CUIMR-R-92-059 C2

Reprinted from CIVIL ENGINEERING IN THE OCEANS V
Proceedings International Conference
Committees: Ocean Engineering & Wave and Wave Forces/ASCE
College Station, Texas/November 2-5, 1992

RE-QUALIFICATION OF OFFSHORE PLATFORMS

By R. G. Bea[1] Member, ASCE

ABSTRACT

Older platforms that are being re-qualified for extended service involve many technical problems that are unlike those associated with design of new platforms. Procedures to help resolve some of these problems are presented. Case studies used to verify the procedures are summarized.

INTRODUCTION

As offshore platforms age:

- **Their strength and reliability decreases.** The rate at which strength and reliability decrease is a function of the initial design and how the structure was constructed and maintained.

- **They must not necessarily be discarded.** The challenge is to define Inspection, Maintenance, and Repair (IMR) programs to maintain strength and reliability and to choose operations that can be successfully completed.

- **They become more likely to reflect the effects of defects and damage.** A primary function of the IMR process is to give early warning of defects and damage, define good alternatives to help manage the effects of aging, provide a basis for decisions on the best alternatives, and provide a continuing framework for implementation, monitoring and recording the results.

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[1]Professor, Dept. of Civil Eng. and Dept. of Naval Architecture & Offshore Eng., Univ. of California, Berkeley, California, 94720.

- **There comes a time for them to retire.** One of the primary functions of an IMR process is to enable one to know when it is time to decommission the platform.

IMR PROGRAMS

Inspection, maintenance, and repair (IMR) programs (Fig. 1) are intended to preserve the strength and serviceability of the platform at an adequate level (Bea, 1992). The most critical part of an IMR program is knowledge. The IMR process can be no more effective or efficient than the knowledge, data, and experience that forms the basis for the process.

The IMR process should investigate a wide variety of alternatives to accomplish its fundamental objectives. Inspections can range from general to detailed, visual to acoustic, and periodic to continuous monitoring (Marine Technology Directorate Ltd. 1989). The first priority of inspections is on those parts of the structure that have combinations of high consequences of loss of strength and ductility and high likelihood's of damage or defects (Fig. 1).

Maintenance can be proactive, focused on prevention. Or it can be reactive, focused on correction. Repairs can range from patching to replacement, temporary to permanent, and from comprehensive to judicious neglect. Repairs need to be carefully engineered (Winkworth and Fisher 1992). More harm than good can be done by inappropriate repairs and careless repair operations.

The IMR process can be periodic (time based) or it can be condition oriented (occasion based). Combinations of proactive, reactive, periodic, and condition based approaches can be appropriate for different IMR programs (Marine Technology Directorate Ltd 1989). There are common elements in many IMR programs. But, each platform has its unique aspects and these unique aspects should be recognized when an IMR program is developed for a given platform.

A major challenge in development of IMR programs is to find a combination of IMR that best fits a particular platform or group of platforms, their operations, and the organizations that operate and regulate the structures. The IMR process should define the combinations of IMR that will produce the lowest total costs (initial and future) and optimize the use of resources without compromising minimum safety and reliability requirements.

The IMR process must be diligent and disciplined and have integrity. There must be a focus on the quality of the performance of the process. The quality of the product (adequate strength and reliability) will be a natural by-product.

