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PROBABILISTIC MODELLING FOR RELIABILITY ANALYSIS OF JACKETS

Jan Mathisen
Det Norske Veritas, N-1322 Høvik, Norway
Email: Jan.Mathisen@dnv.com

Knut O. Ronold
Det Norske Veritas, N-1322 Høvik, Norway
Email: Knut.Ronold@dnv.com

Gudfinnur Sigurdsson
Det Norske Veritas, N-1322 Høvik, Norway
Email: Gudfinnur.Sigurdsson @dnv.com

ABSTRACT

Experience from recent reliability analyses of jacket platforms is used to discuss selected aspects of probabilistic modelling in more detail. These modelling details can have a significant effect on the computed reliabilities. An overview of basic considerations and failure modes in jacket reliability analysis is included to set the various details into context. Ultimate limit states for jackets in relatively shallow water are emphasised; i.e. quasi-static structural response is applicable. The following topics are considered:

(a) Failure modes and some requirements to load and resistance analysis.
(b) Directionality in loading and resistance.
(c) Random periods of individual extreme waves.
(d) Foundations – axial and lateral capacity modelling for multiple piles and model uncertainty for pile capacity.

Keywords: jackets, structural reliability, directional effects, pile capacity.

INTRODUCTION

Two of the present authors published a paper on the reliability analysis of a jacket at OMAE last year [3]. That paper and subsequent jacket analysis work gave us the idea of exploring a few aspects of the reliability analysis of jackets in somewhat more detail, and led to the present paper. Although this paper is less extensive than we planned, we hope it may generate some discussion and contribute a little to the development of reliability analysis procedures for jackets. The exploration of random directionality presented here arises naturally in relation to jackets, but should have implications for reliability analysis of other types of structures, too.

NOMENCLATURE

g	limit state function
l	applied base shear force
l_C	characteristic base shear capacity
l_M	mean load in an environmental state
l_S	load standard deviation in an environmental state
l_0, l_2	load parameters
r	resistance
r_C	characteristic resistance
r_0, r_2	resistance parameters
u_C	model uncertainty factor on capacity
u_L	model uncertainty factor on loading
λ_A	capacity factor, normalised wrt. individual load cases
λ_B	capacity factor, normalised wrt. characteristic base shear capacity
θ	direction
θ_L	direction with largest load coefficient
θ_R	direction with largest resistance
ς	height of line of action of base shear force

OVERVIEW

Basic Analysis Considerations

In relatively shallow water, the natural periods of the dominant modes of global lateral vibration of jackets tend to lie well below the periods of the incoming regular waves. This permits quasi-static structural analysis. Dynamic structural analysis is required in deeper water, when the natural periods are longer and can be excited by the incoming waves. Nonlinear load-effects due to drag forces and to wave elevation can produce excitation forces at other frequencies, typically at two or three times the wave frequency. These effects may also have to be taken into account when choosing the method of structural analysis.

The sub-division between response analysis methods is also reflected in the methods applied to load analysis. The relative contributions of the various wave periods in a sea state are important for accurate assessment of dynamic response, and tend to require application of a wave spectrum, through frequency or time domain methods. For quasi-static structural response, the peak values of the applied loads are of most importance, permitting use of regular waves; i.e. a single wave period in the load model. Of course, the regular wave height and period have to be carefully selected to replace the more detailed spectral representation of the sea state.

Dynamic amplification factors can be used to extend the quasi-static analysis procedure in regular waves to include jackets with some dynamic response. However, detailed comparison of dynamic and quasi-static analyses results may be required to derive accurate dynamic amplification factors.

The structural elements of jackets tend to be relatively slender, such that drag forces give rise to a significant portion of the hydrodynamic loads due to waves and current. The nonlinear nature of the drag forces is a little awkward in a frequency domain analysis and requires some form of linearization. The nonlinear drag forces do not present any difficulty to load analysis in regular waves and hence, tend to favour this approach. Furthermore, it is straightforward to represent simultaneous wave kinematics at all points within the jacket in regular wave analysis, but more arduous to do so in irregular waves.

