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# Assessment and Requalification of Offshore Production Structures

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## Workshop Location

Doubletree Hotel
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# ACKNOWLEDGMENTS

# EXECUTIVE SUMMARY

Oil and gas production from offshore waters started in the 1940s. At present there are more than 7,000 offshore production structures worldwide and a large supportive infrastructure. Damage produced by Hurricane Andrew in the Gulf of Mexico and other recent offshore accidents pointed out that some of these offshore structures may not meet today's requirements for safety and reliability, either because of deterioration of the structures or because of the more stringent standards that have been developed as new knowledge has become available. This has caused regulatory agencies to consider the need to assess the ability of older structures to perform their original function while maintaining environmental quality and the efficient use of scarce petroleum resources. At the same time it is recognized that if the requirements are too stringent, older platforms with marginal production and high retrofit costs could be shut down and valuable resources lost. Thus, there is an enormous need for more sophisticated and rational requalification procedures that will enable these structures to be used for society's greatest good.

To address the issues of reassessment and requalification of offshore structures, an international workshop was held with the stated purpose of improving the understanding of the requalification processes and procedures. More than 250 participants from eight countries attended this workshop, representing marine structure owners and operators, consulting firms, contractors, manufacturers and fabricators, government agencies, academic and research institutions and trade associations. The workshop was originally conceived by Charles Smith, Minerals Management Services, U.S. Department of the Interior. Initial funding for the workshop was also provided by MMS with additional funding provided by the American Petroleum Institute, American Bureau of Shipping, McDermott Inc., and PMB/Bechtel. Planning for the workshop was conducted by an international steering committee composed of well-recognized industry and academic experts who had both special interest and knowledge in assessment and requalification of offshore structures.

The steering committee set the stage for the workshop, recommending that a series of keynote talks be followed by concurrent sessions of six topical working groups considering the following:

- Inspection, Surveys and Data Management
- Environmental Conditions and Forces
- Structural Elements, Systems and Analysis
- Foundation Elements, Systems and Analysis
- Operational Analysis
- Policy Considerations and Consequences

These working groups were led by several experts in their respective areas who first produced "white papers" which were distributed to the workshop participants and which served to provide the focus for the ensuing workshop discussions. The working group discussions were conducted in varied ways depending on the makeup of the attendees. The common elements in all sessions were the openness of the comments and the freedom of time to allow all participants who so desired the opportunity to present their comments.

The workshop was opened by Bud Danenberger who presented the interests and needs of MMS in the requalification process. This was followed by the keynote speakers who presented an international view of assessment and requalification of offshore structures. Two of the keynote talks also presented the charge to the working groups. The working groups then met in individual sessions to discuss their respective topic.

These Proceedings document the results of the workshop, including the keynote talks, the white papers for each working group, and the results of the working group discussions. One of the important tasks of each working group was to determine what research, data or new development was needed for rational requalification procedures. The working groups were given license to portray the results of their discussions as they saw fit. In one case the white paper was rewritten to embed the main discussion comments in the text. No attempt was made to change the format as received to ensure that the spirit of the individual workshop discussions was not altered.

The major inputs from each working group, in terms of research, data or new developments needed to aid in the reassessment and requalification process, are summarized below. Individual working group discussion provide more detailed information.

## Inspections, Surveys and Data Management

- Real time, economical capture of inspection information directly into a data base, particularly for pictures and sketches, would enhance the storage and retrieval of inspection information.
- More work is warranted on the effects of fatigue in the actual collapse of structures in the Gulf of Mexico
- Remotely operated vehicles (ROVs) which are often the only feasible means of conducting underwater surveys, should have their capabilities extended in the areas of cleaning members and zones to be inspected and in the ability to conduct more sophisticated nondestructive tests, for example, ultrasonic tests.
- Remote sensing systems, such as "leak before break" detectors, acoustic emissions and others, should be developed and/or improved to enhance field inspection methods and options.
- There is currently no standard for defining the qualifications of acceptable offshore platform inspectors. Industry and regulatory agencies should develop and share training and certification programs for inspectors.

## Environmental Conditions and Forces

- In both shallow and deep water the statistics of wave crest heights (symmetry of the waves) are of significant importance to determine the air gap of the topside equipment. Wave loading could become very large if the wave crests hit the topsides.
- Qualification criteria for the Gulf of Mexico are dependent on platform survival statistics in Hurricane Andrew. Consequently, there is a need to reevaluate the hindcast of Andrew in view of measured data that were not available when the first hindcast was done.
- The forcing of dynamically sensitive structures should be evaluated with particular attention to nonlinear transient loading effects.
- Utilization of relative velocity for dynamically sensitive structures has been widely discussed for some time. Further assessment is recommended in view of the need for review of all aspects producing hydrodynamic damping.
- The crest of the wave contributes the largest drag loading on an offshore structure. Further research is needed to accurately predict wave crest kinematics in irregular seas.
- Laboratory and field measurements have produced large scatter in predicted versus measured wave force. There is need to understand this scatter and review its effects on reliability analysis of offshore structures.
- There is a need to understand the scatter in hindcast versus measured storm wave peaks and to review its effects on reliability analysis

## Structural Elements, Systems and Analysis

- The API approach for requalification can be appropriate for areas other than the U.S. Gulf of Mexico, but it should be tailored for the area. Other areas needing consideration include Africa, Borneo, China and Thailand.
- There should be a better standardization of deterministic and probabilistic approaches for requalification. There needs to be training in the application of probabilistic methods by the analysts and decision makers.
- There is a significant difference in the application of factored design approaches: in some cases the factors are applied to individual parameters that make up the load and resistance formulation and in others a single factor is applied. Basic data need to be reviewed and the differences resolved.
- It is possible to use both limit state and working stress methods in assessment, but at the present, the methods of interpreting test data are not consistent among researchers and analysts. There is a need to determine how the test data can be incorporated in each case and whether the results can be calibrated to result in the same outcome.
- Guidelines are needed for methods of nonlinear modeling and finite element analysis. Existing techniques should be compiled along with existing data and there is a need to address complex structures, e.g., multiplanar joints.
- More research is needed to provide adequate data to specify assessment procedures that account for component ductility, denting, cracking and low cycle fatigue, particularly for complex structures such as multiplanar

joints.
- To evaluate the applicability of both simplified methods of analysis as well as detailed finite element analysis, a benchmark structural model is needed that exercises the most important nonlinear characteristics of structural behavior. Testing is also needed to validate analytically developed post-buckling behavior.
- A means of calibrating to the operational environmental load needs to be established, so the operating environmental condition can be defined. Because some structures are not dominated by environmental loading, there should be an operational check provided for such structures. Before, it has been left entirely to the operator, since it has no effect on the safety on the structure. If the structure can survive the extreme event, then it can satisfy the operating environmental case.

## Foundation Elements, Systems and Analysis

- Many older structures have missing or questionable foundation data and perhaps even the foundation configuration is unknown. There is a need for non-intrusive methods for assessing or confirming soils and pile properties in older in-place platforms.
- There is a continuing need to assess geotechnical properties from geophysical data. Even with significant past efforts it is still not possible to distinguish soil types from these data, much less specific soil data.
- In keeping with the recommendation above, there is a need to identify soil properties using less traditional geophysical methods, such as stress wave measurements.
- The various computer methods currently available to analyze the effects of cyclic loading on soils need to be calibrated with physical tests both in the field (mini-piles and full-scale piles) and in the laboratory (model piles, centrifuge tests).
- Analytical methods of foundation analysis currently presented in API recommended design practices should be refined based on actual load tests of piles which more closely approach prototype sizes and load levels.
- Recommended design methods should be improved for new or unfamiliar soil types, for nonstandard installation techniques and for severe load regimes.
- Research needed in the application of reliability analysis to foundation assessment include:
    — reduce model uncertainty by obtaining and analyzing more high quality performance data from laboratory or field tests, construction records, and storm and earthquake performance records,
    — conduct comprehensive probabilistic sensitivity studies to identify the effects that contribute most to the overall uncertainty in predicted capacity,
    — study the performance of the foundation system relative to the individual components that form the system to better develop the definition of failure that best reflects the actual performance,
    — quantify failure probabilities for different offshore structures in cooperation with owners and policy makers to make requalification decisions considering costs and benefits.

## Operational Considerations

- Platforms should be equipped with monitoring equipment to:
    — record earthquake-induced motions in seismic areas,
    — obtain deck motions on compliant minimum structures to verify dynamic modeling and ascertain effects on motion sensitive equipment,
    — monitor gradual changes with time in areas subject to foundation subsidence or movement.
- There is a need for compilation and dissemination of worldwide high quality near-miss and accident data, not only for statistical manipulation of the data base, but for evaluation of incident cause and best remedy to aid designers and equipment operators.
- Operators should include structural safety assessments in their loss control and safety management programs.
- To improve communication between the facility and civil/structural communities, and between in-house cross functional teams and support staff, there should be more cross-referencing between structure-related guidelines, facility-related guidelines and other drilling and production standards and related practices.
- There should be further development of disciplined, systematic evaluation techniques for the better control of human and organizational errors.

## Policy Considerations and Consequences

- Although the approach embodied in the draft API guidelines are efficient, flexible and credible, the following important policy decisions will have to be made by MMS for these guidelines to become operational:
  - what is a significant environmental impact?
  - how should the requalification process be integrated with the MMS ongoing inspection process?
  - what should "trigger" the requalification process?
  - how should industry and the public be consulted during the implementation process?
- Further research is warranted to identify and deal with the so-called "bad actors" (firms whose operating practices and policies consistently fall below minimal industry standards), but the term should not be applied indiscriminately to smaller or independent operators.

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# SECTION I

**Keynote Papers**

# REQUALIFICATION OF OFFSHORE PLATFORMS

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## Introduction

This paper presents some considerations related to requalification of offshore platforms as seen from Det Norske Veritas in Oslo. It is mainly based on our experience with the steel platforms installed in the Norwegian part of the North Sea.

The first production of hydrocarbons in the North Sea started at the Ekofisk Field in 1971. The design of the first jacket structures in the North Sea was based on use of the API rules. The first design rules for fixed offshore platforms from Det Norske Veritas were developed in 1974. They were based on an allowable stress format. A major effort was put into the development of rules based on limit state design with use of partial safety factors in 1977: *DNV Rules for Design Construction and Inspection of Offshore Platforms,* and the *Regulations for the Structural Design of Fixed Structures on the Norwegian Continental Shelf* by the Norwegian Petroleum Directorate. The Progressive Collapse Limit State was introduced in these rules to avoid catastrophic failure due to accidental loads such as ship impact, fire and explosion.

The status of these rules is explained as follows: The design of load-bearing structures for exploitation of petroleum resources in Norway is governed by the Norwegian Petroleum Directorate placed in Stavanger while the DNV rules are used for classification of structures around the world.

The Regulations by the Norwegian Petroleum Directorate have been revised several times since 1977. The existing DNV rules for classification of fixed offshore installations are dated 1989.

Both the DNV rules and the NPD regulations opened the use of reliability analysis in design in 1977. This concept has mainly been used for evaluation and calibration of safety coefficients; however, during the last 10 years it has been frequently used for systematic planning of in-service inspection (Lotsberg and Kirkemo 1989; Lotsberg and Marley 1992). In the same period a major research and development project on reliability analyses has been performed at Det Norske Veritas. Guidelines on use of reliability methods have been developed in addition to efficient computer programs for reliability analyses. A classification note on use of structural reliability analyses for evaluation of practical problems was developed in 1992 (Det Norske Veritas 1992).

## Safety Level in Design Rules

The limit state design from the 1977 rules is divided into:

* the *Ultimate Limit State*
* the *Progressive Collapse Limit State*
* the *Fatigue Limit State*
* the *Serviceability Limit State*

Safety coefficients for the *Ultimate Limit State* were aimed to give theoretical annual probability of failure less than $10^{-4}$ - $10^{-5}$ for manned platforms. This implied an environmental load coefficient equal to 1.3, (Fjeld 1978).

For unmanned platforms the Norwegian Petroleum Directorate may decide that the load coefficient for wave, current and wind loads can be reduced to 1.15. This is evaluated based on whether a collapse will:

* entail danger of loss of human life,
* cause significant pollution,
* have considerable financial consequences.

The safety factors used for design of structures are items governing the overall safety. However, accidents and gross errors are a major failure cause. Thus, in order to achieve a high safety level one has, in addition, to control the possibilities for accidental events and reduce the possibilities for gross errors during design, construction, and installation.

Accidental events are controlled through use of risk analysis as normally required by the Norwegian Petroleum Directorate (Norwegian Petroleum Directorate 1990). The Norwegian Petroleum Directorate may however, decide that the control of the limit state for progressive collapse (including accidental events) may be omitted if an overall evaluation shows that a collapse will not:
- entail danger of loss of human life,
- cause significant pollution,
- have considerable financial consequences,

the requirement to risk analysis is a function of consequence (similar to that of environmental load coefficients).

Risk analysis shall be carried out in order to identify accidental events that may occur in the activities and the consequences of such accidental events for people, for the environment and for assets and financial interests. The results of risk analysis shall be included as part of the basis for the decision-making process in the course of ensuring that the safety aspects of the activities are in accordance with requirements laid down by law or regulations, with the operator's safety objectives and acceptance criteria. Risk analyses that have been carried out shall be updated to follow the progress of the activities in order to ensure continuity in the basis for decisions relating to the safety of the activities. An updating of risk analysis includes: updating and extension of basic assumptions and data to include new experience.

The *Progressive Collapse Limit State* is checked according to the following steps:
- it shall be possible to document that the structure will suffer only local damage when subjected to accidental events,
- following local damage which may have been demonstrated under the above, the structure shall continue to resist defined environmental conditions without suffering extensive failure, free drifting, capsizing, sinking, or extensive damage to the external environment.

Gross errors are controlled by requirements to organization of the work, verification of design, and quality assurance during fabrication and construction. Robustness in the design as normally following from requirements to the Progressive Collapse Limit State also reduces the consequence of a gross error (Moan 1993).

Other items which are of significance with respect to safety are:
- material selection and documentation,
- welding procedure specifications and qualifications including testing requirements,
- requirements to non-destructive examination during fabrication.

The *Fatigue Limit State* is controlled by calculation of fatigue damage through the design life of the structures. Safety factors on fatigue life are used to achieve a similar safety level for this limit state as for the other limit states. These factors on fatigue life are dependent on consequence of failure, possibility for inspection and repair as shown in Table 1.

## Safety Level for Requalification

A safety level during requalification is aimed at keeping the safety level during a life time extension above the minimum requirements of the inherent safety level of the design code. For considerations on inherent safety level also see the section on Target Reliability.

## Items to be Considered During a Requalification

### Foundation

When evaluating the pile capacity at a platform life extension the overall conservatism in the initial design has to be considered such as the design procedure and deviations between soil test strength and *in situ* strength (Tjelta 1992; Dahlberg and Ronald 1993). One may consider the long term capacity (consolidation of clay). Further, one has to consider new information on soil data and experience with pile design.

Also information might have been obtained during installation of the piles that can be of significance for evalua-

**Table 1 Design fatigue factors on life.**

| Classification of structural components based on damage consequence | Access for inspection and repair | | |
|---|---|---|---|
| | No access or in the splash zone | Accessible | |
| | | Below splash zone | Above splash zone or internal |
| Substantial consequences | 10 | 3 | 2 |
| Without substantial consequences | 3 | 2 | 1 |

tion of the capacity of the piles. This includes pile installation data and information about penetration into end-bearing layers if available. The knowledge of the load effect might have been improved through measurements which also can be used for evaluation of the reliability of the pile design for a requalification.

Reliability methods based on Bayesian updating can be used for updating the capacity and the load effects (Ang and Tang 1975; Madsen et al. 1986). Guidance and a procedure on this should be developed. Work on a guideline is planned to be performed in a joint industry project at DNVI this year.

### Grouted pile/sleeve connection

The basis for the design equations has improved as the data base on grouted pile/sleeve connections has increased. However, the form of the design equations in the various codes are different and this leads to significant differences in the requirements for pile sleeve lengths (Figure 1) (Sele 1993). It is indicated by Sele that the design can be efficiently improved by reducing the modeling uncertainty in the design equations by using parameters in the equations better representing the physical behavior.

### Corrosion

The design of the corrosion protection system is normally made more conservative for the North Sea than for the Gulf of Mexico in order to reduce possibilities for fatigue cracking from corrosion pits.

The condition of the corrosion of a platform in the North Sea is determined as follows:
- by measurements of anode consumption,
- by potential measurements.

Provided that there are anodes left and that the potential is within an acceptable range, further inspection is not required. The lifetime extension of the corrosion protection system can be estimated based on the amount of anodes left.

The cathodic protection system will also normally be sufficient at the upper region of the piles. At lower depths the corrosion is prevented due to the lack of new oxygen. If the measured potential is not sufficient, then the actual elements are visually inspected. The corrosion should be rather serious before ultrasonic measurements of the member wall thicknesses are required. The most serious corrosion attacks are expected to occur at the nodes, as welds are more subject to corrosion than the base material. Also the cleaning of the nodes in connection with detailed inspections may reduce calcareous deposits and favor corrosion at these areas.

The cost of reinstallation of anodes is a magnitude larger during in-service as compared with the fabrication of the platform.

The splash zone and the atmosphere region is painted and is controlled by visual inspection. Possible damages of the paint are looked for and, in case of severe damages, thickness measurements of the members should be performed. Repair of the painting in the splash zone is considered rather costly. Use of habitat may be considered.

Research needs: Improve the coating to be used in the splash zone for the construction phase, for permanent use, and for repair painting.

### Static Strength

For evaluation of the static strength an updating of the existing geometry of members and modes including deterioration due to corrosion and dents and damages due to impacts should be performed, e.g., with the capacities from test data such as Landet and Lotsberg (1992). In case of subsidence it is also important to update the water level. The capacity of the structural elements may be updated through information obtained during fabrication such as updating the yield strength data based on material certificates. The testing strain rates and the number of tests should be included in such updating.

The loads should be updated including the latest information on weights, loaded areas and environmental data. Then structural analysis should be performed and design checked based on the latest revised rules or the rules used for the original design if they have not been experienced to be unsafe. The reanalysis may be based on a traditional design approach where the capacity of single elements is checked against the load effects from quasi-static linear elastic frame analysis.

For redundant structures the ultimate capacity can hardly be assessed without performing nonlinear analysis in terms of nonlinear geometrical and material behavior (Figure 2). Improved knowledge on acceptable strain at fracture for real structural components at the ultimate limit state is wanted for evaluations of acceptance criteria for this type of analysis. Also considerations on low-cycle fatigue become important when using such analysis for a design verification (Figure 3) (Stewart 1993).
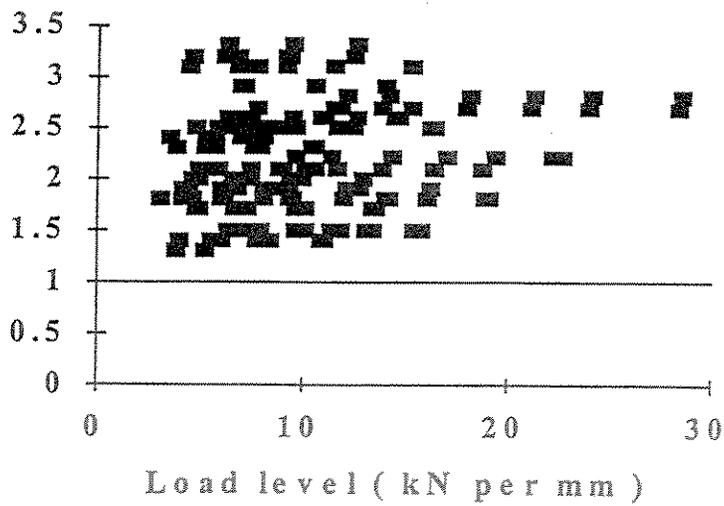
PILE SLEEVE
LENGTH
API / DNV



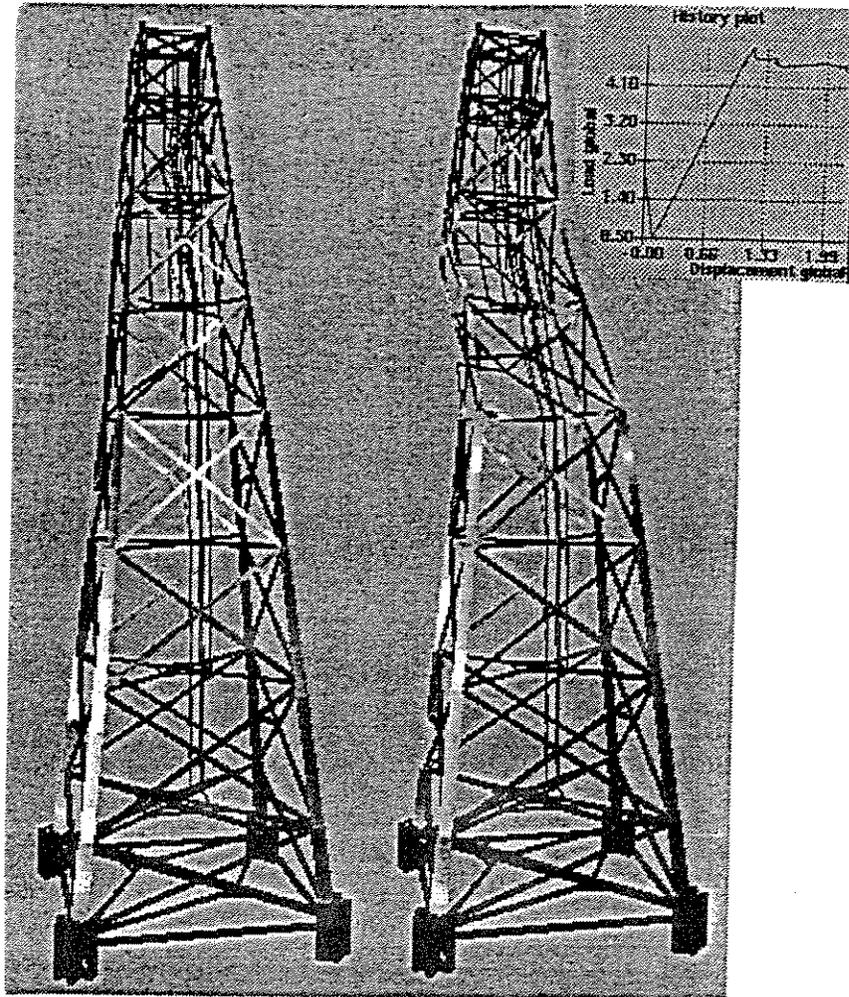Figure 1  Required pile sleeve lengths for a number of different design cases for API and DNV.

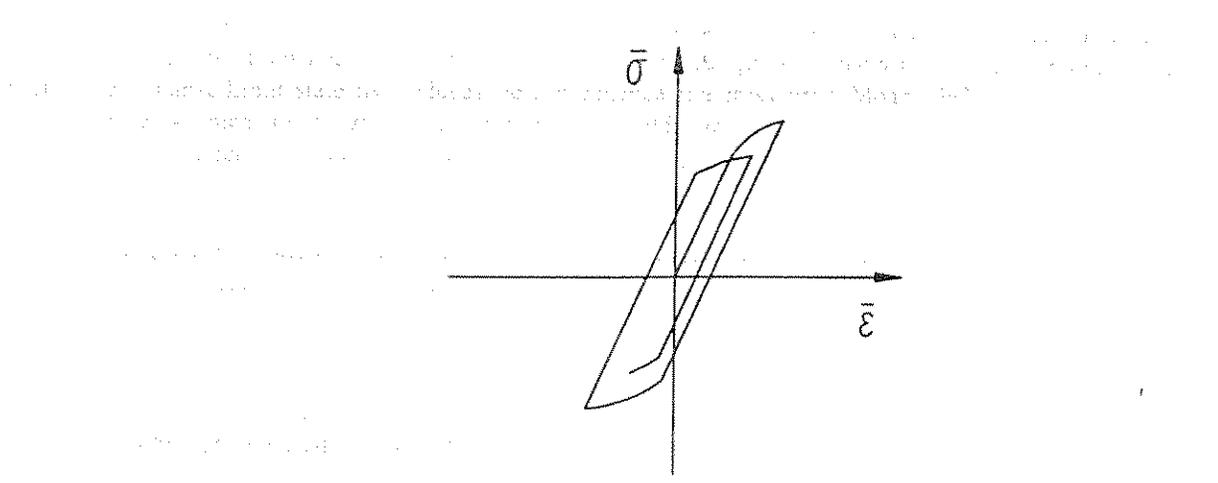Figure 2 Nonlinear collapse analysis of jackets.

Figure 3 Cyclic loading.

For older installations one should keep in mind that there is less documentation on:
- through thickness properties of the material,
- fracture toughness,
- nondestructive examination of the welds after fabrication.

Use of a new analysis technology where the structural capacities become more utilized also implies that the probability of failure may increase. This may be acceptable provided that it can be documented that the traditional design approach has been conservative and that one by new knowledge is able to predict the inherent safety in the new procedure that is still acceptable. This requires a good understanding of actual failure modes for the structure and a good knowledge of the main parameters governing the failure modes. It further requires knowledge on the failure criteria for the different failure modes. Reliability analysis may be a helpful tool for assessment of allowable increase in utilization from that of existing design practice (Sigurdsson et al. 1994). This requires a good understanding of the different physical failure modes together with statistical distributions of the most important parameters involved.

Items to be considered for a documentation of a nonlinear structural analysis as basis for acceptance of a design or a requalification are:
- Considerations that the nonlinear program used can simulate the actual physical behavior of the considered structure and structural details such that the actual failure modes are captured.
- Effect of local details for end restraints or force-deformation relationships for the joints.
- Effect of fabrication tolerances (member straightness and joint eccentricities) and residual stresses on buckling capacity.
- Failure criteria in terms of maximum strain at failure for components containing relevant imperfections, e.g., from welding and at regions containing notches.
- Repeated yielding in case of reversed loading due, e.g., to wave action.
- Sensitivity of input parameters and analysis assumption for evaluation of acceptance criteria (reliability analysis may be used).

This means that an evaluation of the structural integrity of a structure based on use of nonlinear computer programs will require more effort by skilled engineers than that of traditional analysis since an accepted design procedure or code based on use of nonlinear analysis of offshore structures does not at present exist. Establishing a guideline for this is a recommended area for research and development.

### Fatigue

The total number of reported fatigue cracks in primary structures in the Norwegian part of the North Sea is small (Hamre et al. 1991). However, several fatigue cracks have been observed in secondary structures not designed for fatigue.

As the computer capacities have increased the refinement of the fatigue analysis has improved (Gibstein et al. 1989). Traditional fatigue analysis procedures have been based on approximate values of stress concentration factors related to single braces. A procedure has been developed in-house in which a finite element analysis of the considered tubular joint is integrated with the global analysis of the platform. This allows determination of the stress at each considered point as the sum of simultaneous contribution from all members of the joint for each wave position. This reduces conservatism introduced in the hot spot load effect calculation.

Reliability methods have been shown to be efficient for linking of in-service inspection to that of probability of fatigue cracking (Lotsberg and Kirkemo 1989; Lotsberg and Marley 1992). The reliability after an inspection event is updated based on the results of the inspection using Bayes' theorem. An example of such an updating of probability of failure is shown in Figure 4 for a node in a jacket having a design life equal to 10 years. The jacket was installed in 1972. The considered node has been inspected by MPI (Magnetic Particle Inspection) in the years 1982 and 1986. Another inspection is planned to be carried out in 1994, and provided that fatigue cracks are not found, the probability of failure is updated as shown. System effects are included for evaluation of required safety level in terms of reduction in static strength by removal of the considered element.
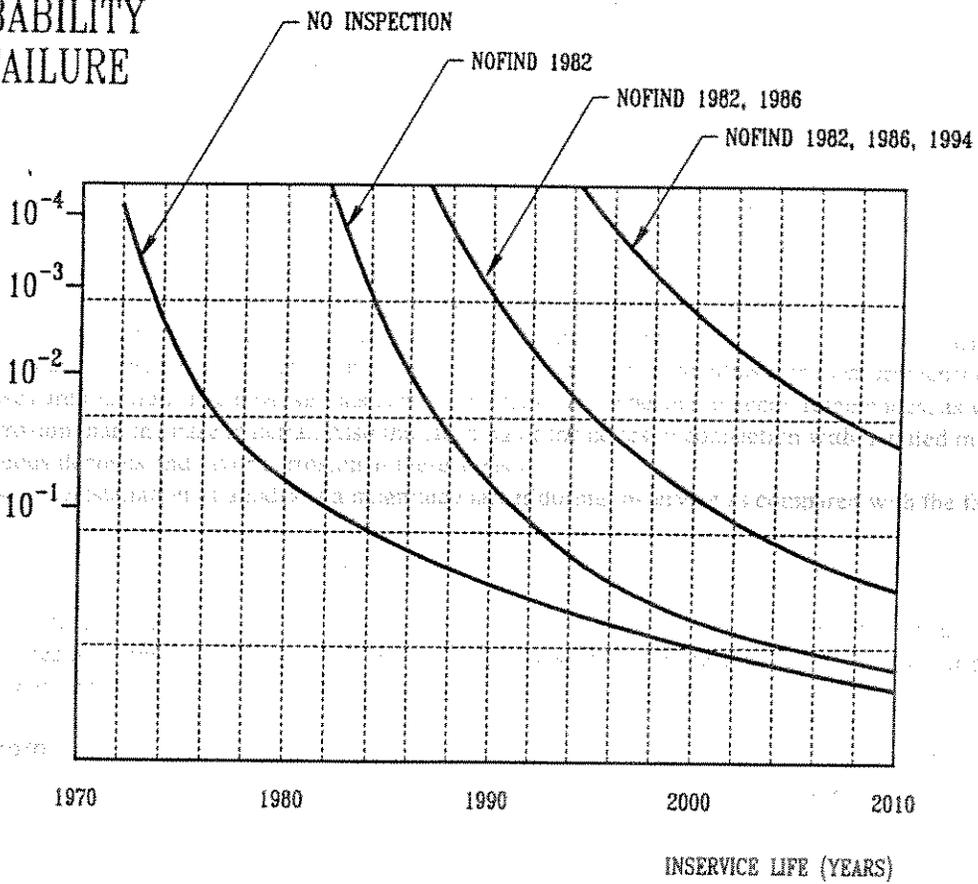
Figure 4  Probability of fatigue failure as function of time for a node having a design life equal 10 years.

# Reliability Analysis

## *Target reliability*

In principle a requalification based on reliability analysis can be performed using the following format

$$\beta_{calculated} \geq \beta_{target}$$

where $\beta_{calculated}$ is the reliability index calculated by reliability analysis and $\beta_{target}$ is a target value that should be fulfilled for the design to be found acceptable.

Some different approaches for the assessment of target reliability levels are considered below.

Requirements to target reliability level can be determined from cost minimization considering cost of design and construction and consequential costs of damage or loss of structure. In this type of model eventual intangible costs should be calibrated by applying the model to well known and accepted cases.

Another approach is the comparison of probabilities of failure from reliability analysis with that of risks associated with other activities in the society. For a comparison between the risk levels obtained from statistics and calculated probabilities of failure obtained from reliability analysis, the following differences between these two procedures should be kept in mind.

- Reliability analyses do not normally account for gross errors which are generally the main reason for structural failures as reported by statistics.
- Statistical and model uncertainties are normally included in the reliability analyses. These uncertainties may lead to larger failure probabilities than those due to the physical uncertainty alone.
- The results from the reliability analysis depend in some cases on the modeling and description of the tails of the distributions, which are difficult to assess in most cases.

Thus, the calculated value of the reliability index is a function of analysis methods used and distributions assumed for the analysis. Therefore, one should not directly compare reliability indices as obtained from different models and sources. A calculation of $\beta_{calculated}$ and $\beta_{target}$ should be based on similar assumptions about distributions, statistical and model uncertainties. Due to dependence on assumptions and analysis models used for reliability analysis the word "reliabilities" in a frequency interpretation of observed structural failures can not be used in a narrow sense. Due to the unknown deviations from the ideal predictions the computed failure probabilities are referred to as nominal failure probabilities.

However, by relating the reliability analysis to well known cases and accepted design practice (the inherent safety level for maximum allowable utilization) it is possible to use reliability analysis for a requalification without specific requirement to target reliability level. On this basis reliability analysis was opened for use in the NPD regulations.

## *Bayesian Updating*

Bayes' theorem is useful for revising or updating the calculated probability of failure as more data and information become available (Ang and Tang 1975). Reliability analysis is based on a limit state function representing the failure criterion through an event $G \leq 0$. For example the information that cracks were not found during an inspection is included through the event $H \leq 0$. Through Bayes' theorem it follows that an updated failure probability is obtained as

$$P(G \leq 0 | H \leq 0) = \frac{P(G \leq 0 \cap H \leq 0)}{P(H \leq 0)}$$

where the probability in the numerator is calculated for components in a parallel system. The situation where a crack is found during an inspection, or where measurements are performed, or where data are observed, can be described in a similar way by calculation of $P(G \leq 0 | H = 0)$ (Madsen et al. 1987).

# Summary and Conclusions

For a requalification of a platform:

- Update the risk analysis in case of new experience that may be of significance to the integrity of the structure.
- Evaluate the safety of the structure by including the reduction in capacity due to deterioration and by updating the structural reliability due to knowledge on data from its service experience.

- Evaluate the safety of the structure by performing refined analysis to reduce analysis uncertainty with respect to the Ultimate Limit State, the Progressive Collapse Limit State and the Fatigue Limit State if required to document sufficient safety.
- Keep the safety level also during a life time extension above the minimum requirements of the inherent safety level of the relevant design code.
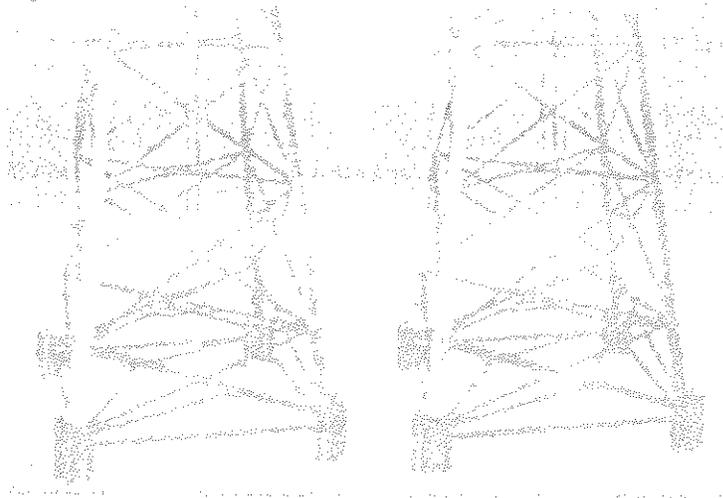
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# ON THE REVIEW OF OFFSHORE LEGISLATION AND CERTIFICATION ON THE UKCS

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## Introduction

On behalf of the Offshore Safety Division of the United Kingdom Health and Safety Executive, I am pleased to be addressing this important international workshop today to present an outline of our progress on the review of offshore legislation generally, and the review of certification in particular.

Lord Cullen's report on the 1988 Piper Alpha disaster (Her Majesty's Stationary Office 1990) was published on 12 November 1990, and the 106 recommendations made in the report were fully accepted by the United Kingdom Government. The report was critical of the existing prescriptive offshore safety regime and recommended it be re-formed in favor of goal setting regulations made under the Health and Safety at Work Act (Her Majesty's Stationary Office 1974) which would achieve "...good and well maintained management of safety by all operators (of offshore installations)" (Her Majesty's Stationary Office 1990).

Lord Cullen recommended that the new offshore regulatory regime be based on two complementary elements:

- a requirement for operators and owners of offshore installations to prepare safety cases for each installation, for submission to and acceptance by the Health and Safety Executive (HSE); and
- a program to progressively reform existing offshore legislation by regulations in a more modern form, expressed mainly in terms of objectives (or goals) to be achieved, which would sit more easily with the new safety case regime.

Under existing legislation, offshore installations cannot operate on the United Kingdom Continental Shelf (UKCS) without a valid Certificate of Fitness (CoF) issued by one of six certifying authorities (CAs), which are appointed by HSE. In his report, Lord Cullen suggested that when the new safety case regime was in force offshore, and the associated requirements for operators to demonstrate their safety management system and audit compliance with it, the need for the certifying authorities to continue to perform the same functions as before should be reappraised. He considered that certification might sit uneasily with the safety case regime, under which responsibility would rest with risk creators to identify hazards, assess risks and take appropriate measures, as opposed to relying on a third party to ascertain compliance with regulatory requirements. Consequently, as part of the overall review of offshore safety legislation in the wake of the Cullen Report, the Offshore Safety Division of H.S.E. is undertaking a thorough review of the certification scheme.

In the following I shall outline where we are in the review of legislation and then give some background and indicate some factors which will need to be taken into account in the review of certification.

## The Legislative Review

The first stage of the legislative review was achieved by the Safety Case Regulations (Her Majesty's Stationary Office 1992) which came into force this year and which promote an integrated, risk based approach to the control of hazards offshore. They require operators and owners of offshore installations to establish an effective safety management system, and to identify hazards which may give rise to a major accident, assess the risks arising from those hazards, and take steps to reduce those risks to as low as is reasonably practicable (ALARP). Operators (of fixed installations) and owners (of mobiles) are now submitting safety cases. We expect to receive over 200 safety cases, which have to be assessed and accepted by HSE by 30 November 1995. After that date no installation will be able to operate in United Kingdom waters unless a safety case has been accepted by HSE.

In August last year the Health and Safety Commission announced plans for the next stage—the review and

reform of offshore legislation. The approach we are taking to the reforms follows the Cullen Report recommendations in that:

- The reformed regulations must complement, and not duplicate, the safety case.
- Regulations should be genuinely goal setting, though some specific provisions will need to be included, to implement Cullen recommendations and to give effect to provisions in a European Directive (Official Journal L. 348 1992).
- The HSE will, so far as practicable, pursue consistent regulatory approaches onshore and offshore. There should be distinct offshore requirements only where justified by the special characteristics of the offshore workplace or hazards.
- Consultation on new goal setting regulations should be taken forward rapidly, to minimize the period of uncertainty about the future regime as a whole.

New offshore regulations are planned on the prevention of fire and explosion and emergency response on offshore installations; management and administration; and design and construction (including certification). Although Lord Cullen recommended separate regulations on fire and explosion protection and on evacuation, escape and rescue, we have developed a single, integrated set of regulations dealing with the prevention, control and mitigation of fire and explosions and response to emergencies (H.S.E. 1993). These are the first of the new regulations to be developed. Formal consultation began with the publication of the consultative document on 8 September and it is intended that the Regulations will come into force next autumn.

The Management and Administration Regulations will cover general matters relating to administrative and working arrangements, for example permits to work, about which Lord Cullen made specific recommendations. Work is well underway on these proposals and it is proposed that the Consultative Document be issued in January 1994; with regulations coming into force in December 1994.

The Design and Construction Regulations will reform the existing Construction and Survey Regulations (Her Majesty's Stationary Office 1974). The reform includes a review (as Lord Cullen recommended) of the offshore certification scheme established under those Regulations. The formal Consultative Document will be issued in the Spring of 1994 and regulations will come into force in mid-1995.

# The Review of Offshore Certification

## Background

Offshore certification arrangements need to be seen in the context of the major accident potential offshore, particularly taking into account the hostile environment of the North Sea; the potential for rapid escalation of dangerous circumstances and the difficulties of evacuation, escape and rescue; the different functions of a large, integrated platform - a drilling rig, process plant, hotel, heliport; and the range of types of installation operating on the United Kingdom Continental Shelf.

Changes to the certification scheme will involve changes to the existing regulations. We are therefore conducting the review in conjunction with the development of new offshore Design and Construction Regulations which are intended to set out goals for design and construction of offshore installations to ensure their integrity throughout their entire life-cycle. (By 'integrity' we mean the state in which the installation can safely fulfill its purpose).

The following are some key elements of the offshore certification arrangements:

- HSE does not certify offshore installations. This is undertaken by independent Certifying Authorities appointed and audited by HSE.
- The scope of the offshore certification scheme includes complete structures, plant and systems, including process plant.
- The scheme is not a product certification scheme. It goes much wider. Certifying authorities check conformity against requirements in Regulations (Her Majesty's Stationary Office SI 1974). The requirements in the Regulations have been supplemented over the years by a large quantity of guidance.
- The certification scheme involves not only independent assessment of the complete design and initial survey of the construction against design, but also in-service surveys. It therefore provides continuity through the life-cycle of the installation.
- The certification scheme concentrates on hardware. But it also, crucially, provides an overview of the many systems which make up an offshore installation and a means of ensuring that the interactions between different systems are considered. For example, changes to the process system which introduce new equipment, might

affect the loadings on the primary structure. These interactions need to be understood and considered, and certification provides a mechanism for this. The certifying authority provides an integrated overview rather than piecemeal certification of individual items of equipment, parts of the structure or systems.

- Each offshore installation is different, so the system therefore has to cope with the unique nature of each installation. We are not dealing with batch products.
- The offshore industry is international. Many installations are designed and constructed outside the United Kingdom. Certification arrangements therefore have to be capable of handling this international dimension.
- There is also a marine dimension, in that mobile installations are also ships, and therefore subject to maritime classification requirements. Five of the six certifying authorities also act as Classification Societies.

The certifying authorities are quasi-regulators. They have powers to refuse a CoF, or place limitations on operations as a condition of granting or maintaining a CoF. Hence, Lord Cullen's consideration that the scheme might sit uneasily with the safety case regime which he recommended for the UKCS.

### Factors Influencing the Review

There are a number of factors to be taken into account in the review. Any new arrangements should be consistent with the general philosophy of modern health and safety legislation. For example, the underlying principles of the health and safety legislation already in place should be followed. Thus health and safety standards should be maintained or improved by any new scheme; and operators must demonstrate that they have in place an adequate safety management system and have taken measures to reduce the risks from major accident hazards as low as reasonably practicable (ALARP). Verification arrangements might contribute to this demonstration, but do not constitute an alternative to it and must not be regarded as such. (We have used 'verification' as a generic term to cover reviewing, inspecting, testing, checking, auditing or otherwise verifying and documenting whether items, processes, services or documents conform to specified requirements - definition from BS 5882: Total Quality Assurance program for Nuclear Installations).

Any change in approach should not lead us to discard the strengths of the existing certification scheme, such as the expertise and experience which the certifying authorities can bring to bear. It is very difficult to quantify the safety benefits of certification since we have no recent experience of a regime without it; but there is ample evidence of problems identified by the certifying authorities at the design and construction stages which would have had major implications for cost and production, as well as safety, had they gone undetected.

Any new offshore verification arrangements must not bring us into conflict with present and likely future European requirements, while at the same time we want to ensure that we can achieve our own safety objectives. Many European community developments are of interest offshore, such as the directives covering equipment and the trend towards the development of European standards for certification, inspection and accreditation to give assurance of competence in a given field of work is growing in the United Kingdom, partly in response to the European drive to remove barriers to trade.

We have explored other models from other industries or in other countries with offshore industries. Some countries have followed the United Kingdom approach; some do not have an equivalent. We have not identified other models which we can take off the shelf and transplant offshore.

New arrangements must also complement, not cut across, the marine classification and flag state requirements applied to mobile installations.

We have a number of stakeholder interests to consider. The concerns of industry, design, fabrication and installations contractors, the six certifying authorities, and the work-force cannot be expected to coincide in every respect, and we must seek a way forward which will be workable and gain the cooperation of all parties.

### Timetable for the Review

We are in the process of developing proposals for the new arrangements for verification offshore and will be discussing these with interested parties over the coming months. They will be integrated into the new regulations governing design and construction to ensure the integrity of offshore installations, which will be the subject of a formal consultation document we aim to publish in mid-1994; the regulations should be in place by mid-1995.

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# ASSESSMENT OF EXISTING STRUCTURES: DEVELOPMENT OF AN INDUSTRY PRACTICE

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## Introduction

In recent years, some of the focus in offshore facilities has shifted from new technology and advances in new facilities to the maintenance and determination of adequacy of existing structures. The offshore industry is relatively young by structural standards getting its Gulf of Mexico start less than 50 years ago. It wasn't until 1969 that the industry had its first offshore design standard with the publication of the 1st Edition of API RP 2A. Prior to that time, everyone was on their own.

As the industry nears the diamond anniversary of the 1st Edition of its "bible", new challenges appear. The industry has seen hurricanes of considerable magnitude rip through the heart of the offshore arena with significant impact in both the early 1960s with Hilda and Betsy and again in 1992 with Hurricane Andrew. But it is not only the storms that bring the challenges, it is also the aging of the structures, changing public and governmental policy pertaining to the environment and life safety issues, the world economic picture and even new technology extending the life of the old oil fields or finding new pools of oil underneath existing fields.

There are over 7,000 offshore structures worldwide with more than 3,800 of these in the Gulf of Mexico. A recent study (James K. Dodson Co. 1993) indicated 1,157 platforms in the Gulf of Mexico were installed prior to 1974. With the oil economics of today, fitness of existing facilities for future operations becomes a high priority as the capital investment in these facilities ranges from thousands of US dollars to several hundred million U.S. dollars.

The American Petroleum Institute has been the leader in development of design standards for offshore production facilities. Since the release of the RP 2A 1st Edition, 19 subsequent editions have been published incorporating lessons learned and new technology.

With the new challenges comes changing standards. In early 1992, the API Offshore Standardization Committee's subcommittee on Fixed Offshore Platforms charged a task group to develop sections for RP 2A to address the topic of Assessment of Existing Platforms To Demonstrate Fitness For Purpose.*

## Charge

The background, core philosophy and initial work of API in the area of existing structures has been outlined in earlier papers (Wisch 1993; Wisch 1992). In essence, the API philosophy has been to establish guidance to designers as to what constitutes good practice in the design of offshore structures while allowing flexibility to the owner and designer in designing and building a facility to meet the owner's needs. A fundamental objective has been to bring a baseline level of consistency to the industry without establishing criteria, procedures or guidelines that penalize.

In meeting the present challenge as well as keeping with the historical objectives, the Task Group was given several charges:

- develop text consistent with the intent of RP 2A for the assessment of existing facilities,
- develop the text in the framework that the process is general and can be applied in areas of the world other than the Gulf of Mexico; i.e., a framework that can be passed to the ISO group for consideration as the basis for assessment section in the upcoming ISO offshore standard,
- develop for inclusion into both the WSD and LRFD versions of RP 2A,

*Editor's note: A draft of the API standard discussed in this paper is included as Section IV in these Proceedings.*

- use of Performance Service Levels (PSL's) based on consequence was preferred as opposed to a single acceptance criteria,
- provide a process that linked inspection levels to the degree or sophistication performed in any structural analysis that provides adequate "as is" information without having an inconsistency between inspection and analysis assumptions,
- do not set environmental policy,
- use sound engineering practice and avoid emotional criteria,
- follow the general principle that the criteria should follow generally accepted practice focusing on life and environmental safety with the further refinement due to economics left to the owner.

## Makeup

In accepting the challenge, a Task Group was formed containing 14 members, 20 corresponding members and seven sub-task groups. The seven sub-task groups focused on specific technical areas and involved an additional industry staff. In excess of 70 individuals representing large, medium and small owners, consultants, academia and regulatory agencies have been working together in formulating the proposed guidelines.

## Issues

A number of issues come up when dealing with existing facilities. While most are philosophical and emotional issues, the final product in a code or standard must provide some rational, objective guidance and/or criteria. Most of the charges listed earlier fall under the philosophical umbrella. It fell to the task group to draw up the objective text.

Assessment of existing facilities is not unique to the offshore community. The electric power generation industry and the chemical industry have been addressing the problem. In the public sector, many building codes have some mention of existing facilities. There was considerable focus on existing facilities in California following the 1971 San Fernando earthquake where many public infrastructure facilities were severely damaged or destroyed. The 1989 Loma Prieta earthquake again focused attention on existing infrastructure, particularly the transportation segment after the general public saw the impact of the damage to the Bay Bridge and the Cypress Viaduct. Again in 1992, the damage due to Hurricane Andrew in the Gulf of Mexico brought about questions concerning adequacy of facilities.

In stepping back to look at the issues, some fundamental questions come to the forefront: Who "owns" the development of public policy and the societal acceptance values? Who sets these values?

Public policy issues should be driven by view of the broad picture and objectives. It is unwise to allow public policy to be developed by special interest groups having exclusive benefit or from groups having no direct economic or safety ownership.

Often the view of acceptable lies in the eye of the beholder. The degree of personal involvement, whether being an owner, a neighbor or potential impacted party from either an economic or safety perspective, has a direct relationship on acceptable. Public policy is the effective balance of all the views.

Public policy is owned by the public at large. It is often stated by governmental or regulatory bodies through adoption of codes, by legislation or by executive orders.

It has been the objective of the Task Group and its parent committees to develop a process that accounts for currently perceived public policy and permitting this policy to be incorporated into a process to be followed for offshore facilities. The process is then offered to the regulatory agencies as a basis for part of the overall policy regarding existing facilities.

Where does economics enter the picture and on what basis in establishing public policy? It is often heard that public policy should not be sensitive to economics, that it should focus on the issues of life and environmental safety. However, it is quickly obvious that economics has been a key factor in the establishment of codes and criteria.

Examples abound in establishing this link. Following the 1971 San Fernando earthquake, California adopted a plan to strengthen many bridges only to see the plan stretched out over 17 years as funding was continually reduced over time. Building codes allow for existing buildings to remain provided they are maintained to the meet the codes in effect at the time the facility was built. Several California building codes require an existing structure to withstand 75% of the lateral forces of a new building when the structure undergoes modification.

It has been the conscious decision of the API groups to let the economics be dictated by market forces, i.e. do not allow economics to be directly incorporated into the acceptability criteria. Each owner has different economic incentives and views risk from a different perspective. It is not within the scope of the offshore standards committee to

establish uniform risk taking or risk aversion standards relative to economics.

What does the public accept as societal risk values? Reviews of literature indicate a sliding scale on acceptable limits. The public generally accepts some 50,000 traffic fatalities in the U.S. each year when most fatalities occur one or two at a time. It is generally not acceptable to have 300 fatalities in a plane accident. There is a general impression that more focus is placed on the reduction of the plane accidents than on the reduction of traffic fatalities. A single event with large consequences is unacceptable, while a series of unrelated events with much smaller individual, but collectively greater, consequences can be acceptable. There is a perceived direct impact on costs and freedom for automobile safety to individuals whereas the airline safety issue appears more indirect to most of the public.

From an overview perspective, where do offshore facilities fit in relation to onshore facilities for which some generally accepted criteria have been established? Offshore facilities have generally been viewed by the public in a class somewhat to their own. On land, there is a multitude of facilities (e.g., single family homes, multifamily homes, apartments, warehouses, large office complexes, schools, hospitals, public gathering facilities, manufacturing plants, etc.) for which codes or practices exist. Offshore facilities differ in their location, but not necessarily in their manning or industrial process compared to land based activities.

The "Seismic Safety Requalification of Offshore Platforms" report (Iwan et al. 1992), commonly called the THIC report, provided some overview as to the relationship of offshore facilities to land based facilities relative to the seismic design/assessment process. The starting point for the Task Group's evaluation came from this report.

What is the relationship between a single facility, a fleet of facilities and the collective whole? What are acceptable fleet performance characteristics pertaining to life safety, environmental safety and delivery of product? This question was dramatically illustrated with Hurricane Andrew. Two types of fleets were involved. First was the complete fleet in the Gulf of Mexico, in excess of 3,800 platforms. Second were the fleets of each individual owner. Some owners had in excess of 200 facilities while others had as few as one. Success or failure of any given facility is binary, it either succeeds or it fails. The regulatory process should address the collective fleet performance relative to safety and overall delivery of product. Owners address safety and the economics of their individual fleets.

In Hurricane Andrew, only part of the entire Gulf of Mexico fleet was affected. If viewed from a whole, less than 5% of the structures suffered significant structural damage. Production was temporarily halted but in excess of 90% of the pre-storm production was back in service within 90 days, and not all of the lost production was due to structural damage: much was due to pipeline or transmission situations not related to any structural problem. From this view, the fleet of structures performed very well.

It was seen that a major storm had no catastrophic impact on the energy supply to the public infrastructure. The impact on deliverability and public reliance is one element that enters public policy and varies from region to region.

Each individual operator has to assess the performance of their fleet. Overall economic impact varies from operator to operator and falls outside the scope of the RP 2A objective.

What constitutes *acceptable* for existing versus new design structures? Engineering practice over the recent history has seen building codes and other standards and practices evolve over time incorporating new technology. The resulting changes have generally resulted in safer structures with increasing criteria having the effect that older structures would not meet today's codes or standards. In essence, older structures have lower margins of safety than their more contemporary counterparts and the public accepts this situation.

Not only is this a generally accepted practice, but many codes explicitly state the relationship in various ways. An example of this for a building exposed to hurricanes is Section 104 of the South Florida Building Code, entitled Application To Existing Buildings. There are several provisions identifying requirements when additions or alterations are made to the buildings. Section 104.8 is entitled Existing Buildings and states: "(a) Any existing building, which complied with the Code in effect at the time of its construction or at the time of establishment of its present Group of Occupancy, may continue in its approved Group of Occupancy but such continued approval may not be construed to prohibit the inspection authority from at any time requiring that the minimum standards of safety such as, but not limited to, strength, egress, fire-resistance, openings in walls, or electrical, or plumbing, mechanical or elevator equipment or fire extinguishing or apparatus be maintained during the period of use of the building in accordance with minimum standards at the time of construction."

This section is typical of many locally applied building codes. Similarly, as noted in the THIC report for seismic areas, a 25% reduction in base shear is the target level on structures in seismic zones undergoing modifications. In regards to bridges, AASHTO (American Association of State Highway and Transportation Officials) is drafting new code provisions to address existing bridges. Again, the acceptance criteria is more stringent for new bridges than for existing bridges. Part of the justification, for both the building and bridge codes, has been economics.

While many precedents exist for lesser criteria for existing than for new buildings, no clear trend or consistent criteria has been observed regarding the quantitative difference.

## Guidelines

After reviewing the general questions, some principles guiding the API work developed. These principles are consistent with the premises upon which RP 2A has developed over the years and in accordance with the above discussion. In addition, they recognize the changing aspects of technology, economics and societal views while still being based in sound engineering practice.

- Good engineering judgement was to be followed relative to generally accepted industry practices,
- The resulting process based on accepted industry practice would be no less than equivalent land based practice,
- Environmental "baselines" would not be established in this document,
- A minimum level of structural safety would be incorporated to ensure safe access to unmanned facilities by field personnel,
- Consequence would be explicitly accounted for by use of multilevel acceptance criteria,
- A multistep process, each step being less conservative that the previous step, would be used to allow owners flexibility,
- Explicit, detailed analyses involving probability computations would be unacceptable due to the number of facilities, the cost of this level of work and the limited availability in the industry of performing the level of work,
- The process would be developed with the full knowledge and interaction of U.S. regulatory agencies,
- The process would be limited to structural behavior and risks associated with structural performance.


## Process

The objective was to devise a procedure which could be flow charted to provide a road map for owners, engineers, consultants and regulatory agencies when evaluating facilities undergoing the assessment process.

The approach can be categorized as:

> Condition Assessment
> Categorization of facility type
> Determination of exposure category
> Assessment and Fitness Determination
>> Screening
>> Design Level Check
>> Ultimate Strength Check

The approach is based on low level engineering effort at the screening level which provides a conservative approach followed by increasing levels of engineering and inspection efforts but decreasing levels of conservatism as the Design Level Check and Ultimate Strength Check. This approach is illustrated in Figure 1.

Many of the key aspects are discussed in the workshop session reports contained in the proceedings of this workshop. The reader can refer to these papers for technical details.

The flow chart discussed at this workshop has undergone seven cycles in the past 12 months as the Task Group has deliberated. One of the key factors in the sequence of flow charts has been the goal of providing a "workable" flowchart, not just a philosophical guidance. The Task Group tested the flow charts against this concept continually. The constant testing and evolution has been a trademark of API Offshore Standards. The draft issue of the proposed new API guidelines needs discussion and testing to further refine the process.

A second key factor was the constant influx of new information: several Joint Industry Projects, individual company initiatives and a continual stream of information from Hurricane Andrew assessments. By focusing on the "workable" concept and incorporating new information, the flow charts evolved.

A summary of each of the eight versions follows. The intent of the summary is to briefly indicate the evolution of a process, and the rationale involved in deriving a practical consensus based modification to accepted industry practice as captured in RP 2A. Though details of each version are not included, it is the intent to identify the process changes as the process is tested with new information and the practicality of application to real world structures.

*Version 1*

# Evaluation Process



Condition Assessment → Categorization of Facility → Exposure Category → Assessment

Conservatism

Screening
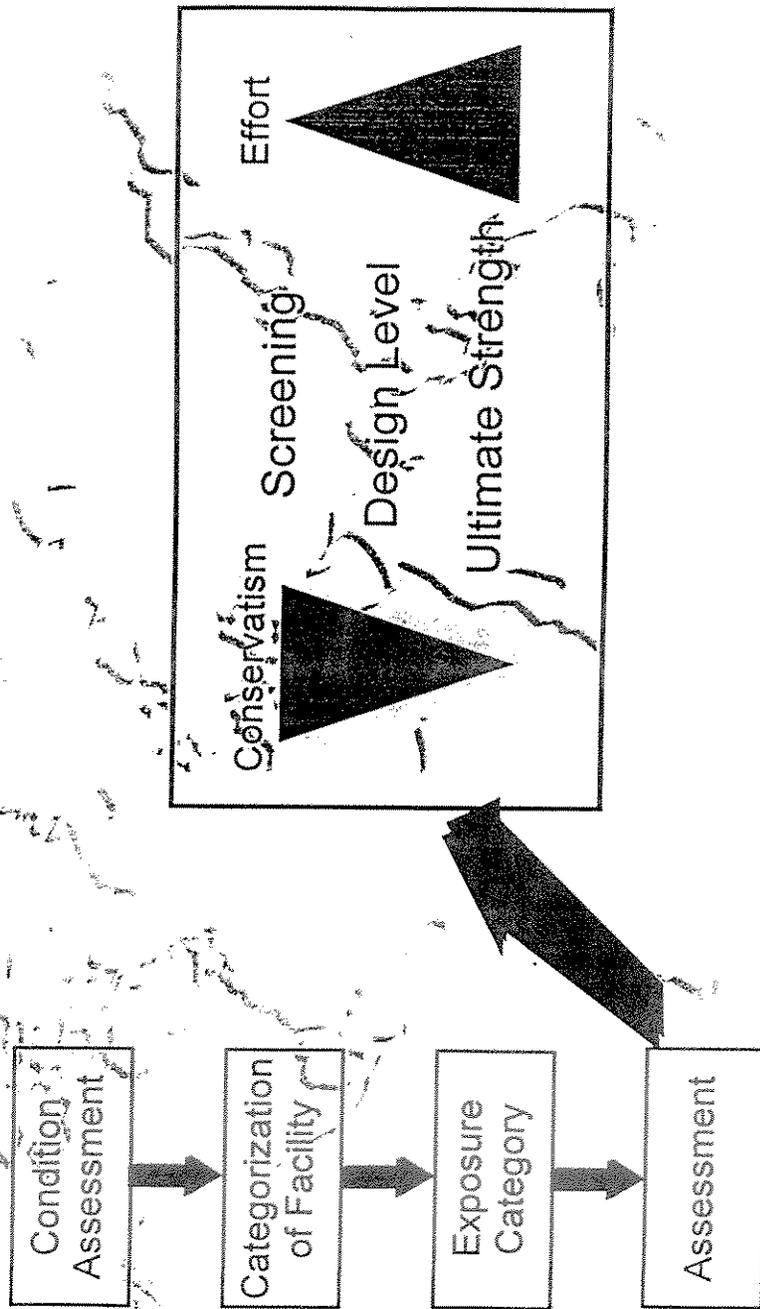Design Level
Ultimate Strength

Effort

Figure 1 Evaluation process.

The first version was modelled after the process presented in the Seismic Safety Requalification of Offshore Platforms report. There were separate charts for Life Safety and Environmental Safety. Three tiers of assessment were provided:

Screening - facility meeting a stated prior edition of RP 2A,

Analysis - meeting 20th edition of RP 2A with reduced wave heights,

Risk Analysis - computed probability of failure less than 1/1000 ( Pf < 0.001).

In the environmental safety area, the "acceptable" spill size associated with rare failures was quantified. Computed risks for SSSV (Sub-Surface Safety Valve) and pipeline valve failures could be used to justify acceptance of platforms. No geographic distinctions are made regarding acceptance criteria. Figures 2-a and b illustrate the Version 1 flow charts.

## Version 2

The provision for Risk Analysis is replaced with ultimate strength analysis, in keeping with industry capability. Return period based criteria were specified for linear and ultimate strength analyses. Screening is based on either the platform having been designed to a specified edition of RP 2A, or using a "simplified approach". A minimum safety flowchart is added to assure adequate safety for temporary manning conditions. No "acceptable" spill size associated with rare failures is specified, in keeping with API guidelines to technical standards committees. SSSV and pipeline valve reliability are not used as part of the assessment process.