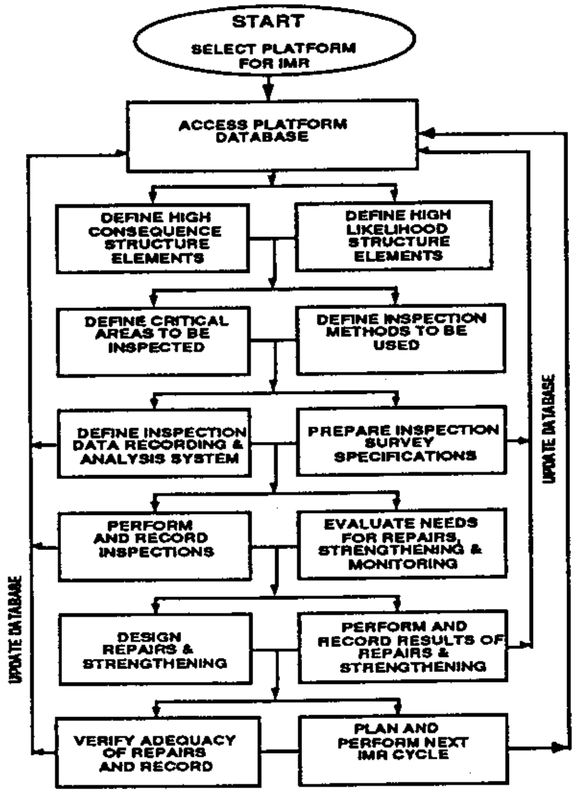

Fig. 1 - Development of IMR Programs

GENERAL APPROACH

A general approach to the re-qualification of offshore platforms has been developed (Aggarwal et al. 1990; Bea et al. 1991a). The approach is based on a progressive screening process involving four cycles of analyses of increasing detail and difficulty (Aggarwal et al. 1990):

- **Cycle-1 - qualitative "scoring" factors** are used to evaluate the platform capacity and loadings and the potential consequences associated with failure of the platform.

- **Cycle-2 - coarse quantitative analyses** are used to define the capacity and loadings and the potential consequences of failure.

• **Cycle-3 - detailed quantitative "state-of-the-practice"analyses** of platform capacity, loadings and potential consequences are performed (Fu et al. 1992).

• **Cycle-4 - very detailed quantitative "state-of-the-art" analyses** are used to evaluate platform performance characteristics, the probabilistic aspects of the loadings and capacities, and the likelihoods and consequences associated with failure (Stewart et al 1988; Martindale et al. 1989; van de Graff and Tromans 1992; Bea et al. 1992e).

This approach attempts to qualify platforms at the earliest possible cycle, reserving marginal, moderate and high consequence platforms for the later cycles.

RSR Analyses

In the first three cycles, the platform capacity to withstand environmental loadings is expressed by a Reserve Strength Ratio, RSR. The RSR is the ratio of the ultimate lateral load capacity of the platform to a "reference" lateral loading. The reference lateral loading is taken to be the minimum lateral loading expressed or implied by current design codes or guidelines (e.g. API RP 2A) (API 1991; Fu et al. 1992).

Major difficulties are associated with development of estimates of the platform capacity. Static loading nonlinear capacity analyses are not easy, and dynamic loading nonlinear capacity analyses are even more difficult. Given a large population of platforms, it is prohibitive to perform nonlinear capacity analyses on all of the platforms. To help overcome this problem, a qualitative ranking method and coarse quantitative method have been developed to estimate the RSR for storm loadings for Cycle-1 and Cycle-2 screening, respectively (Bea et al. 1991, Aggarwal, 1991). The method employs simplified methods to estimate the total lateral loadings and platform capacities.

The Cycle-1 RSR is based on five factors that address the platform capacity and three factors that address the platform storm loadings (Aggarwal, 1991):

$$RSR = \frac{[R_1 R_2 R_3 R_4 R_5]}{[S_1 S_2 S_3]} \qquad (1)$$

where R_1 - Material factor, R_2 - Condition factor, R_3 - Modifications factor, R_4 - Configuration (framing, redundancy) factor, R_5 - Construction factor, S_1 - Design criteria factor, S_2 - Deck elevation factor, R_3 - Loadings modifications factor. Guidelines have been developed for evaluations of quantitative "scores" for each of these factors. Depending on the final score, the platforms are ranked in RSR categories that range from "very low" (≤ 0.8) to "very high" (≥ 2.0).

The Cycle-2 RSR is based on two techniques to estimate the platform lateral load capacity. The lower bound is based on the lateral loading that would cause the first primary lateral load resisting member at a given elevation to fail (undamaged or damaged, unrepaired or repaired, including joint capacities at the ends of this member). The upper bound estimate is based on the summed capacities of all of the primary lateral load resisting members at a given elevation that are required to form a mechanism.