The present paper is primarily concerned with jackets in relatively shallow water, implying quasi-static structural analysis and load analysis in regular waves.

Failure Modes

The ultimate limit state (ULS) tends to be critical for the global design of jackets in relatively shallow water, whereas the fatigue limit state (FLS) can be more critical in deeper water, with dynamic response. The FLS can sometimes be critical for local design in shallow water. Pushover analysis is commonly applied to the ULS; e.g. as implemented in ref.[1] & [2]. The passage of a regular wave past the jacket is discretised into a number of time steps with associated hydrodynamic (and wind) loads. The instant giving the highest load is selected. This environmental load is gradually applied in the pushover analysis and further incremented by a load factor, while the displacement is computed to describe a series of equilibrium states. Plastic behaviour of the structural elements of the jacket is taken into account. Nonlinear models for the behaviour of the foundation piles are included. The maximum capacity is found at the peak of the load and displacement curve, as indicated in Figure 1. Thus, the capacity may be expressed in terms of a capacity factor λ_A, which is equal to the value of the load factor at this point. If the capacity factor equals 1.0, then the specified load lies exactly on the limit state surface. The details of the failure mode may be found from the underlying pushover analysis, and include plastic collapse of struts or legs, lateral soil failure, axial pile failure, and combinations of these modes, etc.

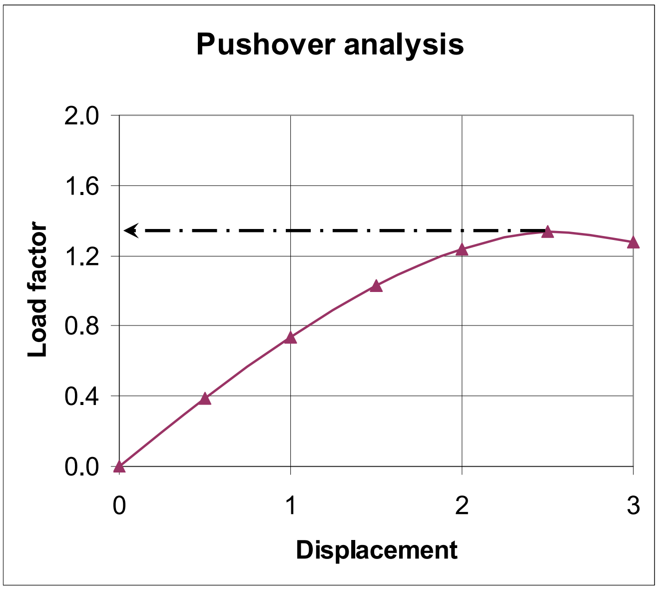

Figure 1 Load displacement curve from pushover analysis in a regular wave. The maximum is indicated as a capacity factor.

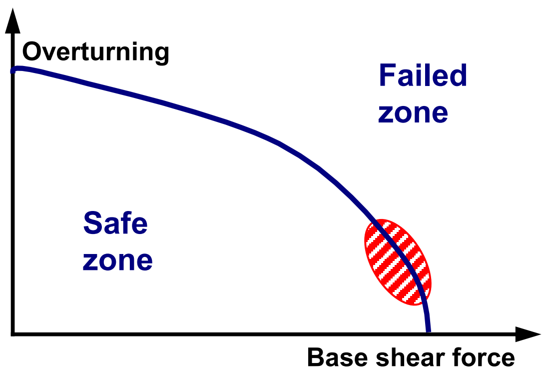

Figure 2 Interaction sketch for jacket capacity, where cross-hatched zone indicates range of probable limit states in shallow water

In shallow water the load and capacity is fairly well characterised in terms of the base shear force acting on the jacket. If only base shear is considered, then a limit state function might be defined as:

(capacity for base shear force) - (applied base shear load)

Of course, the environmental load is really distributed over the jacket structure and is more fully characterised in terms of the base shear force and an overturning moment, as indicated in Figure 2. The limit between safe and failed zones may be thought of as the locus of all combinations of base shear and overturning moment with a capacity factor of $\lambda_A = 1.0$.