## Version 3

Life safety, environmental safety and minimum safety flowcharts were combined into a single flowchart using a platform classification table for a more logical flow.

## Version 4

With the exception of minimum consequence platforms, criteria based upon explicit wave height curves or factored 100 year lateral loadings were introduced. This came about due to the wide company-to-company variability in estimating return periods associated with factored 100-year loads. The adequacy of minimum consequence platforms could be based upon prior exposure.

## Version 5

Platforms were categorized according to a unique exposure category, eliminating the earlier situation where a platform could be placed into more than one category. The design basis check was distinguished from the screening analysis and required that the reference level loading from RP 2A 9th edition be utilized. Separate assessment criteria tables were provided for Gulf of Mexico and other U.S. locations.

## Version 6

The use of exposure category designations (i.e., A=1-3, B=E,N) was eliminated. The design basis check is limited to Gulf of Mexico platforms. The screening analysis criteria were eliminated from the criteria tables because quantitative criteria could not be set without making assumptions regarding screening analysis methods.

## Version 7

Undamaged platforms which have not seen an increase in vertical or lateral loading (generally due to additions or modifications to the facility) from the original design "pass" the assessment, providing the platform is in the "minimum consequence" category. This replaces acceptance by prior exposure in the flowchart (although acceptance via prior exposure remains an option in the final version). Dual criteria for minimum consequence platforms are provided: the platform may either meet the specified loading criteria, or may meet factored loading associated with the intact "as built" capacity. This replaces return period based criteria for minimum consequence platforms.

## Version 8

The dual criteria for minimum consequence platforms is eliminated from the criteria tables as being too complicated, although the acceptance of such platforms having negligible increase in load or decrease in strength from the original/"as built" condition is retained in the text. Criteria for minimum consequence platforms are specified as explicit wave heights for the Gulf of Mexico and as factors on 100 year loading for US areas. The flowchart provides a path around the design basis check for non-Gulf of Mexico platforms. Figures 3-a and b illustrate the latest flowchart.
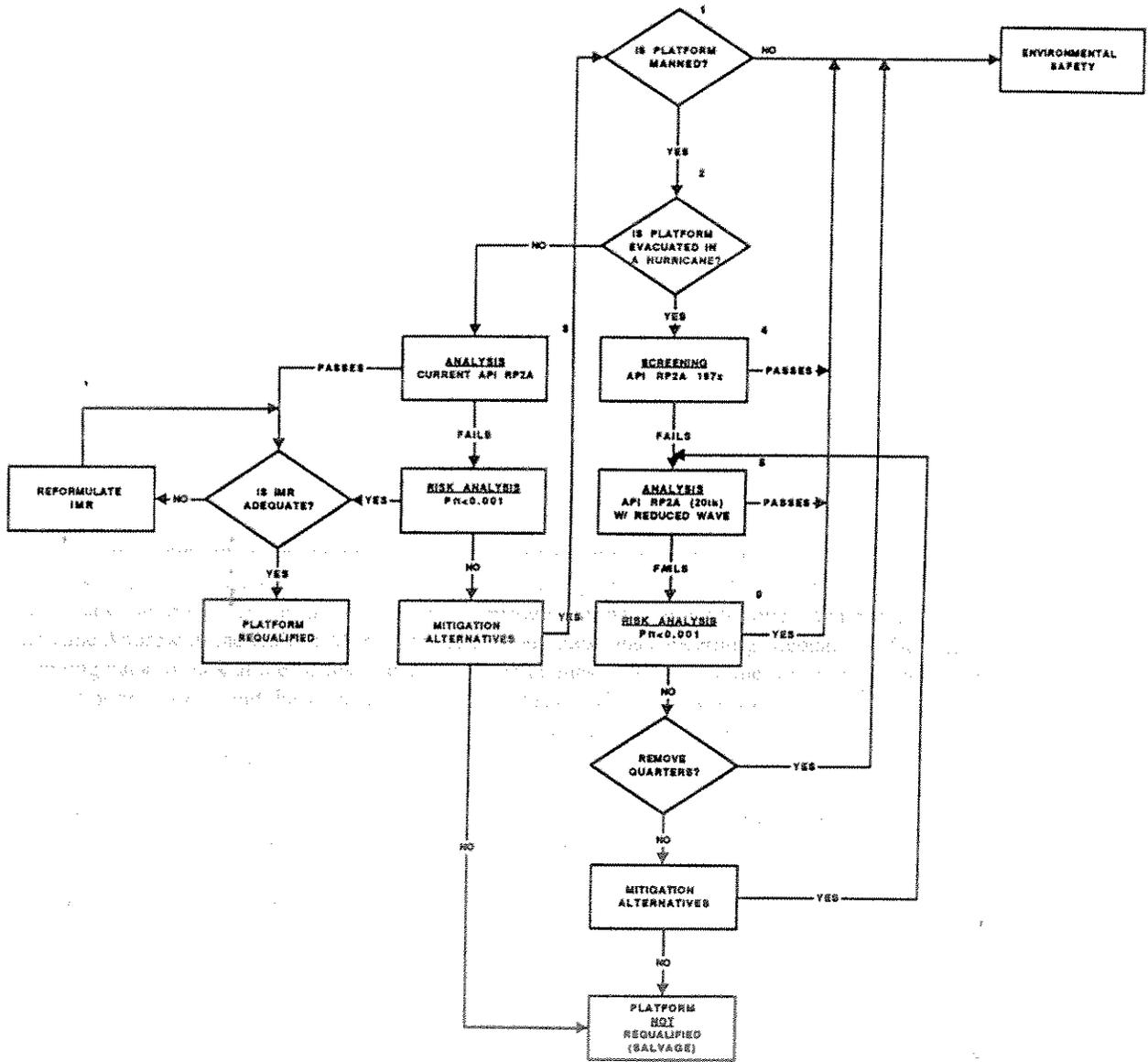
Figure 2-a  Life safety.
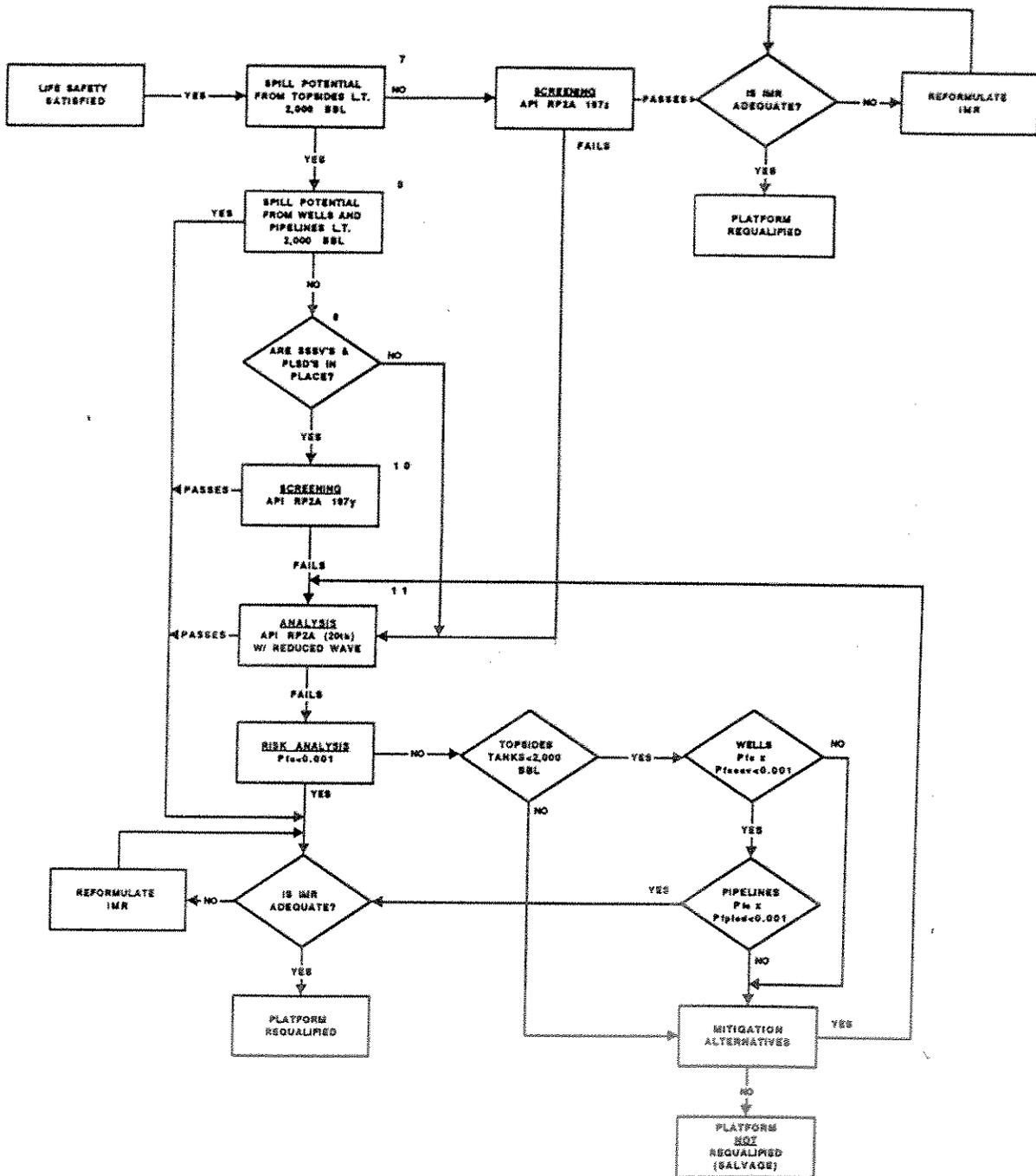
Figure 2-b  Environmental safety.
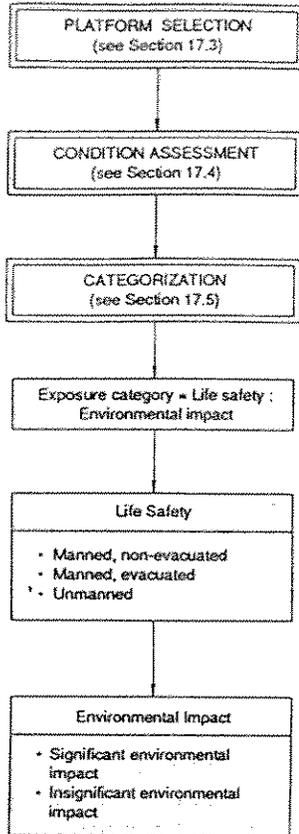
# ASSESSMENT CRITERIA - GULF OF MEXICO

| Exposure Category | | Design Level Analysis (see Notes 1 and 2) | Ultimate Strength Analysis |
|---|---|---|---|
| Sig. Env. Impact | Manned, evac. | Environmental safety design level analysis loading (see Figure 17.6.2-2) | Environmental safety ultimate strength analysis loading (see Figure 17.6.2-2) |
| | Unmanned | | |
| Insig. Env. Impact | Manned, evac. | Sudden hurricane design level analysis loading (see Figure 17.6.2-3) | Sudden hurricane ultimate strength analysis loading (see Figure 17.6.2-3) |
| | Unmanned | Mimimum consequence design level analysis loading (see Figure 17.6.2-5) | Minimum consequence ultimate strength analysis loading (see Figure 17.6.2-5) |

# ASSESSMENT CRITERIA - OTHER U. S. AREAS

| Exposure Category | | Design Level Analysis (see Notes 1 and 2) | Ultimate Strength Analysis |
|---|---|---|---|
| Sig. Env. Impact | Manned, non-evac. | 85% of lateral loading caused by 100-year environmental conditions (see Section 17.6.2b) | Reserve strength ratio (RSR) $\geq 1.6$ (see Section 17.6.2b) |
| | Unmanned | | |
| Insig. Env. Impact | Manned, non-evac. | | |
| | Unmanned | 30% of lateral loading caused by 100-year environmental conditions (see Section 17.6.2b) | RSR $\geq 0.5$ (see Section 17.6.2b) |

Notes: (1) Design level check not applicable for platforms with inadequate deck height.

(2) One-third increase in allowable stress is permitted for design level analysis (all categories).

PLATFORM SELECTION (see Section 17.3)

CONDITION ASSESSMENT (see Section 17.4)

CATEGORIZATION (see Section 17.5)

Exposure category = Life safety : Environmental impact

Life Safety
- Manned, non-evacuated
- Manned, evacuated
- Unmanned

Environmental Impact
- Significant environmental impact
- Insignificant environmental impact

Is platform damaged, deck height inadequate, or has loading increased? (see Sections 17.6, 17.7) — No → Is platform unmanned: insig. env. impact? — No → DESIGN BASIS CHECK

Is platform location GOM? — No

Yes ↓

Platform designed to 9th ed. or later with reference level environmental loading? (see Section 17.6)

Yes → Platform passes assessment

No

Yes (from platform damaged) ↓

A

**Figure 3-a  Platform assessment process - metocean loading.**
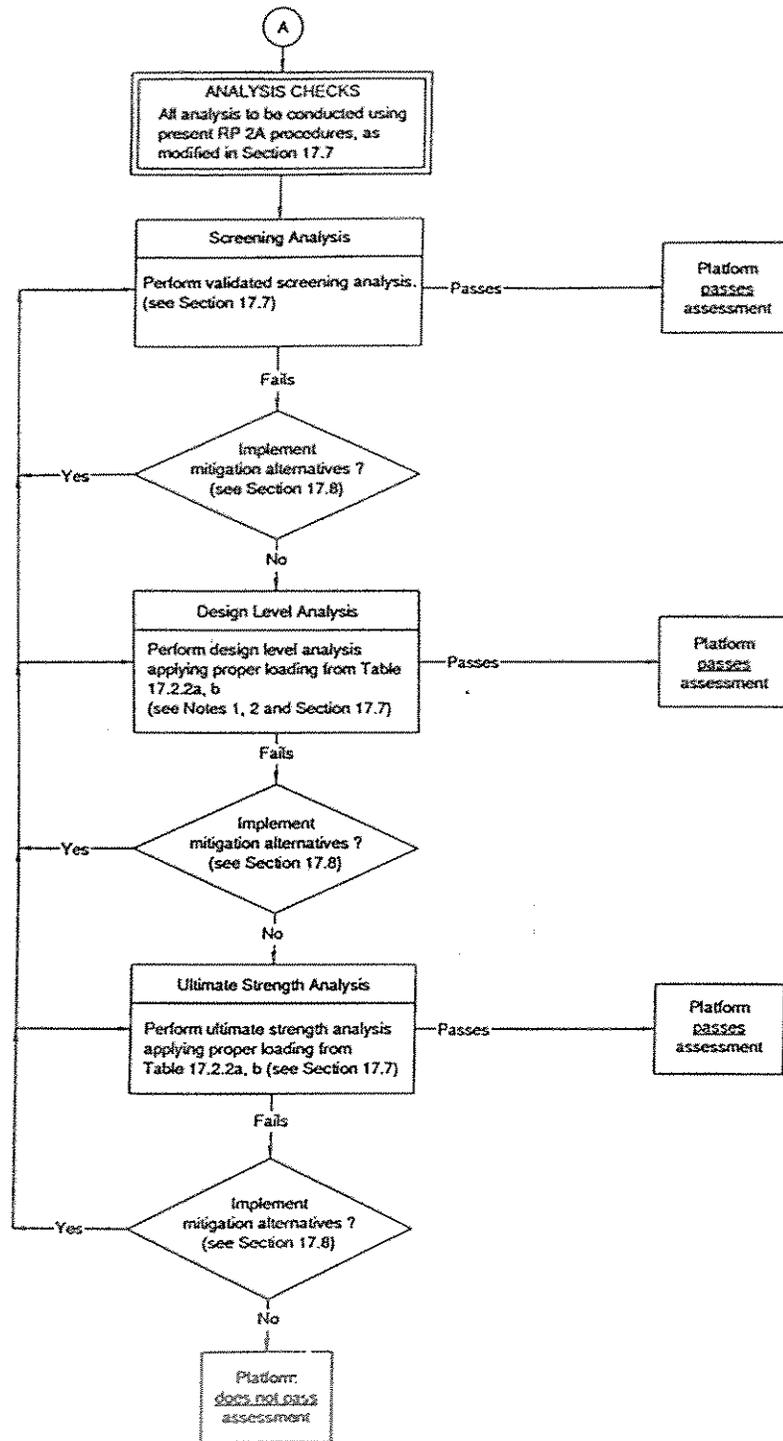
**Figure 3-b  Platform assessment process - metoocean loading (continued).**

## Third Party Review

One aspect of the process outlined in the THIC report not expressly addressed in the API guidelines is the topic of third party review. While there has been an element of third party review for many of the new Gulf of Mexico and offshore California facilities since 1981 through the MMS CVA program, API has not explicitly addressed this issue as it falls outside of the scope of API standards. From a practical view, a modified CVA type program could be implemented by regulatory agencies in conjunction with the RP 2A guidelines and provide the third party independent view.

## Hurricane Andrew

Hurricane Andrew has had a significant effect on both the philosophy and the technical information available for review. Not often in the development of standards or criteria does a full scale event present itself and provide benchmark activity. The API effort has drawn tremendously on the impacts and after-effects of Andrew. In addition to providing raw statistics, it provided an opportunity to review the adequacy of various criteria adopted in RP 2A as it evolved concurrently with the state of knowledge. A wealth of information was developed from a joint industry project that cataloged the damage as well as performed a number of ultimate strength analyses in efforts to quantify any bias in the design/analysis process. Though much of the information is proprietary, the project was structured to release summary information to the API project for use in developing the section on existing structures.

One of the key observations, once again reinforced by the aftermath of Hurricane Andrew, was that there is always an element of risk when dealing with the forces of nature. The quantification of risk levels and explicit choice of risks to be taken should be a fundamental choice of the owner. Historically, an owner could meet the design requirements of RP 2A with the perception of b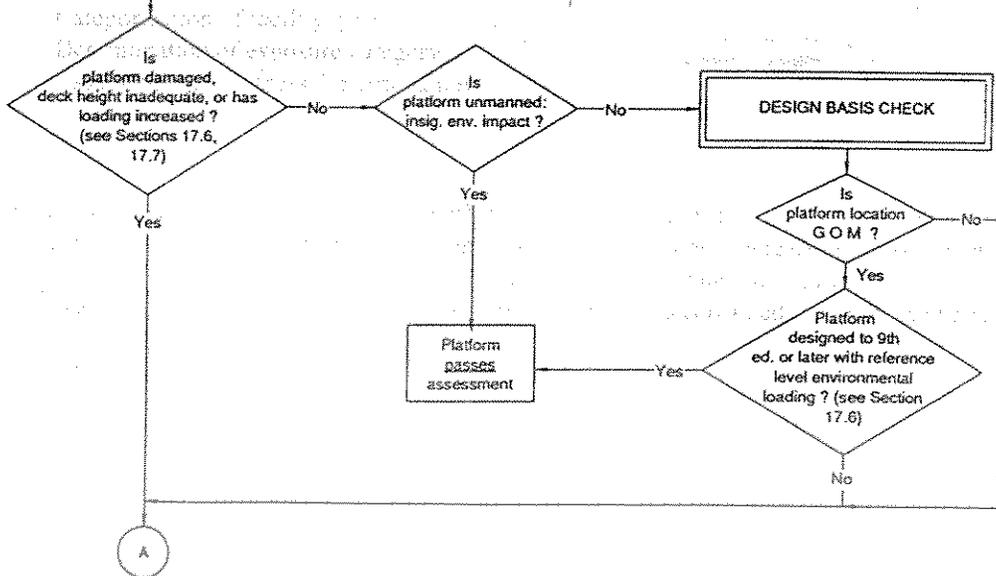eing safe and never quantifying the level of risk. With the introduction of multilevel criteria, a more explicit determination will need to be made by an owner. In the past, an unmanned, single well caisson was designed to the same standards and criteria as a deep-water, multi-well, manned platform. Even though they met the same criteria, and unknown to many within the industry, there were significant differences in the margins of safety against structural failure and even broader differences in consequences. Use of a single criteria was not providing uniform safety or economic risks.

## Summary

The overall assessment and acceptance process can be summarized as one based on logic meeting the needs of a regional industry. While regional in primary focus, the concept has been developed around a logical framework that can be extended through the use of Product Service Levels to account for the consequences of the facilities and the societal values in other areas where offshore structures are utilized.

Identification of multilevel acceptance criteria based on consequence is somewhat a departure from traditional practice is some areas of the world.

As the industry matures and the global economics become more pronounced in the oil industry, rational criteria and approaches must be taken in the offshore arena. API and ISO are moving towards harmonized standards that address issues on a consequence basis taking into account the life and environmental safety standards of the regions where the structures are operating.

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# KEY QUESTIONS IN THE REASSESSMENT AND REQUALIFICATION OF PERMANENT OFFSHORE DRILLING AND PRODUCTION PLATFORMS

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***Charge to the Working Groups***

## Abstract

Recent experiences in the reassessment and requalification of permanent offshore drilling and production structures suggests that there are three key questions that should be addressed in the following order.
- What should be reassessed and requalified?
- What should be the requalification standards?
- How should the reassessments be performed?

These questions are discussed in this paper in the context of present efforts in the United States and North Sea to develop and implement practical methods to reassess and requalify platforms.

## Introduction

The problems associated with aged offshore production platforms are particularly difficult in comparison with those associated with the initial design, construction, and operation of these structures. Aged platforms generally not only reflect the inevitable effects of corrosion, fatigue, and operating wear and tear, but technical obsolescence as well. Rehabilitation and life-extension can be expensive propositions that can typically occur when they are least affordable.

The process of defining the basic characteristics of a particular platform and the condition of its elements is extremely challenging. Generally, there are significant uncertainties concerning the in-place characteristics of the platform.

The objective of this paper is to discuss three key questions that overshadow efforts to develop definitive guidelines and procedures for the reassessment and requalification of offshore platforms.

### What Should Be Reassessed and Requalified?

The fundamental objective of reassessment (analysis of existing platforms) and requalification (evaluation of suitability for service) efforts is to assure the platform owner and regulator that the platform is fit for its intended purposes.

The platform owner represents industrial interests. The platform regulator represents the general public interests. Both of these interests want to determine that the platform and its associated operations do not pose undue risks to property, productivity, human life, or the environment.

It has been suggested that considerations of property and productivity risks are not the purview of government, and that they are solely the purview of industry. Others suggest that the general public has a stake in the industrial activity because ultimately the public must pay the costs for the resources that are developed.

In 1993, Hurricane Andrew resulted in approximately 164 structures being toppled, leaning, or condemned (Smith 1993). This tally included some 22 major platforms. Many more platforms suffered some form of significant damage. Approximately four hundred segments of pipelines were damaged or ruptured. Seven pollution events and two fires occurred. Five mobile drilling rigs were set adrift causing damage to structures and pipelines. One mobile drilling rig narrowly missed entangling its dragging anchors in the LOOP (Louisiana Offshore Oil Port) 36-inch diameter pipeline.

Pollution was not substantial. The maximum pollution event was reported to have resulted in the loss of 2,000 barrels of oil. Unlike the experiences with this storm as it swept across Florida, there were no lives lost offshore as a result of this very severe storm.

However, there was a major loss in property and productivity. One-third of the oil and one-fourth of the gas

production of the Gulf of Mexico were shut-in in the aftermath of Andrew. The total damage and productivity losses in the Gulf of Mexico have been estimated to be in excess of $2 billion. The onshore losses were estimated to be in excess of $4 billion, making this storm the most expensive environmental catastrophe in the U.S. history.

Much of the offshore loss will be born by insurance and will be reflected in increased premiums. Other parts of the loss will be reflected in increased operating costs. Given the short- and long-term industrial requirements for profitability, it is the general consuming public that ultimately must pay these costs.

This experience would suggest that there is a common set of concerns among industrial and governmental interests. They both should be interested in the same things: the sensible preservation and utilization of property and productivity, life, and the environment. These interests represent different constituents and thus emphasize different considerations.

The experience in Hurricane Andrew served to point out another consideration: the performance of a fleet or group of platforms as contrasted with the performance of individual structures. Temporary loss of one-third of the production capacity of the Gulf of Mexico had significant implications to some. This was an expression of an aversion toward large losses from a group of platforms. This aversion potentially has important implications regarding the requalification of a group of platforms as opposed to the requalification of individual structures.

The experience in Hurricane Andrew also served to again point out the importance of deployment and anchoring of mobile offshore drilling units (MODUs). While the losses associated with damage to or loss of MODUs can be relatively small, the damage that they can cause in congested areas can be substantial. It may be time to reexamine anchoring and deployment standards for MODUs located in highly developed areas.

It is proposed that a primary objective of the reassessment and requalification process is intelligent risk management for both industrial and governmental interests. That is, collectively these interests should want to minimize the likelihoods of undesirable events and consequences that are associated with operations of an aging energy infrastructure. There are both costs and benefits associated with this effort and both the industry and public should want the benefits to exceed the costs.

Reassessment and requalification efforts should be focused on those operations that represent high likelihoods of not having sufficient performance capabilities and that also have potentially high consequences associated with less than desirable performance. Limited resources should be devoted to those structures that have high likelihoods of poor performance and high consequences, and as resources can be made available, the reassessment and requalification efforts should be directed to those structures that represent lesser likelihoods and consequences.

The next question is: given the aging offshore infrastructure, what should be requalified? Should the requalification be focused on the platform itself (i.e., the structural components of the decks, jacket, and piles that comprise the platform structurally) or on the total system? There is sufficient evidence, both historical and analytical, that suggests that if the requalification effort is to be successful, the entire system must be reassessed; not only the platform itself, but also its topsides, and its associated operations (personnel operating procedures, process safety, drilling safety, pipelines, and roles in the production infrastructure).

Based on data from the World Offshore Accident Database (WOAD), Figure 1 summarizes the statistics on major damage (in excess of $2 million) to permanent drilling and production platforms for the period 1980 to 1990. The statistics apply to worldwide operations. The accident rates are the average for this period. The statistics indicate that the primary causes of major damage to fixed platforms are related to operations: fires and explosions, blowouts, and collisions. At the present time, accidents due to these operations hazards have comparable rates of occurrence. During this period, the annual rate of accidents that could be attributed to structure causes was less than 1/10,000. The total annual accident rate was less than 1/700.

Figure 2 shows the analytical results from probability based requalification studies of three platforms located off the West Coast of the U.S. (Bea and Craig 1993). These three almost identical platforms were installed during the period 1966-1977. Even though these platforms are located in a very active seismic region, a primary hazard is the force from storms. Earthquakes are indicated to result in a comparable probability of failure. Due to the lack of high hazards associated with the operations on these platforms, the operations related failure rates are relatively insignificant. They are a factor of about ten less than the environmental loading related rates. In this case, the priority for resources should be given to the platform structure.

Figure 3 summarizes similar results for a platform located in Cook Inlet, Alaska (Bea et al. 1992). The results indicate that the topsides related operating likelihoods of failure are the primary contributor to the total risk. They are about a factor of five greater than the total of the environmental loading related risks of failure. This change in relative importance is due solely to the nature of the operations (high pressure gas, enclosed modules) and the present condition of the piping and other critical vessels. In this case, the first priority for allocation of resources should be to the opera-
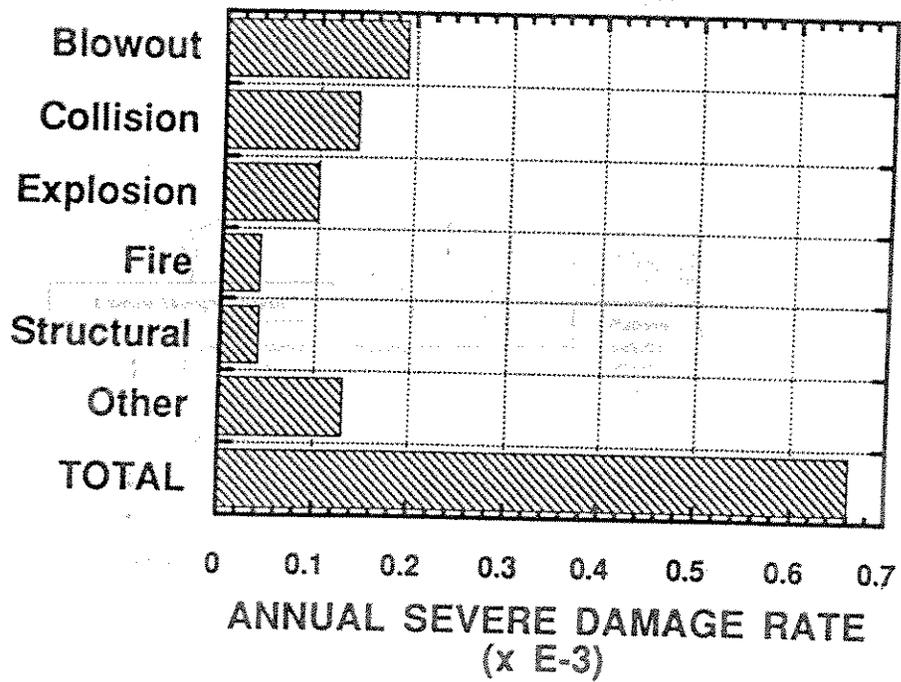
Figure 1  Initiating events leading to severe damage to fixed offshore platforms  (1980-1990).

Operating Pf = 0.1 E-3 pa

Quakes Pf = 1.4 E-3 pa
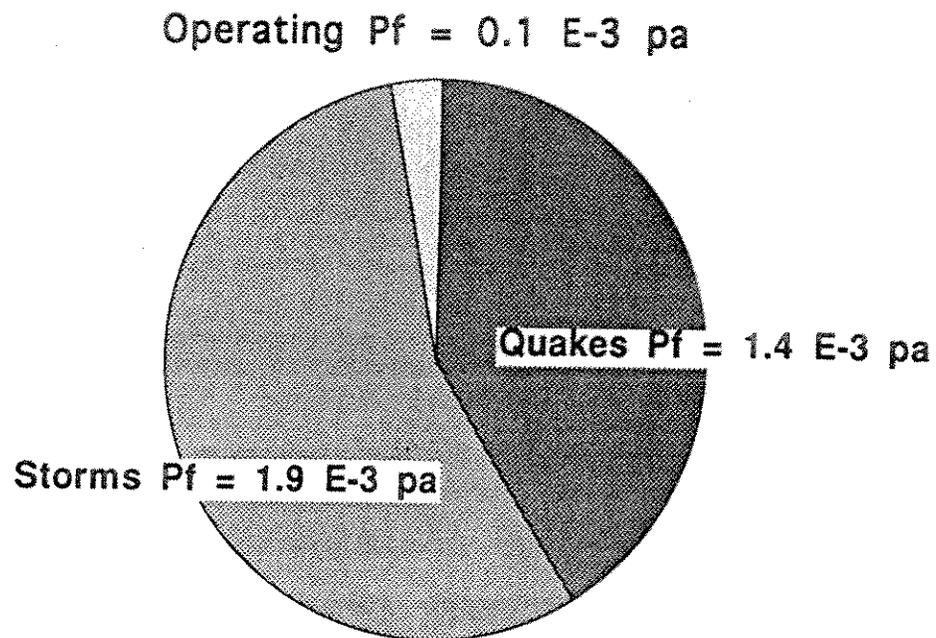
Storms Pf = 1.9 E-3 pa

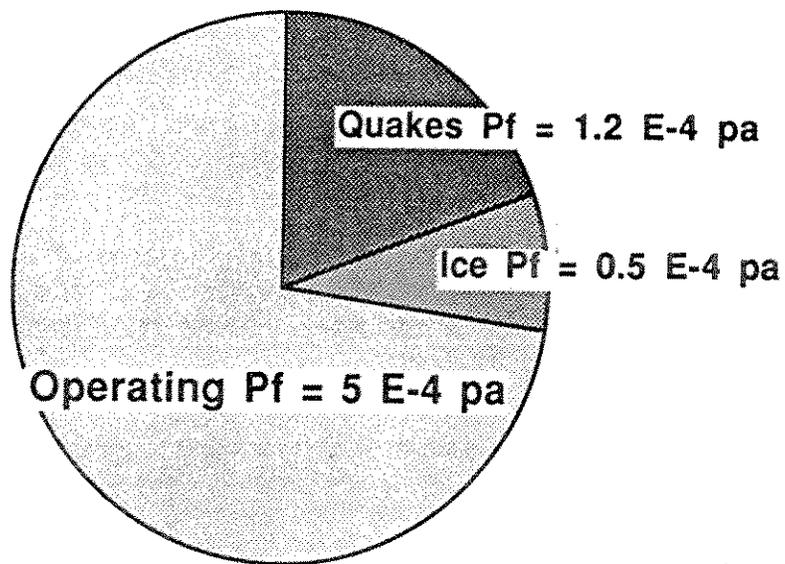Figure 2  Notional annual probabilities of failure for three West Coast platforms.

Figure 3  Notional annual probabilities of failure for Cook Inlet platform.

tions and related facilities.

While there may be different groups of engineers and regulators working on the operations and structure related parts of the requalification efforts, experience indicates that it is important to keep balance and coordination among the different elements of the efforts in order for a comprehensive evaluation of the overall risk of a platform and its operations to be realized. This is particularly true when it is recognized that generally there are limited resources available for reassessment and requalification efforts. One would like to be able to devote these limited resources to where they could do the most good.

There is another important reason for taking a "full-scope" approach to the requalification of platforms. As platforms age, due to corrosion, fatigue, general wear and tear, their strength can be expected to decrease. Thus, their likelihoods of failure in extreme environmental events can be expected to increase. These increases can be mitigated by modifications in operations related risks. The risks can be reduced by reducing the likelihoods of accidents and the potentials for adverse consequences. Examples are de-manning, weight reductions, raising or deleting lower working decks, employing additional well and riser controls, and removing unnecessary storage and appurtenances.

It is important that the question regarding what should be reassessed and requalified be answered first, because the answer to this question has a direct link to the second key question and its answers. It is hoped that the working groups at this workshop will thoroughly address this first question in the contexts of their particular focus topics.

### What Should Be the Requalification Standards?

Requalification standards (goals) are needed so that both the regulatory and the industrial interests can determine if a platform is fit for purpose. From both engineering and regulatory standpoints, it is desirable that these standards be as unambiguous as possible.

Requalification standards can be developed from different points of view. One is from the point of view of history: what we have accepted in the recent past. This becomes a guide to what we may accept in the future proposed life of a given facility.

There are difficulties with this line of development. The first is associated with the term *we*. What industry has accepted may not be acceptable to the public it serves. What has been accepted in the past by industry, government, or the public may not be acceptable in the future. What one segment of the population or industry has deemed acceptable may not be acceptable to other segments.

A second difficulty regards defining what has been accepted. The historical average probability of failure of Gulf of Mexico platforms associated with overloading in extreme hurricanes has been less than 1/2000 per year for the past ten years (Visser 1993). In this context, failure is defined as damage extension enough so that the platform was salvaged. With the experience of Hurricane Andrew, this rate increased to about 1/1000 per year. There has been no hue and cry from industry, government, or the public. Is this new rate of failure acceptable to industry and regulatory interests for Gulf of Mexico operations?

The average total probability of failure of Gulf of Mexico platforms has been in excess of 1/1000 per year during the past decade (Visser 1993). This total rate of failure apparently has been acceptable. The consequences apparently have also been acceptable. Thus, the total risk has been accepted implicitly.

In the North Sea, there is a different history. The historical rate of failure of major permanent platforms due to environmental causes is zero. There has never been a fixed drilling and production platform lost in a North Sea storm.

The historical total rate of failure of North Sea platforms is less than 1/1000 per year (Figure 1). This rate is due solely to blowouts, fires, explosions, collisions, and construction accidents. The Piper Alpha catastrophe lead to substantial revisions in both the industrial and regulatory approaches to the risk management of offshore platforms. The *ALARP* (As Low As Reasonably Practical) fitness for purpose goal developed from this experience (Barrell 1993; Kam et al. 1993). The reliability goals are not definitive; judgments are developed on a case-by-case basis. This places very high demands on the regulatory processes and personnel.

In the North Sea, platform system (structure, topsides, operations) safety case studies have become the mechanism for examining the fitness for purpose of both existing and new platform systems. These safety case studies focus on the structure, equipment, and operations aspects of the structure. All of the platforms in the United Kingdom sector should have submitted safety case studies by the end of 1995 or face shut-in of their operations. For a major drilling and production platform, an average safety case study has been estimated to cost between $1.5 and $2.0 million (U.S.) and to take approximately one year to perform.

A panel on Seismic Safety Requalification of Offshore Platforms (American Petroleum Institute 1992) has addressed the issue of requalification standards for offshore platforms sited in earthquake areas from the standpoint of the accepted standards for onshore occupied buildings. This panel came to the conclusion that an annual probability of

failure of major platforms of 1/1000 per year could be an acceptable target. The guidelines developed by the panel placed limits on environmental and life safety risks. This figure was not based on historical figures associated with the performance of existing occupied buildings. Rather, it was based on analyses of existing buildings that seem to represent acceptable structures to the general public and to their regulatory representatives.

It is interesting to note that analyses of recently designed and well maintained Gulf of Mexico platforms indicate a comparable figure for the probability of failure due to hurricanes (1/1000 per year). In most cases, these structures are shut-in and evacuated in advance of hurricanes. In the case of rapidly developing hurricanes such as Hurricane Juan in 1985 (Dyhrkopp 1987), this goal may not be reached and operating personnel must ride out the storm on board the platform. As will be discussed in the next part of this paper, this experience has lead the API to define requalification guidelines for the conditions in which personnel might not be able to be evacuated.

Initially, guidelines for design of platforms in the Norwegian sector of the North Sea required design for accidental events that had return periods of 10,000 years (Vinnem 1993). Experience indicated that in many cases the engineering analyses were intended to demonstrate compliance with the 10,000 year return period criteria rather than being directed toward comprehensive safety management objectives. As a result, the specific reliability targets have been removed. The experience in the Norwegian sector has emphasized that how one demonstrates compliance is more important than what one calculates. It is the continuing process of attempting to achieve adequate reliability that is the primary focus. Acceptable reliability is the basic product of the effort (Vinnem 1993).

The recently drafted API guidelines for the requalification of platforms in the Gulf of Mexico (American Petroleum Institute 1993) have referenced acceptance standards to criteria used to design platforms that have generally proven to be acceptable to industry. For example, platforms designed according to the 9th edition of API RP 2A (1977) or later have proven to perform acceptably in intense hurricanes. This design basis has been chosen one of several screening bases for requalification. It is stipulated that the platform must have no significant damage, have an adequate deck height, and there must have been no changes from the design premises which would significantly increase either vertical or lateral loadings.

The API platform requalification guidelines are focused on the structure. Other API guidelines are referenced to enable demonstration that the platform equipment and operations are fit for purpose (API 1993).

Different structure requalification standards are defined for different categories of platforms. The different categories of platforms are based on their potential consequences associated with failure. The evaluations of consequences include the potential for significant or insignificant environmental impacts and manning conditions in extreme environmental events.

A second general approach to development of requalification standards is founded in economics: cost-benefit analyses (Bea 1991). The oil and gas industry is very skilled in the performance of such analyses. The industry is also very used to dealing with the uncertainties that pervade the input and results from such analyses.

Some interesting insights regarding requalification standards can be developed from a simplified cost - benefit analysis. It can be shown that the annual platform probability of failure (Pfo) that produces the minimum total cost (sum of initial and future costs) can be approximated as:

$$Pfo = \frac{0.4348}{CR\ (PVF)}$$

where CR is a cost ratio and PVF is a present value discount function. Cost ratio is the ratio of the total costs associated with failure of the platform (CF) to the costs required to reduce its Pf by a factor of 10 ($\Delta$Ci). Note that CR is non-dimensional. For short life operations, PVF is equal to the life in years of the proposed operations. Note that the dimensions of the denominator is years and that the failure rate, Pfo, is the probability of failure per year.

This is a very sensible expression. As CF increases, Pfo decreases (greater exposure leads to a requirement for greater reliability). As $\Delta$Ci increases, Pfo increases (greater costs of achieving reliability leads to a requirement for lesser reliability). As the life of the operations increases, there is a requirement for greater reliability.

Instead of the minimum total cost, if one were to assume that an acceptable Pf could be defined when the increment of invested cost equaled the future cost increment saved, then:

$$Pfm = 2\ Pfo$$

This indicates that the "marginal" Pfm is twice the "optimum" Pfo. Comparisons of the historical probabilities of failure associated with new and existing engineered systems has indicated approximately the same relationship: for older engineered systems we accept about a doubling of the likelihood of failure (Bea 1991).

The author's experience with the historic and economic based approaches indicates that they can produce very comparable results. When used as complimentary approaches, they can provide important insights into the definition of goals and standards for the requalification of platforms. Figure 4 shows one result from these two approaches. Results from recent decisions associated with design of new platforms and requalification of existing platforms are shown. The acceptable and marginal lines are based on the economics considerations that have been outlined here. As contended, the results are very similar.

At the present time, two general types of requalification standards have developed. In the North Sea, the standards are defined primarily as performance goals. The standards are matched to the particular platform and its operations. Demonstration of compliance with these standards are the responsibility of the platform owner and are usually based on state-of-the-art approaches (Barrell 1993).

In the U. S., the industry standards are fundamentally prescriptive (API 1993). The primary approach is founded on traditional engineering methods that have been justified with historic performance of platforms in the Gulf of Mexico. The standards are focused on the performance of the platform itself. Demonstration of compliance with these standards are fundamentally based on state-of-the-practice approaches.

Perhaps these two different requalification standards are appropriate. The platforms and operations in the two different areas generally are different. The industry, regulatory, and engineering cultures and capabilities in the two regions are different. It is hoped that the working groups at this workshop will address this key question in the contexts of their focus topics. Its answer will lead directly to the fourth question posed in this paper.

### How Should the Reassessments be Performed?

The procedures that are used to requalify platforms should be consistent with the procedures and background that are used to define the requalification standards. The specification of reassessment goals and standards should be accompanied by a coherent and detailed specification of how compliance with the goals and standards should be demonstrated.

It is similarly important to define the latitude or tolerances for acceptance. While it is very desirable to have unambiguous goals and standards, experience in the requalification of platforms indicates that it is difficult to realize such results. Generally there are few black and white answers. This places a heavy burden of qualifications, judgment, experience, credibility, and integrity upon the industrial and regulatory personnel that are involved in the processes.

The procedures used to requalify platforms need to be practical in the context of industry capabilities to perform the analyses. The results must be practical in the context of regulatory capabilities to evaluate and approve the results from the analyses. The attributes of practicality include: ease of use, versatility, compatibility with accepted procedures, workability, and consistency (yielding similar results for similar problems). Requalification requirements must be consistent with the resources of both the platform owner and the regulator. Requirements that transcend the capabilities of either group will fail to accomplish the desired objectives.

The API platform requalification guidelines have been based primarily on historic experience with design and performance of Gulf of Mexico platforms in hurricanes. Topsides safety considerations have been left to other API guidelines. For these structures the reassessment process has been founded primarily on traditional elastic analysis-based analysis methods. Working stress-based allowable stress interaction ratios for the platform elements (legs, joints, braces, piles) and factors of safety are used to judge suitability for service. To accommodate the difference between the design criteria for new platforms and that for requalification, environmental loading reduction factors have been developed to reduce the environmental loadings. Specialized storm criteria have been developed for low consequence structures and for rapidly developing hurricanes for which the platforms can not be evacuated. In the initial screening phase, the guidelines require that significant damage be repaired. In the design level analysis there is an implicit requirement for such repairs.

Only when it is not possible or practical to requalify the platform with the design level analysis, then nonlinear ultimate limit state analyses of platform performance can be undertaken. Return period based guidelines for the environmental conditions are specified. Probability based methods are cited but not detailed.

At this time, the U. S. regulatory guidelines for requalification have not been defined. It is expected that they will be based primarily on the API guidelines. From the regulatory viewpoints, what must be requalified and when remains to be defined. An important unresolved aspect regards third-party verification of the requalification analyses and results. Similarly, the review and approval processes and procedures remain to be defined. Given the relatively meager U. S. regulatory resources and the very large number of platforms to be requalified, it seems fair to say that the review and approval processes must be very efficient and generally produce unambiguous results.

The API guidelines are very practical in the context of conventional engineering practices. The basic analytical
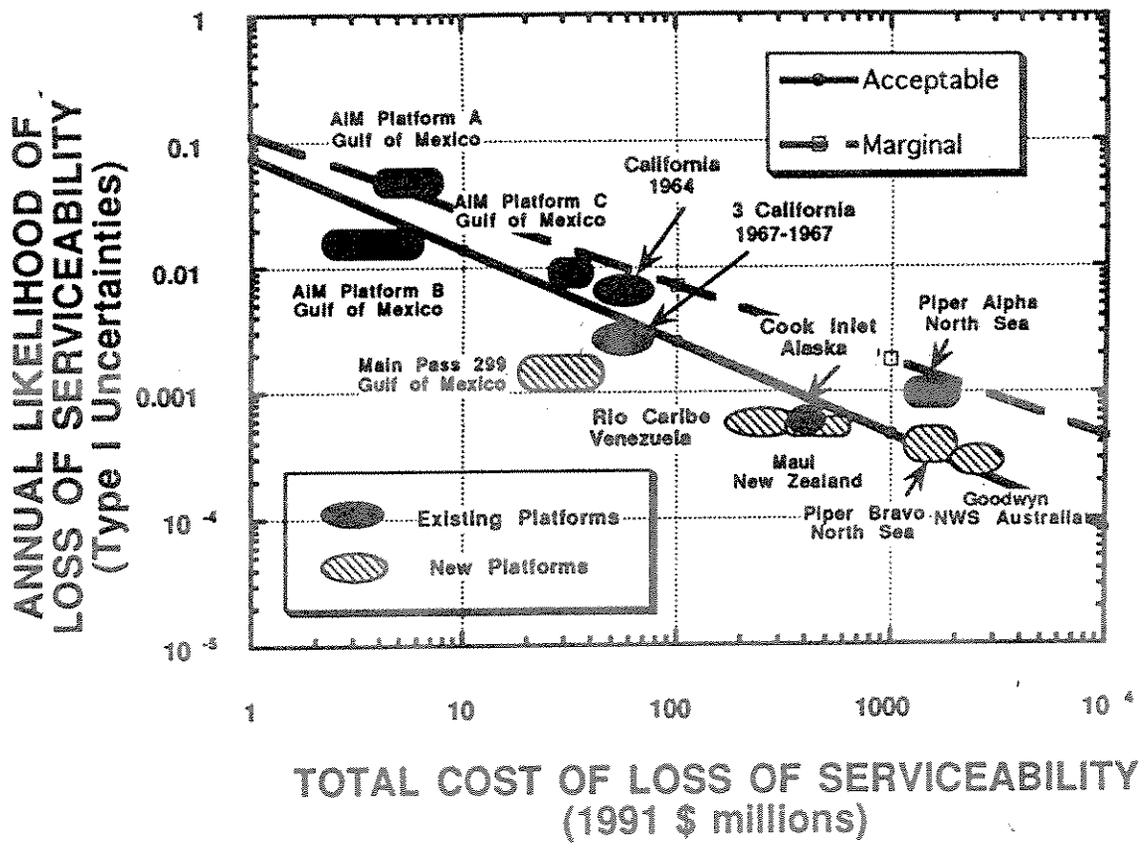
Figure 4  Fitness for purpose standards based on notional probabilities of failure and total estimated costs associated with failure.

procedures are well understood by most platform designers. The guidelines also are versatile in that a variety of approaches and procedures can be used to demonstrate fitness for purpose (screening analysis, design level analysis, ultimate strength analysis). It is only with experience that the industry's capabilities to implement these guidelines and the regulatory agencies' capabilities to verify and approve the results will indicate how successful this approach will be.

The U.K. Safety Case based approach and the approach developed in the Norwegian Sector are very different. The platforms in the North Sea generally are very different from their counterparts in the Gulf of Mexico (they have many similarities to those located in Cook Inlet, Alaska). The North Sea structures represent very high exposure category platforms. They handle large volumes of hydrocarbons in comparison with their Gulf of Mexico counterparts. They represent very significant investments. Because of the long periods of severe inclement weather, these platforms are generally enclosed, making fires and explosions a greater hazard. It is not practical to evacuate North Sea platforms in advance of severe storms, thus, personnel must ride out these events on board the platforms.

Very high degrees of industry and regulatory capabilities have been developed in response to the approach that has developed in the North Sea. Verification, evaluation, and approval processes are focused in the regulatory agencies, thereby avoiding the need for third-party verifications. Given the goal-oriented standards that have been prescribed, review and evaluation of the process of demonstrating fitness for purpose is very important. Highly developed regulatory capabilities and credibility are required.

Again, it is hoped that each of the Working Groups will address the question of reassessment and requalification procedures in the contexts of their focus topics. The coherency of standards and goals and the procedures and methods used to demonstrate fitness for purpose is a high priority concern in this Workshop.

## Conclusions

Two different approaches are evolving in the requalification of platforms. The first being developed in the North Sea is based fundamentally on performance standards, and state-of-the-art procedures. This approach requires highly developed regulatory and industry capabilities.

The second approach being developed in the U. S. is based on historic experience, prescriptive standards, state-of-the-practice procedures. This approach requires much more modest regulatory and industry capabilities.

Are these two approaches complimentary? Are these two approaches fundamentally a product of the two different problems they are addressing? Are these two approaches fundamentally a product of the two different sets of industrial, regulatory, and engineering cultures that they represent?

It is anticipated that the deliberations of this Workshop will shed some light on these important questions.

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# REQUALIFICATION OF OFFSHORE INSTALLATIONS
# A NORTH SEA PERSPECTIVE

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**Charge to the Working Groups**

## Abstract

The reasons for requalification and the procedure used, in the North Sea, are described. The use of system reliability as opposed to design code checks is considered and inspection requirements are discussed. Areas of uncertainty are highlighted.

## Introduction

Lloyd's Register has been involved in the analysis and approval of over 500 fixed offshore platforms throughout the world. Many of these platforms already existed before the requirement for certification came about. The earliest of these were installed in the latter half of the 1960s in the southern North Sea and also in Indonesian waters.

The assessment and reanalysis of existing platforms subject to damage and deterioration has always been an ongoing part of Lloyd's Register's work. In the North Sea this work has generally been called reassessment rather than requalification.

I understand the term requalification to describe the process of inspection, analytical analysis and assessment that may be applied to existing installations in order to assure the operator, regulatory bodies and third parties that the installation is still fit for purpose.

Requalification may be applied in the following circumstances:

- When the installation will be subject to increased loading due to modifications. This has been a very common reason for reassessment, as existing platforms are frequently required to accept new equipment such as compressors or additional accommodation. New requirements from the HSE (United Kingdom Health and Safety Executive) for increased topside safety will also lead to increased deck loading. The structural implications of any significant change in topside loading should always be considered since even a weight decrease can reduce the safety factors of piles in tension. Additional conductors, caissons etc. also need to be considered since these increase wave loading and even where the overall effect is small they may increase the loading on an individual member significantly.
- When the installation is damaged due to severe storms or an accident or suffers deterioration due to fatigue or corrosion. When serious damage occurs it is generally advisable to reanalyze the platform. This serves two purposes. Firstly, it determines the safety for ongoing certification and the requirements for repair. Secondly, in the case of fatigue failure, a "state of the art" fatigue reanalysis may indicate other areas of likely damage and set priorities for further inspection.
- When guidance on inspection and maintenance is required. Since underwater inspection is a time-consuming and costly exercise it is normally possible to inspect only a fraction of the total number of welds on a large offshore platform. It is vital therefore to concentrate the detailed inspection on any welds that are considered to be critical. Unfortunately, the original design analysis may be unreliable from this viewpoint. In these circumstances, it makes good sense both from a safety and an economic point of view to reanalyze the platform using the latest techniques.

It is not normal practice to undertake repairs simply on the basis of a low calculated fatigue life. Where low fatigue lives are predicted by the reanalysis then inspection would normally be undertaken. Only where the fatigue calculations are substantiated by inspection (i.e., cracks or failures are found) will repairs be consid-

ered. Once inspection has confirmed that the results of the fatigue analysis are correct then it may be considered prudent to strengthen uncracked joints if the calculations indicate these are also at risk. This type of preventative repair must be carefully considered since, for some types of repair, further inspection may not be possible after the repair has been fitted. Reanalyses have also been undertaken in order to refine and reduce the requirements for the removal of marine growth.

- When the installation reaches an age where requalification is deemed necessary. There are no industry or government guidelines on when a platform may require requalification based on age alone. In the UK it would be unusual that some form of assessment had not already been carried out for some other reason.
- When the installation will experience a change of use. This would normally apply to mobile drilling units or ship-type units that are converted into permanently moored floating production systems. This is a frequent cause of reassessment and reanalysis in the UK but is not dealt with here as this paper concentrates on fixed platforms.
- To remove de-manning limitations. When the certification scheme was first introduced, in the early seventies, some of the existing platforms were found not to comply with the normal code requirements. The reason for this was that the environmental criteria and the design code requirements had generally increased since the platforms were originally designed. The solution adopted in this and other similar cases was to impose a de-manning restriction on these platforms. That is, all personnel must be taken off the platform if wave heights above a certain level are forecast. In recent years more sophisticated reanalyses have been undertaken in order to remove the de-manning restriction by establishing that the structure is in fact safe and fit for purpose.

## Establishing the Present Platform Condition

The first step in requalification of an existing platform is to establish its present condition. In some situations, such as a reanalysis in the case of damage, there may be very little time for a detailed review of the platform condition. Ideally however, the platform condition should established with confidence before a reanalysis. This information may well be available from annual survey reports, but if not, some special survey may be required. Particular attention should be given to the following points.

### Confirmation of Topside Loading and Jacket Appurtenances

This does not necessarily require a detailed survey but may simply be a question of establishing the number of conductors and risers in place and whether the drilling derrick, for example, has been removed. In other cases, particularly for major installations, a much more detailed weight audit may be required in order to establish the maximum weight and center of gravity for the topside facilities.

### Corrosion

Most platforms have a "corrosion allowance" in the splash zone. If the survey shows no corrosion, or very little, then part of this corrosion allowance may be used in the strength calculations. This would of course be subject to continued monitoring of corrosion in these areas.

### Marine Growth

The thickness of marine growth is required in order to calculate the wave loading. When measuring marine growth it is preferable to base the measurements on the thickness of the compressed growth. This may be done by wrapping a broad measuring tape (say, three inches wide) around the member and pulling it tight. Special tapes calibrated to read off diameter directly may be used. It is also important to ensure that the diver appreciates that it is the average thickness of marine growth on each member, or at each platform level, that is generally required for wave load calculations. A formalized method of reporting marine growth will help to ensure the correct results are obtained.

### Water Level

On some older platforms the still water level with respect to the platform is not known with any certainty. On shallow water platforms particularly, a reduction in the water depth can have a major impact on wave loading and

fatigue at the first horizontal level below the waterline. Alternatively, an increase in the water depth will reduce the air gap.

### Damage

In the case of damage a detailed structural survey will be required to determine the extent of the damage and to aid repair.

### Scour

The depth of scour is important for the pile/soil interaction at the mudline and this should be established during the underwater survey. The survey should distinguish between local scour which mainly affects pile bending at the mudline and overall scour which also reduces pile capacity.

### Structure

It is vital that we have confidence in the integrity of the general platform structure. Inspection should be carried out on any overstressed welds or those with low fatigue lives. Detailed weld inspection should always include NDT since close visual inspection by itself is unlikely to find anything but a major through-thickness crack. However, it is not usually feasible, even on small platforms, to inspect every weld. Therefore the detailed inspection must be supplemented by some other method such as flooded member checks (FMD). Where there are no critical welds a flooded member survey may be sufficient. A thorough overall visual inspection using RCV's should also be performed. Experience indicates that where a totally unexpected damage has occurred this has often been first detected by overall visual inspection or FMD.

## Design Criteria

In addition to establishing the condition of the platform the design criteria should be reviewed with particular emphasis on the environmental criteria.

### Environmental Criteria

In many cases it will be appropriate to review the environmental criteria used in the initial design and the assumptions made in calculating the environmental loads. It is fundamental that the assessment must be honest and consider not only conservative assumptions but also review those assumptions which may have been optimistic.

There may have been considerable improvements in the basic measured data base since the original environmental report was commissioned. This data should be assessed by a competent oceanographer to determine whether there have been any changes in the predicted criteria. Bearing in mind the possibility of future reanalysis requirements, operators should consider undertaking ongoing monitoring of the environment at their platforms. It is essential that any data is recorded in such a way as to be accessible by computer in order for it to be used statistically.

### Directional Data

The original platform may well have been designed on the basis of an omnidirectional maximum wave and current. It would be normal practice in a reassessment analysis to use directional data for wind, wave and current as this could give considerable reduction in load in some directions.

Wave rosettes do not always have a major impact on the design of new platforms because these are often symmetrical in design. However, it is very important for the analysis of existing platforms where the problem of platform overstress, or perhaps overloading of piles, may only occur in the case of one or two directions of wave loading.

### Assessment Criteria

In some cases "assessment criteria" have been agreed that are lower or less conservative than the criteria used for new installations at the same location.

### Combined Probability Of Wave And Current

Under previous editions of the API RP2A (API) the probability of combined occurrence of wave and current, and the possibility of the level of current combined with the extreme wave, was not considered. This was due to the unrealistically low Cd values used and the fact that the combined extremes of wave and current were regarded as an essential part of the design package.

However, the advent of API LRFD (API 1993) has provided a more rational method of wave load calculation and realistic values of Cd for rough members. With this new method it is essential to consider combined probability of wave and current if wave loads are not to be increased.

## Strength Criteria

One of the problems with reassessment of older platforms is that strength criteria, particularly the strength criteria for tubular joints given in the API RP 2A, have changed considerably in recent years. The changes in predicted strength can show considerable reductions for some joints. This obviously causes problems when a platform, which was designed to an early edition of the codes, is checked to a later one. In one particular case a reanalysis of a platform indicated almost 50 joints overstressed. None of these joints failed if the previous edition of the code was used.

The introduction of the API LRFD will lead to further changes both in the calculation of load and of resistance. Components that have a high percentage of environmental loading will probably have higher interaction ratios using API LRFD. If these members were marginal under API WSD then they will probably be shown to be overstressed using API LRFD.

Because of the uncertainty in calculating joint strengths in the codes, some operators have decided to perform full scale punching shear tests on a particular problem joint. This can be very cost effective when there are a number of existing platforms with very similar joint geometries.

For most structures however the joints and associated failure loads will simply be too large for practical testing. In some cases a nonlinear analysis can, when properly specified and performed, give a good indication of the ultimate strength.

Nonlinear finite element packages can now simulate such ultimate behavior to a fair degree of accuracy. Large deformation and elastic-plastic behavior with strain hardening can be modeled. Good solution schemes can also pass the critical point into the post-buckling branch of the solution. Rupture strength can be specified to simulate the tensile tearing limit. For tubular geometry, good shell and curved beam elements are also required and these are now available.

Greater use of this type of analysis may be expected in the future. It is important that any method should be correlated with full scale test data.

## System Reliability

The design of an offshore structure is a compromise between cost and safety, and ideally the designer would be required to demonstrate that the risk of a total system failure is less than a given statutory required level reflecting the consequences of such failure.

At present the strength acceptance of an offshore structure is based mainly on the satisfaction of code checks which check the integrity of the component members for an envelope of design cases. Uncertainties in estimating the design loads and component strengths are allowed for by the application of safety factors. The analyses undertaken are linear and do not account for the redistribution of member loads as yielding occurs.

Code checks do not provide a ready guide to the overall strength reliability of a platform. It is impossible to quantify statistically the conservatism built into the codes by way of safety factors and characteristic material and strength values. The codes are also component based and do not consider the overall failure of a structure.

It is very probable that more emphasis will be given to system reliability in future, rather than component failure. To date, most, but not all, requalification work in the North Sea focuses on code compliance. However I believe this will change in the future for the following reasons:
- The introduction of the Safety Case approach will allow a move away from pure code compliance to consider the full safety implications. Installations that can show a satisfactory level of overall strength (system reliability) can be accepted even if individual components do not meet the code requirements. This will also allow aspects such as manning and the risk of environmental pollution to be included in the equation.
- The availability of nonlinear analysis programs and the increasing power of computers has made it commercially feasible to estimate the collapse strength of a platform. Such a calculation allows for the robustness of the design and is clearly a better guide to the overall strength than the present limited code checks.

One disadvantage of this approach is that once we move outside the design codes and recommended practices, such as API RP2A, there are no industry guidelines as to how safe a structure must be to be acceptable.

It could be based on overall reliability and the achievement of a satisfactory safety index. Considerable efforts have been made to establish the applied loading and strength of a platform as stochastic variables, so that the strength reliability of the platform over its design life can be estimated. However, in our opinion the industry is not yet at the stage where such a calculation is sufficiently accurate to be relied upon and further work needs to be undertaken.

Alternatively the assessment could be based on a safety factor against overall collapse based on the design load. This may be a better way forward. However there is no guidance available as to how large this safety factor should be. What level of overall safety factor would enable a platform, with seriously overstressed or failed local components, to be accepted? This will obviously depend on the consequences both in terms of safety of life, pollution and economic loss.

It would in our opinion be very helpful if some guidance on these questions could be included in the API RP 2A, particularly as it is now being developed as an international design guide.