Reference loadings are based on a wind and wave-current kinematics profile that is defined by specified wind, wave, and current characteristics. This profile can be modified to recognize directional spreading, shielding, current blocking, and near-surface effects (API 1992). The loading formulations are based on API RP 2A guidelines (API 1991) with special provisions for deck elements that may be inundated by the specified wave crests (McDonald et al. 1990). The kinematics profile and loadings formulations are translated to a profile of loadings based on the projected areas of the structure and deck elements.

One of the structures used to test the Cycle-1 and Cycle-2 RSR approaches was a conventional Gulf of Mexico 8-leg self-contained drilling and production platform (Fig. 2) designed for a water depth of 271 ft (83 m) (Aggarwal 1991). The lateral loading and capacity patterns for this platform in end-on and broadside directions (Fig. 3) allowed definition of a minimum local RSR (ratio of platform lateral capacity to storm shear at given elevation) to define the elevation at which failure would be initiated. The analyses indicated that the mean (of high and low bounds) RSR for both the end-on and broadside cases is RSR = 1.7 The Cycle-1 evaluation of this structure indicated an RSR that fell in the "moderate" category (RSR ≈ 1.0).

Detailed results on storm loadings and platform capacities are available for this platform (Aggarwal 1991). The detailed analyses indicated RSR = 1.6. Thus, there was a "bias" (nonlinear analysis / simplified method) in the Cycle-2 RSR analysis of $B_{RSR} \approx 0.9$. Comparisons of the results of the Cycle-2 RSR estimating process with results from the analyses of three "typical" Gulf of Mexico platforms (8-pile, 4-pile, and 5-pile with vertical and horzontal K-bracing) indicated an average RSR bias of $B_{RSR} = 1.0$ and a coefficient of variation $V_{BRSR} = 26$ % (Aggarwal, 1991). A similar approach has been used for estimating the RSR associated with platforms having damaged (dented, holes, corrosion) and repaired elements with similar success. Additional structures need to be studied to further evaluate the bias characteristics of the Cyle-1 and Cycle-2 evaluation techniques.

