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# A SIMPLIFIED STRUCTURAL RELIABILITY ANALYSIS PROCEDURE FOR USE IN ASSESSMENTS AND REQUALIFICATIONS OF TEMPLATE-TYPE OFFSHORE PLATFORMS

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## ABSTRACT

This paper summarizes development of a reliability based screening procedure that can be used in platform assessments and requalifications. The maximum static force acting on a platform system is treated as a function of random variables. Its statistical properties are derived considering the uncertainties associated with environmental conditions, structure conditions, kinematics, and force calculation procedures. The statistical properties of the capacity of the platform system are characterized using a combination of parallel elements and series components. The series components are the superstructure (deck), each bay of the substructure (jacket), and the foundation. The capacity of the platform is reached when the capacity of anyone of these components is reached. Within each component there are parallel elements: deck legs, braces, joints, and piles. In order for a component to reach its capacity, all of the parallel elements have to fail.

The proposed reliability analysis in this paper is based on a First Order Second Moment (FOSM) approach. A study is made of the implications of the simplified FOSM method. In the case of an eight-leg drilling and production platform located in Gulf of Mexico, the results from FOSM reliability analysis are compared with those from First and Second Order Reliability Methods (FORM and SORM). Lognormal and Type I Extreme Value distributions were selected for characterization of the expected annual maximum wave heights. In both cases the results are in good agreement with those from the simplified FOSM analysis. The FOSM safety indices are close approximations to those from FORM and SORM analyses. In addition to reliability indices for different failure modes, bounds for the system probability of failure are estimated.

Detailed structural reliability analyses of template-type offshore platforms are prohibitive, costly and experts are needed to perform the analyses. The simplified procedure introduced in this paper is meant to enable the average structural engineer to perform reliability analyses of jacket-type offshore platforms rapidly and with sufficient accuracy. The reliability based screening procedure identifies the potential failure modes and 'weak-links' of a platform system. The procedure can incorporate damaged, defective, and repaired elements. This procedure can be used to identify the critical platforms that need to undergo a detailed assessment so that limited available resources can be most efficiently utilized. Moreover, this method can be used in the design optimization of platform structures to help assure that desirable robustness and damage tolerance is incorporated into the structures.

## APPROACH

The simplified reliability analysis procedure presented in this paper is based on deterministic load and resistance formulations for template-type offshore platforms that have been previously developed and verified by the authors (Bea and DesRoches, 1993; Mortazavi and Bea, 1994; Bea and Mortazavi, 1995; Bea et al., 1995a; 1995b). Collapse mechanisms are assumed for the three primary components that comprise a template-type platform including the deck legs, jacket, and pile foundation. Based on presumed failure modes, the principle of virtual work is utilized to formulate the ultimate lateral loading capacity for each component. Where of significance, geometric and material nonlinearities are considered. The environmental loading conditions combination is chosen to be the wind speed component and current component that occur at the same time and in the same principal direction as the expected maximum wave height. The wave period is taken to be the expected period associated with the expected maximum wave height.

The reliability analysis procedure formulated in this paper is based on the assumption of two-state structural components; a component can be in a safe-state or fail-state. Furthermore it is assumed that the uncertainties associated with the state of the component can be described by random variables. For each platform component with the resistance $R$ and load $S$, the

probability of failure is equal to the probability that the load exceeds the resistance

$$p_f = P[R < S] \tag{1}$$

Assuming that $R$ and $S$ are random variables with the joint probability density function $f_{RS}(r,s)$, the probability of failure can be written as

$$p_f = \iint_{r \le s} f_{RS}(r,s)drds \tag{2}$$

In general, the resistance $R$ and the load $S$ are themselves functions of random variables. Assuming $X(x_1, x_2, \ldots, x_n)$ to be a set of random variables that completely describe the load and resistance characteristics with a joint probability density function $f_X(x)$, and further assuming that the state of the component is described by a function $g(x)$ so that $g(x)=<0$ indicates failure, the probability of failure can be given by the n-fold integral

$$p_f = \int_{g(x) \le 0} f_X(x)dx \tag{3}$$

$g(x)$ is often referred to as the limit state function. Problems associated with evaluating the above integral include: a) $f_X(x)$ may not be completely known due to lack of statistical data, b) the limit state function, $g(x)$, may not completely describe the true state of the component, and c) even in absence of the problems stated above, integrating the integral in Equation 3 can be a formidable task (Der Kiureghian, 1994).

To circumvent these problems, reliability measures under incomplete statistical information have been developed. Much of the early work on structural reliability analysis was based on such measures. The complete handling of this subject is not within the scope of this paper. The background used to develop simplified reliability analysis formulations for jacket-type offshore structures is summarized in the following.