The strength reliability needs to take into account the probability of periods of reduced strength caused by damage or fatigue. The probability of a fatigue failure occurring on a critical member should be kept to an appropriate level by a schedule of inspections.

Where structures have been damaged for a period of time it is necessary to take account of the increased fatigue damage caused by the higher stress level in members adjacent to the damage. Increased inspections in this area may be appropriate.

# Fatigue

Fatigue is an important aspect for areas of the world such as the North Sea. For example fatigue life can be very significant when considering the frequency of cleaning required to remove marine growth. It will also have a significant impact on inspection requirements.

Traditional fatigue analysis bases the selection of SCF on joint geometry alone. In addition only loading in a single plane can be considered. While these assumptions are generally conservative they are not uniformly so for all joints. This not only leads to conservative predictions of fatigue life but also to an unreliable hierarchy of critical joints. This is misleading information for inspection.

Because of recent advances in fatigue analysis these generally conservative assumptions are no longer necessary. The SCF can be determined from the actual load pattern in the joint. In addition the loading in members in different planes can be considered. (Fisher and Fidler). This provides more realistic fatigue lives and a more accurate hierarchy of critical joints to guide inspection.

# In-Service Inspection

After requalification, a planned inspection program should be implemented. Ideally, the requalification process will provide guidance on the inspection required. Parts of the structure where the design code has been exceeded, or where there are low fatigue lives, repairs (such as bolted clamps), corrosion damage etc., will require more frequent inspections.

However, in many cases there will be no design necessity (no low fatigue lives, etc.) and no suspected damage necessity (due to dropped objects, corrosion, etc.) for inspection. In the United Kingdom, as elsewhere I am sure, there are considerable pressures to reduce inspection as it can represent up to 40% of operating costs. However, there is no general agreement in the industry as to what is the minimum level of inspection consistent with safe and reliable operations, particularly for installations where there is no special need for inspection.

It is a fact that unexpected and unpredictable problems do occur occasionally on some installations. But what inspection is required to reliably detect these before a major incident occurs?

The inspection techniques adopted by various operators differ. Winkworth and Fisher (1992) gives details of the different inspection philosophies adopted by United Kingdom operators to date. However, at present there is no reliable research data or other information as to which is best.

The level of inspection will also depend on the following:
* The type of environment.

- Whether the platform is considered manned or unmanned. This consideration is now possible under the new safety case legislation.
- Economic consequences of loss.
- Regulatory requirements.

Other aspects such as platform redundancy and the safety factor against overall collapse may also be taken into account. Again there are at present no accepted practices or industry guidelines as to how these factors should be used in determining the level of inspection required.

If there are no safety of life or environmental/pollution risks is it then a question of satisfying company economics and insurance requirements, or are there questions of public confidence to consider?

It is probable that in the future inspections will be risk driven. However the goals, in terms of the reliability required, and the input data, in terms of the reliability of inspection methods and probability of unsuspected failures, need to be defined and developed in order to achieve satisfactory results.

## Verification

In the North Sea offshore installations are subject to verification by an independent body. In the author's view this will become even more important for the requalification process. The reasons for this are that in the future requalification will involve the following:
- Increasingly sophisticated review of environmental data and design assumptions.
- Complex analyses to prove system reliability.
- Available conservatism in the strength assessment will increasingly be utilized.
- Inspection program optimized to reduce cost.
- Greater subjectivity in the assessment process.

## Charge to the Working Groups

I propose the following questions for the working groups to consider:
- Requalification. When is requalification required?
- Strength Assessment. What use can and should be made of overall strength/system reliability in requalification and can the industry provide guidelines on this?
- Inspection. What is the safe minimum level of inspection required and what methods of inspection should be used when there is no design need to inspect?
- Verification. What verification is required in view of the increasing complexity, sophistication and subjectivity of the requalification process?

## Conclusions

The present position regarding requalification of offshore installations in the North Sea has been summarized. Some of the more significant areas of uncertainty, as far as the author is concerned, have been raised and hopefully the conference will address these.

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# SECTION II

White Papers

# INSPECTIONS, SURVEYS AND DATA MANAGEMENT

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## Abstract

Reassessment and requalification of offshore production structures require a thorough understanding of the existing condition of the structure. The current state of inspection practice includes a variety of methodologies to plan, capture, and record information. Different inspection schemes are employed by industry, and these vary by region and operator. This paper outlines the current state of practice related to inspections, surveys, and data management for offshore structures. The paper includes the advantages, disadvantages, and costs of the various inspection schemes. The types of structural damage that are critical to reassessment are identified. An estimate of the level of acceptance and utilization of each of the inspection schemes by the industry is noted. Finally, important research and development needs exist and are noted.

## Introduction

The number of offshore structures throughout the world exceeds 7,000. Many, if not most, structures are installed in mature producing basins and have provided acceptable service for decades. Some of these structures have, in fact, provided service far beyond their original intended life (Bea et al. 1988). Steel template type structures were installed in the Gulf of Mexico (GOM) beyond the sight of land as early as 1947. Major existing structures in the GOM, critical to the pipeline infrastructure, date from the late 1940s. The development of the North Sea (NS) lagged the GOM by several years; nevertheless, there are many significant structures in the North Sea approaching 25 years of age.

Operators, owners, and regulatory bodies face a difficult challenge: to safeguard life and resources, yet provide an environment in which oil and gas reserves can be developed and profitably produced. The collapse of oil prices in the 1980s has put tremendous pressure on producers to eliminate waste. Inspection and intervention programs have been heavily scrutinized and pressured to cut costs. All inspection programs in today's environment should be as inexpensive as possible while meeting the needs of the assessment and requalification practices.

This paper provides a brief overview of platform inspection practices which include relative costs, advantages, disadvantages, and general acceptance by the industry. It includes a discussion of the types of damage that have been discovered and provides a general overview of inspection strategies based on business, safety, and regulatory requirements.

## General Inspection Considerations

The underlying philosophy of a platform inspection program is to:
* Identify and quantify all damage or conditions in a structure that can affect the integrity or future serviceability of the structure
* Obtain the necessary information as cost effectively as possible
* Minimize the chances of missing damage that could compromise the integrity of the structure.

The planning, scope, and documentation of the inspection must be in harmony with the assessment process. An inspection plan that does not detect serious conditions, inadequately identifies conditions, or spends too much time on insignificant anomalies is wasteful.

To properly plan and identify conditions and defects which can seriously affect the integrity of the structure in the long or short term, it is necessary to have an understanding of the structure's design criteria, behavior under load, its

intended purpose, expected life, operational considerations, and past exposure. The United Kingdom's Department of Energy Guidance Notes (Offshore Installations) state that:

"In-service inspections of a fixed installation should be planned by an experienced engineer who has examined the design characteristics, the records of severe environmental and other loads to which the structure may have been exposed and any available records of structural behavior such as settlement, differential settlement, tilt, distortion or abnormal response. The initial inspection schedule should take account of the nature of the deterioration to which steel and concrete structures are liable in a marine environment and of the regions in which defects are most prone to occur (e.g., sudden changes in section, discontinuity, etc.) and of members or regions known to have been, or likely to have been, highly stressed or subjected to severe fatigue loading."

The aforesaid cannot be overstated. It is critical in the inspection process to include guidance by individuals with appropriate knowledge and expertise in this area.

The consequence of failure should be weighed in the planning and scope of the inspection process. A small, unmanned well jacket with appropriate subsurface safety valves has a significantly smaller consequence of failure than a major manned structure. The level of consequence impacts the expected life cycle economics of a structure for the greater the consequences of failure, the more advantageous to prevent failure.

## Level and Frequency of Inspections

The appropriate level and frequency of inspection are difficult to establish due to a multitude of interrelated factors which affect the likelihood of damage and their consequences. In general, one must consider the:

Robustness of the design (e.g., conservatism, redundancy)
Construction practices during fabrication
Quality control during fabrication and installation
Material properties
Loading history
Cathodic protection history
Experience with structures in the general location
Performance of similar designs
Consequences of failure
Cost of repairs versus inspection costs
Age and fatigue susceptibility
Desired life of the facility
Past inspection results

All too frequently, owners and regulatory bodies lack some of the information listed above. Lack of information may require additional inspection procedures to determine what loads, materials, wall thickness, member sizes, etc. are in place.

## Planning and Documentation

Without proper planning, inspection programs are likely to be inefficient and not provide the necessary information for the assessment phase. Proper planning should include careful selection of the inspection contractor (and possibly personnel), a detailed scope of the inspection plan including specifications for the inspection and documentation, and a clear strategy of how to alert appropriate personnel of significant damage. Up-to-date drawings that effectively convey important locations and details of the structure to the inspection personnel are very important. It is recommended that a checklist be developed to insure that all necessary locations are examined and documented.

Reports that effectively communicate the true condition of the structure are critical to this process. There have been many glossy inspection reports produced over the years that have little intrinsic value. Proper documentation must include a clear description of all anomalies including exact location and accurate measurements. A description of damage that does not include required engineering measurements is generally of little value. The phrase "dent in horizontal" provides inadequate information for proper assessment. The phrase "dent in 24 in. horizontal 14 in. long

and 3 in. deep top dead center 5 ft. from node A1" with a backup sketch and photograph gives the assessment engineer sound information to make the proper decision.

## Data Management

The large amount of information collected in an inspection and the long period of time for which the information must be maintained make careful consideration and planning of data management imperative. The current state of practice is to maintain some information such as sketches, drawings, and pictures in hard copy format while other information, such as cathodic potential readings, may be available in electronic form as well as hard copy. The trend is to capture more information in electronic format due to the formatting and summation capabilities available in many computer systems. The current cost of capturing and storing pictures and drawings in an easily accessible electronic format generally limits its use for pictures and sketches. Relational databases have proven to be effective for the storage and retrieval of inspection information. Several companies edit and enter data into a database during the actual inspection process in the field. Real time capture of information on video tape, such as cathodic potential readings, is standard practice; however, real time capture of information directly into a database is rare. As hardware and software evolve, we expect to see greater application of computer graphics and digital capture of pictures, sketches, and information in real time. More sophisticated database techniques such as object oriented databases are likely to be employed also.

## Above Water Inspections

Because of different strategies and costs, above water inspections are treated separately from underwater inspections. Above water inspections cover those parts of the structure that can be accessed without physically entering the water. The splash zone presents a special case because of the difficulty accessing the area. Since this location is often the site of damage, it is important to inspect this area from above and below water as thoroughly as possible.

The types of damage or defects typically noted during above water inspections include deflected members, dents, tears, cracks, holes, deformed shapes, unusual deflections, missing members, and severely corroded members. Other conditions that are factors in structural assessment include the deck height elevation, tilt of the structure, and possible structure motion. Most damage is a result of corrosion or vessel impact. Vibration of structural members due to large unbalanced forces in rotating machinery has resulted in some cracking of structural members near the connection point of skids. One GOM operator reported instances of cracks in piling in the jacket-to-piling connections due to the material used as filler bar.

## Frequency

The relative low cost and potential high benefit of above water inspections makes them an annual requirement by many governing bodies. The U.S. requires that the above water portion of all fixed offshore platforms (beyond 3 miles from shore) undergo a structural inspection each year (API 1993; MMS 1991). The U.K. and other North Sea countries also require annual topside structural surveys.

## Inspection Methods and Tools

The primary topside inspection technique is visual. Trained individuals survey the platform for anomalies. A hammer is sometimes used to knock away corrosion to determine the soundness of material beneath. Nondestructive testing (NDT) methods such as ultrasonic testing (UT), magnetic particle inspection (MPI), eddy-current testing (ET), and radiographic testing (RT) are used to a much lesser extent than visual—generally only when damage is suspected by visual surveys. Each of these NDT techniques have certain advantages over others. UT is best suited for full penetration welds of simple geometries and is effective in locating subsurface cracks and slag in the weldment. MPI is favored for detecting surface imperfections such as fatigue cracks and is judged better than UT in detecting small tight cracks. ET is a relatively new NDT technique used to identify shallow cracks primarily beneath coatings.

RT is most suited for detecting inclusions, porosity, and some cracks; however, the orientation of the crack must

be perpendicular to the film. RT has the advantage over UT and MPI in that it naturally produces a film record of the test, but suffers from safety concerns related to the radioactive source. UT and MPI are not as effective on painted surfaces as on bare metal due to coupling problems and loss of magnetic flux and, therefore, careful procedures must be employed when using either technique on painted surfaces (Mahmoud and Abrams 1992).

## Underwater Inspection

Costs for underwater inspections are significantly higher than above water inspections. Typical GOM underwater inspection rates range from $5,000 to $20,000 per day and considerably higher when saturation diving is involved. Inspection rates in the North Sea are as high has $50,000 per day. Examples of underwater damage are similar to those topside, but also include collapsed members. Other conditions that should be noted include loose risers and cables, excessive wear, scour at the base of the structure, thickness and location of marine growth, condition of protective coating systems, and cathodic protection potentials. Loose connections or locations where one piece of material rubs against another are frequently the location for very high corrosion rates. Cables draped over and along braces have worn holes and severed members in a few years. Scrap metal in contact with the structure reduces the cathodic protection of adjacent areas and can cause severe local corrosion.

## Frequency and Level of Inspection

In general, most underwater inspections are scheduled on a periodic basis ranging from one to five years. A nonscheduled inspection may be conducted after a major storm, boat impact, or analysis that would indicate a potential problem exists. However, where there are no specific problems (e.g., low fatigue lives, overstressed parts, known damage), the question arises as to how often and to what level should a structure be inspected to insure that unexpected or random damage does not exist.

Factors that influence the frequency and level of inspection are listed earlier. Differences in these factors can have a profound effect on the philosophy and strategies behind inspection programs. It is worth contrasting the factors that differ between the Northern North Sea and the Gulf of Mexico to demonstrate how these factors influence the appropriate level and frequency of inspections:

- Gulf of Mexico—The prevailing wave climate is relatively mild in comparison to the extreme loading event (i.e., hurricane). Fatigue damage, while a factor, is not as significant as in other parts of the world. Platforms can be de-manned and shut-in in advance of a hurricane which reduces the consequence of failure. The cost of repairs is relatively low and the long-term experience with structures is very high.
- Northern North Sea—Extreme loading events occur frequently and without sufficient warning to evacuate. The structures tend to be major investments and manned which makes the consequences of failure very high. The prevailing wave climate is high compared with the design loading event which increases the likelihood of fatigue damage. The cost of repairs is significant and long-term experience with structures is moderate.

Because of these factors, the inspection strategies differ between these two geographic areas.

## Gulf of Mexico Inspection Frequency and Level

The inspection strategy recommended for the GOM by API RP2A and required by law is briefly described as follows (API 1993; MMS 1991). A manned structure in good condition is to be inspected by a Level II survey every 3-5 years. A Level II survey is a general visual inspection for gross underwater damage. It includes a survey of cathodic potentials, debris, marine growth, and scour. RP2A recommends a Level II survey every 5-10 years for an unmanned structure in good condition. Federal law, however, requires that all structures beyond three miles from shore be inspected at least once every five years. A higher level survey which includes close visual inspections of cleaned joints (Level III) and optional nondestructive testing (Level IV) is required of selected joints at least once every 6-10 years for a manned structure and once every 11-15 years for an unmanned structure.

The number of member ends to clean and inspect in a Level III survey is not prescribed by code. Level III inspections are to be performed on areas of known or suspected damage and on pre-selected areas "based on an engineering evaluation of areas particularly susceptible to structural damage or to areas where repeated inspections are

desirable in order to monitor their integrity over time." Typically, less than 10 percent of the primary member ends on a given platform are cleaned and inspected in a Level III survey.

Level IV surveys are intended to quantify damage. They consist of underwater NDT, such as MPI, and are conducted on areas suspected to have damage. Many operators in the Gulf of Mexico do not inspect routinely with MPI during fabrication so performing MPI underwater for the first time on a weldment can lead to many false alarms, particularly if inexperienced personnel are involved. MPI generally is reserved for locations where cracks are judged likely or potentially significant such as in non-redundant, critical joints. While the authors have not completed a detailed analysis, it is believed that the cost of performing random or extensive underwater MPI on a routine basis would far outweigh the savings in preventing premature joint failures. Data collected by Sea Test Services shows the incidence of cracks to be fairly low. Of 16,000 joints inspected (1500 with MPI) only 53 had crack indications. Forty-nine of these indications were in conductor guide framing. Virtually all of the damage detected was in the top 100 feet of water and generally located in the toe of weldments (Sea Test Services 1989).

The collapse of redundant structures in the GOM are few and are almost exclusively confined to extreme loading events such as hurricanes. Waves impacting decks are thought to be the prime contributor to the collapse of structures in the GOM. The effects of prior fatigue damage, including low cycle, high stress fatigue, on the failure mechanisms are difficult to estimate due to the wide scatter in fatigue behavior and level of information required to make reasonable estimates. Often, the contribution of fatigue in postmortem discussions is omitted due to the difficulty in estimating the effects. More work on the effects of fatigue in the ultimate collapse of structures is warranted.

Corrosion damage has been a problem in many GOM structures, especially in those installed before the mid 70s. Random cleaning of small areas in the structure during a Level III inspection has proven to be effective in assessing the level of corrosion in representative areas and in providing early warning of serious problems. Severe local corrosion damage in weldments and in conductors at conductor guides have been observed using this technique and subsequently arrested.

## North Sea Inspection Frequency and Level

The frequency and level of inspection in the North Sea are generally greater than in the GOM. While require-ments for inspections are still evolving, the current practice is to perform detailed inspections of all portions of the structure every five years. A "major survey" which encompasses inspection requirements similar to API Level II through IV is completed for all portions of the structure.

Platforms are generally manned during the design loading event so that loss of life is a major consideration in selecting the level of inspection. Fatigue is the primary source of joint damage in the North Sea. This type of damage is difficult to predict with great accuracy due to the inherent high sensitivities to cyclic stress levels and stress concentra-tion factors. A prudent operator must inspect relatively often and at a sufficient level to insure that the structure is not in imminent danger of collapse due to fatigue damage.

Because the cost to repair fatigue damage can be minimized by catching the damage early and by grinding out the damage, there is a strong impetus to detect small fatigue cracks (Tweed and Freeman 1993; Winkworth and Fisher 1992). This is also important if use is to be made of crack propagation data in planning future inspections. Close visual inspection (CVI) of cleaned welds was not found to be very effective for detecting damage in the North Sea for a number of reasons. Full scale tests of tubular joints conducted in the laboratory revealed that small through thickness cracks are not visible unless the member is under some tensile load. In the calm weather required for diver inspection, most jacket members are in compression and it is unlikely that anything but serious damage could be discovered by close visual inspection alone.

Considerable emphasis is given to detailed joint inspection using MPI or ET to detect fatigue cracks. However, for structures that have been properly designed to avoid low fatigue lives, it would appear that flooded member detection (FMD) of all major braces would generally be considered adequate for a Major Inspection since the process will detect through thickness cracks at any point on an unflooded member. It must be noted, however, that cracks in joint cans of leg members may not be uncovered since legs are often flooded.

## Underwater Inspection Equipment and Procedures

Underwater inspection equipment and procedures vary significantly depending upon the water depth, level of

inspection, and local costs. Inspections in less than 200 feet of water are generally conducted by qualified divers. Divers have good mobility and are easily adaptable to different situations. As the depth increases, however, the cost and logistics of divers increase significantly. Divers are less mobile vertically below 200 feet due to decompression requirements. While divers have worked at deeper depths, the practical limit of most diving operations is 1,200 feet.

Remotely operated vehicles (ROVs) have been successful in working at water depths well beyond the current depth of offshore platforms. Operation at depths beyond 6,000 feet is common. ROVs are gaining in abilities, but the most sophisticated are still less capable than a diver. Remotely operated vehicles are well suited to general visual inspection with cameras and video. Cleaning and inspection packages can be added to the larger vehicles. The use of an ROV to economically clean and perform close visual inspection of member ends in 1,025 feet of water was successfully demonstrated in 1989. Manipulators have evolved significantly over the years. Nine function spatially correspondent arms are the most advanced in use. Force feedback manipulators are still experimental in these vehicles, but are expected to have a significant impact on improving the utilization of the equipment for cleaning and NDT.

Atmospheric diving suits (ADSs) fill a niche between divers and ROVs. This equipment permits a human to descend to depths of approximately 2,000 feet without the need for decompression. They have an important advantage over ROVs in that the operator has a better perception of the surroundings, has dexterous arms, and receives force feedback. These suits have been quite successful at performing activities including NDT, FMD, operating valves, grinding out cracks, drilling holes, and other light duty activities.

Cameras are very necessary tools in the inspection process. Cameras form a permanent record and give those making assessment decisions very clear and detailed information. The mainstay is the still camera. Many inspection firms have on-site film developing and printing capabilities which permits film to be checked while the crew is in the field. Stereo photography, such as provided by the Photosea 2000, provides excellent depth perception and photogrammetric capabilities, but requires careful film development on shore. Video cameras are used extensively to provide documentation of the inspection. Monochrome cameras can produce excellent images in low light and at very close ranges. These cameras are often used to document crack indications. Color cameras offer additional information relative to color cues. Locations with very turbid water require special "clear water box" cameras. These cameras provide some type of shroud around the area to be photographed and flush clear water in the space to provide a clear medium for photography.

The first step in a Level III inspection (close visual inspection) is the removal of marine growth in a specific area. A member end or node, for example, may have marine growth removed 6 inches on either side of all welds to investigate for cracks. Several techniques are available and each has found a niche. Wire brush cleaning is one of the most common methods for cleaning surfaces under water. Brushing alone is an acceptable technique for cleaning thin layers of marine growth, algae, and black oxide, but is not very efficient for cutting through heavy, encrusted marine growth. Brushes have been employed by ROVs with limited success. The major problems with ROVs are the difficulty in accessing tight locations and the lack of force feedback that makes control very difficult. Water blasting with a high pressure stream of sea water is a better technique to knock off highly encrusted areas. It is used frequently by divers and to a lesser extent by ROVs. Water blasters are, however, inefficient at removing black oxide from steel surfaces. A common technique is to knock off the heavy marine growth with water blasters and follow behind with a wire brush. Typical cleaning rates for this approach vary between 4 and 13 minutes per foot of weld (i.e., a swath 6 in. on either side of the weld). One efficient practice is to water blast only and follow with MPI. This technique quickly locates cracks, but generally provides a poor video or pictorial image of the geometry. It is often very difficult to determine the precise location of the crack relative to joint features (e.g., the weldment or intersection points) due to the camouflaging effects of the black oxide.

A variation of the water jet is to add sand or grit to the water stream. This has shown to be more effective in cleaning down to bare metal, but has experienced operational problems. Another system which has been found to be highly effective in water depths up to 300 feet is a traditional sand blasting system modified for higher working pressures. Sand and compressed air are ejected at about 150 psi over nominal pressure. This system quickly removes black oxide and leaves a mat surface well suited to photography. It has proven to be 100 to 200 percent faster than the traditional water blaster plus wire brush approach. A cautionary note with water and sand blasting is the danger of injury to the diver.

Linear measurements underwater tend to be done as on dry land. A ruler or tape is often used to make measurements. A taught wire stretched between two points, which can be measured on the surface, is another solution. Photogrammetry has been highly successful in a few instances, but it requires good visibility. A promising new technology is laser imaging (Leyat and Mutius 1986).

Cathodic potential measurements are a very important part of the inspection process. Cathodic protection (CP) systems such as impressed current or sacrificial anodes protect the steel from the corrosive forces of sea water. The

importance of these systems has shown to be very dramatic, but these systems must be checked periodically. Significant damage has been done to new structures in a few short years with improperly designed or maintained cathodic protection systems. Impressed current systems require a fair amount of maintenance to ensure proper operation, and sacrificial systems exhaust their supply of material in time. Both systems can leave certain parts of a structure unprotected. A potentially more serious condition is to have different electric potentials between risers and the structure, or between well conductors and the structure that can cause very high corrosion rates where parts come in close contact.

One of the simplest and least expensive survey techniques to gauge the overall condition of the CP system is the "drop cell." The drop cell is a probe lowered into the water from above. The drop cell measures the general level of protection for the area immediately next to the probe. The probe may not pick up all local deficiencies, but is very reliable in determining the global protection level provided to the structure. Drop cell measurements are typically conducted each year in the GOM as part of the MMS mandated topside inspection.

A variety of other cathodic potential measuring devices exist which can be held by divers or ROVs. These devices are useful in obtaining detailed information on the local coverage of the CP system. A general survey of these devices is given by Britton (1991).

Nondestructive testing has been described earlier for above water applications. Underwater applications are similar, although there are a few additional challenges (API 1990). The three major difficulties with ultrasonic testing (UT) underwater are that it is not well suited to detect shallow surface cracks typical of fatigue damage, it is difficult to make a permanent record of the flaw, and there are few individuals properly qualified as both a UT technician and commercial diver. Magnetic particle inspection (MPI) is more common underwater. MPI has shown to be highly effective in detecting cracks on cleaned steel surfaces and on surfaces that have been cleaned only to the black oxide coating. Photographs or video can be made of the dye on the surface being tested and reviewed by appropriate staff. Pouches with dye and epoxy are available which can make a permanent impression of the flux field (Watt et al. 1989).

Flooded member detection (FMD) devices can be used to see if a normally unflooded member has a through wall crack. This technique is gaining acceptance in both the NS and GOM to more economically determine if a member has a through wall crack versus performing MPI of the member ends. The disadvantages of the technique are: it can only detect cracks extending into normally unflooded members, it cannot distinguish between a pin hole and a crack, and it requires good technique. Flooded member detection devices use radiographic or ultrasonic technologies. The ultrasonic FMD device requires a relatively clean surface near the low point on the member to be tested. North Sea operators generally regard radiographic (gamma transmission) FMD as easier to apply and more reliable. Flooded member detection checks are generally considered Level III type inspections rather than Level IV inspections because additional NDT is required to fully identify the extent of the suspected damage.


## Schemes to Monitor for Damage

To reduce inspection costs, a number of monitoring systems have been developed to monitor the status of some given condition. One technique, which was successfully employed in the North Sea, monitored crack growth through acoustic emissions (Mitchell and Rogers 1992). This system was installed to ensure structural integrity until repairs could be carried out under better weather conditions. Another approach that has had limited success attempts to monitor the dynamic signature of a structure using changes in the dynamic response to the random sea conditions (Rubin and Coppolino 1985). This technique has difficulty detecting all but major damage and requires a fair amount of instrumentation. Difficulties with accounting for deck mass changes also cause problems with this technique.


## Underwater Inspection Strategies

Most companies inspect platforms on a routine schedule with little justification for the frequency or particular inspection strategy. Effort has been made by a few operators to develop a scientific rationale for their inspection level and frequency based on a probabilistic approach. One such operator, Conoco, has applied Monte Carlo simulation to their inspection program for the Viking field. The simulation used fatigue life predictions, the capabilities of the various inspection techniques, remedial action plans, and past inspection records to analyze various inspection strategies (Carr et al. 1986). The study developed MPI inspection interval charts for joints and made provisions for increasing the inspection interval for specific joints when no crack-like indications were found during the previous inspection.

## Underwater Inspection Personnel

There is currently no standard for defining the qualifications of an acceptable inspector. Personnel should be qualified and certified similarly to that required for U.S. bridge inspections. This will require training and testing of inspection personnel. It is recommended that industry and regulatory organizations such as the ADC, ABS, Lloyd's, and DnV jointly develop and share such training and certification programs with the industry.

## Conclusion

Inspection techniques are advancing as the need for greater understanding of existing offshore platforms grow. Platform inspection and assessment may be the greatest challenges facing the offshore industry today. The level and frequency of inspections are influenced by several factors that have been discussed. Many development efforts are improving the state of inspection art; however, more work remains to be done. In the face of strong economic pressure, the industry needs to share knowledge of inspection techniques, new developments, and the performance of old structures.

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# ENVIRONMENTAL CONDITIONS AND FORCES

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## Abstract

Offshore structures are designed to withstand extreme environmental forces due to storms (winds, waves and currents) and, in some cases, earthquakes and ice. Environmental conditions for design are selected on the basis of their probability of occurrence during the design lifetime of the structure; typically, conditions with an exceedence probability of 1/50 to 1/100 per year are selected. The forces caused by those environmental conditions are calculated using the most appropriate methodology consistent with the current state of technology. As offshore technology has matured, both the environmental conditions and the force calculation procedures used to design new offshore structures have evolved. Meanwhile, many older structures, designed for different conditions using different procedures, have continued to be used beyond their original design life. In many situations, there are compelling economic forces to extend the use of these structures. But, before doing so, we must assess their "fitness for purpose."

This paper raises questions about the environmental conditions and force calculation procedures that should be used to requalify older offshore structures for continued use, and how and why they might differ from those used for designing new structures. These questions are posed from a historical perspective, with an international point of view, within the context of the uncertainties of extreme environmental forces, and with a full awareness of the economic considerations. This paper does not provide answers to these questions. Rather, its purpose is to provide food for thought and to stimulate discussion at this workshop.

## Introduction

The problem of requalification of offshore structures is perhaps best introduced by Table 1 (Whitfield 1993), which shows the worldwide distribution of offshore platforms. There are more than 7,000 offshore platforms worldwide. Many of these structures were designed, constructed, and installed more than 25 years ago and are still in service.

In the Gulf of Mexico, offshore development began around 1950. Many of the structures in less than 150 ft water depth were constructed before 1970. Some of these were designed to the 25-year wave, not the 100-year wave, with no consideration of current, and with drag coefficients appropriate for cylinders without marine growth. Many had no cathodic protection in their early life. After Hurricanes Hilda (1964) and Betsy (1965), some operators reevaluated their platforms, with the result that some decks were raised to reduce the risk of inundation in large waves, and some jackets were strengthened with pin piles and tripods. Many of the platforms from that era have little or no excess strength above the environmental forces that would result from application of criteria and procedures in the most recent, 20th edition of API RP2A. The 20th edition requires that new structures be designed for forces caused by the 100-year wave and associated storm current of about two knots, with drag coefficients appropriate for marine roughened cylinders.

Experience gained in development of Gulf of Mexico oil and gas reserves was transferred by major international oil companies to the subsequent development of offshore fields in other continents. Notable examples are the "first generation" Bass Strait structures and the earliest southern North Sea gas field structures.

Recognizing that: estimates of extreme, "design" storm conditions evolve as more wind, wave, and current data become available, either from measurements or from numerical simulation of historical storms; wave force calculation procedures evolve as more data become available from laboratory experiments and from full-scale platform response measurements; and it is impractical and hazardous to evacuate platforms in the North Sea in the face of frequently and rapidly approaching storms, the United Kingdom government set a requirement that every offshore structure be

**Table 1  Worldwide Distribution of Operational Platforms.**

| | |
|---|---:|
| Northwest Europe | 379 |
| Southern Europe | 118 |
| Eastern Europe | 43 |
| North Africa | 145 |
| West Africa | 509 |
| South and East Africa | 1 |
| Middle East | 806 |
| India | 145 |
| Southeast Asia | 811 |
| East Asia | 41 |
| Australasia | 36 |
| North America - Alaska | 14 |
|           - California | 35 |
|           - Canada | 3 |
|           - Gulf of Mexico* | 4000 |
| Latin America** | 428 |

| | |
|---|---:|
| World Total | 7514 |

*Estimate
**Excludes Lake Maracaibo

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recertified at five-year intervals. In this recertification process, deviations may be made from the current government guidelines for environmental forces, provided the operator can justify to the certifying authority that there is a good basis for the deviations.

In Norwegian waters, requalification is required when the platform function changes, i. e., when topside weight or number of appurtenances changes. Furthermore, requalification is necessary if evidence of deterioration is found or if the design life is exceeded. Requalification is done on the basis of the original design; however, new criteria are applied in the case of apparent under-design, e. g., if the wave height or hydrodynamic force coefficients in the original design basis were too low. For platforms with little risk of pollution and small economic consequences in the case of loss, the Norwegian Petroleum Directorate (NPD) allows a reduction of the load factor from 1.3 to 1.15.

In the United States, the government has not had formal requalification requirements, unless there has been a substantial change in use of a platform, in which case the platform must comply with current requirements. Indeed, there has not been a compelling need for requalification, since there have been only a few failures of fixed structures in hurricanes in the last 30 years (33 before Hurricane Andrew and 14 in Andrew), and these have not resulted in significant pollution or loss of life. However, even with today's low oil prices, it is still economically attractive to continue to produce oil and gas from some platforms that have already exceeded their design life. Therefore, encouraged by the Minerals Management Service, and in the face of a changing business climate (many more smaller operators), the API has begun to address the problem of platform requalification.

While the specific requalification criteria being developed by the API are limited to U.S. waters, the philosophy underlying their development is perhaps universally applicable. Therefore, the API criteria and underlying logic are presented more fully in the next section.

## API Requalification Criteria

In engineering practice, it is widely recognized that if an existing structure does not meet present day design standards, that does not necessarily mean that the structure is inadequate or unserviceable. Examples of this fact include buildings and bridges. With this background, and with the favorable survival experience in Hurricane Andrew (1992) in mind, the API Task Group 92-5 on "Assessment of Existing Platforms to Demonstrate Fitness for Purpose" has developed a draft process for platform requalification. It should be emphasized that the API requalification process is still evolving, and its final form may differ from what is presented here.

The plan is initially, over a reasonable period of time, to have all existing platforms go through the assessment process. Subsequently, an assessment will be triggered if: damage is found during inspections, manning conditions have changed, operational conditions have changed by the addition of facilities, or the design is altered such that environmental or functional loading is significantly (>10%) higher than for the original design.

A platform can pass the assessment process based either on design basis checks or analysis checks. A platform that passes the design basis checks need not be subjected to analysis checks.

### Design Basis Checks

Based on studies of platform exposure loads, ultimate strength, and survival experience, the API Task Group has proposed that a Gulf of Mexico platform designed according to the 9th edition of RP2A (1977), or later, is acceptable, provided that none of the four "triggers" listed above exist, and provided that the deck height is adequate. It must be demonstrated that the platform was indeed designed for the 9th edition reference level hydrodynamic loading; specific procedures are defined for this purpose. The required deck height is provided as a function of water depth. While experience indicates that platforms located in other U.S. waters that were designed to the 9th edition are also acceptable, blanket acceptance of such structures is not recommended at this time.

### Analysis Checks

The criteria for analysis checks depend on the platform's exposure category. There are six exposure categories, depending on the possible consequences of failure. There are three *life safety* categories: manned - not evacuated, manned - evacuated, and, unmanned. Similarly, there are two *environmental safety* categories: significant environmental impact and insignificant environmental impact.

For each exposure category, there are three levels of analysis. In order of increasing complexity but decreasing conservatism, these are: screening analysis, design level analysis, and ultimate strength analysis. A structure that passes a screening analysis need not have a design level analysis, and a structure that passes a design level analysis need not

have an ultimate strength analysis. The exception to this hierarchy is that any platform whose deck is lower than the crest elevation of the wave specified for ultimate strength analysis must be submitted to an ultimate strength analysis, accounting for wave loads on the inundated deck structure. Screening analysis is a simple procedure that has been validated as being more conservative than the corresponding design level analysis. Design level analysis is like that used in new platform design, including the application of all safety factors, the use of nominal rather than mean yield stress, etc. Ultimate strength analysis excludes all sources of conservatism, providing an unbiased estimate of platform capacity. In all three levels of analysis, wave forces for specified metocean criteria must be calculated using the methodology of the 20th edition of RP2A.

A different approach to defining metocean criteria is taken for the Gulf of Mexico than for other areas, since the industry standard contingency plan is to evacuate platforms in the Gulf when hurricanes approach.

For manned platforms without significant environmental risk, the criteria are based on "sudden hurricanes," i. e., those that arise too quickly for platforms to be evacuated. These hurricanes are a subset of the full population of hurricanes and are significantly less severe than the hurricanes that have their origin outside the Gulf of Mexico. Evacuations also do not take place during winter storms. The metocean criteria for design level analysis consist of the sudden hurricane wave and associated wind and current that give the 100-year base shear for the combined population of sudden hurricanes and winter storms. This requires a sudden-hurricane wave height, and associated wind and current, with a return period of about 150-200 years, depending on water depth. These criteria are applied omnidirectionally. In the ultimate strength analysis, metocean criteria are selected consistent with a reserve strength ratio of 1.8 relative to the 100-yr base shear for the combined population of sudden hurricanes and winter storms. This requires a sudden hurricane wave height with a return period of about 1000-2000 years, depending on water depth. Reductions in criteria are allowed for non-principal directions.

For platforms with significant environmental risk, the full hurricane population is used to derive metocean criteria. Here, emphasis is placed on exposure experience, especially that in Hurricane Andrew. The design level analysis criteria are selected to give a base shear equivalent to the 9th edition reference level, which is about 2/3 that of the 20th edition. This requires a wave height, and associated wind and current, with a return period of about 30 years. These are applied omnidirectionally. The ultimate strength analysis criteria are selected to give a reserve strength ratio (RSR) of 1.2 relative to 20th edition forces; an RSR of 1.2 relative to 20th edition forces is equivalent to an RSR of 1.8 relative to 9th edition forces. This requires a wave height with a return period of about 170 years, which is reduced for non-principal directions.

For regions outside the Gulf of Mexico, platforms are not evacuated. If the platform is manned or has significant environmental risk, the design level analysis is based on a reduction of 15% in the lateral loading caused by 20th edition 100-yr metocean conditions, and the ultimate strength analysis requires an RSR of 1.6 relative to the lateral loading caused by 20th edition 100-yr metocean conditions. The above raises several issues.

*Should a statistically significant sample of platforms that would pass the requalification process based on design basis checks be evaluated to verify that they would also pass the analysis checks, if required? (Issue #1)*

*Should the criteria for requalification of an old platform depend on the length of extended life sought by the owner? (Issue #2)*

*Should requalification be required at regular intervals? (Issue #3)*

*Should requalification be required whenever new information regarding the wave climate or hydrodynamic force coefficients becomes available? (Issue #4)*

*Should the criteria for requalification depend on the consequences of failure? Is the correct priority as follows: first, personnel safety; second, environmental pollution; third, economic / resource loss? (Issue #5)*

*What is the minimum acceptable RSR (ratio of ultimate capacity to the current estimate of load for the 100-yr metocean conditions) for platforms with either life risk or significant environmental risk? (Issue #6)*

# Environmental Conditions

While some offshore platforms are dominated by earthquake or ice loads, the vast majority are dominated by wind, wave, and current loads in severe storms. For this reason, and because of space limitation, this paper will be limited to the discussion of metocean (meteorological and oceanographic) conditions. Recent information on ice criteria and loads can be found in papers by Visser (1993) and by Utt and Turner (1992). Recent information on seismic criteria and loads can be found in the discussions by Craig et al. (1993) and by Iwan et al. (1992).

Here, we provide an overview of metocean data bases and procedures used to select "design" metocean conditions. Sources of uncertainty and magnitude of uncertainty are discussed, with an eye toward identification of implicit safety margins that could be stripped away in the reevaluation of an older platform.

## Data Sources

Estimates of rare, extreme storm conditions are based on extrapolation of historical data. The data may be either measured or hindcast. Hindcast data are wave, current, and surge data generated with numerical simulation models forced by wind fields reconstructed from historical meteorological charts. In order to be used confidently, hindcast models must be calibrated with measured data. As illustrated in Figure 1, careful application of wave hindcast models can lead to accurate prediction of wave heights (within ± 1m significant). To obtain such accuracy requires that the storm wind fields be reconstructed as thoroughly as possible, accounting for all the observed wind data, and enforcing time and space continuity of moving low pressure centers (Cardone et al. 1980; Reece and Cardone 1982). Also, the wave hindcast model must be of the directional, spectral variety, and must consider in some way the transfer of energy between different wave frequencies, as well as the transfer of energy to the waves from the wind and the dissipation of wave energy due to whitecaps and bottom friction.

Ideally, one would use measured wave data to extrapolate to the rare, extreme conditions used for design or reevaluation. In regions such as parts of the North Sea, where there are long-term measured wave data, this is viable. However, in most offshore areas, there is a paucity of measured wave data. If the data span is only a few years, no matter how accurate the data may be, they cannot be used confidently to extrapolate to extreme events with long return periods. As shown in Figure 2 (Wang and Le Mehaute 1983), for regions with frequently occurring storms, the uncertainty in 100-year wave height is very large for databases shorter than 10 years, and reaches "comfortable" levels only for databases longer than 20 years. This figure is based on the assumption that the underlying statistical distribution of storm intensities is stationary over the period of record. In the last few years, the frequency and intensity of storms in the Norwegian Sea have shown a significant increase above those of the previous 15 years (Barstow and Krogstad 1993). Therefore, even in regions with frequent storms, such as the Norwegian Sea, the database needs to be at least 20 years and preferably 30 years or longer to accommodate the apparent cycles and/or trends in storminess that may occur. In regions where the dominant storm type is a hurricane, and the frequency of occurrence of extreme hurricanes at a site is low (such as in the Gulf of Mexico), the uncertainty in the 100-year wave height might not reach a "comfortable" level until the database approaches 100 years. Therefore, a long-term database with no bias and moderate uncertainty (such as hindcasts) can provide a more reliable extrapolation to extreme values than a short-term database with no bias and small uncertainty (such as measurements).

Even if a long-term measured database does exist, one must be careful in extrapolating it. Of course, one must first remove all known biases in the measurements through careful calibration. However, even if the instruments were perfect, there remains the "sampling variability" (Tucker 1993). Although modern data collection systems have the capacity to collect and process long (greater than a half hour) records of wave data at frequent intervals (hourly), much of the historical measured data consist of a single 1,024-second sample once every three hours. This "sampling variability" inflates the variance in the historical wave distribution and leads to extrapolations of extreme wave heights that are biased high (Earle and Baer 1982), as illustrated qualitatively in Figure 3. A cursory review of data from 17 storms at the Forties field indicates that the storm-peak significant wave heights determined from a single 1,024-second sample every three hours are biased about 6% high, relative to three-hour-average values.

*Should the bias in design wave heights based on extrapolation of measured wave data be removed in the reevaluation of old platforms? (Issue #7)*

## Extrapolation of Data

Two methods are commonly used to extrapolate wave data to long return periods. The "continuous data" extrapolation has been commonly used in Europe. In this method, if "continuous" hourly or three-hourly data exist, either from
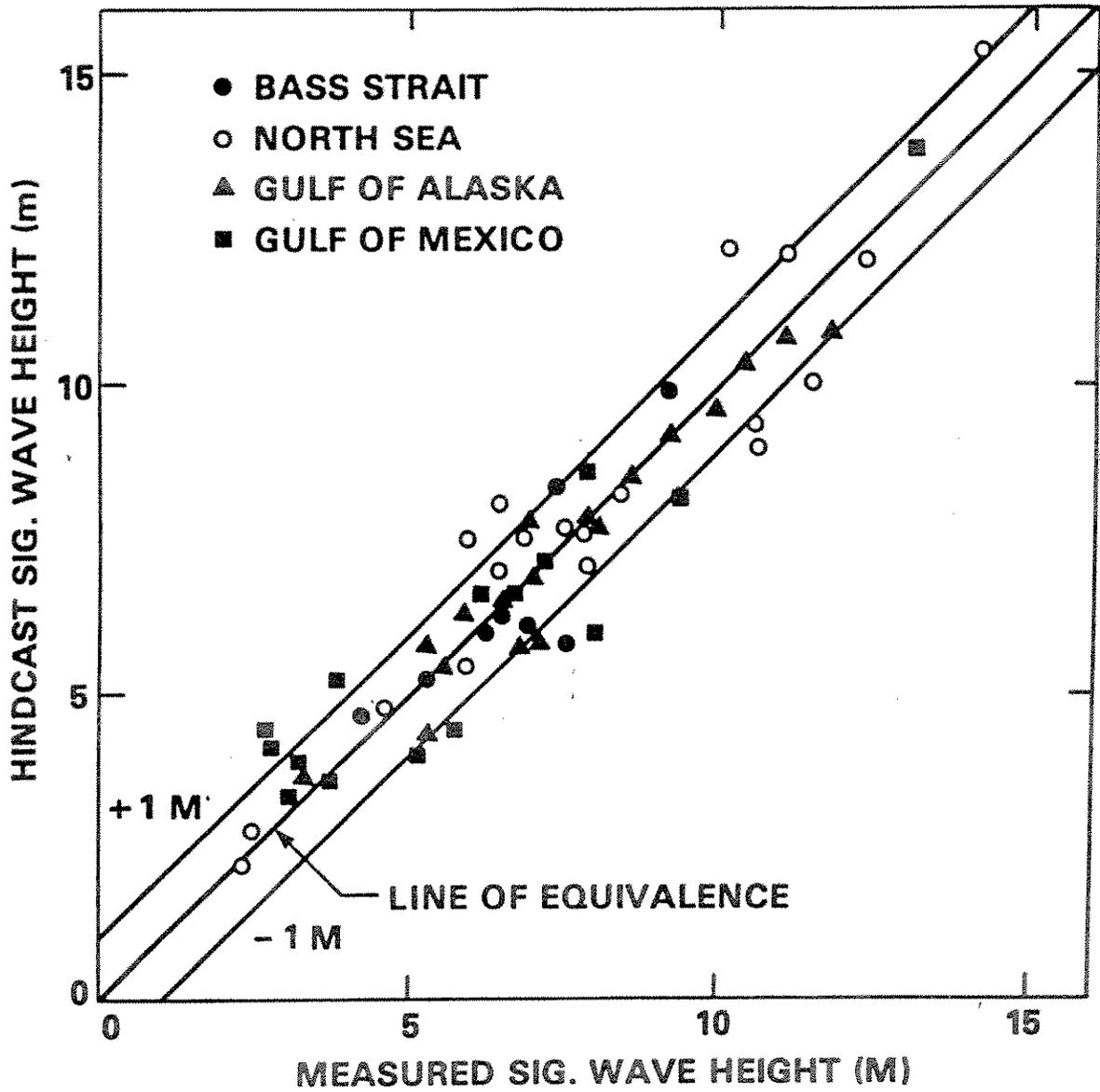
Figure 1 Comparison of hindcast and measured storm-peak significant wave height in four offshore areas.
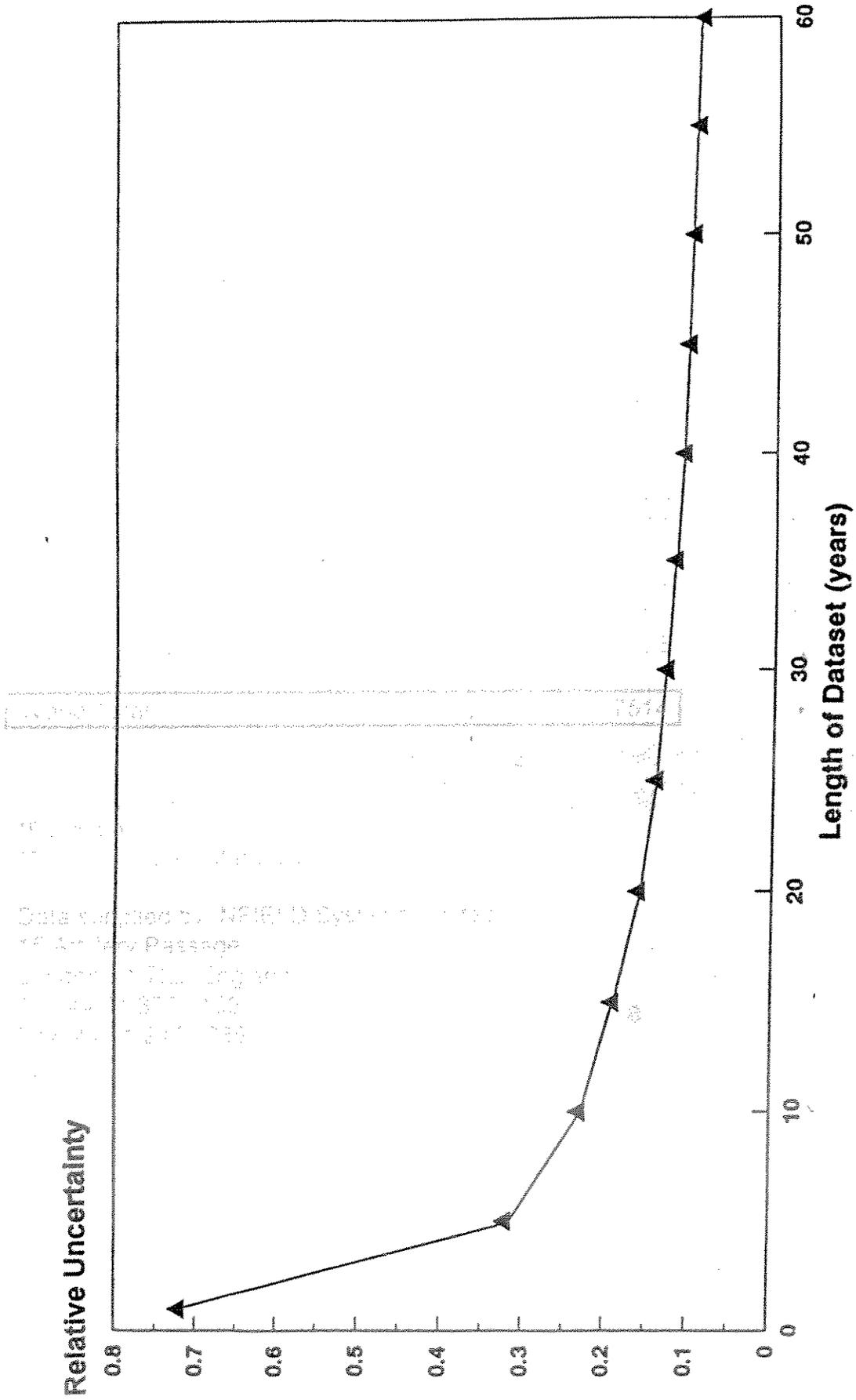
Figure 2  Dependence of the reliability of the 100-year significant wave height estimate on the length of the dataset.
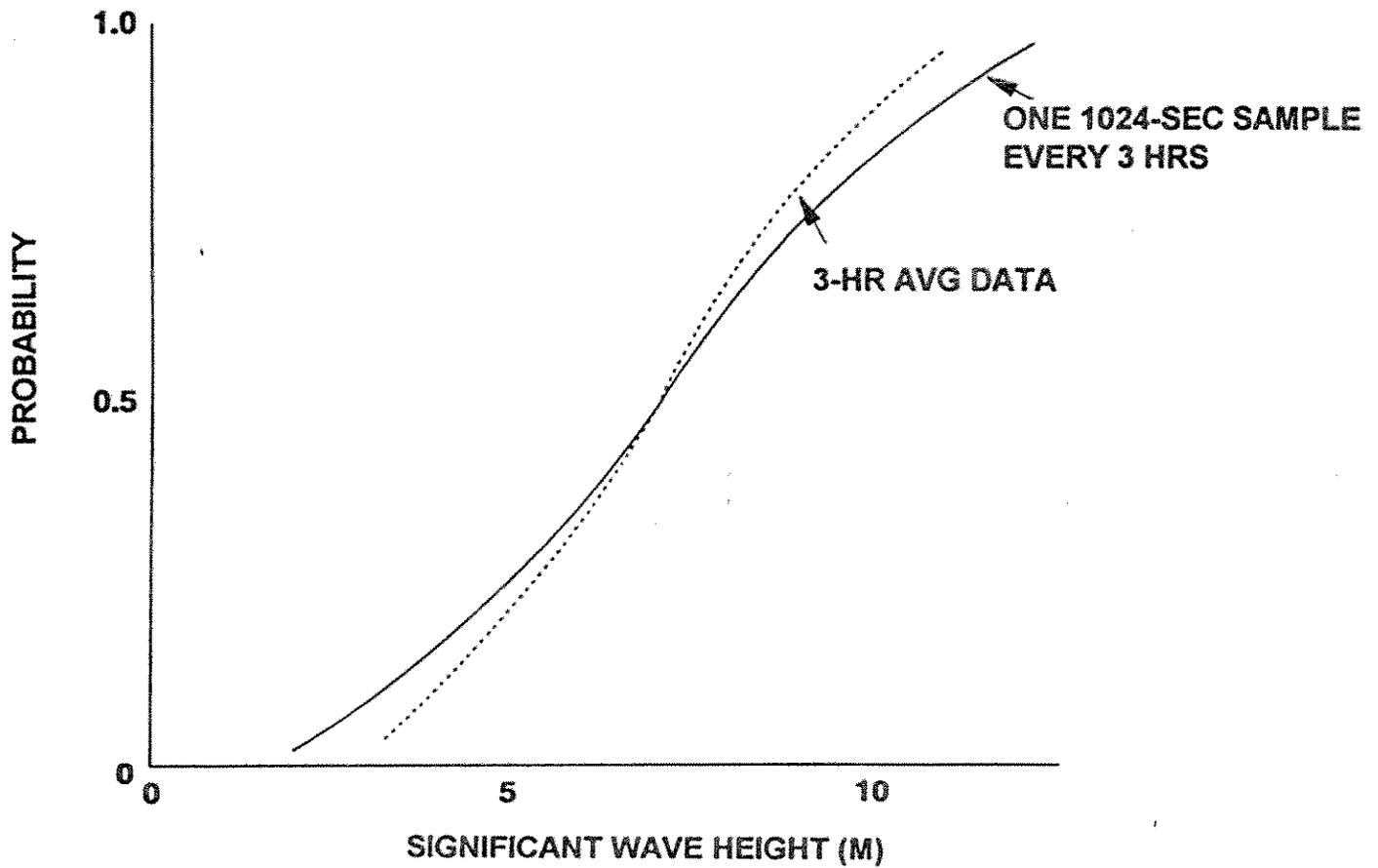
Figure 3  Illustration of the effect of sampling variability on the variance in the distribution of significant wave height.

measurements or hindcasts, the cumulative distribution function of all the data points is used. This distribution gives the fraction of all data points below successively higher thresholds, or, equivalently, it gives the fraction of time that wave heights are below various thresholds. If the duration associated with each data point is H hours, then the Y-year value is that which is exceeded once in Y * 365 * 24 / H data points, which may be read from a curve fitted to the cumulative distribution function and extrapolated beyond the data. Some statisticians criticize this method because of the inherent correlation among data points. For example, the highest few data points in the cumulative distribution function may all come from the same storm. An advantage of this method often quoted by its practitioners is that it makes full use of all the available data, and is less sensitive to uncertainties in the few largest data points. Detractors of the method counter-argue that it gives too little emphasis to the extreme tail of the data in the extrapolation process.

Outside of Europe, the most commonly used method is the "peak over threshold" (POT) method (Jahns and Wheeler 1973; Petrauskas and Aagaard 1971). In this method, only a single data point, the storm-peak significant wave height, is used for each storm in the extrapolation to the Y-year significant wave height. The "storms" that are used are separated by about 18 hours or longer in an effort to ensure that they are statistically independent, and only storms exceeding a prescribed threshold are used. If the average frequency of storms is F storms per year, then for a long return period Y, the Y-year storm is that which is exceeded once in Y * F storms, which may be read from a curve fitted to the cumulative distribution function of storm-peak significant wave heights. Many statisticians favor the POT method because only extreme events are used in the database that is extrapolated to the Y-year extreme event, and the extreme events in the database are statistically independent. Of course, the method is sensitive to errors in the few highest storm-peak significant wave heights used in the extrapolation.

Regardless of whether the "continuous data" or the "peak over threshold" method is used, there is uncertainty in the extrapolation. The underlying probability distribution function just cannot be determined confidently from the limited data. As illustrated in Figure 4 for a typical extrapolation of 50 storm-peak significant wave heights spanning 25 years of history, the 95 % confidence bounds are fairly tight for return periods less than 25 years but expand quickly at higher return periods. In specifying "design" conditions, oceanographers have generally not explicitly accounted for the confidence interval; that is, they have specified the central estimate of the Y-year wave height. This contrasts with foundation engineers, who have traditionally used lower bound estimates of soil strength, which is rather conservative.

*If the central estimate of Y-year wave height is used for design of new structures, can the lower bound estimate of Y-year wave height be used in the requalification of older structures? (Issue #8)*

## Different Storm Types

Different offshore areas have different types of extreme storms. For example, storms in the North Sea are all of the extratropical type, whereas the Gulf of Mexico has both extratropical and tropical storms, with the latter being much more severe but less frequent. As illustrated in Figure 5, a region dominated by extratropical storms would be expected to have a lower rate of change of wave height with return period than a region dominated by tropical storms. Therefore, even if the two regions had identical Y-year wave heights for calculating design wave loads, the region dominated by tropical storms would require a larger factor of safety in its platforms in order to have the same platform reliability as the region dominated by extratropical storms.

*How should the steepness of the wave height versus return period curve affect the wave height used for requalification of older structures, relative to the wave height used to design new structures? (Issue #9)*

In some regions, such as the U.S. East Coast (Ward et al. 1977), the 100-year winter storm and the 100-year hurricane produce comparable wave heights. In the Gulf of Mexico, if hurricanes that originate outside the Gulf are excluded (because platforms are evacuated), the remaining "sudden" hurricanes (for which it may not be possible to evacuate platforms) produce extreme wave heights comparable to those produced by severe winter storms. For regions such as these, where two storm types must be considered, the annual nonexceedence probability for a given wave height in the combined storm population is the product of the annual nonexceedence probabilities for the two different storm types, that is, Pc = Ph * Pw. However, in evaluating a platform's reliability, it must be kept in mind that a wave of a given height will generally be more forceful in a winter storm than in a hurricane, due to less directional spreading of the wave energy. This is illustrated in Figures 6 and 7 for a site in 60 ft water depth in the Gulf of Mexico. While the "sudden" hurricanes dominate the wave height statistics for return periods above 30 years, they do not dominate the force statistics on a drag-dominant structure until the return period exceeds 100 years.

**SIGNIFICANT WAVE HEIGHT (M)**

**RETURN PERIOD (YRS)**

Goodness of fit results
Chi-squared             73.77 %
Kolmogorov-Smirnov      89.28 %
Linear Correlation      .995
Return values for Hs :
  5 Year :    11.16 m
 10 Year :    11.77 m
 20 Year :    12.33 m
 50 Year :    13.02 m
100 Year :    13.52 m
Threshold     8.950 m

Method              :  POT
Direction Sector    :  OMNI
Season              :  All Year
No. of Data Points  :  49
Period              :  1964-1989
Confidence Interval :  95.0 %
Distribution        :  WEIBULL
Sigma               =  1.209
Alfa                =  1.242
X0                  =  8.900

→ Probability of non-exceedance (%)

Figure 4  Sample extrapolation of 49 storm-peak significant wave heights illustrating the expansion of the 95% confidence band for longer return periods.