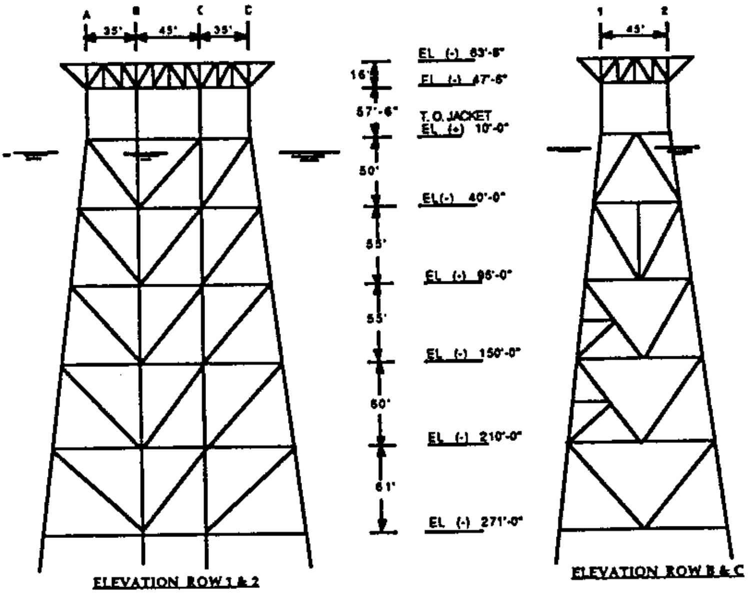

Fig. 2. Gulf of Mexico Test Platform

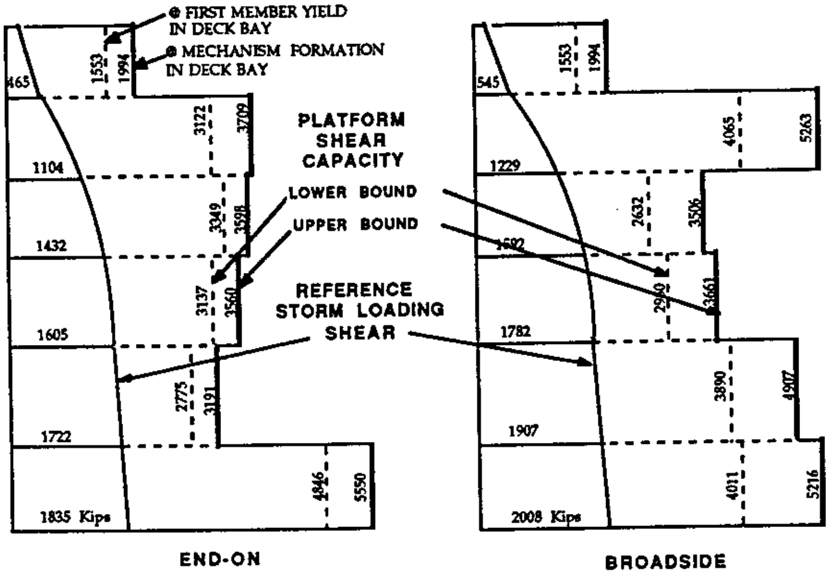

Fig. 3. Storm Loading Shears and Platform Shear Capacities

Determination of Fitness for Purpose

Fitness for Purpose (FFP) evauations are based on the comparisons of the RSR and a measure of consequences associated with the platform. In Cycle-1, (Fig. 4), platform potential consequences associated with "failure" (complete loss of serviceability) are classified into five general categories that range from "very high" to "very low." Guidelines are given for determination of each of the categories. For example, a Medium Consequence platform would be one which produces crude oil, has no other major operations (e.g. processing, storage, and compression), is manned, and which transfers crude to a central platform.

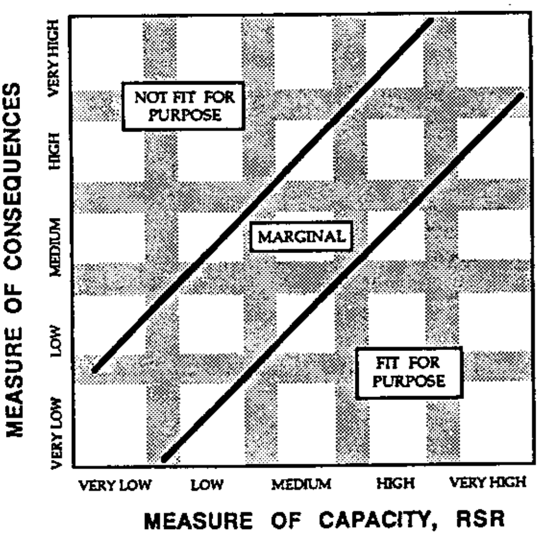

Fig. 4. Cycle-1 Fitness for Purpose Criteria

Cycle-2 FFP criteria are based on quantitative evaluations of a Consequence Measure, CM (Fig. 5) (Bea 1992b):

$$CM = \frac{CF}{\Delta Ci} L \qquad (2)$$

Potential costs of failure, CF, include loss of property, loss of reserves and productivity, salvage and clean-up, injuries, pollution, and if warranted, replacement of the platform and its facilities (Martindale et al. 1989, Bea 1991, Fu et al. 1992). ΔCi are the total costs required to decrease the total annual probability of failure of the platform by a factor of 10. L is the desired platform life in years. Formulation of the Cycle-2 FFP criteria is detailed in Appendix II.