Based on a mean value First Order Second Moment (FOSM) approximation, the reliability index and probability of failure are estimated for all potential failure modes. Using a first-order Taylor-series approximation around the mean point of the load and resistance equations formulated in a previous work (Mortazavi and Bea, 1995), the first two statistical moments of loads inposed on and capacities of platform components can be computed. Given that the resistance $R$ of a structural component is a function of random variables $(x_1, x_2, \ldots, x_n)$, its mean and standard deviation can be given by

$$\mu_R \approx R(M_x) \tag{4}$$

and

$$\sigma_R^2 \approx \nabla R|_{M_x} \Sigma \nabla R^T|_{M_x} \tag{5}$$

where

$$M_x = [\mu_{x1} \;\; \mu_{x2} \;\; \ldots \;\; \mu_{xn}] \tag{6}$$

is the mean vector of the resistance function and

$$\Sigma = \begin{bmatrix} \sigma_{x1}^2 & \rho_{12}\sigma_{x1}\sigma_{x2} & \cdots & \rho_{1n}\sigma_{x1}\sigma_{xn} \\ \rho_{21}\sigma_{x2}\sigma_{x1} & \sigma_{x2}^2 & \cdots & \rho_{2n}\sigma_{x2}\sigma_{xn} \\ \cdots & \cdots & \cdots & \cdots \\ \rho_{n1}\sigma_{xn}\sigma_{x1} & \rho_{n2}\sigma_{xn}\sigma_{x2} & \cdots & \sigma_{xn}^2 \end{bmatrix} \tag{7}$$

defines the covariance matrix, whereas

$$\nabla R = \left[\frac{\partial R}{\partial x_1} \;\; \frac{\partial R}{\partial x_2} \;\; \ldots\ldots \;\; \frac{\partial R}{\partial x_n}\right] \tag{8}$$

is the gradient vector of the resistance function which is evaluated at the mean vector in Equation (6). Similar formulations can be derived for the load function $S$. Defining a safety margin as

$$M = lnR - lnS \tag{9}$$

the probability of failure can be given by

$$p_f = CDF(U) \tag{10}$$

where

$$U = \frac{(M - \mu_M)}{\sigma_M} \tag{11}$$

is a standard variate with zero mean and unit standard deviation. $\mu_M$ and $\sigma_M$ are the mean and standard deviation of the safety margin respectively. Presuming Lognormal distribution for loads and capacities, the exact reliability index can be given as

$$\beta = \frac{\mu_M}{\sigma_M} \tag{12}$$

where

$$\mu_M = ln\left(\frac{\mu_R}{\mu_S}\sqrt{\frac{1+V_S^2}{1+V_R^2}}\right) \tag{13}$$

$$\sigma_M^2 = ln(1+V_R^2) + ln(1+V_S^2) - 2ln(1+\rho_{RS}V_R V_S) \tag{14}$$

and

$$P_f = \Phi(-\beta) \tag{15}$$

where $V$ denotes the coefficient of variation, $\rho_{RS}$ is the correlation coefficient between loading and capacity, and $\Phi(.)$ is the cumulative Standard Normal distribution function. Note that these equations and those derived for jointly normally distributed loads and capacities are the only known exact and

closed form solutions of the probability of failure for non-trivial distributions of loads and capacities. Unimodal bounds on probability of failure of a series system, $p_f$, can be estimated by

$$max\, P_{fi} < P_f < \sum_i P_{fi} \tag{16}$$

where $p_{fi}$ denotes the probability of failure of the $i^{th}$ component. The lower bound is based upon the assumption of perfect correlation among all component failure modes. The upper bound is based upon the assumption of no correlation among the component failure modes. In general, unimodal bounds are useful when there exists a dominating failure mode. However, in case of offshore platforms, the failure of different structural components have been shown to be strongly correlated mainly due to common dominating uncertainties in loading variables (Nordal, et al., 1988).

## LOADING AND CAPACITY FORMULATIONS

### Storm Loadings

All of the structure elements are modeled as equivalent vertical cylinders that are located at the wave crest (Bea, Mortazavi 1995). Appurtenances (boat landings, risers) are modeled in a similar manner. For inclined members, the effective vertical projected area is determined by multiplying the product of member length and diameter by the cube of the cosine of its angle with a horizontal axis. A combination of storm wind load and hydrodynamic wave and current loads is considered

$$S = S_w + S_h \tag{17}$$

The wind load is given by

$$S_w = K_w V_{wd}^2 \tag{18}$$

where $K_w$ is a structure dependent loading parameter, and $V_{wd}$ is the wind speed that occurs at the same time as the maximum wave height. The total integrated hydrodynamic drag force acting on a surface piercing vertical cylinder can be expressed as

$$S_h = K_d K_u H^2 \tag{19}$$

$K_u$ is an integration function that integrates the velocities along the cylinder and is a function of wave steepness and the wave theory used to estimate the velocities. $K_d$ is a force coefficient and a function of mass density of water $\rho$, diameter of the cylinder $D$, and drag coefficient $C_d$. The mean forces acting on the elements are integrated and the shear force at each component level is calculated. These integrated shear forces define the means of the load variables $S_D$ for deck, $S_J$ for each jacket bay, and the base shear $S_F$ for the foundation bay. The coefficient of variation of wave load is given as

$$V_S^2 = V_{K_d}^2 + V_{K_u}^2 + (2V_H)^2 \tag{20}$$

Since the dominating storm condition is the maximum wave height and its associated period, evaluation of the uncertainties in the wind forces does not play a major role and is not included.

