is related to the Safety Index as follows

$$Pf = 1 - \Phi[\beta] \quad (7)$$

where $\Phi[\beta]$ is the Standard Normal distribution cumulative probability of the variate, β, in the interval $-\infty$ to β. With a high degree of accuracy $\Phi[\beta]$ can be determined as follows ($\beta \geq 0.5$)

$$\Phi[\beta] = 1 - 0.5(1 + C_1\beta + C_2\beta^2 + C_3\beta^3 + C_4\beta^4)^{-4} \quad (8)$$

where $C_1 = 0.19685$, $C_2 = 0.11519$, $C_3 = 0.00034$, and $C_4 = 0.01953$.

Similar to a factor-of-safety, the Safety Index is a normalized index of the platform reliability

$$Pf \approx 10^{-\beta} \quad (9)$$

Another useful approximation for $1 \leq \beta \leq 3$ is

$$Pf \approx 0.475 \exp(-\beta^{1.6}) \quad (10)$$

Given the foregoing expressions, the median ULS capacity, R_{50}, can be related to the median expected annual maximum loading effect, S_{50}, as follows

$$R_{50} = S_{50} \exp(\beta\,\sigma) \quad (11)$$

Rearranging Eqn.. 11, the median or central factor of safety (FS_{50}) can be expressed as

$$FS_{50} = \exp(\beta\,\sigma) \quad (12)$$

The factor of safety (Fig. 5) is directly dependent upon the desirable or acceptable reliability of the platform and the total uncertainties in the platform loadings and capacities. Due to the exponential dependence upon $\beta\,\sigma$, small changes in either of the parameters can lead to large changes in the required factor of safety.

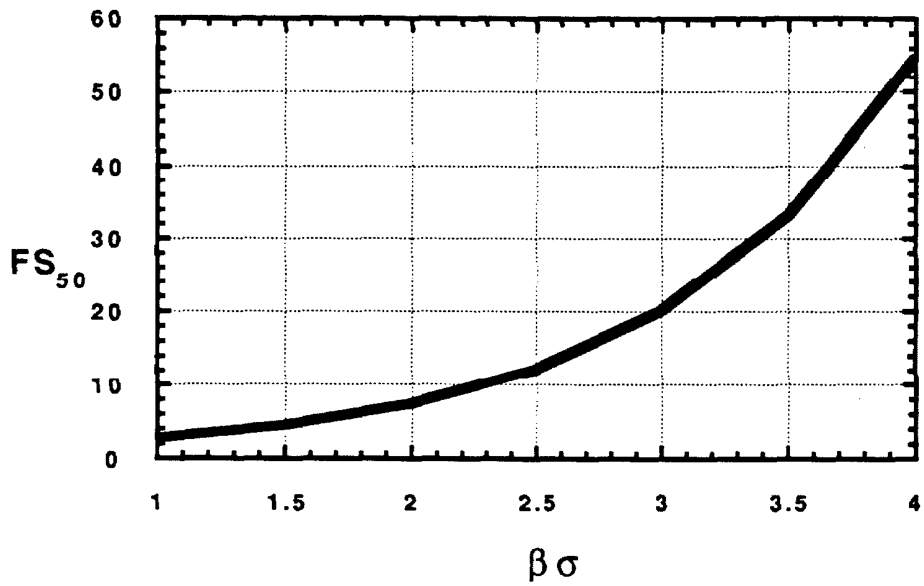

FIG. 5. Relationship between the required central factor of safety for a given reliability and uncertainty in loadings and capacities

Uncertainties and Biases

In this development, uncertainties are organized into two general categories. The first category is identified as natural or inherent randomness (Type I = "randomness").

The second category is identified as unnatural, cognitive, or modeling uncertainty (Type II = "modeling"). This type of uncertainty applies to fixed or deterministic, but unknown values of parameters (parameter uncertainty); to modeling uncertainty (imperfect understanding of problems, simplified theories or analytical models used in practice); and to the actual state of the system or element (imprecise knowledge of properties and characteristics). Most importantly, Type II uncertainties are information sensitive in that they can be changed by information gathering and analyses. This is an important consideration in development of reliability based SRM.

Type II uncertainties will be expressed in the form of "biases." Bias, Bx, in a parameter, x, can be expressed as the ratio of the true value to the predicted or nominal value of the parameter

$$Bx = \frac{X_{true}}{X_{predicted}} = \frac{X_{true}}{X_{nominal}} \qquad (13)$$

The Type II uncertainties will be assumed to be Lognormally distributed and described by their means (X) and Coefficients of Variation (Vx). Type I and Type II uncertainties will be assumed to be uncorrelated.

In this format, **R** and **S** must be replaced by the nominal values (**R_N**, **S_N**) times a mean Bias factor, **B**, for the particular nominal value

$$\mathbf{R} = \mathbf{B_R}\ (\mathbf{R_N}) \quad (14)$$

and

$$\mathbf{S} = \mathbf{B_S}\ (\mathbf{S_N}) \quad (15)$$

Similarly, the Coefficients of Variation will be determined as follows

$$V_R \cong \sqrt{V_{RN}^2 + V_{BR}^2} \quad (16)$$

and

$$V_S \cong \sqrt{V_{SN}^2 + V_{BS}^2} \quad (17)$$

If it is presumed that the distributions of S and R are based on nominal values

$$R_{N50} = \frac{B_{50S}}{B_{50R}} S_{N50} \exp(\beta\ \sigma) \quad (18)$$

It is critical to address both types of uncertainties in as clear and unambiguous way as is possible (Bea 1992). One of the most important parts of SRM is the identification implicit sources of bias in the processes or evaluations that lead to identification of the platform loading and capacity characterizations. This is one of the most important differences between SDM and SRM. What can frequently be tolerated or afforded in the way of implicit conservatisms in an SDM can not be tolerated or afforded in an SRM (Dolan, Crouse, Quilter 1992).

In the author's experience, these implicit sources of conservatism are frequently integrated into the results by those that do not understand that uncertainties can be explicitly recognized and accounted for in development of the SDM or SRM. As has been demonstrated, uncertainties result in larger required factors of safety and larger capacities in the platform. In addition, the decision making processes frequently become very dependent upon how the different sources of uncertainties have been recognized and integrated into the analyses (Bea et al. 1992).

An important example of Type II uncertainties in SRM are the attenuation uncertainties that can be integrated into seismic exposure characterizations. These uncertainties can be very large and are directly dependent upon the particular attenuation relationship that is used and how this attenuation relationship is compared with measured data (Bea 1991e; Bea 1992d). The API Panel SRM clearly identifies this source of uncertainty in seismic exposure characterizations and requires that this source of uncertainty be directly integrated into the analyses of Pf (Panel on Seismic Requalification of Offshore Platforms 1992).

It is not always possible to develop unambiguous definitions of Type I and Type II uncertainties. Natural or inherent randomness can be inexorably mixed with modeling uncertainties. Comparisons of earthquake attenuation relationships with measured data is one excellent example. Not only are attenuation model uncertainties involved in such comparisons, but as well natural or inherent variability in the ground motions. However, in the author's experience, the processes of identifying and analyzing these sources of uncertainties are very worth the trouble because of the new information and understanding that is frequently developed. A primary benefit is development of an understanding of how data gathering and additional research and development activities can lead to improvements in the reliability of the platform, and hence in reductions in the required margins of safety.

Table 1 summarizes the general ranges for Type I and Type II uncertainties associated with annual maximum environmental loadings and loading effects and with ULS capacities of new and existing template type platforms (Bea 1979, Bea et al. 1987; Bea 1990b; 1991b; 1991d; 1991e; 1992d; Bea, Landeis, Craig 1992; Ekngesvik, Olufsen, Karunakaran 1987; Moses 1990).

Table 1
General Ranges of Uncertainties in Loadings and Capacities

Source of Uncertainty	Type I V	Type II V	Type I & II V
Operating Loadings			
- drilling	0.10 - 0.15	0.05 - 0.10	0.11 - 0.18
- production	0.10 - 0.15	0.05 - 0.10	0.11 - 0.18
Storm Loadings			
- tropical cyclone areas	0.30 - 0.40	0.30 - 0.40	0.42 - 0.56
- extra-tropical cyclone areas	0.10 - 0.20	0.30 - 0.40	0.32 - 0.45
Earthquake Loadings			
- plate border areas	1.0 - 2.0	0.40 - 0.80	1.1 - 2.2
- interplate areas	1.0 - 3.0	0.40 - 0.80	1.1 - 3.1
Ice Loadings			
- sheet (first year)	0.35 - 0.40	0.30 - 0.40	0.46 - 0.57
- iceberg	0.60 - 1.50	0.40 - 0.50	0.72 - 1.60
Soil Movement Loadings	0.50 - 0.60	0.30 - 0.40	0.58 - 0.72
New Platforms			
- structure fail modes	0.10 - 0.20	0.10 - 0.20	0.14 - 0.30
- foundation fail modes	0.20 - 0.40	0.20 - 0.30	0.28 - 0.50
Existing Platforms			
- structure fail modes	0.15 - 0.25	0.15 - 0.25	0.21 - 0.35
- foundation fail modes	0.20 - 0.40	0.20 - 0.30	0.28 - 0.50

In general, loading effects uncertainties dominate. However, when information sensitive Type II uncertainties are examined, in the case of existing platforms, the capacity uncertainties can be important, particularly in the case of failure modes that are concentrated in the foundations of the platform (Tang 1988; 1990). Very large uncertainties can accompany some types of foundations (e.g. drilled and grouted), and some types of soils (e.g. calcareous) (Bea 1990a).

The larger uncertainties associated with existing platforms are primarily attributable to the additional uncertainties in the capacities of these structures contributed by time related damage such as corrosion and fatigue, damage from operations (e.g. denting), and uncertainties contributed by lack of complete information on the present condition of the structure.

PLATFORM CAPACITY CONSIDERATIONS

A set of terms that are intended to describe the ULS capacity characteristics of a platform will be introduced (Fig. 6). The first term is the "Reserve Strength Ratio" (RSR):

$$RSR = R_U / S_D \qquad (19)$$

R_U is the ultimate or maximum load resistance developed by the platform. S_D is the design load effect. In the case of SDM and SRM, S_D generally results from the application of the SLE response spectra and the determination of the global or resultant lateral loadings that are associated with the SLE.

The second ULS characterizing term is the "Residual Strength Ratio" (α)

$$\alpha = R_R / R_U \qquad (20)$$

R_R is the residual strength or load resistance of the platform that is developed at the point of collapse (platform no longer able to support its vertical gravity loads).

The third ULS characterizing term is the "ductility" (μ):

$$\mu = \Delta p / \Delta e \qquad (21)$$

Δp is the maximum plastic deformation that can be developed by the platform at the point of collapse. Δe is the deformation at which the platform first exhibits significant inelastic behavior.

In this development, it is assumed that RSR, α, and μ will be defined using static push-over analyses. In such analyses, a lateral loading pattern acting on the platform is developed based on the loadings determined from the SLE response spectra analyses. The lateral loadings are then uniformly increased until the platform is no longer able to support the vertical loadings. At this point, it is assumed that platform collapse has occurred.

The RSR indicated by a static push-over analysis should not be interpreted as the true RSR. It is a nominal value based on a simplified analysis of the nonlinear, inelastic response of the structure. It is an "index" of the strength or capacity of the structure (Bea 1992c, 1992e). Similarly, the deformation capacity of the structure indicated by a static push-over analysis is a vague reflector of the true deformation capacity of the structure when it is subjected to an intense earthquake that brings the platform system to its ultimate limit state.

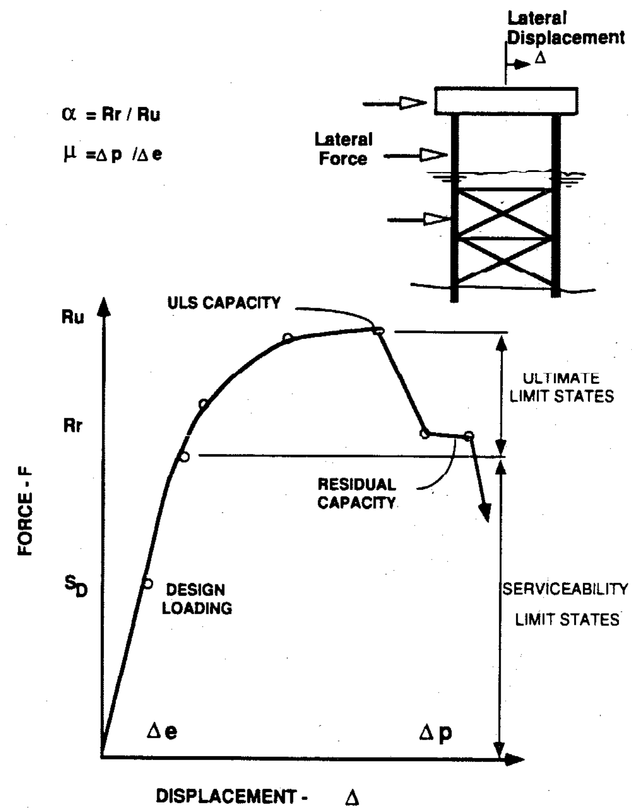

FIG. 6. Characterization of platform ULS load and deformation capacity

The true ULS capacity of the structure is not only a function of the load resistance, but as well of the ductility developed by the structure. The ductility is influenced by the nonlinear hysteretic characteristics of the components and elements that comprise the platform system. Also, it is influenced by the configuration of the elements, as this determines how effectively load-redistribution paths can be developed when elements in the structure loose their load carrying capacity and stiffness ("effective redundancy").

Based on response analyses of a variety of nonlinear hysteretic systems having natural periods $T \geq 1$ sec. subjected to a variety of recorded and synthetic earthquake acceleration time histories (Bea 1992) the following expression has been developed to express the mean effects of ductility and residual strength (Fig. 7):

$$RSR = RSR_S\ (\alpha\ \mu) = RSR_S\ (Fv) \qquad (22)$$

RSR_S is the static push-over strength, and μ is the ductility that can be sustained in repeated plastic cycling (numbers of cycles determined by the specific sequences of earthquake energy "pulses", and the duration of these pulses and of the earthquake). The scatter of results around the mean trends shown in Fig. 7. is in the range of V = 30% to 40% with increasing variability attributed to increasing levels of ductility developed in the system. This is a natural or inherent variability that is due to the natural variability in earthquake ground motions. The differences in the sequences of ground motion pulses and the numbers of strong pulses develop differences in the nonlinear response characteristics of the system.

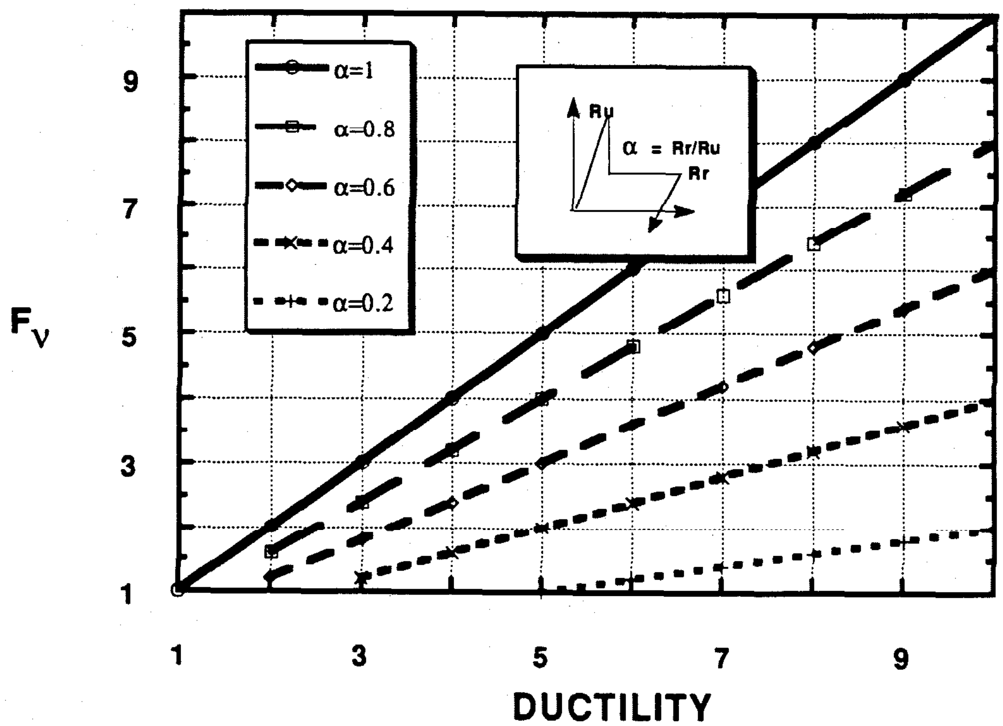

FIG. 7. Mean earthquake capacity modifier for Tn = 1 sec., D = 5% for range of residual load resistance ratios and ductility

Very comparable analytical results for an "experimental offshore platform test frame" (Popov, Mahin, Zayas 1980; Zayas, Shing, Mahin, Popov 1981) have been recently published (Bazzurro and Cornell, 1992). These results also are in substantial agreement with previous studies of the nonlinear response characteristics of idealized and realistic structural systems similar to those of offshore platforms subjected to earthquakes (Iwan 1980; Riddell, Newmark 1979).

There can be similar types of ductility and residual strength dependent loading effects factors that can have important influences on the characterizations of platform capacities in extreme ice impact and wave loading conditions (Bea 1991e; 1992b; 1992d; McDonald, Bea 1992; Stewart 1992). An example of the extreme condition wave loading mean capacity modifiers is given in Fig. 8. As for the earthquake capacity modifiers, there is a variability associated with these mean trends that generally falls in the range of V = 30% to 40%. As for the earthquake results, this is a natural or inherent variability that is attributed primarily to the natural variability in extreme sea state characteristics.

Results from analyses of the nonlinear response characteristics of multi-degree of freedom structures characteristic of those associated with conventional template type offshore platforms indicates that in most of the cases studied, the systems are able to develop global or system ductility in the range of μ = 3 to 4 and residual strength ratios in the range of α = 0.8 to 1.0 (Bea

1979; 1992b; 1992c). This behavior is attributed to the interactions of tensile yielding and compressive buckling braces in the jacket, the development of ductile plastic hinges in the compact sections that comprise the deck legs and supporting system, and the ductile behavior of the foundation piles and conductors. In some cases however, brittle behavior of the tubular joints can result in much less ductility and residual strength.

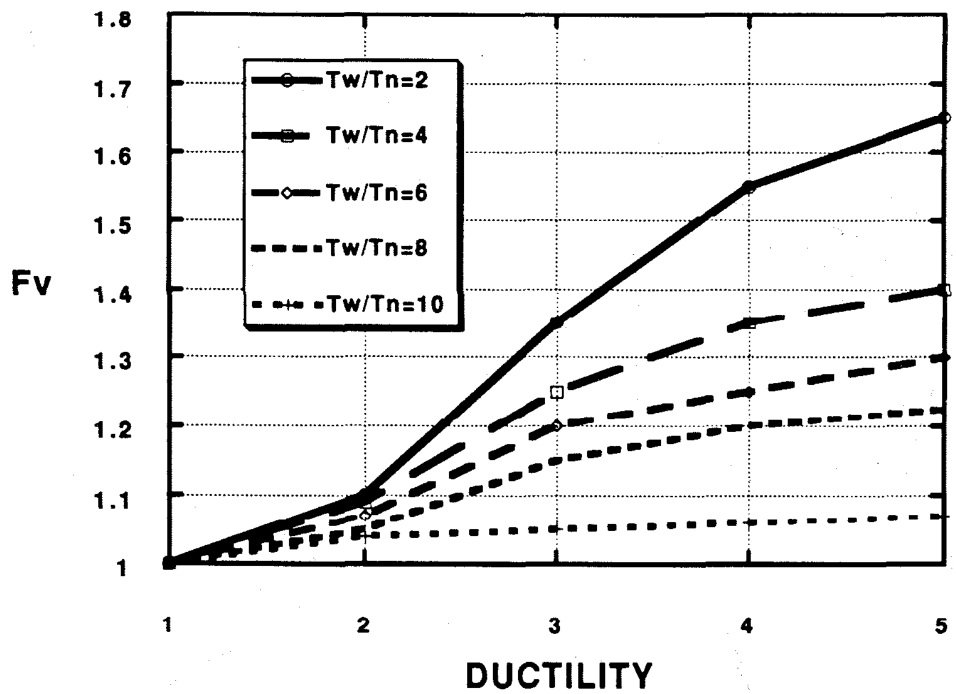

FIG. 8 Mean wave loading effects capacity modifier as a function of the ratio of the maximum wave height period (Tw) to the platform natural period (Tn)

In the case of SRM, remedial grouting of the deck legs, the jacket legs, joints and braces, and the insides of the piles can prove very beneficial if the quality of such remedial work can be assured in the field. These results have very important ramifications for the characterization of the ULS capacity characteristics of platforms as the equivalent ULS load resistance is directly dependent upon the ductility and residual strength that can be developed in the critical components of the platform.

An alternative to static push-over analyses are nonlinear time history analyses (Bea, Landeis, Craig 1992; Dolan, Crouse, Quilter 1992). Nonlinear time-history analyses are able to capture the transient loading characteristics that are lost in the fixed loading pattern of static push-over analyses. Further, time-history analyses are able to preserve the phasing of the ground motions and the resultant phasing of the motions and forces induced in the structure that are lost in static push-over analyses. Due to the lack of uniqueness of a single ground motion time history, a number of time histories must be used to bracket the potential response characteristics of the structure. These time histories should be chosen to simulate the primary sources of extreme earthquakes that are expected to influence the platform site.

In the case that nonlinear time history analyses are used to determine the ULS capacity of the structure, then the total lateral loading that produces collapse of the structure can be used directly as the estimate for R_{50}. Note that the median R should be estimated (not best or worst from the analyses) together with an estimate of the uncertainties associated with the evaluated capacities. Also, note that unbiased (best estimate) values should be used for all of the structural and foundation element properties and characterizations. Nominal values used in design and intentionally conservatively biased values frequently used in analyses should not be used.

Given the foregoing developments, the design RSR can be expressed as

$$RSR = \frac{B_{50S}}{B_{50R}} R_F \exp(\beta\,\sigma) \qquad (23)$$

R_F is the "Force Ratio"

$$R_F = \frac{S_{N50}}{S_D} \qquad (24)$$

Given the assumption that the probability distribution of the earthquake maximum forces is reasonably described by a Lognormal distribution, the Force Ratio is

$$R_F = \exp(-K\,\sigma_S) \qquad (25)$$

where

$$K = \Phi^{-1}(1-T^{-1}) \qquad (26)$$

T is the average return period in years associated with the design event. For T = 200 years, K = 2.576. For T = 400 years, K = 2.807.

Based on the foregoing developments

$$RSR_S = \frac{B_{50S}}{B_{50R}} \frac{R_F}{Fv} \exp(\beta\,\sigma) \qquad (27)$$

Earthquake Induced Loadings

Fig. 9 shows the variation of the SDM or SRM RSR with the product $(\beta\sigma)$ for a design or requalification SLE having a return period of 200 years, and for "unbiased" estimates of the earthquake induced loadings and capacities (B_{50S}, B_{50R}= 1.0). The RSR is strongly influenced by the required reliability of the structure, the total uncertainties in the platform capacities and loadings. As the requirements for safety increase and as the total uncertainties increase, the RSR must increase. For a value of $(\beta\sigma) = 3.8$ ($Pf \approx 1$ E–5), RSR = 3.4. Given Fv = 2.0, RSRs = 1.7.

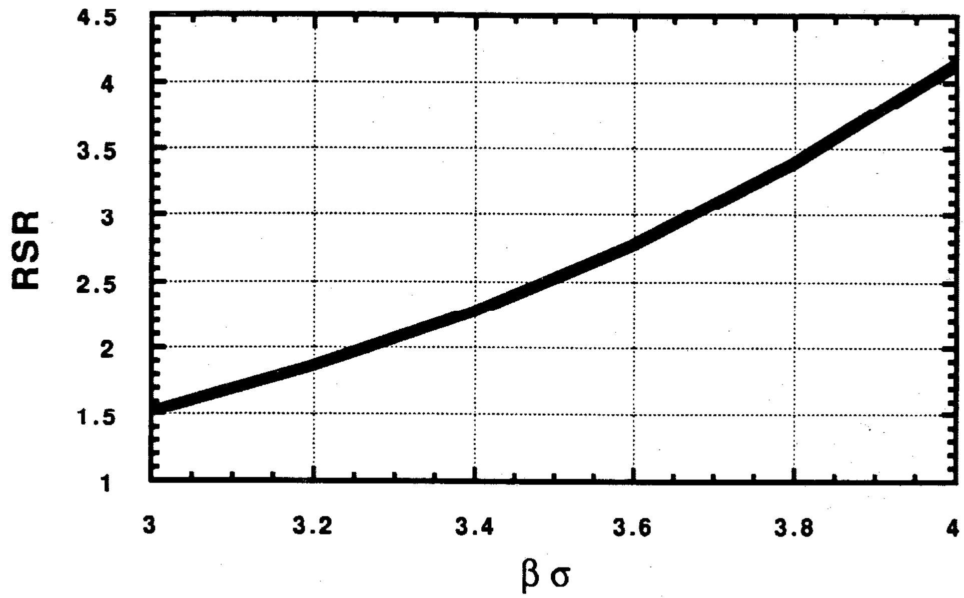

FIG. 9. Reserve Strength Ratio as function of the product of the annual Safety Index and the total uncertainty for the SLE defined on the basis of a 200 year return period

Fig. 10 summarizes the required platform RSR as a function of the design SLE return period for a Safety Index of $\beta = 4.0$, and total uncertainty $\sigma = 1.0$. For SLE return periods in the range of 100 to 300 years, RSR = 5.3 to 3.5. As the design SLE return period increases, the design RSR decreases. For Fv = 2.0, the equivalent design static push over RSRs = 2.7 to 1.8.

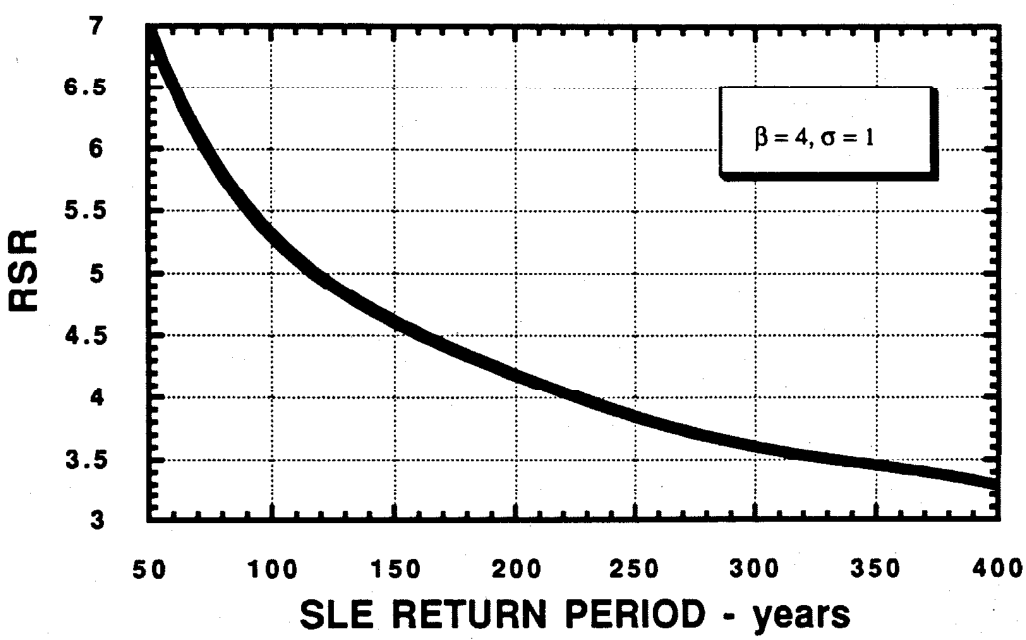

FIG. 10. Reserve Strength Ratio as function of the SLE return period

The Average Return Period (T) associated with the expected annual maximum loading effect, S, can be expressed as:

$$T = \frac{1}{1 - F(s)} \qquad (28)$$

where

$$F(s) = Pa\,[\,S \le s\,]) \qquad (29)$$

F(s) is the annual cumulative probability (Pa) that the expected maximum loading, S, is equal to or less than a given value, s.

Using the approximation for the Lognormal distribution given in Eqn. 10, T associated with a given value of the expected annual maximum loading is

$$T_{ULS} = 2.1 \exp\,(\beta\,\nu)^{1.6} \qquad (30)$$

where

$$\nu = \frac{\sigma}{\sigma_S} \ge 1.0 \qquad (31)$$

ν is the uncertainty ratio. The uncertainty ratio reflects the relative contributions of the uncertainties in the projected future demands (loadings) imposed or induced in the platform and the uncertainties in the projected future ULS capacity of the structure. For $\nu = 1.0$, there is no contribution from the capacity uncertainties.

Fig. 11 summarizes the relationship between the safety index, the uncertainty ratio, and the return period (years) associated with the ULS capacity. In the case of earthquake induced loadings, this return period could be interpreted as the return period associated with the DLE. For example, for $\beta = 3.2$ and $\nu = 1.05$, $T_{ULS} = 2{,}200$ years and for $\nu = 1.1$ $T_{ULS} = 3{,}800$ years. The ULS return period for a given Safety Index is dramatically influenced by the uncertainty ratio. As the uncertainty contributed by the platform capacity increases, there is a commensurate increase in the return period associated with the DLE.

Note that if $T_{ULS} = 1{,}000$ years (as suggested by the API Panel SDM) and $\nu = 1.05$ (typical for existing offshore platforms in plate boundary seismic zones), then $\beta = 2.9$ and $Pf \approx 2$ E–3 per year.

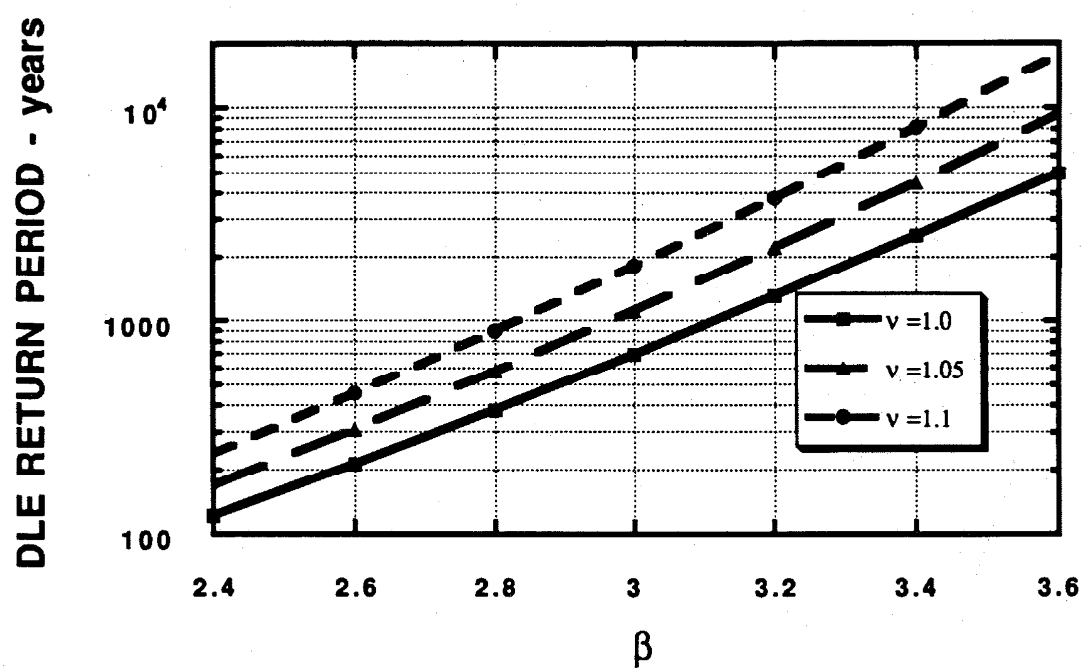

FIG. 11. Relationship between the annual Safety Index and the DLE return period

Other Environmental Loading Hazards

Typically, for other environmental loading hazards, the platform is designed or requalified using elastic analyses and allowable stresses for 100-year conditions. For storm loadings, the loading uncertainties (expressed on an annual basis) can fall in the range of $\sigma_I = 0.30$ to in excess of $\sigma_{I\&II} = 0.80$ (Bea 1990a; 1990b; 1991d). There is a comparable range for ice loadings (Bea 1991e; Bea, Landeis, Craig 1992).

Given that unbiased estimates have been used for the platform loadings and capacities, Fig. 12 summarizes the annual probabilities of failure for a typical range of Reserve Strength Ratios, a capacity uncertainty $\sigma_R = 0.25$, and design according to API guidelines for loading events having return periods of T = 100 years. For this set of conditions, for an RSR = 2, Pf ≈ 1 E–3 to 2 E–4 ($\sigma = 0.3$ to 0.8).

Fitness for Purpose (FFP) Considerations

Two approaches to the definition of FFP criteria will be discussed (Bea, Landeis, Craig 1992; Bea 1992e). The first will be termed the "utility optimization" approach. The second will be termed the "standard of practice" approach.

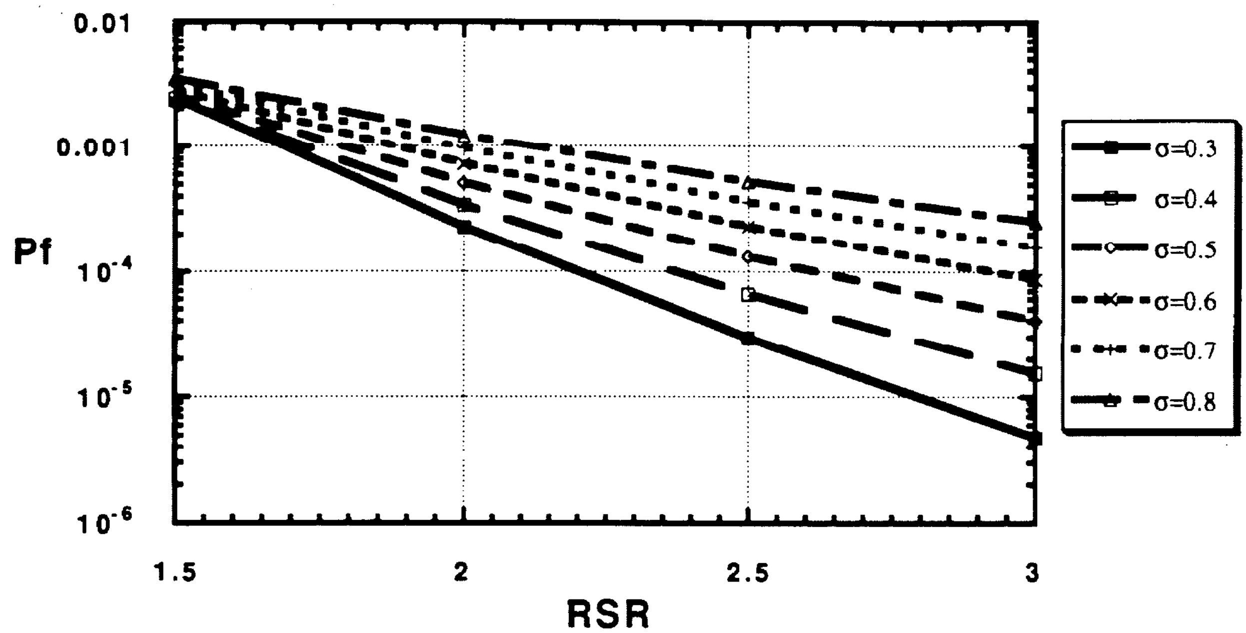

FIG. 12 Relationship of RSR and Pf for a range of environmental loading uncertainties

The utility optimization approach is based on an evaluation of the positive and negative utilities associated with risks. The utilities can be measured in a variety of ways. Costs and benefits measured in monetary terms are one of the most common examples of this approach. The objective of this approach is frequently expressed as total expected cost minimization.

The standard of practice approach is based on current design and requalification decisions associated with other platforms. The general premise of this approach is that the profession, industry, and regulatory agencies over time and through experience define acceptable combinations of probabilities of failure and consequences associated with failure.

Utility Approach

Given that there is a linear relationship between the logarithm of the probability failure and the initial cost, the Pf that produces the lowest total expected combination of initial and future costs (Pfa) can be estimated from

$$Pfa = 0.435 / (PVF\ R_C) \qquad (32)$$

R_C is the cost ratio: the ratio of the expected cost of the platform loss of serviceability (Cf) to the cost needed to decrease the annual likelihood of the platform loss of serviceability by a factor of 10 (ΔC)

$$R_C = Cf / \Delta C \qquad (33)$$

PVF is a Present Value Function intended to discount potential future costs to present value terms. In the case of a continuous replacement based operation that has an exposure period (L, in years), and a net discount rate (r) the PVF can be expressed as

$$PVF = [1 - (1 + r)^{-L}] / r) \qquad (34)$$

For long lives and continuous replacement, PVF = 1/r. For short lives, PVF = L. For the cases of non-replacement conditions, and deferred revenue considerations, more complex PVF's need to be considered (Fu et al. 1992).

The marginal probability of failure (Pfm) can be defined as the condition when the increment in money spent to improve reliability equals the increment in expected present valued future costs saved. For this assumption (Bea 1991c; Bea 1992e)

$$Pfm = 2(Pfa) \qquad (35)$$

Fig. 13 summarizes the foregoing results as a graph of the consequences of failure measured by the product of Rc times PVF and the annual probability of failure that would be "acceptable" and "marginal." As would be expected there is an inverse logarithmic linear relationship between Pf and the consequence measure; high consequences require high reliability and low probabilities of failure.

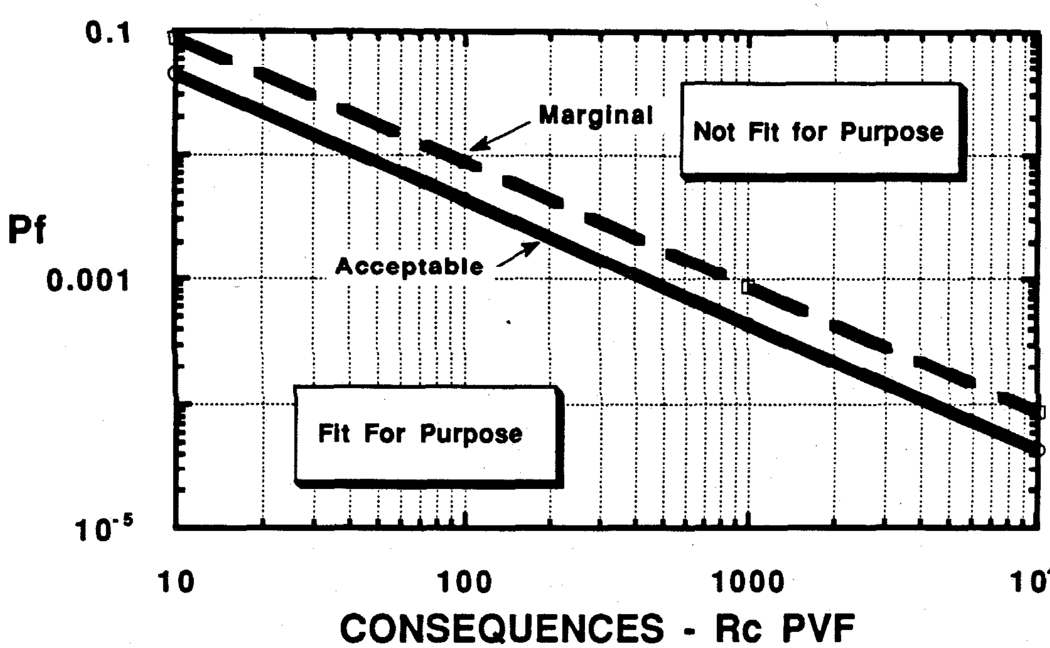

FIG. 13. Economics based fitness for purpose criteria

Standard of Practice Approach

Results from the development of the standard of practice approach are summarized in Fig. 14 (Bea, Landeis, Craig 1992; Bea 1992b; 1992e). This development is founded on notional or calculated reliabilities (including only Type I uncertainties) for both environmental and operational hazards). This avoids many of the problems associated with historic or actuarial failure data which reflects a variety of causes and problems associated with the small numbers of platform failures (Bea 1990c, 1991c).