Hence, a more precise limit state function may be formulated as:

$$g = \lambda_A - 1 \quad (1)$$

There are finer details in the effects of the load and capacity distributions over the jacket on the ULS which are not brought out in Figure 2, but they are included in Eq.(1), when the capacity factor is based on a detailed pushover analysis.

The capacity factor in eq.(1) is a function of all the physical random variables defining the structural capacity and applied loads in the pushover analysis. If the details of the load distribution over the structure are of less importance, then the load and capacity can be separated to some extent. In this case, the load capacity factor λ_B may be normalised relative to a characteristic base shear force l_C and expressed as a function of height of the line of action of the force ς and the compass direction of the force θ. Then the limit state function may be written in terms of the base shear force as

$$g = \lambda_B(\varsigma, \theta) \cdot l_C - l \quad (2)$$

where l is the applied base shear force. The base shear force l, the force height ς and the force direction direction θ are all dependent on the environmental conditions, including the directions of wind , waves and current. The characteristic force l_C is not varied with direction. Dependency on the height of the force can be omitted in some cases; e.g. for purely lateral failure of the foundation.

Model uncertainty factors for capacity u_C and load u_L can conveniently be included in the limit state function as

$$g = u_C \cdot \lambda_B(\varsigma, \theta) \cdot l_C - u_L \cdot l \quad (3)$$

The loads and capacity are dependent on a large number of physical parameters, many of which may need to be modelled as random variables. Details of one case study are given in [3], where response surface methods are applied to handle these dependencies in the reliability analysis. Some aspects of the dependencies on load direction and on wave period are discussed in the following sections

DIRECTIONAL EFFECTS

It tends to be computationally arduous to take full account of directional effects in a reliability analysis. A simplified model is developed in the following, to permit investigation of some aspects. Fourier polynomials with only a few terms are used to model some typical features of jacket load and resistance here. This type of function can be extended with additional terms, for increased accuracy, and used to interpolate between numerical results from detailed jacket analyses at discrete headings.

Directional model

Directional limit state

The limit state function is written as

$$g = r(\theta) - l(\theta) \quad (4)$$

where both resistance r and load l are dependent on direction θ, and the same direction is assumed for both environmental effects and the induced loads. Symmetry about a vertical plane in the longitudinal axis through the centre of the jacket is assumed.

Directional resistance

The resistance is expressed by

$$r(\theta) = r_C \left(r_0 + r_2 \cos[2(\theta - \theta_R)] \right) \quad (5)$$

where r_C is the characteristic resistance, r_0, r_2 are directional resistance parameters and $\theta_R = 0$ indicates the direction with the greatest resistance; i.e. along the longitudinal axis through the centre of the jacket.

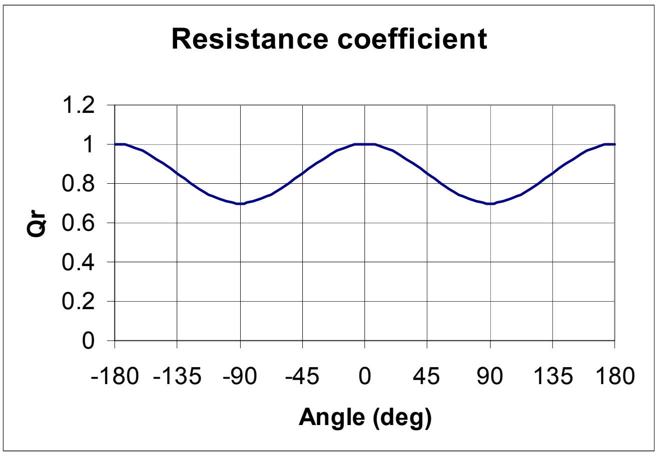

Figure 3 Variation in resistance coefficient with direction.