### Deck Legs Shear Capacity

A mechanism in the deck leg bay would form when plastic hinges are developed at the top and bottom of all of the deck legs. Using this failure mode as a virtual displacement, virtual work principle can be utilized to estimate the deck leg shear resistance $R_d$ (Bea, Mortazavi 1995)

$$R_d = \frac{1}{H_d}(2n M_u - Q\Delta) \tag{21}$$

where

$$\Delta = M_u H_d \left( \frac{H_d}{6EI_d} + \frac{1}{C_r} \right) \tag{22}$$

$$\frac{M_u}{M_{cr}} - \cos\left( \frac{\pi}{2} \frac{Q/n}{P_{crl}} \right) = 0 \tag{23}$$

$H_d$ and $I_d$ are the height and moment of inertia of the deck legs. $M_u$ is the ultimate moment that can be resisted by the cross-section in the presence of axial load and is derived from the $M$-$P$ interaction equation for tubular cross-sections (equation 23). $\Delta$ denotes the deck bay drift at collapse. $C_r$ is a rotational stiffness parameter that accounts for the presence of jacket at the bottom of deck legs. $Q$ denotes the total vertical deck load and $n$ is the number of supporting deck legs. The moment capacity of the legs $M_{cr}$ and the local buckling capacity $P_{crl}$ are treated as random variables. According to equation (5) and assuming perfect correlation between $M_{cr}$ and $P_{crl}$, the variance of the deck legs capacity can be given as

$$\sigma_{Rd}^2 = \sigma_{M_{cr}}^2 \left( \frac{\partial R_d}{\partial M_{cr}} \right)^2 + \sigma_{P_{crl}}^2 \left( \frac{\partial R_d}{\partial P_{crl}} \right)^2 + 2\sigma_{M_{cr}} \sigma_{P_{crl}} \left( \frac{\partial R_d}{\partial M_{cr}} \right) \left( \frac{\partial R_d}{\partial P_{crl}} \right) \tag{24}$$

where $\frac{\partial R_d}{\partial M_{cr}}$ and $\frac{\partial R_d}{\partial P_{crl}}$ are the partial derivatives of the deck legs shear capacity, $Rd$, with respect to critical moment and buckling capacities, $M_{cr}$ and $P_{crl}$, evaluated at the mean values $\mu_{Mcr}$ and $\mu_{Pcrl}$.

### Jacket Bays Shear Capacity

Lateral shear capacity in a given jacket bay is assumed to be reached when the vertical diagonal braces or their associated joints are no longer capable of resisting the lateral load acting on the jacket bay. Tensile and compressive capacity of the diagonal braces, the associated joint capacities, and the batter component of axial forces in the legs due to overturning moment are included to estimate the jacket bay shear capacity (Bea, Mortazavi 1995).

It should be noted that the axial capacities of diagonal braces are negatively correlated with the lateral loading. This correlation is implicitly accounted for. Assuming a three-hinge failure mode for laterally loaded compression diagonals, the following equilibrium equation can be derived for the ultimate axial capacity of such members (Bea, Mortazavi, 1995)

$$P_u = \frac{M_u}{8\Delta_0\left(\frac{1}{1+2\frac{\sin 0.5\varepsilon}{\sin\varepsilon}}\right)\frac{1}{\varepsilon^2}\left(\frac{1}{\cos\frac{\varepsilon}{2}}-1\right)} - \frac{wl^2}{8\Delta_0} \quad (25)$$

where $\varepsilon = l\sqrt{\frac{P}{EI}}$ and $\Delta_0$, $P$, and $l$ are the initial out-of-straightness, axial force and unbraced length of the diagonal brace respectively. Thus the variance of the axial compression capacity of a transversally loaded brace can be given by

$$\sigma_{Pu}^2 = \sigma_{Pcr}^2 + \left(\frac{l^2}{8\Delta_0}\right)^2 \sigma_w^2 \quad (26)$$

where it is assumed that $\Delta_0$ is a deterministic parameter and that the first term in Equation (25) equals the buckling load of the brace in the absence of lateral distributed load $w$.