This development also is based on current estimated total costs of loss of serviceability (property, injuries, lost production, salvage, pollution). Again, this avoids some of the problems associated with historic data based assessments of the costs associated with loss of serviceability.

The objective of the standard of practice approach is to utilize notional or calculated likelihoods and projected costs associated with catastrophic loss of serviceability to help assure consistent and reasonable judgments for a wide variety of offshore platforms and operations.

The values shown in Fig. 14 have been used recently by the offshore industry and associated regulatory bodies to assist development of judgments and decisions regarding design of new platforms and re-qualification of existing platforms. These judgments have ranged from design criteria for billion dollar installations (Bea, Lee, Moore 1991) to re-qualification of platforms that have very low consequences associated with their failure (Bea 1992c). They have included manned and unmanned platforms and platforms that are not required to evacuate personnel in advance of severe environmental conditions.

The lines identified as marginal and acceptable are intended to help discriminate between fitness for purpose criteria for new and existing platforms. It is interesting to note that in several recent cases, and using a total reliability management philosophy, the requalification efforts have resulted in platforms having overall reliabilities commensurate with those of new platforms.

Comparison of the results summarized in Fig. 13 and Fig. 14 indicate very similar results for comparable measures of the consequences of failure. For example, based on the Utility approach (Fig. 13) given that $\Delta C \approx 0.1$ Cf and PVF ≈ 10, for Cf = \$100 millions, Pfa ≈ 4 E–3 per year and Pfm ≈ 8 E–3 per year. Based on the Standard of Practice approach (Fig. 14) for Cf = \$100 millions, Pfa ≈ 3 E–4 per year and Pfm ≈ 7 E–3 per year.

Total Probability of Failure

The total probability of failure of the platform, Pf_T, can be represented as the sum of the probability of loss failure due to operating hazards, Pf_O, and due to environmental hazards, Pf_E

$$Pf_T = Pf_O + Pf_E \qquad (36)$$

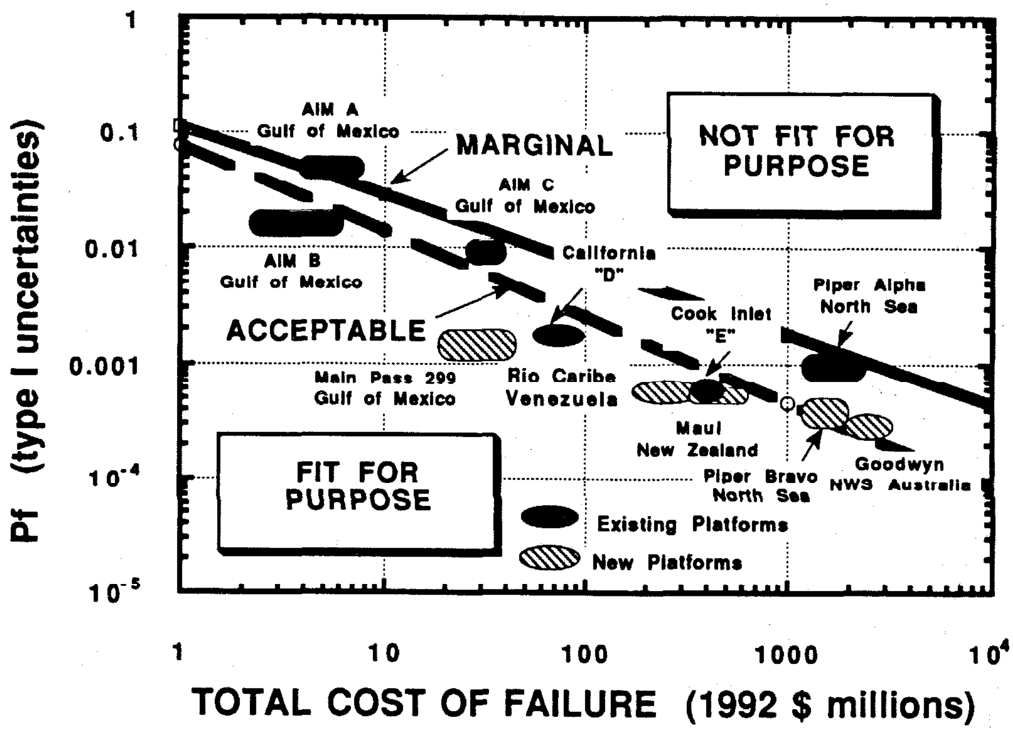

FIG. 14. Standard-of-practice fitness for purpose criteria

Pf_O is a function of the probabilities of platform failure due to events such as blowouts (Pf_B, during primary drilling or workovers), fires (Pf_F) and explosions (Pf_{EX}) due to process systems failures, and collisions (Pf_C). Pf_O also recognizes the probabilities of failure due to fires, explosions and blowouts conditional on the occurrence of extreme environmental events. Assuming independent operating hazards

$$Pf_O = Pf_B + Pf_F + Pf_{EX} + Pf_C \quad (37)$$

For offshore operations associated with self-contained drilling and production platforms, the data summarized in Fig. 15 indicates that Pf_O generally accounts for 70% to 80% of Pf_T (Bea 1990c; 1991c; Bekkevold et al. 1990; Veritec 1985). Human and organization errors are a major contributor to Pf_O (Bea 1990; Bea, Moore 1991).

Given a moderate consequence platform that had Cf = $500 millions, Fig. 14 indicates $Pf_{T\,(acceptable)} \approx 1$ E–3 and $Pf_{T(marginal)} \approx 5$ E–3. If this were a typical drilling and production platform, then one could expect $Pf_O \approx 0.8\, Pf_T$. This would indicate $Pf_E = 0.2\, Pf_T$ or $Pf_E = 2$ E-4 to 1 E–3.

Pf_E is a function of the probabilities of loss of serviceability due to events such as extreme storms (or ice loadings in the case of platforms in the Arctic), Pf_S , and extreme earthquakes, Pf_Q:

$$Pf_E = Pf_S + Pf_Q \qquad (38)$$

Given the results in Table 1 and Fig. 12, one could observe that in many cases Pf_Q can be approximately equal to Pf_S. In such a case, given the moderate exposure platform, Pf_Q = 1 E–4 (acceptable) to 5 E–4 (marginal). For these conditions, the results indicate somewhat more stringent requalification criteria than suggested by the API Panel (1992). This has been the general experience in several recent seismic requalification verification studies (Miller et al. 1991; Bea, Landeis, Craig 1992; Dolan, Crouse, Quilter 1992).

As applied to a SDM for a new platform, these results would indicate an DLE with a return period in the range of 10,000 years (Fig. 11) and an SLE with a return period in the range of 200 years for an RSR = 4. Given that the structure could develop Fv = 2, then the static push-over RSRs ≈ 2.0. In general, these results agree quite well with present SDM practice and results.

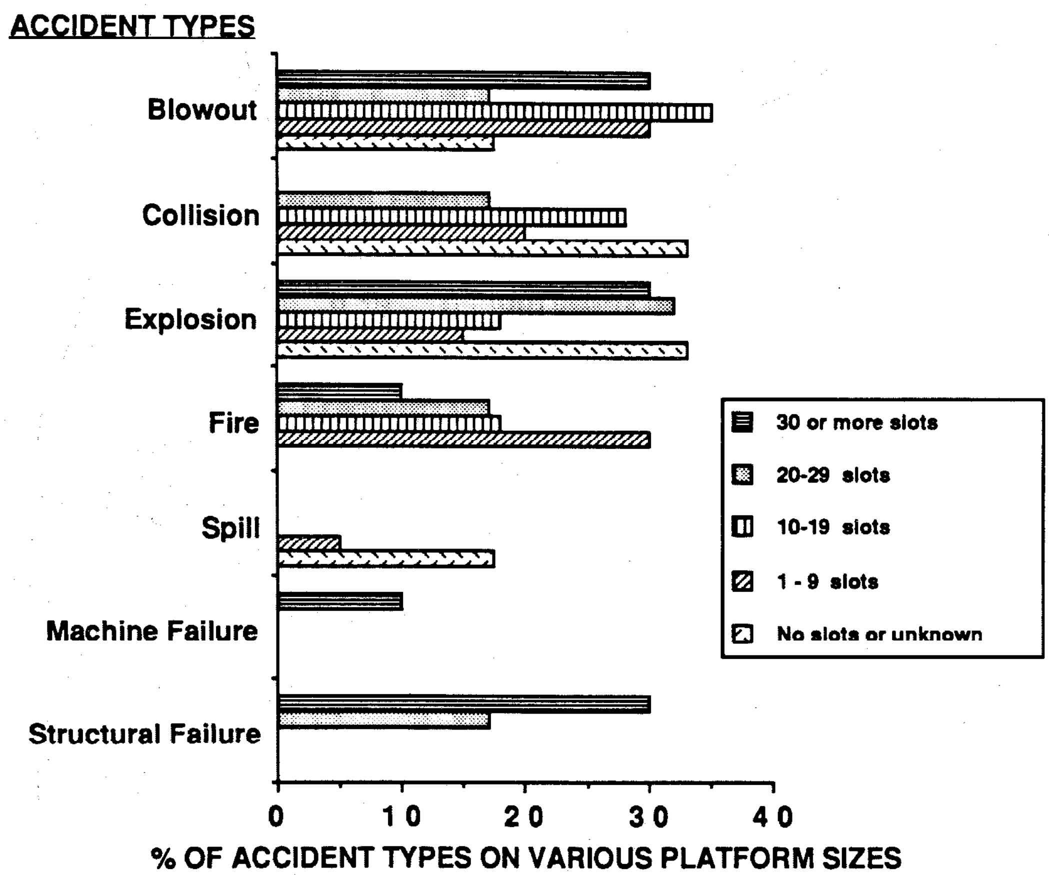

FIG. 15. Drilling and production platform accidents based on WOAD

CONCLUSIONS

Based on the developments summarized in this paper, it is concluded that SDM have been highly developed. Explicit and detailed guidelines associated with the API SDM have been developed, tested, and implemented during the last 20 years. Experience with the API SDM has been good.

However, SRM have not been so highly developed. It has been only during the past five years that formal SRM have been developed. The two SRM that have been published to date have substantial differences. Neither of these SRM have been translated to definitive engineering guidelines. The industry has the necessary technology to develop such guidelines. Developing definitive SRM and the associated general requalification guidelines should be given a high priority.

The reliability, platform capacity, and fitness for purpose approaches summarized in this paper provide important background for development of SDM for new locations and platforms. These approaches also have important ramifications for development of efficient and effective SRM that address not only seismic hazards but as well other equally important environmental hazards and the generally more critical operating hazards. Generally accepted FFP criteria that are dependent on consequences and likelihoods of failure need to be developed for both new and existing offshore platforms.

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The API Requalification Project

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Introduction

Within the offshore petroleum industry a lot of the research has been underway on the topic of fitness of existing structural facilities. This paper will focus on structural and structural related issues pertaining to the three elements of life safety, environmental safety and economics. Aspects of the three elements pertaining to the operational aspects (production systems, well systems, transport, etc.) will not be addressed. The Subcommittee for Fixed Systems Criteria of the Offshore Standardization Committee within the Exploration and Production Department of the American Petroleum Institute (API) addresses structural issues. It is within this context the following overview is made.

The offshore oil industry is relatively young by building standards. The birth of the industry came with the first offshore structure, out sight of land, installed in the Gulf of Mexico in late 1947 (there were near shore structures connected to land by bridges as early as the 1910's in Southern California). Analogous to all technologies, evolution takes place in the knowledge and design of these facilities. Over time, changes in codes and practices tend to establish definable periods for structures based on performance of the structures. Structures can generally be classified as a member of "design" eras which indicates overall performance characteristics.

For offshore structures in U. S. waters, eras can be loosely classified as:

Seismic Design:

Early generation	Prior to mid 1970's
Transition	Mid 1970's to early 1980's
Present Day	Early 1980's to present

Storm/Wave Design:

Early (25 Yr. Storm)	Prior to mid 1960's
Transition	Mid 1960's to Mid 1970's
Present Day	Mid 1970s' to present

It can be seen that not only will there be differences in performance due to design criteria, but aging of the structures resulting in possible deterioration of capacity may also be occurring. In view of these two circumstances, many operators have been reviewing their platform inventories for both fitness and economic value. This has brought about an increase in assessment research and application.

There have been several catastrophic incidences, some with offshore facilities, which focused public attention on existing and aging offshore structures. The explosion and subsequent fire on Piper Alpha focused a lot of attention on the topsides hazards. While the Loma Prieta earthquake did not affect any offshore structures, it has sure peaked the interest in Southern California offshore structures. Most recently, Hurricane Andrew, which is a design loading event, came through the Gulf of Mexico and has provided a wealth of structural performance information.

In looking at facilities, fitness for purpose is the key criteria, but there are three critical factors that are usually included in the evaluation. One is life safety, one is environmental safety and, of course, there is the economics. When reviewing existing structures, there are a number of outstanding questions:

- What role should codes play?
- Where do codes stop?
- Do the codes get into the economics and dictate things based on economics? Do they not?
- How much of the fitness criteria, including life and environmental issues as well as economics, is the operator's purview?

There seems to be a generally held, though not unanimous, view that economics should generally be the operators (owners) decision. When you get to high consequence facilities, for example where a facility is providing a large percentage of a natural gas supply to a region, economics may also be more of a regulatory issue. A lot of debate exists as to where the lines are drawn and who should make the decisions.

In addressing many of the above questions, two main topics will be addressed. First, the recently API funded study entitled Seismic Safety Requalification of Offshore Platforms[1] will be reviewed. Second, an overview of the offshore industry activity in the area of assessment of structures for fitness will be provided.

API Funded Study

The late 70's and the early 80's saw quite a few changes in the design of offshore structures for seismically active areas. As the industry developed its knowledge and extended its capabilities in this design area, significant changes were incorporated into API RP 2A[2] (Recommended Practice for the Planning, Designing and Constructing Fixed Offshore Platforms). Prior to the 1970's, many platforms were designed using a variation of the principles found in the Uniform Building Code in that era. Research and advances in analytical capabilities ushered in new era of design during the late 1970's. Coupled with an increase in oil exploration offshore California during this period, the new generation of platforms were relatively robust. However there were a number of platforms installed prior to 1970 that were not of modern day vintage and the structural capacity relative to potential seismic events was unknown.

In the summer of 1991 the API funded a project, partially as an effect of the Loma Prieta earthquake and requests from the California State Lands Commission and the Minerals Managements Service to operators of older platforms, to address the issue of fitness of existing offshore platforms in seismic areas. The project involved an expert panel chaired by Dr. Bill Iwan of the California Institute of Technology (Caltech). Dr. George Housner, also of Caltech, and Drs. C. Allin Cornell and Chuck Thiel, private consultants and affiliated with Stanford University, served on this panel. The panel was given a three-fold charge from API:

- Prepare a written document that provides a rational basis for the seismic safety requalification of offshore platforms ;
- Focus specifically on issues related to performance objectives and requalification methodologies;
- Relate recommended performance objectives and requalification methodologies for offshore platforms to those for onshore structures such as buildings, lifelines and other facilities.

It should be noted that the panel was asked to focus strictly on the seismic structural performance issue and not take a look at the total risk picture of an offshore platform. There were not asked to take a look at issues relating to storm or wave loadings, accidental loadings such as collisions, or operational risks due to operation of the wells systems, production trains, etc. The panel did consider structural performance issues of appurtenances such as the adequacy of tiedowns for equipment, piping supports, etc. when the facility was subjected to a seismic event. These bounds need to be remembered in order to view the report in a proper context. The panel focused on one part of the risk pie, however, they formulated a framework and methodology that is quite applicable for extension to the other areas also.

The panel made note of four principles that guided their work:

- The focus of the seismic safety requalification should be on limiting to an acceptable level the risk due to catastrophic impacts of earthquakes. A catastrophic impact is one that has unacceptably large life and/or environmental safety consequences.
- The seismic life safety hazard posed by a requalified platform should be of the same order as that posed by well-designed onshore conventional building structures.
- The seismic environmental hazard posed by a requalified platform should be no greater than that posed by other major offshore petroleum release sources.
- Offshore facilities must have rigorous site hazard and engineering behavior analysis to achieve these goals, more rigorous than onshore facilities even though they have compa-

rable quantitative risk limits. This is due in large part to the generally lower level of knowledge of faults offshore than on land.

The report is publicly available through the API office in Dallas. It is not an API position. The report does not constitute an API position, rather it is a report to the API by four experts with their opinions on acceptable procedures methodologies that can be utilized for requalification of platforms in seismic areas.

As the panel deliberated, they addressed several major issues.

- Should requalification be probabilistic or code based?
- What are values, criteria, for acceptable performance?
- How are include cost benefit factors included in an analysis?
- Can risk be partitioned, i.e., can life safety be separated from environmental safety for requalification purposes?
- What is the adequacy of current design practice with an RP 2A[3] document, which was in the 19th edition at the time they performed the study?
- Can the risk of offshore structures be related to that of land-based structures?

The framework of the panel's report is provided above. The panel's recommendations will be summarized below followed by some of the details leading to the recommendations.

The panel developed their recommendations with minimum acceptable levels performance that are consistent with the public's safety expectations. They then provide comments on procedures by which these levels may be demonstrated. The performance levels are compatible with present day (1992) land-based practices for buildings and industrial facilities. The panel uncovered no safety or environmental issues associated with the seismic safety requalification process that indicate platforms should be subject to risk criteria more restrictive than those for land based industrial facilities. Thus, use of the seismic performance expectations developed for land based structures was deemed appropriate to meet the safety expectations of the public.

The recommendations are:

1. Seismic safety requalification performed under the guidelines recommended in this report should yield platforms whose seismic performance will be comparable to the balance of commercial and industrial structures onshore. The focus on performance should be on limiting the catastrophic impacts of earthquakes to an acceptable level.

2. Risk associated with life safety and environmental safety can be treated separately for the purpose of requalification.

3. Seismic life safety hazard posed by requalified platforms should be of the same order as that posed by well designed onshore conventional building structures.

4. The seismic environmental hazard posed by requalified platforms should be no greater than that posed by other major offshore petroleum release sources.

5. To meet these goals, offshore facilities should have more rigorous site hazard and engineering behavior analysis to achieve these goals due to generally lower level of knowledge of offshore fault knowledge, slip rates, etc..

6. Use of RP 2A 19th edition will produce a structure with life safety comparable to that of well engineered structures provided with the following qualifications, (see Fig. 1):

   - the hazard study and structural analysis/assessment should be to peer reviewed.
   - the seismic hazards should be determined in accordance with strength and ductility levels set at 200 and 1000 year return periods, respectively.
   - ductility level analysis should be explicitly made.
   - proper allowance need to be made for life safety risks associated with platform appurtenances as mentioned above.

7. The decisions made by owners based on cost and economic return are properly theirs as is the decision to design for a lower level of risk. Benefits may be significant and the owners are strongly encouraged to include economics in risk assessments.

The recommendations have been flow charted by the panel essentially using a modified API RP 2A 19th Edition seismic procedure as the starting point. The process addresses environmental issues, life safety issues and then a quality assurance program. Fig. 2 is a flow chart of the panels procedure for seismic requalification of offshore platforms. The process starts with the modified RP 2A analysis, if passing, life safety protection for appurtenance response is addressed. If passing, the platform is seismically requalified and a continuous quality assurance program is executed.

If the structure does not meet the modified RP 2A criteria, the environmental and the life safety issues are addressed explicitly for comparison to the threshold values. Figs. 3 and 4 are schematics of the life safety and environmental aspects flowcharts respectively that depict the process for the structures not meeting the modified RP 2A analysis criteria. Further explanation is contained in the panel's report.

Further explanation of the RP 2A seismic requirements is warranted. A probabilistic seismic hazard analysis for the platform site is necessary. It should be peer reviewed to assure that the study utilizes current day practices and the proper seismic sources and attenuation models have been addressed and used in an appropriate manner. There should also be an independent peer review of the seismic analysis evaluation, whether it be linear, nonlinear (static or dynamic), or whatever is chosen.

A prerequisite to the actual structural analysis is that prior to starting any of the analyses, a survey using the guidelines of the 19th edition of RP 2A needs to be performed. The survey is needed to determine what the existing conditions of the structure. Some of the questions the survey addresses are as follows:

Has it degraded?

Is it damaged?

Has it been repaired? If so, what is the effect?

What are the loading and functional use situations?

The next topic for further explanation pertains to life safety. There are three essential steps in determining whether or not the platform poses acceptable life safety risks.

1. Is the facility manned, thereby posing a risk to workers to the time the earthquake occurs?

2. Can the likelihood of platform collapse be limited if it is manned?

3. Can the likelihood of injury/death due to failure of appurtenances in a seismic event be limited?

The life safety flow chart is shown as Figure 3. It starts with the modified RP 2A 19th edition analysis. If failing, the structure is checked against a yearly probability of collapse 1×10^{-3}. If the probability is less, life safety criteria is met. For the appurtenances to meet the life safety criteria, they need to be checked against the 1988 UBC, or equivalent criteria, as a minimum. At the time the study was made, the API RP 2A 20th Edition[4] was in review and the recommendations for appurtenances had not been formally adopted by API (the seismic sections of the 20th Edition have subsequently been approved as proposed). The 20th Edition guidelines, were in the panel's opinion, equivalent or better than the UBC requirements, and would be appropriate.

Median value results from the hazard analysis should be used in a probabilistic analysis. Median results are also consistent with what is used in the basic UBC and land-based type analysis, the basis for the land-based comparisons.

Environmental performance addressed three sources of potential release. Hydrocarbon releases could come from wells, from pipelines or from onboard storage. The panel recommended a 2,000 barrel value, that was in their view, as an appropriate value that would guard against catastrophic impact for the southern California environment.

The 2,000 barrel threshold merits some discussion. The value is subject to debate, both public and industry. The important aspect is that it provides a framework to talk about some very well founded values for corollary purposes and address the ultimate goal, protecting the environment. If there is less than a 2,000 barrel potential release from a well, from a pipeline, or from the processing train, the environmental safety threshold is met. If the release potentials are greater, and the probability of a release greater than 2000 barrels if an event occurs, but the probability of release, based on structural failure or mitigation methods, is less than $1x10^{-3}$, the environmental safety threshold is also met. In the development of the threshold and procedure, the panel notes that API RP 14B[5], which is an API Recommended Practice for subsurface safety valves, should always be followed. It should be recalled that the panel's stated objective was to recommend values to safeguard the environment from a catastrophic impact due to hydrocarbon release. The 2,000 barrel value is not magic, but a value, that in their opinion, was not too large or too small from a regional perspective based on historical information.

The discussion for life safety and environmental thresholds have, to this point, centered primarily on U. S. West Coast activity. However, the framework with possibly modified thresholds, can be applied any place around the world. Many of the regional design and operating practices are similar. Regional characteristics, due to both governmental and societal values, to do get involved in establishing thresholds for acceptance criteria.

The foregoing addresses impacts due to structural consequences. It is not a total offshore risk pie picture. The procedures do not address or handle hazops or any of the other operational considerations that are inherently present when the total risk for an offshore facility is faced. By definition, the API Offshore Standardization Subcommittee for Fixed Systems Criteria deals with the structural aspects of a facility and only identifies the interfaces with the operations, drilling and production practices. Other standardization committees handle these issues.

There are efforts to try and bring more of the various disciplines together to do a total facility requalification or reassessment. In that effort, the whole operation needs to be addressed in an integrated manner for the results to be most meaningful. The Fixed Systems Subcommittee is focusing primarily on structural aspects as noted.

Fig. 5 illustrates typical land-based practices for requalification of structures in seismic areas. Though the Uniform Building Code (UBC) doesn't directly address requalification per se, this figure illustrates what is being done within jurisdictions who use the Uniform Building Code. Many agencies, including governmental agencies having jurisdiction of the governmental inventory of buildings often outside the jurisdiction of local building codes, do have requalification criteria. Many of the these codes or practices target a requalification for a standard facility at approximately 75% of the effective base shear of a new building designed in accordance with the current UBC design practice. A standard building would be defined as one not falling into the categories requiring higher performance standards, i.e., increased base shear factors. The panel made subjective correlations based on their years of experience and familiarity with design codes and practices which have provided notional, not quantitative, values.

Fig. 6 expands Fig. 5 with the addition of the panel's recommended performance levels. The panel concluded, when viewing from an established public policy precedence, that a requalified offshore platform would not necessarily have to meet the new build criteria (19^{th} Edition of RP 2A), which in their opinion, would substantially provide for better performance than a new build building would at the current UBC. The precedent on land recognized that many structures could not be upgraded within reasonable economic bounds.

Expanding more on the environmental issues, some information taken from Eskijian[6] and Ross et al[7] and discussed in some detail in the panel's report, indicates that there is approximately one order of magnitude difference in the potential release, size of the hydrocarbon release, from tankers and storage tanks than what there is from pipelines and platforms. This is illustrated in Fig 7. In addition to the oil release data, which has been plotted to a best fit straight line, a third curve, this time vertical to show the uncertainty, is superimposed indicating the daily natural seepage of oil into the Southern California waters of 50 to 200 barrels per day. The threshold of 2000 barrels from any one of the sources is then approximately equal to the quantity released every 4 days to 40 days naturally.

In establishing the threshold value, the panel looked at the American Trader incident in the Santa Barbara Channel with release of 9,500 barrels of crude. This spill impacted the environment, but not in a catastrophic manner. Total prevention of hydrocarbon release can't be met given the volume handled in Southern California waters from all sources, but catastrophic damage or impact can be managed. It is from the review of this type of information that the panel made their recommendation. Recall from above, the value of 2,000 is not magic. It represents a reasonable value, in the panel's opinion, of a realistic target. A value of 1,000 or 5,000 barrels could also be concluded from the data. Values of 1 barrel or 100,000 barrels would not be realistic.

This has provided a summary of panel's report which compared seismic safety structural performance to land-based along with some of the historical practices going on off the U.S. West Coast pertaining to tanker operations and pipeline transfer. The report is providing a very good set of information and a baseline for further work. Some of the data has already been used by operators in responding to government agencies in reviews of existing facilities.

Wind & Wave Loading - Gulf of Mexico Information

Broadening the view to address another source of structural loading will provide a view on how industry is using data in addressing the question of assessment of existing structures for fitness. Typically, an operator does perform inspections, evaluates his platform inventory, does planning, certain set of implementation and then back through the inspections as indicated in Fig. 8. This process is typically followed for platforms in seismic areas, non-seismic areas, for platforms in ice infested waters, etc. It's a very typical operation which has evolved over the years by experienced operators.

Experience and data generally provides the background for establishing criteria. Though the offshore industry is relatively young, having been established just 45 years ago, a number of structures have either failed or experienced design loading conditions from a several hurricanes.

Hurricane Andrew, passing through the Gulf on August 27 and 28, this past summer was the most recent event in this category. The exact number of structural failures have not been determined yet, but the data is proving invaluable for assessing design criteria and fitness criteria.

It is important to look at what the data has provided as a source of design practice evolution when coupled with developing technology. Major changes in the design criteria for Gulf of Mexico structures came after the Hurricanes Hilda and Betsy in 1964 and 1965 respectively. How information from these events affected wave design is shown in Fig. 9. Though the figure is a little bit cluttered, five key items are indicated:

- Design wave height for the early years of activity, 1950's -1960's;
- Design wave height for present day (RP 2A 20th Edition in press);
- Recommended deck height above mean water for the early years;
- Recommended deck height for present day
- Deck heights of some actual platforms designed in the 1950's-1960's.

It can be seen that for a number of the platforms based on earlier practice, the waves of a Hurricane Andrew, or Camille or Betsy, which were approximately equal to the design heights indicated, would have been inundated and the platforms expected to be lost if they were exposed to the maximum winds and waves. In fact, some of the platforms were lost in each of these storms. Though seismic design loading events have not taken place offshore California, a parallel development in criteria to some extent has occurred for the seismic design criteria within RP 2A in the late 70s and early 80s. There was a lot of exploration and development work being done off the coast of California from the mid 1970's to the mid 1980's. Criteria was upgraded. New advances were recognized and incorporated into the codes to the hazard studies and the analytical techniques. This established a second generation of platforms which either meet, or almost meet, today's criteria even though they were built 10 or 12 or 15 years ago. There is a another set that don't even come close to meeting today's criteria. These are some of the structures that will be the first for assessment and fitness evaluation.

Procedures and acceptance criteria have been addressed so far. Performance issues have real substance. As of September 11, 1992, Minerals Management Service statistics, provide an initial look into platform performance after exposure to Hurricane Andrew. The statistics below are based on visual inspections, no underwater inspections had been performed and reported as of this date.

3,852	Total platform inventory in federal offshore waters;
2,000	Platforms exposed to hurricane force winds or greater, i.e., 75 mph or greater;
	Note: not all platforms exposed to hurricane force winds experienced the maximum winds, and associated waves, sustained winds in excess of 125 mph which approxi mates current level design (100 year conditions).
800	Platforms in excess of 20 years old;
34	Platforms were toppled;
28	Platforms leaning;
104	Platforms had other damage;

5 Mobile drilling rigs set adrift;
2 Offshore fires.

On October 21, 1992, the New York Times reported, referencing a source from the MMS, 249 platforms were damaged, leaning or lost. Platforms are defined as any freestanding structure fixed to the seabed. Individual caissons and multi-leg steel templates are each considered a platform for the purposes of this database.

To the person acquainted with the offshore industry and the fleet of platforms, this information indicates:

1. From a design point of view, there may have been 100 - 200 platforms that actually saw 100 year design criteria. Other platforms were exposed to significant loading, though not necessarily design level. Overall performance was excellent.

2. With Hurricane Andrew, there was no loss of life. All facilities had been evacuated well in advance of the passage of the storm and the facilities secured for storm conditions. Mitigation measures such as demanning in advance of a storm are workable mitigation measures.

3. There was, in many people's opinions, no catastrophic environmental impact from structural collapse. Only a small percentage of hydrocarbon release was attributed to platform failure. The vast majority of hydrocarbon releases was from pipeline damage, not platform structural induced failures. Many wells had been manually shut-in as part of the standard storm preparation sequence. Others that were still producing and controlled by remote systems, had fail-safe subsurface safety valves in operation. There were no reports of free flowing wells following the storm. Again, mitigation procedures worked.

4. A number of platforms were lost recognizing that a number of platforms out in the Gulf of Mexico would not come close to meeting today's design criteria. There was major infrastructure dollar impact. Estimates are running anywhere from $500,000,000 to $2,000,000,000.

5. Modern designs, from the mid 1970's to today, faired excellent, especially the common tubular steel space frame jacket and deck platforms. There was substantial damage in shallow water areas as expected. Many of these installed in the 1950s or 1960s to substantially lower criteria and the waves entered or overtopped the decks.

What this lead to? One view is that the life safety and environmental safety issues are being well managed by the industry and the regulatory bodies for large storms. Questions remain about the entire risk picture concerning existing facilities that might be subject to storms that do not have the advance warning that a storm the size of Hurricane Andrew provided. Regardless of which view one holds, additional work will prove beneficial.

There appear to be two major topics concerning policy that should be further explored. First, it needs to be noted that life safety issues and environmental safety issues are one component of public policy issues. Secondly, economics, and this is still subject to a lot of debate, is primarily the custody of the owner/operator. Part of the debate centers on what part should public policy play in addressing the economics of design, retro-fit or fitness for purpose. There are both direct and indirect interactions between the policy issues and the economic issues. If the public policy becomes too restrictive, the industry dies due to costs. If costs alone are driving forces, some safety issues may be compromised depending on the economic or consequence models an owner may choose. A common view is that once public policy issues are satisfied, the economic benefits/consequences of performance criteria should be the owner/operator's decisions. This is the most common economic and public policy balance within American society.

What are some of the next steps in the assessment process within the industry and API? The seismic requalification study is just one of a long series of steps underway. Other topics and activities are:

- Reviewing case studies for trends. Hurricane Andrew has given us a lot of case studies. These supplement information from earlier hurricanes in the U.S. and other studies around the world.

- Additional studies looking at the seismic arena will continue.

- Questions to be answered include:

  - Can platforms be categorized as to their expected response based on type or age?
  - Can somewhat measures such as the evaluation of base shear versus Reserve Strength Ratios (RSRs) or beta ratios, etc., be established?
  - How valuable is multi-level screening criteria?
  - Can an effective screening procedure be developed for 80 to 95% of the Gulf of Mexico type facilities to avoid performing a detailed case-by-case assessment of every structure? There's not enough facilities in the seismic regions of the world to categorize and, hence it becomes a case-by-case for these facilities.
  - How do you incorporate the method of independent review or peer review? Do you do a modified in the U.S., a modified CVA program for that?
  - Outside of the U. S., some of the regions have certification, some verification. How can procedures be formulated to address these issues?
  - In a somewhat fragmented industry (some owners have a single facilities whereas others have hundreds), can a cost effective, practical method also incorporate a peer review that is of value?
  - Is peer review needed for all assessment cases? or just a subset? Within the U. S., can peer review be effectively incorporated into the existing Certified Verification Agent (CVA) program?
  - In establishing acceptability criteria, what falls under the purview of recommended practices within the industry? What falls under the purview of codes or regulatory requirements? What falls under the purview of economic decisions by the operators?

Take a couple of the issues above to further elaborate on the issues. Acceptance criteria for individual, high consequence facilities, imposes additional policy issues. A caisson with a single producing well that's producing out 2,000,000 cubic feet of gas daily through a flowline with no processing or living facilities, is significantly different in function and consequence, from a public impact and policy view, when compared to a platform supplying the vast majority of the natural gas to a whole region in Australia. The caisson is also nowhere near the same consequence, from both life safety and economics, as a Piper Alpha. In partitioning responsibility or ownership of acceptability limits, the economic issues become the most difficult to address. Low consequence facilities, both from an economic investment and impact due to disruption of supply, belong rightly to the owner/operator. For the high consequence facilities, it becomes a partnership linking product prices with risk and costs.

These two examples are the easy ones. Where, in the consequence realm, does economics move from the owner/operator to a partnership? Many believe the economic issues belong solely to the operators while life safety and environmental safety are regulatory and public policy issues. Typically, this partitioning goes on within the building industry now. Is there a need to deviate from usual practice for the offshore industry?

Extending beyond individual facilities involves addressing fleets. Fleets can be looked at from a couple of different views. One is from an operators view, where the facilities comprising the fleet are spread over some region or series of regions. There is also the regional view comprised of facilities from a number of operators, but all resident within a given geographical region. There's a fleet of structures offshore California, extending from Point Conception down to below Long Beach. This comprises a regional fleet with no one operator owning all or even a majority. Not all of the facilities will be exposed to a given seismic event at the same time. In the circumstance of a design level seismic event (of which level most earthquakes are far below), only a few of the exposed facilities would be exposed to forces of design magnitude. Therefore, actual risk is to a subset of the fleet due to spatial spread of the fleet and distributed to a series of owners/operators. Development of acceptability criteria needs to include these characteristics.

Current API & Industry Activity

The broadest effort underway is an API task group looking at the whole issue of assessment and fitness for purpose of existing facilities. The work is covering waves, ice and seismic. It's a very concentrated effort over the next 15 months to develop draft guidelines prior to December, 1993. There is also a large number of individual company assessments going on, both here in the U.S. and overseas. There has been some significant work on structures in Australia as well as work addressing facilities off the coast of Africa and in the North Sea. Hurricane Andrew has given the industry a wealth of data. Effort is directed at making heads and tails out of the information.

A number of university projects are ongoing around the world dealing in these initiatives. Many of the projects are sponsored by oil companies and regulatory bodies. These studies are addressing, among other topics also, questions such as:

- What are valid methods for inspection and evaluation?
- What inspection methods give us useful information?
- What's the cost effectiveness to the various methods?
- How does the inspection information play into the analysis and the eventual determination of fitness?

Workshops like this one help in the exchange of ideas and interaction among all facets. The industry is interested in getting feedback, obtaining views from concerned parties and working with the public and various governmental agencies in continuing the safe and economic operation offshore.

On a broader scale, the LRFD version of RP 2A[8] is now forming the basis of the new ISO document for offshore structures. It is anticipated that a framework for assessment and determination of fitness guidelines will eventually go straight into an ISO type document for worldwide application. Worldwide answers will not be available for the first drafts. The initial effort is to develop a framework, not necessarily just a procedure that can be followed black and white, due to the complexity of the issue.

This effort might then lead to discrimination in design criteria based on consequences. If consequence is a factor for existing facilities, the natural question of should variation in design criteria be adopted? As an example, if there are no life safety concerns, negligible environmental safety impacts for a given facility, should the design be at a different level than if you have significant environmental or life safety concerns? The API Task Group will be collating research and project information on this question as the work for existing structures is underway since they are parallel from a consequence view.

There are several university and joint industry initiatives that will be utilized, even though the majority are being funded by sources other than API. Not all of them are being funded by operators. Some of them are funded through government agencies, some of them are university grant supported, some are internally funded from various companies and agencies that are consultants and some are combinations. The API initiative is trying to take and use all available information. There is no one obvious answer.

Summary

In summary, the requalification project that was funded and completed past summer has provided one framework for one piece of the pie. Many of us in industry think this framework is an excellent approach. By no means is it conclusive. By no means is it exhaustive. There is a lot of other work that has to be pursued, but it has provided a very good foundation to continue building upon. It also provides an independent assessment of seismic safety relative to what the public right now accepts with land-based buildings, with bridges, with other types of petrochemical complexes along with environmental impact of hydrocarbon releases in Southern California waters. We in the industry are striving forward. We have a ways to go, we need a lot more help and a lot more discussion.