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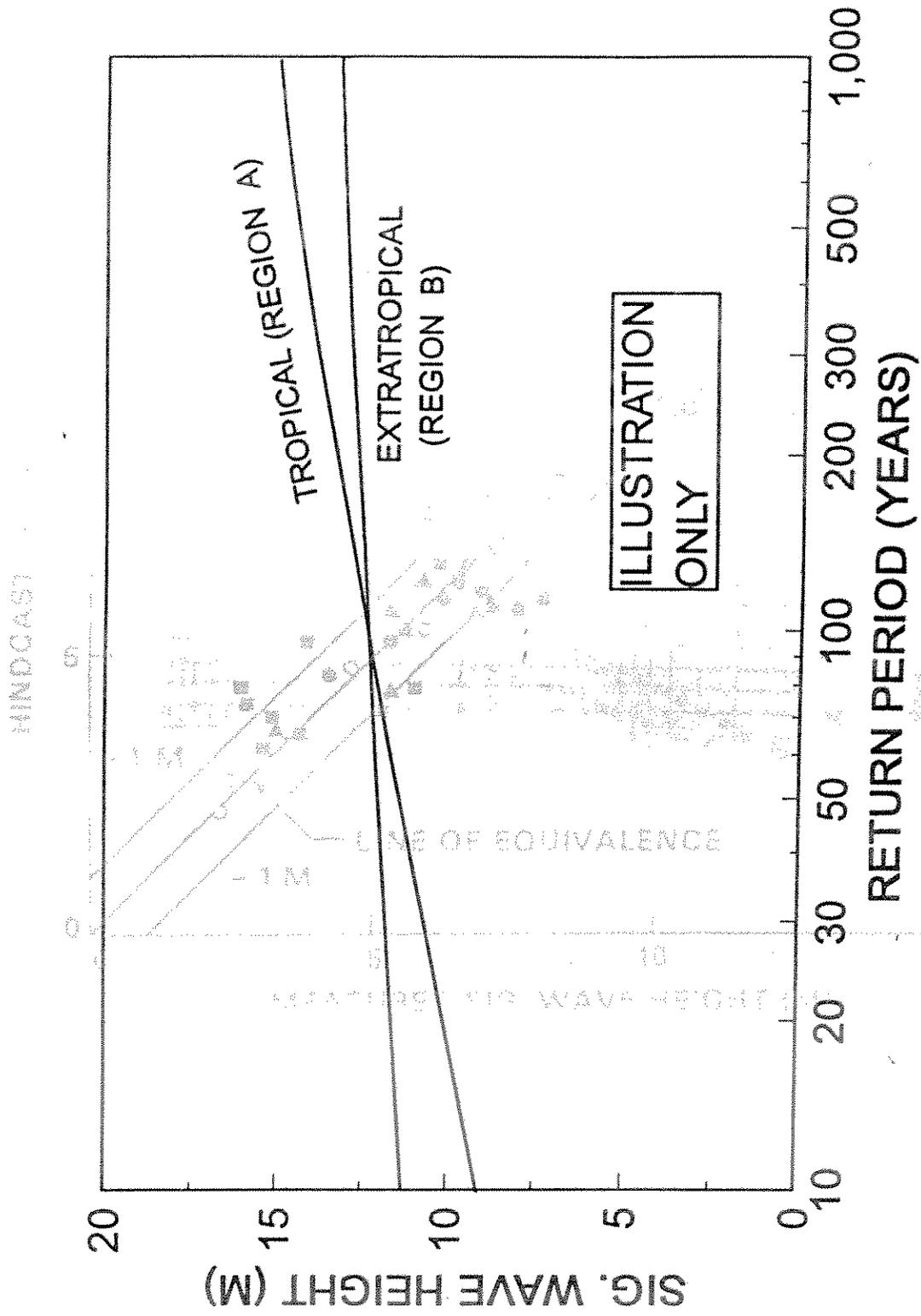
Figure 5  Illustration of the difference in steepness of the wave height versus return period curve between a region dominated by tropical storms and a region dominated by extratropical storms. Both regions in this illustration have the same 100-year wave height.
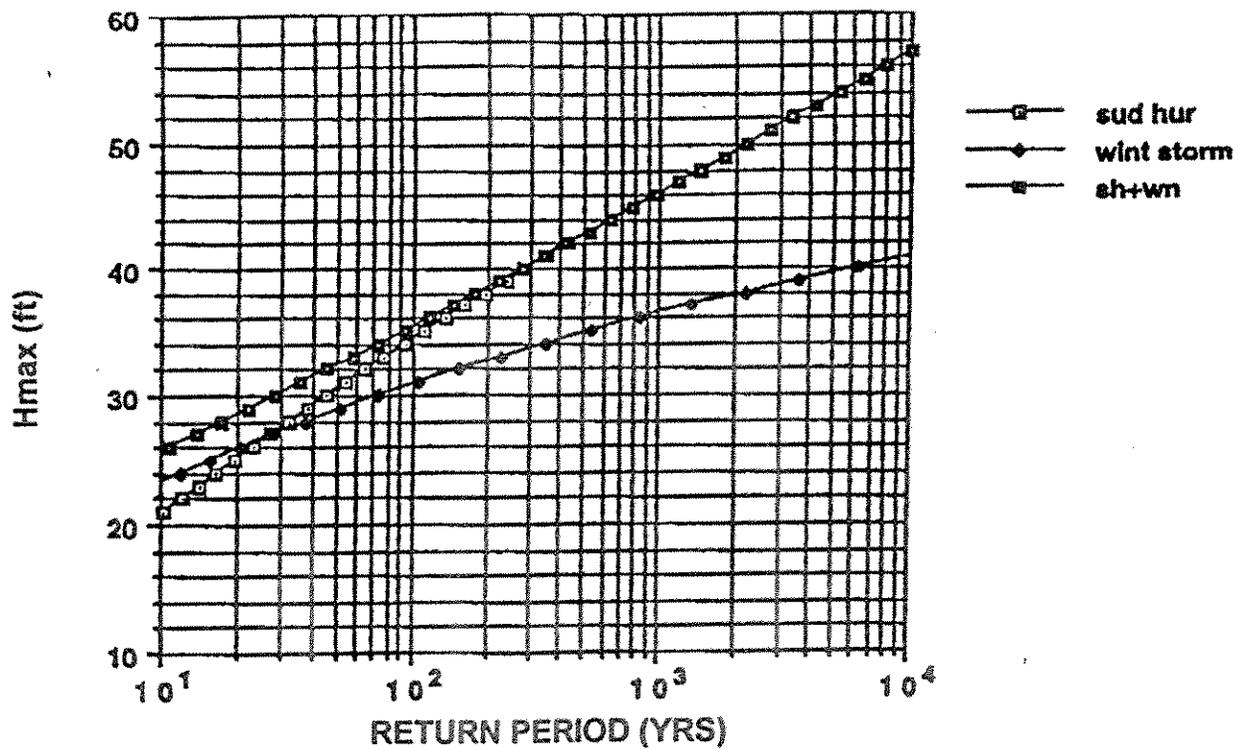
Figure 6 Wave height versus return period for winter storms, sudden hurricanes, and the two combined, for the Gulf of Mexico, in 60 feet water depth. Note that winter storms produce higher waves for return periods less than about 30 years, while sudden hurricanes produce higher waves for longer return periods.
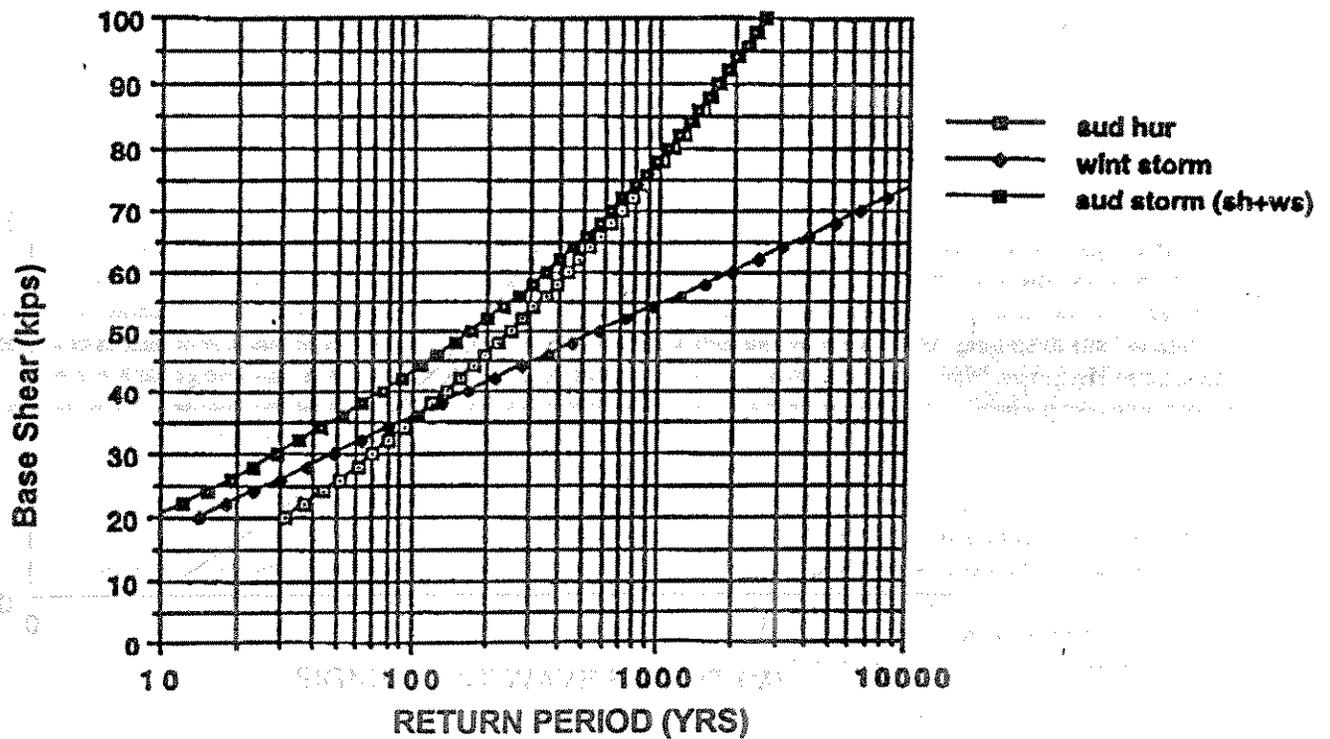
Figure 7  Base shear on a pile versus return period for winter storms, sudden hurricanes, and the two combined, for the Gulf of Mexico, in 60 feet water depth. Note that winter storms produce higher forces for return periods less than 100 years, while sudden hurricanes produce higher forces for longer return periods. Winter storm waves are more forceful than sudden hurricane waves of the same height because they have less directional spreading of the kinematics.

*How should different storm types be considered in the requalification of older platforms? Should the requalification storm be selected on the basis of its Y-year wave height or its Y-year forcefulness? (Issue #10)*

*How can wave spreading be adequately taken into account in calculation of wave forces? Is the procedure in the 20th edition of RP2A (see below) adequate? (Issue #11)*

### Selection of Return Period

For a site with only a single dominant storm type, several possibilities for selecting the return period of the design storm event for a new platform as a function of the planned platform life are illustrated in Figure 8 (after Lloyd 1985). Some have suggested that the uniform annual storm risk curve is appropriate for platforms that may be manned during the design storm. In Norway, for example, all platforms are designed to withstand a wave with an annual exceedence probability of 1/100. The logic is that the risk taken by platform personnel every year should be independent of the platform's design life. In any given year, for example, why should the personnel on Platform A, with a 10-year design life, be exposed to a greater risk than the personnel on Platform B, with a 30-year design life? For platforms that can be evacuated when severe storms approach, and for which the remaining concerns are risk of pollution or loss of revenue, perhaps the middle curve, which gives a uniform probability of failure over the platform life, is appropriate.

*Should the return period of the metocean conditions used in platform requalification depend on the duration of the planned life extension (see Issue #2)? If so, how should the return period differ between the consequences of personnel risk, pollution risk, and revenue loss? (Issue #12)*

*Should the storm return period for requalification differ from the storm return period for new design? If so, on what basis should it be selected? (Issue #13)*

### Site-Maxima versus Basin-Maxima

In a region such as the Gulf of Mexico, the Y-year wave height at a particular site is strongly affected by the proximity of the site to the tracks of extreme historical hurricanes. Since it would not be reasonable to expect future hurricanes to have exactly the same intensities and follow exactly the same tracks as historical hurricanes, a certain amount of "site averaging" of Y-year wave heights from site to site has traditionally been used (Petrauskas et al. 1993; Haring and Heideman 1980; and Ward et al. 1979). The resulting estimate of the Y-year wave height at a site still incorporates the probability that a hurricane will not pass directly over the site, but in an average sense. Use of this Y-year site-average wave height for design force calculations is logical for each platform taken individually. However, it should be kept in mind that in a basin with many broadly distributed platforms such as the Gulf of Mexico, the risk of losing one platform is roughly proportional to the number of platforms present, all else being equal. This fact has led some to suggest that perhaps platforms should be designed to the basin-wide Y-year wave height, to provide an extra margin of safety. The basin-wide Y-year wave height is obtained from the distribution of the storm-peak significant wave heights for all hurricanes, regardless of where in the Gulf the peaks occurred. This is equivalent to assuming that the platform moves around so that, no matter where the hurricane goes, the platform is always located where the most extreme waves occur. Figure 9 shows that the basin-wide Y-year wave height is considerably higher than the site-average Y-year wave height.

*Should the site-average or the basin-wide Y-year wave height be used in platform requalification? Is a two-level check appropriate, in which the site-average value is used for nominal loads and the basin-wide value is used for survival loads? (Issue #14)*

### Joint Probability

At a site, extreme waves, extreme currents, and extreme winds do not necessarily occur at the same time or in the same direction. The metocean criteria in API RP2A permit the designer of a new platform to take full advantage of this fact (Petrauskas et al. 1971). The guideline criteria in the United Kingdom recognize this fact, but do not allow the designer to take advantage of it. As compensation, the United Kingdom allows the designer to use an unrealistically low drag coefficient. The Norwegian Petroleum Directorate (NPD) guidelines allow a small reduction in metocean criteria due to joint probability considerations. The NPD allows use of 10-year currents with 100-year waves, but still requires the two to be applied simultaneously and collinearly. As shown by the NOCDAP project (Heideman et al. 1992), collinear currents associated with extreme wave conditions on parts of the Norwegian shelf are less than a half
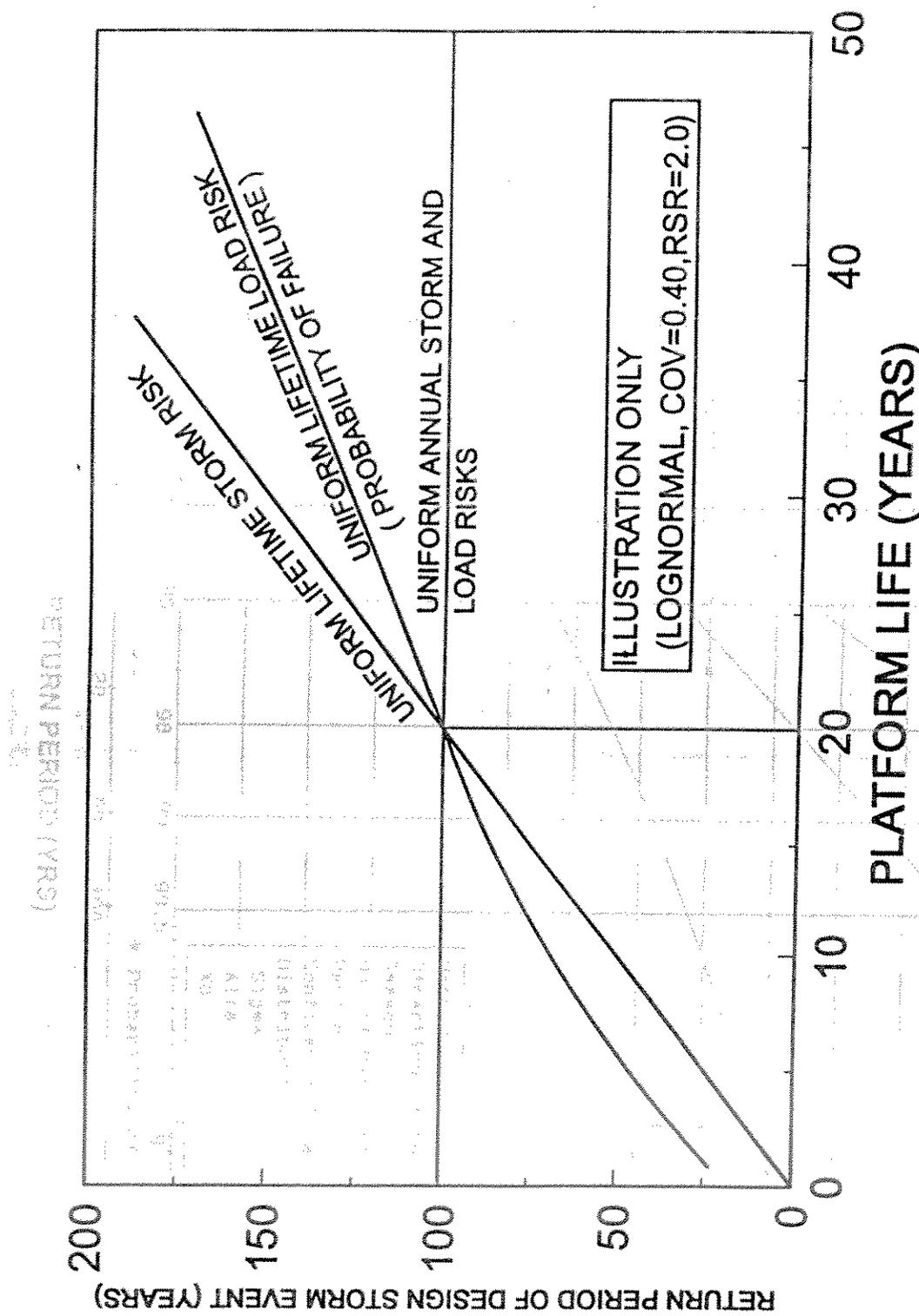
Figure 8 Dependence of the design storm return period on the design life of the platform. Curves are shown for uniform lifetime storm risk, uniform lifetime load risk, and uniform annual storm and load risk.
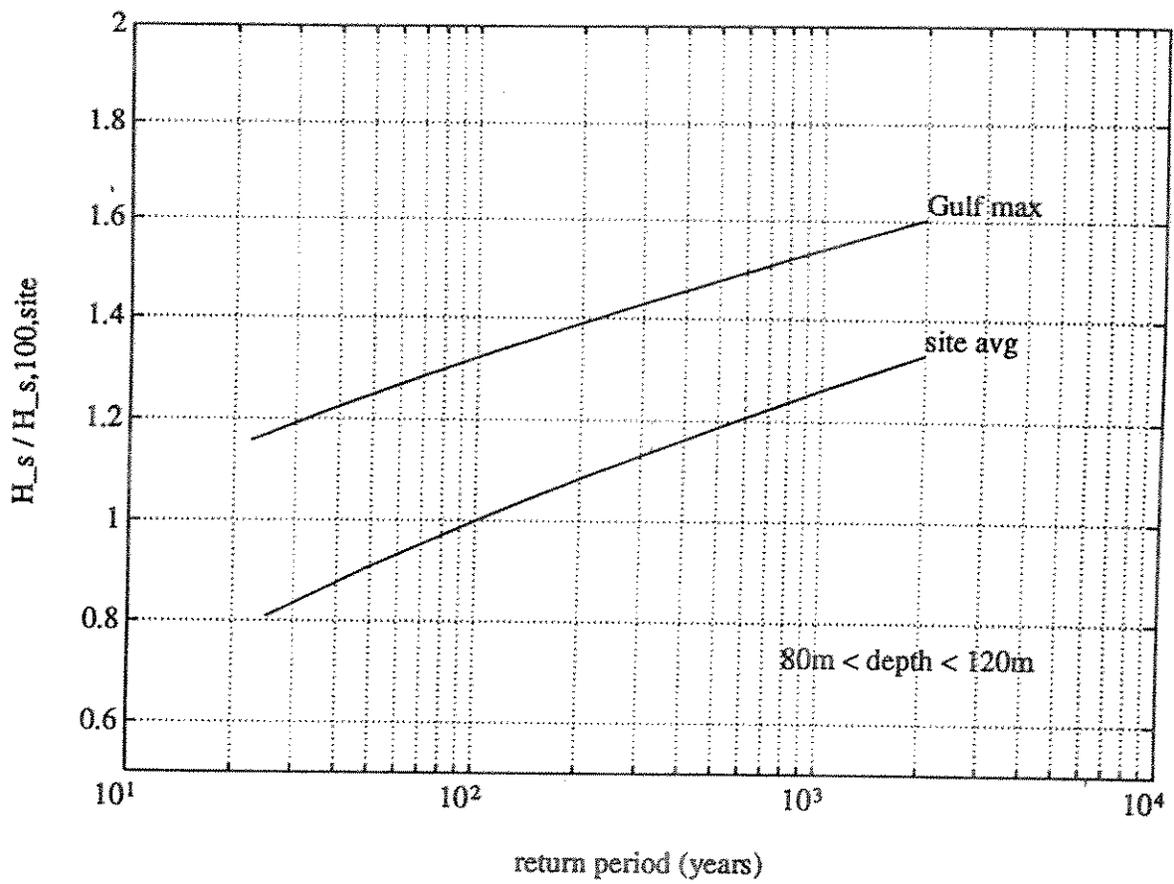
**Figure 9** Comparison of basin-wide extreme waves with site-average extreme waves for the Gulf of Mexico. The latter accounts for the distance between the site and the storm tracks.

knot, which is much less than the NPD current. Like the United Kingdom, the NPD allows the use of low drag coefficients partly as compensation for the conservatism in the guideline metocean criteria.

*In platform requalification, should the metocean criteria take advantage of the joint probability of occurrence in time and direction of extreme winds, waves, and currents? (Issue #15)*

*Should it be required that realistic force calculation procedures (specifically, realistic force coefficients) be used with joint-probability-based metocean criteria in platform requalification? (Issue #16)*

One way to take advantage of wind, wave, and current joint probability, when simultaneous time series are available, is to make use of a platform wave-induced response model (Tromans et al. 1992; Heideman et al. 1992). For steel jackets and jack-ups, with negligible dynamics, the platform response model may be a simple parametric equation of the form F = C1 * ( H + C2 * Vil ) ** C3, where F is base shear or overturning moment, H is wave height, Vil is the in-line component of current, and C1, C2, and C3 are empirical constants. For other structures, the response model may be somewhat more complicated. In any event, it is generally possible to construct a simple response model that can be efficiently evaluated at every time for which winds, waves, and currents are available, and then to develop response statistics. Once the Y-year response has been determined, one can back-calculate different combinations of winds, waves, and currents that might lead to that response, and perform detailed structural evaluations for them.

# Wave Forces

Once the metocean conditions for platform requalification have been selected, the forces induced on the platform must be calculated and compared with the platform capacity. In this section, procedures for calculating forces due to waves and currents will be discussed. Forces due to winds are of less importance for most platforms that will be submitted for requalification in the near future, so wind forces will not be discussed here. Similarly, dynamic response will be ignored, as most fixed platforms that will be submitted for requalification in the near future are in water depths less than 200 ft and therefore have negligible dynamics. Discussion will focus on the new static wave force procedure in the 20th edition of RP2A. Differences between United Kingdom and Norwegian guideline procedures and RP2A will be identified, and alternative methods will be mentioned where appropriate.

## API RP2A, 20th Edition

With this edition of RP2A, major changes were made to the procedure for calculating static wave forces (Petrauskas et al. 1993). The new procedure attempts to account for the principal characteristics of oscillating, turbulent, separated flow about smooth and rough circular cylinders in lattice-type structures, as revealed in laboratory and field experiments over the last decade or so. The procedure is intended to be as realistic as possible, without being so complicated as to preclude its use in routine design calculations. The procedure begins with the specification of the design wave height and direction and associated wave period, wave spreading, current profile (speed and direction at different elevations), storm surge and tide, and marine growth profile (thickness and roughness height at different elevations). Then, as illustrated in Figure 10, the procedure takes the following steps in sequence.

- An apparent wave period is determined, accounting for the Doppler effect of the current on the wave. This is the period seen by an observer moving with the steady current. The change in wave period is generally less than 10% and the consequent change in global wave force is generally only a few percent.
- The two-dimensional wave kinematics are determined from an appropriate regular wave theory, such as stream function, for the specified wave height, storm water depth, and apparent period. Other regular wave theories besides stream function can be used, provided they can be shown to produce comparable results when applied consistently.
- The horizontal components of wave-induced particle velocities and accelerations are reduced by the wave kinematics factor, which accounts primarily for wave directional spreading. The reduction can be substantial in hurricanes (8-12%) but is generally much less in winter storms (0-5%). There can be a further reduction in kinematics to account for the "irregularity" of actual wave shapes, if supporting documentation is provided.
- The effective local current profile is determined by multiplying the specified current profile by the current blockage factor. The blockage factor depends on how "dense" the structure is. Values range from 0.7 to 0.9 for typical lattice-type structures but are much lower for structures as dense as the Lena guyed tower.
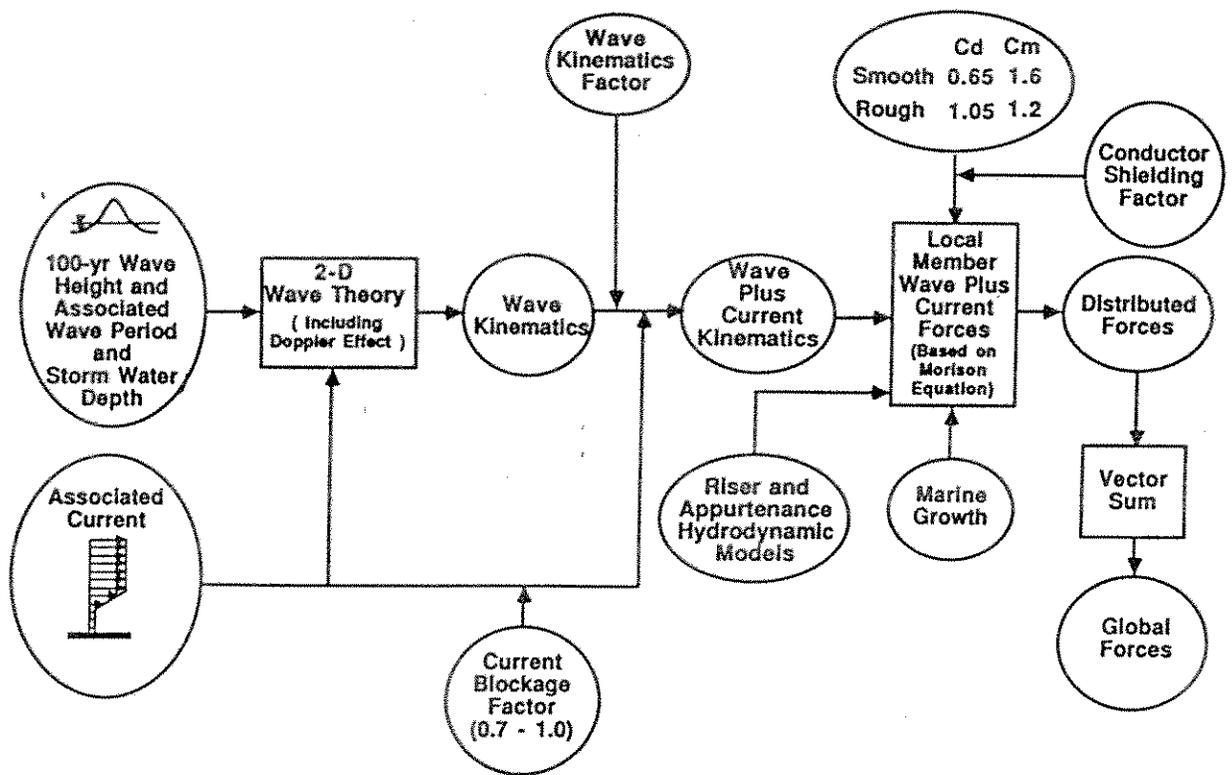
Figure 10 API procedure for calculating wave-current force for static analysis. Default drag and inertia coefficients for reference Keulegan-Carpenter numbers above 30 are shown. For lower Keulegan-Carpenter numbers the coefficients are greater.

- The local current profile is stretched to the wave surface and then combined with the vectorial wave kinematics to determine locally incident fluid velocities for use in Morison's equation. Stretching the current profile can produce considerably lower forces than vertical or constant-shear extrapolation if the profile is highly sheared near mean water level. However, for typical "slab" profiles, the differences are negligible.
- Member dimensions are increased to account for marine growth.
- Drag and inertia force coefficients are determined as functions of wave and current parameters, depth below mean water level, and member shape, roughness (marine growth), size, and orientation. Default values of drag and inertia coefficients are specified for a reference Keulegan-Carpenter number of Umo * Tapp / D > 30, evaluated at storm mean water level, where Umo is the maximum orbital velocity, Tapp is the apparent wave period, and D is average leg diameter, including marine growth. These values are Cd = 1.05, Cm = 1.2 for members in the marine growth zone and Cd = 0.65, Cm = 1.6 elsewhere. For Umo * Tapp / D < 30, guidance is provided on the dependence of the coefficients on Keulegan-Carpenter number.
- Wave force coefficients for the conductor array are reduced by the conductor shielding factor, which depends on conductor spacing. The conductor shielding factor varies from 0.5 at two-diameter (center-to-center) spacing to 1.0 at four-diameter spacing. For typical conductor spacing, the factor is close to 1.0.
- Hydrodynamic models (simple structures with equivalent hydrodynamic characteristics) are developed for risers and appurtenances.
- Local wave / current forces are calculated for all platform members, conductors, risers, and appurtenances using Morison's equation. Morison's equation uses only the components of fluid velocity and acceleration normal to the cylinder axis. It neglects convective acceleration, axial Froude-Krylov forces, lift forces, and slam forces.
- The global force is computed as the vector sum of all the local forces.

Forces computed with the new API procedure, accounting for the measured characteristics of the actual waves and currents, have been compared with forces measured on the Ocean Test Structure in the Gulf of Mexico and the Magnus and Tern platforms in the North Sea (Heideman and Weaver 1992). The ratio of measured-to-predicted force, as a function of reference Keulegan-Carpenter number, is shown in Figure 11 for variable Cd, Cm; in Figure 12 for constant Cd, Cm. The ratios for individual waves fall between 0.5 and 2.0. The distributions of the force ratios in Figures 11 and 12 are plotted in Figures 13 and 14. For variable coefficients, the empirical distribution is well approximated by a normal distribution of the square root of the force ratio. The parameters of the distribution are u = 1.001 and s = 0.146. Thus, the mean value of the force ratio is u ** 2 = 1.002, which indicates negligible bias. Also for constant coefficients, the empirical distribution is well approximated by a normal distribution of the square root of the force ratio. The parameters of this distribution are u = 1.057 and s = 0.152. Thus the mean value of the force ratio is u ** 2 = 1.117, which indicates about 12% under prediction, on average, using the API default coefficients. This bias was to be expected, since many of the waves had a reference Keulegan-Carpenter number less than 30, for which the constant coefficients are not recommended by API. The significant scatter in the wave force predictions, as quantified in Figures 13 and 14, can be accommodated in procedures that calculate platform reliability.

It is believed that the scatter in wave force predictions can be reduced significantly only through abandonment of regular wave theory in favor of a wave theory that can better model the irregularity and three-dimensionality of real ocean waves. In this regard, the performance of various regular and irregular wave kinematics theories are compared by Gudmestad (1993), and the effects of wave steepness on wave forces are discussed by Gudmestad and Haver (1993). Recent developments by Zhang et al. (1993) show promise for improved kinematics predictions, relative to those from traditional "stretching" methods.

*Should the API RP2A, 20th edition, procedure for calculating static wave forces on new structures also be used in reevaluating older structures? (Issue #17)*

*In view of the large scatter in wave force predictions, is there a need for more wave kinematics or wave force research? If so, which areas should be emphasized? (Issue #18)*

*Can experience with typical 4 - 8 leg platforms be used as an indicator of the reliability of optimized tripod structures, for which there has been little exposure to extreme storms? (Issue #19)*

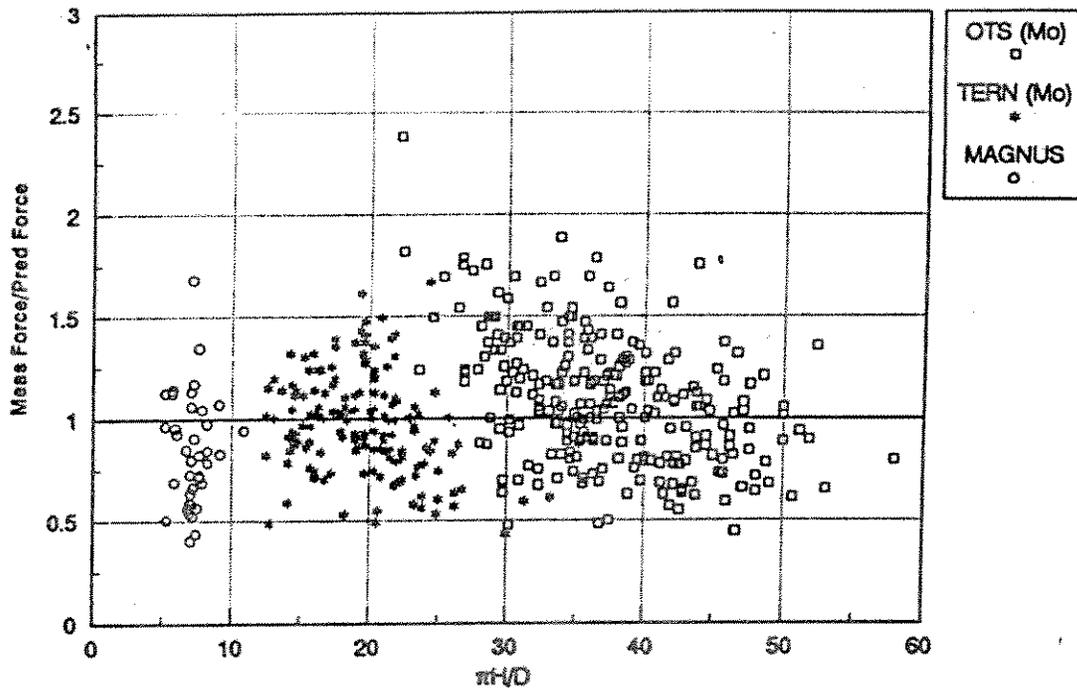Figure 11  Dependence of the measured-to-predicted force ratio on the reference Keulegan-Carpenter number; API procedure with variable Cd, Cm.
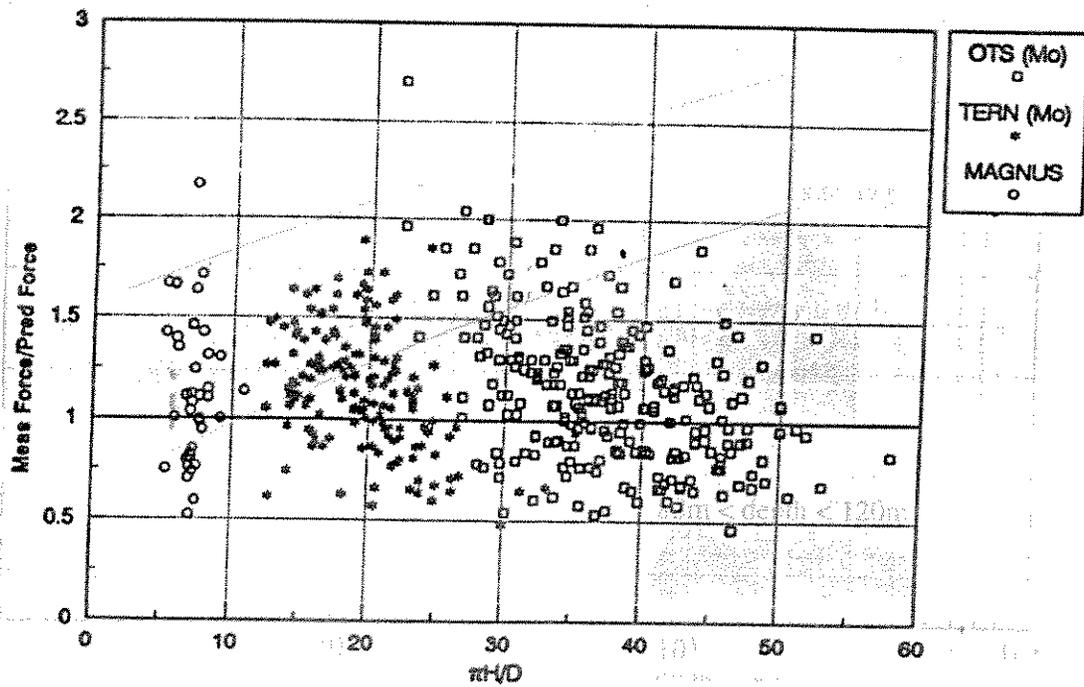
Figure 12  Dependence of the measured-to-predicted force ratio on the reference Keulegan-
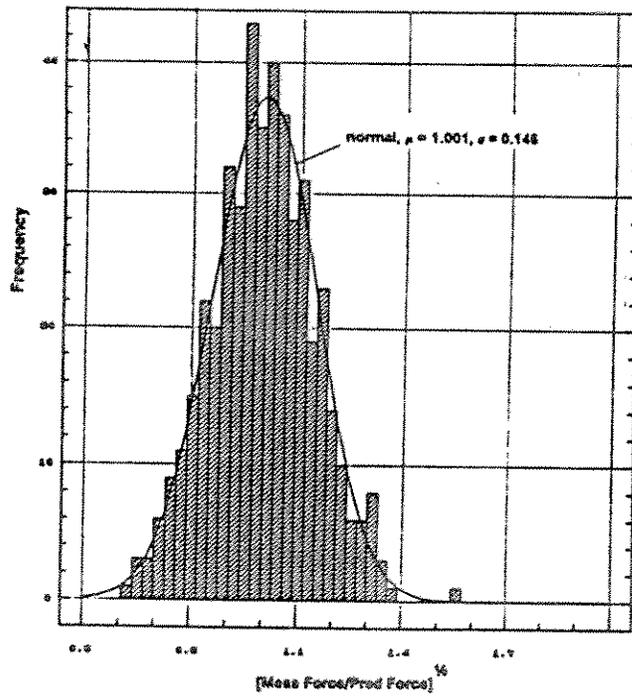Carpenter number; API procedure with constant, default Cd, Cm.

Figure 13  Distribution of the square root of the force ratios; variable Cd, Cm.
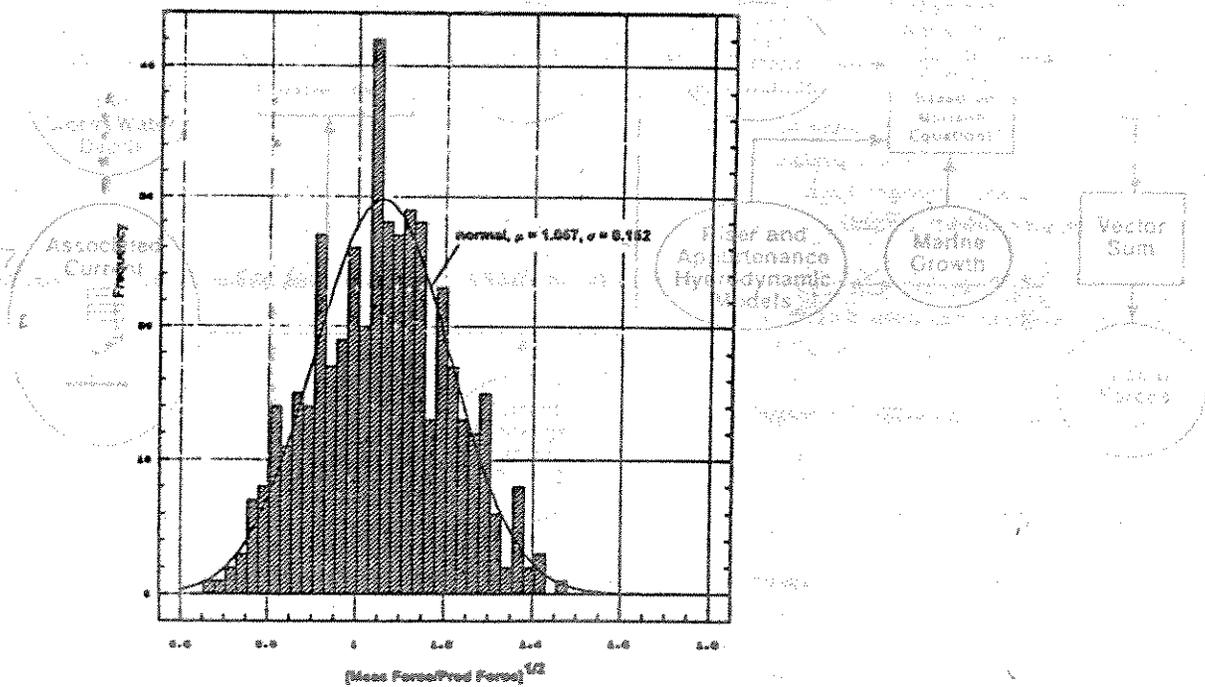
Figure 14  Distribution of the square root of the force ratios; constant Cd, Cm.

### United Kingdom and NPD Guidelines

The United Kingdom guidelines provide a good summary of wave force technology in a commentary, but in the end opt for using artificially low drag coefficients of Cd = 0.7 in the marine growth zone and Cd = 0.6 elsewhere. This is done to compensate for conservatism in other parts of the guidelines. Specifically, the guidelines specify that an extreme current be combined with the extreme wave, with no accommodation of wave/current joint probability. Furthermore, there is no provision for load reductions due to current blockage, conductor shielding, or wave spreading. The NPD guidelines are similar to the United Kingdom guidelines. The low drag coefficients may, however, lead to a need for an extensive marine growth removal program or use of antifouling cladding.

*If the metocean criteria for platform requalification are reduced below those in the guidelines (by the consideration of wave / current joint probability, for example), should a more realistic force calculation procedure than in the guidelines be used? (Issue #20)*

*Should there be a requirement for cleaning marine growth from certain platforms? (Issue #21)*

Table 2 shows how the design wave force level on a large northern North Sea space-frame structure changes as metocean criteria and force coefficients are changed. In this case, the increase in drag coefficient to the more realistic values espoused in the 20th edition of API RP2A is more than offset by the less conservative wave height and current criteria.

### Deck Forces

Historically, a platform whose deck was not high enough to provide clearance above extreme wave crests was considered to be in great peril. However, experience in the Gulf of Mexico has shown that many platforms have survived a few feet of green water into the deck with only minor damage to equipment. Furthermore, laboratory experiments indicate that five feet of green water into the cellar deck gives only about 50% increase in wave load in a typical case; this is within the safety margin of many platforms. A procedure to calculate wave force on inundated decks, as a function of the wave parameters and wave direction, has been developed in API working groups. Comparison of predicted and measured deck forces on model structures shows only moderately more scatter than Figures 11 and 12 show for platform forces.

*Should older platforms submitted for requalification be required to have their deck above the Y-year crest elevation, or will it be sufficient to show that their calculated wave load on the deck and platform combined is below the calculated capacity of the structure? Should it be required that any equipment on the deck whose loss could result in pollution be above the Y-year crest height? In such checks, is 100 years the appropriate value of Y? If not, what is the appropriate return period for crests? (Issue #22)*

## Concluding Remarks

In this paper, the focus has been on environmental forces to be used in platform requalification, and how they might differ from those used in designing new platforms.

Does this line of questioning imply that there is implicit conservatism in the environmental forces used to design new platforms? It is worth recalling that those who specify the metocean conditions *for a prescribed return period* have traditionally given their best estimate (as opposed to an upper bound estimate) of individual metocean parameters, and recently have even begun to account for the joint occurrence of the parameters. The Gulf of Mexico metocean criteria in the 20th edition of API RP2A are a prime example of this. Also, it has been demonstrated that, if the metocean conditions are well known, the new API procedure provides an unbiased prediction of the wave force. Therefore, there is no basis for believing that there is implicit conservatism in the Gulf of Mexico wave forces caused by metocean conditions with a prescribed return period, as specified in the 20th edition of API RP2A. However, for other areas, particularly outside the US, there may be some implicit conservatism that can be exorcised.

The acceptance of less severe metocean conditions in requalification implies a willingness to accept a greater risk of failure for existing platforms than for new platforms. For purely economic reasons, older platforms cannot be required to have the same margin of safety above the current estimate of design load as new platforms. To require them to do so would mean in many cases that either the platform would have to be strengthened or replaced with a new

Table 2  Relative Wave Force on Tern.

| | H (m) | T (sec) | V* (m/s) | Cd** | Cm** | Shear | Moment |
|---|---|---|---|---|---|---|---|
| 1990 UK | 30 | 16.5 | 0.8 / 0.8 | 0.60 / 0.70 | 1.7 / 1.7 | 1.00 | 1.00 |
| 1991 NPD | 29 | 15.5 | 1.0 / 0.2 | 0.70 / 0.70 | 2.0 / 2.0 | 0.82 | 0.89 |
| 1993 Proposed | 26 | 13.8 | 0.3 / 0.3 | API variable | API variable | 0.65 | 0.70 |
| 1993 Proposed | 26 | 13.8 | 0.3 / 0.3 | API constant (0.65 / 1.05) | API constant (1.6 / 1.2) | 0.61 | 0.67 |

* Surface / mid-depth speeds; reduced by 0.8 factor in 1993 proposed procedures

** Values above and below MWL
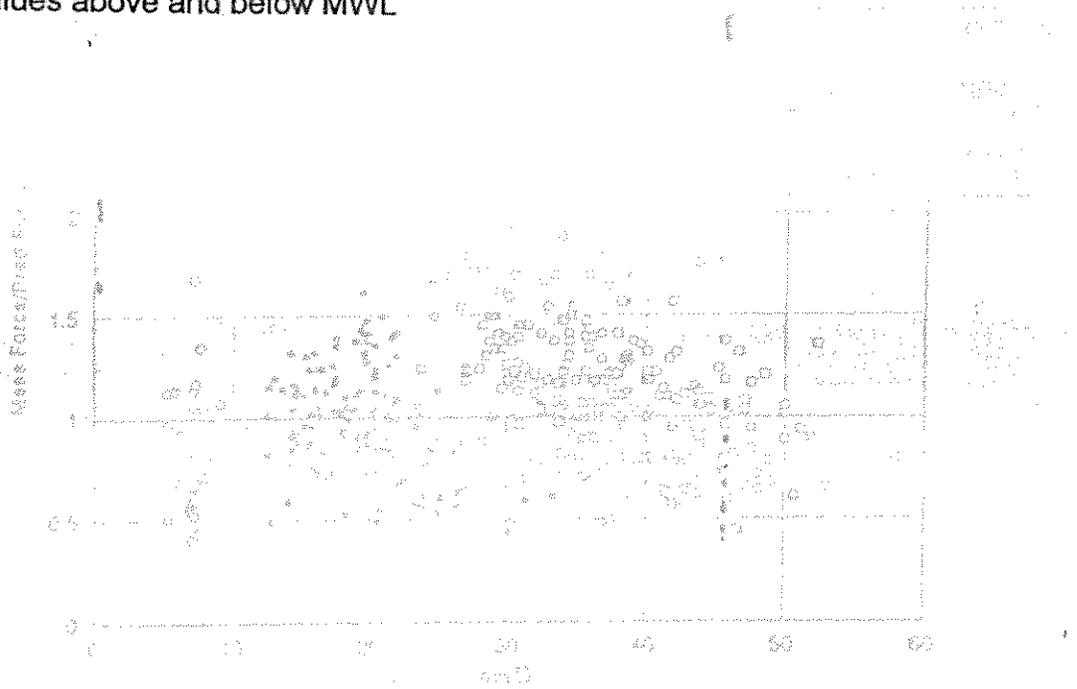
platform, at very high cost, or the reservoir would have to be abandoned, thus leaving scarce natural resources unused. Some supporting justifications are that: the older platforms may in some cases be requalified for a shorter life than considered for new platforms, new platform design criteria do not take advantage of the fact that in some areas platforms are evacuated when extreme storms approach, the consequences of failure may be less for old platforms, (4) the foundation capacity is known better for an in-place structure than for new design, and the demonstrated performance of older structures may have validated their strength.

*If the API and various government authorities allow older platforms that have the same extended lifetime as the design lifetime of a new platform, and that have the same consequences of failure as a new platform, to have a lower ultimate capacity, then why shouldn't they allow the new platform to also have a lower capacity? (Issue #23)*

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# STRUCTURAL ELEMENTS, SYSTEMS AND ANALYSES

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## Introduction

This working group has the objective of considering the current state-of-the-art of assessment and requalification of offshore production platforms and in particular the structural "capacity" or "resistance" and remaining useful life of elements (members and joints) and structural systems. The working group will also assess analytical methods available to assist in calculating such capacities or life. The scope covers intact and aging or damaged structures and methods of strengthening or repair. This working group should consider its role in the context of the Workshop as a whole. Other working groups will consider:

- Inspection and condition surveys - to provide information on the state of the structure
- Environmental conditions and forces - to provide information on past and future natural forces to which the structure may be subjected
- Foundation elements, systems and analyses
- Operational considerations - to provide information on operational and accidental hazards and loads to which the structure may have been or may be subjected
- Policy considerations and consequences - to consider the framework and background within which offshore structural assessments and requalifications may be carried out.

This paper addresses and outlines for review some of the specific issues encountered in assessing the structural adequacy of existing structures. The intent is to highlight topics for working group discussion starting points with the goal of reaching agreement on state-of-the-art and determining appropriate research needs.

In several parts of the world, industry bodies or regulatory authorities are developing guidelines for assessment and requalification. API has developed a draft assessment process described in a flowchart. Separate criteria are set for

- manned platforms,
- platforms that will be evacuated in forecastable design events,
- not normally manned platforms,
- significant environmental impact platforms, and
- insignificant environmental impact platforms.

The assessment process considers both life and environmental safety. An important aspect of the process is the implied safety level or event return period required for each level of assessment. In the United Kingdom the Health and Safety Executive has not prescribed specific safety levels but requires the operator to prepare and maintain a Safety Case for each platform. In some cases, event return periods of up to 10,000 years have been considered. In Norway, safety responsibility is left with the operator, although the Norwegian Petroleum Directorate has final approval authority to allow production operations to continue.

*Is the API approach for requalification of platforms in USA waters appropriate for other geographic areas? What significant change would need to be made? (Issue #1)* (API *under final review*; Iwan et al. 1992; Sharp et al. 1992)

## Approach to Assessment and Requalification

Our main concern is to ensure that a structure will have sufficient capacity to resist the applied loads ("actions") during its remaining operational life in order to meet required criteria for life-safety, operational serviceability and environmental protection. These criteria may be expressed in absolute terms against which the structural capacity may

be determined (deterministic approach) or in probabilistic terms based on a desired probability of the structure resisting the probable actions to which it may be subjected during its remaining life.

***Are both deterministic and probability based approaches suitable for requalification? Is there adequate technology to apply both approaches? What qualifications, if any, should be placed on each approach in so far as interpreting results is concerned? (Issue #2)*** (Banon et al. *to be published*)

The starting point for a structural assessment is data collection, i.e.,:
- Loading
- Structural properties (geometry, material properties)
- Design, construction and in-service history
- Structural or functional modifications
- Structural design/analysis documentation and results
- Inspection data (marine growth, dents, corrosion, anode condition, scour, flooded members, wall thickness, crack length and depth, crack growth rate, missing members, appurtenance positions, etc.).

Historically offshore structures have not been designed, constructed and installed with future assessment in mind and therefore much of the above data is difficult to locate.

***Are there suitable guidelines and standards to assure adequate information is archived during design, construction and installation? If not, what guidelines and standards are needed. (Issue #3)*** (see Issue #19)

Once the best available data are collected, the next step might be to carry out an elastic global analysis of the structure assuming rigid joints. Member and joint utilizations would be calculated to the latest code requirements. Design codes, safety factor levels, and capacity formulations depend on the type and consequences of component failure, e.g., joint capacity equations, section classification for moment capacity (compact, semi-compact, etc.), beam column capacity, etc.

***Should assessment practices adopt the same relative safety factors among the different components as with new design or should all component reserves be the same? (Issue #4)*** (Stewart et al. 1988)

Inspections may reveal some damage or deterioration, the effect of which on stiffness may affect the results from a global analysis. Approximations may be modeled, for example, by member removal or reduced section properties. Similarly, in cases where damage has not occurred but where component utilization exceeds code values (e.g., due to increased loading or code changes), components may be removed or given reduced stiffness.

***When approximating damage in a global analysis is it possible that analytically approximating components stiffness or removing a component from the system could lead to unconservative results? If so, how serious can the error be and what steps can be taken to avoid such errors or limit their consequences? (Issue #5)*** (Jacobs and Fyfe 1992; Loh 1993; Moan and Taby 1987; Duan et al. 1993)

If, at this stage, component "code failures" are found, it may be appropriate to examine
- the basis of the capacity formulation,
- the loading condition producing the failure or
- the basic material properties.

In each case, there is usually better knowledge of the important parameters at the time of an assessment than during design. For example, during the design process assumptions on dead and live load levels are often based on "blanket" or uniform floor loads. At assessment, loads of all types can usually be estimated much more accurately. (Loading is outside the scope of this working group, but consideration of the accuracy may play an important role in the assessment.) Furthermore, code capacity formulations are known to be conservative in a number of areas (e.g., overlapping K joints, multiplanar joints with sympathetic loading), and perhaps unconservative in others (e.g., tensile capacity in the presence of "acceptable" defects, on multiplanar joints with unsympathetic loading). In assessment, advantage of such knowledge can be taken into account that will result in a perceived higher level of safety, though

stripping out conservatisms does not change the existing level of safety. One danger is that stripping out conservatisms in one area (e.g., structure strength) may expose otherwise masked unconservatisms in another area (e.g., foundations). Structures that respond dynamically to environmental loading can be instrumented to validate the predicted design response. In many cases, dynamic response has been found to be much lower than predicted. Such measurements could justify the extension of service life, when coupled with favorable inspection results, especially when predicted (but uncalibrated) fatigue lives may be less than the future service life.

*Are there adequate safeguards in proposed assessment and requalification processes to guard against undesirable interactions among traditionally separate design disciplines? Is it possible that, within a given discipline, integration of case specific test results into an interaction formulation could lead to unconservative outcomes? What steps should be taken to guard against hidden introduction of unconservative bias in the process of stripping away conservatisms? Can dynamic measurements be used to calibrate the response to severe environments? What limitations on seastates would be needed? (Issue #6)* (van de Graaf and Tromans 1991; Visser 1993)

Material properties may also be reexamined. It is unlikely that material yield and UTS values will be at the specified minimum or characteristic values, and in many cases a significant margin will exist which may be documented in the form of coupon test certificates.

*Is the use of mill-certified coupon test results appropriate for assessment? If so, are there any limitations or considerations that should be included in their use? In the case of various conventional steels for which coupon test results are not available, is it OK to use recognized material norms, e.g., 43 ksi for A36 steels? (Issue #7)* (Tromans and van de Graaf 1992)

When component capacity or life cannot be shown to resist the applied load ("action") by a sufficient margin, the effect of component failure on the performance of the surrounding structure should be considered prior to committing to expensive offshore strengthening or repair. Nonlinear static or dynamic pushover analysis methods are being developed and increasingly need to investigate reserve and residual strength, redundancy and ductility or structural systems. These concepts, the methods and their usefulness are discussed later in the paper.

## Component Behavior and Analysis

Design codes are intended to ensure that *component* limit states are not exceeded. Limit states considerations relevant to offshore structures are:
- Ultimate strength
- Fatigue
- Deformation
- Ductility
- Damage tolerance

The principal structural components of offshore platforms are members, welded joints and grouted connections. In most cases, ultimate strength and fatigue of these components are covered in design codes, albeit with several areas of significant conservatism as mentioned above. Ultimate strength and fatigue S-N curves are generally based on fitting empirical equations to test data. In limit state approaches, a mean fit to the data is derived, and the scatter of the data about the mean is studied in order to arrive at a "characteristic" value. Alternatively, lower bound formulations are developed by deriving an equation which lies under all available test data. Characteristic or lower bound formulations are reduced by an appropriate "materials" or "resistance" partial safety factor to obtain "design" values. On occasion specific tests may be carried out to determine the capacity of components where a conservatism of the design code is suspected or where a strengthening technique is being quantified (e.g., grout filling of members or joints).

*Is it appropriate or possible to have both limit state and working stress methods used in assessment? How should test data be incorporated in each case and can the results be calibrated to result in the same outcome? How should*

*"safety" factors be established in each case? (Issue #8)* (HSE 1993; Her Majesty's Stationary Office 1990)

Increasingly, nonlinear finite element methods are being used to study component strength. A wide variety of element types, approaches to meshing and material models are available and yet no comprehensive bench marking or convergence exercises appear to have been performed and no authoritative guidance exists. Furthermore, the level of analytical results in relation to "mean", "characteristic" or "lower bound" and appropriate safety factors to obtain "design" values are other considerations.

Evaluating fatigue damage (by the S-N approach) requires calculation of hot spot stress ranges (combination of nominal stress range from global analysis and SCF from parametric equations or specific tests or analytical results). Surprisingly perhaps, a recent study for the Tubular Joints Group revealed there has been only very limited bench marking even for elastic stress analysis of tubular joints and no industry standard approach to choice of elements, mesh size, weld modeling and methods of extrapolation to hot spot values.

***Should guideline methods of modeling and evaluating finite element analyses be established? What guidelines would be appropriate? (Issue #9)*** (Billington Osborne-Moss Engineering Ltd. 1993)

Other aspects of component behavior mentioned above, e.g., deformation, ductility and damage tolerance have received little attention but are becoming increasingly important for assessment. Both elastic and nonlinear joint flexibility has been found to have a significant affect on the behavior of redundant structures. In order to carry out nonlinear collapse analyses, large deformation joint characteristics are required. Unfortunately, most reported ultimate load tests and analyses have not been continued very far into the post ultimate range. There is virtually no test data for low cycle fatigue of tubular joints. This information can be relevant in earthquake dominated structures.

Few test data are available on damaged (other than dented) components. Dents in members with a low probability of detection could have a significant effect on compression capacity. There has been considerable recent work done on damaged members.

The effects of fatigue cracks on static strength and joint flexibility are now receiving attention both analytically and experimentally.

***Are there adequate data to specify assessment procedures that account for component ductility, denting, cracking or low cycle fatigue? If not, where should more research be done? (Issue #10)*** (Moan and Amdahl 1989; Landet and Lotsberg 1992; Ricles et al. 1993; Ostepenko et al. 1993)

Fracture mechanics methods are being widely used to assess defects both to calculate probable crack growth rates and to investigate criticality for fast fracture. Results of these assessments can be sensitive to input data (e.g., material properties, residual stress levels, SCFs, SIFs, defect dimensions) and in many cases such data, particularly material properties, are not readily available and have to be estimated. Increasing reliance is being placed on such methods to rank the seriousness of defects and to justify ongoing inspection and monitoring rather than immediate repair.

***Is the state-of-the-art for fracture mechanics suitable to be used in assessment recommendations or standards? If not, what needs to be done to develop the know-how? (Issue #11)*** (Haswell 1992; BS PD 6493 1991)

## System Behavior and Analysis

Structural system behavior has been investigated on an ad hoc basis with a variety of objectives. Principal areas of interest have been:

- Reserve strength
- Post ultimate residual strength
- Ductility
- Redundancy
- Robustness
- Residual strength of damaged structures
- Forecasting response to extreme events
- Hindcasting to investigate implications of unexpected failures or survivals (Bayesian updating)

- System reliability.

There are many variations in definition of the above terms. There is not a common understanding of these terms among designers, regulators, and the engineering community as a whole.

*Is there a commonly accepted definition for each of these performance measures? If so, should there be numerical targets associated with them in requalification recommendations? If there are not accepted definitions, who should define them or how should they be developed? (Issue #12)* **(Billington et al. 1993; Lloyd and Clawson 1984)**

Global structural system analysis may be carried out at various levels of sophistication:
- Linear elastic members/rigid joints (with or without dynamics)
- Linear elastic members and joints
- Members with geometric nonlinearity (large deformation beam-column buckling)
- Nonlinear member material properties (plasticity, elasto-plastic, strain hardening, ductility limits, fracture)
- Members with both geometric and material nonlinearity
- Nonlinear joint behavior
- Nonlinear members and joints with ductility limits and fracture assessments

There are about ten nonlinear frame analysis packages available for offshore structural analysis, although none has all the above capabilities. This is a rapidly developing field. Nonlinear frame analysis techniques are being widely used in assessment and to a lesser extent in design, particularly to assess extreme or accidental conditions such a ship impact, fire, blast, dropped object impact, and extreme environmental conditions (hurricanes, typhoons, seismic events, etc.).

To date very limited validations of analytical methods against frame test results and bench marking of alternative analysis methods have been carried out. The results of nonlinear analyses of redundant structures can be sensitive to input data. System reliability approaches are being developed to model this variability in order to obtain probability distributions of strength. These nonlinear analyses are being widely used in assessment of complex structures.

*How accurately can nonlinear calculations for complex structural systems be expected to model true structural behavior? What levels of calculation precision can be expected? What additional research is needed to validate and benchmark these methods and computer programs? Should a standard benchmark structural model be developed to validate programs and component modeling techniques?*
*Are there appropriate means to infer system probabilistic behavior from the uncertainties associated with components of complex, nonlinear systems? If so, what means should be adopted? (Issue #13)* (Billington Osborne-Moss neering Ltd. 1993)

Various studies into system behavior have considered structural configuration, bracing arrangements, interaction between redundant frames, their effect on overall redundancy and the ability to mobilize alternative load paths. Vertical bracing within a panel bounded by vertical or inclined legs and horizontals can be a single diagonal, K or X. With failure of the bracing system the panel relies on the portal frame action of its boundary members (legs and horizontals) for strength. X-braced panels with both tension and compression diagonals have greater redundancy than other types. Interaction between vertical and plan bracing is particular important for X-braced structures without horizontals and K-braced structures with diamond plan bracing. A number of analyses have been reported where failure of the first critical component equates to system ultimate capacity (i.e. no system reserve).

*Is it possible to characterize the behavior of systems by reference to analyses of common structural systems? Is it possible to develop a criterion for establishing loading scenarios, based on these analyses, for which structures may require assessment? (Issue #14)* (Bea and Craig 1993)

## Acceptance Criteria

Assessment or requalification implies consideration of a structure part way through its original design life.
- There will be an established and at least reasonably well documented service history.

- Mill certificates may be available to provide better material information than available for design.
- There may be inspection records.
- The structure may have deteriorated from its as-installed condition and the structure may have construction defects not anticipated in its design.
- There may be accidental or other damage.
- The remaining required operational life may be less or greater than the original design life.
- The original design criteria and structure function may have changed during the structure's life. For example, better environmental data may be available or the structure may now be unmanned.

The foregoing sections consider the approaches available (1) to determine the distribution of loads within the structure, (2) to consider the resistance to such loads at a component and system level and (3) to analyze load and global behavior.

The economics of platform operation and assessment and associated time scales give structural engineers the opportunity to use their ingenuity to minimize the cost of inspection, maintenance and repair, particularly as assessment approaches are not fully codified and safety factors or target risk levels are not prescribed.

At several instances in the foregoing sections, reference is made to the need to consider partial safety factors, and their sensitivity to the input data and risk levels.

It is assumed that the operational, safety and environmental targets will be established by other working groups.