The tubular joint capacity equations given in the design guidelines API RP2A-LRFD (API, 1993) are adopted in this work. To obtain the statistical properties of the joint-brace element resistance, it is assumed that the tensile and compressive capacities of joint and vertical diagonal braces are Lognormally distributed. Using the results of structural system reliability for series systems, the cumulative distribution function of the ultimate axial loading capacity of a joint-brace-joint system can be given as

$$F_R(r) = 1 - \prod_i \left[1 - \Phi\left(\frac{\ln r - \lambda_i}{\xi_i}\right)\right] \quad (27)$$

where

$$\lambda_i = \ln(\mu_i) - \frac{1}{2}\xi_i^2 \quad (28)$$

$$\xi_i^2 = \ln\left(1 + \frac{\sigma_i^2}{\mu_i^2}\right) \quad (29)$$

where $\mu_i$ and $\sigma_i$ ($i$=1 to 3) denote the mean and standard deviation of the tensile or compressive capacity of the brace and its associated joints. Given the capacity distribution function $F_R(r)$, $\mu_R$ and $\sigma_R$, the mean and standard deviation of the capacity of the joint-brace system can be estimated using numerical integration.

To estimate the lateral loading capacity of a given jacket bay, it is assumed that interconnecting horizontal brace elements are rigid. Further, the notion of Most Likely To Fail (MLTF) element is introduced. MLTF member is defined as the joint-brace element with the lowest expected capacity over stiffness ratio. Thus, the lower-bound capacity of the $n^{th}$ jacket bay $R_{ln}$, which is associated with the first member failure in that bay, can be given as

$$R_{ln} = \sum_i \overline{\alpha}_n K_i + F_L \quad (30)$$

where $F_L$ is the sum of batter components of axial pile and leg forces in the given bay and

$$\overline{\alpha}_n = \frac{P_{u,MLTF}}{K_{MLTF}} \quad (31)$$

is the lateral drift of the $n^{th}$ jacket bay at the onset of first member failure. $K_i$ are deterministic factors accounting for geometry and relative member stiffness ($\overline{\alpha} K_i$ = horizontal shear force of brace element $i$ at the onset of first brace or joint failure within the given bay). Assuming no correlation between the capacity of the MLTF member and lateral shear in the jacket legs, the variance of the lower-bound capacity of the $n^{th}$ jacket bay can be given as

$$\sigma_{Rln}^2 = \sigma_{\overline{\alpha}}^2 \sum_i K_i^2 + B_{F_L}^2 \sigma_{F_L}^2 \quad (32)$$

where

$$\sigma_{\overline{\alpha}} = \frac{\sigma_{Pu,MLTF}}{K_{MLTF}} \quad (33)$$

$B_{FL}$ denotes the bias associated with the batter component of axial leg force, $F_L$, at the given bay. The upper-bound capacity of the $n^{th}$ jacket bay $R_{un}$, which is associated with failure of all main load carrying members in that bay, can be given as

$$R_{un} = \sum_i \alpha_i \overline{R}_i + F_L \quad (34)$$

where $R_i$ is the horizontal component of the ultimate axial capacity of the joint-brace element $i$. $\alpha_i$ account for the post-yielding behavior of semi-brittle brace or joint elements ($\alpha_i R_i$ = residual strength of element $i$) and are assumed to have deterministic values. Assuming perfect correlation among the member capacities $R_i$ and $R_j$ within the given bay, the variance of the upper-bound capacity of the $n^{th}$ jacket bay can be given as

$$\sigma_{Run}^2 = \sum_{i,j} \alpha_i \alpha_j \sigma_{Ri} \sigma_{Rj} + \left(B_{FL}\sigma_{FL}\right)^2 \quad (35)$$

## Foundation Capacity

Lateral and axial failure modes are considered in the pile foundation. The lateral failure mode is similar to that of the deck legs. In addition to moment resistance of the piles, the lateral support provided by foundation soils and the batter component

of the axial pile forces are considered. For clay and in case of linearly increasing shear strength with depth, the ultimate lateral capacity of the pile, $P_{u,l}$, can be estimated from the following equation (Mortazavi and Bea, 1995)

$$P_{u,l}(C+\xi)-2M_u-(A+\eta\xi)\frac{C^2}{2}-\left(\frac{\eta}{2}\right)\frac{C^3}{3}=0 \qquad (36)$$

where

$$C=\frac{1}{\eta}\left[(-A+\eta\xi)+\sqrt{(A+\eta\xi)^2+2\eta P_{u,l}}\right] \qquad (37)$$

$$\eta=\frac{B-A}{L_p} \qquad (38)$$

$$\xi=1.5D+X \qquad (39)$$

$$A=9S_{u1}D \qquad (40)$$

and

$$B=9S_{u2}D \qquad (41)$$

$S_{u1}$ and $S_{u2}$ denote the undrained shear strength at the mudline and pile-tip. $Lp$, $D$ and $X$ denote the embedded pile length, diameter and scour depth around the pile respectively. For a constant shear strength distribution over depth, Equation (36) reduces to that given by Tang and Gilbert (1990). For cohesionless soils, the distribution of lateral soil pressure along a pile at a depth $z$, is assumed to be

$$P_z=3\gamma z K_p \qquad (42)$$

$\gamma$ denotes the submerged unit weight of soil and $K_p$ is the passive lateral soil pressure coefficient and given by

$$K_p=\tan^2\left(45+\frac{\Phi}{2}\right) \qquad (43)$$

$\Phi$ is the effective angle of internal friction of the soil. The ultimate lateral force that can be developed at the pile top is (Tang and Gilbert, 1990)

$$P_{u,l}=\frac{2M_p}{\left[X+0.544\left(\frac{P_{u,l}}{\gamma D K_p}\right)^{0.5}\right]} \qquad (44)$$