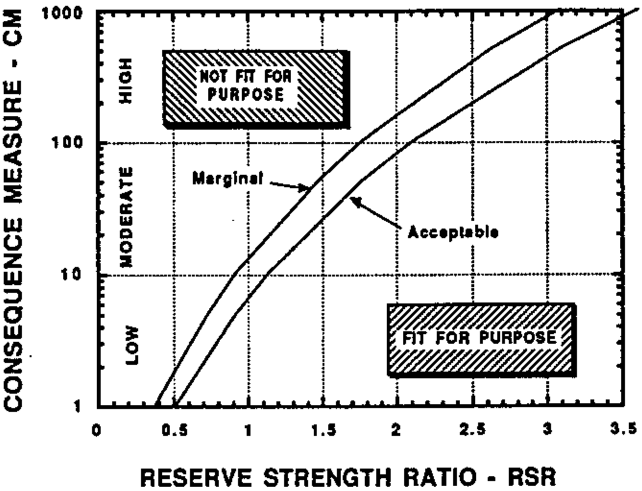

Fig. 5. Cycle-2 Fitness for Purpose Criteria

Older platforms should not be expected to have capacities or structure reliability that are equal to that of comparable new platforms. Aging will take its toll in the forms of corrosion, fatigue, damage, defects, and technical obsolescence (improvements in criteria, materials, and construction - maintenance practices). Given the limited resources that can be spent on platform re-qualification programs, frequently the best use of a major part of those resources is applied to reduce the likelihood of failure associated with operating hazards. Reductions in operating risks help keep the total risk associated with the platform within acceptable limits. This full-scope approach offers a variety of ways that a platform can be qualified as FFP including increasing the platform capacity, reducing its loadings (dead, operating, and environmental), reducing the potential consequences associated with its failure (pollution and injury prevention), and reducing the likelihoods of operations related failures.

Judgments concerning FFP in Cycles 3 and 4 are based on detailed evaluations of the projected performance characteristics of the platform. A "standard of practice" approach to the evaluation of FFP in Cycle-4 has evolved from recent experience with a variety of platform requalifications and development of design criteria for new platforms (Fig. 5) (Bea et al. 1992e). This approach avoids many of the difficulties associated with historic - actuarial data based approaches, calibration (to current design codes) approaches, and economics - utility based approaches.

The remainder of this paper will be devoted to a summary of verifications of the Cycle-2 approach for three Gulf of Mexico platforms and of the Cycle-4 approach for two West Coast platforms.

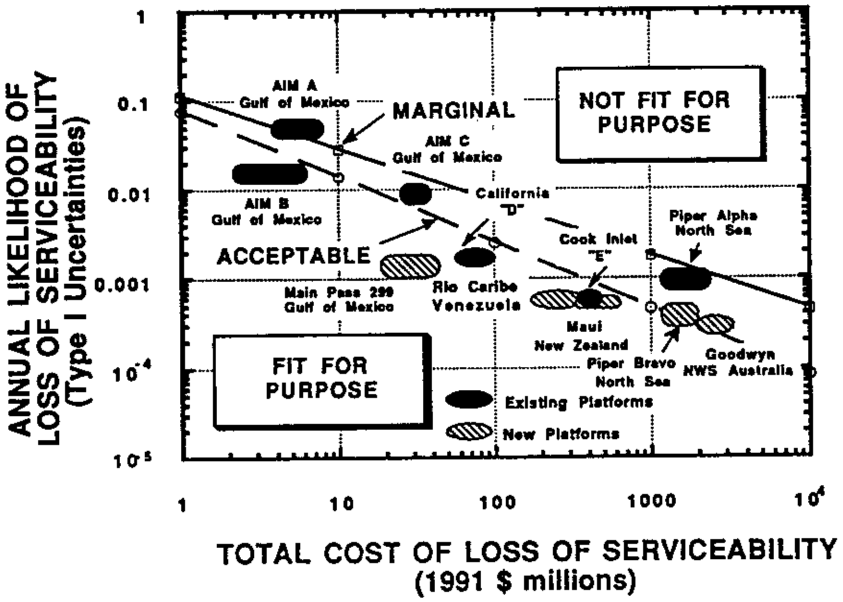

Fig. 5. Cycle-4 Standard of Practice FFP Criteria

CYCLE-2 VERIFICATIONS

Gulf of Mexico 8-Leg Platforms

Platform "A" is a self-contained drilling and production structure that was installed in 1964. The platform was designed for a 25-year return period storm with a wave height of 47 ft (14 m) without any air gap. The lateral design load was 1,000 kips (4,500 kN). The platform was built with A-36 steel and there were no heavy wall joint cans. Due to an error in determining the water depth, the platform was placed in a water depth 2 ft (0.6 m) greater than the structure had been designed for.