*Is assessment (or design) of structural components loaded primarily by environmental loads needed for operational environmental conditions? (Issue #15) (see Issues #14 and #16)*

Risk analyses will indicate critical loading scenarios for evaluation. First pass structural analysis will demonstrate the criticality of components for each case. From this analysis the components may be ranked according to importance within the intact structure. Reserve strength or redundancy analysis may be required to further evaluate the effect of overloaded or damaged components.

*What criteria should be used to rank the importance of components in extreme or accidental loading events? Level of overload? Alternative load paths? Strength utilization? (Issue #16) (Marshall 1992)*

*As a structure ages, its reliability may reduce, but inspections of "critical" areas may increase confidence and enable new assessments of reliability to be made. As a structure nears the end of its life, the probability of its being subjected to an extreme event diminishes, though the annual risk remains constant.*
*Is it practical to establish target reliabilities for requalification acceptance criteria? Can calculated probabilities be expected to represent actuarial statistics? If calculated probabilities cannot be used for acceptance criteria, what is the best alternative measure of acceptability? Should there be a simple quantified measure?*

*Should "target reliability levels" be modified to account for remaining length of life? (Note that the API approach does not address this point explicitly. The API approach adjusts acceptance criteria based on consequences.) (Issue #17) (Titus and Banon 1988; Piermatti et al. 1990)*

## Mitigation

Once mitigations are shown to be necessary, there are three alternatives:
- reduce loads,
- increase strength, and/or
- modify consequences of failure.

Load reduction methods include:
- marine growth control,

- removal of unused conductors,
- removal of unnecessary conductor guide framing or modification to reduce hydrodynamic loading,
- removal of redundant installation aids (e.g., pile guides, launch rails, launch trusses, lift attachments),
- topsides load reduction,
- deck elevation or relocation of cellar deck equipment to higher elevations,
- removal of unnecessary appurtenances, boat landings, walkways ladders, etc.,
- consideration of shielding effects, and/or
- prevention of scour.

Alternatively, strengthening may be necessary. Any repair project will be very structure specific. The ability to obtain accurate underwater data will be a primary consideration in selection of the repair method. Diving and other underwater or atmospheric working conditions will also affect the method of repair. Once such constraints are established the repair method can be selected and designed. The following are some of the methods available:
- welding (wet, atmospheric, cofferdam, hyperbaric, friction),
- clamping (mechanical, grouted, stressed grouted, pressurized, neoprene lined, resin, short and long bolted),
- grout filling (members, joints, annuli, piles),
- weld improvement (grinding, peening),
- defect removal (grinding, arrester holes), and/or
- adhesives.

The success of repairs is closely linked to careful consideration of the installation conditions and capabilities. This includes:
- diver capability,
- installation aids/impediments,
- lifting/rigging capacity,
- sea conditions (waves, current),
- visibility, and
- ROV capabilities.

Other considerations are corrosion protection, sealing, bolting and post installation inspection. Existing repair methods have an extremely good record for reliability and many have achieved upwards of 10 years service in the North Sea and elsewhere.

An alternative to reducing loading or increasing strength is modification of the consequence of failure. There are three factors to consider: life-safety, environmental consequence, and economic consequence. While all three are related, failure consequences may be addressed separately. The following mitigations can be considered:
- reduce manning levels
  - de-man during extreme events (as in the Gulf of Mexico)
  - unmanned platform (remote operation) verify adequacy for time being manned
- minimize environmental consequences
  - downhole safety shutdown valves for flowing wells
  - pipeline shutdown valves
  - remove any large oil storage vessels on the platform
- consider economic consequences of failure
  - move key facilities to other "safe" platforms
  - re-drill at other locations instead of platform under consideration
  - cost versus risk economic trade-off considering field life, reserves, etc.

*Mitigation measures have varying degrees of effectiveness and reliability. Which measures are relatively ineffective and which need substantial development work to be reliable? (Issue #18)* (Martindale et al. 1989; Kallaby et al. 1993)

## Design and Construction for Assessment and Requalification

Assessment projects are frequently hampered by lack of information which could have been readily collected

during the design and construction period. The design should be undertaken against the background of target reliabilities and design lives and therefore the inspection and requalification programs should be a design consideration. The opportunity can be taken to adjust the design to minimize the inspection effort required to achieve the ongoing reliability requirement. An example is the specification of high minimum fatigue lives (e.g., 10 x design life) for which some European Certification Authorities allow significantly reduced joint inspection.

In any event the preparation, archiving and preservation of important design and construction data have negligible cost implications, if planned into the project. Information which should be preserved includes:

- design basis,
- design report,
- data files for design analytical model,
- analysis results (member forces, member and joint utilizations, fatigue lives),
- plate mill certificates,
- coupon test results,
- weld procedures and procedures test results,
- impact test results,
- fracture test results,
- material usage records,
- NDT records and details of fabrication defects, and
- as built dimensions.

The data can be stored in electronic form and paper records can now be scanned very efficiently into a knowledge base using optical character recognition software.

*What amount of structure archiving can be automated? Is it practical to establish standards for archiving based on emerging storage devices? Is there existing software that can be used to put all the information on a single ROM, including photographs, etc.? (Issue #19)* (Billington and Vasudevan 1992)

## Summary of Issues

The number of reference dealing with "issues" raised in this working group document are considerable. Some references relate to several of the "issues" and it is intended that together the papers should give further background to the debates currently surrounding the assessment and requalification of structural elements and systems and the available analytical techniques. A more complete set of references related to assessment of existing platforms will be included in the new Chapter 17 of the API RP2A-WSD.

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# FOUNDATION ELEMENTS, SYSTEMS AND ANALYSIS

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## Scope of Workshop Discussions

The purpose of this paper is to identify the significant foundation related issues affecting the assessment and requalification of offshore production platforms. These issues will provide a format for detailed discussion by workshop participants. Specifically participants will attempt to define for each issue:

- The state of the practice
- Major deficiencies in the practice
- Recent advances or on-going work to improve the practice
- Directions for future research and development

There are many problems in offshore foundation design practice that are common to those of assessment of existing structures. Furthermore, there are many foundation issues that overlap with the subjects of other work groups such as loading, structural interaction, etc. A full discussion of such topics within the brief allocated time is impractical. Therefore, in the foundation sessions, attention will be focused strictly on foundation issues that are particularly pertinent to the requalification question.

The authors have chosen to group the discussions under three major topics, each with a set of issues to be discussed at the workshop:

- data gathering and review
- assessment
- upgrading options

As indicated above, not all issues could be included. We believe that it is preferable to have a more thorough discussion of a few issues than superficially covering all aspects.

## Data Gathering and Review

The first step in assessment of existing foundations is to collect and review the available data that pertain to present conditions. This includes a wide range of information and is discussed in more detail in the following paragraphs.

### Site data and other design documentation

A partial list of the information that may be useful in an assessment is as follows:

- Shallow seismic, side scan, regional soil borings and *in-situ* tests
- Regional/local geology
- Soil borings at the site-number, location, depths etc.
- Geotechnical data reports
- Site information from neighboring structures
- Bias and uncertainty in the design parameters
- Hazards considered in the design or new ones identified
- Analysis/design models and calculations

For the purpose of this discussion let us consider two situations: critical elements of the above information are not available or do not meet 'new design' standards and the basic design information is adequate.

The first situation inevitably engenders considerable 'soul searching'. A key issue that must be addressed in this case is:

*To what extent should indirect information such as geophysical, regional trends (e.g., data from nearby sites), geologic history, etc. be used in establishing analysis parameters such as strength, density, etc.? In the Gulf of Mexico? In the North Sea? Discussion of this issue should include ancillary questions such as how far from an actual boring can extrapolations be made? When should a new boring be recommended? Can minimum strength profiles be established for a given geologic setting? (Issue #1-Use of Indirect Information)*

The second situation is not always straight-forward either. A key issue here is:

*What parameter corrections are justified where 'old' methods of sampling and testing were used in the original site investigation? For example, is it appropriate to apply corrections to data from driven samples? From unconfined compression tests? What should be the standards for parameter determination? To what extent should "old" (perhaps inadequate) analysis models/correlations be considered/corrected for? (e.g., pore pressure corrections for CPT?) (Issue #2-Assessment of "Old" Site Specific Data)*

### Construction Records and Geotechnical History

After the structure is installed and has been in service, the engineer has access to more information than was available during design and, thus, may have a basis for modifying the foundation model. Construction records (if and when available) should indicate structure and foundation revisions that occurred during construction and installation. In addition, these records may help to assess the design assumptions, e.g., location of end bearing sand layers from pile driving blow counts. The foundation's history, such as survival of storms, instrumentation records of structural response, pore pressure response, etc., can also be valuable sources for verifying/revising design models and assumptions.

The following information is useful for evaluation of all types of foundations:
- Structural modifications (e.g., may affect foundation load distribution)
- Historical platform loading (deck loads; drilling loads; estimated wave, wind, current, ice and earthquake loads) and relevant checks of component capacities
- Instrumentation (accelerometers, strain gages, pore pressure transducers, etc.)
- Remedial measures taken during installation
- Observations during platform operations such as scour depths

The following types of foundation information are specific to pile foundations:
- Integrity of grouted connections for piles (e.g., pile sleeves for skirts, sleeve piles for drilled and grouted inserts, etc.) such as indications of visible grout returns
- Incidences of jetting, drilling out plugs, drilling pilot holes, etc. to advance driven piles to grade
- Installed pile lengths and wall thickness schedules, particularly as they differ from design
- Pile and conductor driving records including pile driving hindcasts where available

In most cases, particularly in mature operating areas, pile installation is carried out essentially as planned. Furthermore, pile and conductor installation records are often among the data that are preserved and retrievable. These records, for the most part, contain blow count versus depth recordings plus notations of add-ons and other delays. Further, any remedial measures required to advance the pile are usually noted such as jetting or drilling pilot holes. As such, these data are an important potential source for verifying and/or improving the axial pile capacity design model. The considerable controversy regarding the interpretation of such data gives rise to a key issue:

*Should pile installation data (blow counts, instrumentation, locations and size of pilot holes etc.) be used to update axial pile capacity estimates? What are the limitations of such revisions? How does installation data relate to long term capacity? Is restart or retap data more appropriate? (Issue # 3-Use of Installation Data, Pile Foundations)*

The following types of information are specific to shallow foundations (e.g., gravity structure foundations, jack-up spud cans and mats, anchors, mudmats, etc.)
- Installation records including skirt penetration, under base grouting,
- Foundation performance over time including scour, long term settlement

- Settlement during severe environmental loading
- Instrumentation results including pore pressures during loading and accelerations (for stiffness assessment)

For shallow foundations, observations and response during installation and during subsequent environmental loading can provide useful indications on foundation behavior under different conditions. These can allow one to verify design assumptions. Back-calculations are not always done, instrumentation results are often limited to data reduction without interpretation in terms of the observed data significance from the foundation response point of view.

*Should foundation engineering also include a reassessment of the soil parameters from the results of observations during installation or later on in the life of the platform? Should the parameters be adjusted to reflect an "as installed" foundation in case specific events or penetration observations suggest that changes may be warranted? If so, how can this be done? Should pore pressure response be compared to what is expected on the basis of the assumptions made when defining the soil parameters for the different components of the foundation analysis? (Issue # 4-Use of Performance Observations, Shallow Foundations)*

### Physical Survey Data of the Structure Foundation System
A physical survey of the structure and foundation during the platform's life, and especially near the time of the assessment can be very useful for updating the foundation model. Any change in the structure can give rise to changes in the load distribution in the foundation. An inspection of the seabed in the proximity of the structure's base can be useful in evaluating the contribution of mudmats and horizontal framing members to the strength of the foundation system. The existence of scour (or lack thereof) around piles can be helpful in updating design models. It is particularly helpful if periodic survey data (e.g., ROV) is available to assess long term trends or changes due to specific events (e.g., especially severe storms, harsh winters, etc.)

## Assessment

Having acquired the available data for analysis, the next step is to carry out the assessment(s). Three sequential analysis checks of existing structures have recently been proposed (Draft Revisions to API RP2A for Assessment of Existing Platforms): screening analysis, design level analysis and ultimate strength analysis. For the purposes of discussion, this three-level format will be adopted here.

### Level 1-Screening Analysis
Prior to conducting a screening level analysis, a review of the data is carried out to determine whether the platform has been damaged, loading has increased, air gap has been reduced, or in general the design environment has changed. If the environment is significantly different from the design assumptions then the screening analysis check is skipped and the design level analysis is conducted. If not, the screening analysis check is carried out. This simply involves confirming that the platform was designed to a specified edition of API RP2A or equivalent. For this level of checking it is appropriate to confirm that the design soil parameters are valid, e.g., that the platform location with respect to the soil boring(s) is as assumed in the design. If the platform passes the screening level check, no further action is required. If not, a design level analysis is required.

### Level 2-Design Level Analysis:
For the purposes of conducting a design level analysis of an existing structure, the use of 'nominal' design parameters as would be used in new design (as opposed to 'best estimate' parameters) is prescribed. However, during the original design process, various conservatisms are introduced to account for unknowns and uncertainties that inevitably arise in the construction, installation, and service of the platform. Furthermore, new platform specific information as well as new research results may be available since the original design. In these instances it may be appropriate to use the following in the analysis:
- Revised loads
- Revised soil parameters
- Hindcasts of component behavior based on pile driving, skirt penetration, measurements, etc.
- Identification of all important potential failure modes (especially identification of 'new' modes; i.e., those different from ones identified in the original design)
- New engineering analysis including

Steel piled jacket (axial capacity, laterally loaded pile stress analysis, earthquake response)
Shallow foundations (bearing capacity, hydraulic stability, soil structure interaction, settlement, earthquake response)
- Probabilistic analyses to better understand the sensitivities of the foundation performance to parameter bias and uncertainty
- Reconcile observed versus calculated behavior

Key issues in carrying out a design level foundation analysis are:

*Should axial pile capacity determinations explicitly account for cyclic and rate of loading effects? If so, should nominal factors be used or should laboratory tests be the basis? What factors? What laboratory tests? Should assessment include new developments (in favorable or unfavorable directions) with respect to pile capacity that have been published since the design calculations were completed? To what extent should research results (recently published studies) be used for the assessment (especially when results are in disagreement with current practice)? (Issue #5- Use of Recent Research Results in Pile Capacity Assessment)*

*What is the appropriate role for probabilistic analysis in foundation assessment? Are estimated safety indices for axial pile capacities believable? How should the acceptable reliability level be established (probability of failure or safety index equivalent to a safety factor?). What should be done, if anything, to resolve differences in working strength and LRFD design values? (Issue #6-Role of Probabilistic Analysis)*

### Level 3-Ultimate Strength Analysis
The third level of checking (for platforms that do not pass screening or design analysis levels) is to carry out an ultimate strength analysis. This is typically done by scaling up the environmental forces for the design level event on a nonlinear structural model until the collapse load is achieved. This is frequently referred to as a 'pushover' analysis. The general considerations are similar to those enumerated above for design analysis except that 'best estimate' rather than nominal parameters are used and other known conservatisms are removed. An issue that is particularly important for and unique to nonlinear analysis is

*What criteria should be used for modeling the lateral soil resistance along a pile for 'pushover' analysis? Should so-called 'static' or 'cyclic' models be used? Should a displacement limit be placed on such analyses? (Issue #7-Criteria for Pushover Analysis)*

## Upgrading Options

At some point during an assessment it may become apparent that remedial action of some type is indicated (especially where a platform fails to pass the Level 3, ultimate strength analysis, check). There are a number of actions that can be carried out to upgrade the foundation (or perhaps downgrade the requirements, e.g., de-manning) to render it suitable for continued service. Among the alternatives are
- Collect new soils data (improved quality, more borings, closer to site, etc.) to justify more optimistic interpretation
- Install instrumentation to verify design/analysis assumptions
- Set criteria for shutdown, evacuation, etc.
- Enhance pile performance
    Retap or pull conductors (or piles if practical) to assess capacity
    Upgrade capacity by adding insert piles or artificial plugs
    Add 'outrigger' piles
- Enhance shallow foundation performance
    Add berms around base of structure
    Add scour protection
    Add piles in skirt compartments or around periphery to enhance sliding stability
    Add ballast to structure to enhance overturning or sliding stability

A key issue regarding what is often considered one of the first lines of defense is as follows:

*How does the foundation engineer prioritize or weight the competing factors in his recommendations to the owner, such as: safety requirements; the reality of having a structure that is already installed with a safety factor below standard or with key pieces of information missing; the expected conservatism built into parameter selection and assessment methods; the huge costs that may be involved; the uncertainty in effectiveness (and even risk of making things worse) of the proposed remedial action (e.g., how effective are remedial measures aimed at strengthening piles such as adding inserts and artificial plugs?). (Issue #8-Remedial Measures )*

## Closing Remarks

There are many actions that can be taken during the design, construction, installation and service life of a structure that can facilitate its subsequent assessment and requalification. As such, assessment requirements should be made a part of the plan from inception. A carefully thought out system of archiving pertinent data, calculations, measurements, and inspection results throughout the life of a platform, is one of the most important aspects of such a plan. Such a system is greatly enhanced by a clearly marked trail over the life of the structure indicating what decisions were made and why. Effort spent on the front end in this manner will inevitably pay for itself many times over.

# OPERATIONAL CONSIDERATIONS

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***Working Group #5***

## Operational Issues—Definition and Intent

What is meant by 'Operational Considerations' in the context of this workshop on aging structures is addressing risk sources to a functioning offshore production platform that are unrelated to overload of the structure from the natural environmental hazards of storms, earthquakes and ice.

It means the identification of 'non-structural' risk sources capable of causing injury, pollution or destruction of the structure - what type of risks, how potentially damaging, what are their likelihoods of occurrence, and what mitigating measures control them.

Such risk sources can be the under-performance of aging equipment - process equipment, drilling equipment, instrument and control systems, fire and safety systems, SCV's and SSCV's, risers, cathodic protection systems, navigational warning aids, and the deck structure itself.

Such risk sources can also be the under-performance of the designers and operators of this aging equipment - which raises questions on such diverse issues as operator training and retraining, human and organizational errors, safety review procedures, safety team makeups, operating and safe work practices, blowout control procedures, effects of dropped objects, weight control, inspections, walk-downs, evacuation procedures, and de-manning.

The goal of this working group is to identify the important 'non-structural' sources of risk, their damage potential, their perceived likelihoods of occurrence, what is being done to control them, and where/if such controls need improvement, all in the context of existing aging facilities. The intent is to provide an awareness brief on these hazards. It is not to provide an in-depth treatment of process hazards, a subject which more than justifies a workshop of its own.

Issues are listed in what follows to promote discussion in the working session that should help achieve the above stated goal. Also included at the end of this paper is an analysis of events from MMS file data and MMS accident statistics. These are presented to enhance discussion.

## Operations Related Risks-Sources

The risk of the loss of serviceability of an operating production platform includes many components that are unrelated to storm, earthquake or ice overload. Figure 1 from API RP 14J Design and Hazards Analysis for Offshore Production Facilities illustrates some of these sources. The figure attempts to identify operational sources which could lead to an event (pollution, fire, explosion, or injury).

*Does this figure adequately represent risk sources for production facilities? Is there a better way of depicting significant sources? (Issue #1)*

*Can this figure be modified to include drilling and workover activities? How should it be modified or how can this be identified? Are blowouts age-related? (Issue #2)*

*Does the figure adequately represent risk sources from construction and maintenance activities (concurrent with production, or otherwise)? Can the figure be modified, or is there a better way to identify these sources? Is concurrent construction activity a major culprit? (Issue #3)*
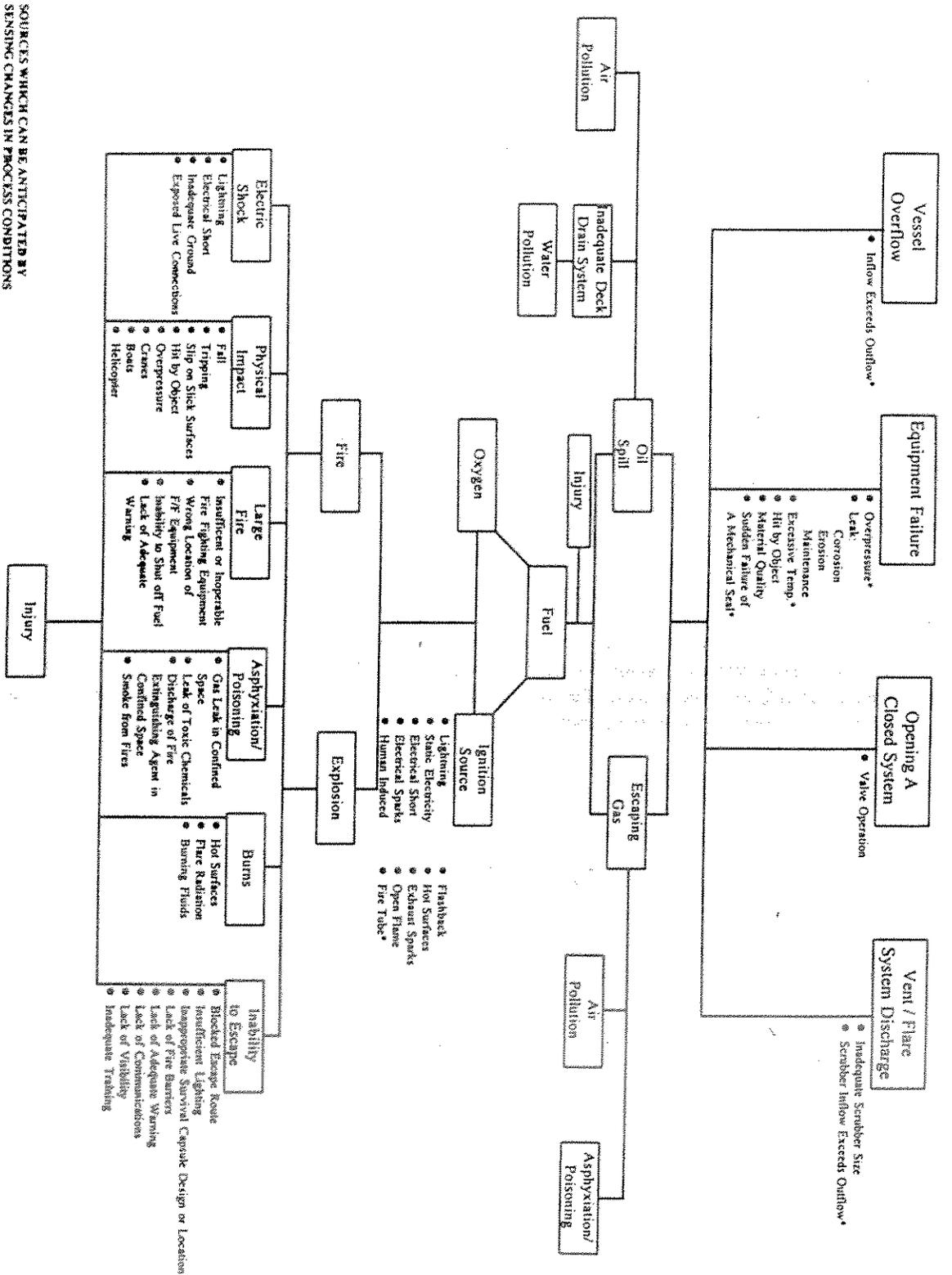
**Figure 1 Hazard tree for production facility.**

\* SOURCES WHICH CAN BE ANTICIPATED BY
SENSING CHANGES IN PROCESS CONDITIONS

Vessel Overflow
• Inflow Exceeds Outflow*

Equipment Failure
• Overpressure*
• Leak:
  Corrosion
  Erosion
  Maintenance
• Excessive Temp.*
• Hit by Object
• Material Quality
• Sudden Failure of
  A Mechanical Seal*

Opening A Closed System
• Valve Operation

Vent / Flare System Discharge
• Inadequate Scrubber Size
• Scrubber Inflow Exceeds Outflow*

Air Pollution

Water Pollution

Inadequate Deck Drain System

Oil Spill

Injury

Escaping Gas

Air Pollution

Asphyxiation/ Poisoning

Oxygen

Fuel

Ignition Source
• Lightning
• Static Electricity
• Electrical Short
• Electrical Sparks
• Human Induced
• Flashback
• Hot Surfaces
• Exhaust Sparks
• Open Flame
• Fire Tube*

Fire

Explosion

Large Fire
• Insufficient or Inoperable
  Fire Fighting Equipment
• Wrong Location of
  F/F Equipment
• Inability to Shut off Fuel
• Lack of Adequate
  Warning

Asphyxiation/ Poisoning
• Gas Leak in Confined
  Space
• Leak of Toxic Chemicals
• Discharge of Fire
  Extinguishing Agent in
  Confined Space
• Smoke from Fires

Burns
• Hot Surfaces
• Flare Radiation
• Burning Fluids

Inability to Escape
• Blocked Escape Route
• Insufficient Lighting
• Inappropriate Survival Capsule Design or Location
• Lack of Fire Barriers
• Lack of Adequate Warning
• Lack of Communications
• Lack of Visibility
• Inadequate Training

Electric Shock
• Lightning
• Electrical Short
• Inadequate Ground
• Exposed Live Connections

Physical Impact
• Fall
• Tripping
• Slip on Slick Surfaces
• Hit by Object
• Overpressure
• Cranes
• Boats
• Helicopter

Injury

*Should the figure be modified to include the potential effects of ship collisions and dropped objects, or should they be referenced elsewhere (draft API RP 2A, Section 18, Accidental Loading)? Are dropped objects a real risk? Are riser guards effective? (Issue #4)*

*Should the figure include risk sources from organizational and human (operator) errors? (Issue #5)*

*Where are potential shortcomings in the deck structure (and even in the jacket structure) addressed or referenced (API RP 2A Section 18, Fire & Blast)? Where is maintenance of adequate cathodic protection addressed? What about weight control? (Issue #6)*

*Which are the more important of these risks? Do we have adequate data? Do we need more data? Is age a factor? Is this figure relevant for the 'assessment' of existing facilities? (Issue #7)*

The object is to identify significant risk sources (risk = consequence x likelihood), considering all aspects of an existing production operation - production, construction, maintenance, drilling, workover, etc., and to develop a logical framework for their further review. This may be in the form of API RP 14J's Figure 1, or something different.

## Operations Related Risks- Likelihoods and Mitigation by Design

The likelihood that a source will develop into an event may be a function of several design-related factors, including the age of the facility, the complexity of the process, fluid properties (pressure, corrosiveness), the degree of documentation of material and construction QA/QC, the codes and standards employed in design, the hazards analysis and safety case techniques employed, and whether the facilities are designed for manned or unmanned operations.

*Can we identify design related factors which effect the likelihood of a source developing into an event? (Issue #8)*

*What quantitative or qualitative evidence can be used to aid in developing probabilities? What are the uses of failure databases (OREDA, etc.)? (Issue #9)*

*Do cost/benefit relationships exist in determining the effort to expend on analyzing risks? (Issue #10)*

## Operations Related Risks- Mitigation by Safety Management

Safety Management elements involve managing the operations occurring on the platform which are generally unrelated to the specifics of the design itself. These elements are explained in API RP 75, and include:
- Safety and Environmental Information
- Hazards Analysis
- Management of Change
- Operating Procedures
- Safe Work Practices
- Training
- Assurance of Quality and Mechanical Integrity of Critical Equipment
- Pre-Start-up Review
- Emergency Response and Control
- Investigation of Incidents
- Audit of Safety and Environmental Management Program Elements

The extent to which these program elements are in place can affect the likelihood that a source will develop which could turn into an event.

*Are other program elements needed? (Issue #11)*

*What objective evidence exists to show that these elements reduce risks? (Issue #12)*

*How to measure the degree of compliance with these elements? (Issue #13)*

*Hazards Analyses. What triggers such? What are their appropriate scopes and frequencies? (Issue #14)*

*Management of Change. How is this process implemented and audited? What are the right roles and responsibilities?*
*Is this the key to aging-independence? (Issue #15)*

*Mechanical Integrity Maintenance. What are the focus areas, frequencies and scopes? Are risk-based inspection and maintenance (Asset Integrity Management) programs justified, especially for short-lived operations? How else to justify the I&M costs (show they result in significant risk reduction)? Does 'mechanical integrity' include that of the jacket? Who is responsible for the maintenance of subsea cathodic protection? What of the effects of new tie-ins of pressured production? (Issue #16)*

*How should the errors by organizations and by operators be quantified and better controlled? Should there be minimum, mandated training and retraining requirements for operators? What should they be? (Issue #17)*

## Operations Related Risks - Consequence Control Measures

The risk of a source developing into an event or the risk of a small event developing into a larger event can be reduced by mitigation and control methods. These methods can include layout choices, use of fire and blast walls, fire fighting and control philosophy, manning decisions, evacuation philosophy, structural design considerations, etc..

*Which mitigation and control measures should be addressed? (Issue #18)*

*Does manning reduce or increase risks? To what extent does manning philosophy influence design? Is there a need for generic Safety, Health and Environment Management Systems? Is the issue safety or loss of a job? (Issue #19)*

*What are the goals of and design criteria for fire and explosion control/protection? What are the implications for structural design? (Issue #20)*

*Are there hard and fast rules for layout considerations other than those contained in API RP 14J? (Issue #21)*

*Have we learned anything from Safety Case studies that have general application? Now that a number of Safety Cases have been performed, is there a demonstrated rationale to continue to require that they be performed in all instances in the North Sea? (Issue #22)*

## Operations Related Risks - Potential Improvements

Improvements can evolve from a better understanding of operational risks and trade-offs. A better understanding of probabilities and consequences is needed.

*How can we improve data on likelihoods in a cost effective manner? When is detailed probability analysis neces-*

*sary? (Issue #23)*

*How can we improve data on effectiveness of mitigation/consequence control measures? When is detailed consequence analysis necessary? (Issue #24)*

*How can we better assess cost/benefit relationships for attention to activities which increase efforts devoted to safety analysis and documentation? (Issue #25)*

*How can operators better understand the full, integrated risk to their facility (operational and structural)? (Issue #26)*

*How can human error be better quantified and controlled? (Issue #27)*

*Are there new products that can significantly enhance mechanical integrity evaluation or preservation (e.g., glass fiber reinforced epoxy in fire water systems)? (Issue #28)*

## Summary and Conclusions

Risk sources related to the operation of production platforms are identified, as are the measures used to control them. Questions on how to improve on this control are listed. These 'non-structural' risks appear to be more significant than those associated with the overload of the structure from natural hazards.

# FATALITIES RELATED TO PRODUCING OPERATIONS

| YEAR | NUMBER |
|------|--------|
| 1982 | 12 |
| 1983 | 7 |
| 1984 | 4 |
| 1985 | 1 |
| 1986 | 1 |
| 1987 | 0 |
| 1988 | 0 |
| 1989 | 7 |
| Total | 32 |
| Average/yr 1982-85 | 6.0 |

Source: "Improving Safety of Production Operations in the U.S. OCS," Kenneth E. Arnold, P.S. Koszela and J. C. Viles, World Oil, July 1990.

**Analysis of MMS Events File Data**
*Courtesy of Paragon Engineering Services, Houston, Texas*

## CAUSE OF FATALITIES, 1982-1989
## FROM MMS EVENTS FILES

| CAUSE OF FATALITIES | NO. OF EVENTS | NO. OF FATALITIES |
|---|---|---|
| Opening a pressurized system for maintenance | 6 | 12 |
| Drowning | 5 | 5 |
| Handling heavy loads (including crane accidents) | 4 | 4 |
| Fall from height | 4 | 4 |
| Helicopter | 1 | 2 |
| Unknown[1] | 1 | 2 |
| Illness and heart attack | 1 | 1 |
| Electrical shock | 1 | 1 |
| Vapors from drain lines | 1 | 1 |
| TOTAL | 24 | 32 |

NOTE:[1] = Supply boat docked at production platform exploded.

Source: "Improving Safety of Production Operations in the U.S. OCS," Kenneth E. Arnold, P.S. Koszela and J.C. Viles, World Oil, July 1990.

**Analysis of MMS Events File Data**
*Courtesy of Paragon Engineering Services, Houston, Texas*

# FIRES AND EXPLOSIONS RELATED
# TO PRODUCING OPERATIONS

| YEAR | NUMBER |
|---|---|
| 1982 | 50 |
| 1983 | 65 |
| 1984 | 55 |
| 1985 | 58 |
| 1986 | 38 |
| 1987 | 35 |
| 1988 | 27 |
| 1989 | 15 |
| Total | 343 |
| Average | 43 |

Source: "Improving Safety of Production Operations in the U.S. OCS," Kenneth E. Arnold, P.S. Koszela and J. C. Viles, World Oil, July 1990.

**Analysis of MMS Events File Data**
*Courtesy of Paragon Engineering Services, Houston, Texas*

## EVALUATION OF EVENT LISTING FOR ACCIDENTS
## CAUSING INJURIES IN THE GULF OF MEXICO, 1982

| EVENT TYPE | NO. OF EVENTS | NO. OF INJURIES |
|---|---|---|
| Falling from height | 36 | 36 |
| Handling of heavy loads (including crane accidents) | 35 | 35 |
| Loss of footing/walking into objects | 25 | 25 |
| Improper use of tools or equipment | 17 | 17 |
| Opening pressurized equipment | 11 | 15 |
| Engine/Compressor/Turbine maintenance and operation | 6 | 11 |
| Boat accidents | 5 | 9 |
| Sandblasting operations | 5 | 5 |
| Welding and cutting operations | 4 | 5 |
| Illness/heart attack | 4 | 4 |
| Walkway failures | 3 | 3 |
| Drain and sump systems | 1 | 4 |
| Design violations | 1 | 3 |
| Helicopter accidents | 1 | 2 |
| Electrical shorting | 1 | 1 |
| Diving operations | 1 | 1 |
| Unknown | 1 | 1 |
| TOTALS | 157 | 178 |

Source: "Improving Safety of Production Operations in the U.S. OCS," Kenneth E. Arnold, P.S. Koszela and J. C. Viles, World Oil, July 1990.

**Analysis of MMS Events File Data**
*Courtesy of Paragon Engineering Services, Houston, Texas*

# EVALUATION OF EVENT LISTING FOR ACCIDENTS
# CAUSING POLLUTION IN THE GULF OF MEXICO

| DESCRIPTION OF CAUSE | NO. OF EVENTS | NO. OF BARRELS |
|---|---|---|
| Unknown | 15 | 38 |
| Drain and sump systems | 13 | 34 |
| Liquid discharge through vent | 8 | 18 |
| Handling heavy loads | 5 | 16 |
| Pipeline leak/failures | 5 | 15 |
| Equipment failures | 5 | 13 |
| Drip pan design | 3 | 13 |
| Boat collisions | 2 | 6 |
| Safety devices bypasses | 2 | 6 |
| Poor operating procedures | 1 | 48 |
| Welding and cutting operations | 1 | 2 |
| Opening pressurized system | 1 | 2 |
| Electrical shorting | 1 | 1 |
| Improper tool or equipment use | 1 | 1 |
| Control component failure | 1 | 1 |
| Totals | 64 | 214 |

**Analysis of MMS Events File Data**

*Courtesy of Paragon Engineering Services, Houston, Texas*

## SUMMARY OF 1982 OFFSHORE EVENT LISTING
## (DRILLING)

| CAUSE OF EVENT | NUMBER OF EVENTS |
|---|---|
| Falling, handling heavy loads, etc. | 209 |
| Tank runover, ruptured hose | 16 |
| Electrical systems | 9 |
| Loss of well control | 8 |
| Pollution from mud circulating system | 5 |
| Welding | 4 |
| Opening a pressurized system | 3 |
| Equipment overpressure | 1 |
| Premature firing of perforating gun | 1 |
| Boat collision | 2 |
| Unknown | 4 |
| Totals | 262 |

**Analysis of MMS Events File Data**

*Courtesy of Paragon Engineering Services, Houston, Texas*

# EVALUATION OF EVENT LISTING
## FOR ACCIDENTS CAUSING FIRES OR
## EXPLOSIONS IN THE
## GULF OF MEXICO, 1982

| EVENT TYPE | NO. OF EVENTS | NO. OF INJURIES |
|---|---|---|
| Engine/Compressor/Turbine maintenance & operation | 13 | 10 |
| Unknown | 8 | 1 |
| Welding & cutting operations | 7 | 1 |
| Equipment failure | 5 | 0 |
| Electrical shorting | 3 | 0 |
| Opening pressurized system | 2 | 3 |
| Improper tool or equipment use | 2 | 1 |
| Poor operating procedures | 2 | 0 |
| Lightning | 2 | 0 |
| Drain and sump systems | 1 | 4 |
| Design violations | 1 | 3 |
| Sandblasting operations | 1 | 1 |
| Improper material storage | 1 | 0 |
| Liquid discharged through vent | 1 | 0 |
| Control component failure | 1 | 0 |
| Totals | 50 | 24 |

Source: "Improving Safety of Production Operations in the U.S. OCS," Kenneth E. Arnold, P.S. Koszela and J. C. Viles, World Oil, July 1990.

**Analysis of MMS Events File Data**
*Courtesy of Paragon Engineering Services, Houston, Texas*

STATISTICS FROM MMS 88-011

Accidents Associated with Oil and Gas

Operations (Outer Continental Shelf)

I    BLOWOUTS

A.    Type of Wells

| | | |
|---|---|---|
| Gas Wells ------------ | 101 | (70%) |
| Oil Wells ----------- | 7 | ( 5%) |
| Oil & Gas Wells ------ | 9 | ( 6%) |
| Gas/Condensate Wells | 3 | ( 2%) |
| Unclassified ---------- | 25 | (17%) |

TOTAL    145

B.    Operations at time of Blowout

| | | |
|---|---|---|
| Drilling ----------- | 92 | (63%) |
| Workover ----------- | 20 | (14%) |
| Producing ---------- | 24 | (17%) |
| Unclassified -------- | 9 | ( 6%) |

C.    Statistics Breakdown

1.    28 (19%) blowouts resulted in a fire and/or explosion.

2.    9 (6%) blowouts were the result of drilling into a shallow gas hazard.

3.    2 (1.4%) blowouts were the result of drilling collision with existing (and producing) wellbores.

4.    3 (2%) blowouts resulted from ships colliding with the platform.

5.    9 (6%) blowouts resulted directly or indirectly from hurricane damage.

6.    There was a total of 72 fatalities attributable to these blowouts.

**Statistics from MMS 88-011**

7.   12 platforms were lost from blowouts.

8.   During the period 1964-1986 there was an average of 2,190 producing platforms in the Gulf of Mexico.

D.   Blowouts by Operator

| | | | | | |
|---|---|---|---|---|---|
| Conoco | - | 14 | Cities Service | - | 2 |
| Gulf | - | 12 | Champlin | - | 2 |
| Mobil | - | 11 | CNG | - | 2 |
| Shell | - | 11 | Kerr-McGee | - | 2 |
| Amoco | - | 9 | Marathon | - | 2 |
| Tenneco | - | 9 | Occidental | - | 2 |
| Chevron | - | 7 | Pan American | - | 2 |
| Pennzoil | - | 7 | Transco | - | 2 |
| Union | - | 7 | CAGC | - | 1 |
| Sun | - | 6 | Hall-Houston | - | 1 |
| Exxon | - | 5 | Phillips | - | 1 |
| Placid | - | 5 | Signal | - | 1 |
| Texaco | - | 5 | Sinclair | - | 1 |
| Mesa | - | 4 | Skelly | - | 1 |
| ODECO | - | 3 | Sonat | - | 1 |
| McMoRan | - | 3 | Texoma | - | 1 |
| ARCO | - | 2 | Trans Ocean | - | 1 |

II.   FIRES AND EXPLOSIONS

There were 49 fatalities and 300 injuries in 744 fires and explosions that were not the result of a blowout.

In 595 (79.9%) of the cases, there was either no damage or minor (less than $200,000) damage to either the platform or facilities.

A.   Welding

Welding-related activities were the primary cause of 125 (16.8%) of the reported fires and explosions. The vast majority of these accidents (approx. 80%) resulted from welding slag falling on flammable material, usually hydrocarbons in drip

**Statistics from MMS 88-011**

pans or on the surface of the Gulf. The remainder were caused by welding operations igniting fugitive hydrocarbon vapors or, in some cases, a welding operator inadvertently cutting into a vessel containing flammable materials, usually hydrocarbons.

B.    Glycol Systems

Glycol systems were either directly or indirectly responsible for 43 or 5.8% of the reported fires and explosions. These accidents can, for the most part, be characterized as resulting from glycol leaking or spilling on hot surface, usually the exhaust piping of an internal combustion engine. In a few cases, these accidents were the result of a ruptured firetube in a direct-fired glycol reboiler.

C.    Compressors

Compressors were involved in 140 (18.8%) of the fires and explosions. The vast majority of these accidents occurred as the result of the failure of a compressor gasket or valve allowing leakage of hydrocarbons which were subsequently ignited. In a few cases, the accidents were the result of using natural gas in the starter motors without proper venting. In a few cases, fires and explosions resulted when there was a catastrophic failure of a compressor head. These incidents occurred as the result of human error, safety system failure or both.

D.    Electrical

Electrical systems were the cause of 64 (8.6%) of the reported fires and explosions. In almost every case, these accidents were the result of a short circuit and the failure of a breaker to actuate. In a few cases, the accidents resulted from frayed wiring arcing and igniting a flammable material or from electric motors seizing, overheating and igniting.

E.    Lightning

In 14 (1.9%) of the reported cases, lightning was the cause of fires and explosions. Every one of these reported accidents was the result of lightning striking either an atmospheric vent pole or a tank vent.

F.    Equipment Failure

**Statistics from MMS 88-011**

Equipment failure, as used here, is defined as the catastrophic failure of a piece of equipment which immediately results in a fire and/or explosion. 106 (14.3%) such incidents were reported. These figures are exclusive of those equipment failures discussed in other statistics (i.e., compressors and electrical systems). Most of these failures fall into one of the following general categories:

1.  Failure of a pump head, usually on a positive displacement pump.

2.  Failure of a weld in a pressure piping system.

3.  Failure of a pressure vessel or heat exchanger or some appurtenance attached to the vessel or exchanger.

G.  Collisions

Of the reported fires and explosions, 5 (0.7%) resulted from a collision with the platform of a vessel or in one case a helicopter. In one case, a service vessel rammed and broke the gas sales line which resulted in a fire. In the other cases, the vessels themselves caught fire first and subsequently ignited the platform.

H.  Miscellaneous

There were 247 (33.2%) fires and explosions of a miscellaneous nature which do not logically fall into any of the other categories. Examples of these types of accidents are: small cooking fires in the gallery, members of the crew igniting flammable materials while smoking, use of unauthorized and flammable materials for cleaning, water heater failure, sump fires caused by static electricity, etc. Most of these fires were small and resulted in little or no damage, with the exception of the water heater failure which resulted in 3 fatalities and extensive damage to the quarters.

Statistics from MMS 88-011

# CONSIDERATIONS AND CONSEQUENCES OF PUBLIC POLICY

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*with contributions by*
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## Introduction and Abstract

Those responsible for safety of offshore structures confront three primary enigmas: (1) How safe is safe? (2) Who determines this criterion and how? and (3) What strategies facilitate a workable and legitimate partnership between the public and private sectors to achieve socially satisfactory outcomes?

Underpinning these questions are several realities:
- Offshore oil and gas resources are deemed by U.S. law and international treaty a common property resource with the federal and state governments having a fiduciary duty to prudently manage the resource as a public trust.
- Exploration for and development of these resources in the United States has been conducted by the private sector motivated by traditional market incentives.
- The extraction process introduces such externalities as hazards to life, property and the environment that are not always internalized by the operators.
- Risk mitigation must then be sought through public policy to supplement corporate policy on risk management.
- Acceptable levels of public risk are social judgments embedded in policy, unformed by media, by expert opinion and by political bargaining.
- Because of conflicts of parties at interest, efficient functioning to extract economic benefits with an acceptable level of risk requires a partnership between the public and private sectors.

## Context

Applying this perspective to offshore structures first requires illumination of twin elements of context— the physical risk environment and the socio-political stage on which questions of risk management are settled. The physical environment of surface wind, waves, currents and ice, and subsurface geologic structures imposes high uncertainties of loading, fatigue effects and aging. Although engineering and management practice accommodate these defining risk variables, there are other strenuous, ambiguous and often competing exogenous factors which enter the risk equation. For example, the market place for petroleum products dictates an economic imperative. Simultaneously, social expectations and value norms call for minimal risk to human life and to the natural environment. Harmonizing these requirements is not simple. Because public and private interests collide, professional practice is caught in the middle. All parties must thus define what constitutes acceptable risk. In other words, how safe is safe?

This question becomes even more perplexing because of two social dilemmas. First, human behavior is neither predictable as for machines nor in accord with familiar clockwork precepts of causality. Secondly, the public has lost confidence in its institutions and in expert opinion and increasingly demands risk management through government regulation. The outcome ultimately finds expression in public demands for accountability and legal liability, both of which conditions have vital economic and social consequences.

## Traditional Engineering Practice of Risk Management

Engineers and managers of technology typically reflect social responsibility of risk management through safety margins to hedge against uncertainty. This practice, however, is limited to hardware components of energy delivery systems. Moreover, safety has its economic costs. The practitioner is thus not free to adopt high margins arbitrarily, but must grind through the risk-cost trade-offs. The parameter of time also enters the calculus of risk, in that choices based on short run costs may overlook a powerful reality. While safety costs money, inadequate safety may cost even more.

The problem is further complicated by uncertainties in the field which are beyond the control of designers and managers. Extraordinary seismic, hydrodynamic or ice loading, deficiencies in construction and installation, human error in operation and in organizational behavior, and material aging all have potentially significant consequences. After such events or risk exposures, idealized structures may not be what they used to be. They are thus more vulnerable to failure even under standard conditions of service. Because risks may increase, we turn to their anticipatory management. This is one of the most basic attributes of public policy.

## Public Policy and Politics

Operational products of public policy are brewed from social needs or wants in a crucible of politics. Warfare erupts between interested parties, sometimes informed by history, sometimes inflamed by the media, sometimes driven by a highly stressed society inclined to sue for damages, but sometimes tamed by processes of bargaining. The outcomes depend both on the values held by different interests and on the skills of leadership in building consensus.

The messy and arcane system for making public policy amidst the hullabaloo of political decision making contrasts with the cold style of reasoned engineering design.

An appeal to rationality requires asking, "rational for whom?" Contributing to the heat of argument is the high diversity of viewpoints within the United States, exacerbated by a corrosive adversarial atmosphere, with threats looming of costly lawsuits. In turn, this overhanging threat can lead to uneconomical measures of defensive engineering and to administrative atmospheres which instruct personnel to deal with perceived errors by an unstated policy of "don't ask and don't tell."

This theatre may be an anathema for those who grew up with slide rules at their sides. Not surprisingly, technical managers shy away from examining political dynamics as though "politics" were a fundamental flaw in human affairs. As engineers study psychology and sociology, they recognize that all aspects of human interaction involve politics—at every scale from family, to work environment, to the nation, to the planet.

## Safety as a Social Judgment

It may be surprising, therefore, to learn that engineering design and policy design are remarkably similar. Both sets open with a statement of requirements. Both operate within certain constraints of natural law, administrative law, or economics. Both developments are nourished by factual information. Both reveal incompatible conditions in design which must be reconciled by the art of trade-offs. If these parallel qualities were more widely recognized, all parties would understand how the technology-intensive policy process in both government and industry is a fundamental part of the design of technological megasystems.

Most dramatically, engineering design and policy design converge with the fundamental question, "how safe is safe?" Adequate answers cannot be found in mathematical equations or observations of natural phenomenon alone. Almost universally, engineers deal unselfconsciously with danger by applying margins mandated by safety codes. Seldom are they moved to question why the numbers are four instead of three or five. Seldom is it explained that margins to assure proper functioning and safety are empirical, usually iterated from lessons taught by failures.

Over time, practice reveals what levels of risk the public will accept. The political system responds accordingly employing the art of compromise. Inevitably, so does corporate management and its response with internal policies for risk management are even more significant than the more visible policies set in the public domain. This distinction in scale of influence is drawn because corporate policies are implemented continuously while public policies can steer the system only in the wake of sporadic inspections or accident investigations.

Clearly, determining how safe is safe is a social judgment beyond the purview of expert knowledge. Moreover,

what is deemed safe at one time is subject to violent amendment in the aftermath of calamity. Indeed, the full range of impacts and impacted parties are often not known until a technological system is agitated by a serious accident. Then we witness with a sharper vision those elements which heretofore were obscured: which parties are the source of risk and which the recipients, including innocent bystanders who were oblivious of exposure. We conduct postmortems to establish cause and, most importantly, interpose measures of prevention. Acceptable risk extracted from public expressions in the political theatre ultimately is crowned by adoption of both public and corporate policy. No policy choice has the comfort of the product proving immutable.

Despite the inherently political nature of defining acceptable risk, it would be foolish to expect each source of risk to be subject to a public referendum on safety. Our system of governance depends on public policies being made through representative government. They are then interpreted and implemented by executive branch agencies through regulation by administrative rules.

This policing side of government inevitably triggers hostility by those regulated. Thus emerges a combative atmosphere between the public and the private sector that can block productive problem solving. On the other hand, the absence of stress may signal a pathology of one advocate completely dominating the process. Excessive tension can be similarly unproductive. Uncomfortable it may be, but we are obliged to accept the notion that every technology entails risks, and that their resolution inherently generates conflict.

## Redefining Technology as a Social Process

With that perspective, it becomes clear that both design and operation of offshore structures demand accommodation of parameters beyond the purely technical. Understanding the problem may be advanced by thinking about how we think about it. Since we are dealing with technology, consider defining technology as a social process. While every technology has a core of specialized knowledge, it is more than hardware, more than planes, trains and automobiles, or offshore structures.

Every technology encompasses software and socialware. As in computer lingo, the software can be visualized as the operating instructions for a specific technological system. These may be highly detailed for the hands-on function. But there are also other more subtle and more powerful instructions coursing through the system that emanate from industrial management at all levels. These may constitute the reward system of managers, or attitudes toward short term profit versus long-term. In addition, there are operating instructions from government in the form of regulations.

The socialware referred to earlier comprise institutions that write the operating instructions and monitor their employment. In the first instance, these are government regulators and industrial producers, but also significant are public interest groups.

The point is that "safety" is a product of both sets of operating instructions, a synthesized social process that reflects both commercial and public interest. Ideally, we can imagine a single set of instructions prepared by industrial management in which all of the untoward externalities of high social consequence have been integrated by the owners and operators. Regulation would then be unnecessary. Unfortunately, history teaches that while we honor the capitalist economic system which animates the hardware component of every technology, and while the overwhelming majority of these systems perform with acceptable risk, there have been enough cases of neglect or bald abuse as to warrant protection of the public by instruments of regulation. Incentives are too narrow for industry to internalize risks beyond the envelope of the firm.

Risk management becomes all the more intractable in fields subject to economic boom and bust, reflected dramatically in offshore platforms by the number being sold, moved and possibly poorly maintained.

## Evolving Public Attitudes Toward Risk Management

Trends of government regulation so conspicuous since 1970 can be explained both by the proliferation of technologically-triggered risks, and by lowered tolerance of the public to accept artificial risk as compared to risks injected by natural phenomena. This cultural shift contrasts public attitudes toward a beneficent technology. During World War II and for several decades later, the major question was, "Can we do it?" Around 1970, the question shifted to "Should we do it?" The thrust of this paper illuminates a new pair of questions: "Can we manage it?" and "Can we afford it?"

A sensible balance among public and private management interests has often been disabled by highly vocal groups that react to risk exposure on general principle without considered judgment on a case-by-case basis. Some enter the lists only from alienation or institutional self-interest. Moreover, examples abound where the estimates of risk by experts and by the public are in sharp disagreement with smoking and nuclear power as two prime examples. The problem lies not in ambiguities of data but in the lack of trust because of technical illiteracy, breeches of ethics, media excitation and because of the gulf in risk tolerance between perceptions of all parties of voluntary versus involuntary risk. The act of regulation simply reflects the fact that economic incentives, harsh as they are, are not alone sufficient.

All too often, the act of regulation carries a burden of hostility to government. For years, industry has marched to a slogan of "get government off our backs." The paradox is that many of those complaining were beneficiaries of governmental largesse. Indeed, from our national origins two centuries ago, government has responded to requests of the private sector for subsidies, tax write-offs, even total bailouts. The national interest requires a viable economy through private enterprise.

Closer examination reveals government involved in technology in yet six other ways: defraying social overhead by funding research, development and technical education; fostering innovation as a customer of defense-related high-technology; and funding major projects such as highways and dams which are beyond private purse or risk acceptance. Most important in the current context are the roles of government in managing the economy by tax and fiscal policies, and as steward of the public trust, those common property resources such as offshore oil and gas. Then, of course, there is the role of regulation.

A deeper comprehension of the symbiosis between government and technology could suppress hostility toward government regulation by those who believe that the invisible hand of the market place operates for overall social benefit so that the best government is the least government. For many decades, analysts have confirmed that socially efficient utilization of oil and gas resources will not follow from the calculus of the free market. The literature is full of examples of how this industry distrusted the free market and resorted to monopolistic practices. Policy making, therefore, requires a harmony of public and private interests to which long-standing prejudices can get in the way.

## Policy for Risk Management

The design of public policy, like engineering artifacts, must begin with facts. So, then, must risk management policy. An opening wedge in the policy process is for all sides to seek a data base of casualties on which they can substantially agree. This is not as straightforward as it would appear because the presence or even threat of lawsuit chills good intentions of sharing information. Nevertheless, without basic information on platform performance, policy is likely to evolve on the basis of negotiated self interest where the appearance of compromise is vulnerable to subsequent revelations. A second principle in policy design is to face a harsh truth that safety costs money, at least in the short run. In the long run, accident prevention may prove the most economic strategy. Public policies must take account of private costs, but the debate between public and private interests should be moved from the tempting and familiar stance of short run costs to examine the longer run profit. Moreover, failures now provide a baseline of total costs from harm to people and to the environment. This proposition, incidentally, to illuminate the longer-term dimensions of the safety/cost trade-off is not ideological. Policies always bridge the present and future because of the long time elapsed between problem identification that spawns policy, its design, political bargaining, and enactment, to implementation for goal achievement. The National Environmental Policy Act of 1969, NEPA, had its roots exposed in 1962 and was resolved in an extraordinarily short time because of unusually strong public sentiment that drove the political campaigns of 1968. A major cultural shift occurred. Much later in 1993, legal repercussions still require policy refinement. It is impossible to deal with policy as though it were a short-run, economic phenomenon.

That National Environmental Policy Act which engendered so much subsequent antagonism contained a number of principles which have proved durable in underpinning policy for risk reduction. These include:
- The role of facts about risk: what we know and what we don't know but should.
- Generation of alternatives which embody lower risk.
- Identification of stakeholders.
- Estimate of consequences of each alternative by asking "What, if?", "When?" and "To Whom?"
- Opportunity for input from all stakeholders, BEFORE action.
- Identification of trade-offs which are present in all options.

In addition to these steps, the social history of the issue is essential to generate a context and perspective for

effective generation of policy.

In short, five streams of information are required: Goals; a map of the technological delivery system, its institutional components and their interlacing; existing operating instructions; data on past performance; and identification of options and their estimated impacts on different stakeholders.

What confounds analysis of offshore platform risks is the low frequency of casualties. Those situations characterized by low frequency but high impact do not yield readily to methodologies of probability nor, frequently, the public process. Since there may be few failures in similar structures to learn from, the literature should be consulted on all classes of technological systems where risks are high and prophylactic action taken. Indeed, a body of such information is growing from accidents and heightened risk management in nuclear power generation, commercial air transport, chemical processing, and military operations such as nuclear weapons management. From such studies comes insight, particularly on the role of human and organizational factors.

One discovery is the lack of attention by watchdogs expected to protect the public. The saga of Prince William Sound is a perfect example, with complacency and neglect discovered at numerous elements of the technological delivery system. Thus we find the necessity of having public bodies oversee the watchdogs to confirm that they function as intended.

Realities of risk management confirm that public policies are made by political leadership with the assistance of experts and significant inputs of vested interests that are intimately involved. Overlooked is the fact that policy makers are elected and the policy is thus the ultimate authority on risk acceptance. The problem is taking the public pulse. As said before, we cannot depend on referenda. Media polls have their own biases, and sounding out public opinion in an uninformed public can be seriously misleading.

To understand the process involved, consider the medical analogy of "informed consent." The challenge begins with providing information to all parties, information that is comprehensible to the laity, timely, authentic, with emphasis on what contending parties agree on and then what they disagree about. Different interpretations should be publicly available so as to illuminate the inevitable trade-offs between risk and benefits. Unfortunately, the media are not primarily dedicated to education; the public rewards their focus largely on entertainment. Photogenic catastrophes thus become springboards to inform the public, but seldom is public opinion thus generated fully or objectively informed.

## Techniques of Policy Design

A major issue naturally evolves on a process to consult the public in policy design. Basic principles flow from the perspective of technology assessment that was imbedded in NEPA:

- Institute quality assured, risk management through auspices of an independent facilitator.
- Involve all potentially impacted interest groups, owners and operators, federal, state and county regulatory bodies, insurance underwriters, representatives of potentially impacted marine industries and the general public.
- Map the socio-technical system that is involved so as to reveal all of the institutional players and their relationships.
- Map the hardware that contributes to risk, including platform jacket and topside equipment, subsurface safety valves, and related elements, including their history of exposure to extreme conditions and details of past inspections and repairs.
- Inventory the protocols under which the system operates, including the potentially conflicting or overlapping requirements of separate regulators.
- Identify those elements that experience has proved to be especially powerful or frequent sources of concern or accident.
- Develop a comprehensive data base of information from past inspections or accident investigations, including findings from other classes of socio-technical systems and results from computer modeling.
- Generate risk mitigation measures.
- Conduct an impact analysis to ascertain what might happen if each of these were to be introduced, the benefit and the costs, segregated as to impacted part.
- Seek consensus on action to be imbedded in future policy.
- Specify who is responsible to implement such a policy and how the process can be judged so as to entertain

future refinement.

- Publicize the process from its inception and follow through to facilitate healthy public understanding.

The methodology of what might be called risk management technology is still in a primitive stage. Much research and development is required to resolve the challenges involved in defining and realistically evaluating uncertainties in offshore platform performance. Simplified criteria to trigger inspections should be adopted with great caution. Fortunately, there is an enormous body of information available from environmental impact analysis that can be applied to the policy process. We can then learn from two types of failure, the structural and the political.

Success ultimately depends on good faith by all parties. Even with the vagaries of acceptable risk, a high level of trust of all parties, in all parties, is a *sine qua non* of achieving deployment of risk-encumbered technology, with socially satisfactory outcomes.

## Acknowledgments

Discussions and Summaries

... the use of scales, ... by ... adversarial atmosphere, ...
... In turn this overhang ... log ... one ... measures of defense ...
... adversarial atmosphere which instead persuades to deal with perceived errors by an attitude of the "... the risk and don't tell"

This theme may be an anathema for those who grew up with slide rules at their sides. Not surprisingly ...
... the areas from examining political dynamics as though "politics" were a fundamental flaw in human ...
... they recognize that ... human interaction ...

## Safety as a Social Judgment

... remarkably similar ...
... natural law ...
... public ...

... fundamental question ...

# Discussion and Summary on

# INSPECTIONS, SURVEYS AND DATA MANAGEMENT

Neal Hennegan, *Shell Offshore Inc.*
Wade Abadie, *Oceaneering Solus Schall*
Larry Goldberg, *Sea Test Services*
Walter Winkworth, *Lloyd's Register of Shipping*
***Working Group #1***

## Major Topics Discussed

Best Practices
Lack of Consensus Issues
Research and Development Needs

## Best Practices

- Planning
    - Design factors (criticality, robustness, QA/QC)
    - Experience
    - Prioritization of inspection points (criticality, likelihood of damage)
    - Environmental factors (e.g., soil, visibility, current)
    - Commercial factors

- High Benefit To Cost Ratio Inspection Methods
    - Topside structural inspections
    - Underwater visual inspections (ABDF)
    - CP measurements
    - FMD (through wall damage, unfolded members)
    - Close visual inspection of member ends/critical points
    - MPI
    - Photographs, video

- Inspection influenced by consequence, redundancy, past experience

- Random MPI not cost effective

- Random limited scale cleaning effective for assessing corrosion damage

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*\* Editor's note: The results of the Working Group discussions were embedded in a revised version of the White Paper. These are the highlights of the discussion.*

## Best Practices

- If looking for incipient cracks, water blast only plus MPI

- Crack experience
  - Relatively rare
  - Generally in top 100', around conductor guides, toe of weld

- UT not suited to in-service detection of fatigue cracks

- Qualified/certified inspection personnel
  - Similar to bridge practice
  - Will require some training and testing
  - ADC, ABS, Lloyd's, DnV, etc. should develop/share program

- Data Management/Reports
  - Relational databases
  - Offshore data entry
  - Damage inventory files

## Consensus Issues

- Right mix of inspection techniques
  - Best way to find random damage
  - Lack of industry hard data and knowledge

- Need for verification of inspection program (i.e.. is third party required?)