The axial resistance of a pile is based on the combined effects of a shear yield force acting on the lateral surface of the pile and a normal yield force acting over the entire base end of the pile. Thus the ultimate axial capacity, $Q$, can be expressed as

$$Q=Q_s+Q_p=q\,A_p+f_{av}A_{sh} \qquad (45)$$

$Q_p$ denotes the ultimate end bearing and $Q_s$ is the ultimate shaft capacity, $q$ is the normal end yield force per unit of pile-end area acting on the area of pile tip $A_p$, and $f_{av}$ denotes the ultimate average shear yield force per unit of lateral surface area of the pile acting on embedded area of pile shaft $A_{sh}$. It is assumed that the pile is rigid and that shaft friction and end bearing forces are activated simultaneously. It is further assumed that the spacing of the piles is sufficiently great so that there is no interaction between the piles.

Taking into account the uncertainties in soil and pile material properties and biases associated with capacity modeling and using the FOSM approach described earlier, the uncertainty in foundation capacities can also be estimated. In the case of lateral pile capacity, the uncertainty associated with the batter component of the pile force is added to the total capacity uncertainty for vertically driven piles.

## EXAMPLE APPLICATION

Using the formulations presented in this paper, the structural reliability of an actual offshore platform has been evaluated. Located in the Main Pass area of the Gulf of Mexico, the eight-leg template-type platform is installed in a water depth of approximately 271 feet (Figure 1). Designed and installed in 1968-70, the platform has been exposed to high environmental loading developed by hurricanes passing through the Gulf (Bea, et al 1995). Because the wind loading is less than 10 % of the total lateral loading, only wave force is considered in the reliability evaluation. According to oceanographic studies performed for the site, the 100 year return period wave height, $H_{100}$, is 70 feet. The uncertainties associated with the expected annual maximum wave height predictions are summarized in Table 1 (Bea 1990). Considering the uncertainties in the predicted wave heights results in a total bias (true value / predicted value) of $B_H$ =1.1 and a coefficient of variation of $V_H$ =0.34. Assuming Lognormal and Type I Extreme Value distributions, the probabilistic characteristics of the expected annual maximum wave heights are summarized in Table 2. The wave height distribution parameters were determined by fitting the tails of the distributions to the predicted extreme wave heights. The variabilities of the force coefficients developed by Bea (1990) are used to estimate the uncertainties associated with the wave forces (Table 3). These estimates are consistent with the simplified analytial models employed to calculate the loadings.

It should be noted that once the wave crest elevation exceeds the platform lower deck elevation, the load pattern changes significantly and the total forces acting on the platform increase much faster than before. In this example, this fact has not been accounted for. In general, the problem can be circumvented by considering conditional probabilities ($p_f|H$). In this case, the total probability of failure can be estimated by

$$p_f=\int_h p_f|_H f_H(h)dh \qquad (46)$$

Analytical models have been developed and verified to calculate the wave-in-the-deck forces (Bea, et al. 1995; Loch, Bea, 1995).

The structural reliability of the example platform is studied for the two principal orthogonal directions. For each load direction, eight different failure modes are identified and analyzed; one in the superstructure, five in the substructure, and two foundation failure modes.

For critical bending moment, $M_{cr}$, local buckling capacity, $P_{crl}$, and global buckling capacity of diagonal braces, $P_{cr}$, the mean-value curves given by Cox (1987) are utilized. These are represented by the following equations

$$M_{cr} = M_p\left[1.113 exp\left(\frac{-1.638 f_y D}{Et}\right)\right] \tag{47}$$

$$P_{crl} = P_y\left[1.79 - 0.25\left(\frac{D}{t}\right)^{0.25}\right] \tag{48}$$

$$P_{cr} = P_y\left(1.03 - 0.24\lambda^2\right) \quad \text{for } 0 < \lambda < 1.7 \tag{49}$$

where

$$M_p = Z f_y \tag{50}$$

$$P_y = A f_y \tag{51}$$

$$\lambda = \left(\frac{1}{\pi}\right)\left(\frac{KL}{r}\right)\sqrt{\frac{f_y}{E}} \tag{52}$$

where $Z$, $\lambda$, and $K$ are the plastic section modulus of the cross-section, slenderness ratio of the member, and buckling length factor respectively. For bending resistance, a combined coefficient of variation (COV) of 0.106 is given by Cox(1987). The COV for local buckling is 0.117, which includes the test uncertainties, uncertainties in steel yield strength (COV =0.08), and uncertainties associated with fabrication. This value is reported to be constant over the entire range of practical values of $Et/f_yD$ and $D/t$ respectively. The uncertainties of column resistance over a practical range of $\lambda$ are given in Table 4 (Cox, 1987). In addition to uncertainties associated with test and fabrication, the uncertainties associated with yield stress $f_y$, elastic modulus $E$, and effective column length factor $K$ are included in the column resistance uncertainty.