Application of Cycle 2 analyses indicated that the platform had an RSR = 0.75 to 0.8. The lower-bound platform capacity technique indicated a static capacity of 1,500 kips (6,700 kN) with failure initiating in a diagonal brace at the top of the jacket. The upper-bound technique indicated a capacity of 1,600 kips (7,100 kN). The API reference lateral load is 2,000 kips (9,400 kN). This reference load includes a wave crest loading on the lower cellar deck of the platform of 500 kips (2,200 kN) (McDonald, et al, 1991).

The costs associated with failure (without replacement) together with a projected life L = 10 years resulted in CM = 20 to 30. With an RSR = 0.75 to 0.80 and CM = 20 to 30, based on the FFP criteria in Fig. 4, this platform is not indicated to be FFP. The platform could be qualified for service through a series of upgrading measures including reducing loadings (removing equipment from the lower decks, unused boat landings, well conductors, and risers) and installing additional well shut-in equipment. In the upgraded condition, the platform would have an RSR ≈ 1.0 and CM = 10 to 15 placing it in the marginal range.

This platform failed one year after it was installed (during hurricane "Hilda"). Based on the hindcast hurricane conditions that were present at the platform (Meeting Transcript 1964), the maximum total lateral loadings were estimated to be 1,900 kips (8,500 kN) (maximum wave height = 59 ft = 18 m). The maximum wave resulted in significant inundation of the lower deck. This loading exceeded the platform capacity. It should have failed, it did, and in the way predicted by the Cycle 2 loading and capacity analyses. All of the platform operating personnel had been evacuated before the storm and all of the subsurface safety valves in the wells (three completed) operated properly. There was no environmental pollution and there were no personnel injuries.

Platform "B" is a nearby similar structure installed in a water depth of 217 ft (66 m) in 1963. The platform was designed for a 100-year return period storm with a maximum wave height of 55 ft (17 m)with an air gap of 5 ft (1.5 m). The design hydrodynamic forces were determined based on Stokes Fifth Order Theory kinematics, no current, and a drag coefficient Cd = 0.5. The total lateral design loading was 1,800 kips (8,000 kN) (Marshall and Bea 1976).

Application of the lower bound lateral capacity method indicated a total lateral capacity for this platform of 3,500 kips (16,000 kN) and an upper bound capacity of 4,900 kips (22,000 kN). These lateral capacity estimates were confirmed with results from sophisticated nonlinear analyses published by van de Graff and Tromans (1991). These lateral capacity estimates also involved recognition of foundation dynamic loading characteristics (Bea 1992b) and removal of biases from the foundation pile axial capacity evaluations (Bea 1983). The present day API reference lateral loading is 2,800 kips (12,000 kN). This indicates an RSR = 1.3 to 1.8. The CM for this structure is estimated to be in the range of 30 to 40 based on a 10-year projected life. Based on the FFP criteria summarized in Fig. 4, this platform is indicated to be FFP.

This platform survived the same storm that resulted in the failure of Platform "A." The storm wave crests did not reach the lower decks and the maximum lateral loadings were estimated to be 1,600 kips (7,100 kN) (Bea 1974, Marshall and Bea 1976). The platform should have survived and it did. The platform was damaged by the storm (Meeting Transcript 1964) and later repaired. This platform is in service today.

Platform "C" is a more recent 8-leg, 12-pile structure that was designed for a 100-year storm wave height of 58 ft (18 m) with an air gap of 5 ft (1.5 m), the storm associated currents (4 fps, 1.2 m/s, at surface), and Cd = 0.5. The total lateral design load was 3,300 kips (15,000 kN) (Marshall and Bea 1976). The platform was installed in 1968 and was designed according to the draft guidelines of the first API RP 2A.

Application of the lower bound lateral capacity method indicated a capacity of 6,600 kips (29,000 kN)and an upper bound capacity of 8,600 kips (38,000 kN). These lateral capacity estimates are larger than those originally estimated for this platform (Marshall and Bea 1976). In this earlier work, Bayesian updating methods were employed to bring the observed behavior (no failures) into conformance with the predicted behavior (high likelihood of failures). Recent analyses published by van de Graff and Tromans (1992) confirm these Cycle 2 loading and static capacity results. Recent work indicates that recognition of transient loading - nonlinear capacity performance effects could result in additional increases in the capacity of the order of 25 % (Bea 1992c; Stewart 1992).