- API requirements for surveys
  - Difference between operators
  - Recommend commentary section for inspection

# Research and Development Needs

- Technology transfer
  - Information between geographical regions (ECM, MPI on black oxide, etc.)

- Remotely Operated Vehicles
  - Enhanced capabilities in areas of cleaning, UT, NDT
  - Operational reliability

- Remote sensing
  - Acoustic emissions
  - Leak before break detectors
  - Durability of instrumentation systems

- Data Management
  - Real time data entry
  - Graphical representation

# Discussion and Summary on

# ENVIRONMENTAL CONDITIONS AND FORCES

Ove T. Gudmestad, *Statoil*
John C. Heideman, *Exxon*
Charles Petrauskas, *Chevron*
*Working Group #2*

## Introduction

A White Paper on "Environmental Conditions and Forces for Use in Requalification of Offshore Structures" was presented at the workshop. The present paper summarizes the discussions in the Workshop's Working Group #2 on this topic.

Working Group #2 gave in general a strong endorsement of the draft "API RP2A-WSD, Section 17, Assessment of Existing Platforms" dated November 3, 1993. In this paper, therefore, answers to the different questions raised in the White Paper will be grouped into main categories while reference will be made to the API Section 17 document for more detailed questions.

Further endorsement was given to the idea that the API Section 17 document represents a good background for an ISO standard regarding assessment of existing platforms. In view of the ongoing work to produce an ISO standard for design of offshore structures based on the new API RP2A-LRFD version of July 1, 1993, the relevant ISO format for a section on assessment of existing platforms would, however, be the LRFD format (Load and Resistance Factor Design) rather than the WSD format (Working Stress Design).

The LRFD format involves use of load and resistance factors, which have to be calibrated to the particular environmental conditions encountered in the area where the LRFD code is to be used. It should, furthermore, be realized that the authorities and the operators should cooperate to determine the required reliability level for the new as well as for existing platforms considered for requalification. As lives of personnel and a clean environment are considered to have equal value around the world, ideally the same reliability level should apply. In some areas, such as in the Arctic, however, the impact of pollution may be higher than in other areas; for these areas a higher reliability may be considered by the operators.

It should be noted that there are inherent assumptions in the API Section 17 document and that steps must be taken to ensure that these assumptions are met:

* With reference to safety for personnel, it is assumed that evacuation of platforms takes place in the case of a severe hurricane. This will require that a reliable hurricane warning system and contingency plan be kept operational.

* Related to the environmental issue is the assumption that subsurface safety valves will function when there are such needs. During Hurricane Andrew, all of the valves which had to be closed worked as planned. The members of Working Group #2 base their recommendations on the assumption that the subsurface safety valves must have a documented high reliability level.

# Best Practices Related to Determination of Environmental Conditions and Forces for Use in Requalification of Offshore Structures.

## General Issues (White Paper Issues #1, 3, 4, 5, 12, 13, 23, 6, and 22)

The philosophy that the environmental criteria can depend on the consequences of a platform failure was, in general, accepted as logical. As both authorities and operators are concerned about the consequences of human lives and the environment, priority should be given to these areas, while the economic decisions should be left with the operators of the platforms. In some instances, however, production of certain fields could be so important for the government of a country that it could be expected that the authorities would like to set requirements for safe production during a storm.

While new offshore platforms have to be designed in accordance with the latest rules and regulations, it was documented during Hurricane Andrew that no platforms designed according to API RP2A 9th edition or later versions of API RP2A collapsed. Although this hurricane struck a limited sample of platforms in the Gulf of Mexico, API RP2A 9th edition should represent a suitable reference level for requalification such that old platforms do not have to meet the requirements of new ones. In this respect it should, however, be noted that the API Section 17 document recommends RSR's (Reserve Strength Ratios) only for U.S. waters. For other areas, the RSR's must be determined for the actual environmental conditions encountered and the reliability levels selected.

Related to the discussion on whether requalification should be based on set periods or on technical triggers, as listed in the API Section 17 document, the members of Working Group #2 felt strongly that requalification at set periods could endanger the safety of production units as the operators in the periods in-between requalification could be tempted to limit inspection and maintenance to under-critical levels.

The requalification recipe as presented in the API RP2A Section 17 document represents an enclosed format where attempt is made to consider all aspects related to environmental conditions and forces. Implementation of new theoretical information or calculation methods could therefore be in contradiction with the contents of the rest of the procedure and should only be done by the API committee in view of the full procedure. New information on environmental data could, however, more readily be implemented, as such new information will be based on actual measurements from the locations where the data will be utilized.

In order to maintain a consistent risk level for personnel involved in the operation of platforms to be requalified, the criteria for requalification should not depend upon the length of the extension period sought for the platform under requalification. Most of the participants of Working Group #2 agreed that this philosophy should also apply to environmental risk. With respect to economic risk, different operators may, however, decide on a more stringent risk level for platforms considered for a long extension period compared to platforms requalified for a shorter period.

## Environmental Statistics (White Paper Issues #7, 8, 10, 9, 14, and 15)

The environmental criteria for requalification should be based on the best available statistics for the area where the platform under requalification is situated. To utilize more conservative wave statistics, e.g., based on basin-wide extreme waves, would give unrealistically high load levels, which would not be consistent with the idea of combining best possible (most correct) environmental statistics with a consistent recipe for calculation of environmental forces for the reliability level selected. An operator with a large inventory of platforms may, however, view the economic risk for losing the entire population of platforms to be sufficiently high to select a higher reliability level for the platforms he operates.

The discussion on the use of joint probability for waves and current (and wind) has been ongoing for many years. Where reliable data exist, the documented joint probability functions could be used. It is in this context implicitly assumed that a realistic and consistent force calculation procedure will be applied. The API Section 17 document recipe satisfies these requirements.

The selection of the appropriate Reserve Strength Ratios (see previous section) for the area under consideration will depend upon the environmental statistics of the area. Of particular concern in this respect is the rate at which the wave height increases as a function of return period. This should be used as one of the main factors in the selection of the appropriate RSR's. The design criteria should, furthermore, be based on force statistics, rather than on wave height statistics as the degree of nonlinearity in forcing will vary for the different platforms depending upon the relative contribution of drag versus inertia loading.

*Force Calculation and Methodology (White Paper Issues #16, 20, 17, 11, 18, 19, and 21)*

The reserve strength ratios recommended for requalification purposes are calibrated with respect to the force recipe of API RP2A 20th edition. It is therefore important that the 20th edition wave force calculation procedure be used for requalification purposes to maintain the consistency implicit in the API Section 17 document.

It should be noted that the drag coefficient to be used in the Morison equation for calculation of loading to a very large degree depends upon whether the platform member is roughened or not. The diameter of the member will, furthermore, depend upon the thickness of the marine growth attracting higher loading in the platform. In this respect it could be considered to remove marine growth to obtain a smooth surface and to reduce member diameter. Present cleaning technology does not, however, seem to provide a smooth surface over a substantially long period unless there is a very frequent, extensive cleaning program. Use of anti-marine-growth painting or coating could, however, be considered although the costs could be considerable.

The requalification procedure described in the API Section 17 document is calibrated against traditional four or six legged jacket structures. Until further calibration assessments are carried out for tripods and well caissons, etc., the procedure must be used with caution for special types of structures. Of main concern in this respect are dynamics of very slender structures, the effect of transverse loading and the potentially low redundancy.

During requalificaiton of offshore structures, the wind load on the structures and the topsides should be taken into account properly. The possibility for gust or suction effects leading to damage of topside equipment should in particular be considered.

# Research Areas Related to Requalification of Offshore Production Structures

Within the area of environmental conditions and forces on offshore production structures, the following research areas are in particular considered relevant with respect to requalifications of these structures:

- In both shallow and deep waters the statistics of crest heights (i.e., the asymmetry of the waves) are considered of large importance to determine the appropriate air gap of the topside equipment. Note that the wave loading could become very large if the wave crests hit the topsides.
- Since the qualification criteria for the Gulf of Mexico are dependent on platform survival statistics in Hurricane Andrew there is a need to reevaluate the hindcast of Andrew in view of measured data that were not yet available when the first hindcast was done.
- The forcing dynamic sensitive structures should be evaluated with particular attention to nonlinear transient loading effects ("springing" and "ringing").
- The utilization of the relative velocity formalism has been widely discussed over the years for dynamically sensitive structures. Further assessment of this formalism is recommended in view of the need for a careful review of all aspects producing hydrodynamic damping.
- The largest contribution to the drag loading on an offshore platform comes from the crest of the wave. In view of the squaring of the water particle velocity in calculating the loading in accordance with the Morison equation, there is a need for further research in order to predict accurately wave crest kinematics in irregular seas.
- During all laboratory experiments and offshore measurement programs there has been a large scatter in predicted versus measured wave force. There is a particular need to understand the background for this scatter and to review its effects in reliability analysis of offshore production structures.
- The measured wave peaks in a storm will not necessarily coincide with the hindcasted storm wave peaks based on metocean information collected during the storm. There is a need for further research to understand the scatter in hindcast versus measured storm wave peaks and to review its effects in reliability analysis of offshore production structures.

## Need for Interdisciplinary Collaboration

The participants of Working Group #2 called for collaboration with the other disciplines. Of particular concern was cooperation with the operations discipline as to confirmation of assumptions related to weather warning to accommodate evacuation and the reliability of subsurface safety valves.

Furthermore, Working Group #2 encourages the policy makers to adopt a uniform risk for personnel and environment worldwide. The work on an ISO standard for requalification of offshore production structures is thought to enhance this recommendation. The draft of API RP2A Section 17 will, in this respect, provide an excellent basis for a further international standard.

## References

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# Discussion and Summary on

# STRUCTURAL ELEMENTS, SYSTEMS AND ANALYSES

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*Working Group #3*

## Executive Summary

### Scope

Working Group #3 addressed the structural and related reliability issues that must be considered in the reassessment or requalification of offshore production structures. All topics covered were considered for their suitability for international use.

### Activities of Work Group

- Addressed API RP-2A Section 17.0
- Reviewed Andrew JIP
- Reviewed Testing and Analysis Case Studies
- Reviewed Active and Proposed Research Programs
- Discussed Technical Issues

### Overview of Work Group Conclusions

There was consensus that the general API approach being proposed for reassessment of existing platforms for the Gulf of Mexico would provide an appropriate framework for worldwide application. However, the consequences for failure are inherently more likely to be greater in areas like the North Sea than in the Gulf of Mexico.

Structures may be accepted as fit-for-purpose at less than design capacity provided there is adequate reliability to serve their intended purpose and/or the consequences of failure are acceptable. This practice is followed for other land-based structures; reduction standards have been established for earthquake resistant structures, bridges, buildings, dams, etc.

The main thrust of the assessment process from a structural perspective is a progressive level of assessment complexity that could require a calculation of the ultimate system resistance. With the resistance convolved with the applied loads the reliability of the structure can be estimated. There was not consensus on whether these calculated reliabilities could be regarded as "actuarial."

Sophistication of analysis has had a profound effect on the ability to estimate the structural behavior of a platform under extreme loads. Computer programs have been developed that are capable of considering nonlinear behavior of members and joints in assessing the system behavior of complex structures. Most of these programs are proprietary, but are available for contracted services of the program developer or for licensed use.

There is a modest database available to provide some of the nonlinear characteristics needed to carry out the computer modeling and analysis. There is, however, a substantial need to gather further information and calibrate the computer programs' ability to estimate a platforms ultimate failure behavior. In some cases, the data is readily available, but has yet to be processed with an application to reassessment in mind. For example, ultimate strength of a joint is not sufficient; the nonlinear loading and unloading characteristics are very important to the redistribution of loads within the structure.

# Working Group Discussion of Issues

*Is the API approach for requalification of platforms in U.S. waters appropriate for other geographic areas? What significant change would need to be made? (Issue #1)*

### Discussion

The API approach is based on pragmatic acceptance criteria that permits requalification to conditions less than normally required for new design. The amount of reduction depends on the platform design basis and condition, as well as the consequences to personnel and the environment.

Yes, the API approach is appropriate for other geographic areas. The provisions, however, should be tailored for the specific area. In the case of Norway, there is no allowed reduction from design requirements unless the installation is unmanned. It was noted that in the North Sea, there are no low consequence structures; they usually have a large complement of personnel and substantial production.

In the case of bridges, a reduction of about 30% in design strength in the U.S. is permitted for requalification. There was some discussion of how to choose probabilistic targets, where they were deemed more appropriate for reassessment.

### Research Needed

Other areas needing consideration: Africa, Borneo, China, Thailand

### Unresolved Issues

If probabilistic reassessment is used instead of the API approach, there was no unanimity on how the safety targets should be set. Is an under-strengthened structure considered of low consequence, if it is bridged to a safe haven structure for personnel?

*Are both deterministic and probability based approaches suitable for requalification? Is there adequate technology to apply both approaches? What qualifications, if any, should be placed on each approach in so far as interpreting results are concerned? (Issue #2)*

### Discussion

Yes, both deterministic and probability based approaches are suitable for requalification. Deterministic is easier. The probabilistic approach should be considered an optional "higher level" approach. The probabilistic approach is not a panacea.

### Research Needed

There needs to be better standardization of approaches in order to achieve more consistent outcomes. In the case of the probabilistic approach, there needs to be training in the application of the methods by the analysts and interpretation of the outcomes by the decision maker.

*Are there suitable guidelines and standards to assure adequate information is archived during design, construction and installation? If not, what guidelines and standards are needed? (Issue #3)*

### Discussion

Time limitations prevented this issue from being discussed.

*Should assessment practices adopt the same relative safety factors among the different components as with new design or should all component reserves be the same? (Issue #4)*

### Discussion

No, it is not necessarily desirable to have all structural components achieve the same target reliability. Among the factors that enter into accepting different levels are the incremental cost to achieve improved safety and the statistical

characteristics of the supporting test data (mean or lower bound). It was noted that AISC depends on "lower bound" data rather than "mean" data.

### Research Needed

There is a substantial difference in the application of factored design approaches. In some cases (concrete design), the factors are applied to individual parameters that make up the load and resistance formulations, and in others (API-LRFD) a single factor is applied to the load and resistance formulations. This difference has a significant effect on how experimental results are interpreted. Basic data needs to be reviewed and this difference resolved. Presently, the draft for the new ISO standard for offshore structures allows both approaches.

*When approximating damage in a global analysis, is it possible that analytically approximating component stiffness or removing a component from the system could lead to non-conservative results? If so, how serious can the error be and what steps can be taken to avoid such errors or limit their consequences? (Issue #5)*

### Discussion

It was pointed out that joint failure could "protect" the integrity of its member. It was noted, however, that joint failure often results in serious damage to the host chord member, which may experience a significant reduction in strength. If analysis shows that removal of a member increases the system strength, then consideration should be given to removing the member.

### Unresolved

There was no agreement on the seriousness of these non-conservative outcomes.

*Are there adequate safeguards in proposed assessment and requalification processes to guard against undesirable interactions among traditionally separate design disciplines? Is it possible that, within a given discipline, integration of case specific test results into an interaction formulation could lead to non-conservative outcomes? What steps should be taken to guard against hidden introduction of non-conservative bias in the process of stripping away conservatisms?*
*Can dynamic measurements be used to calibrate the response to severe environments? What limitations on sea states would be needed? (Issue #6)*

### Discussion

There was general agreement that such dangers do exist and that diligence needs to be practiced to avoid such unfavorable outcomes. When practices are changed in one area, their effect should be addressed in other areas. This is especially relevant in requalification, where reduced levels of acceptance are tolerated. There was no discussion of the dynamics issue.

*Is the use of mill-certified coupon test results appropriate for assessment? If so, are there any limitations or consid-erations that should be included in their use? In the case of various conventional steels for which coupon test results are not available, is it okay to use recognized material norms, e.g., 43 ksi for A36 steels? (Issue #7)*

### Discussion

Time limitations prevented this issue from being discussed.

*Is it appropriate or possible to have both limit state and working stress methods used in assessment? How should test data be incorporated in each case, and can the results be calibrated to result in the same outcome? How should "safety" factors be established in each case? (Issue #8)*

### Discussion

It is possible to use alternative methods; however, it is important to be consistent.

### Research Needed

At the present time, the methods of interpreting test data are not consistent among researchers and analysts. In some cases the data populations are extremely sparse and substantial probabilistic interpretation is unjustified. The questions posed in this issue need further consideration to develop an appropriate requalification guideline.

### *Should guideline methods of modeling and evaluating finite element analyses be established? What guidelines would be appropriate? (Issue #8)*

### Discussion

Yes, guidelines for nonlinear modeling and finite element analysis are needed. Examples cited were:
- How to define the Stress Concentration Factor.
- How to define nonlinear capacity.
- Where to place strain gages in component testing.

This should be part of the structure bench marking work identified in Issue #13.

### Research Needed

Compile existing techniques.
Compile existing data.
More work needs to address complex structures, e.g., multiplanar joints.

### *Are there adequate data to specify assessment procedures that account for component ductility, denting, cracking or low cycle fatigue? If not, where should more research be done? (Issue #10)*

### Discussion

There is not enough nonlinear fracture criteria. There is not enough large deformation data available for component data. Testing is often limited by the testing equipment actuating mechanism.

Discussion digressed to the problem of who should coordinate and sponsor research work in general. Regulatory bodies and industry have carried a fair share of the research funding. There is, however, a steady shrinking of industry and government financial resources. Code writing bodies, like ISO, have no resources to fund code development; they depend on national standards associations to fund their individual activities.

Joint Industry Projects have been a primary source of funding for large projects. There are two problems with JIPs: sponsors are reluctant to allow free use of the work that just a few companies supported, and there are fewer companies willing to sponsor the work; their cost burden therefore is larger for a given piece of work.

### Research Needed

Work on component ductility, denting, cracking and low cycle fatigue are in various stages of development at Lehigh University, Sintef, TWI, and possibly Korea. More work is needed on multiplanar joints.

### *Is the state-of-the-art for fracture mechanics suitable to be used in assessment recommendations or standards? If not, what needs to be done to develop the know-how? (Issue #11)*

### Discussion

Time limitations prevented this issue from being discussed. The fracture mechanics issue was briefly mentioned in the discussion of Issue 10. It was generally felt that fracture mechanics considerations are important to the assessment process, since some level of cracking is expected and some limits need to be established to determine if cracking is sufficient to warrant repair.

### *Is there a commonly accepted definition for each of these performance measures? If so, should there be numerical targets associated with them in requalification recommendations? If there are not accepted definitions, who should define them or how should they be developed? (Issue #12)*

## Discussion

This topic refers to the large number of definitions for *ductility, redundancy, robustness, reserve strength,* and *toughness.* There was not time to discuss this topic, though it was evident during the discussion that some terms still do not have universal understanding.

*How accurately can nonlinear calculations for complex structural systems be expected to model true structural behavior? What levels of calculation precision can be expected? What additional research is needed to validate and benchmark these methods and computer programs? Should a standard benchmark structural model be developed to validate programs and component modeling techniques?*
*Are there appropriate means to infer system probabilistic behavior from the uncertainties associated with components of complex, nonlinear systems? If so, what means should be adopted? (Issue #13)*

## Discussion

An HSE study is in progress to address the issue of consistency among nonlinear structural analysis software. The study encompasses *members, joints* and *foundation* considerations. Though the study is not complete, it has disclosed that *member* analysis consistency is better than for *joint* analysis.

There was general agreement that a standard benchmark structural model would be desirable to validate existing and developing software.

Several software packages are capable of inferring system probabilistic behavior from uncertainties associated with components. There was no discussion of the appropriate methodology.

## Research Needed

A benchmark structural model is needed that exercises the most important nonlinear characteristics of structural behavior. Evaluate the applicability of simplified methods and detailed FE analysis. Supporters of FE analysis claim that the methods are becoming so simple to apply that there is no need for approximate methods. Testing is needed to validate analytically developed post-buckling behavior.

*Is it possible to characterize the behavior of systems by reference to analyses of common structural systems? Is it possible to develop a criterion for establishing loading scenarios, based on these analyses, for which structures may require assessment? (Issue #14)*

## Discussion

It may be possible for an owner of a large number of platforms to develop a screening process that would be self consistent, but such a process may not carry over to a different owner who may have a different data base of information from which to work.

*Is assessment (or design) of structural components loaded primarily by environmental loads needed for operational environmental conditions? (Issue #15)*

## Discussion

Because some structures are not dominated by environmental loading, there should be an operational check provided for such structures. Such checks would be appropriate for decks and structures in very benign areas.

Discussion did not identify how the operational environment would be selected.

## Research Needed

A means of calibrating to the operational environmental condition needs to be established, so that the operating environmental condition can be defined. Heretofore it has been entirely left to the operator, since it has had no effect on the safety of the structure; e.g., if the structure can survive the extreme event, then it can satisfy the operating environment case. The translation of Section 17 of the RP2A-WSD into the RP2A-LRFD will have to address this issue.

*What criteria should be used to rank the importance of components in extreme or accidental loading events? Level of overload? Alternative load paths? Strength utilization? (Issue #16)*

### Discussion

It was questioned whether accidental loading events was a reassessment issue. The United Kingdom, Norway and Australia felt that it was. (Safety cases may be required in Australia next year.) Early designs might not have been designed for accidental loads that may occur subsequent to requalification.

It was unresolved whether future accidental loading cases should be part of requalification.

*Is it practical to establish target reliabilities for requalification acceptance criteria? Can calculated probabilities be expected to represent actuarial statistics? If calculated probabilities cannot be used for acceptance criteria, what is the best alternative measure of acceptability? Should there be a simple quantified measure?*
*Should "target reliability levels" be modified to account for remaining length of life? (Note that the API approach does not address this point explicitly. The API approach adjusts acceptance criteria based on consequences.) (Issue #17)*

### Discussion

It is practical to establish target reliability levels. The reliability levels may be in terms of probabilities or acceptable performance standards. Some reassessment reference level is needed.

Calculated reliabilities should still be considered "notional," however, the calculated values are becoming more defensible in an actuarial sense, but we are not there yet.

The issue of including the remaining life of the structure in setting acceptance criteria hinges on the condition of the platform, the condition of the cathodic protection system, the level of the achieved strength in relation to the target level. It was not clear how this question is answered for onshore structures whose failures are not dominated by environmental loading.

Unresolved issue-when a structure is requalified, what is the duration of the requalification period? Assuming none of the original reassessment triggers exist, when or should the structure require future requalification?

*Mitigation measures have varying degrees of effectiveness and reliability. Which measures are relatively ineffective and which need substantial development work to be reliable? (Issue #18)*

### Discussion

Time limitations prevented this issue from being discussed.

*What amount of structure archiving can be automated? Is it practical to establish standards for archiving based on emerging storage devices? Is there existing software that can be used to put all the information on a single ROM, including photographs, etc.? (Issue #19)*

### Discussion

Time limitations prevented this issue from being discussed.

# Appendix I

During the Working Group sessions, several individuals presented information regarding ongoing or planned research falling within the purview of the work group topic. In some cases, handouts were given to the participants. It is not appropriate to reproduce these handouts in the proceedings, however, a brief synopsis of the topic is given below along with a point of contact.

### Corrosion Damage-Effect on Strength
The effect of "patch" and "overall" corrosion damage on the strength of tubular members is being studied. Particular and immediate emphasis will be on the effect of localized "patch" corrosion which produces reduction in thickness in a relatively small area and thus invites formation of local buckles which precipitate the ultimate load. (A. Ostapenko, Department of Civil Engineering, Fritz Engineering Lab, Lehigh University, Bethlehem, PA 18015)

### Corrosion Damage-Assessment in Field
This study involves a survey of existing and emerging nondestructive evaluation (NDE) techniques, and an evaluation of their applicability to the problem of measuring *in situ* the loss of section due to corrosion in offshore structures. (S.P. Pessiki, Department of Civil Engineering, Fritz Engineering Lab, Lehigh University, Bethlehem, PA 18015)

### Repair of Columns-Whole Column Approach
The experimental phase will involve axial testing of long columns with dents. Some unrepaired columns will be tested first to establish a reference, followed by testing of columns repaired using internal grouting and perhaps a repair using a grouted sleeve. Analytical work will involve assessing the effects of grout repair on member overall behavior. Part of this work involves the development of a finite element model for analyzing grout-repaired dented columns. (J.M. Ricles, Department of Civil Engineering, Fritz Engineering Lab, Lehigh University, Bethlehem, PA 18015)

### Repair of Columns-Segment Approach
The objective of this work is to develop a reliable moment-thrust-curvature relationship for short dented grout-repaired column segments. Such a relationship would then be used in integration-type methods (analogous to DENTA, BCDENT, etc.) for analyzing the pre- and post-ultimate axial behavior of dented grout-repaired long columns. (A. Ostapenko, Department of Civil Engineering, Fritz Engineering Lab, Lehigh University, Bethlehem, PA 18015)

### Static Strength of Cracked Tubular Joints
This project will develop a methodology for assessing the static strength of cracked tubular joints, building on existing knowledge and addressing issues identified in a definition study. The objectives are to develop guidance on the static strength of cracked tubular joints and to provide recommendations for in-service assessment of cracks using a failure assessment diagram based approach. The work involves finite element modeling of a range of cracked geometries and fully instrumented static tests on cracked large scale joints. The work will cover joint geometries, crack sizes and loading modes shown to be of concern in the project definition study. (G.S. Booth, TWI, Abington Hall, Abington, Cambridge CB16AL United Kingdom)

### Requalification of Offshore Production Structures-Synthesis of Some European Initiatives
This paper was presented after the workshop as a response to questions raised in the Working Group #3 discussions and to address some of the fundamental issues raised in the workshop charge by Bob Bea. It presents a synthesis of some of the European initiatives in developing a rational approach to reassessment and requalification of offshore production structures, with emphasis on probabilistic approach to decision making and assessment of structural reliability. The paper discusses the following main topics: what should be requalified and when; what should be the requalification standards; framework for requalification decisions; assessment of environmental hazards; assessment for operational hazards; and issues for consideration. The overall approach was based on a number of previous and ongoing collaborative research efforts in the European community. (N.K. Shetty or J.T. Gierlinski, W.S Atkins Consultants Ltd. Woodcote Grove Ashley Road, Epsom, Surrey KT18 5BW, United Kingdom)

### Reassessment, Strengthening, Modification and Repair of Steel Offshore Structures

This paper discusses the state-of-the-art in structural reassessment techniques and strengthening/repair options for existing installations. It is demonstrated that adequate and advanced structural engineering techniques are available to economically and safely upgrade or strengthen/repair existing offshore platforms. The paper discusses structural assessment techniques, strengthening and repair techniques and intervention methods. Although the techniques are discussed with reference to jacket structures, the writers claim the methods described are equally applicable to topside structures, from a conceptual standpoint. (M. Lalani or N. Sondhi, MSL Engineering Ltd., T.C.C. Silwood Park, Buckhurst Rd., Ascot, England)

# Discussion and Summary on

# FOUNDATION ELEMENTS, SYSTEMS AND ANALYSIS

James D. Murff, *Exxon Production Research Co.*
Suzanne Lacasse, *Norwegian Geotechnical Institute*
Alan G. Young, *Fugro-McClelland Engineers*
*Working Group #4*

## Scope of Workshop Discussions

The purpose of this paper is to report the findings of the workshop with regard to significant foundation related issues. The foundations workshop sessions were attended by a subset of approximately 25 workshop participants. Most of the attendees were specialists in offshore geotechnical engineering. A set of issues selected prior to the workshop provided a format for detailed breakout discussions. Specifically, participants were encouraged to discuss the following aspects of each issue:
- The state of the practice
- Major deficiencies in the practice
- Recent advances or on-going work to improve the practice
- Directions for future research and development

There are many problems in offshore foundation design practice that are common to those of assessment of existing structures. Furthermore, there are many foundation issues that overlap with the subjects of other work groups such as loading, structural interaction, etc. A full discussion of such topics within the brief allocated time was impractical. Therefore, in the foundation sessions, we attempted to focus strictly on foundation issues that are particularly pertinent to the requalification question. It is worth mentioning that maintaining this focus turned out to be surprisingly difficult as the discussions tended to drift naturally toward more generic foundation issues.

As will be described in the following sections, six topics were selected by the authors for detailed discussion during the foundation breakout sessions. Each session was conducted by three invited contributors: a presenter, a discussion leader and a recorder. More specifically, the sessions began with the invited speaker briefly presenting his/ her interpretation of the issues related to the topic posed by the authors. The discussion leader then served as moderator for an open forum. The recorder took notes of the discussions and presented a summary at the end of the session. A seventh session was used to plan the overall summary report to the general session.

The authors have chosen to group the report of the discussions under the following three major topics, each with its own set of issues:
- Data gathering and review
- Assessment
- Upgrading options

In preparing for the workshop, eight issues from the three major topics were selected by the authors as having particular interest to participants. These issues are identified in the following sections. As workshop preparation proceeded, however, it was concluded that only six issues could reasonably be addressed in the time allocated for the breakout sessions. It was therefore decided not to have specific sessions on *Issue #4 (Use of Performance Observations, Shallow Foundations)* and *Issue #7 (Criteria for Pushover Analysis)* since these are perhaps of somewhat less general interest. However, because of their importance, it was decided to leave these issues identified in the report and invite write-in contributions from participants.

The following is a presentation of the selected issues within the overall framework of assessment topics. The

description of each issue is introduced by a summary of the invited presenter's remarks followed by a summary of the discussions held during the breakout sessions. These summaries are intended to reflect an overview of what was said in the sessions as provided by the presenters' written notes and notes taken during the discussions by the session recorders. We have attempted to properly acknowledge those who contributed, however it was impractical to have everyone involved review this report. Therefore, while these comments reflect the work and ideas of the session presenters, discussion leaders, recorders and participants, the authors take final responsibility for accurately reporting what was said.

The length of this report makes it somewhat difficult for the reader to keep the material in proper context. Therefore the following notes are provided to assist the reader in this regard:

- The report is presented in the same format as in the "white paper." The summaries of breakout sessions, including names of session leaders, have been inserted after the definition of each issue
- The white paper was rewritten in a report context. Additional outline identifiers have been added to clarify the major topics and subtopics.

# Data Gathering and Review

The first step in assessment of an existing foundation is to collect and review the available data that pertain to its present condition. This includes a wide range of information and is discussed in more detail in the following paragraphs.

## Site data and other design documentation

A partial list of the information that may be useful in an assessment is as follows:
- Shallow seismic, side scan, regional soil borings and *in-situ* tests
- Regional/local geology
- Soil borings at the site- number, location, depths etc.
- Geotechnical data reports
- Site information from neighboring structures
- Bias and uncertainty in the design parameters
- Hazards considered in the design or new ones identified
- Analysis/design models and calculations

For the purposes of the breakout session discussions, two situations were considered: (1) critical elements of the above information are not available or do not meet 'new design' standards and (2) the basic design information is adequate.

The first situation inevitably engenders considerable 'soul searching'. The issue that was originally posed to the breakout session leaders was as follows:

*To what extent should indirect information such as geophysical, regional trends (e.g., data from nearby sites), geologic history, etc. be used in establishing analysis parameters such as strength, density, etc.? In the Gulf of Mexico? In the North Sea? Discussion of this issue should include ancillary questions such as how far from an actual boring can extrapolations be made? When should a new boring be recommended? Can minimum strength profiles be established for a given geologic setting? (Issue #1 Use of Indirect Information)*

## Session Leaders:
Presenter: A. G. Young, Fugro-McClelland Marine Environmental, Inc.
Discussion Leader: E. H. Doyle, Shell Offshore, Inc.
Reporter: R. Ingersoll, Mobil Research and Development Corp.

## Summary of Presentation by A. G. Young
Among the more important sources of indirect information are:
- Geophysical data
- Geologic history
- Regional strength data

• Pile driving data from nearby sites

All of this information may play an important role when the evaluator has a good understanding of the geologic regime and recognizes the interrelationship that exists between geologic processes and geotechnical engineering parameters.

A good model of the geologic history of an area is critical to ensure that the geologic factors affecting the design, construction, and later assessment of the foundations are recognized and adequately interpreted into useful engineering data. Further it is important for the geotechnical engineer to have a sound understanding of the site conditions and sediment types and how they may influence the foundation performance. It was emphasized that indirect information can certainly be more fully exploited when the geologic model is well defined and there is a long history of observed foundation performance in the area.

Because of the similarities and overlap in *Issues #1* and *#2*, it was decided to combine their discussion summaries into one section reported under *Issue #2* below.

As alluded to above, the situation where the basic design data are adequate is not always straight forward either. A key issue that arises in this case is:

***What parameter corrections are justified where 'old' methods of sampling and testing were used in the original site investigation? For example, is it appropriate to apply corrections to data from driven samples? From unconfined compression tests? What should be the standards for parameter determination? To what extent should "old" (perhaps inadequate) analysis models/correlations be considered/corrected for? (e.g., pore pressure corrections for CPT?) (Issue #2 Assessment of "Old" Site Specific Data)***

### Session Leaders
Presenter: R. E. Olson, University of Texas
Discussion Leader: H. Kolk, Fugro, BV
Reporter: R. Ingersoll, Mobil Research and Development Corp.

### Summary of Presentation by R. E. Olson

Site-specific "old" data includes (1) geophysical data (subbottom profiler, shallow profiler, deep penetration system, side scan sonar, and marine magnetometer), (2) soil borings (soil descriptions, water contents, Atterberg limits), sampling (by different methods) and undrained shear strength of cohesive soils (different testing methods, generally rather crude) and other data, and (3) pile driving records.

In applications of current pile design methods, the required data depend on the model used for the analysis. It is imperative that the data are interpreted in light of the data in the database underlying the empirical calculation model used. There are a number of databases of various sizes. Under partial sponsorship of API, the presenter has assembled a data base with a wide range of soil and pile properties (all steel pipe piles). Soil profiles were cohesive (78 tests), cohesionless (97), and mixed (142); loading was compressive (246) and tensile (71); soil properties were determined by standard penetration test (238), unconfined compression (161), Q-type triaxial (53), minivane or torvane (41), quasi-static cone (27), and field vane (21); data quality ranged from good to excellent (145) to relatively poor (172); pile diameters ranged from 4.5 inches to 60 inches; pile lengths ranged from 9.9 feet to 315 feet; and the best samples were 3-inch thin wall or better (59), 2-inch thin wall or better (44), thick wall (185), or unknown (31).

The following important issues were identified:
• Can correlations be developed between strengths from "earlier" tests, e.g., unconfined compression tests, and the tests commonly used today, e.g., unconsolidated undrained triaxial tests and field vane tests? Such correlations are not simple as they depend on sampler, trimming, storage time, soil sensitivity, strain rates, stress state, etc.
• For a number of years driven sampling was the standard sampling method. Today, pushed samples are common. Are there correlations that can be used to account for the effects of sample disturbance due to sampling technique?
• If the database (and thus the empirical calculation methods based on it) represents piles with set-up times of one week to one month, what capacities may be expected over several decades?

As mentioned above, the discussions of *Issues #1* and *#2* are combined here because of their similarities and overlap.

### Summary of Discussion of Issues #1 and #2

There was clearly a consensus that all data sources should be carefully considered in any platform assessment. Almost any data can be useful if sound judgment is used in its interpretation. This includes indirect data such as geophysical surveys and site specific data obtained using 'old' methods such as wire line hammer sampling and unconfined compression tests. It was further clear that participants recognized the extreme diversity of situations that can arise and were in favor of interpreting the data on a case by case basis rather than developing general rules. The following specific examples were discussed:

- The horizontal distance one might extrapolate stratigraphy and associated engineering properties from a boring using geophysical data should be dependent on the nature and quality of the data as well as the geology of the site and the soil characteristics themselves.
- Rules of thumb for relating different test data, e.g., field vane versus UU triaxial versus unconfined strengths, can be valid but such correlations can be very sensitive to subtle variabilities in the soil properties and geologic histories. No fixed set of rules for such correlations is likely to be valid in general.
- Great care should be taken when tracking geophysical reflectors horizontally between borings as the 'cause' of a specific reflection may not be indicative of uniform engineering properties along its length.

On the other hand there was a degree of optimism about the value of indirect and old data. It was emphasized that such data should be considered as a whole within the context of the geology and the entire set of engineering and physical properties of the soil. Where variability is not great, sensible extrapolations away from borings and interpolations between borings can be made. A number of participants emphasized the need for engineering judgment in such circumstances. There was general agreement that one should always strive to get a logical profile. For example, if a particular sample strength is obviously affected by sample disturbance and is inconsistent with a reasonable interpretation of the site geology and other data, then one should take this into account in the interpretation. One should not feel compelled to arbitrarily honor data that is not logical.

Among the research needs identified in this area were:

- For some old structures, particularly those that have changed ownership several times, there are not only problems with missing or questionable soils data but the foundation configuration (pile depth, wall thickness schedule, etc.) itself may not be known. Thus, there is a need for nondestructive methods for assessing or confirming pile properties on an in-place platform.
- There remains a keen interest in assessing geotechnical properties from geophysical data as the potential technological gains and cost savings would be enormous. Significant efforts have been placed on this topic in the past, but, in many situations, one is still not able to distinguish sand from clay much less quantify specific properties. The participants are still optimistic about the possibilities of this technology and encourage further work.

Finally, the participants were unanimous in their endorsement of improved record keeping and storage of all geotechnical information. The lessons of the past certainly underscore the frequent need to have a comprehensive data set available for a range of purposes, not the least of which is requalification.

### Construction Records and Geotechnical History

After the structure is installed and has been in service, the engineer has access to more information than was available during design and thus may have a basis for modifying the foundation model for post installation analyses. Construction records (if and when available) should indicate structure and foundation revisions that occurred during construction and installation. In addition, these records may help to assess the design assumptions, e.g., location of end bearing sand layers from pile driving blow counts. The foundation's history such as survival of storms, instrumentation records of structural response, pore pressure response, etc. can also be a valuable source for verifying/revising design models and assumptions.

The following information is useful for evaluation of all types of foundations:

- Structural modifications (e.g., may affect foundation load distribution)
- Historical platform loading (deck loads; drilling loads; estimated wave, wind, current, ice and earthquake loads) and relevant checks of component capacities
- Instrumentation (accelerometers, strain gages, pore pressure transducers, etc.)
- Remedial measures taken during installation
- Observations during platform operations such as scour depths

### Pile Foundations

The following types of foundation information are specific to pile foundations:

* Integrity of grouted connections for piles (e.g., pile sleeves for skirts, sleeve piles for drilled and grouted inserts, etc.) such as indications of visible grout returns
* Incidences of jetting, drilling out plugs, drilling pilot holes, etc., to advance driven piles to grade
* As-installed pile lengths and wall thickness schedules, particularly as they differ from design
* Pile and conductor driving records including pile driving hindcasts where available

In most cases, particularly in mature operating areas, pile installation is carried out essentially as planned. Furthermore, pile and conductor installation records are often among the data that are preserved and retrievable. These records, for the most part, contain blow count versus depth recordings plus notations of add-ons and other delays. Further, any remedial measures required to advance the pile are usually noted such as jetting or drilling pilot holes. As such, these data are an important potential source for verifying and/or improving the axial pile capacity design model. The differences of opinion regarding the interpretation of such data give rise to the third key issue.

*Should pile installation data (blow counts, instrumentation, locations and size of pilot holes etc.) be used to update axial pile capacity estimates? What are the limitations of such revisions? How does installation data relate to long term capacity? Is restart or retap data more appropriate? (Issue #3 Use of Installation Data, Pile Foundations)*

### Session Leaders

Presenter: A. Puech, Geodia, Inc.
Discussion Leader: R. G. Dahlberg, Veritec
Reporter: R. Dutt, Fugro-McClelland Marine Geosciences, Inc.

### Summary of Presentation by A. Puech

Installation data can play a role in three aspects of assessment: (1) confirmation of stratigraphy (changes in stratum elevation, thickness and soil type), (2) qualitative information on soil strength and need for remedial measures, and (3) quantitative information on soil strength (reassessment of design parameters and capacity).

The main issue proposed for discussion was whether, on the basis of the old state of practice and the present state of knowledge, one can expect reliable estimates of axial pile capacity from pile installation observations. Possible avenues to improve the situation include dedicated model pile tests, more systematic use of hammer and pile driving monitoring techniques and research on pile driving parameter assessment techniques.

Other related issues identified were:

* It was emphasized that blow counts alone are inadequate to assess soil resistance to driving, primarily because the energy delivered by a hammer can vary so greatly. Efficiencies between 40-60% are not uncommon for a hammer at a given site.
* The questions were raised— Are the simplified wave equation analysis models (quake-damping models) sufficient to model dynamic soil response? What is the reliability of the signal-matching procedure for long open-ended steel piles? Examples show that the results of signal-matching can vary significantly from one operator to another.
* Additional questions posed were— Is the profession confident in the current methods for determining the soil resistance to driving? What are the factors (soil related and driving-history related) affecting soil resistance to driving? For clays, are all the key parameters governing set-up properly identified, for example, apparent set up (due to pore pressure evolution) versus true set-up (due to soil reconsolidation)?

### Summary of Discussion of Issue #3

During the discussions, there was clearly a consensus that any additional information will be useful to foundation reassessment, first to confirm some of the assumptions made, and second to provide means for an overall assessment of the pile capacity. All agreed that blow count observations are useful for the quantification of time effects. As a minimum, one should make soil plug measurements and do set-up test(s) (restriking the pile after a specified set-up period) to estimate increased resistance with time for piles in clays.

There was some agreement, although not a consensus, on the ability of dynamic monitoring and signal-matching to assess soil resistance. Blow counts coupled with instrumentation results (hammer efficiency, velocity and acceleration) could provide a reasonable basis for reassessing pile capacity. The effect of different variables is however somewhat uncertain. The models for predicting pile capacity from pile driving records were debated at length and a separate

written discussion by Professor Randolph (a strong advocate of the technique) on the potential of the interpretation of dynamic pile load tests to evaluate soil-pile resistance is presented in Appendix I.

Among the research and development needs identified in this area were:

- Methods to determine the length of an in-place pile (considering that the pile will have been welded or grouted to the jacket)
- Methods to identify site-specific soil properties from stress wave measurements

Important issues related to pile driving should also include: (1) for sands, whether a pile cores or plugs during driving, the skin friction distribution and degradation along the pile wall and whether there are effectively limiting values with penetration depth; and (2) for clays, whether a relationship between the dynamic and static skin friction can be established and whether there is a limiting value of skin friction at depth.

### Gravity Foundations

The following types of information are specific to shallow foundations (e.g., gravity structure foundations, jack-up spud cans and mats, anchors, mudmats, etc.)

- Installation records including skirt penetration, under-base grouting,
- Foundation performance over time including scour, long term settlement
- Settlement during severe environmental loading
- Instrumentation results including pore pressures during loading and accelerations (for stiffness assessment)

For shallow foundations, observations and response during installation and during subsequent environmental loading can provide useful indications of foundation behavior under different conditions. These can allow one to verify design assumptions. Back-calculations are not always done and instrumentation results are often limited to data reduction without interpretation in terms of the observed data significance from the foundation response point of view.

*Should foundation engineering also include a reassessment of the soil parameters from the results of observations during installation or later on in the life of the platform? Should the parameters be adjusted to reflect an "as installed" foundation in case specific events or penetration observations suggest that changes may be warranted? If so, how can this be done? Should pore pressure response be compared to what is expected on the basis of the assumptions made when defining the soil parameters for the different components of the foundation analysis? (Issue #4 Use of Performance Observations, Shallow Foundations)*

This issue was not discussed at the workshop but is included as it is an important issue for gravity structures. While the number of gravity structures is relatively small these structures tend to be extremely large, manned facilities and usually involve large investments. They are somewhat unique in that they have often been heavily instrumented, thus, providing the potential for reassessing design parameters based on measured performance. While no open discussion of this issue was conducted, a written contribution by Susan Lacasse is included as Appendix II.

### Physical survey data of the structure foundation system

A physical survey of the structure and foundation during the platform's life, and especially near the time of the assessment can be very useful for updating the foundation model. Any change in the structure can give rise to changes in the load distribution in the foundation. An inspection of the seabed in the proximity of the structure's base can be useful in evaluating the contribution of mudmats and horizontal framing members to the strength of the foundation system. The existence of scour (or lack thereof) around piles can be helpful in updating design models. It is particularly helpful if periodic survey data (e.g., ROV) is available to assess long term trends or changes due to specific events (e.g., especially severe storms, harsh winters, etc.). However, one must be mindful that observations of scour after a storm may be not fully reflect the worst case, as there may be a tendency for scours to be backfilled during the subsidence of the storm.

## Assessment

Having acquired the available data for analysis, the next step is to carry out the assessment(s). Three sequential analysis checks of existing structures have recently been proposed: screening analysis, design level analysis and ultimate strength analysis (Draft Revisions to API RP2A for Assessment of Existing Platforms). For the purposes of discussion, this three-level format will be adopted here.

### Level 1-Screening Analysis

Prior to conducting a screening level analysis, a review of the data is carried out to determine whether the platform has been damaged, loading has increased, air gap has been reduced, or in general the design environment has changed. If the environment is significantly different from the design assumptions then the screening analysis check is skipped and the design level analysis is conducted. If not, the screening analysis check is carried out. This simply involves confirming that the platform was designed to a specified edition of API RP2A or equivalent (this is intended to be a 'modern' version). For this level of checking it is appropriate to confirm that the design soil parameters are valid, e.g., that the platform location with respect to the soil boring(s) is as assumed in the design. If the platform passes the screening level check, no further action is required. If not, a design level analysis is required.

### Level 2-Design Level Analysis

For the purposes of conducting a design level analysis of an existing structure, the use of nominal design parameters as would be used in new design (as opposed to best estimate parameters) is prescribed. However, during the original design process, various conservatisms are introduced to account for unknowns and uncertainties that inevitably arise in the construction, installation, and service of the platform. Furthermore, new platform specific information as well as new research results may be available since the original design was completed. In these instances it may be appropriate to use the following in the analysis:

- Revised loads
- Revised soil parameters
- Hindcasts of component behavior based on pile driving, skirt penetration, measurements, etc.
- Identification of all important potential failure modes (especially identification of new modes, i.e., those different from ones identified in the original design)
- New engineering analysis including: steel piled jacket (axial capacity, laterally loaded pile stress analysis, earthquake response) and shallow foundations ( bearing capacity, hydraulic stability, soil structure interaction, settlement, earthquake response)
- Probabilistic analyses to better understand the sensitivities of the foundation performance to parameter bias and uncertainty
- Reconcile observed versus calculated behavior
- Two key issues that arise in carrying out a design level foundation analysis are described below.
- Hazards considered in the design or new ones identified

*Should axial pile capacity determinations explicitly account for cyclic and rate of loading effects? If so, should nominal factors be used or should laboratory tests be the basis? What factors? What laboratory tests? Should assessment include new developments with respect to pile capacity (in favorable or unfavorable directions) that have been published since the design calculations were completed? To what extent should research results (recently published studies) be used for the assessment (especially when results are in disagreement with current practice)? (Issue #5 Use of Recent Research Results in Pile Capacity Assessment)*

### Session Leaders

Presenter: M. F. Randolph, University of Western Australia
Discussion Leader: J. L. Briaud, Texas A&M University
Reporter: R. J. Jardine, Imperial College

### Summary of Presentation by M. F. Randolph

Two problems often occur in practice:
- Newer pile load test data are in conflict with the current design guidelines.
- Phenomena that are observed in the field (or the laboratory) are not accounted for in routine design.

The variability in the pile capacity reference data is associated with:
- Natural (but unquantified) variation in soil properties across the site
- Imprecise quantification of key soil parameters, such as shear strength
- Variation in installation and testing procedure (e.g., pile tested before full reconsolidation)
- Other effects such as loading rate, multiple tests on a single pile, failure definition, etc.

The different factors can often be compensating with, for example, high loading rates, load redistribution due to creep and cycling, and aging being beneficial while strain-softening, degradation due to cycling and creep being detrimental.

Three main issues were considered important:

- If an adverse change in foundation or loading characteristics has occurred since original design, should a lower capacity be adopted in reassessment?
- Is it reasonable or ethical to ignore results of recent research until guidelines are updated (especially in the light of implied lower capacity)?
- Should one include, in analysis, effects of strain-softening and high loading rates under storm loading? How should these factors be accounted for?

### Summary of Discussion of Issue #5

The group discussion focused on the implications of new research data by addressing the following specific issues:

- the assimilation of new pile load test data into the data base and the resulting code changes,
- the use of site specific pile load test results without changes in design code, and
- the need for future research to understand phenomena of pile behavior that are not addressed explicitly at present.

There was a strong consensus that research will take a long time and should be conducted in a phased approach. Phase I should be directed towards the identification and discovery of the basic physical model, any governing phenomena, and fundamental reasons for divergence with practice. In Phase 2 we should attempt to quantify by geotechnical analysis the findings from Phase 1. Finally, Phase 3 should simplify and codify the analysis, so it can be applied retrospectively to different geologic regions and diverse site conditions around the world.

On Point 1 above, the group agreed that caution should be exercised when using new load test data to justify changes in the design code. Such changes are much more acceptable when the geologic conditions and soil types are not considered unusual when compared with those originally used to develop the empirical design method. The group agreed that the design code should be changed, to include both beneficial and adverse effects, only if the test results can be explained, quantified, and applied sensibly in terms of soil types, stress history, and geologic conditions.

Site specific pile load tests were considered a reasonable method for improving pile design parameters for a specific structure or area when the design code is recognized to have a sparsity of pile load tests in the particular soil type or geologic regime. For example, the Shell Oil Co. Long Beach (California) Beta Test (J. H. Pelletier and E. H. Doyle, (1982) Tension Capacity in Silty Clays- Beta Pile Test. Proceedings of the 2nd International Conference on Numerical Methods in Offshore Piling, Austin, Texas) provided valuable pile design information for nearby offshore structures. However, these site specific test programs should also be planned to understand basic soil-pile behavior. This allows one to generalize the results with the goal of making improvements in the design codes and not merely using the results for the parochial requirements of the project.

All participants agreed in the discussion of Point 3 that improved understanding of the basic behavior is needed from future research, so the effects of strain softening, loading rate, and cyclic loading can be evaluated relative to the actual field capacity. However, most agreed that the basic phenomena are not well understood in the context of a physical model and thus practitioners are rightly reluctant to consider a single phenomena in a design without accounting for the full effect of all others.

Most agreed that strain softening occurs, but the effects may differ from one region to another depending upon the susceptibility of the soils. The effects are not considered to be substantial in some soils such as Gulf of Mexico clays, but there is a pronounced influence in partially cemented, highly structured carbonate sands. Analytical methods for numerically modeling strain softening exist, but need to be treated carefully due to some anomalies that may develop such as numerical errors.

Rate of loading effects can increase the design capacity, but the benefits of same should not be used without including the effects of cyclic loading or strain softening which typically reduce capacity. However, a need exists to quantify the effect for various types of soil by a field or laboratory test such as the *in situ* field vane or direct simple shear, respectively. These tests must duplicate the rate of loading associated with the environmental loading such as the maximum storm wave.

A majority of the participants agreed that cyclic loading will reduce design capacity when the magnitude of the cyclic load is significant in comparison with the shear strength of the soil. Foundation analyses of gravity base structures typically incorporate these effects explicitly in the design method in contrast to the current practice of offshore

pile design. There are computer methods currently available to analyze the effects of cyclic loading, but these methods need to be calibrated with physical tests such as a field mini-pile, laboratory model pile, centrifuge model test, and/or full-scale field pile. Some studies of this type have been carried out, but no generally accepted methods of design or analysis have yet been developed

In summary, the group of participants agreed that further research is needed to refine the analytical methods currently presented in the API recommended design practice. This research should continue to extend the pile load test data base to more closely approach actual prototype pile sizes and load levels. The research should also be planned to improve design methods in new or unfamiliar soil types, for nonstandard installation techniques or for severe load regimes. Application of such research to the assessment problem is inextricably linked to such advancements.

*What is the appropriate role for probabilistic analysis in foundation assessment? Are estimated safety indices for axial pile capacities believable? How should the acceptable reliability level be established (probability of failure or safety index equivalent to a safety factor)? What should be done, if anything, to resolve differences in working strength and LRFD design values? (Issue #6— Role of Probabilistic Analysis)*

### Session Leaders
Presenter: S. Lacasse, Norwegian Geotechnical Institute
Discussion Leader: W. H. Tang, University of Illinois
Reporter: R. B. Gilbert, University of Texas

### Summary of Presentation by S. Lacasse
On the premises that (1) uncertainties in foundation behavior and soil-structure interaction exist, (2) probabilistic models are now accessible, (3) solutions have been developed for a range of geotechnical problems, and (4) probabilistic solutions are to be used in addition to deterministic analyses, a number of issues were raised. These issues also reflect requirements to make reliability analysis more useful for geotechnical engineers.

- The uncertainties in the parameters and their effect on the results obtained need to be established, even if this is difficult.
- Engineers need to know how to quantify model uncertainty, especially since the ideal model test that will clearly establish model uncertainty is rarely available. One concern is the large extrapolations that need to be made of the data from reference pile load tests to more typical pile dimensions and loads used offshore. Model uncertainty also should account for human judgment. How does one quantify this?
- With regard to the interpretation of the results of reliability analysis, how should the reliability results be used, and how does one determine an acceptable target probability of failure when requalifying a structure?

There is a need to quantify the uncertainty from "old" design methods where engineering judgment and hidden factors were combined in a single safety factor. It is helpful in the process of code development to establish the implicit probability of failure for "conventionally accepted" designs for structures already in place. That is, probabilistic methods should be calibrated using methods that have successfully worked in the past.

### Summary of Discussion of Issue #6
During the discussions, there was a consensus that probabilistic analyses will serve a valuable role in the assessment of foundations for existing offshore production structures. The probabilistic approach can systematically account for the uncertainties in the parameters and the model and provide a consistent measure of foundation performance that is not achieved using a deterministic approach. The large uncertainty in many geotechnical parameters is a reason for, and not against, using reliability approaches.

Probabilistic analyses are also helpful in designing effective site investigation programs to minimize uncertainties in soil properties, and in performing sensitivity studies to identify critical design parameters. They also enable one to evaluate complex problems with multiple failure mechanisms, where deterministic analyses may lead to compounded conservatisms.

However, several major deficiencies were identified in the current state of practice with respect to probability analysis. First and foremost, calculated failure probabilities are better used at present to provide relative but not absolute measures of foundation performance because they have not been fully calibrated. For example, the reliabilities of several foundations can be compared using a consistent analysis of failure probabilities. It should be noted that the reliability of a foundation is not necessarily comparable with that of a structure. Calculated failure probabilities for foundations are generally higher than for those of structures because the uncertainties in properties and analysis/design models are greater, yet it is the perception of the workshop participants that significantly fewer offshore foundation

failures than offshore structural failures have occurred or at least have been documented.

Workshop participants were of the opinion that the major contribution to uncertainty in foundation performance results from the uncertainty in the performance models. Databases used to calibrate these models are lacking in quantity and quality, and very large extrapolations are commonly required to relate the data to offshore foundation performance. Hence, considerable expertise and judgment are used together with the existing databases to assess appropriate model uncertainty. Although there clearly are not enough data, the geotechnical engineer believes that he has a good understanding of the basic behavior and of the response of the foundation elements he is trying to model.

In the current state of practice, many geotechnical engineers are not familiar enough with probability concepts and have had little formal training in this area. Consequently, the full potential of probabilistic analysis has yet to be realized in practice. Examples of how to use the method are needed.

The research and development needs identified in the area of application of reliability analyses to foundation assessment include:

- Reducing model uncertainty by obtaining and analyzing more performance data of high quality [laboratory or field model tests, construction records, performance records during events (storms, earthquakes, etc.)]. It is important to understand the components of behavior better, rather than lumping together different factors affecting performance. Bayesian updating may prove to be useful for the assessment of existing offshore foundations for requalification.
- Comprehensive probabilistic sensitivity studies should be made to identify the effects that contribute most to the overall uncertainty in predicted capacity.
- More work is required to study the performance of the foundation system relative to the individual components that form the system (e.g., capacity of four-pile system versus single pile components). It is important to develop the definition of failure that best reflects actual performance.
- We should strive to better quantify failure probabilities for different offshore structures in cooperation with owners and policy makers. Probabilistic results will be useful, not only to compare different foundation designs, but to make requalification decisions considering costs and benefits.
- It is imperative that geotechnical engineers communicate with engineering specialists in related fields (e.g., structures, hydrodynamics, environment) to develop appropriate parameter statistics and to maintain consistent assessment criteria throughout the requalification process.

## Level 3-Ultimate Strength Analysis

The third level of checking (for platforms that do not pass screening or design analysis levels) is to carry out an ultimate strength analysis. This is typically done by scaling up the environmental forces for the design level event on a nonlinear structural model until the collapse load is achieved. This is frequently referred to as a pushover analysis. The general considerations are similar to those enumerated above for design analysis except that 'best estimate' rather than nominal parameters are used and other known conservatisms are removed. An issue that is particularly important for and unique to nonlinear analysis is:

*What criteria should be used for modeling the lateral soil resistance along a pile for 'pushover' analysis? Should so-called 'static' or 'cyclic' models be used? Should a displacement limit be placed on such analyses? (Issue #7— Criteria for Pushover Analysis)*

As previously stated this issue was not discussed at the workshop but is included here as it is an important issue for reserve strength analysis of steel piled jackets. It is of particular interest for shallow water structures where the critical failure mode is often found to be the formation of a lateral shear mechanism in the piles. While no open discussion of this issue was conducted, a written contribution is included as Appendix III.

## Upgrading Options

At some point during an assessment it may become apparent that remedial action of some type is indicated (especially where a platform fails to pass the Level 3, ultimate strength analysis check). There are a number of actions that can be carried out to upgrade the foundation (or perhaps downgrade the requirements, e.g., demanning) to render it suitable for continued service. Among the alternatives are:

- Collect new soils data (improved quality, more borings, closer to site, etc.) to justify a more optimistic inter-pretation
- Install instrumentation to verify design/analysis assumptions
- Set criteria for shutdown, evacuation, etc.
- Enhance pile performance: retap or pull conductors (or piles if practical) to assess capacity, upgrade capacity by adding insert piles or artificial plugs, add outrigger piles
- Enhance shallow foundation performance: add berms around base of structure, add scour protection, add piles in skirt compartments or around periphery to enhance sliding stability, add ballast to structure to enhance overturning or sliding stability

A key issue regarding what is often considered one of the first lines of defense is as follows:

*How does the foundation engineer prioritize or weight the competing factors in his recommendations to the owner, such as: safety requirements; the reality of having a structure that is already installed with a safety factor below standard or with key pieces of information missing; the expected conservatism built into parameter selection and assessment methods; the huge costs that may be involved; the uncertainty in effectiveness (and even risk of making things worse) of the proposed remedial action (e.g., how effective are remedial measures aimed at strengthening piles such as adding inserts and artificial plugs?). (Issue #8—Remedial Measures)*

### Session Leaders

Presenter: J. H. Chen, Chevron Petroleum Technology Co.
Discussion Leader: J. D. Murff, Exxon Production Research Co.
Reporter: S. Lacasse, Norwegian Geotechnical Institute

### Summary of Presentation by J. H. Chen

Discussion issues in the planning of structural upgrade of offshore platforms included the target reliability, costs involved, feasibility and effectiveness, and possible disruption in the platform operation. The costs vary over a wide range, from the relatively modest costs of new soil borings to the extremely high costs of pile strengthening using pile tip bell footings as used for North Rankin A on the Northwest Shelf of Australia (Proceedings of the International Conference on Calcareous Sediments, Perth, Australia, 1988).

When considering foundation upgrading the following issues should be addressed in any reassessment:
- Attention should be focused on a system rather than a component (i.e., only foundation) approach.
- The technical feasibility and effectiveness of different options need to be evaluated on a case by case basis. Because there is little or no direct experience with a number of existing options, there can be potential downside risks, and large (perhaps unacceptable) deformations may be needed to achieve the added resis-tance.
- One should focus on selecting an upgrade solution that really improves the situation rather than giving a good solution on paper. In this enterprise details are important.
- There should be special concern for maintaining the integrity of the structure and minimizing disruption of the platform operation.

### Summary of Discussion of Issue #8

In the decision of whether to strengthen a foundation or not, the following considerations were deemed to be of most importance:
- results of conductor pull-out tests on the platform under consideration
- whether the proposed remedial action can weaken other aspects of the foundation or the structure
- the degree of improvement that is needed
- the cost-benefit trade-offs of remedial action; balance between safety level and benefit of an inspection

- the level of deformation needed to mobilize the added resistance, e.g., for load transfer
- the risk incurred if one decides not to strengthen
- the possibility of doing further damage during the strengthening operation (to what extent and with what degree of certainty will the measure improve the situation)

The following options with respect to strengthening or verifying axial pile capacity were mentioned: strengthening—insert piles (grouted or driven); outrigger piles; guy wires; bell footings; struts; grout or gravel plugs; soil modifications (freezing, chemicals); verifying—new site investigations; instrumentation of structure and monitoring during events; pile/conductor load tests; and including drill casings and mudmats in resistance calculations.