A tubular joint failure mode is not included in the presented reliability analysis since the leg-pile annulus and the joints are grouted and the joints are significantly stronger than the braces. In the general case, the joints capacities and the uncertainties and biases can be included in the analyses.

In this example, the upper-bound capacity formulation is used for the jacket bays. Deterministic values are assigned to brace residual strength factors $\alpha$, which are calibrated to give results consistent with those gained from a detailed nonlinear pushover analysis of the studied platform (Bea, et al., 1995). In a general case however, the $\alpha$ factor is unknown and can be considered as a random variable itself. The uncertainty associated with this variable reflects the modeling uncertainty introduced by using simplifying assumptions regarding residual strength of compression braces and stiffness properties of inter-connecting horizontal braces.

Due to lack of data regarding the pile capacity modeling uncertainty, the total uncertainties recommended by Tang and Gilbert (1990) are used, which are based on test results and implicitly include the model uncertainties (Table 5). The uncertainty associated with the batter component of the pile force is added to the total capacity uncertainty given for vertical piles. Available test data on axially loaded piles indicate a very wide range in capacity bias. The uncertainties associated with axial capacities of driven piles are given by Tang (1988) (Table 6). Current studies of the performance characteristics of platforms subjected to storm loadings indicates that the mean biases in lateral and axial pile capacities indicated in Tables 5 and 6 represent a lower bound (mean biases in the range of 2 to 3) (Young, Bea, 1995; Bea et al. 1995a; 1995b).

To study the effect on FOSM results of different probability distributions of maximum wave height and nonlinear limit state functions, the computer program CALREL (Liu et. al., 1989) was used to perform FORM and SORM analyses in addition to FOSM analysis. In the case of Lognormally distributed load and capacity variables, the results from the simplified FOSM analysis and those from more sophisticated FORM and SORM are given in Tables 8 and 9. FORM and SORM analyses have also been performed assuming a Type I Extreme Value distribution for the annual expected maximum wave heights. No significant changes in the reliability indices are observed for the two different distributions of expected annual maximum wave heights. The FOSM safety indices are close approximations to those determined from the FORM and SORM analyses.

The results summarized in Tables 8 and 9 and Figures 2 and 3 indicate that the most probable failure mode in both loading cases involves the failure of the diagonals in the second jacket bay. The large uncertainties in storm loadings are due to uncertainties in force calculations and those associated with predicted wave heights. The large uncertainties in jacket bay capacities are mainly due to uncertainties associated with the lateral loading and the load-capacity correlation which is implicitly accounted for in this analysis. The uncertainties in lateral capacity of jacket bays are larger for the broadside loading direction than the end-on loading direction. This can be explained by the fact that for the broadside loading case, compressive buckling of diagonal K-braces govern the failure of the jacket, whereas in the case of end-on loading, tensile yielding of diagonal braces govern the ultimate lateral loading capacity of the jacket. The compressive buckling of the braces is associated with much larger uncertainties than the tensile yielding. The foundation piles have safety indices that are comparable with those in the superstructure. Given recognition of the additional mean biases cited earlier, the foundation safety indices would be increased substantially.

Based on the FOSM results, unimodal bounds on annual probabilities of failure are estimated for both loading directions and given in Table 7. The failure probabilities range from about 1 % per year to 4 % per year depending on the assumptions regarding correlation of the failure modes. Given the large loading uncertainties relative to those of the component capacities, one would expect the correlation of the failure modes

to be nearly unity (Nordal et al. 1988). Thus, the most realistic failure probability would be represented by the lower bound results.

## SUMMARY AND CONCLUSIONS

A simplified procedure is presented to perform structural reliability analysis of conventional, jacket-type, offshore platforms. Simplified formulations of storm loadings on and capacities of such platforms are utilized to develop a probabilistic failure analysis procedure. These formulations have been developed and verified during recent studies by the authors (Bea and DesRoches, 1993, Mortazavi and Bea, 1994, Bea and Mortazavi, 1995, Bea et al., 1995, Young, Bea, 1995). Comparisons of these formulations with results from full three dimensional nonlinear analyses indicates a sligltly conservative bias of the forces and an essentially unbiased estimate of the components and system capacities. The reliability analysis procedure presented in this paper is based on a first order second moment approach. It is assumed that the loads and capacities are lognormally distributed. The correlation between loads and capacities is implicitly accounted for.

A case study is performed and the structural reliability of an eight-leg offshore drilling and production platform located in Gulf of Mexico is studied. In addition to reliability indices for different failure modes, unimodal bounds for the system probability of failure is estimated. Using the computer program CALREL, first order and second order reliabilities are also computed. Two different distributions are selected for the maximum wave height; Lognormal and Type I Extreme Value. In both cases the results are in good agreement with those from the simplified FOSM analysis. The FOSM safety indices are close approximations to those derived from FORM and SORM analyses.