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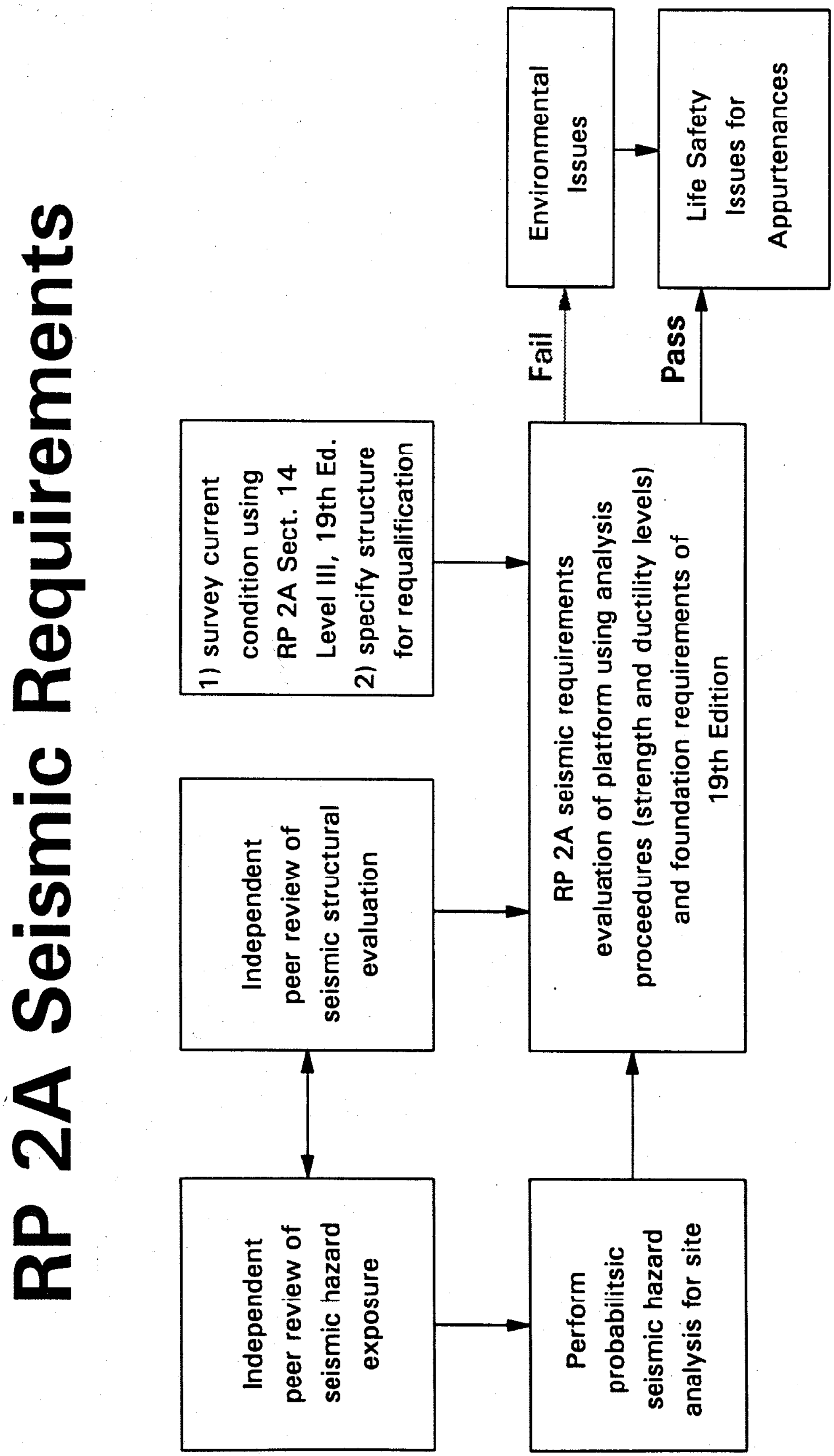

RP 2A Seismic Requirements
Independent peer review of seismic hazard exposure
Independent peer review of seismic structural evaluation
1) survey current condition using RP 2A Sect. 14 Level III, 19th Ed. 2) specify structure for requalification
Perform probabilitsic seismic hazard analysis for site
RP 2A seismic requirements evaluation of platform using analysis proceedures (strength and ductility levels) and foundation requirements of 19th Edition
Fail
Pass
Environmental Issues
Life Safety Issues for Appurtenances

FIGURE 1

Seismic Requalification of Offshore Platforms

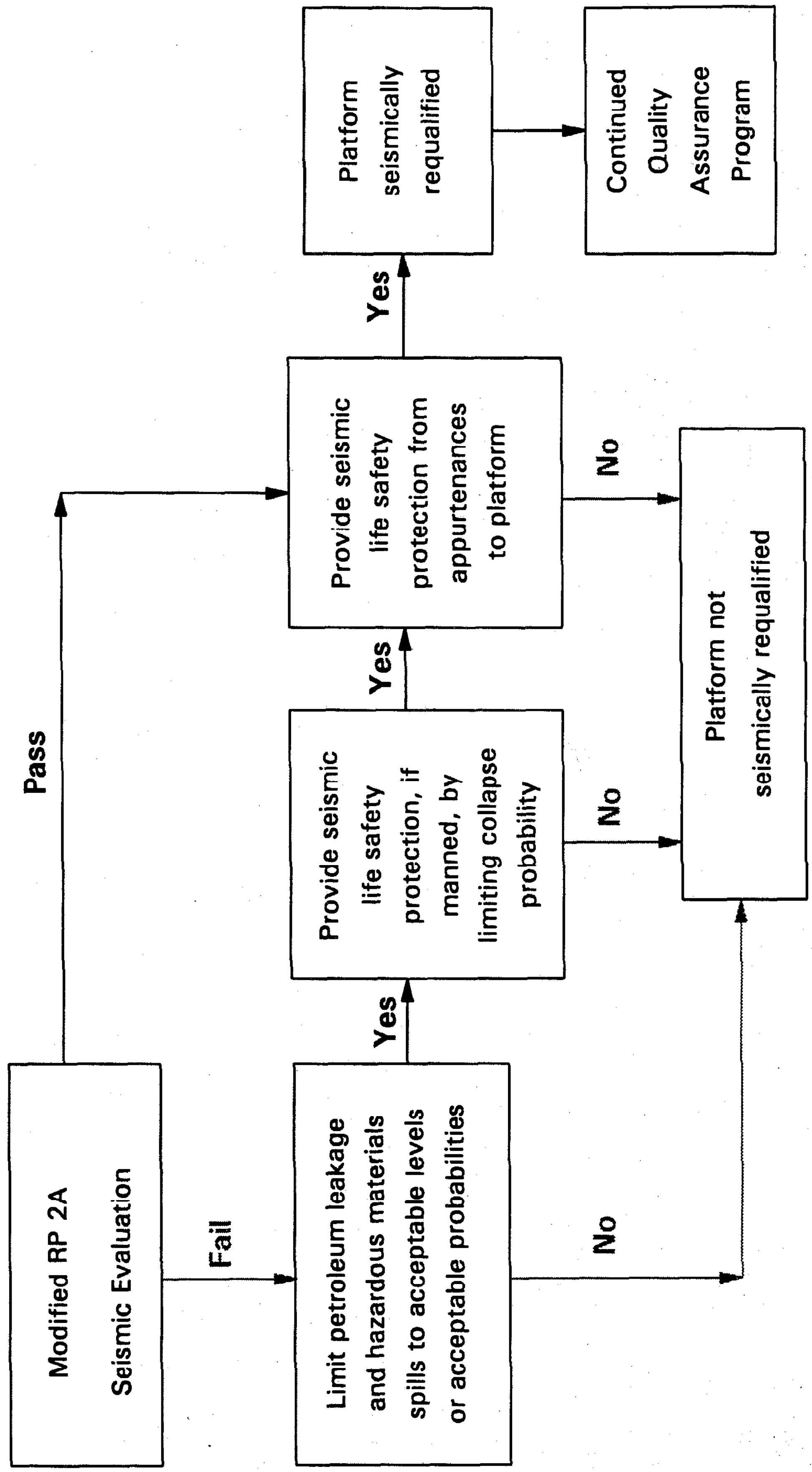


FIGURE 2

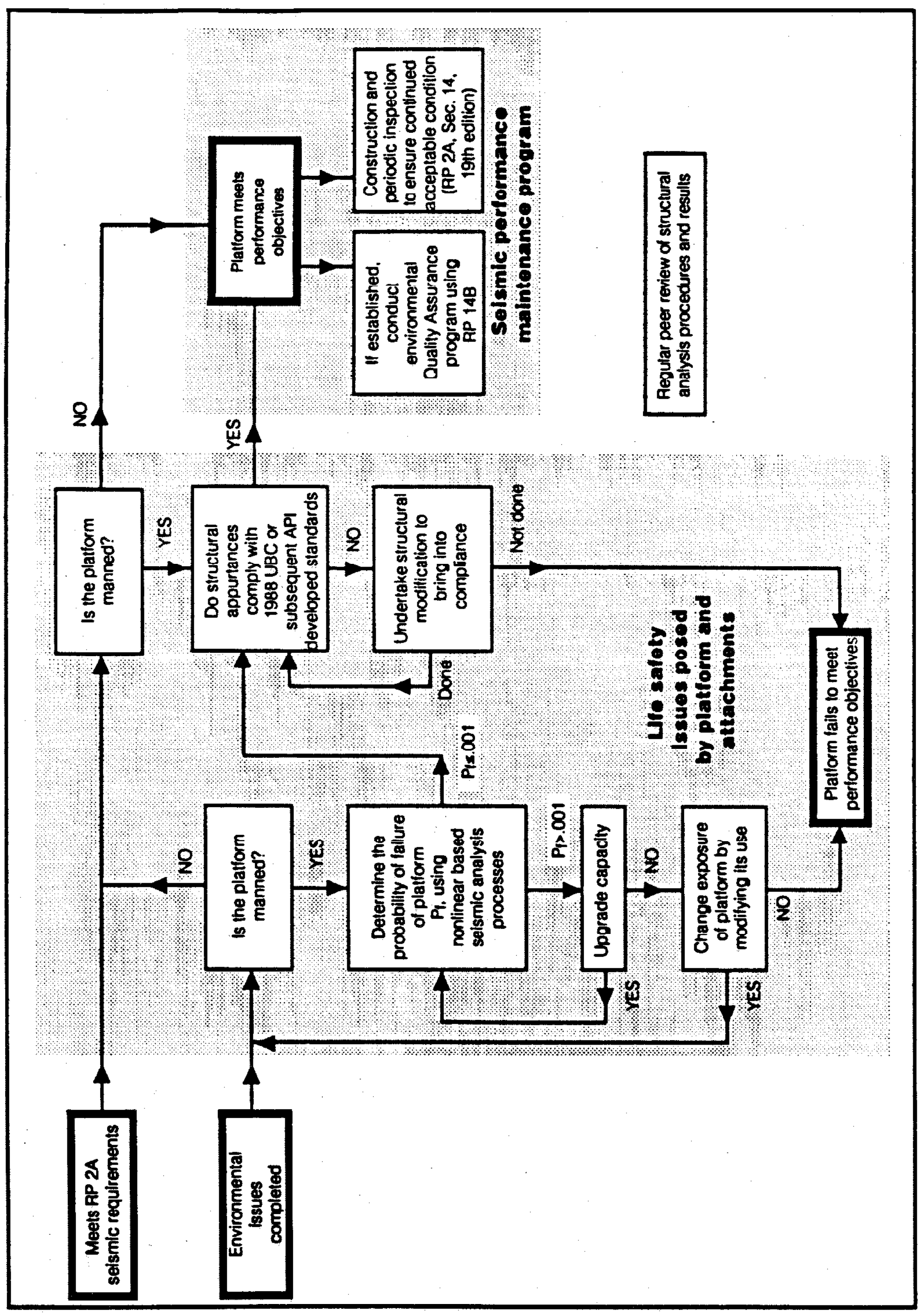

Meets RP 2A seismic requirements
Environmental issues completed
Is the platform manned?
NO
YES
Determine the probability of failure of platform Pf, using nonlinear based seismic analysis processes
Pf≤.001
Pf>.001
Upgrade capacity
Change exposure of platform by modifying its use
Is the platform manned?
Do structural appurtances comply with 1988 UBC or subsequent API developed standards
Undertake structural modification to bring into compliance
Done
Not done
Life safety issues posed by platform and attachments
Platform fails to meet performance objectives
Platform meets performance objectives
If established, conduct environmental Quality Assurance program using RP 14B
Construction and periodic inspection to ensure continued acceptable condition (RP 2A, Sec. 14, 19th edition)
Seismic performance maintenance program
Regular peer review of structural analysis procedures and results

FIGURE 3

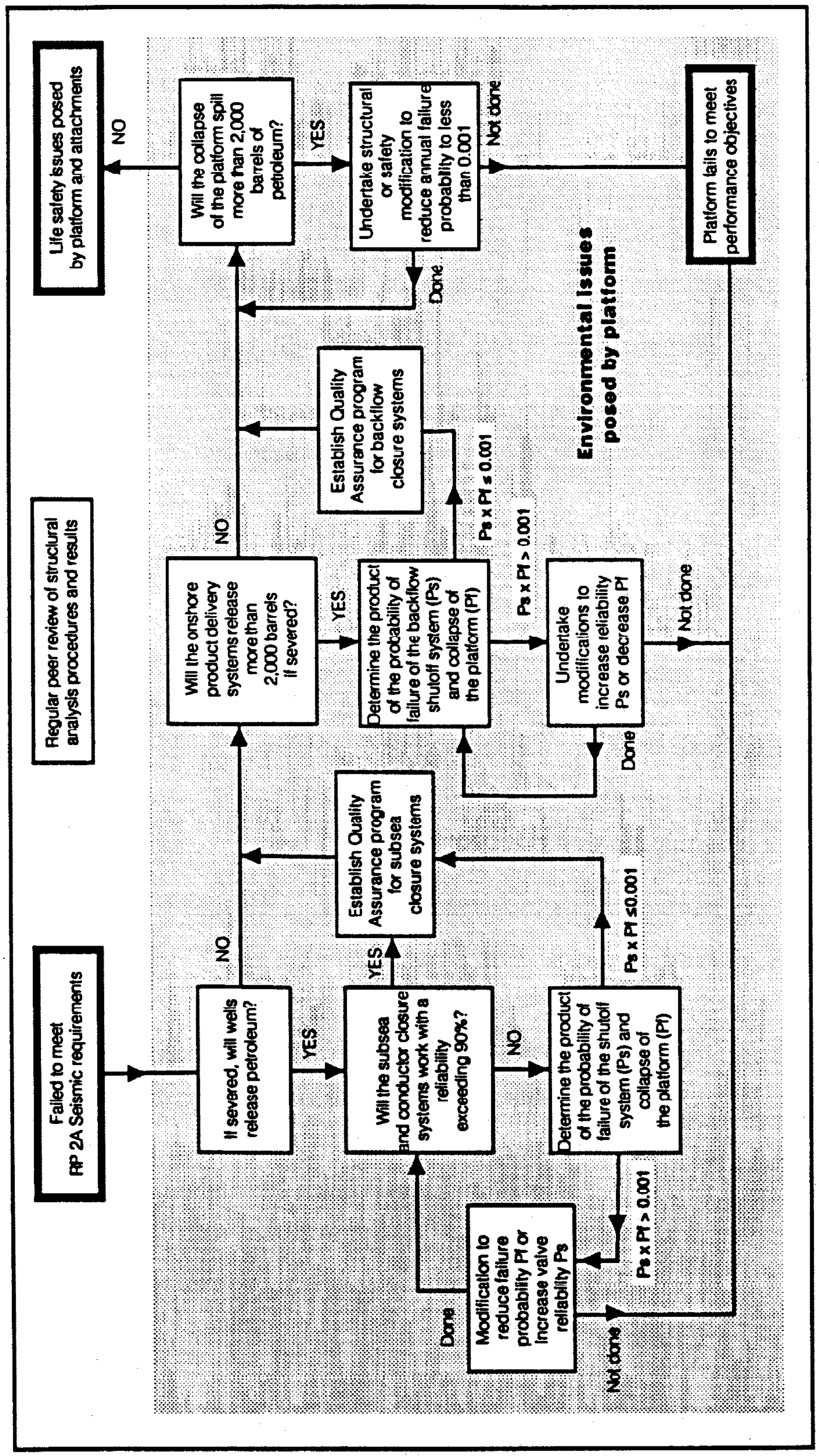

Failed to meet RP 2A Seismic requirements
Regular peer review of structural analysis procedures and results
Life safety issues posed by platform and attachments
If severed, will wells release petroleum?
YES
NO
Will the subsea and conductor closure systems work with a reliability exceeding 90%?
YES
NO
Establish Quality Assurance program for subsea closure systems
Determine the product of the probability of failure of the shutoff system (Ps) and collapse of the platform (Pf)
Ps x Pf ≤0.001
Ps x Pf > 0.001
Modification to reduce failure probability Pf or increase valve reliability Ps
Done
Not done
Will the onshore product delivery systems release more than 2,000 barrels if severed?
YES
NO
Determine the product of the probability of failure of the backflow shutoff system (Ps) and collapse of the platform (Pf)
Ps x Pf ≤ 0.001
Ps x Pf > 0.001
Establish Quality Assurance program for backflow closure systems
Undertake modifications to increase reliability Ps or decrease Pf
Done
Not done
Will the collapse of the platform spill more than 2,000 barrels of petroleum?
NO
YES
Undertake structural or safety modification to reduce annual failure probability to less than 0.001
Done
Not done
Environmental issues posed by platform
Platform fails to meet performance objectives

FIGURE 4

Structural Performance

Typical for
Building Codes

New Build = 100%

Requalification = 75%

Base Shear

FIGURE 5

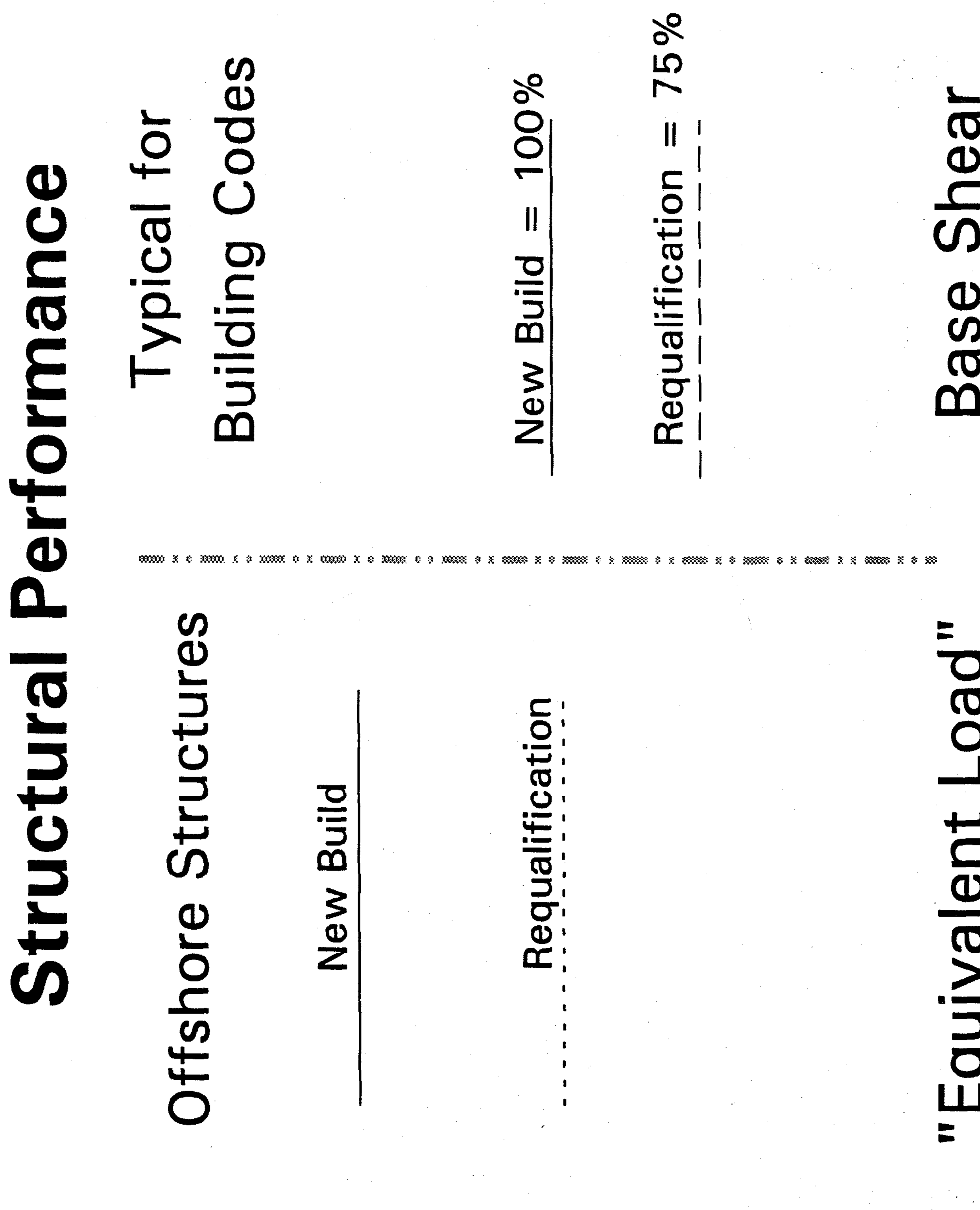

Structural Performance
Offshore Structures
New Build
Requalification
Typical for Building Codes
New Build = 100%
Requalification = 75%
"Equivalent Load"
Base Shear

FIGURE 6

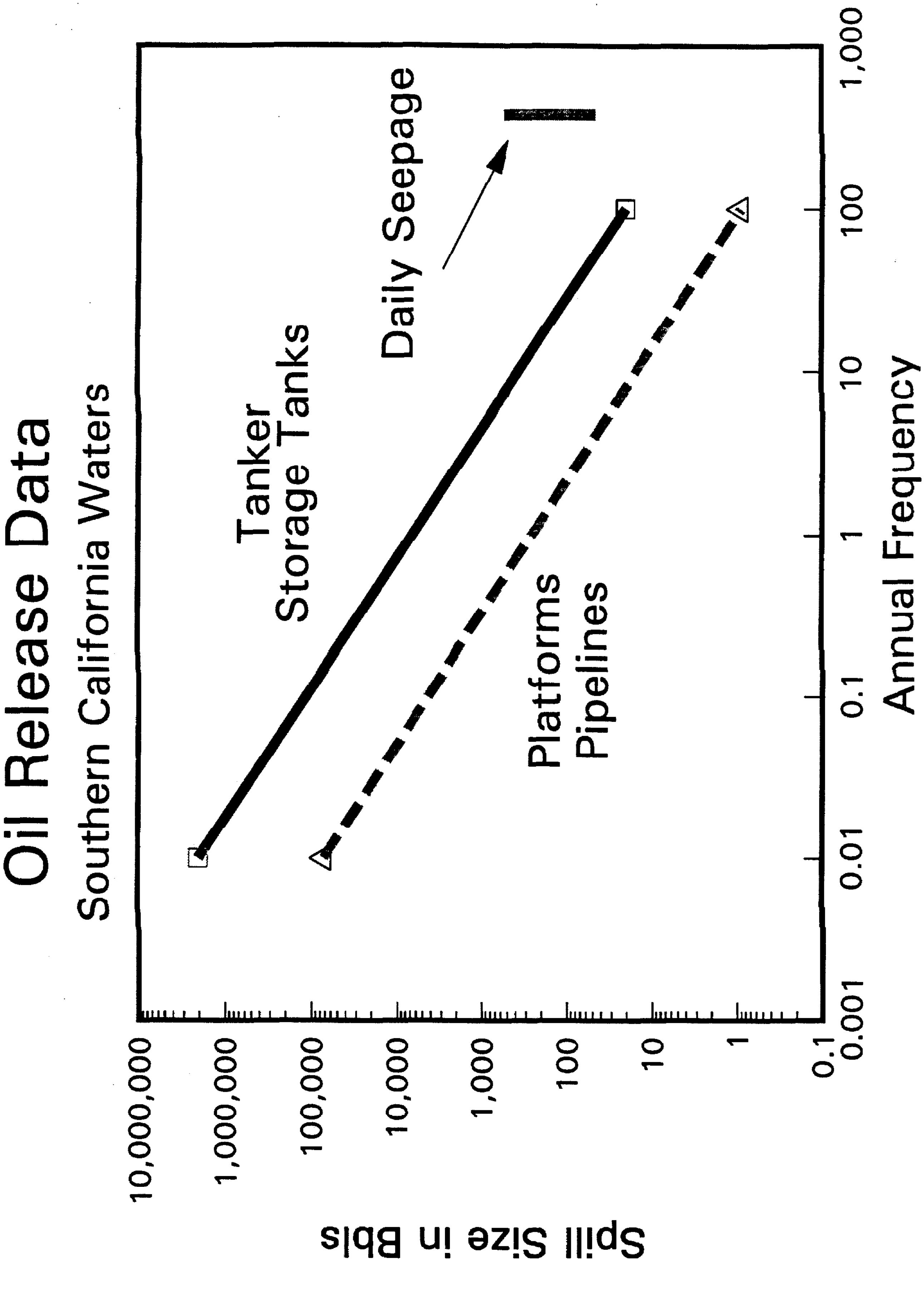

Oil Release Data
Southern California Waters
Tanker
Storage Tanks
Daily Seepage
Platforms
Pipelines
Spill Size in Bbls
10,000,000
1,000,000
100,000
10,000
1,000
100
10
1
0.1
0.001
0.01
0.1
1
10
100
1,000
Annual Frequency

FIGURE 7

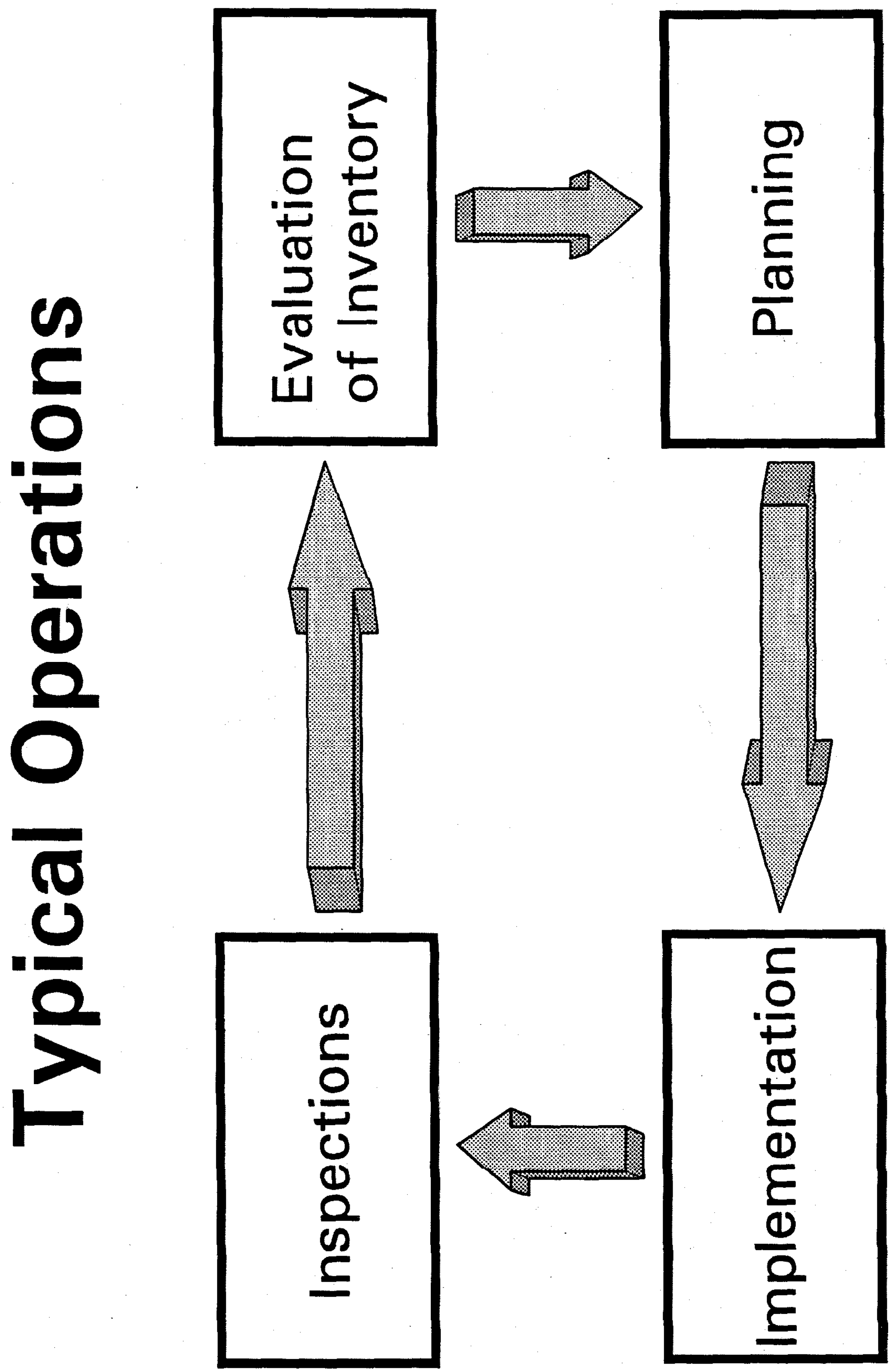
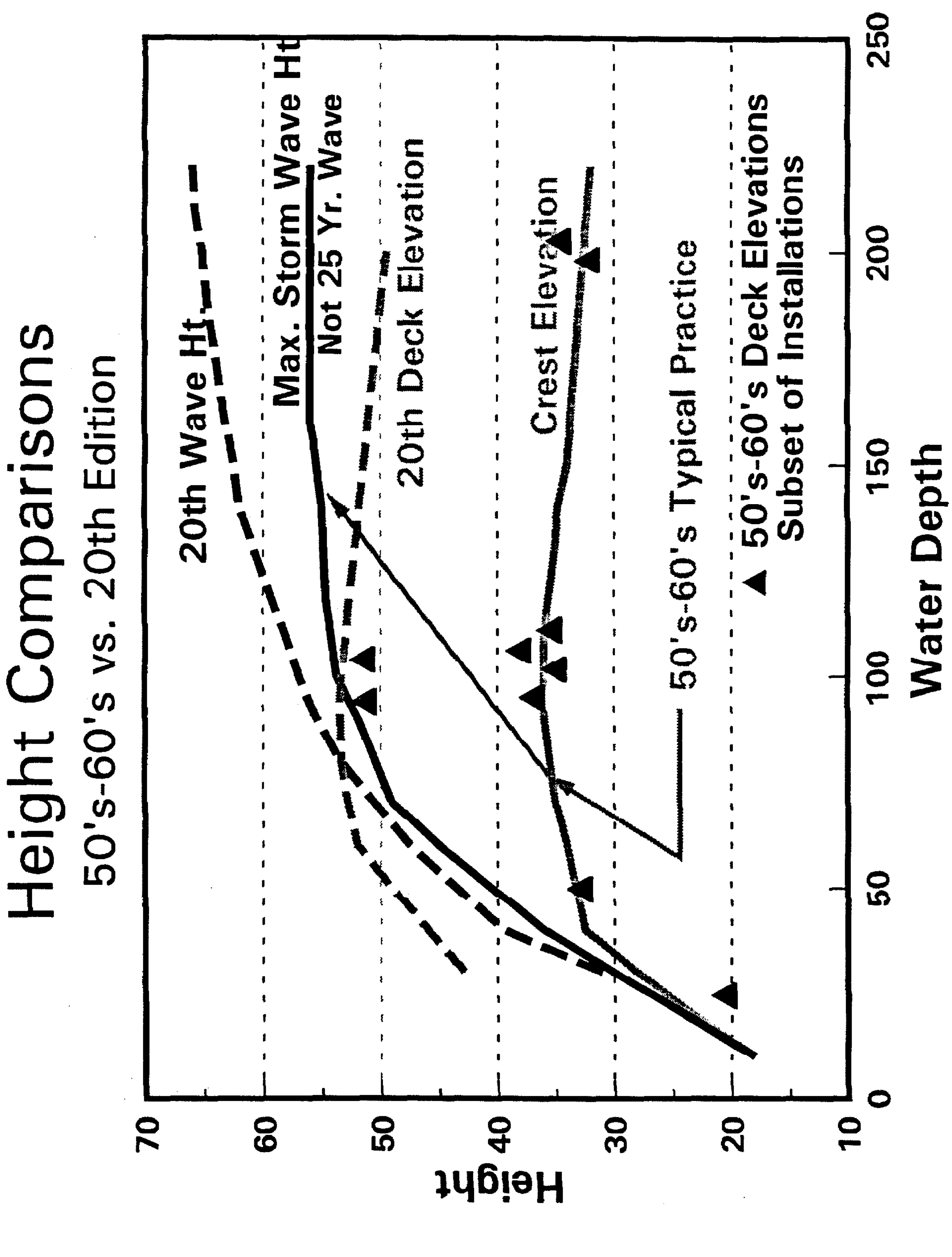

Typical Operations
Evaluation of Inventory
Planning
Implementation
Inspections

FIGURE 8



Height Comparisons
50's-60's vs. 20th Edition
Height
Water Depth
70
60
50
40
30
20
10
0
50
100
150
200
250
20th Wave Ht.
Max. Storm Wave Ht
Not 25 Yr. Wave
20th Deck Elevation
Crest Elevation
50's-60's Typical Practice
50's-60's Deck Elevations
Subset of Installations

FIGURE 9

Case Studies On Seismic Reassessment Analysis

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San Francisco, California

ABSTRACT

The development of reassessment methodology has concentrated primarily on platforms in nonseismic regions. Reassessment has become an important issue for many companies operating in seismically active areas and has initiated the development of new methodologies appropriate for these cases. Analytical methodology, software and assessment techniques have been developed and refined through the course of several site-specific studies that have been performed by PMB over the past ten years. This paper presents by way of illustration a comparison of various assessment procedures.

INTRODUCTION

Improvements in the technologies associated with the drilling and production of offshore oil and gas have provided access to reserves in existing reservoirs which were once unavailable. Many operators are finding that actual field lives are significantly longer than originally estimated, owing partly to this new technology and partly to conservatism in the initial estimates. This has increased the value of many existing platforms which may be close to or beyond their original design service life. The utility and worth of these platforms are increased further as the costs for exploration and new production increase and permitting of platforms, tankers, pipelines and on-shore facilities often becomes prohibitive. Consequently, operators and regulatory agencies have had to deal with the problems associated with ensuring the safe operation of platforms well beyond the original design service life.

Life extension continues to gain in importance as more platforms, in all types of environmental and operational conditions, reach their respective service lives. This is illustrated in *Figure 1* which provides a histogram of platforms distributed by age for all platforms located in U.S. state and federal waters. This data indicates that the rate at which platforms will be reaching their respective services lives will be increasing in future years.

COMPARISON OF EARTHQUAKE AND HURRICANE/STORM REASSESSMENT

The dominant source of extreme loading for platforms located in seismically active areas (such as along the Pacific rim) will often be close proximity, large magnitude, earthquakes. The execution of reassessment analyses in these instances is difficult because of the nature of earthquake loading and the resulting structural response. The determination of failure probabilities, as part of a risk analysis, requires an assessment procedure which is more complicated than that commonly used for platforms dominated by storm or hurricane loadings.

Scope

Industry sponsored projects such as AIM *[1]* have developed the process for platform reassessment but have focused on platforms in the Gulf of Mexico and North Sea where the primary environmental threat is from storms and hurricanes. This focus is the consequence of the distribution of risk toward the Gulf of Mexico, where most of the U.S. platform fleet is located.

A comparison of the number of platforms in hurricane dominant (i.e., Gulf of Mexico) and earthquake dominant (i.e., California and Alaska) regions is also provided in *Figure 1,* which shows over 4000 platforms in the Gulf of Mexico compared with 50 offshore Southern California and in the Upper Cook Inlet of Alaska (i.e., almost 2 orders of magnitude difference). The scope of the problem (i.e., the number of platforms that need to be assessed) is therefore far greater in the Gulf than off the West Coast.

Exposure

In many instances, reassessment philosophy in nonseismic areas is based on the premise that some warning occurs prior to the event which allows for shutdown of operations and evacuation. In these instances, the potential for loss of life and pollution is minimized and failure consequence becomes primarily economic. Earthquakes occur without warning and achieve peak loading very quickly after the initial ground motions, making effective shutdown and evacuation difficult or impossible. A reassessment philosophy for these conditions must therefore reflect the additional failure consequence of loss of life and pollution.

Response

Since most platforms are located in regions where the potential for earthquakes is small or negligible, reassessment analyses have concentrated on evaluating structural response under wave loading conditions. Most of the older platforms are in shallow to medium water depths and exhibit limited dynamic response; therefore, analysis procedures are based on a comparison of ultimate static strength versus a reference level design load. The ratio of ultimate strength to design level loading is typically referred to as the Reserve Strength Ratio.

The ultimate static strength and resulting RSR can be determined through various capacity analysis procedures, most of which utilize a constant load pattern that is increased uniformly until ultimate strength is obtained. An application of this procedure to evaluate the performance of a platform subjected to extreme earthquake loading can result in potentially misleading conclusions.

Failure Database

The following table provides a summary of the number of recorded platform failures resulting from Hurricanes in the Gulf of Mexico since 1948.

PLATFORM FAILURE DATABASE		
Hurricane	**Year**	**Number of Failures**
Grande Island	1948	2
Carla	1961	3
Hilda	1964	14
Betsy	1965	8
Camille	1969	3
Carmen	1974	2
Fredric	1979	3
Juan	1985	3
Andrew	1992	~20

There are several instances of storm and hurricane-related failures throughout the history of offshore oilfield development; however, there has never been a reported failure of a platform due to earthquake loadings. The fact that there have been hurricane related failures is obviously due to the much larger number of platforms in the Gulf of Mexico and the higher frequency of near design level hurricane conditions at platform locations. The experience gained through hurricane related failures provides an important benefit. Improved design and assessment procedures are largely effected by the information generated from failures which is not obtainable by other means (i.e., testing, analysis, etc.). There is significant experience relative to failures of land based structures; however, assessment of offshore platforms remains essentially dependent on the ability to understand the nature of probable future earthquakes and to assess structural response under these conditions.

Conclusion

The Gulf of Mexico and West Coast reassessment problems are therefore dissimilar in a number of ways: total problem scope, the predictability or forecasting of the threat (and resulting differences in life safety and environmental hazard), and the nature of loading and response. The need for developing simplified procedures to assess the numerous platforms in the Gulf of Mexico should not, therefore, impact the procedures developed for earthquake dominant regions, where analytical methods will be relatively more complex due to the nature of the loading and response.

THE SEISMIC REASSESSMENT PROBLEM

There are four primary issues which impact the use of existing platforms in earthquake dominant regions;

- Use of structure beyond original design service life
- Deterioration and damage of structure with age
- Increased earthquake demand due to improved seismological data
- No ductility requirement or recognition during original design

The first two of these problems are generic to both the seismic and storm reassessment problems and thus have been discussed in many prior technical papers.

The criteria used for seismic design of offshore structures have changed significantly over the last 30 years. These changes have resulted from:

- Improved understanding of regional seismicity, site effects and corresponding surface ground motions
- More efficient structural framing arrangements and detailing which provide greater ductility, thereby improving structural response under extreme earthquake loading.

Updated Definition of Earthquake Demand

In the 1960's very little was known about seismicity offshore California. Subsequent large earthquakes (e.g., 1979 San Fernando earthquake M=6.8) have caused substantial damage to buildings, freeways and bridges and have prompted investigators to increase study into the regional seismic environment. Investigation has identified new faults, improved the definition of potential earthquake magnitudes generated by known faults, and refined the definition of site ground motion intensity resulting from postulated earthquakes. As a result, the current definition of site seismicity is generally more severe than that established previously.

The issue of increased earthquake demand is analogous to that of the increase in wave loading resulting from the improved definition of extreme hurricane conditions in the Gulf of Mexico. In the case of the Gulf, the substantial increase in real storm data leads us to define much greater "design level" (i.e., 100-year return period) wave heights than would have been defined 20 years ago. Many of the older Gulf of Mexico platforms are at risk because the definition of the crest elevation in the original design wave resulted in a deck placement which will likely be below the crest of a "true" extreme wave.

Similarly, the increased availability of earthquake records, fault identification, site response information and geophysical data has improved seismic hazard analysis. This new information has resulted in an increase in the level of ground motions defined for new platform design and reassessment of existing platforms. Significant increases have occurred in cases where platforms are sited near faults which were previously undefined or have been reevaluated with respect to fault rupture potential and frequency. This is of particular concern for platforms which were originally designed based on the Uniform Building Code without site-specific seismic criteria.

One example of the degree of change is shown in *Figure 2* which provides a comparison of the newly defined and original site hazard spectra. In this particular case, the acceleration amplitude near the primary mode of the structure increased by 100% over that used for the original design. If the platform was not designed and detailed for ductile response, it is unlikely that it could sustain a 100% increase in load without other mitigating factors such as reduced payload.

Current seismological technology now allows us to define vertical ground motions directly based on vertical ground motion records and attenuation relationships specific to vertical motions [2]. This is a significant improvement over the earlier earthquake design methods used for offshore platforms in which the vertical motions were established as a direct percentage of the peak horizontal motions. The new process provides a vertical spectrum which has a much different shape from that of the horizontal spectrum. In addition, the motion amplitudes in the high frequency range may be greater than 50% of the horizontal. This introduces another potential problem for platforms with vertical modes in this higher frequency range. This is further compounded by the fact that, for many of these older structures, deck structures many not have been designed for earthquake loading. Many of these decks include cantilevers which can be very sensitive to the platforms vertical motions and therefore are affected seriously by the increase in high frequency ground motion spectrum.

Platforms Not Designed for Ductility

API RP 2A is generally recognized as the standard for seismic design of fixed-base platforms. Many revisions have been made to these recommendations regarding seismic criteria, design and analysis methods since the first issue. California platforms installed prior to RP 2A were typically designed based on the Uniform Building Code or other simplistic criteria which, by today's standards, would be considered inadequate.

Most of the substantial development in API RP2A with respect to earthquake design and ductility came in the 1976 through 1979 editions. The current recommendations were first issued in 1984. A histogram showing the distribution of west coast platforms by age is provided in *Figure 3* which shows that many of the platforms were designed without specific ductility requirements. Some of these platforms perform better than expected due to sound design and detailing; however, one would typically expect poor performance from these older platforms if one were to apply a conventional "design" design-based assessment.