The present day API reference lateral loading is 4,600 kips (21,000 kN). This indicates an RSR = 1.4 to 1.9 (static) and RSR = 1.8 to 2.4 (dynamic). The platform is indicated to have a CM in the range of 30 to 50 for a 10-year projected life. Based on the criteria summarized in Fig. 4, this platform is indicated to be FFP.

This platform, and seven other similar structures survived the intense portion of hurricane "Camille." One of the platforms recorded a 72-ft wave height and several of the platforms indicated substantial wave crest damage in the lower decks (Bea 1974). The total lateral loading estimated on this group of platforms ranged from 4,600 to 5,400 kips (20,000 to 24,000 kN) (Marshall and Bea 1976). All of the platforms survived without substantial damage. The analyses indicate that they should have.

CYCLE-4 VERIFICATIONS

Platform "D" is located two miles offshore California in a water depth of 60 ft (18 m). It is a 12- leg drilling and production platform. It was installed in 1964 and has been well maintained. Damage which has been found has been repaired using advanced methods (Miller et al. 1991). Strengthening of the structure and foundation was accomplished to further improve the performance of the platform. The operator wants the platform remain in service for another 20 years.

The operator developed advanced studies of the oceanographic and earthquake environments and the performance characteristics of the platform. Storm wave loading analyses examined the force implications of a subdeck that had been added to the platform and the benefits of marine growth inhibition with cladding; these measures did not result in significant reductions in loadings.

The earthquake loading characterization accounted for the dynamic loading and ductility performance characteristics of the platform (Bea, 1991d). There was a substantial difference between the forces indicated for an elastic system and those for a nonlinear system having load-deformation characteristics similar to those of the platform (Bea 1992c). Static push-over analyses indicated capacities that did not reflect the dynamic, transient loading effects of earthquakes and resulted in indications of capacity that were too conservative.

The annual probabilities of failure for the storm and earthquake loadings considering only inherent uncertainties (not including modeling uncertainties) were estimated to be $Pfs = 1 \times 10^{-3}$ and $Pfq = 2 \times 10^{-4}$ per year, and the operating probabilities of failure were estimated to be 6×10^{-5} per year (Miller et al. 1991). The total annual likelihood of loss of serviceability was estimated to be 1.3×10^{-3} per year. Based on estimated costs of failure in the range of $50 to $100 millions (non-replacement of the platform) and based on the criteria summarized in Fig. 5, the platform is indicated to be FFP. This platform is in service today.

Platform "E" is located in Cook Inlet, Alaska in a water depth of 125 ft (38 m).. This 4-leg, 48-pile platform was installed in 1967 and experienced a blowout in 1985 (Bea et al. 1992e). The blowout created a large crater 80 ft (24 m) deep around one of the legs. The crater was subsequently filled with 100,000 yd^3 (77,000 m^3) of gravel. Pile axial and lateral load tests were performed in gravel to confirm the characterizations used for the pile foundations. A critical part of this analysis was inclusion of the effects of the multiple well casings inside the piles.

Analyses of the platform response to intense earthquakes indicated that the primary failure mode of concern was the horizontal displacement induced in the foundation. Initial static push-over analyses indicated that the platform would fail in an earthquake having an average return period of about 750 years. Analyses of the foundation system indicated that the platform had a lateral displacement capacity in the range of 36 in. (91 cm) to 54 in. (137 cm) before the well tubulars inside the piles ruptured at the connectors. Dynamic, nonlinear time-history analyses of the horizontal displacements induced by a suite of recorded and synthetic earthquake records indicated that the platform would reach its limiting lateral displacements at earthquake intensities having average return periods in excess of 10,000 years.