The simplified procedure introduced in this paper is meant to enable the average structural engineer to rapidly perform structural reliability analyses of jacket-type offshore platforms. This reliability based screening procedure can be used to identify the critical structures that need to undergo a detailed assessment so that the limited available resources can be most efficiently utilized. Moreover, this method can be used in the design optimization of platform structures to help assure that desirable robustness or damage tolerance is incorporated into the structures.

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**Table 1: Wave Height Uncertainties**

| | $\sigma_{lnH}$ | *Bias* ($B_H$) | $\sigma_{lnBH}$ |
|---|---|---|---|
| $H_{max}$ | 0.3 | 1.1 | 0.13 |

**Table 2: Probabilistic Characteristics of the Maximum Wave Height**

| $f_H(h)$ | $\mu_H$ (ft) | $\sigma_H$ (ft) |
|---|---|---|
| *Lognormal* | 34.5 | 11.7 |
| *Type I largest* | 34.0 | 11.4 |

**Table 3: Force Coefficient Uncertainties (Bea, 1990)**

| | $\sigma_{lnK}$ | *Bias* ($B_K$) | $\sigma_{lnBK}$ |
|---|---|---|---|
| $K_u$ | 0.1 | 0.41 | 0.47 |
| $K_d$ | 0.1 | 1.67 | 0.23 |

**Table 4: Column Resistance Uncertainties (Cox, 1987)**

| $\lambda$ | 0.4 | 0.6 | 0.8 | 1.0 | 1.2 | 1.4 |
|---|---|---|---|---|---|---|
| COV | 0.099 | 0.100 | 0.106 | 0.119 | 0.150 | 0.212 |

**Table 5: Lateral Pile Capacity Uncertainties (Tang, 1990)**

| Lateral Capacity in: | Bias | COV |
|---|---|---|
| Clay | 0.92 | 0.20 |
| Sand | 0.81 | 0.21 |

**Table 6: Axial Pile Capacity Uncertainties (Tang, 1988)**

| Axial Pile Capacity in: | Bias | COV |
|---|---|---|
| Sand | 0.9 | 0.47 - 0.56 |
| Clay | 1.3 - 3.7 | 0.32 - 0.53 |

**Table 7: Unimodal Bounds on $p_f$ (Example Platform)**

| Loading | Lower Bound $p_{fs}$ | Upper Bound $p_{fs}$ |
|---|---|---|
| End -on | 0.011 | 0.046 |
| Broadside | 0.013 | 0.042 |

**Table 8: Component Reliabilities, Broadside Loading (Example Platform)**

| BROADSIDE LOADING | LOAD (KIPS) | BIAS | C.O.V | CAP (KIPS) | BIAS | C.O.V | FOSM β | FOSM Pf | FORM β | SORM β |
|---|---|---|---|---|---|---|---|---|---|---|
| DECK LEGS | 197 | 0.83 | 1.03 | 2606 | 1.00 | 0.11 | 3.64 | 1.34E-04 | 3.51 | 3.51 |
| JACKET | | | | | | | | | | |
| BAY1 | 544 | 0.83 | 1.03 | 2932 | 1.00 | 0.08 | 2.61 | 4.52E-03 | 2.48 | 2.48 |
| BAY2 | 621 | 0.83 | 1.03 | 2621 | 1.00 | 0.24 | 2.22 | 1.32E-02 | 2.08 | 2.12 |
| BAY3 | 638 | 0.83 | 1.03 | 4130 | 1.00 | 0.45 | 2.41 | 8.07E-03 | 2.41 | 2.47 |
| BAY4 | 641 | 0.83 | 1.03 | 5702 | 1.00 | 0.49 | 2.67 | 3.85E-03 | 2.75 | 2.80 |
| BAY5 | 643 | 0.83 | 1.03 | 6157 | 1.00 | 0.48 | 2.75 | 2.94E-03 | 2.84 | 2.90 |
| FOUNDATION | | | | | | | | | | |
| LATERAL | 643 | 0.83 | 1.03 | 7700 | 0.81 | 0.56 | 2.69 | 3.58E-03 | 2.67 | 2.67 |
| AXIAL | 1035 | 0.83 | 1.03 | 4063 | 1.5 | 0.31 | 2.52 | 5.79E-03 | 2.49 | 2.49 |