COMPARISON OF SEISMIC DESIGN AND REASSESSMENT PROCESSES

Design

Present seismic design practice for offshore platforms focuses on two primary objectives:

1. To provide sufficient strength and stiffness to ensure no significant structural damagc for earthquakes with a reasonable likelihood of occurrence during the life of the structure

2. To provide reserve strength and/or ductility to prevent collapse during rare intense earthquakes.

These requirements are typically designated as the Strength Level Earthquake (SLE) and Ductility Level Earthquake (DLE) conditions, respectively.

Owners and regulators generally set criteria for the SLE requirement to balance the initial cost of fabrication against the losses associated with earthquake damage to achieve an economic optimum. Such losses could include interruption of cash flow, repair costs and increased maintenance costs. The DLE requirement provides safety against structural collapse and the associated catastrophic consequences to human life, the environment and capital investment.

Reassessment

The seismic reassessment process involves many of the same procedures required in the design of new platforms. However, due to important differences regarding economically feasible options for new design versus reassessment of an existing platform, each process requires a different philosophy. Existing platforms may be evaluated for SLE conditions; however, if a platform does not conform to these requirements, it is unlikely to be cost effective to institute strengthening or other remedial measures to achieve this level of performance. Reassessment methodology therefore concentrates on failure prevention and requires procedures similar to that of DLE analyses.

The reassessment of existing platforms is a relatively new process that has not been standardized in the U.S. in the way that design has been. This lack of standardization creates some difficulty in establishing performance requirements, which must be developed on a case by case basis depending on the risks (i.e., hazards, exposures and consequences) associated with the future operations of the platform.

Platforms are often assessed initially based on current criteria for new design. The reassessment problem becomes trivial if the existing platform is shown to meet this requirement. This is usually not the case; however, in some instances the earthquake loading may be less severe than that associated with the original "as-built" conditions because of reduced payload, for example.

ANALYTICAL PROCEDURES

A new design will typically use linear response spectra analyses (RSA) to define reactions under SLE conditions and inelastic time domain or pushover analyses to evaluate adequate DLE performance. Similar techniques can be used for reassessment analyses; however, the difference in performance requirements must be recognized when developing methodology. Given the premise that the platform cannot be proven to conform to current design criteria, there are two general methodologies for reassessment analysis: static pushover and time domain.

Analysis Objectives

The objective of reassessment analysis is to provide a reliable definition of expected performance for anticipated operating conditions to support decisions relative to future use of the facility. This objective can be pursued through various analytical investigations:

- Confirmation of platform stability for minimum performance requirement (e.g., 1000 year return period earthquake)
- Definition of maximum survivability event (i.e., in support of a failure probability calculation)

Either, or both, of these paths can be taken, depending upon the philosophy of the operator.

Additional analytical objectives include:

- Comparison of possible failure modes
- Evaluation of critical secondary response (i.e., risers, drilling rigs)
- Evaluation of the impact of existing damage
- Evaluation of effectiveness of remedial measures

Modeling Issues

The modeling techniques used for pushover and time domain analysis have some basic similarities. Each should utilize a fully coupled (i.e., deck/jacket/pile/soil) model so that all areas of inelastic behavior are represented and load redistribution properly modeled. Each analysis will utilize identical nonlinear representations of structural components including capacity reduction and post-yield modification for damage and degradation. Large deformation modeling is critical in each analysis since gravity effects (i.e., p-delta) will contribute significantly to the collapse mechanism.

Monotonic Pushover Analyses

Pushover analyses are used to define the load/deformation behavior of a structure up to and beyond ultimate capacity. This method requires the definition of a load pattern which, for seismic conditions, will typically be based on a response spectrum analysis. The nodal inertia forces resulting from this response spectrum analysis are then applied to a nonlinear model and scaled proportionately to induce yielding and buckling of structural members until the structure fails or reaches some other intended target. The results of this analysis can then be used to assess the platform through the evaluation of ultimate strength or energy absorption. These values can be compared to the design reference level loading to establish a Reserve Strength Ratio which can be used to calculate a safety index and probability of failure. Procedures for monotonic pushover analysis include static and pseudo static methods.

Pseudo-Static Pushover

Pseudo-static pushover analysis methods are commonly used to assess the ultimate strength of a platform when post-ultimate response is not required. In these analyses, the applied load increases or remains constant. Pseudo-static pushover analyses produce less computational problems that static pushover and, at times, time domain. The fundamental limitation with this procedure is that it is does not explicitly recognize the difference between brittle and ductile response.

Static Pushover

Static pushover analysis methods are used to assess ultimate strength and residual capacity in the post-ultimate region. These methods provide a basis to calculate ductility and the monotonic strain energy of the structure. Static pushover therefore provides a much enhanced assessment of the resistance under extreme earthquakes; however, these methods require the ability to analyze the response of the structure beyond the ultimate strength which entails a higher degree of nonlinearity beyond that of the pseudo-static method.

The comparison of ultimate strain energy to a reference strain energy has been used for the DLE analysis of new platforms. Earlier versions of API RP 2A (fifteenth edition) included a recommendation of four times the energy at SLE loading as a check for adequate ductility. While this same comparison can be made on existing platforms, it does not provide a simple basis to calculate failure probability.

The static pushover method establishes (1) static ultimate strength, (2) failure sequence for the selected loading pattern, (3) energy absorption for the selected loading pattern, and (4) critical or weak elements in the structure. Aspects of dynamic structural response such as load reversal, soil and structural cyclic degradation, change in vibration characteristics due to nonlinear response, and soil and structural hysteresis are not represented through pushover analysis. These features of structural response are often critical and should be included when establishing the survivability of a platform subjected to extreme earthquake loading.

Time Domain Analysis

Time domain analyses are used to determine if a structure can survive a predetermined event that is characterized by a site ground motion record. These ground motion records are traditionally defined with the use of measurements from actual earthquakes that are scaled to match a site-specific uniform risk spectra. In some instances, synthetic records provide a more rigorous test of a structure owing to the greater uniformity of ground motion intensity relative to structural vibration frequency.

The strength and nonlinear load-deformation characteristics of structural elements and soils are modeled as in the static pushover analyses. In addition, time domain analyses provide bases for evaluating critical response quantities such as dynamic response, load reversal, soil and structural cyclic degradation, change in vibration characteristics due to nonlinear response, increased effective damping due to soil and structural hysteresis, explicit definition of phasing and local vibration effects.

A base shear history from a time domain analysis is compared with a load-deformation result from a static pushover on the same structure in *Figures 4 and 5* [3]. In this example, the base force reached 16,200 kips during the time domain analysis, which was 30% more than the static pushover strength.

Foundation Response

The behavior of the near-field soils adjacent to the piles affects the response and resulting stability of the platform in several ways. The hysteretic behavior of the soils results in substantial energy absorption, which in turn increases the effective damping of the systems and tends to provide some isolation of the motions "imparted" to the platform. The plastic deformation and degradation of the soils increase the vibration periods of the primary modes of the structure, often reducing the dynamic response to the ground motions.

Platform failure modes are typically caused by the large deformation effects (i.e., p-delta) resulting from story drift and global rotations. A prominent source of platform deformation results from the plastic deformations and degradation of the soils. The dynamic resistance of the pile/soil system must be monitored closely to ensure no instability is triggered by the significant reduction in foundation resistance or plastic deformation of the foundation.

An example of a single pile load and resistance time history resulting from an extreme earthquake loading is provided in *Figure 6* [4]. The curves shown in this figure include

- The pile head axial load
- The total soil reaction (friction and end bearing) in resistance to the pile axial load
- The remaining soil capacity (tensile and compressive) in resistance to pile axial load.

The third curve demonstrates the loss of axial strength due to cyclic degradation which occurs for each of the primary loading cycles for the pile. The difference between pile head axial load and total soil resistance indicates the dynamic response of the pile/soil system.

In this example, there are several cycles of loading which exceed the static strength of the pile, but the foundation has not failed. A monotonic pushover analysis would indicate pile failure at this load level. In each cycle that exceeds the existing soil resistance generates plastic deformation of the pile, load redistribution to other piles and change in load path through the jacket. Excessive plastic deformation will result in increased overturning through p-delta, which may initiate platform failure.

OTHER ASPECTS OF SEISMIC REASSESSMENT

The total risk associated with extreme earthquake loading includes aspects of response in addition to total platform collapse. For example, the response of the topside facilities when subjected to extreme vibrations and deformations should be given an suitable level of consideration in a seismic reassessment [5]. The risk associated with the collapse of a drilling mast or gas leaks due to failed piping systems may be comparable to that of platform collapse due to potentially greater failure probabilities. These and other response issues add to the total life and environmental hazard. Proper risk management deals with all hazards and attempts to achieve the most cost-effective mitigation of all sources of risk. It is important that the industry continues to develop guidelines for minimum performance levels for the major structural components of our existing platforms. It is important that operating companies recognize that these aspects of seismic reassessment constitute only a part of earthquake hazard to their platforms.

CONCLUSIONS

A substantial amount of research and development has been completed in the area of Reserve Strength assessments for new and existing platforms, focused primarily on platforms in regions where storms and hurricanes are the dominant environmental loading conditions. The lack of new construction in seismically active areas over the last 10 years has retarded the development of technology associated with earthquake engineering for offshore structures. This lack of development comes at difficult time when public perception regarding the safety of offshore operations is generally negative and there is increased concern about future large earthquakes. Regulatory agencies must be accountable to public perception and need to call on industry to provide assurance that the aging offshore infrastructure can be maintained and operated safely. Industry must therefore respond with new technologies and procedures commensurate with the difficulty of the problem.

Methods traditionally used to assess the structural safety of platforms in hurricane and storm environments can result in erroneous conclusions when applied to platforms in seismically active areas. Methods are available to perform more rigorous analyses, but these require more personnel resources, complex software and computer modeling. The need for simplified assessment procedures for the Gulf of Mexico, where the vast majority of the U.S platform fleet is situated, should not affect the development of procedures for west coast platforms where (1) the total number of platforms is small, (2) the consequence of failure is greater due to the impossibility of shutting down and evacuating prior to an earthquake, and (3) the nature of loading and response is more complex. Simplified analytical methods should be used as one of several logical steps in the assessment process in which the objective is to develop a reliable understanding of the performance of the platform to support reasonable and informed decisions regarding future use.

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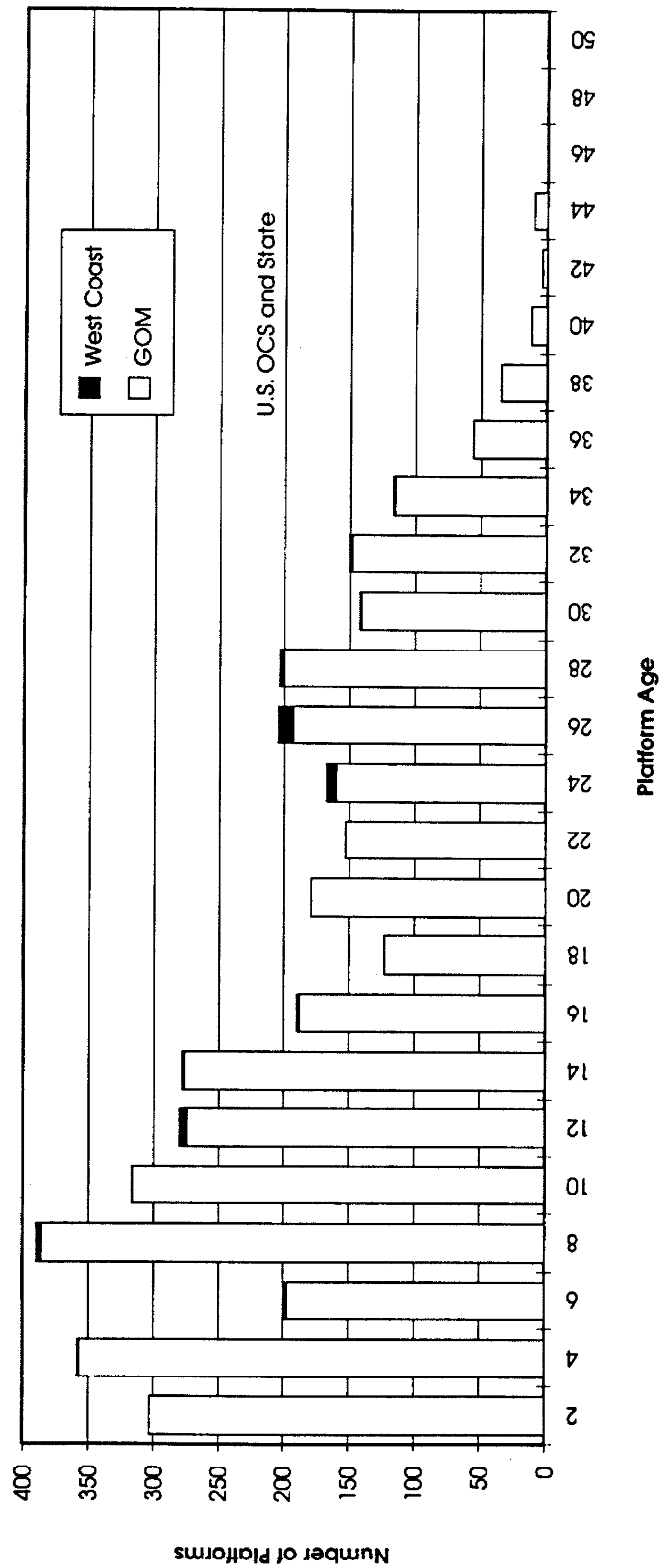
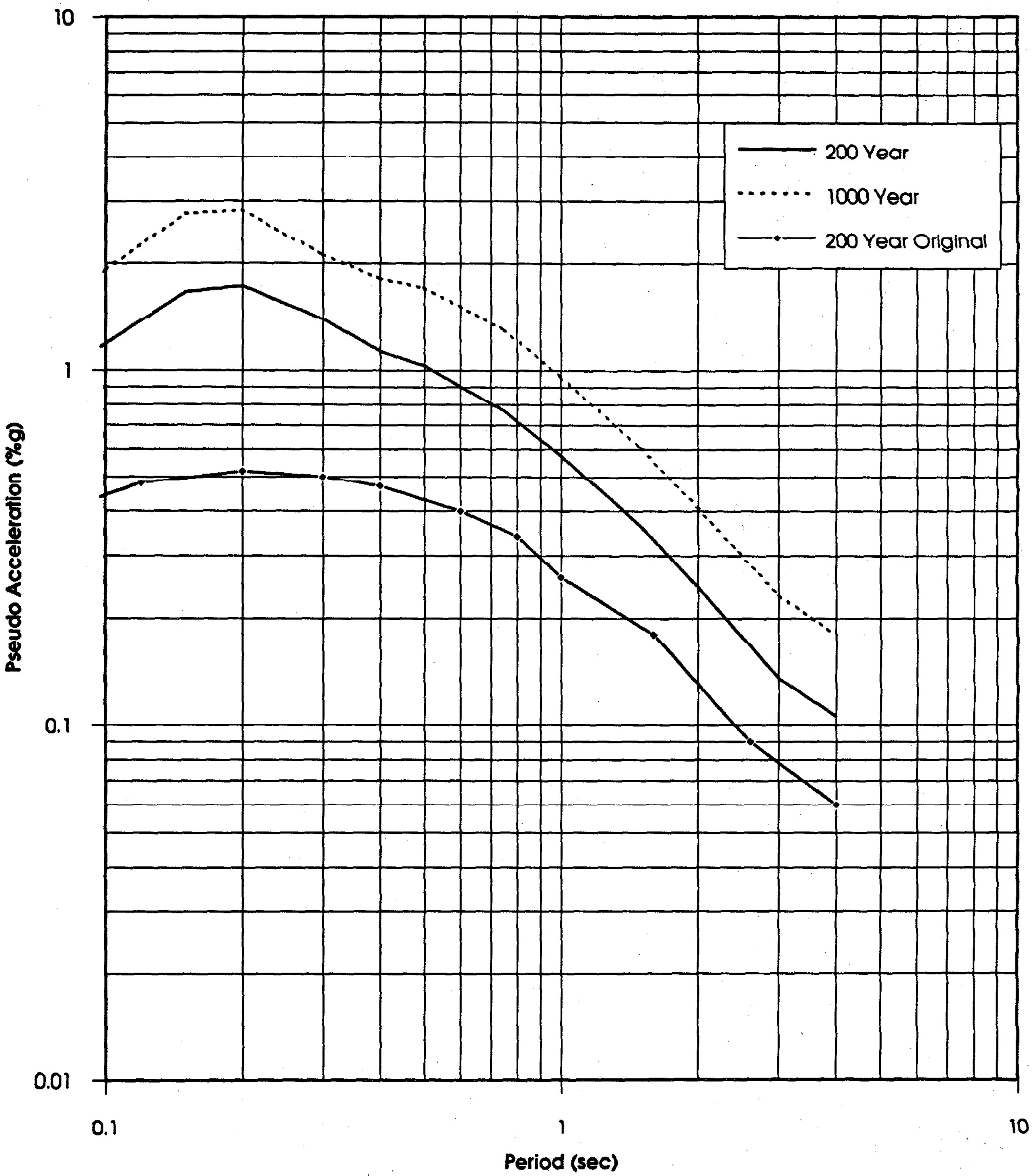
Platform Age by Region

Figure 1



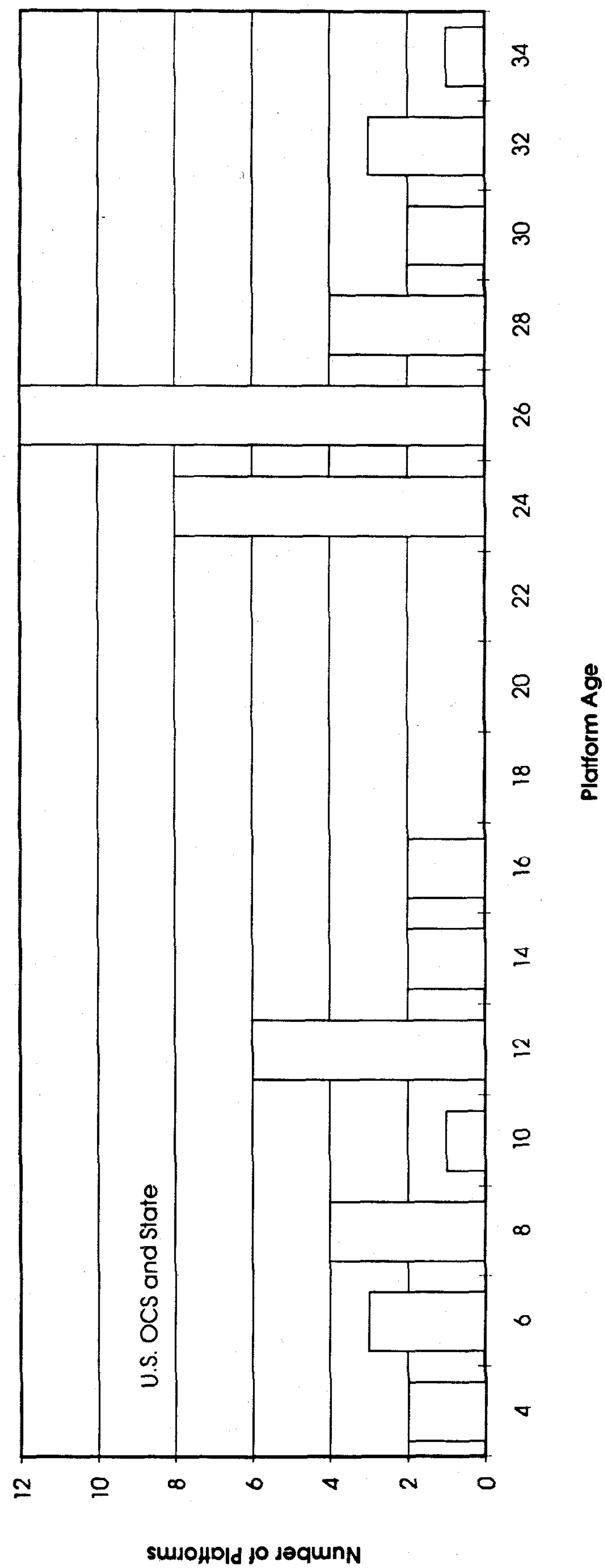
Comparison of Original Design and Current Site Seismic Criteria

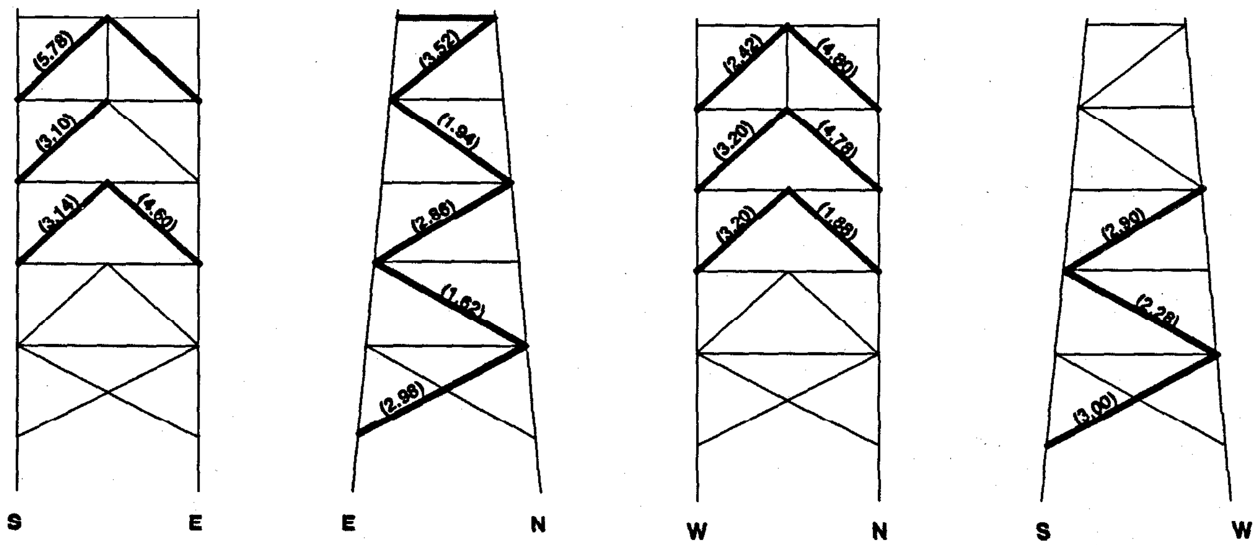
Figure 2



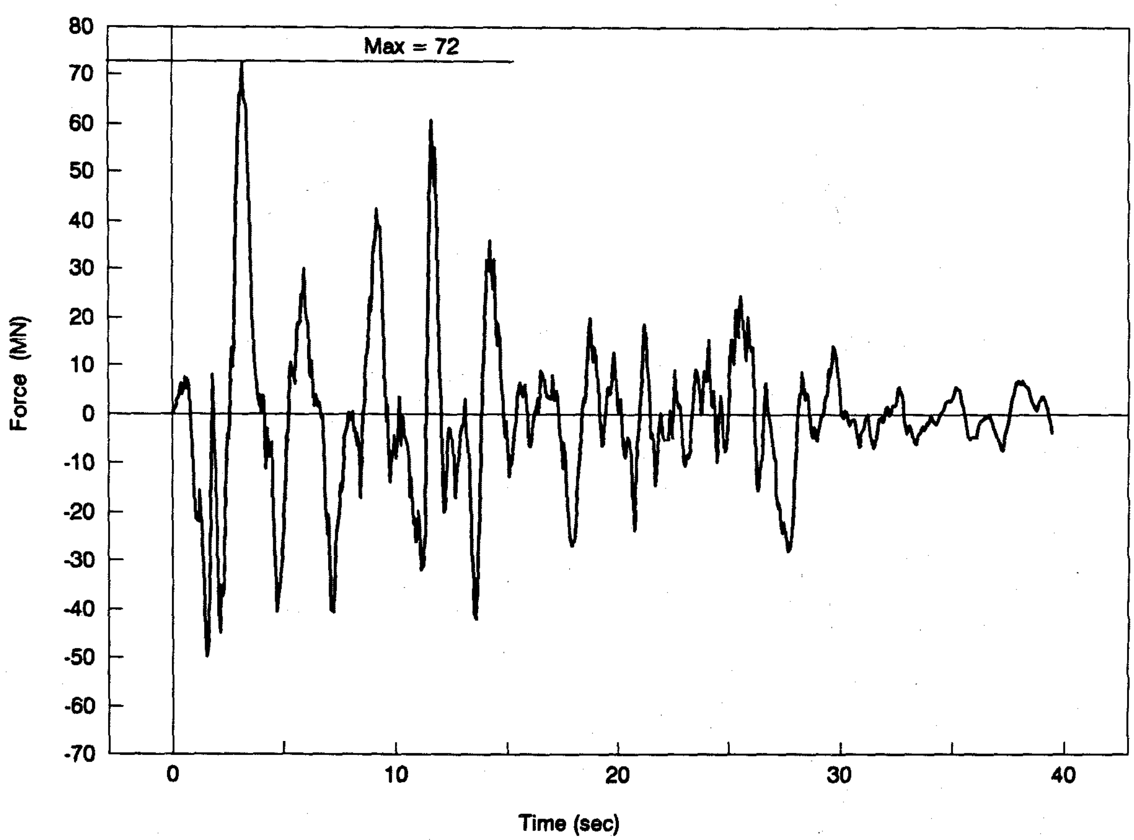
Age Distribution for West Coast Platforms

Figure 3



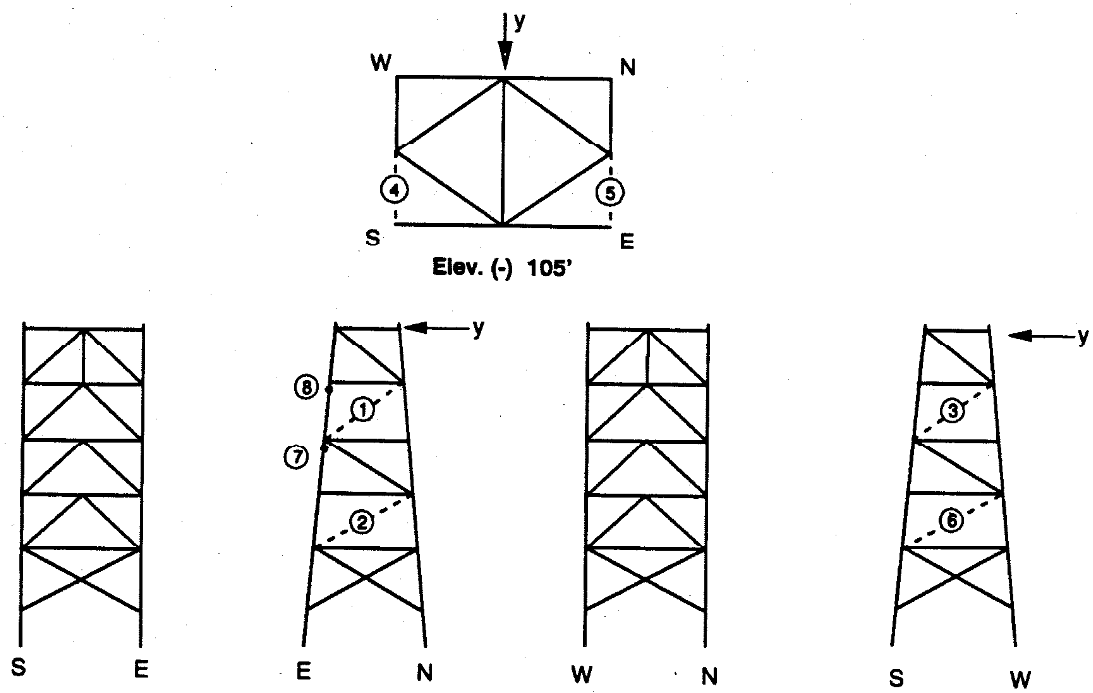


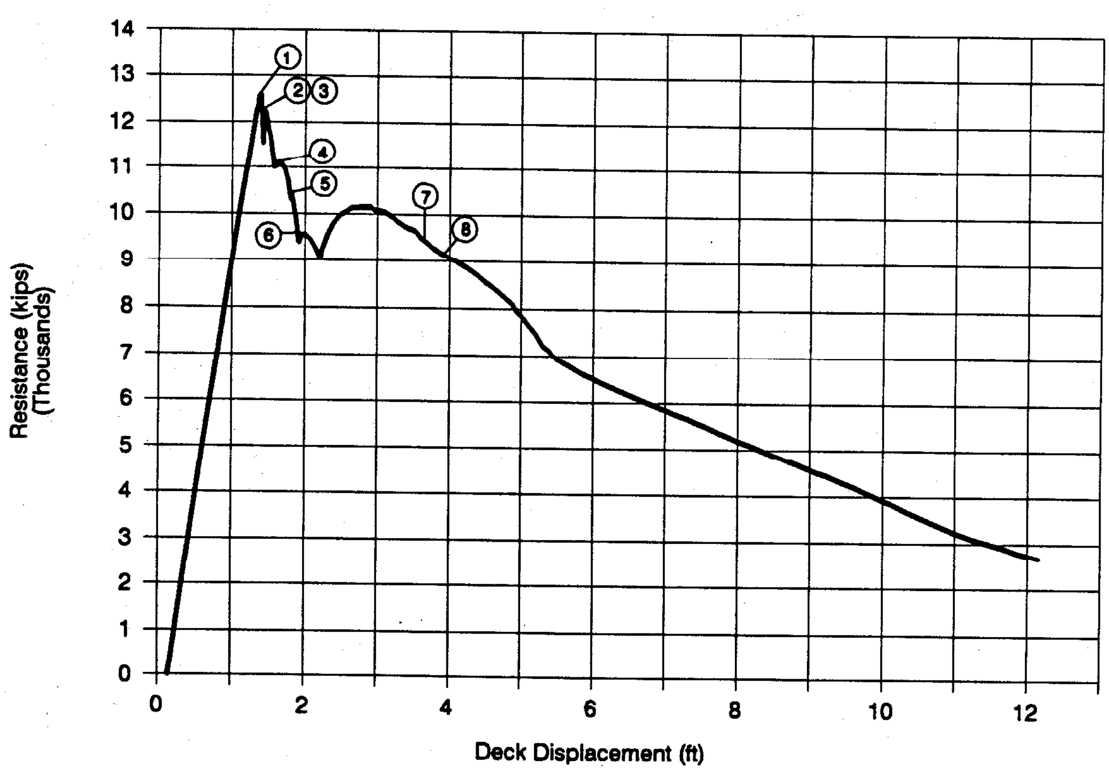
( ) - indicates time of first occurrence



DLE Analysis Results - Base Shear

Figure 4



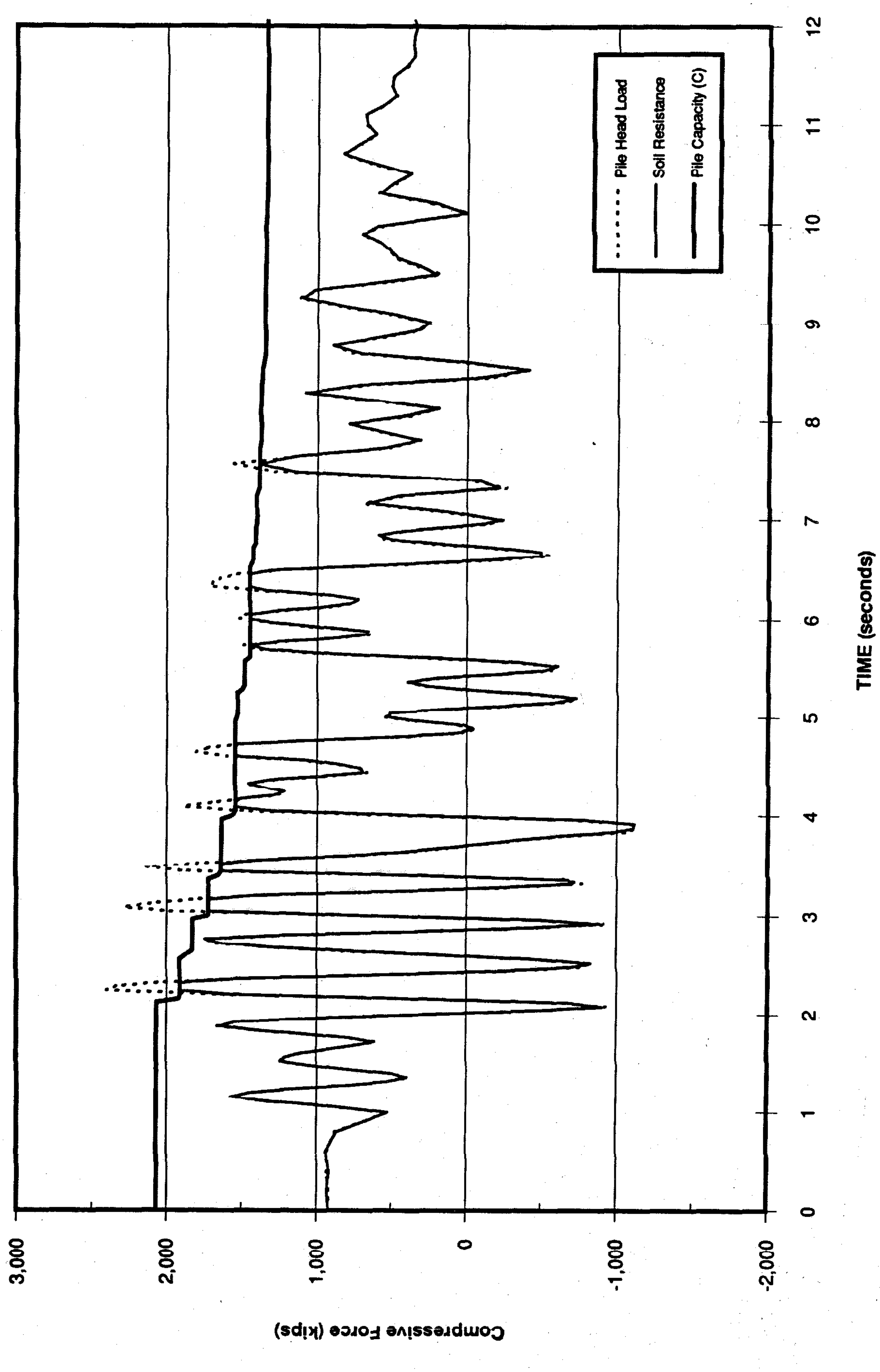


Pushover Analysis Results

Figure 5

Pile Dynamic Response

Figure 6



Issues in the Assessment of Pile Behavior During Seismic Events

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Introduction

This general overview is written to highlight the activities involved in the design and reassessment of offshore structures for seismic loads, primarily fixed structures, associated with the performance of pile foundations and to indicate topics for consideration. This paper is not a state-of-the-art document and is not intended to be a comprehensive literature review. It is intended instead to provide a point of departure for discussion at this workshop.

Relatively little reassessment of pile foundations for offshore structures in seismic zones has been performed to date, and there are no codified procedures. However, the four-step procedure for requalification of offshore structures in general, described by Bea (1992b) and Marshall (1992), provides a rational framework for an approach to cost-effective requalification of foundations for old structures in seismic zones.

Reassessment vs. Design in the Context of Foundations

Reassessment of pile foundations differs from *design* in that:

- Indirect data on pile capacity exist, usually from pile driving records at the time of installation, that permit fairly accurate hindcasting of static axial capacity via wave equation modelling, particularly in sands. Such data do not exist *a-priori* in the design stage, which usually results in a safe bias (higher factor of safety) on axial loads in the design phase than might be needed for requalification. In addition, improved estimates can be made of soil profiles from the driving records, from which more accurate estimates of lateral pile behavior can in turn be inferred.

- Foundation performance under past environmental loadings can be inferred through system identification techniques using soil-structure interaction analyses if the response of appropriate points in the structure were monitored. Such performance can be used to forecast potential performance during stronger seismic events.

- Loads to be resisted by the piles through quasi-static loading and through superstructure feedback are not the same for reassessment and design because reassessment loadings may not need to include loads from drilling operations, and environmental loading criteria may have changed.

- Piles, like other elements in the superstructure, may have suffered damage or deterioration since the time of installation.

Bea (1992b) identifies four steps in the requalification process, ranging from very simple qualitative analyses (Case 1) to research-level analyses (Case 4), with the objective of clearly requalifying the structure as fit or unfit for service at the lowest level possible. This discussion will focus on Case 2 and 3 analyses (coarse and state-of-the practice quantitative analyses), with suggestions about general research for Case 4.

To set the stage for discussion, in terms of Case 2 and 3 analyses, the following general procedure may be representative of those used to reassess the competence of pile foundations in a seismic area.

A. Identify and assign weighting (probability) potentials to faults within 100 km of the structure site. Select a set of explicit events with a return period within about ten times the desired future life of the structure (e.g., 1/100 probability of annual occurrence if qualified life is ten years. This value of return period would be shorter than that for initial design if the future life is shorter than the initial design life.

For example, one could evaluate time histories of rock motion at a specific site for M=6 on a fault 30 km away, M=7 on a fault 40 km away and M=7.5 on a fault 50 km away from the structure, etc., using appropriate attenuation procedures [e.g., Joyner and Boore, 1988 (strike slip faults); Ho and Tsiatas, 1992 (deep subduction faults)].

B. Compute site-specific ground motion using site-specific soil stiffness data and an appropriate mathematical model [e.g., SHAKE (Schnabel et al., 1972)] to simulate ground motion at the level of the foundation.

C. Using this ground motion, conduct linear spectral analyses of the entire structure ("strength analyses") using standard criteria, such as 5 per cent structural damping in all modes, under the most critical seismic events without considering the piles themselves explicitly. Only pile stiffnesses at their connecting nodes are considered. However, in the words of API RP 2A (API, 1991) "pile-soil performance and pile design requirements should be determined on the basis of special studies." These "special studies can come both before and after the spectral analysis of the structure.

In order to estimate pile stiffnesses with appropriate consideration of changes in axial and lateral stiffness due to prior environmental cyclic loadings and cyclic loadings from the modelled seismic events, nonlinear time domain simulations may be made. Such simulations can also be used in the following step to assess the consequences of loadings generated at the foundation nodes from the linear response analysis. Pile analyses could be accomplished by using such software as PAR, SPASM, DRIVE or similar programs (referenced later).

D. From the linear spectral analysis of the structure, which may consider many events with ground motions predominating in different directions, determine the loadings on the piles, identifying the pile with the highest load from each case studied. Ensure that the highest-loaded piles carry structural "feedback" loads combined with static bias loads that are lower than the loads that will produce either (1) a structural failure in the pile (with due consideration of prior damage, corrosion and other factors) or pile failure by pullout in or compression in the soil with an appropriate factor of safety, or (2) that the reaction loads at the pile heads, when multiplied by appropriate load factors, are less than the structural or soil-failure capacities of the piles multiplied by appropriate resistance factors. The latter approach, the "LRFD" approach, is just beginning to find its way into foundation engineering practice, both offshore and onshore.

In a Case 4 study, it would be advisable to assess the resistance factors on a true probabilistic basis, taking account of the probability distribution of soil properties and of the resulting unit load transfer functions if a load transfer function analysis is used.

Pile failure by pullout is most likely in tall, slender jacket structures or in TLP's, while failure in shear or moment are likely for standard jacket structures in shallow water.

E. Since a linear spectral analysis can conceivably give misleading results with respect to the safety of the piling system (and other parts of the structure), due to the highly nonlinear nature of pile response, particularly lateral response, assure that the structure/foundation system has adequate ductility. For seismic loading a ductility analysis can be conducted using equivalent quasi-static loading on the structure, employing a very long return period for the most critical seismic event (e.g., 1/10,000 probability of annual occurrence, such as M=7.75 event on a fault 30 km away from site) to obtain the loads. The objective of such a ductility analysis is to develop a reserve strength ratio (RSR, Bea, 1992b) by dividing the lateral collapse load by the minimum lateral design loading currently specified by design guidelines (e.g., API, 1991). An appropriate acceptance criterion is that the RSR will exceed some minimum value, such as 1.2. Lower bound collapse loads may be developed by observing the load producing, for example, the first plastic hinge in either the structure or a pile (in a pile, this hinge usually occurs at some point below the mudline), while the upper bound collapse load corresponds to complete "pushover" of the structure. The differences in these bounds provide some measure of redundancy.