The annual probabilities of failure for earthquakes and ice loadings based on inherent uncertainties were estimated to be $Pfq = 1 \times 10^{-4}$ and $Pfi = 5 \times 10^{-6}$ per year, respectively. Operating probabilities of failure were estimated to be $Pfo = 5 \times 10^{-4}$ per year. The total annual probability of failure was estimated to be $Pf = 6.5 \times 10^{-4}$ per year (Bea et al 1992e). The estimated costs of failure ranged between $350 to $500 millions. This included costs associated with potential injuries, salvage, and pollution, but not replacement of the platform. Based on the criteria summarized in Fig. 5, the platform was indicated to be FFP. This platform is in service today.

CONCLUSIONS

The experience upon which the developments described in this paper have been founded indicate that the offshore industry has the basic technology necessary to properly and responsibly re-qualify platforms for extended service. This technology needs to be organized and summarized in the form of definitive engineering guidelines and procedures for the re-qualification of offshore platforms. FFP criteria acceptable to industry operators and government regulators and based on use of the specified guidelines and procedures need to be developed to provide sensible goals for re-qualifications. Industry-wide computer based data banks need to be developed to archive important information on platform re-qualifications and to expedite industry and government re-qualification efforts and reporting.

ACKNOWLEDGMENTS

This paper is the result of a research project conducted at the University of California at Berkeley with funding provided by the National and California Sea Grant College Program, the Minerals Management Service, the California State Lands Commission, Phillips Petroleum Co., Chevron Corp., and Unocal Corp. Without this support, this work could not have been undertaken and completed. During this project, information and assistance were provided by Marathon Oil Co., Amoco Prod. Co., and Shell Oil Co. This information and assistance are gratefully acknowledged.

This paper is funded in part by a grant from the National Sea Grant College Program, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, under grant number NA89AA-D-SG138, project numbers R/OE-11 and R/OE-19 through the California Sea Grant College, and in part by the California State Resources Agency. The views expressed herein are those of the author and do not necessarily reflect the views of NOAA or any of its sub-agencies. The U. S. Government is authorized to reproduce and distribute for governmental purposes.

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Appendix II. CYCLE-2 FITNESS FOR PURPOSE CRITERIA

The Cycle- 2 FFP criteria (Fig. 5) are based on the following expressions (Bea 1991b):

$$RSR = \exp(\beta_S \sigma - 2.33 \sigma_S) \quad (1)$$
$$\sigma^2 = \sigma_R^2 + \sigma_S^2 \quad (2)$$
$$\beta = \ln(R_{50} / S_{50}) / \sigma \quad (3)$$
$$Pf = \Phi[-\beta] \quad (4)$$
$$Pf_T = Pf_O + Pf_S \quad (5)$$
$$Pf_{T\,(acceptable)} = 0.4348 / CM \quad (6)$$
$$Pf_{T\,(marginal)} = 2\, Pf_{T\,(acceptable)} \quad (7)$$

S refers to the annual maximum lateral loading and R to the platform lateral loading capacity (subscript 50 indicates the median values). σ is the standard deviation of the logarithim of S or R. Pf_T is the annual total probability of failure, Pf_O is the annual probability of operating failure, and Pf_S is the annual probability of storm (or environmental loadings) failure. β is the Safety Index and Φ [.] is the standard cumulative normal distribution. The formulation presumes that "unbiased" or "best estimate" values are used in the analyses. It is presumed that the "100-year" storm lateral loading is used as the reference loading and that R and S are lognormally distributed variables. It is presumed that the costs required to change the probability of failure of the platform by a factor of 10 vary linearly with the logarithm of Pf.

The FFP criteria are based on the following characterizations for Gulf of Mexico deep water drilling and production platform requalifications: $\sigma R = 0.25$ (capacity uncertainties) (Aggarwal 1991), $\sigma S = 0.75$ (storm loading uncertainties) (Olufsen and Bea 1990), and $PfS = 0.2\, PfT$. The assumption that $Pfo = 0.8\, PfT$ is based on data that shows that what causes platforms to fail is generally not related to environmental loadings or structure capacities. The predominant causes (e.g. 80 %) of platform failure are related to operations associated with the platform (blowouts, fires, explosions, and collisions) (Bekkevold et al. 1990; Bea and Moore 1991). Most of these operations hazards are related to "human and organization errors" (Paté-Cornell and Bea 1992).