Among the needs identified with respect to upgrade options during requalification of offshore structures were:
- Enhanced dialogue between geotechnical engineers and structural engineers when dealing with upgrade decisions.
- More effective exploitation of the existing large database of upgrade case histories. Both the decision-making process and the upgrade solution selected could be documented. It is highly recommended that industry makes an effort to establish a reference database of actual foundation upgrades. The case studies should describe (1) the problem; (2) the decision process; (3) the solution selected; and (4) how the effectiveness of the measure taken was confirmed. It is important that all information pertinent to the condition to be upgraded is captured.
- Means of evaluating the effectiveness of the strengthening solution adopted are needed; it is of questionable value to employ remedial measures without confirming their effectiveness.

It was also suggested that in the future there should be less need for remedial actions and upgrades, as the design process gradually improves.

## Closing Remarks

There are many actions that can be taken during the design, construction, installation and service life of a structure that can facilitate its subsequent assessment and requalification. As such, assessment requirements should be made a part of the plan from inception. A carefully thought out system of archiving pertinent data, calculations, measurements, and inspection results throughout the life of a platform is one of the most important aspects of such a plan. Such a system is greatly enhanced by a clearly marked trail over the life of the structure indicating what decisions were made and why. Effort spent on the front end in this manner will inevitably pay for itself many times over.

At the close of the workshop discussion, the following consensus was drawn on foundation assessment for requalification of offshore structures.

Geotechnical specialists believe that their state of knowledge is moving in the right direction, but there is still a lot of work to do. The details of the geotechnical procedures for foundation assessment and the large number of parameters and relevant factors are such that it is important to keep flexibility in the codes and standards and to avoid imposing too restrictive or too specific requalification requirements.

At nearly all levels, there is a need for greater interaction between the geotechnical engineer and other fields of engineering (specialists on environmental conditions, hydrodynamics, structural analysis, operations, and policy making).

## Acknowledgments and Disclaimer

# Appendix I
## Issue 3: Interpretation of Dynamic Pile Load Tests

*Contribution by*
*Professor Mark F. Randolph, University of Western Australia*

Discussions were held at the workshop on the reliability of the soil resistance to axial loading deduced from dynamic load tests carried out during pile driving. Professor Mark Randolph, from the University of Western Australia, shared the following experience with the participants at the workshop.

The measurement of dynamic force and velocity waves at the pile head during a hammer blow is generally referred to as a dynamic load test. The data allow accurate assessment of the hammer energy delivered to the pile, and may also be interpreted to yield an estimate of the soil resistance mobilized during the blow. A full interpretation, known as signal-matching, involves adjustment of the parameters in a numerical model of the dynamic pile and soil response until a close match is obtained between the measured and computed signals. Generally, measured force and velocity data are combined to give the downward traveling component of the stress wave, which is used as input to the numerical model. The signal-matching is then carried out on the upward component of the stress wave, comparing measured and computed response.

The main parameters that must be adjusted are the limiting values of shaft friction and end bearing. These are taken as equivalent static values and are adjusted to give the full dynamic resistance by means of additional parameters that attempt to quantify inertial and viscous damping. A variety of different soil models have been proposed for the dynamic components of soil resistance, and this is still an area of current research. However, the total static resistance deduced from an accurate signal-matching is relatively insensitive to the precise soil model and choice of dynamic parameters. As such, dynamic tests must be interpreted by experienced personnel to give estimates of the equivalent static soil resistance mobilized during the blow. Such estimates are considered to be more reliable than estimates that would be obtained from a more fundamental approach based on soil parameters and, for example, the API design guidelines. Of course, this is partly a reflection of the variability and limitations of the design guidelines.

While it is considered that the total soil resistance mobilized during the blow may be assessed accurately, the following should be emphasized:

- The dynamic load test can give only information on the soil resistance mobilized at the time of the blow, and any changes in resistance that might occur with time should be assessed by re-testing the pile.
- A distinction must be made between mobilized pile resistance and pile capacity, since if the hammer blow is insufficient to cause significant plastic penetration of the pile, then the full pile capacity may not be mobilized.
- The mode of failure of open-ended piles during driving is generally an unplugged (or partially plugged) one, with relative motion between the complete soil plug and the pile. However, during static loading, the pile will generally perform as a fully plugged pile, and any relative movement will be confined to the lower few diameters of the soil plug. As such, some adjustment of the soil resistance mobilized during dynamic testing may need to be carried out to arrive at an equivalent static resistance.
- While the total soil resistance deduced from a dynamic load test is relatively insensitive to the soil model and 'dynamic' parameters, the detailed distribution of resistance along the pile shaft and at the pile base is rather more sensitive to these parameters. Different operators will generally arrive at a range of different distributions of resistance (but with similar values of total resistance) still achieving an adequate match to the dynamic data. As such, more confidence should be placed in the overall soil resistance than in the deduced distribution.

It should be emphasized that operators of dynamic testing equipment (and experienced signal-matchers) probably will tend to be more assertive of their ability to predict the distribution of soil resistance, while those with little direct experience with dynamic testing will tend to downplay the reliability even of the estimated total resistance.

# Appendix II
## *Issue 4: Use of Performance Observations, Shallow Foundations*

*Contribution by*
*Dr. Suzanne Lacasse, Norwegian Geotechnical Institute*

The geotechnical design of shallow foundations for offshore structures, especially gravity structures, has until now been based on much more detailed soil investigations than, for example, piled structures; and a large number of the existing gravity structures, even the very first structures, were very well instrumented. Early on, the observed data were used to adjust calculation hypotheses and to develop semi-empirical calculation procedures (e.g., methods for predicting skirt penetration resistance).

Throughout the years, a large quantity of performance data have been accumulated for gravity structures, but the data and their geotechnical content have not been fully exploited with respect to interpretation of soil behavior and understanding of the soil-structure response, partly due to lack of time, partly because the exercise was given only superficial attention, and partly because the measured data have been reduced by those accumulating the data but not interpreted further by a geotechnical engineer. This is unfortunate because important behavioral information is locked away in files and may be forgotten. As more time elapses, it will become only more difficult to reconstruct the series of events that have led to the observed behavior.

Gravity structures have the potential of providing long-awaited answers to many aspects of soil behavior or soil-structure response. They represent, in effect, model tests at a very large scale. The unexploited performance observations are a source of additional information in the case of platform strengthening or requalification. The behavior observed on one structure may well prove to be a key element in the understanding of the behavior of another structure. As a minimum, performance observations represent useful model tests for other types of shallow foundations, e.g., spud cans for jack-up structures and suction anchors.

The interpretation of instrumentation results can, among other aspects, enable one to reassess hypotheses made during design, document components of foundation behavior that are controversial, reduce uncertainty in the calculation models, and/or develop further the understanding of different behavioral patterns (e.g., dilative sand behavior, pore pressure build-up and dissipation, stresses and pore pressure on structural elements such as skirts and base). An example is the rate of settlement measured beneath the early gravity structures. The observed settlements tended to occur much faster than assumed in design, thus indicating a more rapid increase in shear strength due to consolidation under the platform weight. This observation is utilized nowadays, where installation is planned such that optimum advantage of the shear strength gains due to consolidation is taken.

Among future needs, the following should be mentioned:

* Whenever one needs to reevaluate a foundation, the soil profile, soil parameters and calculation models, if relevant, should be adjusted to reflect the added knowledge and the in situ condition at the time of the platform re-evaluation. An historical follow-up of the events in the platform history and the observed performance should be carried out to make the observed behavior fit in logically with accepted geotechnical principles.
* The platform operators should allocate the required resources (man-hours, budget) to promote an adequate exploitation of the existing measurements. The conclusions drawn on soil behavior and soil reactions will prove useful not only for other gravity concepts but also for the interpretation and future developments of other foundation types. For example, skirt penetration data, e.g., skin friction, can assist in the understanding of calculation models and existing observations of skin friction on piles; scour observations can help verify analytical models; and accelerometer data can help verify soil stress-strain models.
* A data bank of the information and instrumentation that exists should be established and priorities set up for the interpretation of the results on the basis of the usefulness and the expected gain in the understanding of geotechnical behavior and soil-structure response.
* Especially in the interpretation of performance observations, interaction with other disciplines is of utmost importance. For example, the interpretation of soil-structure interaction response (e.g., cracks in the concrete base can change the back-calculation of the foundation stiffness from accelerometer data) requires a close dialogue between the geotechnical and the structural engineer. A worthwhile reanalysis of foundation stability during a storm depends entirely on the loads derived from the interpretation of the wave height, and therefore the interaction between the hydrodynamicist and geotechnical engineer.

# Appendix III
## *Issue 7-Criteria for Pushover Analysis*

*Contribution by*
*Dr. James D. Murff, Exxon Production Research Company*

In the process of assessing the ultimate strength of an offshore platform it has become standard practice to conduct a nonlinear structural analysis. Typically the 100-year design load is applied to the structural model and incrementally scaled up until the model can carry no more load. In this analysis the foundation piles are usually modeled explicitly with the soil being represented as soil springs which are characterized by so-called p-y (lateral), t-z (axial shaft), and q-w (axial end bearing) curves (see for example, the API Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms, RP 2A, 20th edition).

The nonlinear structural analysis may indicate that the critical failure mode is completely within the structure, completely within the foundation or a combination of the two. Within the foundation there are typically two failure modes: overturning, in which the failure primarily involves axial deformation of the piles, and shear, in which the failure primarily involves lateral translation of the structure base accommodated by the formation of plastic hinges in the piles. Owing to axial load-moment interaction within the pile itself a combined overturning-shear failure mode is also possible.

In modeling the pile's axial behavior the approach is a straight forward extension of the procedures used in design. For lateral behavior, the designer usually employs the so called cyclic p-y curves (as opposed to static curves) to account for the fact that the foundation will undoubtedly undergo many repetitions of significant lateral load prior to experiencing the design condition (usually the 100-year wave). In an ultimate strength analysis the piles undergo relatively large displacements (perhaps 20-30% of the diameter or more) to mobilize the full resistance available. In this situation the question arises whether it is appropriate to use cyclic or static p-y curves. An important step in answering this question is to consider the tests that formed the basis for the p-y curve development.

In general, the cyclic loading in the tests was initiated at low load levels building up to higher levels. Even at the highest load levels the tests were typically carried to displacements that were only a few percent of the pile diameter. The p-y curves developed from these data are thus intended to represent envelopes of soil response and are not intended to be realizations of specific load versus displacement paths. The cyclically degraded curves represent the combined effects of near surface gapping as well as localized remolding and softening of the soil. They have historically been applied in working stress design (where displacements are normally less than 10% of the pile diameter) and it seems reasonable to question their direct applicability for ultimate strength analysis where a one-time large displacement excursion should force the pile into virgin soil that has presumably been minimally affected by previous cyclic loads.

The foregoing hypothesis has been tested in a series of centrifuge tests on laterally loaded piles in soft clay by Hamilton and his coworkers. (The basic test set up and results of static calibration tests are reported by Hamilton, et al., Centrifuge Study of Laterally Loaded Behavior in Clay, Proceedings of the International Conference Centrifuge 91, Boulder; the cyclic load tests will be reported in a future publication). In the tests, relatively rigid, large diameter (prototype) piles were cyclically loaded to large displacements (up to and greater than one pile diameter). The tests showed that the API cyclic criteria for soft clays were very conservative for post cyclic, ultimate strength analysis. A slightly reduced version of the 'static' curves appears to be more appropriate. Obviously care should be exercised in generalizing these results, especially for other soils such as stiff clay or sand. However, the writer would expect these conclusions to directionally apply to many conditions.

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# Discussion and Summary on

# OPERATIONAL CONSIDERATIONS

M. J. K. Craig, *Unocal Corp.*
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*Working Group #5*

## Operational Issues-Definition and Intent

'Operational Considerations' in the context of this workshop on aging structures is defined to mean addressing risk sources to a functioning offshore production platform that are not directly related to overload of the structure from the natural environmental hazards of storms, earthquakes and ice.

It means the identification of 'non-structural' risk sources capable of causing injury, pollution or destruction of the structure - what type of risks, how potentially damaging, what are their frequencies of occurrence, and what mitigating measures control them.

Such risk sources can be the under-performance of aging equipment or appurtenances - process equipment, drilling equipment, instrument and control systems, fire and safety systems, SCV's and SSCV's, risers, cathodic protection systems, navigational warning aids, and the deck structure itself.

Such risk sources can also be the under-performance of the designers, constructors and operators of this aging equipment - which raises questions on such diverse issues as operator training and retraining, human and organizational errors, safety review procedures, safety team makeups, operating and safe work practices, blowout control procedures, effects of dropped objects, weight control, inspections, walkdowns, evacuation procedures, and demanning.

The intent of this paper is to identify important 'non-structural' sources of risk, their damage potential, their perceived likelihoods of occurrence, what is being done to control them, and where/if such controls need improvement, all in the context of existing, aging platforms.

The intent of this paper is not to provide an in-depth treatment of the facility-related (equipment) issues, but to provide more of an awareness brief on these hazards. A separate workshop addressing facility-related hazards may be justified. However, considerable efforts are presently underway to implement updated process safety management guidelines. These efforts should not be diluted. If a workshop is held, process-related questions are listed in Appendix I to facilitate discussion.

Operations/Structure interface issues are discussed first, followed by facilities issues. Areas demanding further development, as identified by the Operations Working Group, are outlined at the end of the paper.

## Operations/Structure Interface Risk Issues

Operations/Structure interface issues are those in the grey area between the purely structural and purely facility issues. These interface issues can influence the integrity of the structure.

The findings and recommendations of the Work Group on Operational Considerations are listed below, in approximate priority order. As such, and given the nature of the issues, the subject topics vary considerably. The statements made below are those developed by consensus in the Operational Considerations Work Group.

### Cathodic Protection

Adequate cathodic protection is recognized as the first line of defense in the preservation of subsea structural integrity. Collection of data annually on cathodic protection levels on each OCS structure is required by law. It is crucial that these data are acted on appropriately. Underprotected structures must be retrofitted, or their integrity will rapidly decrease. This is emphasized because collection of the data is commonly supervised by operations (non-

structural) personnel. Development of clear purchase order specifications for new anodes (including connecting rod steel) is recommended. Proper anode placement and connection details to avoid tearout is encouraged. Impressed current systems perform satisfactorily provided they are properly fabricated and hooked-up.

### Fire and Blast Design

Draft guidelines proposed for a new Section 18 in API RP 2A are being developed on this subject. It is recommend that, although this topic is a complex one to quantify, the new guidelines be as pragmatic and designer-usable as possible. The process must recognize the successful operational experiences in U.S. waters.

### Risk Reduction

In the fitness for purpose assessment of a platform, there are operations-related (non-structural) issues that influence the outcome, in the form of reliability improvements or lower performance targets. First, housed personnel can be removed from a platform, and it can be re-categorized as an unmanned platform. Second, marine growth can be periodically or permanently removed, as can appurtenances such as unnecessary conductors, risers and boatlandings, to lower wave forces. Third, hydrocarbon inventory stored on the platform can be limited or removed. Fourth, upgraded surface controlled subsurface safety valves (SCSSV's) can be installed on naturally flowing oil wells, to potentially improve shut-in performance. Safety valves can be added to process piping, risers and pipelines to help limit oil spillage; more valves can be added to reduce process segment sizes. Fourth, personnel and platform structural safety can be enhanced by the selective provision of fire and blast protection, and additional means of safe escape and evacuation.

### Blowouts, 'Complex Platforms'

A 'complex platform' is one whose facilities process significant quantities of oil in closed-in, modular type process packages. For these platforms, and especially for ones located in harsh and sensitive environments, a 'total risk' type analysis may be necessary, as discussed in the next section. Note, that few such platforms exist in U.S. waters. In a 'total risk' analysis, the probability of occurrence of a blowout that would undermine the global integrity of the platform must be estimated. The development of a comprehensive database on such blowout statistics to aid with these estimates is encouraged.

### Shut-In Systems

A preliminary analysis of the limited statistics on surface and subsurface controlled shut-in system reliability shows that these systems are relatively reliable. However, there is no comprehensive documentation of this. The development of such documentation and its associated database is recommended, to aid in a) low consequence platform categorizations, and b) 'total risk' analyses. However, past attempts to develop such a database have not been successful, due in large part to poorly conditioned data when it is reported (was the cause of valve failure due to the valve itself, or to other unknown causes?)

### Ship Collisions

This issue was considered, first, very germane to global platform safety, especially for those platforms located near active ship channels, and second, complex to treat. Issues on where to place what type of navigational system (active or passive, on the vessel or on the platform) crossed boundaries beyond the experience base of the Work Group. The creation of an improved database on vessel/structure collisions is encouraged. For local vessel impacts (impacts which do not threaten the global stability of a structure), in the design of boatlandings and the analysis of bent waterline braces, it is recommended that the final Section 18 guidelines in API RP 2A-WSD on this subject be simple, pragmatic and designer-usable. Operators' field personnel are encouraged to notify the civil engineer(s) when such damage occurs.

### Dropped Objects

It is recommended that operators have in place clear reporting directives for significant objects dropped over the sides of platforms, and that their reporting to relevant parties be encouraged. One relevant party is the company's civil engineer, who may sometimes be left out of the reporting loop. This is particularly important when damage has occurred, and qualified repair is necessary. It is hoped that the new guidelines on offshore crane operations and maintenance proposed for API (American Petroleum Institute 1994) will result in fewer objects dropped by cranes.

## Topsides Seismic Performance

Equipment performance under lateral load is a critical safety issue for field personnel on platforms located in earthquake-prone areas. Appropriate restraint is required, especially of large movable items like the drilling rig. The drilling rig is particularly vulnerable due to its flexibility and its being regularly repositioned. Detailed guidance on appropriate restraint and other safety measures are given in the updated Section 2.3.6e.2 and associated commentary, API RP 2A-WSD, 20th edition.

## Riser Guards

It is recommended that, in light of the satisfactory past performance of riser guards, the present non-prescriptive approach to the design of these structures be retained. Explicitly quantifying vessel impact loads for riser guard design is considered not warranted, but will likely result in overdesigned riser guards and jackets.

## Evacuation

For a platform's exposure category to qualify as "manned, evacuated" it is presumed that sufficient resources are available and deployed in advance of a hurricane's impact in the Gulf of Mexico to accomplish full evacuation. The potential of shrinking evacuation resources (helicopters), and growing numbers of personnel further offshore, highlights the need to assure adequate resources.

## Platform Movements

First, for compliant platforms - either deepwater or minimum structures -there exist relative motions between the well conductors and the platform. It is important that these motions be recognized in the design and maintenance of piping and connections between the wellhead and deck-supported production manifold. Second, relative platform/riser or platform/J-tube motions can induce significant stresses in the short, stiff standoffs typically used to connect the riser of the J-tube to the platform, especially in more compliant platforms, and in standoffs that are directly welded (instead of clamped) to the jacket members. These stresses can cause premature fatigue failure. Third, the recording of earth-quake-induced motions on platforms located in seismic regions is strongly encouraged, given the present paucity of real data against which to measure the accuracy of numerical models. Fourth, it is recommended that similar deck motion recordings be collected on compliant 'minimum' structures, in order to: verify the modelling (accurate dynamic modelling can be critical to the survivability of these structures); improve the human perception graphs used for assessing motion effects on humans; and better determine the effects on motion-sensitive production equipment, such as unbaffled separators. And finally, for structures which may be susceptible to foundation subsidence or movement - platforms in active mudslide areas, for example - onboard tiltmeters could be useful in monitoring gradual changes with time, and help prevent catastrophic platform failure.

## Vibration Control, Weight Control

Vibrations from reciprocating equipment (compressors) can cause gas leaks and deck support frame cracking. Proper location on stiff decking is the first step to vibration control, followed by proper balancing or the use of dynamic absorbers. Deck support stiffening may help. It may also move the problem to an adjacent location. Fatigue cracking concerns must be communicated, from civil engineer to field personnel, and vice versa, and either permanently rectified or monitored through a documented, regular inspection program. Effective weight control can only be achieved through thorough, formal documentation of significant weight changes with time. Good communication between civil and operations personnel is crucial to achieving this end. An accurate reference or baseline weight takeoff is a necessary starting point. This should be performed by an engineer(s) experienced with structures and facilities. Walkthroughs on the platform by this engineer(s) may be necessary.

## Coatings

Continued maintenance of adequate corrosion protection of the platform steel at the waterline, and above water, is crucial to structural integrity preservation. Proper surface preparation and application during construction are necessary, especially on elements in the splash zone area. Special coating protection of risers should be considered. The continued development of coatings with reduced volatile organic compounds and isocyanates is encouraged. Communication between field personnel and civil engineers can help minimize expenses relating to maintenance painting in the field, to obtain adequate asset preservation at minimum lifecyle cost.

### Illegal Docking, Terrorism

Tie-up by pleasure craft and other boats to platforms or their mooring systems is illegal. Its prevention by law enforcement authorities (Coast Guard) is, first, encouraged, and, second, recognized as being ineffective. The threat of acts of terrorism, or the willful infliction of damage by non-industry parties, is recognized as real. It is also recognized that the prevention of the acts themselves is difficult. Ongoing vigilance on the part of all field personnel is encouraged.

## Operations Risk Issues

Topsides risk issues relating strictly to the operations of the platform are discussed in the following. First, how are topsides facilities assessed to assure continued fitness for purpose? And second, how are the results of these assessments integrated with the fitness for purpose assessments of the platform's structure, if at all?

### Topsides Facilities-Ongoing Assessment

Unlike the structure-related fitness for purpose question, there are, and have been for many years, significant industry programs and agency regulations in place that assure the ongoing fitness for purpose of topsides process facilities. These topsides safety management programs ensure that 'requalification' or assessment of topsides facilities is performed on an ongoing basis.

Topsides safety management programs have evolved over the years, as experience, and the prevailing regulatory and safety climates, dictate. These programs have become more conservative, more pro-active, more concerned about human and organizational errors, more focused on key safety concerns, and much more demanding in terms of required documentation and paperwork.

Current topsides safety management programs include:

API RP 14A through API RP 14G, recently enhanced by API RP 75, Recommended Practice for Development of a Safety and Environmental Management Program (SEMP) for Outer Continental Shelf (OCS) Operations and Facilities, First Edition, May, 1993, and by API RP 14J, Recommended Practice for Design and Hazards Analysis for Offshore Production Facilities, First Edition, September, 1993. These programs are in effect on a voluntary basis in U.S. coastal waters.

The Offshore Installations (Safety Case) Regulations, 1992, United Kingdom. These regulations are in effect in waters off the United Kingdom, and they are mandatory.

Regulations Concerning the Implementation and Use of Risk Analyses (1990), and Emergency Preparedness, in Petroleum Activities, Norwegian Petroleum Directorate, (1992). These regulations are in effect in Norwegian waters, and are mandatory.

The more recent documents have a common genesis—the Piper Alpha disaster. They each have a common structure. They have remarkably similar performance goals, whereby facilities with increasing complexity and increasing failure consequences are evaluated by and assessed to more complex analyses and more stringent standards. Each of the programs address hardware as well as human issues.

Based on 40 years of successful offshore production experience, the net result for offshore oil and gas processing is that compliance with these programs appears to result in topsides integrity that: is assessed on an ongoing basis, appears generally to be unrelated to age, generates process facilities that are fit for purpose.

As discussed, the scope and detail of process hazards analysis should be commensurate with the facilities' complexity and consequences.

The vast majority of production platforms in U.S. waters, by virtue of their layout and equipment makeup, have low escalation and consequence potential in the event of a process-related incident. There typically exists the ability for personnel to readily escape in the event of an incident (by jumping overboard, in most cases). The equipment is typically arranged on skids on 'open' decks with good ventilation.

For these types of production platforms, and where a rational political climate prevails, hazards analyses that are pragmatic, experienced based, check-list type evaluations, as allowed by SEMP are considered appropriate. Separate evaluations of topsides process safety and structural safety are also considered appropriate.

These types of production platforms can be found in the Gulf of Mexico, offshore California, and in many overseas waters, such as offshore Indonesia and West Africa.

For the small subset of 'high consequence' platforms - typically those with complex facilities processing high volumes of oil and natural gas in closed-in modules with little or no natural ventilation, often involving the handling of highly volatile natural gas liquids and $H_2S$ gas by-products, housing many persons onboard on an ongoing basis, often far from shore, in harsh climates (with the potential for extreme seas, ice, earthquakes or temperatures, potentially prohibiting personnel escape), often in environmentally pristine and sensitive areas, and sometimes in areas where the local community has a radical environmental element - a more rigorous and detailed hazards analysis is considered appropriate, commensurate to the potential consequences of failure.

In these cases, an extension of the check-list type hazards analysis to one that includes quantitatively evaluating the total risk associated with the operation of the platform may be appropriate. Such risk analyses can be helpful in assessing the relative distribution of risks, and thus in determining the most effective disposition of limited financial and human resources, using cost-benefit analysis. Total risk comprises threats to platform stability from fires, explosions, blowouts, ship collisions, and environmental overload.

Quantitative, total risk analysis (Bea and Craig 1993) is, however, generally not well understood at the present time, and there are no universally accepted quantitative risk targets or goals. As can be expected in an evolving area of new technology, the validity of the analysis depends to an extent on the knowledge and ability of the risk analyst, as well as the availability and quality of the input data. For these reasons, similar studies by different individuals may sometimes produce markedly different results.

No doubt there is room for improvement in the process safety management programs presently in place. As mentioned earlier, this issue may be worthy of a workshop of its own. To help facilitate discussion on this issue - focused solely on process safety - the Appendix comprises numerous questions and issues that are germane to the issue of production platform topsides process safety.

# Operational Considerations-Development Needs

The Operational Considerations Work Group recommends the following development work:

- Further compilation and dissemination of quality near-miss and accident data worldwide, to allow for the improved deployment of scarce resources, an improvement in the quality and consistency of both qualitative and quantitative risk analysis, the development of clearer safety goals, and the identification of high risk activities or operations. In particular, the following should receive more attention:

    Accurate and consistent near-miss and accident reporting within companies.

    Near-miss and accident investigation by trained investigators.

    The real need is not for a database for statistical manipulation, but to go beyond that with evaluations of incident cause and best remedy, to aid designers and equipment operators.

- Operators include structural safety assessments in their loss control and safety management programs. The API RP 75 SEMP element 8 Mechanical Integrity should be modified to read '...Assurance of quality and mechanical integrity of critical equipment and load bearing structures'. Further '...Critical equipment may include..., and load bearing structures.'

- More cross-referencing between the structure-related guidelines in API RP 2A, and the facility-related guidelines in the API RP 14's, API RP 75 and other drilling and production standards and recommended practices, to improve communication between the facility and civil/structural communities, especially communication between in-house cross-functional teams and support staff.
- The further development of disciplined, systematic evaluation techniques for the better control of human and organizational errors, in support of API 14J recommendations. It is particularly important to avoid the mind set which regards the cause of accidents as a result of equipment or system failure alone.

# References

American Petroleum Institute. Recommended Practice for Operation and Maintenance of Offshore Cranes. Draft Third Edition, Washington, D.C., January 1994.

Bea, R. G., and Craig, M. J. K. Developments in the Assessment and Requalification of Offshore Platforms. *Proceedings:* Offshore Technology Conference, OTC 7138, Houston, Texas, May 1993.

*Shut-in Systems*

*Data Collection*

# Appendix I

Risk associated primarily with the operations of the platform are discussed below - that is, risks not associated with structural overload, nor the operations/structure interface issues discussed earlier. The questions below are provided to help facilitate future discussion on this important topic.

## Operations Related Risks-Sources

The risk of the loss of serviceability of an operating production platform includes many components that are unrelated to storm, earthquake or ice overload. Figure 2.1 in API RP 14J Design and Hazards Analysis for Offshore Production Facilities illustrates some of these sources. The Figure attempts to identify operational sources which could lead to an event (pollution, fire, explosion, or injury).

Questions:

- Does this figure adequately represent risk sources for production facilities? Is there a better way of depicting significant sources?
- Can this figure be modified to include drilling and workover activities? How should it be modified or how can this be identified? Are blowouts age-related?
- Does the figure adequately represent risk sources from construction and maintenance activities (concurrent with production, or otherwise)? Can the figure be modified, or is there a better way to identify these sources? Is concurrent construction activity a major culprit?
- Should the figure be modified to include the potential effects of ship collisions and dropped objects, or should they be referenced elsewhere (draft API RP 2A, section 18, Fire, Blast & Accidental Loading)? Are dropped objects a real risk? Are riser guards effective?
- Should the figure include risk sources from organizational and human (operator) errors?
- Where are potential shortcomings in the deck structure (and even in the jacket structure) addressed or referenced (API RP 2A section 18, Fire, Blast & Accidental Loading)? Where is maintenance of adequate cathodic protection addressed? What about weight control?
- Which are the more important of these risks? Do we have adequate data? Do we need more data? Is age a factor? Is this figure relevant for the assessment of existing facilities?

The object is to identify significant risk sources (risk = consequence x likelihood), considering all aspects of an existing production operation - production, construction, maintenance, drilling, workover, etc., and to develop a logical framework for their further review. This may be in the form of API RP 14J's Figure 2.1, or something different.

## Operations Related Risks-Likelihoods and Mitigation by Design

The likelihood that a source will develop into an event may be a function of several design related factors, including the age of the facility, the complexity of the process, fluid properties (pressure, corrosivity), the degree of documentation of material and construction QA/QC, the codes and standards employed in design, the hazards analysis and safety case techniques employed, and whether the facilities are designed for manned or unmanned operations.

Questions:

- Can we identify design related factors which affect the likelihood of a source developing into an event?
- What quantitative or qualitative evidence can be used to aid in developing probabilities? What are the uses of failure databases (OREDA, etc.)?
3. Do cost/benefit relationships exist in determining the effort to expend on analyzing risks?

# Operations Related Risks-Mitigation by Safety Management

Safety management means those elements of managing the operations occurring on the platform which are generally unrelated to the specifics of the design itself. These elements are explained in API RP 75, and include:

Safety and Environmental Information
Hazards Analysis
Management of Change
Operating Procedures
Safe Work Practices
Training
Assurance of Quality and Mechanical Integrity of Critical Equipment
Pre-Startup Review
Emergency Response and Control
Investigation of Incidents
Audit of Safety and Environmental Management Program Elements

The extent to which these program elements are in place can affect the likelihood that a source will develop which could turn into an event.

Questions:

- Are other program elements needed?
- What objective evidence exists to show that these elements reduce risks?
- How to measure the degree of compliance with these elements?
- Hazards Analyses. What triggers such? What are their appropriate scopes and frequencies?
- Management of Change. How is this process implemented and audited? What are the right roles and responsibilities? Is this the key to aging-independence?
- Mechanical Integrity Maintenance. What are the focus areas, frequencies and scopes? Are risk-based inspection and maintenance (Asset Integrity Management) programs justified, especially for short-lived operations? How else to justify the I&M costs (show they result in significant risk reduction)? Does 'mechanical integrity' include that of the jacket? Who is responsible for the maintenance of subsea cathodic protection? What of the effects of new tie-ins of pressured production?
- How should the errors by organizations and by operators be quantified and better controlled? Should there be minimum, mandated training and retraining requirements for operators? What should they be?

# Operations Related Risks-Consequence Control Measures

The risk of a source developing into an event or the risk of a small event developing into a larger event can be reduced by mitigation and control methods. These methods can include layout choices, use of fire and blast walls, fire fighting and control philosophy, manning decisions, evacuation philosophy, structural design considerations, etc..

Questions:

- Which mitigation and control measures should be addressed?
- Does manning reduce or increase risks? To what extent does manning philosophy influence design? Is there a need for generic Safety, Health and Environment Management Systems? Is the issue safety or loss of a job?
- What are the goals of and design criteria for fire and explosion control/protection? What are the implications for structural design?
- Are there hard and fast rules for layout considerations other than those contained in API RP 14J?
- Have we learned anything from Safety Case studies that have general application? Now that a number of Safety Cases have been performed, is there a demonstrated rationale to continue to require that they be performed in all instances in the North Sea?

## Operations Related Risks-Potential Improvements

Improvements can evolve from a better understanding of operational risks and trade-offs. A better understanding of probabilities and consequences is needed.

Questions:
- How can we improve data on likelihoods in a cost effective manner? When is detailed probability analysis necessary?
- How can we improve data on effectiveness of mitigation/consequence control measures? When is detailed consequence analysis necessary?
- How can we better assess cost/benefit relationships for attention to activities which increase efforts devoted to safety analysis and documentation?
- How can operators better understand the full, integrated risk to their facility (operational and structural)?
- How can human error be better quantified and controlled?
- Are there new products that can significantly enhance mechanical integrity evaluation or preservation (eg. glass fiber reinforced epoxy in fire water systems)?

## Summary Observations

Risk sources related to the operation of production platforms are identified, as are the measures used to control them. Questions on how to improve on this control are listed.

These 'non-structural' risks appear to be more significant than those associated with the overload of the structure from environmental hazards.

# Discussion and Summary on

# CONSIDERATIONS AND CONSEQUENCES OF PUBLIC POLICY

Robert E. Kallman, *Minerals Management Service*
Richard McCarthy, *Seismic Safety Commission, California*
Allan Pulsipher, *Louisiana State University*
***Working Group #6***

## Working Group Objective

Discuss and summarize the public and private benefits as well as the economic consequences and political realism of the two approaches to the development of requalification policy identified in Bob Bea's "Workshop Charge," i.e., the "performance standards approach" being implemented in the North Sea (SISTS) and the "historic approach" (ABDF) exemplified by the API draft guidelines.

Fundamental Public Policy Question Regarding Requalification at Issue in the U.S.:

Is the "if it isn't broke (and wouldn't make much difference if it were) don't fix it" (ABDF) standard for requalification propounded in the API draft guidelines an acceptable basis for public policy?

### OR

Should a more inclusive and proactive approach resting on state-of-industry technological standards (SITS) like the one followed in the North Sea and similar to the best available technology (BACT) or best practicable technology (BPCT) approaches common in many EPA and OSHA regulations in the U.S. be used as the basis for requalification?

- Questions were discussed in a variety of contexts from a variety of perspectives.

- "Acceptable" has economic, political and regulatory dimensions.

## Summary Conclusion of Working Group #6

The ABDF approach outlined in the API draft guidelines is an "acceptable" and "viable" approach to requalification of offshore platforms in the United States, because it is:

- Practical, efficient and "economic"

- Flexible

- Credible

ABDF approach is practical, efficient and economic because:

- Operating experience, especially in the Gulf of Mexico with Hurricane Andrew, demonstrates that the vast majority of platforms are "not broke." Thus an expensive regulatory system which would expend scarce public and private resources to document this widely accepted "fact" would neither by prudent nor wise, but simply wastefully redundant.
- The attention and resources of both regulators and operators should be focused on platforms with the highest probability of, and most serious consequences from, failure. The screening procedure detailed in the API draft recommendation is a good way to accomplish this.

- Conversely, use of more comprehensive procedures such as the "SITS" or the "Safety Case" approach, in which each platform would be comprehensively reviewed to see if technologically defined, minimum conditions or practices were conformed to, would pose an unwise and unacceptable risk of a loss of production in the Gulf of Mexico.

ABDF is flexible because:

- It treats platforms posing more serious environmental consequences more carefully.

- It uses relatively cheap deterministic screening and analysis to make broad categorization but allows more expensive probabilistic, case-by-case, analyses to be used as necessary to avoid economic mistakes.

ABDF is credible because:

- Difference in treatment correspond to objectively verifiable differences in:

> conditions and

> environmental consequences

**BUT**

- Policy goals and responsibilities for the operator and the regulator are objectively consistent.

Major issues identified but not resolved:

- Although the ABDF approach embodied in the API guidelines is efficient, flexible and credible, important policy decisions will have to be made by MMS in order for API's guidelines to be come operational. Without these decisions by MMS, API's flow chart will not "flow." Among the major decisions MMS will have to make are:

> What is a 'significant" environmental impact?

Although the API guidelines sketch an acceptable conceptual criteria, the criteria are not discriminating enough to make the operational distinctions necessary to implement the guidelines. During working group discussions, MMS indicated that the research necessary to make such decisions was underway and that completion of this research would enable these determinations to be made.

> How should the requalification process be integrated with MMS's ongoing inspection process?

The requalification 'triggers' identified in the draft regulations were deemed sensible, but some ambiguity exists as to when and how the determination is to be made if a particular platform would activate a specific trigger. There was a general, if not unanimous agreement, that MMS should institute procedures to insure that all platforms are subjected to the API screening triggers within five years of their adoption.

Should the sale of a platform trigger a requalification?

How should industry and the interested public be consulted during the design and implementation phases of MMS's part of the requalification process?

There was general agreement that MMS will have to exercise considerable regulatory judgment in face of the inherent ambiguities of some aspects of the requalification process and that deterministic rules-of-thumb should not and could not be applied without the opportunity for regulatory review or recourse.

Peer review and modified CVA processes could improve the efficiency of any implementation of requalification procedures, but they should be applied early and interactively rather than ex post in a deterministic fashion.

- International Standards: The working group recognized the value of harmonization of requalification standards and practices internationally, but did not have the time to consider the public policy or regulatory aspects of this issue during the workshop.

- Concern about the "bad actor" problem, i.e., individual firms whose operating practices and policies consistently fall below minimal industry standards, have played an important role in the requalification dialogue and was discussed by our working group. It was agreed that further research to identify and deal with "bad actors" was warranted but the term should not, a priori, be applied indiscriminately to smaller or independent operators.

## Draft California Seismic Requalification Guidelines

Considerable portions of three separate working group sessions were allocated to draft seismic guidelines for California platforms prepared by Martain Eskijian of the California Lands Commissions and Leslie Monahan of the MMS Pacific Region. The discussion not only resulted in significant and substantive changes in the draft guidelines but served as a very practical case study that focused the discussion of the public policy aspects of requalification in a very concrete and practical way. The discussion below summarizes the consensus reached during the day and half of deliberations of Working Group #6, entitled "Policy Considerations and Consequences." Initially, the intent was to provide guidelines for high seismic areas, but much of the following information will apply to overall requalification of the aging fleet of fixed, offshore platforms in the Gulf of Mexico and the Pacific offshore region (California and Alaska).

- The fundamental policy decision is when should requalification be performed, or what is the 'trigger'. The following summarized the active and passive triggers for platform reassessment.

  The overall policy is that all existing platforms will be considered for structural reassessment. With the large number of platforms to be considered, the time during which the reassessment would be submitted to the regulators would be not more than five (5) years. This is an active trigger that will affect all fixed, offshore platforms.

  A number of secondary trigger mechanisms are also operational, and are basically in conformity with the Draft of Section 17 (American Petroleum Institute 1992). These triggers result from a significant change in the demand, capacity or consequence of the platform:

  Functional or operational changes, with higher loads than in the original design (e.g. water flood operations, additional tanks, etc.).

  Significant, unrepaired damage to primary member (s), following a Level II or higher inspection.

Acquisition of credible environmental data (wind, wave, current or seismic) that would indicate higher loads than postulated in the original design criteria.

Significant changes in the design criteria or methodologies. An example of this type of significant change is the tubular joint equations and their evolution.

- In terms of seismic reassessment for zones 3, 4, 5 (API seismic map), the following criteria are applicable:

  Platforms must meet the median, 1000 year return period seismic event, without loss of global structural stability. The selected seismic events must be site-specific.

  If the platform is unmanned (API 1993) and the pollution risk is nil, then the platform must meet the median, 500 year return period seismic event.

  If neither of the above can be satisfied, then a total risk, probabilistic assessment is required.

  Note:   Topsides and appurtenances must withstand these loads (RP 2A, Section 2.3.6e.2).

- The issue of peer review was discussed during the workshop, and the following consensus statement was obtained. However, the definition and full implementation of the proposed "modified CVA" program was not completely developed.

"For the seismic hazard assessment, a modified CVA approach is recommended, with interaction with the geotechnical team, from the beginning of the project."

The interactive CVA process was initially proposed in the THIC document (Iwan, et al. 1992), and it was agreed during this workshop that it was a good idea. The uncertainty associated with the seismic hazard assessment is large, and the regulator is without any easily obtained "tool" to verify the operator's values.

# References

American Petroleum Institute. RP 2A-WSD Section 17, Assessment of Existing Platforms, draft November 3, 1993.
Iwan, W.D.; Thiel, C.G.,Jr.; Housner, G.W.; and Allin, C. Cornell Seismic Safety Requalification of Offshore Platforms. American Petroleum Institute. May 1992.

conditions and

( ) environmental consequences

BUT

* Policy goals and responsibility for the issues raised in various ways throughout the document

Major issues identified but not resolved:

* Although the AUSI approach is described in the API draft as more coherent, flexible and credible, policy decisions will have to be made by MMS in order for API's guidelines to be come operational. These decisions by MMS "APIs first chart will magnifred". Among the major decision MMS must make are:

Although the API document tends to a sketchy description of the issues involved throughout. Ervels the operational features necessary to implement the guidelines. There are also a g

## ASSESSMENT OF EXISTING PLATFORMS

### 17.1 GENERAL

These guidelines are divided into separate sections describing assessment initiators, exposure categories, platform information necessary for assessment, the assessment process criteria/loads, design and ultimate strength level analysis requirements and mitigations. Several references [1-8] are noted which provide background, criteria basis, additional details and/or guidance including more specific technical references.

The guidelines in this Section are based on the collective industry experience gained to date and serve as a recommended practice for those who are concerned with the assessment of existing platforms to determine their fitness for purpose. The development of these guidelines is documented in *API RP2A DRAFT Section 17 Assessment of Existing Platforms*, by K. A. Digre, et al., BOSS '94, July, 1994 [1].

The guidelines herein are based on life safety and environmental risk. They do not include consideration of economic risk. The determination of an acceptable level of economic risk is left to the operator's discretion. It may be beneficial for an operator to perform explicit cost-benefit risk analyses in addition to simply using this recommended practice.

### 17.2 PLATFORM ASSESSMENT INITIATORS

An existing platform should undergo the assessment process if one or more of the conditions noted in Sections 17.2.1 through 17.2.5 exists.

Any structure which has been totally decommissioned (e.g., an unmanned platform with inactive flowlines and all wells plugged and abandoned) or is in the process of being removed (e.g., wells being plugged and abandoned) is not subject to this assessment process.

**17.2.1 Addition of Personnel.** If the manning condition (as defined in Section 17.3.1) is changed to a more restrictive level, the platform must be assessed.

**17.2.2 Addition of Facilities.** If the original operational loads on a structure or the level deemed acceptable by the most recent assessment are significantly exceeded by the addition of facilities (i.e., pipelines, wells, significant increase in topside hydrocarbon inventory capacity), the platform shall be assessed.

**17.2.3 Increased Loading on Structure.** If the structure is altered such that the new combined environmental/operational loading is significantly increased

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beyond the combined loadings of the original design using the original design criteria or the level deemed acceptable by the most recent assessments, the structure should be assessed. See Section 17.2.6 for definition of "significant".

**17.2.4 Inadequate Deck Height.** If the platform has an inadequate deck height for its exposure category (Ref. Sections 17.3 and 17.6.2, plus for GOM, Section 17.6.2a-2 and Figures 17.6.2-2b, 3b, 5b) and the platform was not designed for the impact of wave loading on the deck, the platform must be assessed.

**17.2.5 Damage Found During Inspections.** The assessment process may be used to assess the fitness for purpose of a structure when significant damage to a primary structural component is found during any inspection. This includes both routine and special inspections as required and defined in Section 14.4. Minor structural damage may be justified by appropriate structural analysis without performing a detailed assessment. However, the cumulative effects of damage must be documented and, if not justified as insignificant, be accounted for in the detailed assessment.

**17.2.6 Definition of Significant.** Cumulative damage or cumulative changes from the design premise are considered to be significant if the total of the resulting decrease in capacity due to cumulative damage and the increase in loading due to cumulative changes is greater than 10%.

## 17.3 EXPOSURE CATEGORIES

Structures should be assessed in accordance with the applicable exposure category and corresponding assessment criteria. Platforms are categorized according to life safety and environmental impact. Exposure categories for life safety are:

- *Manned, Non-Evacuated*
- *Manned, Evacuated*
- *Unmanned*

Exposure categories for environmental impact are:

- *Significant Environmental Impact*
- *Insignificant Environmental Impact*

This results in a potential for six combinations of platform exposure categories. Platforms categorized as *Unmanned, Insignificant Environmental Impact* are termed as having "minimum consequence."

### 17.3.1 Life Safety

**17.3.1a Manned, Non-Evacuated.** The *Manned, Non-Evacuated* category is a condition in which a platform is actually and continuously occupied by persons accommodated and living thereon, and it is not intended that they be evacuated during an environmental design event.

**17.3.1b Manned, Evacuated.** The *Manned, Evacuated* category is a condition in which a platform is normally manned except during a forecasted design

environmental event. For assessment purposes, a platform should be classified as a *Manned, Evacuated* platform if, prior to a design event, sufficient time exists to safely evacuate all personnel from the platform.

**17.3.1c Unmanned.** The *Unmanned* category is a condition in which a platform not normally manned, or a condition which is not classified as either *Manned, Non-Evacuated* or *Manned, Evacuated*.

### 17.3.2 Environmental Impact.

**17.3.2a Significant Environmental Impact.** A structure shall be placed in the *Significant Environmental Impact* (SEI) category if its assumed collapse can be projected to result in a liquid hydrocarbon or sour gas release which would cause an unacceptable impact on the environment. In order to determine the appropriateness of placing a structure in the SEI category, an environmental impact review should be performed. Such a review should consider all pertinent factors, including but not necessarily limited to:

- estimated volume of the release
- location and availability of containment equipment
- the proximity of environmentally sensitive areas, such as a coral reef, wildlife refuge and/or public beach

Except for those cases in which release of hydrocarbons or sour gas would not occur, no one factor should be considered alone when performing an environmental impact review.

**17.3.2b Insignificant Environmental Impact.** Structures not in the *Significant Environmental Impact* category are deemed to be in the *Insignificant Environmental Impact* category. Note that a platform may have potential for liquid hydrocarbon or sour gas release and still be categorized as *Insignificant Environmental Impact*. (See Section C17.3.2)

## 17.4 PLATFORM ASSESSMENT INFORMATION - SURVEYS

**17.4.1 General.** Sufficient information should be collected to allow an engineering assessment of a platform's overall structural integrity. It is essential to have a current inventory of the platform's structural condition and facilities. The operator should ensure that any assumptions made are reasonable and information gathered is both accurate and representative of actual conditions at the time of the assessment. Additional details can be found in C17.4.1 and in both An Integrated Approach for Underwater Survey and Damage Assessment of Offshore Platforms, by J. Kallaby and P. O'Conner, OTC 7487, May, 1994 [2] and *Structural Assessment of Existing Platforms,* by J. Kallaby, et al., OTC 7483, May, 1994 [3].

### 17.4.2 Surveys.

1.  Topside: The topside survey should, in most instances, only require the annual Level I survey as required in Section 14.3.1. The accuracy of the platform drawings should be verified when necessary. Where drawings are not available, or are inaccurate, additional walkaround surveys of the

topside structure and facilities may be required to collect the necessary information, i.e., topside arrangement and configuration, platform "Exposure Category" (see Section 17.3), structural framing details, etc.

2.  Underwater:  The underwater survey should, as a minimum, comprise a Level II survey (existing records or new survey), as required in Section 14.3.2.

    In some instances engineering judgment may necessitate additional Level III/Level IV surveys, as required in Sections 14.3.3/14.3.4, to verify suspected damage; deterioration due to age; lack of joint cans; major modifications; lack of/suspect accuracy of platform drawings; poor inspection records; or analytical findings. The survey should be planned by personnel familiar with inspection processes. The survey results should be evaluated by a qualified engineer familiar with the structural integrity aspects of the platform(s).

**17.4.3  Soil Data.** Available on or near site soil borings and geophysical data should be reviewed. Many older platforms were installed based on soil boring information a considerable distance away from the installation site. Interpretation of the soil profile may be improved based on more recent site investigations (with improved sampling techniques and inplace tests) performed for other nearby structures. More recent and refined geophysical data may also be available to correlate with soil boring data developing an improved foundation model.

## 17.5  ASSESSMENT PROCESS

**17.5.1  General.** The assessment process for existing platforms separates the treatment of life safety and environmental impact issues, and applies criteria that depend upon location and consequence. Additional details regarding the development and basis of this process can be found in *Process for Assessment of Existing Platforms to Determine Their Fitness for Purpose*, by W. Krieger, et al., OTC 7482, May, 1994 [4] with supporting experience in *A Comparison of Analytically Predicted Platform Damage to Actual Platform Damage During Hurricane Andrew*, by F. J. Puskar, et al., OTC 7473, May, 1994 [5].

There are six components of the assessment process:

    1 - Platform selection (Section 17.2)
    2 - Categorization (Section 17.3)
    3 - Condition assessment (Section 17.4)
    4 - Design basis check (Sections 17.5 and 17.6)
    5 - Analysis check (Sections 17.6 and 17.7)
    6 - Consideration of mitigations (Section 17.8)

The screening of platforms to determine which ones should proceed to detailed analysis is performed by executing the first four components of the assessment process.  If a structure does not pass screening, there are two potential sequential analysis checks:

    1 - Design Level Analysis
    2 - Ultimate Strength Analysis

The design level analysis is a simpler and more conservative check, while the ultimate strength analysis is more complex and less conservative. It is generally more efficient to begin with a design level analysis, only proceeding with ultimate strength analysis as needed. However, it is permissible to bypass the design level analysis and to proceed directly with an ultimate strength analysis. If an ultimate strength analysis is required it is recommended to start with a linear global analysis (Section 17.7.3a), only proceeding to a global inelastic analysis (Section 17.7.3c) if necessary.

Note that mitigation alternatives (Section 17.8) such as platform strengthening, repair of damage, load reduction, or changes in exposure category, can be considered at any stage of the assessment process.

In addition, the following are acceptable alternative assessment procedures subject to the limitations noted in C17.5.1:

1.  Assessment of similar platforms by comparison.

2.  Assessment through the use of explicit probabilities of failure.

3.  Assessment based on prior exposure, surviving actual exposure to an event that is known with confidence to have been as severe or more severe than the applicable ultimate strength criteria based on exposure category.
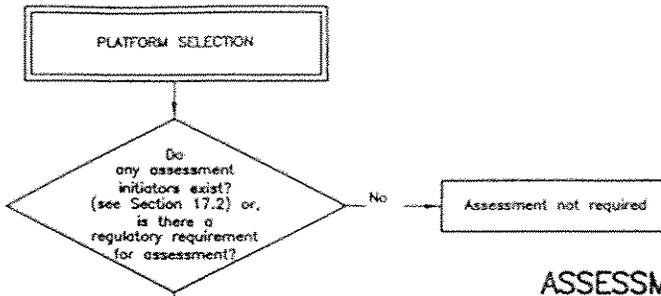
Assessment procedures for metocean, seismic, and ice loading are defined in Sections 17.5.2, 17.5.3, and 17.5.4, respectively.

**17.5.2 Assessment For Metocean Loading.** The assessment process for metocean loading is shown in Figure 17.5.2. A different approach to defining metocean criteria is taken for Gulf of Mexico platforms than for other locations. For the Gulf of Mexico, design level and ultimate strength metocean criteria are explicitly provided, including wave height vs. water depth curves. For other areas, metocean criteria are specified in terms of factors relative to loads caused by 100-year environ- mental conditions. The reserve strength ratio (RSR) is used as a check of ultimate strength. RSR is defined as the ratio of a platform's ultimate lateral load carrying capacity to its 100-year environmental condition lateral loading, computed using present RP2A procedures. Further discussion of metocean criteria is provided in section 17.6.

Platforms that have no significant damage, have an adequate deck height for their category (Ref. Figures 17.6.2-2b, 3b, 5b), and have not experienced significant changes from their design premise may be considered to be acceptable, subject to either of the following two conditions:

1.  **Minimum Consequence:** If the platform is categorized as having minimum consequence, the platform passes the assessment.

2.  **Design Basis Check:** If the platform is located in the Gulf of Mexico and was designed to the 9th edition of RP2A (1977), or later, the platform passes the assessment. However, in this case it must also be demonstrated that reference level hydrodynamic loading was used for platform design. The procedure to demonstrate that 9th edition reference level forces were

# PLATFORM ASSESSMENT PROCESS — METOCEAN LOADING

```
┌─────────────────────────────┐
│     PLATFORM SELECTION       │
└─────────────────────────────┘
              │
              ▼
        ╱ Do          ╲
      ╱ any assessment   ╲        No      ┌──────────────────────────┐
    ╱ initiators exist?    ╲──────────────▶│  Assessment not required  │
    ╲ (see Section 17.2) or,╱              └──────────────────────────┘
      ╲ is there a         ╱
        ╲ regulatory      ╱
          ╲ requirement  ╱
            ╲ for assessment? ╱
              │
             Yes
              │
              ▼
┌─────────────────────────────┐
│      CATEGORIZATION          │
│    (see Section 17.3)        │
└─────────────────────────────┘
```

## ASSESSMENT CRITERIA — GULF OF MEXICO
(see Table 17.6.2–1)

| Exposure Category | | Design Level Analysis (see Notes 1 and 2) | Ultimate Strength Analysis |
|---|---|---|---|
| Sig. Env. Impact | Manned, Evac. | Environmental safety design level analysis loading (see Figure 17.6.2–2) | Environmental safety ultimate strength analysis loading (see Figure 17.6.2–2) |
| | Unmanned | | |
| Insig. Env. Impact | Manned, Evac. | Sudden hurricane design level analysis loading (see Figure 17.6.2–3) | Sudden hurricane ultimate strength analysis loading (see Figure 17.6.2–3) |
| | Unmanned | Minimum consequence design level analysis loading (see Figure 17.6.2–5) | Minimum consequence ultimate strength analysis loading (see Figure 17.6.2–5) |

Table 17.5.2a

## ASSESSMENT CRITERIA — OTHER US AREAS

| Exposure Category | | Design Level Analysis (see Notes 1 and 2) | Ultimate Strength Analysis |
|---|---|---|---|
| Sig. Env. Impact | Manned, Non-Evac. | 85% of lateral loading caused by 100-year environmental conditions (see Section 17.6.2b) | Reserve strength ratio (RSR) ≥ 1.6 (see Section 17.6.2b) |
| | Unmanned | | |
| Insig. Env. Impact | Manned, Non-Evac. | | |
| | Unmanned | 50% of lateral loading caused by 100-year environmental conditions (see Section 17.6.2b) | RSR ≥ 0.8 (see Section 17.6.2b) |

Table 17.5.2b

Notes: (1) Design level check not applicable for platforms with inadequate deck height.

(2) One-third increase in allowable stress is permitted for design level analysis (all categories)

```
┌─────────────────────────────┐
│  Exposure category – Life safety: │
│      Environmental Impact    │
└─────────────────────────────┘
              │
              ▼
┌─────────────────────────────┐
│         Life Safety          │
│  • Manned, Non-Evacuated     │
│  • Manned, Evacuated         │
│  • Unmanned                  │
└─────────────────────────────┘
              │
              ▼
┌─────────────────────────────┐
│     Environmental Impact     │
│  • Significant Environmental │
│    Impact                    │
│  • Insignificant Environmental│
│    Impact                    │
└─────────────────────────────┘
              │
              ▼
┌─────────────────────────────┐
│    CONDITION ASSESSMENT      │
│    (see Section 17.4)        │
└─────────────────────────────┘
              │
              ▼
        ╱ Is          ╲
      ╱ platform damaged, ╲   No        ╱ Is       ╲   No    ┌──────────────────┐
    ╱ deck height inadequate, ╲─────────╱ platform    ╲───────▶│ DESIGN BASIS CHECK│
    ╲ or has loading increased? ╱       ╲ unmanned:    ╱       └──────────────────┘
      ╲ (see Sections 17.6,   ╱          ╲ insig. env. ╱              │
        ╲ 17.7)              ╱            ╲ impact?   ╱               ▼
              │                             │                  ╱ Is        ╲  No
             Yes                           Yes               ╱ platform location ╲────┐
              │                             │                ╲ G O M ?        ╱      │
              │                             │                  ╲            ╱        │
              │                             │                     Yes                │
              │                             ▼                      │                 │
              │                  ┌──────────────────┐              ▼                 │
              │                  │    Platform       │          ╱ Is         ╲       │
              │                  │    passes    ◀────┼──── Yes ─╱ platform      ╲     │
              │                  │  assessment       │          ╲ designed to 9th ╱   │
              │                  └──────────────────┘           ╲ ed. or later with╱  │
              │                                                  ╲ reference level ╱   │
              │                                                   ╲ environmental ╱    │
              │                                                    ╲ loading? (see╱     │
              │                                                     ╲ Section 17.6)      │
              │                                                        │                │
              │                                                       No ───────────────┘
              ▼
            ( A )
```

Figure 17.5.2

# PLATFORM ASSESSMENT PROCESS - METOCEAN LOADING



```
                              ( A )
                                |
          ┌─────────────────────────────────────┐
          │         ANALYSIS CHECKS              │
          │  All analysis to be conducted using  │
          │  present RP 2A procedures, as        │
          │  modified in Section 17.7            │
          └─────────────────────────────────────┘
                                |
          ┌─────────────────────────────────────┐        ┌──────────────┐
          │        Design Level Analysis         │        │  Platform    │
          │                              Passes  │────────│  passes      │
          │  Perform design level analysis       │        │  assessment  │
          │  applying proper loading from Table  │        └──────────────┘
          │  17.5.2a, b                          │
          │  (see Notes 1, 2 and Section 17.7)   │
          └─────────────────────────────────────┘
                              Fails
                                |
                            ◇ Implement
                  Yes ─────  mitigation alternatives ?
                            (see Section 17.8)
                                |
                               No
                                |
          ┌─────────────────────────────────────┐        ┌──────────────┐
          │       Ultimate Strength Analysis     │        │  Platform    │
          │                              Passes  │────────│  passes      │
          │  Perform ultimate strength analysis  │        │  assessment  │
          │  applying proper loading from        │        └──────────────┘
          │  Table 17.5.2a, b (see Section 17.7) │
          └─────────────────────────────────────┘
                              Fails
                                |
                            ◇ Implement
                  Yes ─────  mitigation alternatives ?
                            (see Section 17.8)
                                |
                               No
                                |
                        ┌──────────────┐
                        │  Platform    │
                        │  does not pass│
                        │  assessment  │
                        └──────────────┘
```

Figure 17.5.2 (continued)

applied during design is described in Section 17.6.

Significant damage or change in design premise is defined in Section 17.2.6. For all other platforms, the following applies:

3.   **Design level analysis:** Design level analysis procedures are similar to those for new platform design, including the application of all safety factors, the use of nominal rather than mean yield stress, etc. Reduced metocean loading, relative to new design requirements, are referenced in Figure 17.5.2 and Section 17.6. Design level analysis requirements are described in Section 17.7.2. For minimum consequence platforms with damage or increased loading, an acceptable alternative to satisfying the design level analysis requirement is to demonstrate that the damage or increased loading is not significant relative to the as-built condition, as defined in Section 17.2.6. This would involve design level analyses of both the existing and as-built structures.

4.   **Ultimate strength analysis:** Ultimate strength analysis reduces conservatism, attempting to provide an unbiased estimate of platform capacity. The ultimate strength of a platform may be assessed using inelastic, static pushover analysis. However, a design level analysis with all safety factors and sources of conservatism removed is also permitted, as this provides a conservative estimate of ultimate strength. In both cases the ultimate strength metocean criteria should be used. Ultimate strength analysis requirements are described in Section 17.7.3. For minimum consequence platforms with damage or increased loading, an acceptable alternative to the ultimate strength requirement is to demonstrate that the damage or increased loading is not significant relative to the as-built condition as defined in Section 17.2.6. This would involve ultimate strength analyses of both the existing and as-built structures.

Several investigators have developed simplified procedures for evaluation of the adequacy of existing platforms. To use these procedures successfully requires intimate knowledge of the many assumptions upon which they are based, as well as a thorough understanding of their application. Environmental loadings used in simplified analysis are at the discretion of the operator; however, the simplified analysis method used must be validated as being more conservative than the design level analysis.

**17.5.3 Assessment For Seismic Loading.** For platforms with exposure categories noted in section 17.3 (excluding the non-applicable *Manned-Evacuated* category) that are subject to seismic loading in seismic zones 3, 4 and 5 (see Section C2.3.6c), the basic flow chart shown in Fig. 17.5.2 is applicable to determine fitness for seismic loading with the following modifications:

1.   Assessment for seismic loading is <u>not</u> a requirement for seismic zones 0, 1 and 2 (see Section C2.3.6c);

2.   Assessment for metocean loading should be performed for all seismic zones.

3.   Perform assessment for ice loading if applicable.

4.  Design Basis Check:  For all exposure categories defined in Section 17.3,

    platforms that have been designed or recently assessed in accordance with the requirements of API RP 2A, 7th Edition (1976) which required Safety Level Analysis (referred to as Ductility Level Analysis in subsequent editions), are considered to be acceptable for seismic loading, provided that:

    - No new significant fault has been discovered in the area.
    - No new data indicate that a current estimate of strength level ground motion for the site would be significantly more severe than the strength level ground motion used for the original design.
    - Proper measures have been made to limit the life safety risks associated with platform appurtenances as noted in Section 2.3.6e.2.
    - The platforms have no significant damage,
    - The platforms have been surveyed,
    - The present and/or anticipated payload levels are less than or equal to those used in the original design.

5.  Design Level Analysis:  The design level analysis box in Fig. 17.5.2 is not applicable to seismic assessment (see Section 17.6.3).