The collapse loads of the structure, assuming the piles to be structural members, can be assessed using a nonlinear space frame program [e.g., INTRA (Arnold et al., 1977) or a simplified plasticity method (Murff and Wesselink, 1986)]. Nonlinear soil response must be prescribed for these analyses, hopefully specific to the site and with as little bias as possible, and so-called p-δ effects should be simulated due to the expected large lateral motions of the pile heads. API RP 2A permits exceptions to this level of analysis when it can be demonstrated that no "soil instability" (e.g., significant pore pressure buildup in the free field) will occur under the extreme event, that the intensity of this event is less than 2 times that of the (strongest) event considered in Steps A-D, and certain other structural characteristics exist.

Pile-Structure Interaction Issues

Embedded in the above approach for reassessment of offshore structures are several issues relating to pile-soil interaction. Many of these issues are also pertinent to initial design:

A. Modelling the axial and lateral response of single piles in preparation for the linear spectral analysis (Step C). Shown in Fig. 1 is a rheological representation of the SPASM model for lateral loading (Matlock et al., 1978), in which soil response is modelled by a set of near-field, degradable "p-y" curves that must be specified by the user. A companion model, DRIVE (Foo et al., 1977) is often used for parallel axial pile response analysis. A similar state-of-the practice model is the PAR numerical model (Bea, 1992a). In SPASM and DRIVE selected time histories of lateral, free-field soil motion are applied to the elements representing the soil, which in turn load the pile, whose inertial and dynamic stiffness response is appropriately represented. Radiation damping is specified by the user. The results (deformations, shears, moments) are expressed in terms of time histories.

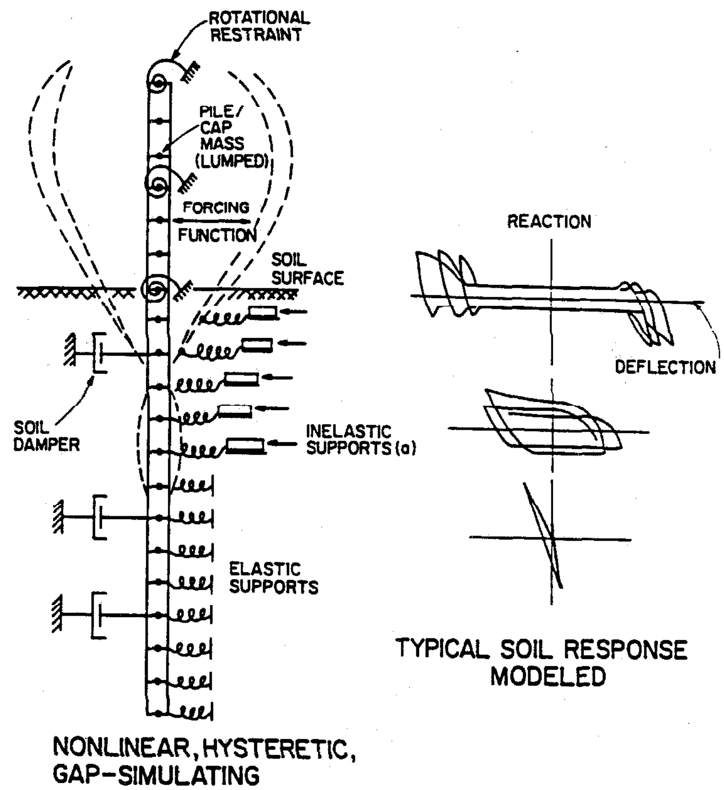

Fig. 1. SPASM Rheological Model

Alternate pile models can also be employed, for example, Nogami and Chen, 1987 (near field p-y element/far field dynamic plane strain element); Otani, 1990 (similar to Nogami and Chen but for pile groups); and Chen and Penzien, 1984 (hybrid finite element/boundary element for single piles or groups). The discrete element models of Nogami, Otani and similar models (illustrated in Fig. 2) combine the capability of simulating nonlinear, degradable soil behavior, including gap development, with plane strain wave propagation, whereas the model of Chen and

Penzien and similar models permit full coupling of pile and soil but restrict soil characterization to linear viscoelasticity.

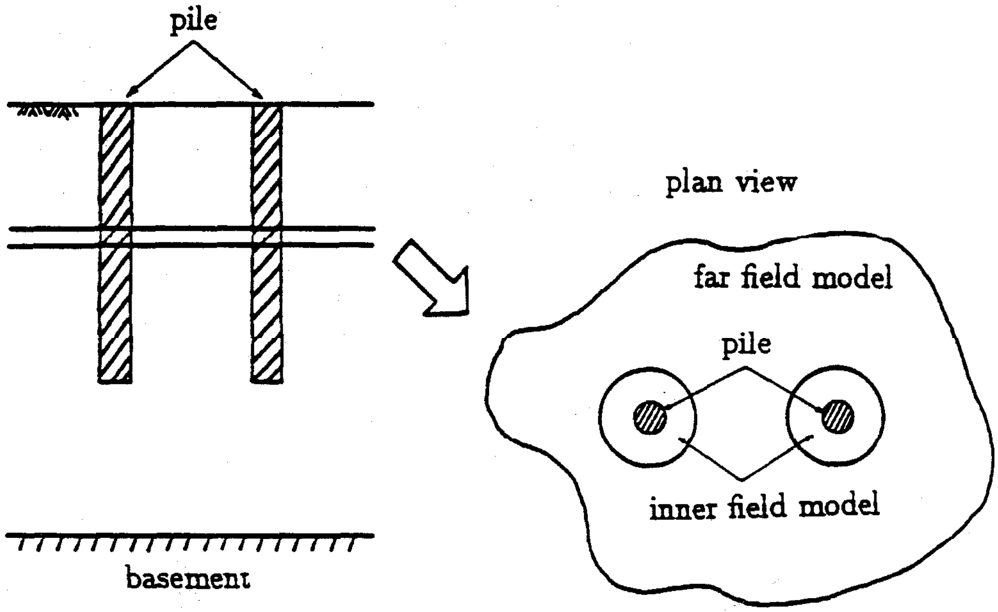

Fig. 2. Otani's Rheological Model

While the SPASM/PAR approach has been the most popular state-of-the-practice pile analysis method, since it can simulate soil degradation and hysteretic damping in terms familiar to most geotechnical engineers, the other models permit more rational assessment of geometric damping and soil-pile coupling, so that Case 3 analyses should logically include both approaches to give the analysis team a thorough picture of the bounds of pile behavior.

Solutions for a particular case of forced dynamic lateral loading at frequencies typical of seismic motion are illustrated to provide some sense of how accurately pile response can be modelled under nearly ideal field conditions. A full-sized pipe pile was vibrated with combined lateral and rocking motion in a simulated superstructure through rapid 30-second downsweeps with a force amplitude sufficient to produce gaps between the soil (submerged, overconsolidated clay) and the pile (10.75 in. steel pipe). The conditions of the test are shown in Fig. 3 and explained in more detail by Blaney and O'Neill (1986). The lateral response spectrum for the base of the simple superstructure was simulated by both the SPASM and the Nogami-Chen methods. For the SPASM solution p-y curves were constructed from undrained soil shear strength profiles for the site using the method prescribed by API for cyclic loading (API, 1991) ("I") and using a simplified method developed by Kagawa and Kraft (1980) ("III"). For the Nogami-Chen solution criteria developed from cyclic lateral pile loading tests on several piles of widely varying diameters at the test site were used to define the p-y behavior. The criteria that were used for cyclic response in stiff clay (Dunnavant and O'Neill, 1989) are shown pictorially in Fig. 4, in which terms are defined by Dunnavant and O'Neill. In the former analysis full gapping was assumed to be produced at all levels, while partial gapping, expressed through the coefficient β, defined by Nogami and Chen (1987), was assumed in the latter. No explicit, cycle-by-cycle soil degradation was assumed in either analysis.

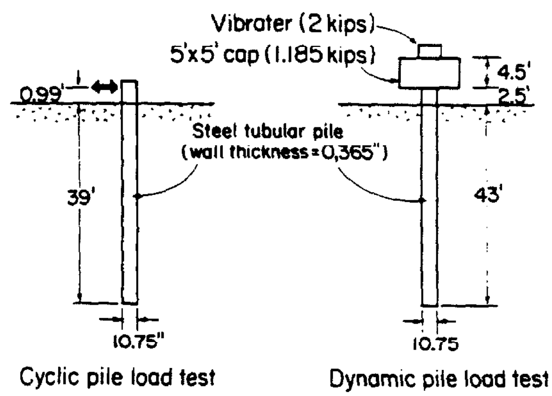

Fig. 3. Conditions of Lateral Response Test (Nogami and Chen, 1987)

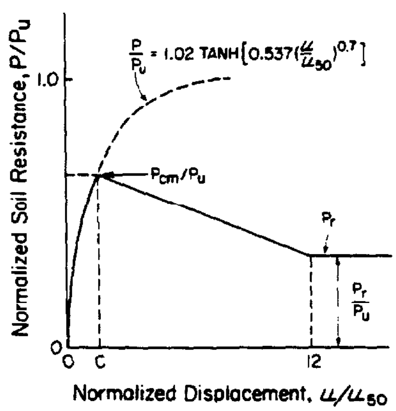

Fig. 4. p–y Criterion of Dunnavant for Cyclic Loading in Stiff Clay (Dunnavant and O'Neill, 1989)

The two solutions are shown in terms of predicted vs. measured displacement response spectra in Figs. 5 and 6. While both methods simulated the response at very low frequency and resonance well, suggesting that the soil stiffness was modelled adequately with either approach, the Nogami-Chen model provided better simulation of response above resonance.

Centrifuge testing, pioneered by Scott (1980) and others for laterally loaded piles, can also be employed to accomplish the same objective. Finn and Gohl (1987) and Kutter (1992) have demonstrated the utility of the centrifuge to measure pile-head response and bending moment distribution in model piles and pile groups under toe-level seismic excitation in both dry and saturated sand, respectively. In the former study strong interaction was observed among two-pile groups. A thorough evaluation of a specific foundation, perhaps at the Case 4 level, would involve both mathematical modelling and centrifuge modelling.

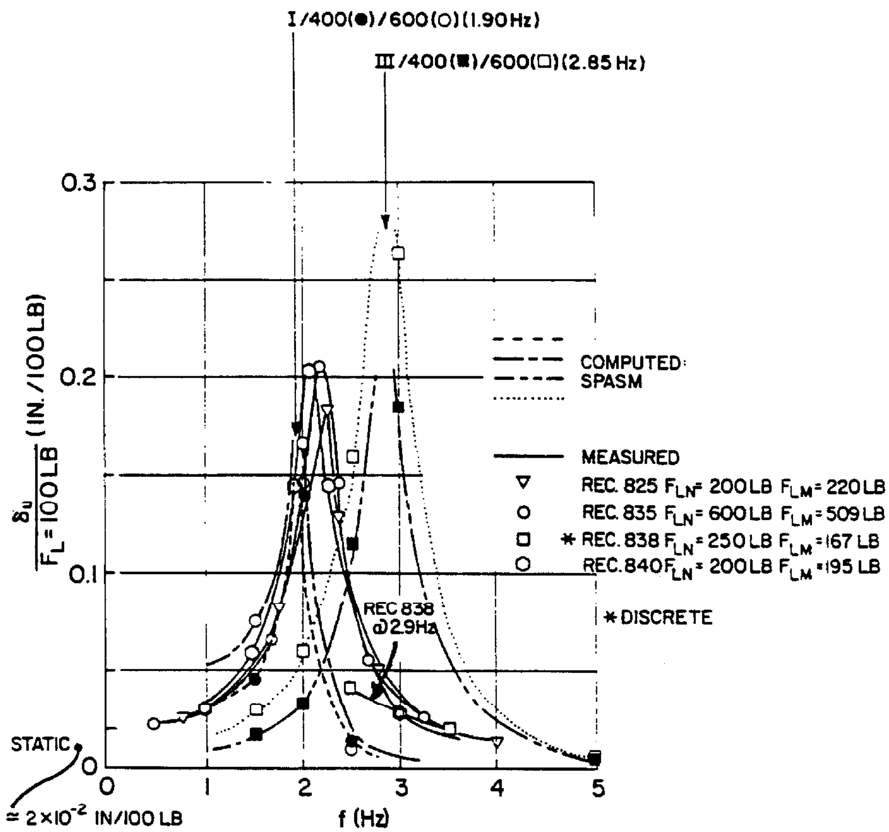

Fig. 5. Displacement Response Spectra for SPASM Analysis (O'Neill et al., 1982)

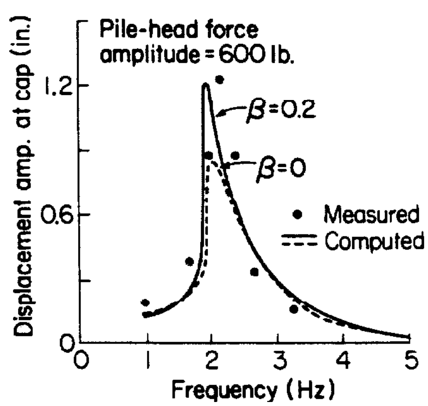

Fig. 6. Displacement Response spectra for Nogami-Chen Analysis (Nogami and Chen, 1987)

When a relatively few cycles of strong shaking exists, unlike the conditions of the test reported above, full cyclic degradation of the soil will not occur around a pile. In time domain analyses of such condition, cycle-by-cycle degradation is typically specified. Degradation is typically accompanied by stiffening and strengthening due to rate effects, so that the net effect of the two phenomena may be small. For dealing with the unit axial pile shaft response relations (e.g., as used in DRIVE), Poulos (1983) suggested that the product of degradation (D_τ) and loading rate (D_R) factors that should be used as a multiplicative correction term for axial unit soil resistance along the pile shaft be of the following form where reversals in plastic stress can occur:

$$D_\tau D_R = \left[(1-\lambda)(D'_\tau - D_{min}) + D_{min}\right]\left[1 + F_p \log_{10}\frac{\zeta}{\zeta_r}\right] \tag{1}$$

in which

D_τ = degradation factor following a plastic stress reversal,

D'_τ = degradation factor that existed before the preceding plastic stress reversal,

D_{min} = minimum degradation factor, to be evaluated at the judgment of the analyst, taken perhaps to be the ratio of the limiting value of shaft resistance to the peak value from cyclic rod shear tests,

λ = degradation parameter, typically ranging from 0.1 to 0.5 according to Poulos,

F_r = rate coefficient, typically increasing from about 0.01 to 0.1 for normally consolidated clays with liquidity indexes increasing from 0.1 to 1, according to Bea (1980), and 0.15 for moderately plastic, normally consolidated clays from the Gulf of Mexico according to Bea (1992a) and becoming as large as 0.25 according to Poulos,

ζ = actual loading rate, and

ζ_r = reference loading rate (e.g., rate normally encountered in a static loading test).

Other degradation rate models have been proposed more recently, but this model expresses the phenomena involved sufficiently accurately for discussion. More fundamental models for clay soils that determine pore pressure changes within the soil mass, such as that proposed by Matasovic and Vucetic (1992), offer encouragement for improving the understanding of degradation effects in clays. Similar models exist for quantifying changes in stiffness and for lateral loading.

An important issue, however, is whether degradation/rate models are required at all for RSR analyses. At the very large monotonic pile deflections simulated in RSR analyses, exceeding significantly the deflections that occur in a seismic event, cyclic loading has no effect on the lateral soil resistance, and if structural failure does not occur in any pile until such deflections are achieved throughout the system, consideration of degradation is not necessary, although rate effects may still be operable. However, if structural collapse or pullout occurs in any pile at small deflections (brittle structural behavior) within or slightly greater than the range of deflections that exists during seismic loading, cyclic degradation criteria may control, and the RSR will be small, possibly disqualifying the structure. In the simplified plasticity model described by Murff and Wesselink (1986), only the ultimate resistance of the soil is used, and cyclic degradation, if it exists, is not considered.

B. Evaluating the effect of combined degradation, rate of loading and pore pressure generation. In sands, seismic loadings can produce both degradation and rate effects, where degradation can occur due to reductions in lateral effective stresses against the pile wall resulting from volume change in the soil around a pile and to the generation of pore water pressures due both to the passage of seismic waves and the motion that those waves induce in the pile, which in turn shears the soil and generates pore water pressures. This behavior, coupled with feedback loading from the superstructure, may produce failure of the pile for consideration in Step D of a reassessment analysis.

O'Neill et al. (1992) describe a laboratory experimental program to evaluate such effects for axial behavior. Seismic acceleration data from the MMS seafloor SEMS unit on station near Long Beach, California, were recorded for several moderate seismic events in 1986. An event of Richter Magnitude 5.8 on a fault just offshore of Oceanside, California, on July 13, 1986, produced a high quality record at the SEMS site 74 km to the northwest. This site was a deep soil site, with mixed granular and cohesive sediments to a depth of at least 400 feet based on borings at the site. The combined horizontal components of the measured event were scaled to a target spectrum in the complex frequency domain, to maintain phase in the time history (proper damping), and then transformed to the time domain to produce the record shown in Fig. 7. The target spectrum was the 50 per cent probability spectrum for a Magnitude 7.0, 7.5 or 8.0 event at 74 km from the epicenter, produced according to a procedure described by Trifunac (1979). This record was converted to displacement by first repeating the train of strong accelerations from 20 sec. to 42 sec to account for the longer period of shaking from the Magnitude 7-8 events.

A model pile was driven into submerged sand that had been placed in a test chamber approximately 24 inches in diameter at various relative densities. The sand column was bonded frictionally to the rigid base of the chamber, to which was imparted rotary motion that produced the linear motion of the modelled seismic event at the radius of the pile from the center of rotation. The remainder of the column was isolated from the chamber by Teflon liners and a latex membrane. Effective stresses were applied isotropically to the sand column from the top and lateral boundaries to simulate various mean depths of pile penetration.

The driven one-inch-diameter model pile was subjected to biased uplift loads ranging from less than one-half of its true or "unbiased" static capacity to 90 per cent of that capacity. In addition, the biased load was applied through a mass-spring system whose scaled natural frequency was in the range of that which would exist for a deep-water TLP. The overall testing arrangement is shown in Fig. 8.

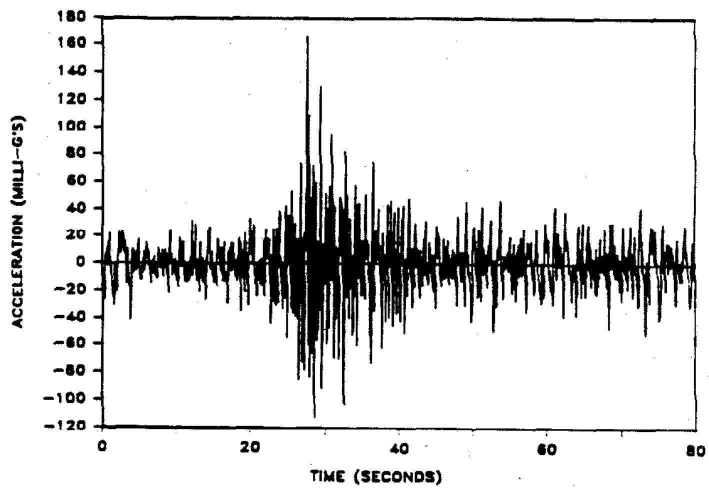

Fig. 7. Oceanside Event Acceleration Record at SEMS Site Scaled to Magnitude 8.0: Vector Combination of Orthogonal Horizontal Components (O'Neill et al., 1992)

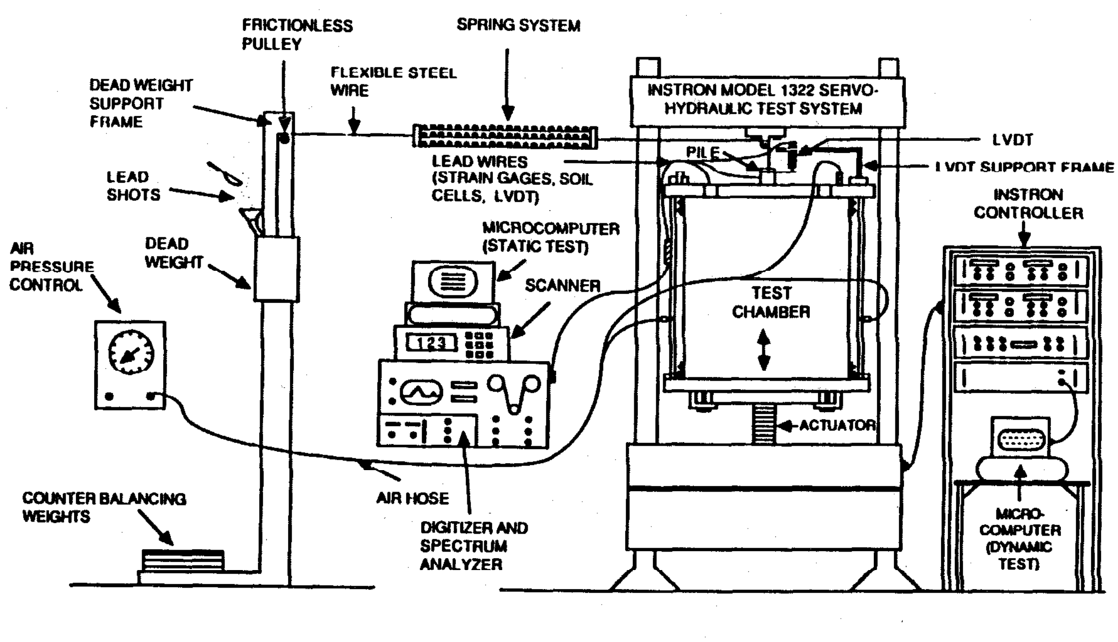

Fig. 8. Testing Arrangement for Laboratory Simulations of Seismically Loaded Tension Piles: MMS Study

Typical results for a failed pile driven into a clean, fine sand, deposited at a relative density of 55 per cent, are shown in Fig. 9. It is noted that the induced pore water pressure in the near field, adjacent to the pile wall, reached only about 0.24 times the ambient effective stress in the chamber when failure (sudden pullout) occurred in this test. Clearly, liquefaction was not the cause of failure, although elevated pore pressures were produced. For this pile the static resistance computed from the API design formula, using K = 1 and δ = 29°, which were both known conditions, was 0.55 times the ambient effective stress, and 0.76 times this value, or 0.42 times ambient effective stress was applied in "skin friction" in static bias. That is, the static factor of safety in this test was 1/0.76 = 1.32. In addition, dynamic excursions in axial load produced from lateral motions of the vertical tension tendon during the shaking test of the pressure chamber, were of the order of 15 per cent of the static bias, thus producing even higher instantaneous loads in skin friction (static plus feedback shear loads of about 0.48 times the ambient effective stress, but without reversals of direction of stress).

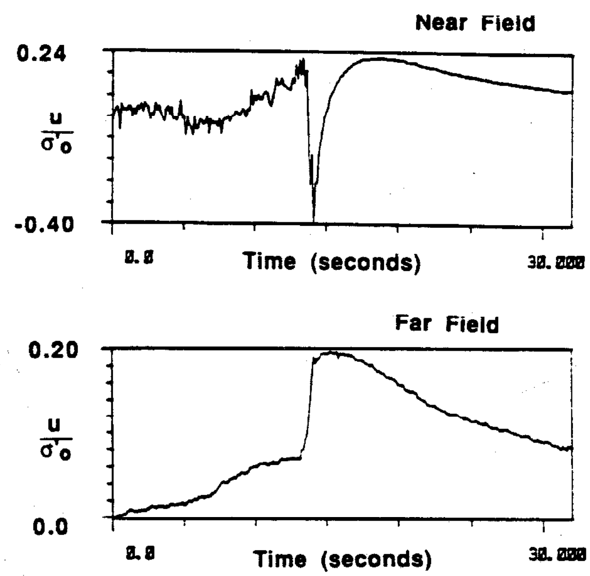

Fig. 9. Pore Water Pressures Measured During Test in Which Bias-Loaded Pile Failed

The pile, therefore, was able to accommodate pore pressures that were higher than should have been necessary to cause failure, since an increase in only 0.07 (0.55–0.48), not 0.24, times the ambient effective stress was necessary to develop the design skin friction resistance of 0.55 times the ambient effective stress. This suggests that the installation of the test pile and/or seismic loading of the system produced locally higher lateral effective stresses against the pile wall than the ambient effective stress. Static tests on the piles suggested that this the actual skin friction was 1.5 to 2.0 times that computed from the simple formula that gives 0.55 times ambient effective stress.

This observation points out the importance of removing biases from estimates of pile capacity prior to conducting a reassessment of a pile foundation. In this simulated case, use of a standard design rule would have unnecessarily penalized the foundation.

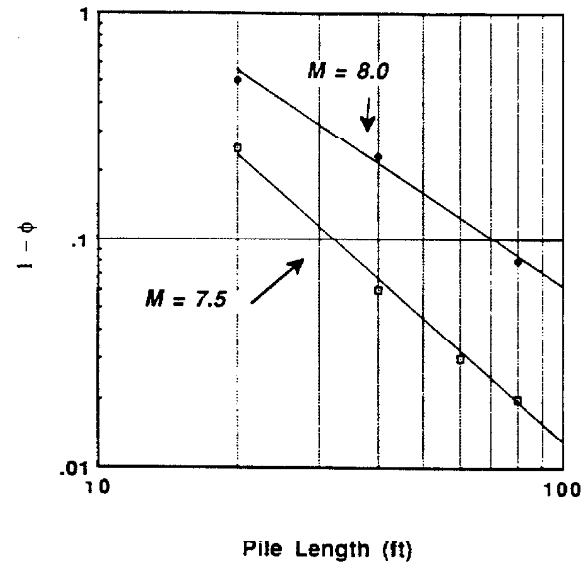

Fig. 11. Capacity Reduction Factor φ for Conditions Modelled for 55% Relative Density

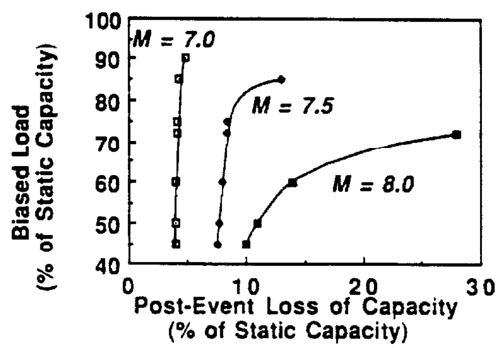

Fig. 12. Loss of Static Side Shear Capacity in Upper 40 Feet Following Simulated Seismic Event; Relative Density = 55%

De Alba (1983) demonstrated experimentally that induced pore water pressures in a model sand deposit due to cyclic shear loading but without inertial loading produced approximately equal loss of capacity in single piles and in groups of four piles spaced 2.5 diameters on center, which tentatively suggests that the results described above are at least approximately applicable to groups of piles as well as to single piles.

While these results should not be generalized, they point out issues that need to be addressed on a site specific basis and describe an experimental methodology that can be used to approach the problem. An interesting question that arises from such studies is whether the vertical component of seismic seafloor motion produces vertical compression waves in the ocean that in turn induce pore water pressures in the soil not accounted for by most upward-propagating wave analyses. Ochoa (1990) demonstrated that these fluid overpressures could exceed pore pressures induced directly by seismic motion.

C. Arriving at unbiased (non-conservative) estimates of static lateral and axial pile capacity. In order to make a rational reassessment of the suitability of pile foundations to withstand seismic loadings, it is important to determine the most likely static axial capacities of the piles as well as their most likely lateral behavior. In a design context, such an approach is not necessary, as conservative estimates usually suffice. Similar comments are appropriate for stiffness computations, except that apparently conservative ("soft") estimates of pile stiffness can result in unconservative predictions of superstructure response when natural periods of the pile-structure system are longer than the fundamental period of seismic motion.

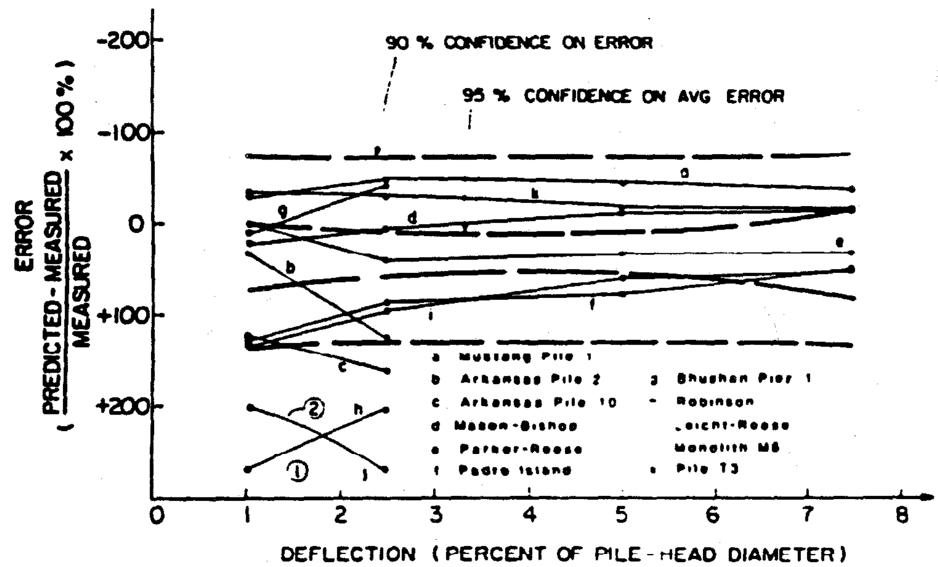

Fig. 13. Error in Prediction of Lateral Pile-Head Using Present (1991) API Criteria for p–y Relations

Design models for unit load transfer curves, such as p-y curves, contain bias and also carry a degree of uncertainty. This observation is illustrated by considering the principal full-scale tests in the data base against which the current API RP 2A criteria for cyclic p-y curves for piles in sand were tested (Murchison and O'Neill, 1984). Since this method was developed for design it has a built-in bias, as suggested in Fig. 13, which shows error in pile-head deflection vs. deflection using this method. The mean error, or global bias, is seen to be between about 0 and +70 per cent, with the + sign indicating overprediction of movement (p-y curves yield pile behavior that is too flexible). Without adjustment for site-specific conditions, pile-head flexibilities derived from use of these relations will produce response spectra, in linear spectral analyses that consider foundation stiffness, that are probably shifted toward periods that are too high (although Fig. 13 suggests that there is a significant probability that the shift will occur in the other direction), which can give misleading pile reactions and other component loads. The updating of p-y curves and removal of bias will need to be generated by reassessment of p-y behavior from prior environmental loading events, where structural response (and hopefully loading) data exist. Alternatively, *in-situ* testing techniques that simulate better the action of laterally loaded piles than the technique presumed to be used to characterize the soil for use in the API method (the dynamic penetration test), such as the pressuremeter test, can serve to reduce bias and uncertainty.

Similar effects can be seen for non-cyclic axial loading for piles in sand in Fig. 14, which is taken from Dennis and Olson (1983) and which uses a version of the API method ("APIS") that is similar, but not identical, to the present (1991) design method. In this case the mean computations for the method are about ten per cent conservative, but a very large degree of scatter exists, more so than for lateral loading, above.It is virtually impossible to eliminate bias from estimates of pile capacity at a given site, however, without direct capacity measurements at that site. For example, O'Neill (1986) showed the variation of measured static plunging capacities of eleven full-sized pipe piles in clay over a relatively small terrestrial site (<10 m wide) with laterally uniform soils. The data are reproduced in Fig. 15, in which the shaft, toe and total capacities, both measured and computed from the present API criteria for clay, are indicated. These data suggest that reanalysis of driving records of each individual pile for a given structure may be necessary in order to establish site-specific mean pile capacities and variances thereof, and that a probabilistic analysis should be made of both the linear response and the ultimate capacity of structures.

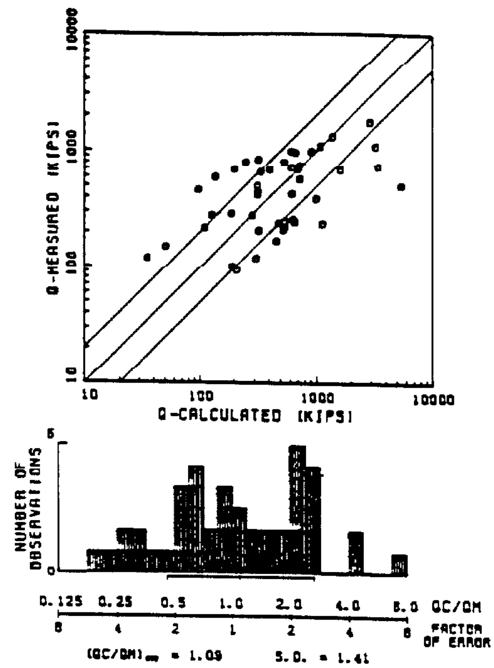

Fig. 14. Variation in Error of Prediction of Static Capacity for Piles in Sand, APIS Method (Dennis and Olson, 1983)

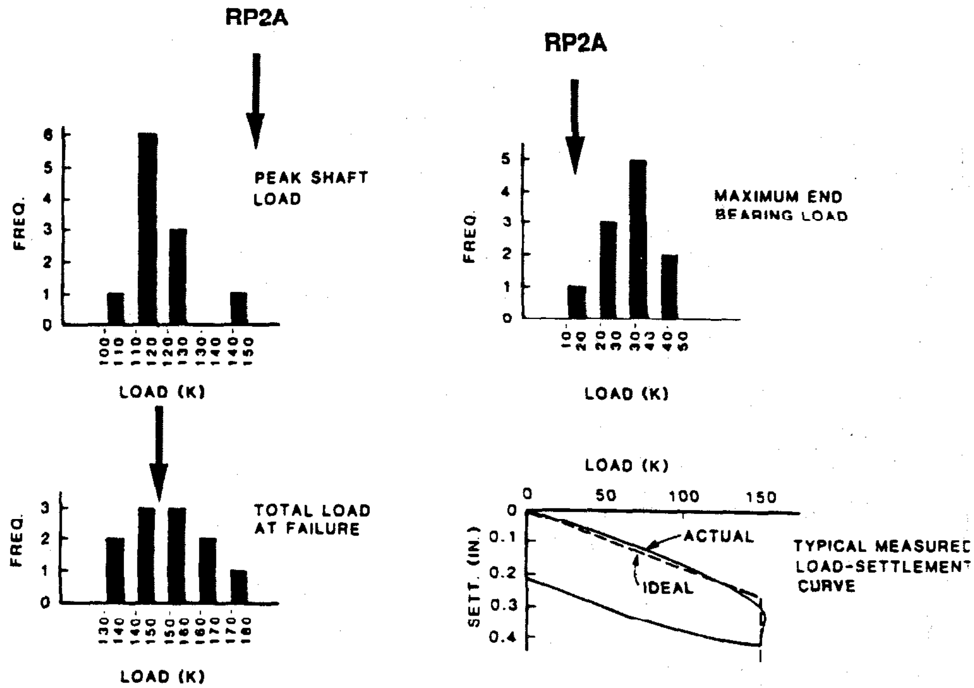

Fig. 15. Variation in Axial Capacity of Pipe Piles in Clay over a Small Terrestrial Site (O'Neill, 1986)

Mean axial capacities and variances can be updated for a given structure if information on pile capacity already exist in a given area through loading tests (e.g., on conductors). A method proposed by Nadim and Lacasse (1992) for updating capacities of legs for jackup structures is directly adaptable to piles for fixed structures. Such an updating method, which can use data from load tests on conductor piles or bearing piles for nearby structures or from nearby test sites on shore or probabilistic models to establish an initial estimate of both mean pile capacities and variances, is preferable in the view of the author to taking only values of pile capacities computed through hindcasting procedures using wave equation methods, since such methods are prone to errors, some of which can be systematic.

D. Modelling the behavior of the soil in the development of spectra for Case 2/3 analysis. In conducting the linear response analysis of the structure, an attempt should be made to assess the effects of ground stiffness on the relevant periods of shaking and the effects of wave propagation through the soil on liquefaction or pore pressure buildup around piles. Proven computational models such as SHAKE (Schnabel et al., 1972) may be sufficient for assessing ground stiffness, while computational models such as APOLLO (Martin and Seed, 1978) may be used to estimate the potential for free-field liquefaction or the generation of sub-liquefaction pore water pressures. Norris (1992) indicates that sub-liquefaction pore pressures can be accounted for in the analysis of piles under seismic loading using a model such as SPASM by reduction of stiffness in p-y curves, presumably in proportion to the reduction in effective stress caused by the pore pressure generation/dissipation behavior. If liquefaction occurs, piles must be able to resist the inertial loadings from the soil above the deepest zone of liquefaction. Norris has also suggested that pore pressure buildup, below liquefaction levels, may have been partially responsible for changes in foundation response, which affected superstructure response, at the Cypress Interchange in the Loma Prieta Earthquake of 1989.

Other Issues

Other issues involved in the design and assessment of pile foundations, which will not be elaborated upon here, are:

- Determining whether shallow flowslides can occur during seismic events for sites on sloping seafloors. Should slides be possible, their additive effects on piles must be addressed (e.g., Lee et al., 1991).

- Assessing whether group action will have an effect on piling response through wave interference with pile motion during the seismic event. Normally, frequencies are small enough and pile spacings large enough that this effect is not important. However, relatively straightforward approximate analyses of this effect can be made (Nogami, 1980; Kaynia and Kausel, 1982).

- Assessing group effects for quasi-static overload analyses. Group effects for quasi-static overload analyses can be accomplished by modifying p-y curves to account for reduced soil stiffness and capacity produced by overlapping stress fields (Brown et al., 1987; Brown and Shie, 1991). Based on both full-scale testing of groups in stiff clay and finite element modelling, Brown and Shie recommend that p and y values on p-y curves for "von Mices" and "Extended Drucker Prager" soils be multiplied by the P and Y factors indicated in Fig. 16 before conducting lateral (overload) analyses. The recommendations are valid strictly for piles in square matrix configurations, hence the identification of "rows."

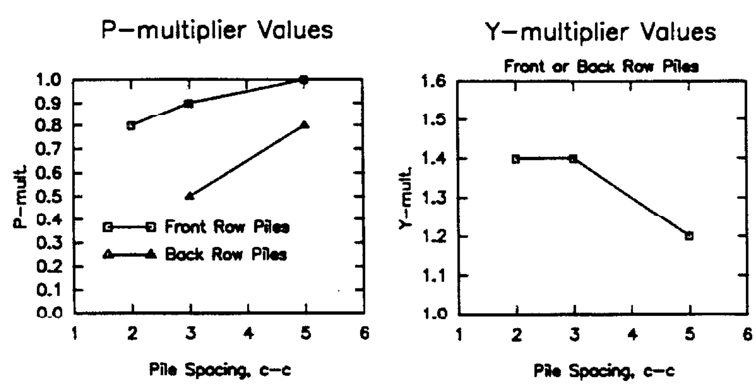

Fig. 16. P and Y Factors for Quasi-Static Pile Group Analysis as Assessed by Brown and Shie (1991)

- Assessing probability of occurrence of significant storm loadings during significant seismic events and their combined effect on pile response. At present, this effect is not normally considered.

- Evaluating soil properties at the actual site of construction of a structure, since structures are often constructed some distance from the position of the nearest soil boring(s). For Case 2/3 analyses, such evaluation should be as expedient as possible.

Acknowledgments

The author acknowledges the helpful information given by Dr. Don Murff and Dr. Todd Dunnavant of Exxon Production Research Company and by his colleague at the University of Houston, Dr. A. N. Williams.

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Operations Issues In Seismic Design And Reassessment

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"If it cannot be expressed in mathematics, it is not fact - only opinion." R.A. Heinlein.

Summary

This paper summarizes the potential risks to the topside facility of an offshore platform from an earthquake. To-date there has been no recorded earthquake damage to any platform located in the active seismic regions offshore California, Alaska, Japan, Indonesia and the Mediterranean.