Table 9: Component Reliabilities, End-on Loading (Example Platform)

| END-ON LOADING | LOAD (KIPS) | BIAS | C.O.V | CAP (KIPS) | BIAS | C.O.V | FOSM β | FOSM Pf | FORM β | SORM β |
|---|---|---|---|---|---|---|---|---|---|---|
| DECK LEGS | 120 | 0.83 | 1.03 | 2808 | 1.00 | 0.11 | 4.22 | 1.20E-05 | 4.10 | 4.10 |
| JACKET | | | | | | | | | | |
| BAY1 | 424 | 0.83 | 1.03 | 1954 | 1.00 | 0.07 | 2.43 | 7.51E-03 | 2.29 | 2.29 |
| BAY2 | 499 | 0.83 | 1.03 | 2046 | 1.00 | 0.10 | 2.28 | 1.13E-02 | 2.12 | 2.14 |
| BAY3 | 515 | 0.83 | 1.03 | 2360 | 1.00 | 0.15 | 2.39 | 8.54E-03 | 2.25 | 2.28 |
| BAY4 | 518 | 0.83 | 1.03 | 2538 | 1.00 | 0.20 | 2.43 | 7.62E-03 | 2.28 | 2.32 |
| BAY5 | 520 | 0.83 | 1.03 | 2892 | 1.00 | 0.26 | 2.51 | 6.02E-03 | 2.38 | 2.43 |
| FOUNDATION | | | | | | | | | | |
| LATERAL | 520 | 0.83 | 1.03 | 7200 | 0.81 | 0.53 | 2.88 | 1.96E-03 | 2.85 | 2.85 |
| AXIAL | 856 | 0.83 | 1.03 | 4063 | 1.5 | 0.31 | 2.74 | 3.12E-03 | 2.70 | 2.70 |

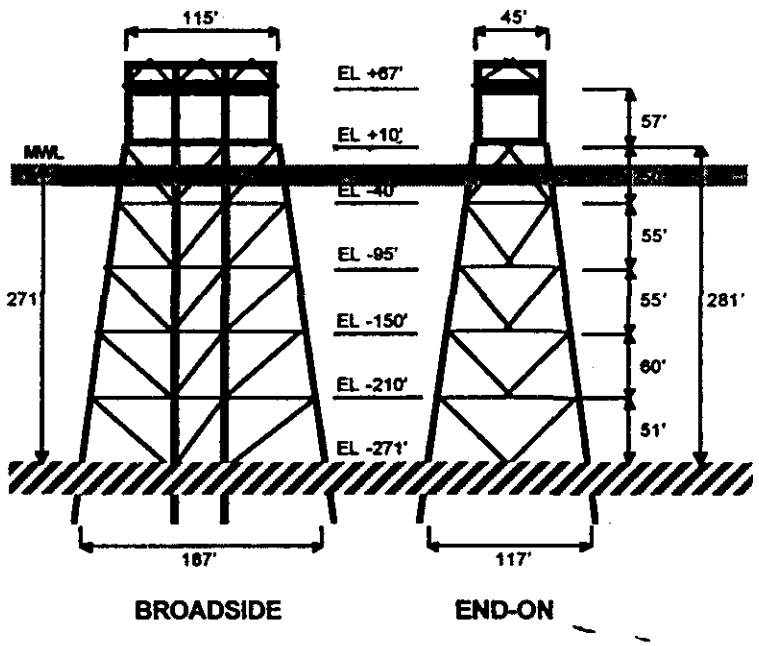


Figure 1: Example Platform Elevations

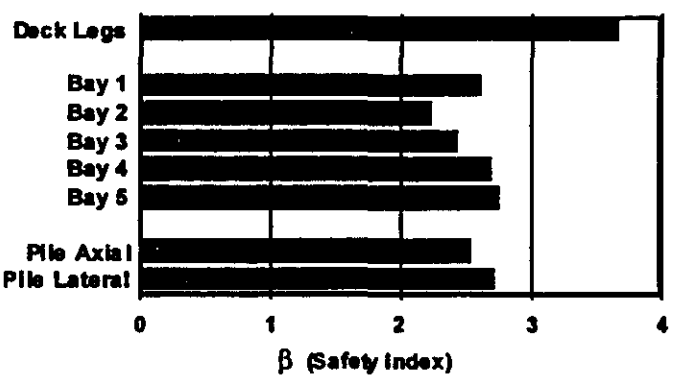


Figure 2: Component Safety Indices (Annual) for Broadside Loading

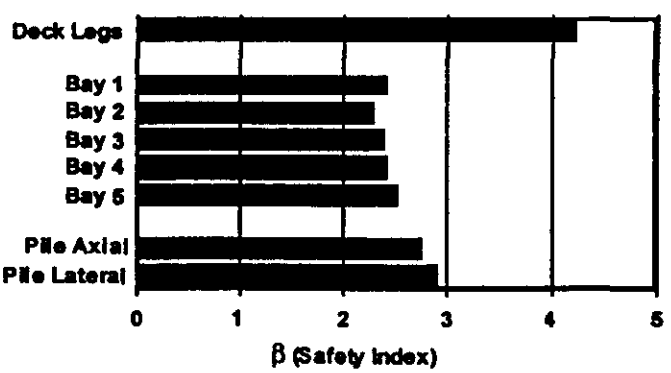


Figure 3: Component Safety Indices (Annual) for End-on Loading