6.  Ultimate Strength Analysis: *Manned, Non-Evacuated* platforms and/or *Significant Environmental Impact* platforms that do not meet the screening criteria may be considered adequate for seismic loading provided they meet the life safety requirements associated with platform appurtenances as noted in Section 2.3.6e.2, and it can be suitably demonstrated by dynamic analysis using best estimate resistances that these platforms can be shown to withstand loads associated with a median 1,000-year return period earthquake appropriate for the site without system collapse.

    In the case of *Unmanned, Insignificant Environmental Impact* platforms, in addition to satisfying the platform appurtenance requirements of Section 2.3.6e.2, it must be suitably demonstrated by dynamic analysis using best estimate resistance values that the platform can withstand earthquake loads associated with a median 500-year return period event appropriate for the site without system collapse.

A validated simplified analysis may be used for seismic assessment (Ref. Section 17.5.2).  It must be demonstrated that the simplified analysis will be more conservative than the Ultimate Strength Analysis.

17.5.4 Assessment For Ice Loading.  For all exposure categories of platforms subject to ice loading, the basic flowchart shown in Fig. 17.5.2 is applicable to determine fitness for ice loading with the following modifications:

1.  Perform assessment for metocean loading if applicable.  Note this is not required for Cook Inlet, Alaska as ice forces dominate.

2.  Perform assessment for seismic loading if applicable.

3.  Design Basis Check:  All categories of platforms as defined in Section

17.3 that have been maintained and inspected, have had no increase in design level loading, are undamaged and were designed or previously assessed in accordance with API-RP 2N, First Edition (1988) or later are considered to be acceptable for ice loading.

4. Design Level Analysis: *Significant Environmental Impact* and/or *Manned* platforms that do not meet the screening criteria may be considered adequate for ice loading if they meet the provision of API-RP2N (1st edition 1988) using a linear analysis with the basic allowable stresses referred to in Section 3.1.2 increased by one-half.

   *Unmanned, Insignificant Environmental Impact* platforms that do not meet the screening criteria may be considered adequate for ice loading if they meet the provision of API RP 2N (First Edition) using a linear analysis, with the basic allowable stresses referred to in Section 3.1.2 increased by 70% which is in accordance with Sections 2.3.6.c4 and 2.3.6e.

5. Ultimate Strength Analysis: Platforms that do not meet the design level analysis requirements may be considered adequate for ice loading if an ultimate strength analysis is performed using best estimate resistances, and the platform is shown to have a Reserve Strength Ratio (RSR) equal to or greater than 1.6 in the case of *Manned, Non-Evacuated* and/or *Significant Environmental Impact* platforms, and a RSR equal to or greater than 0.8 in the case of *Insignificant Environmental Impact* platforms that are either *Manned-Evacuated* or *Unmanned*. RSR is defined as the ratio of platform ultimate lateral capacity to the lateral loading computed with present API RP 2N (1st edition, 1988) procedures using the design level ice feature provided in Section 3.5.7 of RP 2N.

A validated simplified analysis may be used for assessment of ice loading (Ref. Section 17.5.2). It must be demonstrated that the simplified analysis will be as or more conservative than the design level analysis.

## 17.6 METOCEAN, SEISMIC AND ICE CRITERIA/LOADS

**17.6.1 General.** The criteria/loads to be utilized in the assessment of existing platforms should be in accordance with Section 2.0 with the exceptions, modifications and/or additions noted herein as a function of exposure category defined in section 17.3 and applied as outlined in Section 17.5.

**17.6.2 Metocean Criteria/Loads.** The metocean criteria consist of the following items:

- Omni-directional wave height vs. water depth
- Storm tide (storm surge plus astronomical tide)
- Deck height
- Wave and current direction
- Current speed and profile
- Wave period
- Wind speed

The criteria are specified according to geographical region. At this time, only

criteria for the Gulf of Mexico and three regions off the U. S. West Coast are provided. These regions are Santa Barbara and San Pedro Channels plus Central California (for platforms off Point Conception and Arguello). No metocean criteria are provided for Cook Inlet because ice forces dominate.

The criteria are further differentiated according to exposure category (environmental impact and life safety category combination) and type of analysis (design level or ultimate strength).

Wave/wind/current force calculation procedures for platform assessment have to consider two cases:

- Wave clears the underside of the cellar deck;
- Wave inundates the cellar deck, ultimate strength analyses must be performed;

For Case 1, the criteria are intended to be applied with wave/wind/current force calculation procedures specified in Sections 2.3.1-2.3.4, except as specifically noted in Section 17.6.2.

For Case 2, the procedures noted in Case 1 apply in addition to the special procedures for calculating the additional wave/current forces on platform decks, provided in Section C17.6.2.

The following sections define the guideline metocean criteria and any special force calculation procedures for various geographical regions. Platform owners may be able to justify different metocean criteria for platform assessment than the guideline criteria specified herein. However, these alternative criteria must meet the following conditions:

- Criteria must be based on measured data in winter storms and/or hurricanes, or on hindcast data from numerical models and procedures that have been thoroughly validated with measured data.

- Extrapolation of storm data to long return periods and determination of "associated" values of secondary metocean parameters must be done with defensible methodology.

- Derivation of metocean criteria for platform assessment must follow the same logic as used to derive the guideline parameters provided herein. This logic is explained in *Metocean Criteria/Loads for use in Assessment of Existing Offshore Platforms*, by C. Petrauskas, et al., OTC 7484, May, 1994 [6].

17.6.2a Gulf of Mexico Criteria.

1. Metocean Systems:

    Both hurricanes and winter storms are important to the assessment process. In calculating wave forces based on Section 2.3, a wave kinematics factor of 0.88 should be used for hurricanes and 1.0 for winter storms.

2.    Deck Height Check:
The deck heights shown in Figures 17.6.2-2b, 17.6.2-3b, and 17.6.2-5b are based on the ultimate strength analysis metocean criteria for each of the exposure categories. Specifically, the minimum deck height above MLLW measured to the underside of the cellar deck main beams is calculated as follows:

Minimum deck height = crest height of ultimate strength analysis wave height and associated wave period + ultimate strength analysis storm tide.

The wave crest heights are calculated using the wave theory as recommended in Section 2.3.1b.2.

If this criterion for the minimum deck height is not satisfied then an ultimate strength analysis must be conducted with proper representation of hydrodynamic deck forces using the procedure described in Section C17.6.2.

3.    **Design Basis Check (for structures designed to the 9th Edition or later):**

All exposure categories: A single vertical cylinder may be used to determine if the platform satisfies the 9th edition reference level force. Figure 17.6.2-1 shows the 9th edition wave forces as a function of water depth for diameters of 30", 48", 60", and 72". The forces are calculated using the wave theory as recommended in Section 2.3.1b.2. Consistent with the 9th edition, the current is zero and no marine growth is used. The drag coefficient is 0.6 and the inertia coefficient is 1.5.

To verify that the platform was designed for 9th edition reference level loads, the forces on the single cylinder need to be calculated using the original design wave height, wave period, current, tide, drag and inertia coefficients, wave-plus-current kinematics, and marine growth thickness. The cylinder diameter should be equal to the platform leg diameter at the storm mean water level. If the forces are equal to or exceed that in Figure 17.6.2-1, then the platform forces are considered consistent with 9th edition requirements.

A more accurate approach is to build a hydrodynamic model of the structure and compare the base shear using the original design criteria with the base shear that is consistent with the 9th edition reference level force. The 9th edition forces should be calculated using the wave theory as recommended in Section 2.3.1b.2.

4.    Design Level and Ultimate Strength Analyses:

a.    *Significant Environmental Impact/Manned, Evacuated or Unmanned.* The full hurricane population applies. The metocean criteria are provided in Table 17.6.2-1. The wave height and storm tide are functions of water depth; these are given in Figure 17.6.2-2a. The minimum deck height is also a function of water depth; this is shown in Figure 17.6.2-2b. The wave period, current speed, and wind speed do not depend on water depth; these are provided in Table 17.6.2-1.

# Table 17.6.2-1

## GULF OF MEXICO METOCEAN CRITERIA

| CRITERIA | SIGNIFICANT ENVIRONMENTAL IMPACT/MANNED-EVACUATED OR UNMANNED — FULL POPULATION HURRICANES | | INSIGNIFICANT ENVIRONMENTAL IMPACT/MANNED-EVACUATED — SUDDEN HURRICANES | | INSIGNIFICANT ENVIRONMENTAL IMPACT/UNMANNED (MINIMUM CONSEQUENCE) — WINTER STORMS | |
|---|---|---|---|---|---|---|
| | Design Level Analysis | Ultimate Strength Analysis | Design Level Analysis | Ultimate Strength Analysis | Design Level Analysis | Ultimate Strength Analysis |
| Wave Ht & Storm Tide, ft | Fig. 17.6.2-2a | | Fig. 17.6.2-3a | | Fig. 17.6.2-5a | |
| Deck Height, ft | Fig. 17.6.2-2b | | Fig. 17.6.2-3b | | Fig. 17.6.2-5b | |
| Wave & Current Direction | *Omni-Dir. | 20-th Ed. | **Omni-Dir. | Fig. 17.6.2-4 | Omni-Dir. | Omni-Dir. |
| Current Speed, kts | 1.6 | 2.3 | 1.2 | 1.8 | 0.9 | 1.0 |
| Wave Period, sec | 12.1 | 13.5 | 11.3 | 12.5 | 10.5 | 11.5 |
| Wind Spd(1-hr@10m), kts | 65 | 85 | 55 | 70 | 45 | 50 |

* If the wave height or current vs direction exceeds that required by the 20th ed for new designs, then the 20th ed criteria will govern.

** If the wave height or current vs direction exceeds that required for ultimate strength analysis, then the ultimate strength criteria will govern.

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TABLE 17.6.2-2

## 100-YR METOCEAN CRITERIA FOR PLATFORM ASSESSMENT
## US WATERS (OTHER THAN GULF OF MEXICO), DEPTH > 300 FT

| SANTA BARBARA CH | Wave Height, ft | Current, kts | Wave Period, sec | Storm Tide, ft | Wind Speed, kts (1-hr @ 33 ft) |
|---|---|---|---|---|---|
| 120° 30' W | 50 | 1 | 14 | 6 | 55 |
| 120° 15' W | 43 | 1 | 13 | 6 | 50 |
| 120° 00' W | 39 | 1 | 12 | 6 | 50 |
| 119° 45' W and further east | 34 | 1 | 12 | 6 | 45 |

| SAN PEDRO CH. | Wave Height, ft | Current, kts | Wave Period, sec | Storm Tide, ft | Wind Speed, kts (1-hr @ 33 ft) |
|---|---|---|---|---|---|
| 118° 00' to 118° 15' | 43 | 1 | 13 | 6 | 50 |

| CENTRAL CALIFORNIA | Wave Height, ft | Current, kts | Wave Period, sec | Storm Tide, ft | Wind Speed, kts (1-hr @ 33 ft) |
|---|---|---|---|---|---|
| West of Pt Conception | 56 | 1 | 14 | 7 | 60 |
| West of Pt Arguello | 60 | 1 | 14 | 7 | 65 |

BASE SHEAR, KIPS

200

150

100

50

0

10    100    1000

MLLW, FT

30" OD Cylinder
48"OD Cylinder
60" OD Cylinder
72" OD Cylinder

**Fig 17.6.2-1  Base Shear for a Vertical Cylinder Based on
API-RP2A 9th Ed Reference Level Forces**

Fig. 17.6.2-2a  Full Population Hurricane Wave Height and Storm Tide Criteria

Fig. 17.6.2-2b  Full Population Deck Height Criteria

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Fig. 17.6.2-3a  Sudden Hurricane Wave Height and Storm Tide Criteria

Fig. 17.6.2-3b  Sudden Hurricane Deck Height Criteria

Fig. 17.6.2-4 Sudden Hurricane Wave Directions and Factors to Apply to the Omnidirectional Wave Heights in Fig. 17.6.2-3a for Ultimate Strength Analysis
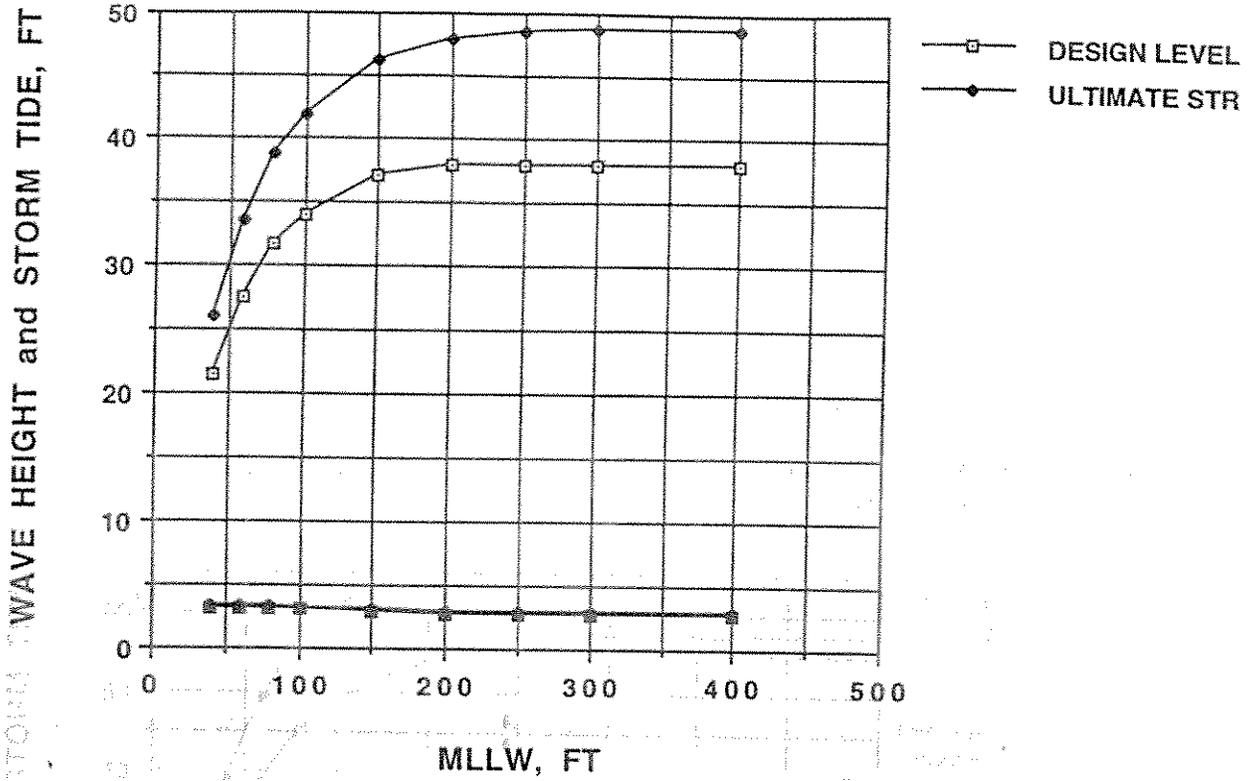
Fig. 17.6.2-5a   Winter Storm Wave Height and Storm Tide Criteria
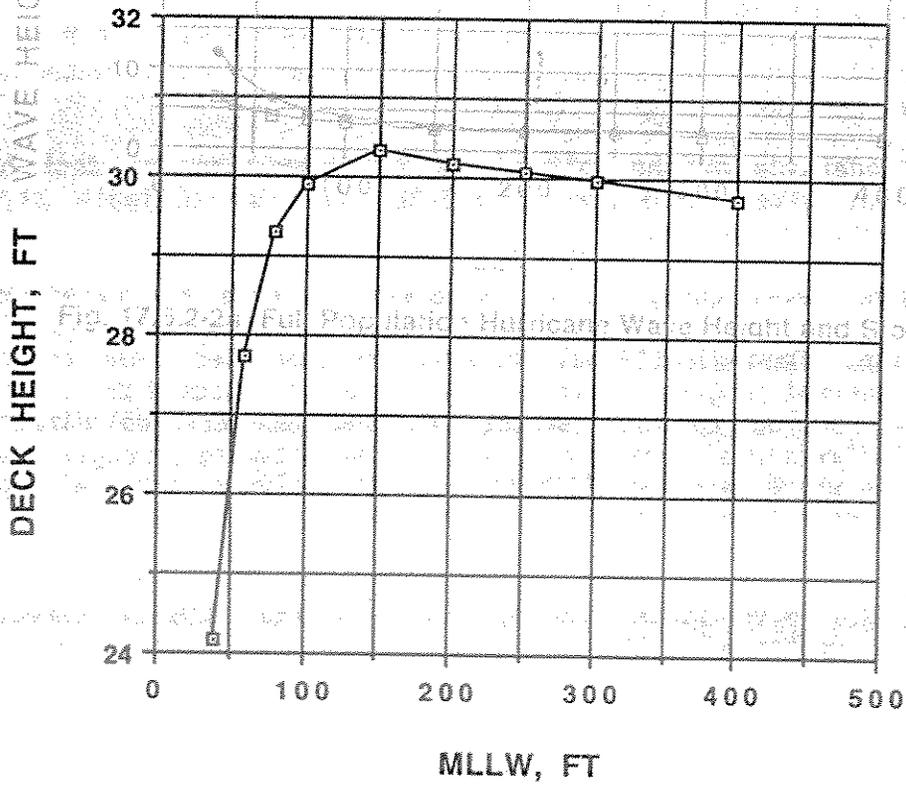


Fig. 17.6.2-5b   Winter Storm Deck Height Criteria

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If the underside of the cellar deck is lower than the deck height requirement given in Figure 17.6.2-2b, then an _ultimate strength analysis_ will be required.

For design level analysis, omni-directional criteria are specified. The associated in-line current is given in Table 17.6.2-1 and is assumed to be constant for all directions and water depths. For some non-critical directions, the omni-directional criteria may exceed the design values of this recommended practice, in which case the values of this recommended practice will govern for those directions. The current profile is given in Section 2.3.4c.4. The wave period, storm tide, and wind speed apply to all directions.

For ultimate strength analysis, the directionality of the waves and currents should be taken into account. The wave height and current speed direction factor and the current profile should be calculated in the same manner as described in Section 2.3.4c.4. The wave period and wind speed do not vary with water depth. Wave/current forces on platform decks should be calculated using the procedure defined in Section C17.6.2.

b. _Insignificant Environmental Impact/Manned-Evacuated._ The combined sudden hurricane and winter storm population applies. The metocean criteria (referenced to the sudden hurricane population) are provided in Table 17.6.2-1. The wave height and storm tide are functions of water depth; these are shown in Figure 17.6.2-3a. The required deck height is also a function of water depth; this is given in Figure 17.6.2-3b. The wave period, current speed, and wind speed do not vary with water depth; these are provided in Table 17.6.2-1.

If the underside of the cellar deck is lower than the deck height requirement given in Figure 17.6.2-3b, then an _ultimate strength analysis_ will be required.

For design level analysis, the metocean criteria are based on the 100-year force due to the combined sudden hurricane and winter storm population. Omni-directional criteria are specified. The associated in-line current is given in Table 17.6.2-1 and is assumed to be constant for all directions and water depths. For some non-critical directions, the omni-directional criteria may exceed the ultimate strength analysis values, in which case the ultimate strength analysis values will govern for those directions. The current profile is given in Section 2.3.4c.4. The wave period, storm tide, and wind speed apply to all directions. Although the criteria are based on both sudden hurricanes and winter storms, the wave forces should be calculated using a directional spreading factor of 0.88 because the criteria are referenced to the sudden hurricane population.

For ultimate strength analysis, the directionality of the waves and currents should be taken into account. The wave height, associated current and profile, as a function of direction, should be calculated in the same manner as described in Section 2.3.4c.4., _except that the_

<u>directional factors should be based on Figure 17.6.2-4</u>. The wave period and wind speed do not vary with water depth. Wave/current forces on platform decks should be calculated using the procedure defined in Section C17.6.2.

c. *Insignificant Environmental Impact/Unmanned* (Minimum Consequence). The winter storm population applies. The metocean criteria are provided in Table 17.6.2-1. The wave height and storm tide are functions of water depth; these are shown in Figure 17.6.2-5a. The required deck height is also a function of water depth; this is given in Figure 17.6.2-5b. The wave period, current speed, and wind speed do not vary with water depth; these are provided in the Table 17.6.2-1..

If the underside of the cellar deck is lower than the deck height requirement given in Figure 17.6.2-5b, then an <u>ultimate strength analysis</u> will be required.

For both design level and ultimate strength analysis, the wave height criteria are omni-directional. The associated in-line current is provided in Table 17.6.2-1 and is assumed to be constant for all directions and water depths. The current profile should be the same as in Section 2.3.4c.4. The wave period, storm tide, and wind speed apply to all directions. Wave/current forces on platform decks should be calculated using the procedure defined in Section C17.6.2.

## 17.6.2b West Coast Criteria.

### 1. Metocean Systems:

The extreme waves are dominated by extratropical storm systems. In calculating wave forces based on Section 2.3, a directional spreading factor of 1.0 should be used.

### 2. Deck Height Check:

The deck height for determining whether or not an ultimate strength check will be needed should be developed on the same basis as prescribed in Section 17.6.2a.5. The ultimate strength wave height should be determined on the basis of the acceptable RSR. The ultimate strength storm tide may be lowered from that in Table 17.6.2-2 to take into account the unlikely event of the simultaneous occurrence of highest astronomical tide and ultimate strength wave.

### 3. Design Basis Check:

Only applicable to Gulf of Mexico platforms.

### 4. Design Level and Ultimate Strength Analysis:

Table 17.6.2-2 presents the 100-yr metocean criteria necessary for performing design level and ultimate strength checks. An ultimate strength check will be needed if the platform does not pass the design

level check or if the deck height is not adequate.

The criteria are for deep water (>300 ft) and should be applied omni-directionally. Lower wave heights, provided they are substantiated with appropriate computations, could be justified for shallower water.

**17.6.3 Seismic Criteria/Loads.** Guidance on the selection of seismic criteria and loading is provided in Sections 2.3.6 and C2.3.6. Additional details can be found in *Assessment of High Consequence Platforms - Issues and Applications*, by M. J. K. Craig and K. A. Digre, OTC 7485, May, 1994 [7].

1.  The design basis check procedures noted in Section 17.5.3.4 are only appropriate provided no significant new faults in the local area have been discovered or any other information regarding site seismic hazard characterization has been developed that significantly increases the level of seismic loading used in the platform's original design.

2.  For seismic assessment purposes, the design level check is felt to be an operator's economic risk decision and thus is not applicable. An Ultimate Strength Analysis is required if the platform does not pass the design basis check or screening.

3.  Ultimate strength criteria is set at a median 1000-year return period event for all platforms except those classified as minimum consequence. For the minimum consequence structures a median 500-year return period event should be utilized. Characteristics of these seismic events should be based on the considerations noted in Sections 2.3.6 and C2.3.6 as well as any other significant new developments in site seismic hazard characterization. The Ultimate Strength Criteria should be developed for each specific site or platform vicinity using best available technology.

**17.6.4 Ice Criteria/Loads.** Guidance on the selection of appropriate ice criteria and loading can be found in API RP2N, first edition, 1988. Note that the ice feature geometries provided in Section 3.5.7 of RP 2N are not associated with any return period since no encounter statistics are presented. All references to Screening, Design Level and Ultimate Strength Analyses in Section 17.5.4 assume the use of the values noted in Table 3.5.7 of RP 2N. Where ranges are noted, the smaller number could be related to design level and the larger related to ultimate strength. Additional details can be found in *Assessment of High Consequence Platforms - Issues and Applications*, by M. J. K. Craig and K. A. Digre, OTC 7485, May, 1994 [7].

## 17.7 STRUCTURAL ANALYSIS FOR ASSESSMENT

**17.7.1 General.** Structural analysis for assessment shall be performed in accordance with Sections 3, 4, 5, 6 and 7 with exceptions, modifications and/or additions noted herein. Additional information and references can be found in *Structural Assessment of Existing Platforms*, by J. Kallaby, et al., OTC 7483, May, 1994, [3].

A structure should be evaluated based on its current condition, accounting for any damage, repair, scour or other factors affecting its performance or

integrity. Guidance on assessment information is provided in Section 17.4. The global structural model should be three-dimensional. Special attention should be given to defensible representation of the actual stiffness of damaged or corroded members and joints.

For platforms in areas subjected to ice loading, special attention should be given to exposed critical connections where steel that was not specifically specified for low temperature service was used.

### 17.7.2  Design Level Analysis Procedures.

**17.7.2a  General.**  Platforms of all exposure categories which do not pass the screening requirements may be evaluated using the design level procedures outlined below. These procedures may be bypassed by using the ultimate strength analysis procedures described in Section 17.7.3.

**17.7.2b  Structural Steel Design.**  The assessment of structural members shall be in accordance with the requirements of Section 3, except as noted otherwise in this section. Effective length factors (K-factors) other than those noted in Section 3.3.1d may be used when justified. Damaged or repaired members, may be evaluated using a rational, defensible engineering approach, including historical exposure or specialized software developed for that purpose.

**17.7.2c  Connections.**  The evaluation of structural connections shall be in accordance with Section 4, except as noted otherwise in this section. Section 4.1 which requires that joints be able to carry at least 50% of the buckling load for compression members, and at least 50% of the yield stress for members loaded primarily in tension, need not be met. Tubular joints should be evaluated for the actual loads derived from the global analysis. The strength of grouted and ungrouted joints may use the results of ongoing experimental and analytical studies, if it can be demonstrated that these results are applicable, valid and defensible. For assessment purposes, the metallurgical properties of API 2H material need not be met.

**17.7.2d  Fatigue.**  As part of the assessment process for future service life, consideration should be given to accumulated fatigue degradation effects. Where Levels III and/or IV surveys are made (see Section 14.3) and any known damage is assessed and/or repaired, no additional analytical demonstration of future fatigue life is required. Alternatively, adequate fatigue life can be demonstrated by means of an analytical procedure compatible with Section 5.

### 17.7.3  Ultimate Strength Analysis Procedures.  Platforms of all exposure categories either bypassing or not passing the requirements for screening and/or Design Level Analysis, must demonstrate adequate strength and stability to survive the ultimate strength loading criteria set forth in Sections 17.5 and 17.6, to insure adequacy for the current or extended use of the platform. Special attention should be given to modeling of the deck, should wave inundation be expected as noted in Section 17.6. The provisions of Section 17.7.2d (Fatigue) apply even if the Design Level Analysis is bypassed.

The following guidelines may be used for the ultimate strength analysis:

a. The ultimate strength of undamaged members, joints and piles may be established using the formulas of Sections 3, 4, 6 and 7 with all safety factors removed (i.e., a safety factor of 1.0). Nonlinear interactions (e.g. arc-sine) may also be utilized where justified. The ultimate strength of joints may also be determined using a mean "formula or equation" versus the lower bound formulas for joints in Section 4.

b. The ultimate strength of damaged or repaired elements of the structure may be evaluated using a rational, defensible engineering approach, including software developed for that purpose.

c. Actual (coupon test) or expected mean yield stresses, may be used instead of nominal yield stresses. Increased strength due to strain hardening may also be acknowledged, if the section is sufficiently compact, but not rate effects beyond the normal (fast) mill tension tests.

d. Studies and tests have indicated that effective length (K) factors are substantially lower for elements of a frame subjected to over-load than those specified in Section 3.3.1d. Lower values may be used, if it can be demonstrated that they are both applicable and substantiated.

The ultimate strength may be determined using elastic methods, Section 17.7.3a and 17.7.3b, or inelastic methods, Section 17.7.3c, as desired or required.

**17.7.3a Linear Global Analysis.** A linear analysis may be performed to determine if overstress is local or global. The intent is to determine which members or joints have exceeded their buckling or yield strengths. The structure passes assessment if no elements have exceeded their ultimate strength. When few overloaded members and/or joints are encountered, local overload considerations can be used as outlined in Section 17.7.3b. Otherwise, a detailed global inelastic analysis is required.

**17.7.3b Local Overload Considerations.** Engineering judgment suggests that overload in locally isolated areas may be acceptable with members and/or joints having stress ratios greater than 1.0 if it can be demonstrated that such overload can be relieved through a redistribution of load to alternative paths, or that a more accurate and detailed calculation would indicate that the member or joint is not, in fact, overloaded. Such a demonstration should be based on defensible assumptions with consideration being given to the importance of the joint or member to the overall structural integrity and performance of the platform. In the absence of such a demonstration, it is necessary to perform an incremental linear analysis (in which failed elements are replaced by their residual capacities), or perform a detailed global inelastic analysis and/or apply mitigation measures.

**17.7.3c Global Inelastic Analysis.**

**1. General.** Global inelastic analysis is intended to demonstrate that a platform has adequate strength and stability to withstand the loading criteria

specified in Sections 17.5 and 17.6 with local overstress and damage allowed, but without collapse.

At this level of analysis, stresses have exceeded elastic levels and modeling of overstressed members, joints and foundation must recognize ultimate capacity, as well as post buckling behavior, rather than the elastic load limit.

**2. Methods of Analysis.** The specific method of analysis depends on the type of extreme environmental loading applied to the platform and the intended purpose of the analysis. Push-over and time-domain analysis methods are acceptable as described in Section C17.7.3c.2.

**3. Modeling - Element Types.** For purposes of modeling, elements may be grouped as follows:

    a.    **Elastic Members:** These are members that are expected to perform elastically, throughout the ultimate strength analysis.

    b.    **Axially Loaded Members:** These are members that are expected to undergo axial yielding or buckling during ultimate strength analysis. They are best modeled by strut-type elements that account for reductions in strength and stiffness after buckling.

    c.    **Moment Resisting Members:** These members are expected to yield during the ultimate strength analysis, primarily due to high bending stresses. They should be modeled with beam-column type elements that account for bending and axial interaction, as well as the formation and degradation of plastic hinges.

    d.    **Joints:** The assessment loads applied to the joint should be the actual loads, rather than those based on the strength of the braces connecting to the joint.

    e.    **Damaged/Corroded Elements:** Damaged/corroded members or joints shall be modeled accurately to represent their ultimate and post ultimate strength and deformation characteristics. Finite element and/or fracture mechanics analysis may be justified in some instances.

    f.    **Repaired and Strengthened Elements:** Members or joints that have been or must be strengthened or repaired should be modeled to represent the actual repaired or strengthened properties.

    g.    **Foundations:** In carrying out a nonlinear pushover or dynamic time history analysis of an offshore platform, pile foundations should be modeled in sufficient detail to adequately simulate their response. It may be possible to simplify the foundation model to assess the structural response of the platform. However, such a model should realistically reflect the shear and moment (MT/PT) coupling at the pile head. Further, it should allow for the nonlinear behavior of both the soil and pile. Lastly, a simplified model should accommodate the development of a collapse mechanism within the foundation for cases where this is the weak link of the platform system. Further foundation modeling guidance can be found in

Section C17.7.3c.3g.

For ultimate strength analysis it is usually appropriate to use best estimate soil properties as opposed to conservative interpretations. This is particularly true for dynamic analyses where it is not always clear what constitutes a conservative interpretation.

## 17.8 MITIGATION ALTERNATIVES

Structures that do not meet the assessment requirements through screening, design level analysis, or ultimate strength analysis (reference Figure 17.5.2) will need mitigation actions. Mitigation actions are defined as modifications or operational procedures that reduce loads, increase capacities, or reduce consequences. *A Review of Operations and Mitigation Methods for Offshore Platforms*, by J. W. Turner, et al., OTC 7486, May, 1994 [8] contains a general discussion of mitigation actions and a comprehensive reference list of prior studies and case histories.

## REFERENCES

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7.  Craig, M. J. K. and Digre, K. A., "Assessments of High Consequence Platforms - Issues and Applications," OTC 7485, Offshore Technology Conference Proceedings, May 1994.

8.  Turner, J. W., Wisch, D. and Guynes, S. "A Review of Operations and Mitigation Methods for Offshore Platforms," OTC 7486, Offshore Technology Conference Proceedings, May, 1994.

Commentary on Draft Section 17.0 Assessment of Existing Platforms

C17.1  GENERAL.   In engineering practice, it is widely recognized that if an existing structure does not meet present day design standards, that does not mean that the structure is inadequate or not serviceable.  Examples of this not only include fixed offshore platforms, but also buildings, bridges, dams and onshore processing plants.  The application of reduced criteria for assessing existing facilities is also recognized in risk management literature, justified on both cost-benefit and societal grounds.

The References noted did not follow the review and balloting procedures necessary to be labeled API documents and in some cases reflect the opinions of only the authors.

C17.2 Platform Assessment Initiators

C17.2.4  Inadequate Deck Height.   Inadequate deck height is considered an initiator because most historical platform failures in the Gulf of Mexico have been attributed to waves impacting the platform deck resulting in a large step-wise increase in loading.   In a number of these cases this conclusion is based on hurricane wave and storm surge hindcast results which indicate conditions at the platform location that include estimated wave crest elevations higher than the bottom elevation of the platform's cellar deck main beams.

Inadequate deck height may result from one or more of the following events:

- Platform deck elevation set by equipment limitations;
- Platform deck elevation set to only clear a lower design wave height;
- Field installed cellar deck;
- Platform installed in deeper water than design for;
- Subsidence due to reservoir compaction.

C17.3 EXPOSURE CATEGORIES

C17.3.1a Manned, Non-Evacuated.   The Manned, Non-Evacuated condition is not normally applicable to the Gulf of Mexico.   Current industry practice is to evacuate platforms for hurricanes.

C17.3.1b  Manned, Evacuated.   In determining the length of time required for evacuation, consideration should be given to the distances involved; the number of personnel to be evacuated; the capacity and operating limitations of the evacuating equipment; the type and size of docking/landings, refueling, egress facilities on the platform; and the environmental conditions anticipated to occur throughout the evacuation effort.

C17.3.1c  Unmanned.  An occasionally manned platform, e.g., manned for only short durations such as maintenance, construction, workover operations, drilling and decommissioning, may be classified as Unmanned.

C17.3.2  Environmental Impact.  This section addresses those concerns associated with the potential release of liquid hydrocarbon or sour gas as a result of an assumed structural collapse of a platform.  Such release could emanate from any,

or all, of three possible sources: (1) the topsides inventory, (2) the wells, and (3) the pipelines. Determining the detailed impact of potential release on the environment is a very complex and somewhat subjective evaluation. It is important to note that the potential amount of liquid hydrocarbon or sour gas release from any of the three sources mentioned above is considerably less than the available inventory of each source. The factors affecting the release of hydrocarbon from each source are discussed below.

As a first step in the overall assessment process, an environmental impact review should be performed by the operator to estimate the consequence of any assumed platform collapse on the environment. This review should consider the amount of anticipated release, proximity of the platform to the shoreline, current direction and/or particularly environmentally sensitive areas such as coral reefs, wildlife refuges, and estuaries. The outcome of this review is the categorization of the platform as having either *Significant Environmental Impact* or *Insignificant Environmental Impact*.

Topsides Inventory:

At the time of a platform collapse, liquid hydrocarbon in the vessels and piping is not likely to be suddenly released. In fact, due to the continuing integrity of most of the vessels, piping and valves, it is most likely that very little of the inventory will be released. Thus, it is judged that significant liquid hydrocarbon release is a concern only in those cases where the topsides inventory includes unusually large capacity containment vessels.

Wells:

The liquid hydrocarbon or sour gas release from wells depends on several variables. The primary variable is the reliability of the sub-surface safety valves, SSSV, which are fail-safe closed or otherwise activated when an abnormal flow situation is sensed. As current MMS regulations require the use and maintenance of SSSV, it is judged that uncontrolled flow from wells may not be a concern for the platform assessment. Where SSSV are not used and the wells can freely flow, e.g., are not pumped, the flow from wells is a significant concern.

Even with the best operation of the SSSV, the liquid hydrocarbon or sour gas above the valve could be lost over time in a manner similar to a ruptured pipeline; however, the quantity is small and may not impact the assessment requirements.

Pipelines:

The potential for liquid hydrocarbon or sour gas release from pipelines is a major concern because of the many possible causes of rupture, (e.g., platform collapse, soil bottom movement, intolerable unsupported span lengths, and anchor snag). Only the first cause (platform collapse) is addressed in this document. Platform collapse is likely to rupture the pipelines or risers near or within the structure. For the hurricane case where the lines are not flowing, the maximum liquid hydrocarbon or sour gas release will likely be substantially less than the inventory of the line. The amount of product released will also depend on several variables such as the line size, the residual pressure in the line, the

contraction of the steel pipe during release of the residual pressure, the gas content of the liquid hydrocarbon, the undulations of the pipeline along its route, and other secondary parameters.

The major concern is the major oil transport lines which are large in diameter, longer in length and have a large inventory. In-field lines, which are much smaller and have much less inventory, may not be a concern and therefore may not need to be included in the platform assessment.

## C17.4 PLATFORM ASSESSMENT INFORMATION - SURVEYS

C17.4.1  GENERAL. The adequacy of structural assessments is measured by the quality of data available. The following is a summary of data that may be required:

1.  General Information
    a.  Original and Current Owner
    b.  Original and Current Platform Use and Function
    c.  Location, Water Depth and Orientation
    d.  Platform Type-Caisson, Tripod, 4-6-8 Leg, etc.
    e.  Number of Wells, Risers and Production Rate
    f.  Other Site Specific Information, Manning Level, etc.
    g.  Performance During Past Environmental Events

2.  Original Design
    a.  Design Contractor and Date of Design
    b.  Design Drawings and Material Specifications
    c.  Design Code (e.g., Edition of RP-2A)
    d.  Environmental Criteria-Wind, Wave, Current, Seismic, Ice, etc.
    e.  Deck Clearance Elevation (Bottom of Cellar Deck Steel)
    f.  Operational Criteria-Deck Loading and Equipment Arrangement
    g.  Soil Data
    h.  Number, Size and Design Penetration of Piles and Conductors
    i.  Appurtenances-List and Location as Designed

3.  Construction
    a.  Fabrication and Installation Contractors and Date of Installation
    b.  "As-Built" Drawings
    c.  Fabrication, Welding and Construction Specifications
    d.  Material Traceability Records
    e.  Pile and Conductor Driving Records
    f.  Pile Grouting Records, if Applicable

4.  Platform History
    a.  Environmental Loading History- Hurricanes, Earthquakes, etc.
    b.  Operational Loading History-Collisions and Accidental Loads
    c.  Survey and Maintenance Records
    d.  Repairs-Descriptions, Analyses, Drawings and Dates
    e.  Modifications-Descriptions, Analyses, Drawings and Dates

5.  Present Condition
    a.  All Decks-Actual Size, Location and Elevation

b. All Decks-Existing Loading and Equipment Arrangement
c. Field Measured Deck Clearance Elevation (Bottom of Steel)
d. Production and Storage Inventory
e. Appurtenances-Current List, Sizes and Locations
f. Wells-Number, Size and Location of Existing Conductors
g. Recent Above Water Survey (Level I)
h. Recent Underwater Platform Survey (Level II minimum)

If original design data, or as-built drawings are not available, assessment data may be obtained by field measurements of dimensions and sizes of important structural members and appurtenances. The thickness of tubular members may be determined by ultrasonic procedures, both above and below water, for all members except the piles. When the wall thickness and penetration of the piles cannot be determined, and the foundation is a critical element in the structural adequacy, it may not be possible to perform an assessment. In this case it may be necessary to downgrade the use of the platform to a lower assessment category by reducing the risk, or to demonstrate adequacy by prior exposure.

**C17.4.3 Soil Data.** Many sampling techniques and laboratory testing procedures have been used over the years to developed soil strength parameters. With good engineering judgment, parameters developed with earlier techniques may be upgraded based on published correlations. For example, design undrained shear strength profiles developed for many platforms installed prior to the 1970's were based on unconfined compression tests on 2.25-inch diameter driven wireline samples. Generally speaking, unconfined compression (UC) test give lower strength values and greater scatter than unconsolidated undrained compression (UU) tests, which are now considered the standard (Section 6). Studies have also shown that a 2.25-in. sampler produces greater disturbance than the 3.0-inch diameter thin walled push samplers now typically used offshore. Therefore, depending on the type of sampling and testing associated with the available data, it may be appropriate to adjust the undrained shear strength values accordingly.

Pile driving data may be used to provide additional insight on the soil profiles at each pile location, and infer the elevations of pile end bearing strata.

## C17.5 ASSESSMENT PROCESS

### C17.5.1 General - Acceptable Alternative Assessment Procedures:

1. Assessment of similar platform by comparison: Design level or ultimate strength performance characteristics from an assessment of one platform may be used to infer the fitness for purpose of other similar platforms, provided the platforms' framing, foundation support, service history, structural condition and payload levels are not significantly different. In cases where one platform's detailed performance characteristics are used to infer those of another similar platform, documentation should be developed to substantiate the use of such generic data.

2. Assessment with explicit probabilities of failure: As an alternative to meeting the requirements herein, the computation of explicit probabilities of platform failure may be performed at the discretion of the owner, provided the failure probabilities are properly derived, and the acceptance criteria used can

be satisfactorily substantiated.

3. **Assessment based on prior exposure:** Another alternative to meeting the requirements herein, for metocean loading assessment, is to use prior storm exposure, provided the platform has survived with no significant damage. The procedure would be to determine, from either measurements or calibrated hind-casts, the expected maximum base shear that the platform has been exposed to and then check to see if it exceeds, by an appropriate margin, the base shear implied in the Ultimate Strength Analysis Check. The margin will depend on the uncertainty of the exposure wave forces, the uncertainty in platform ultimate strength, and the degree to which the platform's weakest direction was tested by the exposure forces. All sources of uncertainty, i.e., both natural variability and modeling uncertainty, should be taken into account. The margin has to be substantiated by appropriate calculations to show that it meets the acceptance requirements herein. Analogous procedures may be used to assess existing platforms based on prior exposure to seismic or iceloading.

**C17.5.2 Assessment For Metocean Loading** - The *Manned, Non-Evacuated* criteria are not applicable to the Gulf of Mexico. Current industry practice is to evacuate platforms for hurricanes whenever possible. Should this practice not be possible for a Gulf of Mexico platform under assessment, alternative criteria would need to be developed to provide adequate life safety. The *Manned, Evacuated* criteria provide safety of personnel for hurricanes which originate inside the Gulf of Mexico where evacuation may not be assured, e.g., hurricane Juan (1985). The *Manned, Evacuated* criteria also encompass winter storms.

In the Gulf of Mexico, many early platforms were designed to 25-year return period conditions, resulting in low deck heights. By explicitly specifying wave height, deck inundation forces can be estimated directly for ultimate strength analysis (Ref. Section 17.6).

**C17.5.3 Assessment For Seismic Assessment.** An alternative basis for seismic assessment is outlined in the API sponsored Report titled: *Seismic Safety Requalification of Offshore Platforms*, by Iwan, W. D., et al, prepared for the API, May 1992. This Report was prepared by an independent panel whose members were selected based on their preeminence in the field of earthquake engineering and their experience in establishing practical guidelines for bridges, buildings and other on-land industrial structures. The basis for separating economic, life safety and environmental safety issues is addressed in this report.

## C17.6. METOCEAN, SEISMIC AND ICE CRITERIA/LOADS

**C17.6.2 Wave/Current Deck Force Calculation Procedure.** The procedure described herein is a simple method for predicting the global wave/current forces on platform decks. The deck force procedure is calibrated to deck forces measured in wave tank tests in which hurricane and winter storm waves were modeled.

The result of applying this procedure is the magnitude and point-of-application of the horizontal deck force for a given wave direction. The variability of the deck force for a given wave height is rather large. The coefficient of variation (standard deviation divided by the mean) is about 0.35. The deck force should be added to the associated wave force.

Other wave/current deck force calculation procedures for static and/or dynamic analyses may be used provided they are validated with reliable and appropriate measurements of global wave/current forces on decks either in the laboratory or in the field.

The deck force procedure relies on a calculated crest height. The crest height should be calculated using the wave theory as recommended in Section 2.3.1b.2., and the ultimate strength analysis wave height, associated wave period and storm tide.

The steps for computing the deck force and its point-of-application are as follows:

Step 1. Given the crest height, compute the wetted "silhouette" deck area, A, projected in the wave direction, $\theta_w$.

The full silhouette area for a deck is defined as the shaded area in Figure C17.6.2-1a, i.e., the area between the bottom of the scaffold deck and the top of the "solid" equipment on the main deck. The silhouette area for deck force calculations is a subset of the full area, extending up to an elevation above mllw that is equal to the sum of the storm tide and crest height required for ultimate strength analysis.

For lightly framed sub-cellar deck sections with no equipment, such as a scaffold deck comprised of angle iron, use one-half of the silhouette area for that portion of the full area. The areas of the deck legs and bracing above the cellar deck are part of the silhouette area. Deck legs and bracing members below the bottom of the cellar deck should be modeled along with jacket members in the jacket force calculation procedure. Lattice structures extending above the "solid equipment on the main deck can be ignored in the silhouette.

The area, A, is computed as:

$$A = A_x \cos \theta_w + A_y \sin \theta_w,$$
where $\theta_w$, $A_x$ and $A_y$ are as defined in Figure C.17.6.2-1b

Step 2. Use the wave theory recommended in Section 2.3.1b.2 and calculate the maximum wave-induced horizontal fluid velocity, V, at the crest elevation or the top of the main deck silhouette, whichever is lower.

Step 3. The wave/current force on the deck, $F_{dk}$, is computed by:

$$F_{dk} = 1/2 \; \rho \; C_d \; (\alpha_{wkf} {*} V + \alpha_{cbf} {*} U)^2 A,$$

where U is the current speed in-line with the wave, $\alpha_{wkf}$ is the wave kinematics factor (0.88 for hurricanes and 1.0 for winter storms), $\alpha_{cbf}$ is the current blockage factor for the jacket and $\rho$ is the density of sea water.

The drag coefficient, $C_d$, is given in Table C17.6.2.-1.

Step 4. The force $F_{dk}$ should be applied at an elevation $Z_{dk}$ above the bottom of the cellar deck. $Z_{dk}$ is defined as 50% of the distance between the lowest point of the silhouette area and the lower of the wave crest or top of the main deck.

Main Deck

Cellar Deck

Scaffold Deck

Wave

Elevation View of Platform Deck

Main Deck

Cellar Deck

Scaffold Deck

deck legs and braces
are part of 'deck' area

deck legs and bracing
are part of jacket

**Figure C.17.6.2-1a  Silhouette Area Definition**

Y

Plan view of deck

X

Ax

Wave
Heading

Ay

θw

**Figure C.17.6.2-1b  Wave Heading and Direction Convention**

## TABLE C.17.6.2-1

### Drag Coefficient, Cd, for Wave/Current Platform Deck Forces

| Deck Type | Cd<br>End-on and Broadside | Cd<br>Diagonal (45°) |
|---|---|---|
| Heavily Equipped (solid) | 2.5 | 1.9 |
| Moderately Equipped | 2.0 | 1.5 |
| Bare (no equipment) | 1.6 | 1.2 |

## C17.6.2a.1 Gulf of Mexico Criteria

The *Significant Environmental Impact* criteria are based on the "full population" hurricanes (all hurricanes affecting the Gulf). The *Manned, Evacuated/ Insignificant Environmental Impact* criteria are based on a combined population consisting of "sudden" hurricanes (subset of full population hurricanes) and winter storms. The *Unmanned/Insignificant Environmental Impact* criteria are based on winter storms.

The sudden hurricane criteria are based on hurricanes that spawn in the Gulf of Mexico. These criteria apply to *Manned* platforms in which there may not be enough warning to evacuate. Hurricanes that spawn outside the Gulf were not included because sufficient warning to evacuate all platforms is available provided that conventional Gulf of Mexico excavation procedures are maintained. An example of a sudden hurricane is Juan (1985). The sudden hurricane population used here provides for conservative criteria because, among the 27 hurricanes that spawned in the Gulf during 1900-1989, platforms would have been evacuated in almost all cases.

## C17.7 STRUCTURAL ANALYSIS FOR ASSESSMENT

**C17.7.1 General.** Structural evaluation is intended to be performed in three consecutive levels of increasing complexity. Should a structure fail the screening or first level, it should be analyzed using the second level, and similarly for the third level. Conversely, should a structure pass screening, no further analysis is required, and similarly for the second level. The first level (screening) is comprised of the first four components of the assessment process, selection, categorization, condition assessment and Design Basis Checks. The second level (design level analysis) allows recognition of the working strength of a member or joint within the elastic range using current technology. The third level (ultimate strength analysis) recognizes the full strength of the platform structure to demonstrate adequacy and stability.

**C17.7.2 Design Level Analysis Procedures.**

**C17.7.2a. GENERAL:** It should be noted that the Design Level Analysis criteria provided in Section 17.6 were calibrated for structures that did not have wave loading on their decks. It is therefore unconservative to consider wave loading on decks for assessments using Design Level Analysis. Ultimate Strength Analysis is required, using the higher environmental criteria contained in Section 17.6. Note, for some wave-in-deck loading only a linear global analysis will be necessary (Ref. Section 17.7.3a).

**C17.7.2b Structural Steel Design.** Should ongoing research be used to determine the strength of members, it must be carefully evaluated to assure applicability to the type of member, its level of stress and the level of confidence in the conclusions of the research. For example, the use of smaller values for effective length (K) factors may be appropriate for members developing large end moments and high levels of stress, but may not be so for lower levels of stress. Because of availability and other nonstructural reasons, members may have steel with yield stress higher than the specified minimum yield stress. If no such

data exists, tests may be used to determine the actual yield stress. Joint industry studies have indicated that higher yield stresses may be justified statistically.

**C17.7.2c Connections.** Joints are usually assumed rigid in the global structural model. Significant redistribution of member forces may result if joint flexibility is accounted for, especially for short bracing with small length to depth ratios, and for large leg can diameters where skirt piles are used. Joint flexibility analysis may use finite element methods, as appropriate. Steel joints may have higher strength than currently accounted for. Similarily, the evaluation of strength for grouted joints, as well as the assessment of grout stiffness and strength can consider higher values than normally used for design.

**C17.7.2d Fatigue.** All offshore structures, regardless of location, are subject to fatigue degradation. In many areas, fatigue is a major design consideration due to relatively high ratios of operational seastates to maximum design environmental events. In the GOM, however, this ratio is low. Still, fatigue effects should be considered and engineering decisions should be consciously based on the results of any fatigue evaluations.

Selection of critical areas for any Level III and/or IV inspections should preferably be based on factors such as joint and member loads, stresses, stress concentration, structural redundancy and fatigue lives as determined by platform design.

In the GOM, Levels III and/or IV underwater surveys may be considered adequate if they indicate no fatigue cracks. Should cracks be indicated, no further analysis is required if these are repaired. The use of analytical procedures for the evaluation of fatigue may be adequate if only Level II survey is done.

**C17.7.3 Ultimate Strength Procedures.** It should be noted that limited structural damage is acceptable and that the more severe environmental loading as noted in Section 17.6 is required.

In ultimate strength analysis structural elements are allowed to carry loads up to their ultimate capacities, and can continue to carry load after reaching those capacities, depending on their ductility and post elastic behavior. Such elements may exhibit signs of damage, having crossed over buckling or inelastic yielding, and in this context, damage is acceptable as long as the integrity of the structure against collapse is not compromised.

Since structures do not usually develop overload stresses in most of their elements at one time, the need to perform complex ultimate strength analyses for the whole structure may not be justified for a few overloaded elements, thus the need to distinguish between local and global overloading.

An efficient approach to ultimate capacity assessment is to carry it out in a step wise procedure. First, perform a linear global analysis to determine whether nonlinearity is a local or a global problem, and then perform local or global ultimate strength analysis as required.

C17.7.3a  Linear Global Analysis. This analysis is performed to indicate whether the structure has only a few or a large number of overloaded elements subject to loading past the elastic range.

C17.7.3b  Local Overload Considerations.  Minimal elastic overstress with adequate, clearly definable alternative load paths to relieve the portion of loading causing the overstress, can be analyzed as a local overload, without the need for full global inelastic analysis, or the use of major mitigation measures. The intent here is not to dismiss such overstress, but to demonstrate that it would be relieved because of alternative load paths, or because of more accurate and detailed calculations based on sound assumptions.  These assumptions must consider the level of overstress as well as the importance of the member or joint to the structural stability and performance of the platform.

Should demonstration of relief for such overstress be inconclusive or inadequate, a full and detailed global inelastic analysis would be required and/or mitigation measures taken as needed.

C17.7.3c  Global Inelastic Analysis.

1.  General.  It should be recognized that calculation of the ultimate strength of structural elements is a complex task and the subject of ongoing research that has neither been finalized nor fully utilized by the practicing engineering community. The effects of strength degradation due to cyclic loading and the effects of damping in both the structural elements and the supporting foundation soils should be considered. Strength increases due to soil consolidation may be used if justified.

2.  Methods of analysis.  Several methods have been proposed for ultimate strength evaluation of structural systems.  Two methods that have been widely used for offshore platform analysis are the Push-Over and the Time Domain methods.  It is important to note that, regardless of the method used, no further analysis is required once a structure reaches the specified extreme environmental loading, i.e., analysis up to collapse is not required.

    a.  Push-Over Method. This method is well suited for static loading, ductility analysis, or dynamic loading which can be reasonably represented by equivalent static loading.  Examples of such loading would be waves acting on stiff structures with natural periods under three seconds, having negligible dynamic effects, or ice loading which is not amplified by exciting the resonance of the structure.  The structural model must recognize loss of strength and stiffness past ultimate.  The analysis tracks the performance of the structure as the level of force is increased until it reaches the extreme load specified. As the load is incrementally increased, structural elements such as members, joints or piles, are checked for inelastic behavior in order to ensure proper modeling.  This method has also been widely used for ductility level earthquake analysis by evaluating the reserve ductility of a platform, or by demonstrating that a platform's strength exceeds the maximum loading for the extreme earthquake events.  Although cyclic and hysteritic effects cannot be explicitly modeled using this method, their effects may be recognized in

the model, in much the same way that these effects are evaluated for pile head response to inelastic soil resistance.

b. **Time Domain Method.** This method is well suited for detailed dynamic analysis in which the cyclic loading function can be matched with the cyclic resistance-deformation behavior of the elements step by step. This method allows for explicit incorporation of nonlinear parameters such as drag and damping into the analysis model. Examples of dynamic loading would be earthquakes and waves acting on flexible structures whose fundamental period is three seconds or greater. The identification of a collapse mechanism, or the confirmation that one does exist, may require significant judgment using this method. Further guidance to nonlinear analysis can be found in Sections 2.3.6 and C2.3.6.

3. **Modeling.** Regardless of the method of analysis used, it is necessary to accurately model all structural elements. Before selection of elememt types, detailed review of the working strength analysis results is recommended to screen those elements with very high stress ratios, that are expected to be overloaded. Since elements usually carry axial forces and biaxial bending moments, the element type should be selected based on the dominant stresses. Beam and column elements are commonly used, although plate elements may be appropriate in some instances. Elements may be grouped as:

   a. **Elastic Members:** The majority of members are expected to have stresses well within yield, and would not be expected to reach their capacity during ultimate strength analysis. These elements should be modeled the same as in the working strength method, and tracked to ensure their stresses remain in the elastic range. Examples of such members are deck beams and girders which are controlled by gravity loading, and with low stress for environmental loading, allowing for significant increase in the latter before reaching capacity. Other examples may be jacket main framing, controlled by installation forces, and conductor guide framing, secondary bracing and appurtenances.

   b. **Axially Loaded Members:** These are undamaged members with high $Kl/r$ ratios and dominant high axial loads, that are expected to reach their capacity. Examples of such members are primary bracing in the horizontal levels and vertical faces of the jacket, and primary deck bracing. The strut element should recognize reductions in buckling and post buckling resistance due to applied inertia or hydrodynamic transverse loads. Effects of secondary (frame induced) moments may be ignored when this type of element is selected. Some jacket members, such as horizontals, may not carry high axial loads until after buckling or substantial loss of strength of the primary vertical frame bracing.

   c. **Moment Resisting Members:** These are undamaged members with low $Kl/r$ ratios and dominant high bending stresses, that are expected to form plastic hinges under extreme loading. Examples of such members may be unbraced sections of the deck and jacket legs, and piles.

d.  **Joints:** The joint model should recognize whether the joint can form a hinge or not, depending on its D/t ratio and geometry, and should define its load-defotmation characteristics after hinge formation. Other evaluations of joint strength may be acceptable if applicable and if substantiated with appropriate documentation.

e.  **Damaged Elements:** The type of damage encountered in platforms ranges from dents, bows, holes, tears and cracks to severely corroded or missing members or collapsed joints. Theoretical as well as experimental work has been ongoing to evaluate the effects of damage on structural strength and stiffness. Some of this work is currently proprietary and others are in the public domain. Modeling of such members should provide a conservative estimate of their strength up to and past capacity.

f.  **Repaired and Strengthened Elements:** The type of repairs usually used on platforms ranges from wet or hyperbaric welding, grouting, clamps, to grinding and relief of hydrostatic pressure. Grouting is used to stiffen members and joints, and to preclude local buckling due to dents and holes. Grinding is commonly used to improve fatigue life and to remove cracks. Several types of clamps have been successfully used, such as friction, grouted and long bolted clamps. Platform strengthening may be accomplished by adding lateral struts to improve the buckling capacity of primary members, and by adding insert or outrigger piles to improve foundation capacity. Modeling of repaired elements requires a keen sense of judgment tempered by conservatism, due to lack of experience in this area.

g.  **Foundations:** In a detailed pile-soil interaction analysis, the soil resistance is modeled as a set of compliant elements which resist the displacements of the pile. Such elements are normally idealized as distributed, uncoupled, non-linear springs. In dynamic analysis hysteretic behavior may also be significant. Recommendations for characterizing non-linear soil springs are provided below.

    - **Soil Strength and Stiffness Parameters.** A profile of relevant soil properties at a site is required to characterize the soil resistance for extreme event analysis. Soil strength data are particularly important in characterizing soil resistance. In some cases, other model parameters (such as initial soil stiffness and damping) are correlated with strength values and thus can be estimated from the strength profile or other rules of thumb.

    - **Lateral Soil Resistance Modeling.** A method for constructing distributed, uncoupled, non-linear soil springs (p-y curves) is described in Section 6.7. These techniques may be useful for modeling the monotonic loading behavior of laterally deforming piles where other site specific data is not available. Due to their empirical nature the curves should be used with considerable caution however, particularly in situations where unloading and reloading behavior is important or where large displacement response such as

ultimate capacity (displacements generally > 10% of the pile diameter) is of interest.

- **Axial Soil Resistance Modeling.** A method for constructing distributed, uncoupled, non-linear soil springs (t-z and q-w curves) for axial resistance modeling is described in Section 6.7. These techniques may be useful for modeling the monotonic loading behavior of axially deforming piles where other site specific data is not available.

To construct a 'best estimate' axial soil resistance model, it may be appropriate to adjust the curves in Section 6.7 for loading rate and cyclic loading effects which are known to have a significant influence on behavior in some cases.

- **Torsional Soil Resistance Modeling.** Distributed, uncoupled, non-linear soil springs for torsional resistance modeling can be constructed in a manner similar to that for constructing t-z curves for axial resistance. Torsion is usually a minor effect and linear resistance models are adequate in most cases.

- **Mudmats and Mudline Horizontal Members.** In an ultimate strength analysis for a cohesive soil site, it may be appropriate to consider foundation bearing capacities provided by mudmats and mudline horizontal members, in addition to the foundation capacity due to pilings, provided that:

1. Inspection was conducted to confirm the integrity of the mudmats.

2. Inspection confirmed that the soil support underneath the mudmats and horizontals has not been undermined by scour. In contrast, for design purpose, the bearing capacity due to mudmats and mudline jacket members are typically neglected.

Mudmats and mudline horizontal members may be treated as shallow foundations. Methods described in Sections 6.12 to 6.16 and the commentary on shallow foundations may be used to estimate their ultimate capacity and stiffness. In addition, other methods may be used in cases in which the shear strength of the soil increases with depth.

Care must be taken in correctly modeling the interaction between the mudmats (and mudline members), and the pile foundation. Depending on soil conditions, the two components of foundation capacity may have very different stiffnesses.

- **Effect of Soil Aging.** For ultimate strength analysis, aging (the increase of soil shear strength with time) has been suggested as a source of additional foundation capacity that is not accounted for in the present design methodology. However, the state-of-the-art of this subject has not been sufficiently developed to justify routine application. Any attempt to upgrade foundation capacity based on

aging will have be justified on a case-by-case basis.

- **Estimate As-installed Pile Capacity.** Pile capacity should be estimated primarily based on the static design procedure described in Section 6.4. However, if pile driving records (blow counts and/or instrumented measurement) are available, one dimensional wave equation based methods may be used to estimate soil resistance to driving (SRD) and infer an additional estimate of as-installed pile capacity.

A conductor pull test offers an alternative means for estimating the as-installed capacity of a driven pile.

- **Conductors.** In an ultimate strength analysis, well conductors may contribute to the lateral resistance of a platform once the jacket deflects sufficiently to close the gap between the conductor guide frames and the conductors.

Below the mudline, conductors may be modeled using appropriate p-y and t-z soil springs in a manner similar to piles. Above the mudline, the jacket model must realistically account for any gaps between the jacket and the conductors.

C17.7.3 Ultimate Strength Procedures.

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