From both quantitative and qualitative analyses it is concluded that the risk of topside earthquake damage is low, provided all equipment and piping are adequately restrained and the drilling rig and other slender structures are designed for the appropriate earthquake loading. Comprehensive seismic design and assessment guidance for topside facilities will be provided in the twentieth edition of API RP 2A.[1*]

Introduction

The seismic safety reassessment of an offshore platform involves the evaluation of the adequacy of a number of platform components and management systems.[2] This paper deals with operational considerations of the seismic safety assessment. The key word is "*operational*" and in this context it includes the design, operation and re-qualification of the platform topsides.

The inherent purpose of an offshore platform structure is to provide the support for the topside equipment, i.e., the drilling rig, the wells, the artificial lift equipment, the process equipment, and the compressors and pumps, needed to get the oil and gas out of the ground and process it for transportation to shore. The potential risk to the safety of this topside equipment from an earthquake is the topic of this paper. This earthquake risk is quite often ignored. In fact, there is very little specific information in the literature regarding the design of topside facilities for earthquake loading.

The paper reviews the potential earthquake hazards to process equipment, pipeline risers, drilling rig and cranes, living quarters, and wells and conductors. Each of these hazards will be

* References are located at the end of the paper.

briefly discussed. Thoughts on potential research that may be beneficial to reduce, or better understand, these risks conclude the paper.

Risk Assessment

Risk assessment is a generic term covering a wide range of techniques used to assess the level of safety by considering both the magnitude of harm or damage and the likelihood of such harm occurring. Such assessments can be qualitative or quantitative or a mixture of the two. Comprehensive probabilistic risk-based assessments have been used as part of the re-qualification procedure on a few West Coast platforms.[3]

The potential risk to an offshore operation comes from many different external factors. The distribution of platform loss causes are shown in Figure 1. The platform can be damaged or destroyed by storms, by a collision, by a blowout, by a fire or explosion in the process facility, and, the subject of this conference, by an earthquake. Platform risks from storms, collisions, blowouts and fires and explosions are reasonably well defined. An extensive database of accidents has been accumulated over the past twenty years by the Minerals Management Service from the almost 4,000 operating platforms in the Gulf of Mexico.[4] As noted on Figure 1 the majority of platform losses are from storms, i.e., hurricanes.

There has, to the best of the author's knowledge, not been any significant earthquake damage to an offshore platform. Seismic environments where platforms are located include offshore California, Alaska, Japan, Indonesia, and the Mediterranean. This excellent record is probably not due to good design, but to the fact that, to-date, no offshore platforms have been exposed to a major earthquake.

Topside Risk Assessment

The seismic reassessment of a topside platform facility involves the analysis of a complex system which includes hazardous materials, equipment, control and safety systems, people and management systems.

In the absence of statistical earthquake damage information, the earthquake risk assessment of an offshore platform facility must be based on a qualitative assessment. In other words, it must be postulated what can happen to the topsides when an earthquake hits, and then the likelihood of such an incident is determined.

This can involve the preparation of a variety of models assessing the consequences of various incidents. Information that may be useful in these models is available from the experience at onshore oil and gas facilities during earthquakes.[5,6,7] The steps involved are to (1) identify potential hazards, (2) postulate a specific accident, (3) determine the potential frequency based on failure rates or educated guess work (heuristic), (4) determine the consequences, (5) determine the risk, and (6) determine whether or not the risk is acceptable.

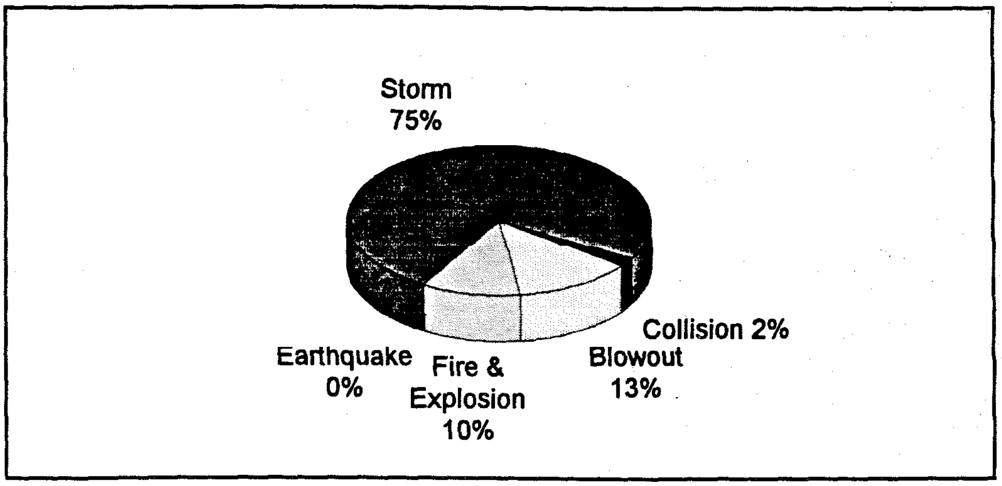

Figure 1. The platform risk pie, distribution of causes of platform losses offshore the United States, 1956 to 1991.

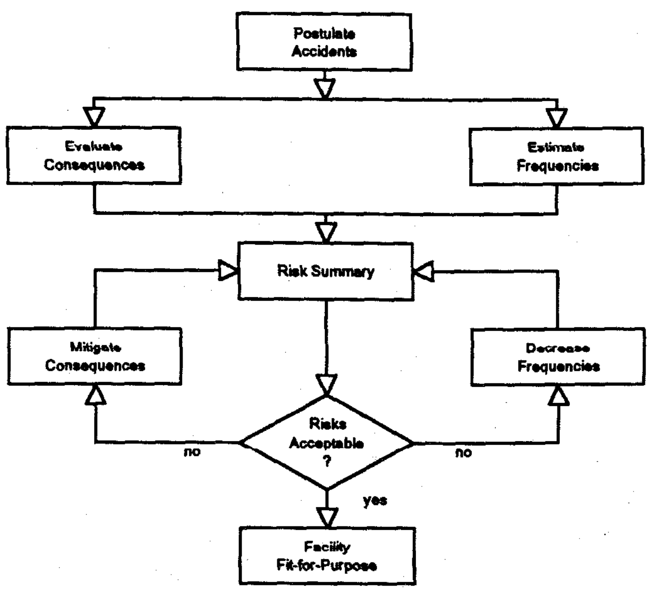

Figure 2. Risk management methodology.

If the risk is not acceptable, the risk can be reduced by decreasing the frequency, by mitigating the consequences, or by doing both. For instance, the consequences of an earthquake can be mitigated by making the platform unmanned or, alternatively, the frequency of failure can be reduced by using more reliable control devices. The methodology is illustrated in Figure 2.

Potential Topside Hazards

The potential hazards to a platform operation from an earthquake are:

1. Process equipment damage,
2. Pipeline riser rupture,
3. Drilling rig or crane collapse,
4. Quarters building collapse, and,
5. Well or conductor failure.

Each of these hazards will be discussed in the following.

Process Equipment Damage

As mentioned earlier, there is no database of earthquake damage to an offshore facility. Yet, as part of the design or re-qualification effort, it must be determined, either implicitly or explicitly, what the risk is, and whether or not that risk is acceptable.

One method to determine the risks is through the use of event trees. The relative risk of an earthquake generated process incident is illustrated in the event tree shown in Figure 3. By analogy with what has happened onshore, accident scenarios can be postulated and the risk estimated at each step. In this example, the platform facility is exposed to the strength level earthquake, in this case, an earthquake with a 200 year return interval.

The assumption is that nothing happens to the structure, because it was designed for that level earthquake. It is assumed in this event tree that there is a 50 percent possibility that some damage occurs to the process equipment or piping. Other events are postulated, such as the probability of the emergency shutdown system (ESD) and/or the fire fighting equipment not working because of the earthquake, that escaping oil and/or gas will ignite, and so on.

The event tree shown in Figure 3 indicates a 2.5 x 10-7 annual possibility that the platform is totally destroyed due to a fire and explosion caused by the 200 year earthquake. In other words, an extremely unlikely event. The frequencies of scenarios with less severe damage are also calculated. With this information it can be determined that the risk is acceptable, or, alternatively, that measures need to be taken at the facility to reduce the risk of damage.

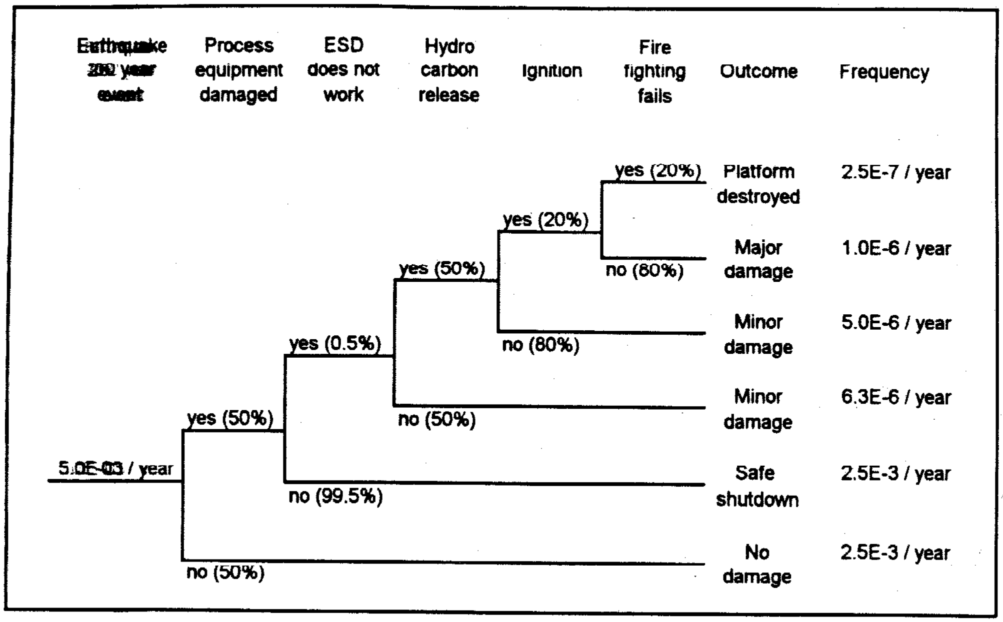

Figure 3. Earthquake event tree for an offshore process facility.

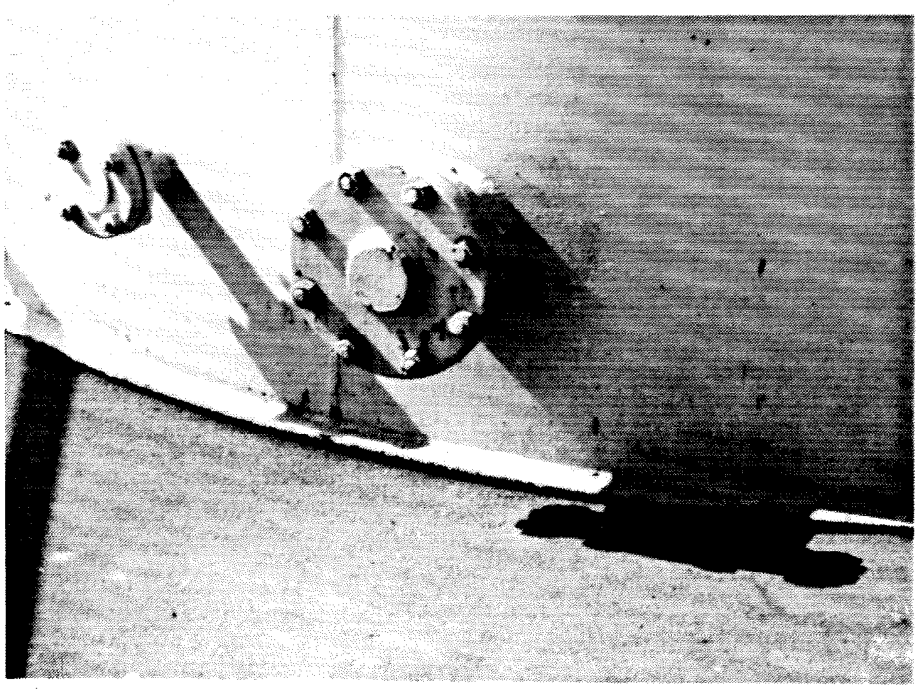

Figure 4. Lack of tie-down on an offshore storage tank.

Actually, reducing the risk of earthquake damage to process equipment is quite easily done on most platforms. Most equipment items are rigid bodies that can be tied to the deck, or to each other, such, that there is no movement in even the most severe earthquake.[8,9] A walk-through on the platform will readily identify equipment and piping that are not properly tied down.

Design and assessment guidelines have recently been developed by an API RP 2A subcommittee to assist in this evaluation. These guidelines will be included, as updated section 2.3.6e.2 and associated commentary, in the forthcoming twentieth edition of API RP 2A.[1]

Onshore experience indicates that most earthquake damage is related to storage tank failures.[6,7] This occurs either because the tank moves across its foundation and thereby damages the piping, or because of sloshing of the liquid in the tank, which causes the roof to fail. Storage tanks on offshore platforms are usually small and not subject to sloshing failure. Offshore storage tanks, however, are occasionally not tied down and may slide across the platform deck in an earthquake. If so, the connecting piping will fail and cause an oil release with the possibility of pollution and/or a fire and explosion. Figure 4 illustrates the lack of tie-down on an offshore platform tank.

It is simple to correct with the right amount of tack welding or the use of clamps. Figure 5 shows a properly tied down oil transfer pump.

Pipeline Riser Rupture

Pipeline risers are normally clamped to the platform structure. The potential earthquake hazards to a pipeline riser are from falling objects, a platform structural failure, or a mud slide.

Falling objects from parts of a collapsing drilling rig, flare boom or crane can conceivably hit the risers and cause a rupture. If there is significant structural damage to the platform jacket it is likely that there is also damage to the risers. An earthquake generated mud slide could pull the pipelines away from the platform and result in a riser failure.

Potential consequences from a pipeline riser rupture are pollution from an oil spill, and, in a worst case, a fire and explosion.

Mitigation of the hazard is effected through the installation of automatic shutdown valves. Offshore the Unites Sates these valves are mandated by regulation. The location, however, of the shutdown valves is not mandated. To provide the intended safety function the valves should be located as close as possible to the point where the lines depart the platform and where the valves are still accessible. This is particularly true for gas pipelines.

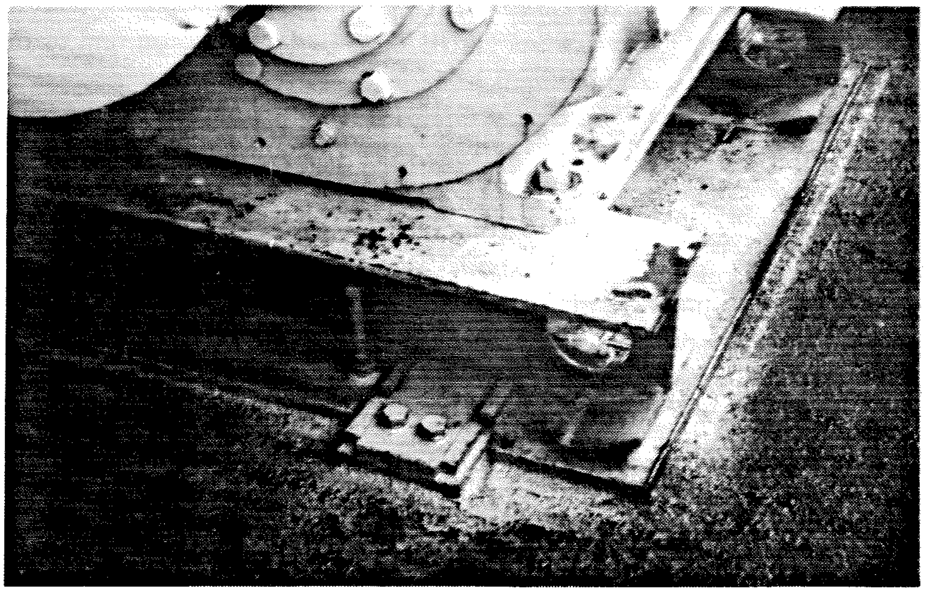

Figure 5. Properly tied down oil shipping pump.

Figure 6. Absence of shut down valves at top of the pipeline risers.

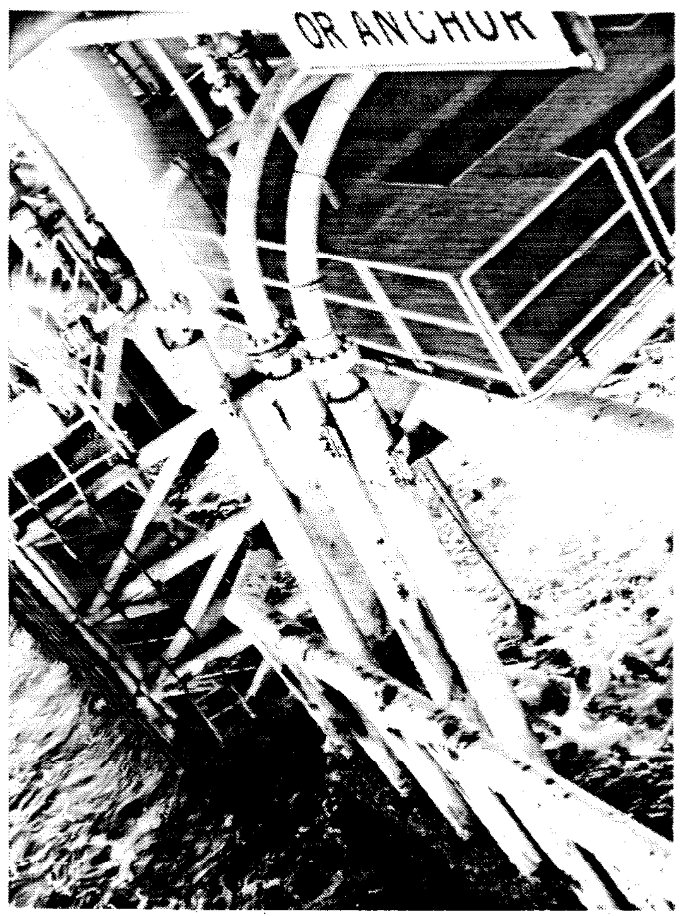

Figure 6 shows pipeline risers that cross underneath the platform. The shutdown valves are located on the other side of the platform. The lines are well protected from falling objects by the solid decks. There is the possibility, however, that the lines could rupture during an earthquake and cause a backflow of gas. This is essentially what happened in the Piper Alpha incident in the North Sea. An initiating fire and explosion caused debris to rupture a main gas transmission line. The gas from this pipeline accumulated under the platform and caused a massive explosion. Although the Piper Alpha incident was not earthquake related, one can easily postulate a similar accident resulting from earthquake damage.

Drilling Rig Collapse

There are a number of slender structures on a platform that may collapse during an earthquake. These are the drilling rig, cranes, flare booms, and cantilevered structures. Debris from these structures could cause damage to process equipment, piping, pipeline risers and to the jacket structure.

Mitigation is accomplished by designing the rig, cranes, flare booms, and cantilevered structures for the appropriate earthquake loading. This may be difficult to do because of the complexity in developing the necessary deck floor spectra.[10,11]

The drilling rigs on offshore platforms are usually set on skid beams to enable movement from well to well. When not being moved, the rig should be tied down to prevent sliding.

Figure 7 shows a picture of a cantilevered deck supporting living quarters and a helicopter deck on a platform in Cook Inlet. Failure of the cantilevered support structure could dump the living quarters into the Inlet. This particular support structure has the further problem that it is not fabricated from low temperature steel. A severe earthquake during extreme low temperatures could create a brittle fracture failure. Means to mitigate such risks, if deemed necessary, include lowering the nominal stresses, heat tracing and material replacement.

Quarters Building Collapse

Potential hazards from an earthquake to a quarters building include the possibility that the building falls off the platform, or, alternatively, that the building itself collapses. Potential consequences are loss of life. To prevent this from happening the building should be designed for earthquake loading.

Figure 8 shows the interior structural framing of a quarters building that was specifically designed for earthquake loading.

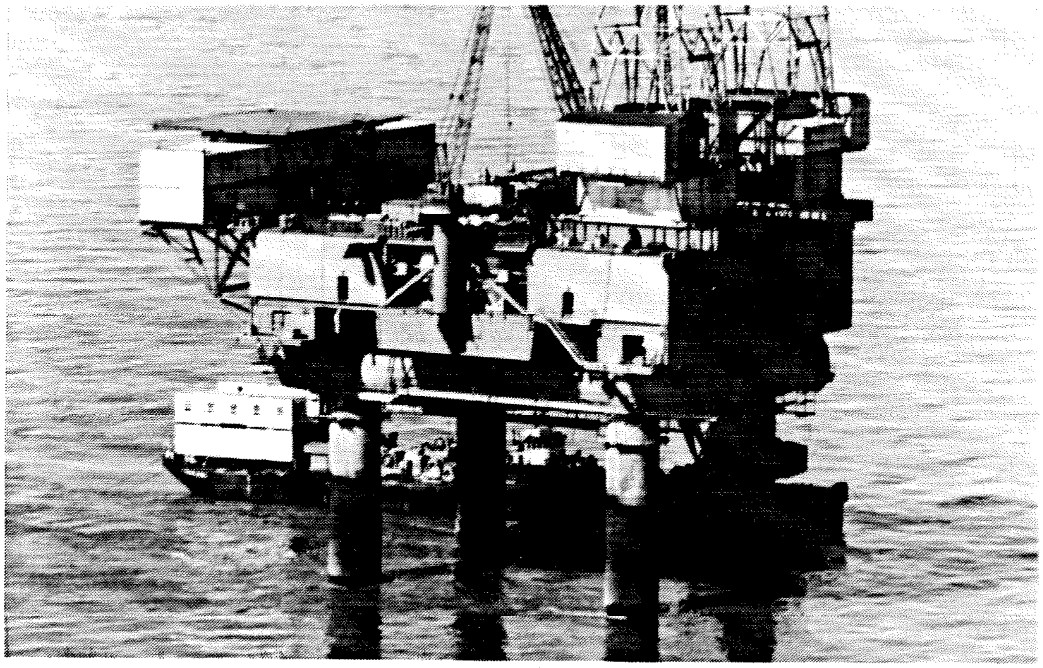

Figure 7. Cantilevered living quarters deck on a platform in Cook Inlet, Alaska.

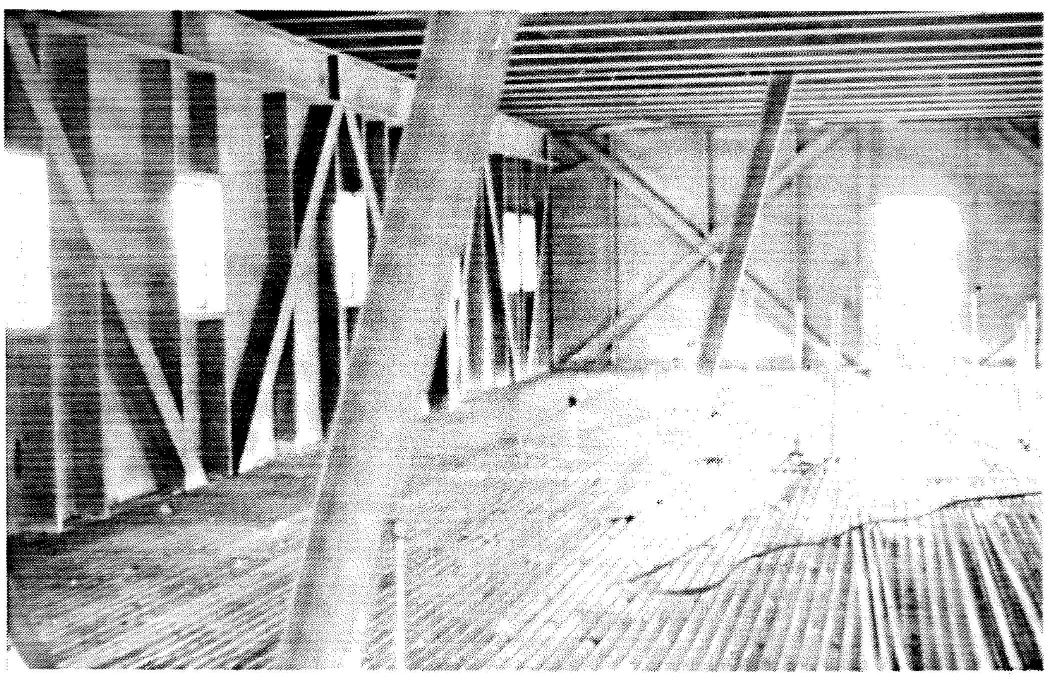

Figure 8. Interior structural framing of a quarters building designed for earthquake loading.

Conductor Failure

Well or conductor failure during an earthquake is potentially possible if there is a massive structural platform failure or an earthquake induced foundation failure. Potential consequences from such a failure would be a blowout and/or pollution from an oil spill.

The potential of a blowout or an oil spill is mitigated by the installation of surface controlled subsurface safety valves (SCSSV) on all wells that are capable of natural flow. The installation of these valves is mandated on all wells offshore the United States. The reliability of these subsurface safety valves has substantially improved over the past twenty years. Additional improvement in their dependability would further reduce the risk.[12,13,14,15]

Conclusions

The risk of damage from an earthquake to the platform topsides is low, provided:

1. All equipment and critical piping is properly tied down, and,

2. The drilling rig, crane booms, flare booms, etc. are designed for appropriate earthquake loading,

The earthquake safety reassessment of an existing topside facility can be accomplished, in part, by a walk-through of the facility to assure that equipment items and piping are adequately tied to the deck structure.

It is also important to have a preventive maintenance program in place to assure that all safety devices are operative at all times. A training program to advise platform operators what the earthquake hazards are, and what to do when an earthquake hits, would be beneficial to avoid panic.

Research Thoughts

Research needs will be discussed in more detail in the work groups. A few thoughts are listed below. There are undoubtedly many more.

There is a fair amount of literature available on onshore earthquake damage to oil and gas process facilities, i.e., Long Beach 1933, Coalinga 1983, Costa Rica 1991. This information is scattered and it would be helpful to collect this data into a single database.

There is a need for an easier method to develop the floor spectra to calculate the behavior of the so-called slender structures. A subcommittee of API RP 2A briefly investigated one such method developed by Professor Bea. There is a need to further explore the validity of this method.

There have been a number of failures of the earthquake recording instruments installed offshore California and, as a result, there is no data available from the most recent earthquakes. There is a need for a simple, foolproof, instrument that is not subject to being activated by boats bumping into the platform or from drilling rig activity.

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WORKING GROUP CONCLUSIONS AND RECOMMENDATIONS

Site Seismic Hazard and Ground Motion	Paul Somerville and Allin Cornell
Design, Reassessment and Requalification	Kris Digre and William Ibbs
Structural Performance	Hugh Bannon and Joseph Penzien
Operations	Michael Craig and David Hopper
Public Policy	Richard McCarthy and Sylvia Earle

Site Seismic Hazard and Ground Motion

Paul Somerville and Allin Cornell, Co-Chairs

1. The State Of Practice

In this section we summarize key aspects of the state of current practice of seismic hazard assessment for engineered facilities with a focus on offshore structures. The keynote lecture by C. B. Crouse at this workshop can be read for more detail and references.

Seismic hazard assessment in offshore practice today invariably includes a probabilistic seismic hazard analysis because modern criteria (e.g., the API RP2A) demand ground motion estimates consistent with specified mean return periods. These probabilistic studies may, however, be supplemented by "deterministic" estimates of ground motions associated with "maximum credible events". We focus here on the former approach; input for it is more than sufficient to permit specification of a deterministic design event, if one is desired.

A probabilistic seismic hazard analysis requires the specification of (1) potential seismic sources (e.g., faults or broad areal zones of presumed uniform seismicity), (2) magnitude recurrence characterization for each source (i.e., the magnitude-frequency relationship together with a temporal stochastic model), and (3) ground motion prediction procedures for given magnitudes and distances. In practice today in active seismic regions such as Southern California the sources are generally characterized by the major neighboring faults supplemented perhaps by areal zones. Faults offshore may not be as well identified and located as those onshore, at least in the open scientific literature. The magnitude recurrence models are generally the truncated exponential magnitude frequency distribution (i.e., the Gutenberg-Richter law with specified mean rate, a, and "slope" b), while the temporal model is usually the basic Poisson model. The parameters a, b, and upper bound magnitude, m1, are estimated from historical seismicity and/or long-term average fault slip rate, as well as fault area. This data may be more limited offshore because of the reduced access. Finally, general practice uses one or more of the published semi-empirical ground motion "attenuation laws" to predict the peak ground acceleration (PGA) and the spectral acceleration (PSA) for a suite of structural periods. These are based primarily on regression analyses using empirical strong ground motion observations. It is only recently that significant attention has been focused on periods in excess of 1 second.

2. The State Of The Art And Its Relationship To Practice

Source and Recurrence Representation. Advanced practice or the state of the art in source characterization includes site-specific studies especially of nearby faults. These may include trenching of faults onshore and analysis of deep geophysical data, where available. Surface geologic data are the common source of onshore fault information. Deep geophysical information is more likely to be available to the hazard analyst in cases where the client has conducted exploration for hydrocarbons. This deep data can be as, or more, useful than the surface data to the extent that they provide more direct imaging of seismic sources at depth. The state of the art exploits this benefit of working in the oil production environment. Given the limited seismicity and surface geological information, this deep geophysical data is particularly critical to offshore fault characterization.

State-of-the-art recurrence models include consideration of characteristic magnitude frequency relationships, especially where historical seismicity rates and those inferred from longer-term paleo-seismic or geological information would otherwise conflict. In addition, one may consider non-Poissonian ("time-dependent") temporal models, especially where paleoseismic data may permit fault-specific estimation of the degree of deviation from Poissonian behavior.

Ground Motion Prediction. The state of the art supplements empirical ground motion models by numerical ground motion modeling. This information is particularly helpful for situations where empirical data is missing or sparse, e.g., very close to major faults, especially at longer periods (≥ 1 second), and at offshore sites. Local soil conditions and their effects on the motions are (in state-of-the-art studies) given site-specific study using nonlinear dynamic analyses; the state of practice may often be limited to the use of generic empirical adjustments to the ground motions.

Sensitivity and Uncertainty Analysis. Where the state of practice is often limited to making simply "best estimates" of each input parameter value, the state of the art includes sensitivity analysis and perhaps formal uncertainty analysis to reflect in the results the impact of the limitations of data and understanding about the sources and the ground motion. These approaches admit multiple hypotheses about, for example, fault dip and faulting style, and ranges of values of parameters such as maximum magnitudes and slip rates. In major state of the art studies these ranges may be provided by two or more independent scientists, and their inputs aggregated. These multiple inputs reflect legitimate variations in the interpretation of the limited existing data that cannot be uniquely resolved at this time. The resulting uncertainty in the output reflects the current, imperfect state of professional knowledge. This situation is not unique to seismic engineering; it may be, however, that the state of the art is more advanced in the seismic area in its attempts at characterizing quantitatively this uncertainty. In the state of practice that provides only single models and parameter values, this range is not displayed, except, often, by the observation that neighboring sites analyzed by different firms have different hazard and design value estimates.

3. How To Improve The State Of Practice And The State Of The Art

Both common and advanced practice offshore can be improved by research. The transfer of technology from one to the other can upgrade the state of conventional practice.

A. Research

Source Characterization. The characterization of blind thrust faults as potential seismic sources is an important issue both onshore and offshore. The deep geophysical data that are usually available offshore should be used to investigate the characteristics of blind thrust faults and assess their relationship to and importance compared to surface faults. Research based on analysis of deep geophysical data in offshore regions has the potential to enhance our characterization of blind thrust faults both offshore and onshore. Deep geophysical data from offshore regions should be used to evaluate slip rates of offshore faults. By establishing the partition between the offshore and onshore components of crustal strain rates, especially near plate margins, this research can benefit seismic hazard analysis in both the offshore and onshore regions.

Earthquake Recurrence. Research is needed in order to understand the relationship between earthquake recurrence estimates derived from geodesy, historical seismicity, paleoseismology, and fault slip rates. The former two categories of information provide short-term estimates, while the latter two provide long term estimates. Geodetic data provide information on total strain rate; historical seismicity data provide information on seismic strain rate and earthquake recurrence; paleoseismology data provide information on earthquake recurrence rate; and fault slip rate data provides information on fault strain rate. This research may shed light on the partition of total strain rate into seismic and aseismic components.

Ground Motions. Ground motion recordings offshore on the sea floor and downhole in the free field are required for the objectives of confirming onshore derived models of depth variation of horizontal ground motions in soft sediments, and of investigating the effect of the water column on vertical ground motions. Ground motion attenuation relations should be developed for long period ground motions that include basin effects. Numerical modeling of ground motions is needed to extrapolate recorded data and perform parametric studies.

B. Technology Transfer to Upgrade the State of Practice

Source and Recurrence Characterization. Deep geophysical data should be routinely used for the assessment of source geometry, sense of slip, and slip rate both offshore and on. Folds should be identified as potential seismic sources (in underlying blind thrust faults). A regional perspective should be used to improve confidence in the characterization of the local (hazard dominating) fault geometry and slip rates. Characteristic magnitude earthquake recurrence relationships that deviate from conventional relationships near the maximum magnitude should be considered.

Ground Motion Prediction. Numerical ground motion models should be used to improve predictions, especially for periods of one second and longer to address near-fault, basin and other effects. The vertical/horizontal ground motion ratio should be site-specific to address near-fault and high frequency conditions. Although the issue of the reference location of the ground motion that is provided is a solved problem in technical terms, in practice there needs to be interaction between seismologists, geotechnical engineers and structural engineers to ensure consistency.

Standard of Practice. Information about the parametric sensitivities and uncertainty bands in hazard estimates should be transferred from the more advanced state of the art studies to the state of the practice, so that at least some level of information is provided about uncertainties inherent in ground motion hazard estimates.

4. Highest Priority Actions

The working group identified several research and technology actions judged to be the most urgent.

1. Research: Record and analyze offshore ocean bottom and downhole ground motions, to understand vertical motions and the vertical variation of horizontal motions.

2. Research: Improve the methods of predicting ground motions in the period range of 1 to 4 seconds using empirical and numerical modeling methods.

3. Technology Transfer: Conduct a short course or workshop jointly for engineers and seismologists on selection of the reference point for ground motions. This should include the issues and solutions for various types of sites and structures.

4. Technology Transfer: Encourage the use of deep geophysical information both on an in-house basis and on a joint industry cooperative basis.

DESIGN, REASSESSMENT AND REQUALIFICATION

Kris Digre and William Ibbs, Co-Chairs

INTRODUCTION

Copies of the discussion paper were distributed (see attached). The objectives of the work group were reviewed. They were define the State-of-the-Art & State-of-Practice for:

* Seismic Design
* Assessment
* Requalification

and recommended future research, development or technology transfer (Fig. 2). Viewgraphs taken from the discussion paper and used during the work group sessions are numbered, referenced herein and attached.

An overview of the work group's scope for the next four sessions was conducted. Seismic Design (Figs. 3,4) was reviewed noting that API RP 2A 19th edition is the current basis with additional topside considerations being addressed in the 20th edition due out in 1993. Seismic Reassessment was introduced noting Information Gathering (Fig. 4), Structural Capacity Unknowns (Fig. 5) and Mitigation Alternatives (Fig. 6). Seismic Requalification was brought up and would be covered in the third session. The distinction was made between assessment (internal, company-directed) and requalification (external, regulatory-managed). Assessment is more than just technical in nature.

SEISMIC DESIGN DISCUSSION

It was noted that current design rules are for new structures. Discussion started with SLE and DLE return periods (Fig. 2). No clear statement exists on how to set or constrain the actual values. Should the return period be tied to the remaining design/operating life of the platform? The strength level return period should somehow tie to exposure risk. If the DLE return period is tied to platform life the traditional 5X rule is very wrong. The consequences of failure need to be considered. Life risk on an annual occurrence versus lifetime should be addressed. API RP 2A does not link the SLE (200 year return period) with the DLE (rare event) and DLE analysis is not always required. A minimum factor of 5 or 1000 years for the DLE may be appropriate.

In design of new structures the SLE design is required. It is recognized that for requalification a SLE analysis is not required, only the DLE. For some high consequence structures the owner will want to meet SLE requirements to protect his investment.

For the design of new structures:

Specify minimum return period tied to safety consequences and uncertainties for both SLE and DLE events i.e. Return Period = f(consequences/uncertainties).

Peer review of seismic hazard analysis was discussed. How to choose the "peer" is unclear. What makes for a good peer review? What's the timing of the peer review? This seems a good idea but hard to follow through on a consistent basis. The API has Response Spectra defined without site hazard analysis.

Damping levels were discussed. The SLE event value of 5% is considered quite acceptable. The DLE event value could be higher but should be justified on hard data.

SEISMIC ASSESSMENT

The Seismic Assessment Process was defined as follows:

- Information Gathering (Fig. 4)
- Structural Capacity Determination (Fig. 5)
  - Unknowns (including loading)
  - What's different from design
- Mitigation Alternatives (Fig. 6)

Reassessment is the iterative process of evaluating the different alternatives in obtaining a satisfactory solution leading up to Requalification.

Discussion was held on economic consequences. Are these the purview of the operating company only? Or, does the government have an interest here? There are two types of cost; the cost to operate and maintain the platform, and the cost of lost resources, life safety and environmental concerns. The second type is of concern to the government or regulator.

Information Gathering(Fig. 4) should also include:

History of Loading
Operator
Designer
Instrumentation – monitoring of platforms may preclude requalification by showing that loading was actually less than designed for.

Discussion also indicated that underwater surveys for damage should be API Level III.

Structural Capacity Determination (Fig. 5) received a large amount of discussion. There is a need for development of realistic joint capacity assessment, grouted and ungrouted (development item) including evaluation of joints in a frame versus individual test data. Concerns regarding the brittleness of older steels was expressed in addition to adequacy of old soil boring information. Conservatisms from design practices which are "codified" into existing computer programs need to be removed for the assessment process. We are interested in actual capacity. Attention to deformation capacity in addition to load capacity need to be made.

Mean loading effects were addressed - what is the difference between static and dynamic loading effects? Considerations should be give to the water column effects. Altered equipment arrangements should be identified as this directly effects the topside mass distribution. Offshore structures should tie research/data to that from other industries. Further information should be developed for added mass especially for members cutting the water surface, slender members and what is the added mass of marine growth? How to determine or measure marine growth in the field was discussed with industry experience indicating that when a 4" wide tape is used to strap a member and the thickness of the growth is back calculated from the measured circumference, values of only 30% for the normal measurements are obtained. There can be excessive conservatism when using "standard" techniques.

Mitigation Alternatives(Fig. 6) were reviewed. Emphasis is placed on the most cost effective mitigations which involve the reduction of loading or consequence of failure. Items to add to the list provided include the removal of unneeded appurtenances such as: boat landings, barge fenders, wells/conductors/risers, etc. Soil improvement may be made through stabilization techniques and grout injection. Underwater welding whether performed wet (watch out for brittleness) or in a dry habitat should not be excluded for m the mitigation possibilities even though it is expensive.

SEISMIC REQUALIFICATION

Seismic Requalification was a topic of much lively discussion. A synopsis of the API sponsored report titled "Seismic Safety Requalification of Offshore Platforms" (Figs. 7–12) was presented and received active participation from most work group members. Seismic life safety hazard is an area of disagreement, needs further research, industry consensus building and technology transfer. Extensive discussion was centered on whether offshore platform risk should be set comparably to the seismic risk of onshore facilities. It is noted that for existing buildings, it is accepted policy to requalify at a level of 75% of that required for new designs. This equates to a return period of around 50% of design for requalifying an existing building.

Peer review was supported. The annual probability of failure <0.001 tying to life safety and the separation of life safety and environmental hazards lead to discussion regarding the marginal utility of people killed versus barrels of oil spilled. Some participants believe that these hazards should be separate, others think that they should be combined. We did agree that we should take out the hard numbers for the explicit recommendations: 5 people ⇒ manned; 2000+ bbls. ⇒ catastrophic spill. However, somewhere these categories have to be defined maybe as low, medium and high. The need for notional graphs tying different levels of return periods with the stated consequence levels is recognized. Can these be developed from other fields?

Discussion highlighted the fact that the probability of structural failure allowed for in the API funded study is a median probability of failure. In addition all engineers and regulators should be aware of the "softness" in the numbers, e.g. 0.001. There is always room for interpretations.

CONCLUSIONS:

Items where Work Group agreement was reached are followed with (A), those where the Work Group could not agree are followed by (D).

SEISMIC DESIGN PROCESS

State-Of-Practice = API RP 2A (A)

- Topsides covered by the 20th edition. (A)
- Structural peer review through CVA process. (A)
- SLE return period is not defined. (A)
- DLE is not required for some cases, but usually performed due to $ savings. (A)

Recommendations for Development:

SLE Be more specific in requirements. (A)
E.G. tie return period to consequences or to uncertainty or require a minimum of meeting API design spectra. (D)

DLE Should be required always. (A)
Develop basis (see SLE). (D)

SEISMIC ASSESSMENT PROCESS

Assessment is different than design. We want to remove conservatism in the assessment process. This is a constant operator process based on the individual operator's economics. As regulatory targets do not exist, these assessments become the initial basis for requalification. (A)

State-Of-Practice (A)

Similar to design but involves:

Information Gathering
- -History of Loading.
- -Underwater Survey
- -Actual Loading and Condition

Determine Present Capacity
- -Model actual structure removing conservatism
- -Unknowns exist in joint and member capacities plus loading

Potential Mitigations are Known
- -Cycling through process is reassessment

Recommendations: (A)

Provide acceleration monitors at each platform.
- -Take care in location.
- -Make sure they are passive.
- -Very cost effective, removing uncertainty in loading experience.

Develop and/or share Technology for:
- -Capacity information
  - Joints (including in frames)
  - Members (including in frames)
- -Loadings/Loading Effects
  - What Cm should be used?
  - How should marine growth be addressed?
  - Static versus dynamic loading

SEISMIC REQUALIFICATION PROCESS

State-Of-Practice varies with each case. There is no common basis. (A)

State-Of-Art is meeting some minimum return period, i.e. demonstrating survival for the return period. Existing buildings use 75% of new design requirements which is similar to using a return period of 50% of new design. (A)

The majority of the Work Group agreed that there should be separation of life safety, environmental consequences and economic decisions but consensus could not be obtained.

Use the API funded independent study as a go-by? (D)

Engineers are not calculating probability of failures but simple yes or no on survival for a given event return period. This is a ductility analysis not a risk analysis. (A)

A SLE event analysis is not required for a requalification. (A)

Peer review is supported. Structural review can be covered using a process similar to the MMS's CVA process. Seismic hazard review need to be performed by "qualified" personnel. How to qualify is not known. (A)

Recommendations:

Develop:

-Manned - unmanned definitions (A)
-Catastrophic pollution definition (A)

-Means to allow engineering judgment if one number (return period) is targeted. Consider: (D)

-Variation in accuracy of seismic return period (D)
-Consequence of failure (D)
-Remaining field life in determining return period(D)

-OR use a notional approach with a matrix: (D)

R.P. = X-years (200?) for Low consequence of failure (D)
R.P. = Y-years (1000?) for Medium consequence of failure (D)
R.P. = Z-years (5000?) for High consequence of failure (D)

The following is a table of priorities for future direction of efforts regarding the Requalification process with 1 = top priority and 4 = low priority:

	RESEARCH	DEVELOPMENT	CONSENSUS FORMING	TECHNOLOGY TRANSFER
REQUALIFICATION	3	1	2	4
ENVIRONMENTAL QUESTIONS	1	2	3	4
PROB. FAIL OR SURVIVAL	4	3	1	2
LIFE SAFETY QUESTIONS	1	2	3	—
PEER REVIEW	—	1	2	3

WORK GROUP ON SEISMIC DESIGN, REASSESSMENT AND REQUALIFICATION OF OFFSHORE STRUCTURES

OBJECTIVES:

DEFINE STATE-OF-THE-ART:

SEISMIC DESIGN

ASSESSMENT

REQUALIFICATION

RECOMMEND FUTURE RESEARCH

WIND, WAVE, CURRENT AND ICE LOADING NOT ADDRESSED

Figure 1

SEISMIC DESIGN

API RP 2A 19TH EDITION

Strength Level Event (SLE) Design - Return Period?

Ductility Level Event (DLE) Design - Return Period?

Time History Analysis is allowed:

Use average of the maximum values

A minimum of three analyses is required.

Response Spectrum Analysis is allowed:

Complete quadratic combination (CQC) of modal loading allowed.

Square root of the sum of the squares (SRSS) is allowed for directional responses.

Should the Naval Research Laboratory method (NRL) of combining modes be used?

Uniform modal damping of five percent should be used for elastic analyses. Is this a conservative assumption? Too conservative?

Figure 2

SEISMIC DESIGN

Allowable Stress Design (ASD) and LRFD allow stresses to reach yield for SLE design.

Major joints should be made with ductile steel and be designed to resist the full tensile yield capacity of any member framing into them.

Additional topside considerations are addressed in the 20th edition of API RP2A, due out in 1993.

How should Marine Growth mass be modeled?

Inelastic analysis is allowed for DLE.

Figure 3

SEISMIC REASSESSMENT

Information Gathering:

- Design drawings
- Design data/criteria
- Soil data
- History of modifications
- Present equipment and loading
- Present list of appurtenances and wells
- Maintenance logs
- Condition of cathodic Protection (CP) system etc.
- Underwater surveys for damage or corrosion – what to look for, etc.
- Etc. (age, location.....)

Figure 4

SEISMIC REASSESSMENT

Structural capacity unknowns for both SLE and DLE assessments:

- Acceptable modeling techniques - elastic and inelastic.
- Realistic member capacity assessment (undamaged).
- Realistic joint capacity assessment (grouted and ungrouted).
- Use mean steel yield strengths instead of nominal, or increase further for strain rate loading effects?
- Evaluation of strength of damaged members.
- Evaluation of capacity of damaged joints.
- More realistic 'K' factors for diagonal and horizontal bracing.
- The removal of conservative computer program defaults.
- Factors to account for increases in soil strength with age.

Figure 5

SEISMIC REASSESSMENT

Mitigation alternatives:

- Remove marine growth
- Remove deck mass - lowers load and changes natural period of the structure.
- Foundation improvements (provide insert piles).
- Reduce consequence of failure (de-man, plug wells, remove equipment, reroute pipelines, etc.).
- Repair damaged members (grout or clamp).
- Provide additional bracing (outrigger piling)
- Strengthen joints (underwater welding - grouting)
- Consider remaining life and potential exposure i.e. reduce criteria.
- Provide more frequent underwater surveys to monitor crack growth

Figure 6

SEISMIC REQUALIFICATION

The panel preparing the report on seismic safety requalification of offshore platforms findings include:

- Seismic requalification should focus on limiting to an acceptable level the risk due to catastrophic impacts of earthquakes.

  "A catastrophic impact is one that has unacceptably large life and/or environmental safety consequences."

- Life safety and environmental safety risks can be treated separately.

- Seismic life safety hazards posed by a requalified platform should be in the same order of magnitude as those of a well-designed onshore conventional building.

- Seismic environmental hazards for a requalified platform should be no greater than those posed by other major offshore petroleum release sources.

- Rigorous site hazard and engineering behavior analyses, more rigorous than those performed for onshore, should be performed to meet stated goals.

Figure 7

SEISMIC REQUALIFICATION

- Using the API RP2A 19th edition for design will produce a structure with adequate life safety provided:
  - "The seismic hazard is determined in accordance with strength and ductility levels set at 200 and 1000 year return periods respectively."
  - "Independent peer review of hazard analysis and structural analysis are performed."
  - "Ductility level analysis is made."
  - "Proper allowance is made for the life safety risks associated with platform appurtenances."

Figure 8

SEISMIC REQUALIFICATION

The Panel's recommendations for minimum life safety performance include:

- Either the collapse probability in an earthquake should not exceed 0.001 per year or the structure should meet the requirements of API RP 2A, 19th edition with the additions noted above.
- The ground motions must be determined from the median results of a probabilistic seismic hazard analysis for the site considering all sources of uncertainty, and must also undergo peer review.
- Appurtenances must meet the 1988 Uniform Building Code (UBC) requirements until equivalent or stricter requirements are incorporated into the API RP2A.

Figure 9

SEISMIC REQUALIFICATION

The Panel's recommendations for minimum environmental safety performance include:

- No probability limits for spills when considered separately and each source of the spill's size is limited to 2,000 barrels.
- "If the size of a potential spill exceeds 2,000 barrels, then the calculated probability of its occurrence should be less than or equal to 0.001 per year. This probability is generally associated with platform collapse."
- The requirements of API RP 14B should be met for flow shut-off devices including regular testing, inspection and maintenance.

Figure 10

SEISMIC REQUALIFICATION

The Panel's recommendations for requalification include:

- If the platform (as is or with modifications) meets the new platform seismic requirements of the 19th edition of API RP 2A (with noted modifications), there should be no further requirements.

- If the platforms does not meet the modified API RP 2A requirements:

    - The likelihood that the platform poses an environmental hazard larger than the prescribed level of 2,000 barrels for each source should be no more than an annual threshold probability of 0.001.

    - The annual probability that the platform poses a life safety risk must be no more than 0.001.

- When both the environmental and life safety performance objectives have been met, a quality assurance program should be implemented.

  •"Independent peer review of the seismic hazard analysis and structural analysis must be provided throughout the requalification process."

Figure 11

SEISMIC REQUALIFICATION

Other important issues discussed by the Panel:

- Their recommendations do not include any consideration of the economic costs (beneficial or not) of seismic retrofitting.
- The probability of structural failure allowed is a median probability of failure and can be approximated by determining the probability that the demand exceeds the median capacity of the platform.
- If a structure is unmanned, the life safety issue is satisfied. The Panel has replaced the API's definition of a manned platform with: "a manned platform is one which is actually and continuously occupied by at least 5 persons."

Figure 12

APPENDIX I

WORKING GROUP ON SEISMIC DESIGN AND REASSESSMENT OF OFFSHORE STRUCTURES

12/7/92

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APPENDIX II

WORKING GROUP ON SEISMIC DESIGN AND REASSESSMENT OF OFFSHORE STRUCTURES

AN INTERNATIONAL WORKSHOP ON

SEISMIC DESIGN AND REASSESSMENT

OF OFFSHORE STRUCTURES

DECEMBER 7-9, 1992

California Institute of Technology
Pasadena, California, USA

WORK GROUP ON SEISMIC DESIGN,

REASSESSMENT AND REQUALIFICATION

OF OFFSHORE STRUCTURES

CO-CHAIRMEN

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WORK GROUP ON SEISMIC DESIGN, REASSESSMENT AND REQUALIFICATION OF OFFSHORE STRUCTURES

INTRODUCTION

This Work Group has the objectives of defining the state-of-art of seismic design, assessment and requalification and recommending future research in this area. The topics of other environmental loading such as wind, wave, current or ice will not be addressed in this workshop other than reminding those involved that for many cases, those loadings may be more critical than the seismic loading. The following paper addresses and outlines for review some of the specific issues involved. The intent is to highlight topics for Work Group discussion starting points with the goal of reaching agreement on state-of-the-art and determining appropriate research needs.

SEISMIC DESIGN

Design guidelines for the seismic design of offshore steel structures have been developed and updated during the past 20 years. The current guidelines are presented in the American Petroleum Institute (API) RP2A, nineteenth edition, August 1,1991 - Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms. The API has written a similar guideline (now in draft format) to be published as a first edition in April 1993, API RP2A-LRFD (Load and Resistance Factor Design method). These guidelines represent state-of-the-art and include several specific provisions:

Strength Level Event (SLE) Design:

The SLE requirements are intended to provide a platform which is adequately sized for strength and stiffness to ensure no significant structural damage for the level of earthquake shaking which has a reasonable likelihood of not being exceeded during the life of the structure. A return period of 200 years for southern California is noted.

Ductility Level Event (DLE) Design:

The DLE requirements are intended to ensure that the platform has sufficient reserve capacity to prevent its collapse during rare intense earthquake motions, although structural damage may occur.

Time History Analysis is allowed:

Use the average of the maximum values for each of the Time Histories considered.

A minimum of three Time History analyses is required for DLE.

Response Spectrum Analysis is allowed:

Complete quadratic combination (CQC) of modal loading allowed.

Square root of the sum of the squares (SRSS) is allowed for directional responses.

Should the Naval Research Laboratory method (NRL) of combining modes be used?

Uniform modal damping of five percent should be used for elastic analyses. Is this a conservative assumption? Too conservative?

Allowable Stress Design (ASD) and LRFD allow stresses to reach yield for SLE design. Is there really any difference between the methods for seismic design?

Major joints should be made with ductile steel and be designed to resist the full tensile yield capacity of any member framing into them.

Additional topside considerations are addressed in the 20th edition of API RP2A, due out in 1993.

How should Marine Growth mass be modeled?

Inelastic analysis is allowed for DLE.

SEISMIC REASSESSMENT

There are no current guidelines on seismic reassessment of offshore platforms. The API has formed a Task Group (TG) on Assessment of Existing Platforms to Demonstrate Fitness for Purpose. One of their goals is to provide guidelines on this topic. They have already identified several items which affect the assessment process for evaluating offshore structure capacity:

Information Gathering:

- Design drawings
- Design data/criteria
- Soil data
- History of modifications
- Present equipment and loading
- Present list of appurtenances and wells
- Maintenance logs
- Condition of cathodic Protection (CP) system

- Underwater surveys for damage or corrosion - what to look for, etc.
- Etc. (age, location.....)

Structural capacity unknowns for both SLE and DLE assessments:

- Acceptable modeling techniques - elastic and inelastic.
- Realistic member capacity assessment (undamaged).
- Realistic joint capacity assessment (grouted and ungrouted).
- Use mean steel yield strengths instead of nominal, or increase further for strain rate loading effects?
- Evaluation of strength of damaged members.
- Evaluation of capacity of damaged joints.
- More realistic 'K' factors for diagonal and horizontal bracing.
- The removal of conservative computer program defaults.
- Factors to account for increases in soil strength with age.

Mitigation alternatives:

- Remove marine growth
- Remove deck mass - lowers load and changes natural period of the structure.
- Foundation improvements (provide insert piles).
- Reduce consequence of failure (de-man, plug wells, remove equipment, reroute pipelines, etc.).
- Repair damaged members (grout or clamp).
- Provide additional bracing (outrigger piling)
- Strengthen joints (underwater welding - grouting)
- Consider remaining life and potential exposure i.e. reduce criteria.
- Provide more frequent underwater surveys to monitor crack growth

SEISMIC REQUALIFICATION

At the present time, there are no standards, regulations or requirements which set forth the criteria for seismic requalification. The API funded an independent study titled "Seismic Safety Requalification of Offshore Platforms", issued in May 1992. The panel that prepared the report consisted of Wilfred D. Iwan, Chairman; Charles C. Thiel, Jr.; George Housner; and Allin C. Cornell. These panel members were selected because of their preeminence in the field of earthquake engineering and experience in establishing practical regulatory guidelines. They have been involved in recommendations for both design and requalification procedures for bridges, buildings and other on-land industrial structures.

The intent of the API funded study was to investigate how to establish a basis for requalification of offshore platforms that is consistent with land-based practices. The API has issued a report on the study. The API TG on Assessment of Existing Platforms to Determine Fitness for Purpose is currently evaluating the study's recommendations for inclusion in whole or in part in their draft recommended practices.

While there are no formal standards for requalification, offshore structures have been requalified for seismic loading (California state waters, Alaska, New Zealand) using sound engineering judgment and techniques. Inherent in the process is evaluating the platforms for site specific loadings, removing conservatism in the standard design process, considering appropriate mitigations and trying to meet present day design criteria. With appropriate mitigation, the criteria have been met in some cases. What is missing is recognition that present design criteria may be too restrictive for some existing structures.

The panel preparing the report on seismic safety requalification of offshore platforms has addressed these issues. Their findings include:

- Seismic requalification should focus on limiting to an acceptable level the risk due to catastrophic impacts of earthquakes.

  "A catastrophic impact is one that has unacceptably large life and/or environmental safety consequences."

- Life safety and environmental safety risks can be treated separately.

- Seismic life safety hazards posed by a requalified platform should be in the same order of magnitude as those of a well-designed onshore conventional building.

- Seismic environmental hazards for a requalified platform should be no greater than those posed by other major offshore petroleum release sources.

- Rigorous site hazard and engineering behavior analyses, more rigorous than those performed for onshore, should be performed to meet stated goals.

- Using the API RP2A 19th edition for design will produce a structure with adequate life safety provided:

  - "The seismic hazard is determined in accordance with strength and ductility levels set at 200 and 1000 year return periods respectively."

  - "Independent peer review of hazard analysis and structural analysis are performed."

  - "Ductility level analysis is made."

  - "Proper allowance is made for the life safety risks associated with platform appurtenances."

The Panel's recommendations for minimum life safety performance include:

- Either the collapse probability in an earthquake should not exceed 0.001 per year or the structure should meet the requirements of API RP 2A, 19th edition with the additions noted above.

- The ground motions must be determined from the median results of a probabilistic seismic hazard analysis for the site considering all sources of uncertainty, and must also undergo peer review.

- Appurtenances must meet the 1988 Uniform Building Code (UBC) requirements until equivalent or stricter requirements are incorporated into the API RP2A.

The Panel's recommendations for minimum environmental safety performance include:

- No probability limits for spills when considered separately and each source of the spill's size is limited to 2,000 barrels.

- "If the size of a potential spill exceeds 2,000 barrels, then the calculated probability of its occurrence should be less than or equal to 0.001 per year. This probability is generally associated with platform collapse."

- The requirements of API RP 14B should be met for flow shut-off devices including regular testing, inspection and maintenance.

The Panel's recommendations for requalification include:

- If the platform (as is or with modifications) meets the new platform seismic requirements of the 19th edition of API RP 2A (with noted modifications), there should be no further requirements.

- If the platforms does not meet the modified API RP 2A requirements:

  - The likelihood that the platform poses an environmental hazard larger than the prescribed level of 2,000 barrels for each source should be no more than an annual threshold probability of 0.001.

  - The annual probability that the platform poses a life safety risk must be no more than 0.001.

- When both the environmental and life safety performance objectives have been met, a quality assurance program should be implemented.

- "Independent peer review of the seismic hazard analysis and structural analysis must be provided throughout the requalification process."

Other important issues discussed by the Panel:

- Their recommendations do not include any consideration of the economic costs (beneficial or not) of seismic retrofitting. The Panel's professional opinion is that an operator should consider explicit cost-benefit-risk analyses for all loading conditions any that these analyses may well indicate it is beneficial to design for a lower level of seismic safety risks than permitted under the Panel's recommendations.

- The probability of structural failure allowed is a median probability of failure and can be approximated by determining the probability that the demand exceeds the median capacity of the platform. This is equivalent to determining the probability of failure from the median seismic hazard curve at the best estimate or "unbiased estimate" of the capacity. The unbiased capacity estimate must be made with care to remove from the structural model all recognized and unrecognized "safety factors" or other assumptions that represent conservative rather than best estimate behavioral properties of the platform's materials and systems.

- If a structure is unmanned, the life safety issue is satisfied. The Panel has replaced the API's definition of a manned platform with: "a manned platform is one which is actually and continuously occupied by at least 5 persons." Thus whether a platform is manned or not depends strictly on whether 5 or more workers are expected to be on the platform at all times of the 24 hour work day. "Occupancy for a short period by more than 5 at all times of the 24 hour work day need not force the platform to be manned; this may occur for such operations as extended maintenance of a limited drilling program."

REFERENCES:

API (RP2A), Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms, API RP2A, 19th Edition, American Petroleum Institute, 1991.

API (RP2A-LRFD), Recommended Practice for Planning Designing and Constructing Fixed Offshore Platforms - Load and Resistance Factor Design, First Edition, April 1, 1993.

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STRUCTURAL PERFORMANCE

Hugh Bannon and Joseph Penzien, Co-Chairs

The discussions in this working group were focused on issues related to seismic reassessment of steel pile jackets (SPJs) and, because of time limitations, the working group participants decided to exclude other types of platforms such as gravity-based structures. Although the participants discussed a few issues exclusively related to new designs, there was consensus that the subject of reassessment deserves more attention than topics related to new designs. The group noted that while the cost of offshore hazard mitigation and repair for existing platforms may be very high, there is a lot more flexibility in a new design where the designer can select a more conservative alternative without seriously impacting the cost.

The discussions in this working group were divided into four general topics: 1) modeling of structures and foundations, 2) dynamic analysis methods and procedures, 3) development of seismic criteria, and 4) laboratory and field measurements. The following sections summarize these discussions and present specific conclusions related thereto. Within the time limitations allotted to each topic, the working group's goals were to highlight the sources of uncertainty in our present procedures, define the important issues and challenges presently facing practicing engineers, and propose research studies for advancing the technology. These discussions covered both the state-of-practice as well as the state-of-the-art; the two could be comparable in some areas while being quite different in other areas. This was an opportunity for the working group participants to compare their assumptions, methods, and procedures, and identify weaknesses in the existing technology for seismic reassessment of platforms. The participants then proceeded to categorize the research needs into: 1) areas where short-term research studies can produce significant improvements, and 2) those that require long-term research plans and solutions. The following is a brief description of both categories, with the emphasis being on topics that the working group has judged to have the most impact on a platform reassessment.

1. Modeling of Structures

1.1 Modeling of Joints

One of the areas of highest uncertainty in reassessment of structures is modeling and analysis of joints, e.g., modeling cyclic behavior of joints and evaluating their seismic performance. The state-of-practice for modeling and analysis of joints in offshore platforms is as follows: 1) Joint flexibility is typically ignored in developing the global stiffness matrix, and 2) resistance of a joint is checked against codified equations such as those proposed by API RP 2A, API LRFD (Load and Resistance Factor Design), or the UK Health and Safety Executive (HSE).

The working group participants felt that the present practice of using code design equations (API or HSE) for calculating resistance of joints may not adequate. The codes specify lower bound equations that have been developed from experimental results; the tests are invariably those of monotonic loading. The code equations are adequate for new platform designs subjected to wave forces, but their application to seismic reassessments may not be appropriate. For seismic reassessments, one should account for joint strength degradation due to the cyclic nature of earthquake loads. In addition, using best estimate or mean strength equations may be more appropriate.

The results and conclusions of a platform reassessment may be sensitive to seismic response of joints. For example, consider the case where the resistance of a critical joint is calculated to be close to the resistance of one or more of the members connected to it. If the joint fails first, then it will limit the load being transmitted to the connecting members. This could be beneficial if the joint strength does not degrade with cycles of loading; however, it could be detrimental if the joint exhibits substantial strength degradation.

It is suggested that, in light of limited resources available today, a short-term study be initiated to evaluate the lower bound joint capacity (design) equations proposed by the API and HSE guidelines. The purpose of this evaluation would be to develop a set of best estimate equations for resistance of joints based on available test data. It is recognized, however, that the existing equations are designed to predict static strength and not cyclic strength. Therefore, a long-term objective of the industry research in this area should be to support experiments in order to better define strength degradation and energy absorption capacity of SPJ joints subjected to cyclic loads. Such experiments may then be further expanded to studying cyclic behavior of damaged joints (e.g., joints with dents).

The working group's experience indicates that including joint flexibility in the global stiffness matrix is not expected to significantly affect a platform's global mode shapes or frequencies. This statement, however, is not true for local vibration modes of a platform such as those associated with vibration of conductor panels or x-brace panels. Modeling joint flexibility may have a moderate effect on frequencies of local modes and the balance of axial load and bending moments in some members; this may be a more important issue in prediction of fatigue damage. Unfortunately, joint stiffness, and the degradation in joint stiffness with high amplitude cyclic loads, has not been studied in previous laboratory tests. This is a subject that can be integrated in the proposed long-term laboratory testing program to study cyclic behavior of joints.

In addition to classes of joints in modern offshore platforms that use joint cans, attention should be given to other joint types which are prevalent in older platforms. Many older platforms were built with joints that use gusset plates. The ductility or strength degradation of gusset type joints may be vastly different than those with joint cans. Another area of concern may be details of joints used to connect cluster piles to a platform leg. Due to the uniqueness of joint designs in older platforms, behavior of such joints may have to be studied through platform-specific laboratory experiments.

1.2 Modeling of Members

Nonlinear behavior of tubular members in SPJs has been a focus of industry sponsored research in the last decade. In general, compared to joints, modeling of stiffness, strength, and cyclic behavior of tubular members can be performed with a much higher level of confidence. In addition to experimental results that cover nonlinear cyclic behavior of intact members, there are several monotonic experimental results available that cover a range of common damage scenarios such as dents and bends.

Currently, there are two procedures for modeling nonlinear behavior of intact tubular members. In the first approach, the member is modeled either as a strut (axial load only) or as a beam-column (that includes both axial loads and bending moments). For a strut, axial compression capacity and post buckling behavior is modeled phenomenologically (using the member k-factor as an input). For a beam-column, axial load shedding is ignored and failure of the member under a combination of axial load and bending moments is modeled by an interaction equation. In the second approach, the member is modeled as an element that can develop three plastic hinges (two at the ends and one in the middle). The latter approach can simulate post buckling load shedding as well as the interaction between axial load and bending moments; this approach also eliminates the need for specifying k-factors. Although the latter approach is theoretically more sound, it has to be calibrated against test data. Both of these approaches are deemed to be acceptable and relatively accurate.

In the past, experimental results for damaged tubular members have mainly focused on bends, dents, or combinations of the two. There are several papers in the literature dealing with this subject, and computer programs are available that specifically calculate the ultimate strength of a damaged member (e.g., DENTA, BCDENT). Because of the proprietary nature of test data, the methods for calculating capacity of damaged members are not widely available to practicing engineers. The participants suggested that simple guidelines for modeling and analysis of damaged members can be developed and made available to the profession. This may be accomplished, for example, through an API committee. Another area of concern is behavior of members that have been damaged due to corrosion (e.g., reduced thickness, pits, holes, or their combinations). Currently, there is only one set of small scale test results for members with holes (Chevron) available. It was felt that additional test and/or analytical results covering this topic would be useful.

1.3 Modeling of Foundations

Using nonlinear springs to model lateral (p-y) and axial (t-z) soil resistance represents the state-of-practice for modeling of offshore pile foundations. In a Ductility Level Earthquake (DLE) structural analysis, the pile steel nonlinear behavior is also included by modeling each pile as a series of beam-column elements; the p-y and t-z elements are in turn connected to the ends of beam elements. The participants felt that using standard p-y and t-z curves (which have been calibrated to a large number of small scale and large scale experiments) to model soil resistance is an acceptable approach for assessing capacity of pile foundations. Additional information, however, is needed to confidently use this approach to predict performance of foundations under cyclic loads. There were also several suggestions on how this approach may be modified, both in the short-term and also in the long-term, to better account for actual behavior of piles in sand or clay. The working group recognized that pile capacity prediction methods, in general, are more uncertain than structural resistance evaluations and that further experiments are needed to reduce some of the known uncertainties.

Several research studies have been sponsored by the industry to enhance the present procedures for (wave and) seismic assessment of pile foundations. A recently completed API sponsored study (by Prof. Tang at the University of Illinois) quantified the bias and coefficient of variation in our best estimates of: 1) lateral pile capacities, 2) axial pile capacities, and 3) capacity of pile foundation systems. These uncertainties will be used to develop pile resistance factors (φ's) in the future API LRFD recommended practice. The University of Illinois study also investigated the effects of soil sampling and testing methods and alternative design procedures on the ratio of predicted to measured pile performance. These results may be useful in determining safety indices of existing pile foundations if the original soil data and analysis methods are known. In a recent joint industry project, the contribution of a platform's mudmat to the foundation capacity was found to be significant.

The working group identified two other aspects of soil/pile behavior that need to be studied in the short-term. The first is the combined effects of rapid and cyclic loading; while rate effects in an earthquake may be beneficial, they may be offset by cyclic degradation. Rate effects are known to vary with soil type. They seem to be most pronounced in highly plastic normally consolidated clays and least pronounced in sands. A second issue is the possibility of softening or liquefaction for piles in sandy soil especially in the surface layers where most of the lateral pile resistance is derived, e.g., one may reduce or even neglect the soil resistance in a sand layer if the liquefaction potential exists. Although methods for prediction of liquefaction exist today, these methods seem to be highly uncertain and do not in general apply to local liquefaction around a pile caused by its lateral movement. Simple guidelines are needed to allow practicing engineers to assess the potential for liquefaction in a given sand layer.

There are several other topics related to pile foundations that also require future research, but fall under the category of long-term research objectives. The following is a brief description of these topics:

- New experiments and methods are needed to model loss of strength due to local pore pressure build-up adjacent to a pile.

- We need to develop a better understanding of axial and lateral capacities for piles in a cluster. While we can predict the overall behavior of the cluster, predicting the behavior of individual piles in a cluster is subject to a high degree of uncertainty.

- The effect of soil aging on pile capacities may be beneficial and needs to be studied. This is a secondary consolidation phenomenon which is not captured in our present procedures. A 1993 API sponsored project will study this topic.

- The present procedures assume independence between p-y and t-z resistance of a pile. In reality, the axial capacity of a pile may be reduced due to its lateral motion. Although the effect is presently judged to be unimportant, we need to assess the interaction between lateral and axial pile capacities.

- The plasticity index has been used as an indicator of cyclic response characteristics for onshore foundations in certain soils. Extension of such methods to offshore pile foundations requires further long-term research.

2. Dynamic Analysis Methods and Procedures

The state-of-practice for Strength Level Earthquake (SLE) design (or linear assessment) of offshore platforms is to use the response spectrum approach and verify the results with limited time history analyses. For DLE design check (or nonlinear assessment) of platforms, it is common to use either the static pushover method or the nonlinear time domain method. The working group felt that these methods are generally well established, and that the offshore industry's practice is consistent with those of other industries. The following is a summary of where some of the perceived gaps in our current modeling and analysis procedures exist. All of these topics can be categorized as those that can be studied in the short-term.

- There is a need for new guidelines regarding the level of necessary detail in a platform structural analysis. In most cases, the level of detail has to balanced against our confidence in the model and the earthquake input. Nonetheless, some general guidelines and further enhancements of existing methods and procedures for offshore structures may be useful. Examples of topics that can be covered are: 1) out-of-plane vibration of x-panels, 2) details of deck models, 3) modeling of conductor friction and/or locking of conductors inside of their guides (especially for deepwater jackets).

- For response spectrum analysis of platforms, there is a need to review and possibly modify the existing API guidelines on: 1) combination of load-effects (e.g., axial loads, moments, or displacements) due to the X, Y, and Z components of an earthquake motion, and 2) combination of moments and axial loads in a code check equation. It was suggested that new load combination procedures, which are consistent with probabilistic load combination methods, should be developed for this purpose.

- The static pushover method, although often used for nonlinear seismic assessments, is more applicable to estimating the ultimate resistance of platforms subjected to wave loads. Because of dynamic nature of seismic excitations, there are three drawbacks to the static pushover method: 1) it does not directly address the hysteretic damping in a platform, 2) it neglects the shift in natural frequency of a structure, and 3) it cannot take into account the sequence of seismic loads acting on a structure. Although the static pushover method is generally conservative, guidelines are needed which describe its merits and limitations.

- The nonlinear time history analysis method, although more accurate compared to the static pushover procedure, is often time consuming and expensive. Its major shortcoming, however, is that failure criteria for members, joints, and systems in a time history analysis do not exist. Lack of clearly established failure criteria can often lead to results that are unconservative. There are two examples of possible unconservatism in a time history analysis: 1) it is not clear that monitoring structural stability is a sufficient criterion for predicting failure, and 2) the amount of hysteretic energy dissipated through cyclic behavior is often not limited in a time history analysis. Given that a single time history analysis may not be sufficient, guidelines are also needed that specify the minimum number of earthquake records and their properties. The working group suggested that further research in these areas is needed to modify and enhance our existing methods and procedures for performing time history analyses.

3. Development of Seismic Criteria

Structural engineers, who perform seismic assessments of offshore platforms, are most often not involved in the development of seismic criteria. Seismic criteria (e.g., site specific response spectra or time history records) are commonly developed by seismologists and geotechnical engineers who are not informed about the details of how their work product is used in a platform reassessment. The group felt that proper communication and interaction between structural engineers, who are the end users of seismic criteria, and the seismologists and geotechnical engineers is critical in avoiding possible errors of omission or misapplication of results. For example, it is well established that site-specific spectra and records should account for seismotectonics of the region, type of faults and their maximum magnitude potentials, seismicity of the region and its faults, seismic attenuation, and local site conditions. Structural engineers need to become familiar with details of site specific seismic criteria development to properly apply the results in linear or nonlinear analyses of platforms.

The group indicated that fitting earthquake records to response spectra is an area that requires special attention. This is a task that is performed by either structural engineers, geotechnical engineers, or seismologists. There are three possible alternatives for this purpose: 1) scaling of natural records, 2) using synthetic records, and 3) varying the frequency content of natural records to obtain a better match with the target spectrum. Because of the subjectivity in each of the above alternative procedures (especially in selection of natural records), it was felt that additional guidelines on this topic in API RP 2A (and API LRFD) would be beneficial.

Another area that requires attention is the ratio of vertical to horizontal spectral values. In the past, because of lack of data, this ratio has often assumed to be 50%. It is important to realize that the ratio of vertical to horizontal spectral values is dependent on type of faulting, proximity of fault to a site, the earthquake magnitude, and the frequency of interest. For example, for a large magnitude earthquake caused by thrust faulting close to a site, this ratio could be 100% or more at the high frequency end of the spectrum. Therefore, it was suggested that whenever possible site-specific vertical spectra be developed to avoid making assumptions about the ratio of vertical to horizontal spectra. Finally, the effect of water column on vertical seismic motions is an area that requires further research. The water column may greatly reduce the vertical component of an earthquake motion if it can be established that the vertical motion is dominated by P-waves.

4. Laboratory and Field Measurements

The discussions on this topic centered on measurements from two platforms (offshore Japan and California) which are operated by Exxon. Because of their location in seismically active areas, these platforms have been extensively instrumented to enable the operator to assess the condition of the jacket after an earthquake and make decisions regarding a post event inspection. The instrumentation system allows the operator to estimate natural frequencies and mode shapes (using system identification techniques) from the observed data. For one of these platforms, the maximum recorded earthquake to date is roughly 30% of its design SLE.

Exxon has analyzed the (seismic and wave) data measured from these two platforms and compared them with analytical predictions; their conclusion is that current procedures under-predict modal frequencies of platforms. It was pointed out that this conclusion has been generally supported by other operators who also have measured natural frequencies of their platforms. A second inconsistency that has been observed in Exxon measurements is that the observed (global) mode shapes do not match the analytical mode shapes. These anomalies cannot be attributed to the foundation flexibility alone; for example, the predicted modal frequencies of Exxon platforms were found to be low even if a fixed foundation was used in the computer model. Even after extensive sensitivity studies, no rational explanation for the anomalies has been found. At present, Exxon's best guess regarding the source of modal frequency inconsistency points to platform mass calculations.

The working group suggested that a first step in resolving the differences in measured and predicted modal frequencies and mode shapes would be to disclose the results (which are currently proprietary) to the entire industry. It was felt that by disseminating the information, Exxon and the industry could seek input from other experts and plan a future course of action.

Based on the working group discussion, there is an immediate need for more seismic instrumentation and data gathering systems offshore. This could include: 1) free field measurements that would be used to calibrate our soil structure interaction models, 2) below mudline measurements that would verify and calibrate soil/pile behavior models, and 3) additional deck and jacket instrumentation systems that would be used to resolve the differences in modal frequencies and mode shapes.

OPERATIONS

Michael Craig and David Hopper, Co-Chairs

1. Scope

Some of the issues addressed by the Working Group on Operations include: 1) What do we mean by 'Operations' in the context of this Workshop? 2) What facilities are covered within this scope? 3) What are the dangers from the seismic-induced failure of the different types of facilities covered? 4) How is this danger being presently managed? And 5) What can be done to improve its management?

This brief focuses on the latter two issues. Namely, how is the danger from the seismic performance of poorly tied down or undersized deck equipment presently being handled; and what can be done to improve its management.

2. 'Operations' Defined

The Workshop's operations working group limited its charge to evaluating the way 'topsides equipment' is, and should be, designed and assessed to minimize the risk of failure due to earthquake loading.

'Topsides equipment' was defined to comprise the drilling rig and associated equipment, all process related equipment, safety and control equipment, cranes, quarters, flare booms, storage tanks, pipeline risers and well conductors.

3. Topsides Equipment - Potential Dangers

Inadequately or inappropriately restrained equipment on an offshore platform that experiences a significant earthquake has the potential to cause injury or death to platform personnel, either directly or indirectly, and the potential to damage the environment through the uncontrolled release of oil.

Direct personnel injury or death could result from the structural failure of the equipment; for example, the drilling rig collapses or topples over. Personnel injury or death could occur as an indirect result of the failure of inappropriately restrained pressure retaining systems which then can leak their contents, catch fire, cause explosions, and injure personnel. Environmental pollution could be caused by the rupture of oil storage tanks or oil transmission piping - pipeline risers and well conductor strings.

Because of the potential consequences described above, significant efforts have been employed, and continue to be employed, to prevent these potential hazards from occurring.

4. Topsides Equipment - Present Design & Assessment Practices

Hazards mitigation practice comprises first recognition of the hazard, then proper mitigation design, proper usage and lastly continued maintenance.

Recognition is driven by the above consequence potential. However, because no significant earthquake shaking has been experienced for the entire duration of offshore west coast operations, complacency can be a major obstacle to earthquake hazards recognition and continued consequence mitigation.

Proper design practices are embodied in the recently updated Section 2.3.6e.2 and associated Commentary in the 20th edition of API RP 2A (working stress design and load and resistance factored design versions). API RP 2A is the standard by which offshore oil and gas platforms are designed in earthquake-prone areas.

The sections referenced define design guidelines and practices that, when properly implemented and maintained, will appropriately mitigate the hazards associated with topsides equipment subject to earthquake loading, for both new and existing facilities.

In the case of existing facilities, it is essential that the documented field inspections be performed by experienced personnel. Reanalysis work, if any, should be minimal and focused.

These API design and assessment guidelines (text and commentary) are copied from the upcoming 20th edition (working stress design version) in Appendix I. The reader is encouraged to read these guidelines in detail. Implementation of appropriate restraint, and sizing of structural elements with adequate strength and ductility, are keys to successful topsides equipment hazards mitigation.

The implementation and usage of appropriate restraint is primarily a matter of common sense. Thus, the need for experienced based re-evaluation as opposed to detailed analyses. Restraint implementation and usage is complicated, though, by lack of recognition of the potential hazards, and by changes to deck equipment, including the moving of the rig from well to well. As discussed, complacency is a major obstacle. Continued vigilance is necessary to ensure that identified seismic hazard mitigation measures remain in place.

'Maintenance' of hazard mitigation measures means ensuring that appropriate restraints are not lost nor left to corrode unduly. This can happen when the rig is moved around the drilling deck infrequently and/or stationary equipment is left in the marine air.

Maintenance can also include recognition and prevention of seismic hazards from equipment not originally designed to withstand lateral seismic forces. This is especially the case for the drilling rig which may be a land-based rig designed without seismic considerations. In this case, the mast or derrick and in some cases the substructure must be analyzed and reinforced as necessary.

Another suspect element can be cantilevered framing, typically supporting living quarters or added process equipment. In light of recent developments regarding the perception of vertical ground motions, it may be necessary to re-evaluate the 'bounce' responses of such framing and remediate as necessary.

In areas which experience subfreezing temperatures, the fracture potential of above-water steels and weldments exposed to such temperatures when subject to significant dynamic bounce forces must also be evaluated. Exposed steels and weldments typically include the type of cantilever framing discussed above. In general, platforms in these areas have been and are being designed using exposed steels and consumables with appropriate toughness properties. If remediation is deemed necessary, options include reducing nominal static stresses, heat tracing or material replacement.

5. Development Areas To Improve Topsides Seismic Hazard Mitigation

Below is a list of focused improvements identified by the Operations Working Group. These ideas are more development (common sense) rather than research oriented.

5.1 Assessment Procedures For Existing Equipment

It is suggested that regular topsides structural inspections of earthquake-prone platforms be formalized, through API recommendations on frequency and the development of a practical checklist. The checklist would be an industry consensus document, reflecting post-earthquake experiences from onshore refineries and processing plants.

Follow-up inspections should succeed a baseline inspection at logical intervals dictated by the amount of ongoing changes to the topsides equipment and the quality of 'maintenance'. Inspections should also follow any significant seismic event.

The checklist should reflect a focused work scope, concentrating on experienced-based evaluations of restraint, strength and ductility of critical elements such as primary instrument and control equipment, fire fighting equipment, piping and pressure vessels handling hazardous materials, and escape capsules. It would clearly recognize the potential 'knock-on' effects of fires and explosions from seismically undersized or inadequately braced equipment. It is recommended that such scope and frequencies be integrated into Section 14, Surveys, in API RP 2A–WSD.

A format for such a checklist could be similar in format to the International Association of Drilling Contractors' (IADC) derrick inspection checklist in API Specification 4F. The working group believe that peer review of these inspections is not necessary.

5.2 Cross-referencing 2A With Other API 'Topsides' Documents

It is recommended by the working group that the updated topsides seismic hazards mitigation guidance in Section 2.3.6e.2 of API RP 2A 20th edition (copied in Appendix I) be referenced in other API Recommended Practices and Specifications which deal specifically with the design and sizing of deck equipment - mechanical items such as drilling rigs (4F), piping (14E) and cranes (2C); electrical equipment (14F); as well as topsides process safety guidelines (750, 14G, PSM).

A paragraph referencing this 2A guidance presently exists in Specification 4F; however a statement that the rig should be tied down at all times except during skidding is lacking. The cross-referencing of seismic guidance in documents used by topsides engineers with little or no seismic design experience is of value. There is benefit in this type of multi-disciplined linkage of technical guidance, to parties within and outside the oil companies.

5.3 In-house integration of Seismic Guidelines

Significant safety-related efforts are underway within operating companies where personnel are responding to new process-related OSHA Process Safety Management requirements and are conducting extensive Hazard and Operability (HazOp) studies and other 'management of change' safety reviews.

Integration of structural seismic assessment guidelines into these programs improves the opportunities for structural engineers to aid production and drilling personnel in recognizing seismic structural hazards as one generic part of the total risk picture, in assessing these hazards and in dealing with them appropriately. Too often, seismic structural hazards tend not to be included in process-related safety reviews even when they can comprise a significant portion of the facility's total risk.

5.4 Spill Potential Evaluation

Evolving industry and regulatory platform structural assessment and requalification guidelines may contain threshold limits on oil spill potential or require some classification of the severity of a facility's oil spill potential. Calculating this potential for any given platform is not as straightforward as it may seem.

Potential oil spill quantities from beam tanks (in Cook Inlet tower structures), piping and other storage vessels onboard the platform are not simple to derive, nor in many cases is the potential quantity of oil backflow in pipelines whose risers are sheared. What is clear is that potential oil spill quantities are not the simple sum of the volumes within vessels, tanks, piping added to the volume within the oil pipeline between platform and shore-based shut-off valves. In most cases realistic potential oil spill volumes are a very small fraction of the total volume, when this quantity is computed correctly.

Existing failure data on well surface-controlled subsurface safety valves (SCSSVs) need to be collated and evaluated in the context of the potential catastrophic structural failure of the platform. Relevant data from the Hurricane Andrew experience should be analyzed. How often do SCSSVs fail to close? What might the influence of ground shaking be on their performance statistics?

5.5 Timely Specification of Seismic Forces

In the design of new structures, accurate and early specification of seismic design criteria for topsides equipment (eg. drill rig, cantilevers, wellhead hookup piping, motion sensitive equipment) is important for non-structural design team personnel and for equipment vendors and manufacturers.

This is particularly the case for flexible equipment such as the drill rig and flare booms. To this end, initial development work on the derivation of accurate floor spectra using simple, coupled structural models should be pursued. Such spectra have use not only in platform design, but also in the design or assessment of 'new' rigs for development work on existing platforms.

5.6 Reliable System Seismic Instrumentation

The development and use of reliable earthquake response data acquisition equipment is encouraged. This acquisition system should capture full system response - free field, near field, pile base and pile top, as well as deck response motions.

Given the paucity of data on system response, the benefits of these data in calibrating our numerical as well as seismotectonic models is of obvious benefit, particularly in the confirmation of results of complex system 'requalification' analyses on older platforms. These data acquisition systems must be reliable - they must not be made inoperative by repeated rig jarring and boat impacts or dead batteries.

5.7 In-house Earthquake Emergency Response

Pre-Quake: Elaborate plans and procedures typically exist for platform personnel emergency evacuation and facility shut-in procedures. It is recognized that proactive mechanical shut-in systems and evacuation procedures may not be included, even considered, in the response of personnel and equipment to a significant earthquake. It is recommended that such plans incorporate earthquake preparedness and earthquake response.

Consideration should be given to developing integrated early warning systems (i.e. 15 seconds or more advance warning), based on the electronic transmission of signals from the site of the earthquake's occurrence.

Post-Quake: A detailed inspection per the proceduralized inspection checklist discussed in Section 5.1 above should be triggered by a 'significant' earthquake. The list would define a clear post-quake inspection scope - where to look, and how to look - so that potential cracking can be identified and remediated as necessary before production operations resume.

5.9 Conductor Support Frame Modelling

It is suggested that numerical models developed for platform seismic strength and ductility analyses include proper detailing in the conductor framing areas. Because of the complexity of such analyses, particularly ductility analyses, the conductor support areas are typically modelled by simple X-frame representations.

The concern then is that the seismic response of conductor guides and conductor support framing may be misrepresented. Such misrepresentation could lead to unexpected failure of this framing, with potential knock-on effects of conductor instability and the undermining of system structural integrity.

6. Funding & Implementation

Some of the ideas recommended above can be implemented by continued industry efforts within the API's organization. Others will require proposals backed by public and private funds in order to be realized.

7. Acknowledgments

The contributions of the Operations core group members and those who participated at the Workshop to help develop these guidelines and recommendations are acknowledged and appreciated - Doug Nyman, Larry Wesselink, Bob Smith, Bob Visser, Ralph Warrington, Ward Turner, Gayle Johnson, Peter Brooks, Gary Gray, Bruce Hesson, John Jepson, Danny Chancellor, David Thomas, David Williams, and Glenn Shackell.

APPENDIX I

WORKING GROUP ON OPERATIONS

Excerpts from the next (20th) edition of API RP 2A–WSD 'Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms — Working Stress Design'

DECK APPURTENANCES AND EQUIPMENT (SEISMIC AREAS)

TEXT

2.3.6d. Ductility Requirements

1. The intent of these requirements is to ensure that platforms to be located in seismically active areas have adequate reserve capacity to prevent collapse under a rare, intense earthquake. Provisions are recommended herein which, when implemented in the strength design of certain platforms, will not require an explicit analytical demonstration of adequate ductility. These structure-foundation systems are similar to those for which adequate ductility has already been demonstrated analytically in seismically active regions where the intensity ratio of the rare, intense earthquake ground motions to strength level earthquake ground motions is 2 or less.

2. No ductility analysis of conventional jacket-type structures with 8 or more legs is required if the structure is to be located in an area where the intensity ratio of rare, intense earthquake ground motion to strength level earthquake ground motion is 2 or less, the piles are to be founded in soils that are stable under ground motions imposed by the rare, intense earthquake and the following conditions are adhered to in configuring the structure and proportioning members:

   — Jacket legs, including any enclosed piles, are designed to meet the requirements of Section 2.3.6c4, using twice the strength level seismic loads.

   — Diagonal bracing in the vertical frames are configured such that shear forces between horizontal frames or in vertical runs between legs are distributed approximately equally to both tension and compression diagonal braces, and that "K" bracing is not used where the ability of a panel to transmit shear is lost if the compression brace buckles. Where these conditions are not met, including areas such as the portal frame between the jacket and the deck, the structural components should be designed to meet the requirements of Section 2.3.6c4 using twice the strength level seismic loads.

   — Horizontal members are provided between all adjacent legs at horizontal framing levels in vertical frames and that these members have sufficient compression capacity to support the redistribution of loads resulting from the buckling of adjacent diagonal braces.

   — The slenderness ratio (Kl/r) of primary diagonal bracing in vertical frames is limited to 80 and their ratio of diameter to thickness is limited to $1900/F_y$ where F_y is in ksi ($13100/F_y$ for F_y in MPa). All non-tubular members at connections in vertical frames are designed as compact sections in accordance with the AISC Specifications or designed to meet the requirements of Section 2.3.6c4 using twice the strength level seismic loads.

3. Structure-foundation systems which do not meet the conditions listed in Section 2.3.6d2 should be analyzed to demonstrate their ability to withstand the rare, intense earthquake without collapsing. The characteristics of the rare, intense earthquake should be developed from site-specific studies of the local seismicity following the provisions of Section 2.3.6b1. Demonstration of the stability of the structure-foundation system should be by analytical procedures that are rational and reasonably representative of the expected response of the structural and soil components of the system to intense ground shaking. Models of the structural and soil elements should include their characteristic degradation of strength and stiffness under extreme load reversals and the interaction of axial forces and bending moments, hydrostatic pressures and local inertial forces, as appropriate. The P-delta effect of loads acting through elastic and inelastic deflections of the structure and foundation should be considered.

2.3.6e. Additional Guidelines

1. Tubular Joints. Where the strength level design horizontal ground motion is 0.05g or greater (except as provided in Section 2.3.6b2 when in the range of 0.05g to 0.10g, inclusive,), joints for primary structural members should be sized for either the tensile yield load or the compressive buckling load of the members framing into the joint, as appropriate for the ultimate behavior of the structure.

   Joint capacity may be determined on the basis of punching shear or nominal loads in the brace in accordance with Section 4.3 except that the allowable punching shear stress in the chord wall, v_{pa} and the allowable joint capacities, P_a and M_a, may be increased by 70 percent in lieu of a ⅓ increase. The factor A used in determining v_{pa} should be computed as follows:

$$A = \sqrt{\frac{\bar{f}_{AX}^2 + \bar{f}_{IPB}^2 + \bar{f}_{OPB}^2}{F_y}} \quad \ldots\ldots\ldots\ldots\ldots \quad (2.3.6\text{-}1)$$

   where $\bar{f}_{AX}$, $\bar{f}_{IPB}$ and $\bar{f}_{OPB}$ are stresses in the chord due to twice the strength level seismic loads in combination with gravity, hydrostatic pressure and buoyancy loads or to the full capacity of the chord away from the joint can, whichever is less.

2. Deck Appurtenances and Equipment. Equipment, piping, and other deck appurtenances should be supported so that induced seismic forces can be resisted and induced displacements can be restrained such that no damage to the equipment, piping, appurtenances and supporting structure occurs. Equipment should be restrained by means of welded connections, anchor bolts, clamps, lateral bracing, or other appropriate tie-downs. The design of restraints should include both strength considerations as well as their ability to accommodate imposed deflections.

Special consideration should be given to the design of restraints for critical piping and equipment whose failure could result in injury to personnel, hazardous material spillage, pollution, or hindrance to emergency response.

Design acceleration levels should include the effects of global platform dynamic response; and, if appropriate, local dynamic response of the deck and appurtenance itself. Due to the platform's dynamic response, these design acceleration levels are typically much greater than those commonly associated with the seismic design of similar onshore processing facilities.

In general, most types of properly anchored deck appurtenances are sufficiently stiff so that their lateral and vertical responses can be calculated directly from maximum computed deck accelerations, since local dynamic amplification is negligible.

Forces on deck equipment that do not meet this "rigid body" criterion should be derived by dynamic analysis using either: 1) uncoupled analysis with deck level floor response spectra or 2) coupled analysis methods. Appurtenances that typically do not meet the "rigid body" criterion are drilling rigs, flare booms, deck cantilevers, tall vessels, large unbaffled tanks and cranes.

Coupled analyses that properly include the dynamic interactions between the appurtenance and deck result in more accurate and often lower design accelerations than those derived using uncoupled floor response spectra.

Drilling and well servicing structures should be designed for earthquake loads in accordance with API Specification 4F. It is important that these movable structures and their associated setback and piperack tubulars be tied down or restrained at all times except when the structures are being moved.

Deck-supported structures, and equipment tie-downs, should be designed with a one-third increase in basic allowable stresses, unless the framing pattern, consequences of failure, metallurgy, and/or site-specific ground motion intensities suggest otherwise.

2.3.7 Accidental Loads. Offshore platforms may be subject to various accidental loads such as: collision from boats and barges; impact from dropped objects; explosion or fire. Considerations should be given in the design of the structure and in the layout and arrangement of facilities and equipment to minimize the effects of these loads.

Potential impact from operational boat or barge traffic for jacket waterline members, risers, and external wells should be considered. Barge bumpers, boat landings, and other external fendering may be used as protection. Certain locations of the deck, such as crane loading areas and areas near the drilling rig, are more likely to be subject to dropped objects. The location of equipment and facilities below these areas should be considered to minimize damage from dropped objects.

Typically, an offshore structure is constructed of an open framework of structural shapes and tubular members which is relatively resistant to blast and explosion. When it is necessary to enclose portions of a platform in locations where the potential for gas explosion exists, the protective siding or walls should include blowout panels or should be designed to collapse at low uniform pressure to minimize the load on primary members. Fire protection precautions are covered in other API and industry codes and specifications.

It is possible that accidents or equipment failures may cause significant structural damage. Inspection of this damage in accordance with Section 14 of RP2A can provide the information for analytical work to determine the need for immediate or eventual repair. Such analysis will also identify under what conditions the installation should be shut-in and/or evacuated. It is not anticipated that the accidental event will occur simultaneously with design environmental loads.

2.4 FABRICATION AND INSTALLATION FORCES.

2.4.1 General. Fabrication forces are those forces imposed upon individual members, component parts of the structure, or complete units during the unloading, handling and assembly in the fabrication yard. Installation forces are those forces imposed upon the component parts of the structure during the operations of moving the components from their fabrication site or prior offshore location to the final offshore location, and installing the component parts to form the completed platform. Since installation forces involve the motion of heavy weights, the dynamic loading involved should be considered and the static forces increased by appropriate impact factors to arrive at adequate equivalent loads for design of the members affected. For those installation forces that are experienced only during transportation and launch, and which include environmental effects, basic allowable stresses for member design may be increased by ⅓ in keeping with provisions of Section 3.1.2. Also see Section 12, "Installation," for comments complementary to this section.

2.4.2 Lifting Forces

2.4.2a. General. Lifting forces are imposed on the structure by erection lifts during the fabrication and installation stages of platform construction. The magnitude of such forces should be determined through the consideration of static and dynamic forces applied to the structure during lifting and from the action of the structure itself. Lifting forces on padeyes and on other members of the structure should include both vertical and horizontal components, the latter occurring when lift slings are other than vertical. Vertical forces on the lift should include buoyancy as well as forces imposed by the lifting equipment.

To compensate for any side loading on lifting eyes which may occur, in addition to the calculated horizontal and vertical components of the static load for the equilibrium lifting condition, lifting eyes and the connections to the supporting structural members should bers are strongly dependent on their D/t and slenderness ratios (48). A significant amount of ductility can be built into the structure by implementation of the generic design guidelines presented in Section 2.3.6d2. Foundation models should consider the effects of cyclic load reversals, strain rate, pore water pressure generation on the strength and stiffness of the soils surrounding the piles (49, 50, 51, 52, 53), and energy dissipation mechanisms (54, 55, 56).

The designer should develop a thorough insight into the performance of the structure and foundation during a rare, intense earthquake. The time history method of analysis is recommended. The structure-foundation response should be determined to multiple sets of ground motions which characterize the likely envelope of ground motion intensity, frequency content, phasing and duration expected at the site. At least three sets of representative earthquake ground motion time histories should be analyzed. Additional more simplistic methods of analysis may be used to complement the results of the time history analysis (13).

C2.3.6e Additional Guidelines

1. **Tubular Joints.** Joints are sized for the yield or buckling capacity of incoming members, so that premature failure of the joints will be avoided and the ductility of the overall structure can be fully developed.

   The recommended practice is to size jacket leg joint cans for full yield in main diagonals, and for the buckling load of principal horizontals. These horizontals typically have small loads for elastic analysis, but are required to pick up substantial compressive loads to prevent the structure from "unzipping" after main diagonals buckle. Excessive joint can thickness may often be avoided by using a conical stub end on the governing member; or by considering the beneficial effects of member overlap (Section 4.3.2) and/or grouted-in piles.

2. Deck Appurtenances and Equipment. The method of deriving seismic design forces for a deck appurtenance depends upon the appurtenance's dynamic characteristics and framing complexity. There are two analysis alternatives.

   First, through proper anchorage and lateral restraint, most deck equipment and piping are sufficiently stiff such that their support framing, lateral restraint framing, and anchorage can be designed using static forces derived from peak deck accelerations associated with the strength level seismic event.

   To provide assurance that the appurtenance is sufficiently stiff to meet this criterion, the lateral and vertical periods of the appurtenance should be located on the low period, 'flat' portion of the deck level floor response spectra. Additionally, the local framing of the deck that supports the appurtenance must also be rigid enough to not introduce dynamic amplification effects. In selecting the design lateral acceleration values, consideration should be given to the increased response towards the corners of the deck caused by the torsional response of the platform.

   Second, in cases of more compliant equipment — such as drilling and well servicing structures, flare booms, cranes, deck cantilevers, tall free-standing vessels, unbaffled tanks with free fluid surfaces, long-spanning risers and flexible piping, escape capsules, and wellhead/manifold interaction — consideration should be given to accommodating the additional stresses caused by dynamic amplification and/or differential displacements estimated through either coupled or decoupled analyses.

   Decoupled analyses using deck floor spectra are likely to produce greater design loads on equipment than those derived using a more representative coupled analysis. This is particularly the case for more massive components, especially those with natural periods close to the significant natural periods of the platform. References 61 through 64 describe coupled procedures, and decoupled procedures which attempt to account for such interaction.

   If coupled analyses are used on relatively rigid components that are modelled simplistically, care should be exercised such that the design accelerations which are derived from the modal combination procedure are not less than the peak deck accelerations.

   Field inspections by experienced personnel of equipment and piping on existing platforms in seismic areas can help identify equipment anchorage and restraint that by experience and/or analysis should be upgraded. To accommodate loadings and/or differential displacements, the addition or deletion of simple bracing and/or anchorage to these components can significantly improve their performance during an earthquake. This is especially important for critical components such as piping and vessels handling hazardous materials, emergency battery racks, process control equipment, etc.

   The use of a none-third increase in basic allowable stresses is usually appropriate for designing deck supported structures, local deck framing, equipment anchorage, and lateral restraint framing under strength level earthquake loads. This lower increase in design allowables for strength level earthquake loads compared to a full yield stress allowable typically used for jackets is intended to provide a margin of safety in lieu of performing an explicit ductility level analysis.

   However, in areas where the ratio of rare, intense ground motion intensities to strength level ground motion intensities is known to be higher than 2.0, an adjustment to the design allowable stresses should be considered. Also, for certain equipment, piping, appurtenances or supporting structures, the degree of redundancy, consequences of failure, and/or metallurgy may dictate the use of different allowable stresses or a full ductility analysis, depending on the component's anticipated performance under rare, intense earthquake ground motions.

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APPENDIX II

WORKING GROUP ON OPERATIONS

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PUBLIC POLICY

Richard McCarthy and Sylvia Earle, Co-Chairs
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Aging offshore structures present challenges to industry and regulators worldwide. Sound public policy must protect lives, the environment, and assure continued profitability. Therefore, procedures and standards need to be developed and implemented to assure that offshore structures will perform acceptably and a balance is reached between competing interests in the offshore area.

Public policy concerns incorporate facts and perception, private and public interests, costs and benefits, shared decision making, and others. With regard to seismic risk to offshore structures, public policy assumes that an older structure has a probability of failure that is greater than its new modern counterpart built to the latest standards. Therefore, public policies must address the probability of failure of the operating system and its consequences by requiring detailed studies of the performance characteristics of the topsides equipment, pipelines, terminals and operations. Probability of failure should not be restricted to earthquakes alone, rather, wind and wave loads, drilling, workovers, and production operations should also be considered.

To accomplish the difficult regulatory goal of adequate seismic design and reassessment of offshore structures, problems in "organizational policy" and "fitness for purpose" need to be addressed. While offshore resources have been developed in the past through ad hoc procedures, future development should be undertaken with better coordination and understanding between all concerned parties. The working relationship between government and industry should be based on trust, professional integrity, mutual respect, and a clear understanding of the role each plays in operating in the offshore realm. It must be absolutely clear not only to the operator and the regulator, but also to the citizens of the coastal country permitting the development, that specific acceptable levels of risk have been developed and will be adhered to under any circumstances. Obviously, oil spilled into the water column from collapsed aging offshore structures not only impacts the environment but can have major financial impacts on the industry as a whole and the host country for years.

The Public Policy Working Group was given a day and a half to formulate a position paper that would represent a consensus on some, if not all, of the following topics:

- reliability goals
- acceptable levels of risk
- life safety
- environmental safety, and
- historical performance of offshore structures.

To approach the issue of public policy as it relates to the reassessment of the seismic design of offshore structures, the working group first developed a definition of public policy and study framework. It found that:

GOOD PUBLIC POLICY is one that can be applied globally, yet is flexible enough to allow case-by-case decision making when needed.

For purposes of discussion, the Working Group took advantage of the availability of the May 1992 Panel Report, "Seismic Safety Re-qualification of Offshore Platforms" prepared for the American Petroleum Institute (the Panel Report). Although not an official API position document, it served as a "strawman" for discussion of reassessment policy issues. Some of the key findings of the Panel Report which were examined by the working group were:

- The risks associated with life safety and environmental safety can be treated separately for the purpose of requalification.

- Life safety hazards are a concern only for "manned" platforms where manned is defined as a platform "which is actually and continuously occupied by at least 5 persons" (from API RP 2A). The seismic hazard posed by a requalified platform should be of the same order as that posed by well-designed onshore conventional building structures. This life safety level could be met either through the requirements of the 1991 API RP 2A, using ground motion levels of 200 years for strength and 1,000 years for ductility or, through a probability of collapse in an earthquake not to exceed 0.001 per year.

- Structural analyses should meet peer review.

- The seismic environmental hazard posed by a requalified platform should be no greater than posed by other major offshore petroleum release sources. If potential spills from a source would be less than or equal to 2,000 barrels, then there is no need to limit the potential for the spill. If potential spills from a source would exceed 2,000 barrels, the annual probability of an occurrence should be less that or equal to 0.001 (or the platform should meet the requirements of the 1991 RP 2A). In this, sources of spills are considered separately, and the 2,000 barrel cut-off value was selected since this is the estimate for releases in the Santa Barbara offshore area due to natural sources.

The group also discussed the usefulness of peer review, alternative review procedures and techniques to develop improved public input.

One major shortfall of existing offshore public policy in the area of Offshore Platform Reassessment is that the current practice uses case-by-case-review procedures, without standardized criteria. There is not now a formalized trigger process to initiate seismic reassessment, although major events have served as informal triggers. The Minerals Management Service viewed the Loma Prieta Earthquake in 1989, as a trigger. In a similar way, the California State Lands Commission uses structural or operational changes to the platform as review triggers. The Panel Report was seen as an effort to move from the current Ad Hoc process to a more structured one.

Due to the wide range of opinions expressed on a variety of topics, it was difficult to reach full consensus on many issues. Good agreement was reached that economics would be the owners prerogative, so long as the minimum standards are maintained (however, there was no consensus on what the minimum standards should be). It was also agreed that independent panels were working well and that they should be use more. Participants also agreed that the peer review concept and the idea of an enhanced Certified Verification Agent (CVA) for seismic reassessment needed more work.

While most of the participants felt that partitioning of risk (life safety from environmental) was working, there was much discussion about how these concerns were addressed in other industries and whether full partitioning is possible when either trigger could ultimately have the same consequences. Through this discussion, the group agreed that the requalification triggers needed to be better defined and that more work was necessary to identify threshold values, possibly on a regional basis. Inherent in this was the possibility that in some regions or situations, a threshold of zero may be the acceptable one.

Greater consensus was reached on types of research that would be beneficial in developing public policy and the priority actions that should be undertaken (Table 1). The Panel Report proposed a reassessment procedure that used triggers for life safety and for environmental risk. There are other processes or models to achieve acceptable levels of risk and, due to extent of discussion about the triggering values, the group recommended that other models be studied and compared to the trigger model that was proposed in the Panel Report. If the triggering model is found to be the preferred procedure, the group recommended that all of the triggering quantities be reexamined—the allowable amount of spillage, the occupancy level, and the probability of failure. Finally, regardless of the model, the group felt that action should be taken to implement a peer review process, possibly following the guidelines developed by the American Society of Civil Engineers, and develop procedures for obtaining public input throughout the reassessment process.

TABLE 1

FINAL RECOMMENDATIONS OF THE PUBLIC POLICY WORK GROUP

PROPOSED RESEARCH

1. Investigate other processes (models) to achieve acceptable levels of risk/requalification procedures.

2. Re-examine allowable amount of spillage; put into context with other sources.

3. Re-examine the 0.001 probability of failure.

4. Re-examine the occupancy issue; consider options to the API RP 2A definition of "manned", such as quartered, person-years, and the need for limited occupancy for maintenance or recommissioning.

PRIORITY ACTION

1. Implementation of a Peer Review Process (see ASCE Guidelines).
2. Investigate the applicability of the procedures to other natural hazards.
3. Define active and passive triggers for requalification.
4. Define catastrophe.
5. Develop mechanisms for getting public input.

PUBLIC POLICY WORKING GROUP PARTICIPANTS

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APPENDIX A

The Public Policy Working Group Co-Chairs questioned numerous individuals before the conference to collect a list of possible topics to be presented for discussion during the conference. Although most of the following issues were not discussed by the Working Group, they do present some ideas for future conferences that wish to address offshore public policy.

Organizational Policy Issues

1. Government/Industry Relationships

   - Should set goals be common or competing?
   - How should responsibilities be divided?
   - Who is accountable to whom?

2. What are the best ways for industry and government to establish better credibility with the public?

3. What level of risk is acceptable?

   - Is risk a function of thc pricc of oil?
   - Is risk a function of the size of the reservoir?
   - Who wins and looses? Who is the riskee—the riskor (the community, all industry members, resource owners, regulators)?

4 What role should public participation play in the development of offshore resources?

5. Could the development of an Information System be developed that would help reduce the overall risk of offshore structures (Information Systems are currently being applied by the Federal Aviation Administration and the Nuclear Regulatory Commission)? An Information System could address the following offshore issues:

   - Design Information—earthquake, storm waves, wind, codes and practices, standards and specifications
   - Fabrication Information—materials, welding techniques, rolling and forming, erection procedures
   - Transportation Information—transpacific tows, routes barges, seasons, towing procedures, launch procedures,

- Installation Condition Information—upending, ballasting, on bottom stabilization, pile driving and attachment, pipeline riser attachments, weld conductors installation

- Operations Information—drilling phase, production transportation, personnel, maintenance

- Inspection/Maintenance/Repair Information—inspection timing and procedures, data recording, repairs planning, cathodic protection, scour protection, marine growth management

- Decommissioning Information—removal of platform equipment, securing wells, removal of risers, removal of platform

- Miscellaneous Issues

—Who would be responsible for the System?
—What would it consist of (more than above)?
—How should it be used?
—Who should have access?

6. Can the risk of developing large offshore lease sale areas be adequately evaluated during the EIR/EIS stages to satisfy seismic/environmental regulatory concerns?

   - Would smaller lease areas help to establish more confidence in determining real world seismic risk? If so, this may help reduce problems 30 years later when structures must be reassessed for extended service.

   - Who should be responsible for the work?

Fitness for Purpose

1. What are the goals?

2. How are they measured?

3. How do you prove fitness for purpose?

   - Development of performance standards
   - Verification process (certified verification agent)
   - Independent third party technical review panel

4. What are the acceptable procedures for calculating fitness for purpose?

5. Should the regulatory emphasis be placed on Total Risk (i.e. earthquake, wave, operations) vs. Focused Risk (i.e. earthquake only) reduction?

- If total risk is the preferred alternative, what procedures should be used to adequately and accurately translate and transfer this knowledge to the public?

- Who is the riskee/riskor?

6. Should regulatory and industry emphasis be placed on overall group risk reduction or the reduction of risk on an individual basis. In other words, given cost factors, is it best to reduce the overall level of risk of many structures or focus resources on higher risk reduction on a structure-by-structure basis?

7. Standardization of Offshore Structures—i.e. would standardization of platform modules/jackets help achieve acceptable levels of risk?

8. Should older platforms be retro-fitted to meet the same performance standards as new platforms? If so, can it be done?

9. What should be the regulatory time requirements to reassess offshore structures?

   - How much time should be given to owner to reassess?

   - How much time to approve?

   - How much time to verify?

   - Who does it?

10. Should bonding be required to assure that structures will be removed at end of service life? Should regulators accept a higher level of risk if bonding assures the removal of a collapsed structure and the cleanup of the affected environment?

11. What is the acceptable degree of safety against undesirable performance during intense seismic or wave loading events? Should the reassessment goal be to:

    - Minimize loss of life?

    - Minimize environmental damage?

    - Reduce costs—return to full production levels quickly?

    - Maintain profitability?

    - All of the above?

12. Deterministic vs. probabilistic method of assessing site strong ground motion. Should one be favored over the other for reassessment of aging structures and the design of new structures. New offshore structures designed around a probabilistic earthquake methodology may have problems years later during regulatory reassessment review. This is due to advances in the fields of seismology and geology and the increase in the numbers of strong ground motion records that have recorded higher than expected accelerations, both vertical and horizontal. The probabilistic method may not be the desired alternative (for either new structures or aging structures undergoing reassessment) in areas where the fault activity rate data are limited for most seismic sources.

13. Can subsea completions be used cost-effectively to reduce the number of new platforms required, yet maintain the desired production levels?

    - Does the use of subsea completions reduce or increase the risk of oil spills?
    - Can subsea completions reduce the size of oil spills?
    - How do subsea completions perform under intense earthquake loads?
    - Are there some instances where an aging structure that cannot be retro-fitted to meet seismic concerns be removed and replaced cost effectively by subsea completions?

14. Onshore policies/approaches applied to offshore.

    - Can public policies developed for onshore structures be effectively applied to offshore development?

15. Does the concept of "risk management" which considers risk reduction and risk management measures as a means to reduce the residual risk hold true for offshore structures?

APPENDIX B

Members at the California State Lands Commission reviewed the materials provided in Appendix A and developed the following pre-conference outline for the Public Policy Working Group:

Outline
Public Policy Recommendations Session
International Workshop on Seismic Design/Requalification of Offshore Structures

I. Risk Assessment and Operator/Regulator Policies

A. Acceptable level of risk

1. Voluntary vs. involuntary
2. Public safety and pollution mitigation (regulatory directive)
3. Corporate (operator) perception of adverse publicity
4. "Whitman-type" diagram for public policy
5. "Whitman-Bea" diagram for economic evaluation and/or a combined risk assessment
6. Revenue losses (operator, community and regulator)

    a. Commerce, fishing, local area economics
    b. Litigation against operators
    c. Clean-up costs (public & private)
    d. Revenue loss (public & private)
    e. Removal/repair costs

7. Other sources of risk (fire, operational upset, vessel impact, etc.)

8. Is the API RP 2A acceptable for new structures?

B. Value prescribed in the API document ("Seismic Safety Requalification of Offshore Platforms, Final Draft, May 1992) is 1.0e–3/year.

C. Risk assessment by interference from the Uniform Building Code and the California Structural Engineers Association "Blue Book".

D. Other

II. Policy issues of the API Draft Document

A. What should constitute the trigger for a requalification effort? What about the frequency for requalification?

B. Is the formation of the "peer group" for the evaluation of the seismic hazard feasible? How would this be implemented?

C. A "peer group" review of the proposed design modifications and analysis procedures is also mandated. How would this be implemented? By whom?

D. The API document states that the current "CVA process" should be enhanced. Should an analysis be reviewed independently for reasonableness? What about the regulator?

E. Applicability to various high seismic zones (CA & Alaska)

F. The 2000 barrel environmental lower-limit

G. Should any structure be considered unmanned? The API document uses a 5 man average as the bounds between unmanned and manned.

H. What about a real risk of adjacent platforms, or other facilities such as pipelines?

I. Human safety versus pollution prevention — is the determination of structural integrity adequate to address both?

J. Inspection frequency and quality assurance programs after requalification

K. Deck appurtenances — level of structural requalification required.

L. Other issues

III. Directions and focus for further research

A. Possibly have a caretaker for all of these peer reviewed requalification processes and information, so that other operators could understand the scope of the problem, and the general methodology and procedures used by others, with the hope of all operators not having to go to the same consulting firm?

B. A research consensus or at least compendium of various regulator and operator methodologies for the assessment of the combined risk of an offshore structures, subjected to seismic loading. This effort should compare the isolated seismic only probability approach with the "basket" approach.

C. A comparison between the analysis and design resulting from the API draft document guidelines, and those used by the "preferred" operator.

D. Other directions for further research

APPENDICES

APPENDIX I – FINAL PROGRAM

APPENDIX II – LIST OF ATTENDEES

APPENDIX I

FINAL PROGRAM

Monday, December 7

WELCOME AND INTRODUCTIONS:

W. D. Iwan, Chair
Paul C. Jennings, Vice President and Provost, Caltech
David C. O'Neal, Assistant Secretary for Land and Minerals, Department of the Interior
Paul Mount II, Chief, Mineral Resources Management Division, California State California State Lands Commission

INVITED LECTURES:

Session 1 — G. W. Housner, Caltech, Chair

Thomas Tobin, California Seismic Safety Commission, "Policy Issues Related to the Seismic Performance of Offshore Structures"

C. B. Crouse, Dames & Moore, "Estimation of Ground Motion for Design or Reassessment of Offshore Platforms"

Session 2 — A. G. Brady, U.S.G.S., Chair

Robert Bea, U.C. Berkeley, "Seismic Design and Requalification Methodologies for Offshore Platforms"

Daniel Dolan, PMB Engineering, "Case Studies on Seismic Reassessment Analysis"

Session 3 — M. Eskijian, California State Lands Commission, Chair

Robert Visser, Belmar Engineering, "Operations Issues in Seismic Design and Reassessment"

Michael O'Neill, University of Houston, "Issues in the Assessment of Pile Behavior During Seismic Events"

David Wisch, Texaco, "The API Requalification Project"

WORKING GROUP SESSIONS:

- Site Seismic Hazard and Ground Motion; A. Cornell and P. Somerville, Co-Chairs
- Design, Reassessment and Requalification; K. Digre and W. Ibbs, Co-Chairs
- Structural Performance; H. Bannon and J. Penzien, Co-Chairs
- Operations; M. Craig and D. Hopper, Co-Chairs
- Public Policy; R. McCarthy and S. Earl, Co-Chairs

Tuesday, December 8

Working Group Sessions

Wednesday, December 9

Presentations by Working Group Co-Chairs — G. W. Housner, Chair

Discussion — W. D. Iwan, Chair

Adjournment

APPENDIX II
LIST OF ATTENDEES

Allen Adams, Minerals Mgmt. Service
Rod Akky, Geomatrix
Hugh Bannon, Exxon Production Res. Co.
Joan Barminski, Minerals Mgmt. Service
Y. Bayazitoglu, Brown & Root
Robert Bea, U.C. Berkeley
James Beck, Caltech
Jared Black, UNOCAL
Thomas Blake, Fugro-McClelland
D.L.R. Botelho, Chevron
Gerald Brady, U.S.G.S.
Peter Brooks, WGP Engineering
Sam Bryant, Fugro-McClelland
Kenneth Campbell, EQE International
Brad Campbell, Exxon
D. Chancellor, Chevron
Mario Chavez, Cuidad Universitaria
Jen-Hwa Chen, Chevron Oil
Lloyd Cluff, PG&E
C. Allin Cornell, Stanford
Michael Craig, UNOCAL
C. B. Crouse, Dames & Moore
Rabi De, Shell Development Co.
Kris Digre, Shell Oil
Daniel Dolan, PMB Engineering
Michael Dowling, WGP Engineering
Tomas Dunaway, Minerals Mgmt. Service
F. Dunn, Shell Oil
Felix Dyhrkopp, Minerals Mgmt. Service
Sylvia Earl, NOAA
Martin Eskijian, California Lands Comm.
S. Fu, Chevron
Wenshui Gan, Caltech
Jerry Geesling, Shell Western E&P
Troy Gillum, WGP Engineering
Catherine Gitkov, Alaska, Env. Conserv.
Gary Gray, Chevron USA
Steve Guynew, ARCO
John F. Hall, Caltech
Marvin Halling, Caltech
T. Hasselman, Engr. Mech. Assoc.
Neal Hennegan, Shell Offshore
Bruce Hesson, Dept. of Conservation
David Hopgood, UNOCAL
David Hopper, Hopper and Associates
George Housner, Caltech
Liping Huang, Caltech
William Ibbs, U.C. Berkeley
Gary Imm, Amoco
Jack Irick, Barnett & Casbarian
Wilfred D. Iwan, Caltech
Paul Jennings, Caltech
John Jepson, Dept. of Conservation
Gayle Johnson, EQE Engineering
Joe Kallaby, Offshore Structures
V. Karthigeyan, Offshore Safety Div.
Hyungwoo Kim, Shell Oil Co.
Frank Kpodo, UNOCAL
William Krieger, Chevron
Steven Kuehn, Hopper and Associates
Griff Lee, McDermott
Mark Legg, ACTA, Inc.
Dick Litton, PMB Engineering
James Lloyd, Exxon Production Research
Terry Lundeen, Ratti Swenson Perbix
Nabil Masri, Minerals Mgmt. Service
Sami Masri, USC
Rolf Mauermann, Caltech
Melinda Mayes, Minerals Mgmt. Service
Bill McCarron, Amoco Prod. Co.
Richard McCarthy, CA Seism. Safety Com.
Leslie Monahan, Minerals Mgmt. Service
Paul Mount II, California Lands Comm.
John Niedzwecki, Texas A&M Univ.
Robert Nigbor, Agbabian Associates
Douglas Nyman, D.J. Nyman & Assoc.
Patrick O'Connor, Amoco Prod. Co.
David O'Neal, Department of Interior

Michael O'Neill, University of Houston
Tom Paez, Sandia Labs
Bazzurro Paolo, Stanford University
John Pelletier, Shell Development Co.
Joseph Penzien, Intl. Civil Engr. Cons.
Stephen Perryman, Amoco Production Co.
Mark Petersen, Univ. Nevada, Reno
Rick Reimer
James Ricles, Lehigh University
Andrew Santos, Cook Inlet Regional Citizens Advisory Council
Shahram Sarkani, George Wash. Univ.
Doug Schmucker, Stanford University
Herb Schneider, Minerals Mgmt. Service
Ronald F. Scott, Caltech
Clay Serrahn, F.D.E.A.S. Inc.
Glenn Shackell, Minerals Mgmt. Service
Arvind Shah, Minerals Mgmt. Service
Allan Shareghi, Minerals Mgmt. Service
Leonard Simkin, F. David Chavez Eng.
Charles Smith, Minerals Mgmt. Service
Robert Smith, ARCO
Paul Somerville, Woodward-Clyde
Douglas Stock, Digital Structures
Paul Summers, WGP Engineering
Takaomi Taira, Nuclear Powr Eng., Japan
Andrew Taylor, NIST/BFRL
Charles Thiel, Stanford
David Thomas, Thomas and Beers
Thomas Tobin, CA Seism. Safety Com.
Tomohiko Tsunoda, Obayashi Corp.
Barbara Turner, UNOCAL
Ward Turner, Exxon
Peter Tweedt, Minerals Mgmt. Service
Robert Visser, Belmar Engineering
Mladen Vucetic, UCLA
Ralph Warrington, Shell Western E&P
Y. Wen, University of Illinois
Joe Wescott, California Lands Comm.
Larry Wesselink, Chevron Res. & Tech.
David Williams, Digital Structures
David Wisch, Texaco
Tsukasa Yamawaki, Nucl. Pow. Eng., Japan
Chi-Ming Yang, Caltech
Solomon Yim, Oregon State University