This Fourier polynomial provides symmetry of resistance about longitudinal and transverse axes, a lower resistance in the transverse directions than in the longitudinal directions, and smooth variation through intermediate directions. These properties are typical of jacket resistance when length and breadth are unequal. An eight-legged jacket in 80 m water depth might typically have $r_2/r_0 = 0.15/0.85$, as illustrated in Figure 3, based on calculations of lateral foundation failure similar to those reported in ref.[3]. Subsequent parameter variation is carried out such that $r_0 + r_2 = 1.0$.

Directional load

The load coefficient is expressed by

$$Q_L(\theta) = \left(l_0 + l_2 \cos[2(\theta - \theta_L)]\right) \quad \textbf{(6)}$$

where l_0, l_2 are directional load parameters and $\theta_L = \pi/2$ indicates the direction with the highest load coefficient; i.e. along the transverse axis through the centre of the jacket. Similar reasoning is applied in the choice of the Fourier polynomial for the load coefficient, as is used for the resistance, but the peak load coefficient is found in the transverse, rather than the longitudinal direction. When the breadth is smaller than the length of the jacket, then the loads on the various elements tend to be more nearly in phase when waves are propagating in the transverse direction. Of course, this is also dependent on the ratio of the wavelength to the platform dimensions. A typical jacket, as mentioned above, might have $l_2/l_0 = 0.10/0.90$, as illustrated in Figure 4.

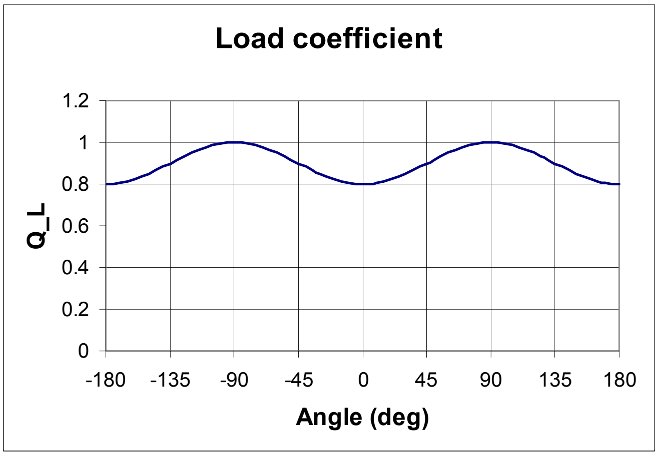

Figure 4 Variation in load coefficient with direction.

The mean and standard deviation of the load in a short term environmental state are expressed by

$$l_M = k \cdot Q_L(\theta) \quad \textbf{(7)}$$

$$l_S = k \cdot Q_L(\theta) \quad \textbf{(8)}$$

where k is an environmental intensity factor, that is also dependent on direction. The load maxima are assumed to be distributed according to the Rayleigh distribution in a short term environmental state, with a mean period of 8s between maxima. The maxima are assumed independent and the distribution of short term extreme largest in a 3-hour state $L^*(\theta;3hr)$ is obtained from a Gumbel distribution, via a probability transformation from Rayleigh to an auxillary exponential distribution. This random variable is applied as the load in the limit state equation (4). This type of distribution model is commonly applied to wave loads.

Directional environment

The long term probability density of environmental directions is written as

$$f(\theta) = \left(1 + a_1 \cos(\theta - \theta_E)\right)/(2\pi) \quad \textbf{(9)}$$

where the 1st order coefficient is set to $a_1 = 0.333$ in the present example and the dominant direction is specified by $\theta_E = 0.0$. This distribution is illustrated in Figure 5. The present example is chosen such that the dominant direction leads to the highest environmental intensity, and this direction is aligned with the platform direction that tends to maximise the resistance and minimise the load coefficient. Such situations are not uncommon, and may be found in the South China Sea, for example, where the north-east monsoon is dominant for part of the year and produces the highest waves. However, the present model is not specifically fitted to this location. Note that the density is defined on a range $(-\pi, \pi)$ and is zero elsewhere. The length of this range is fixed, but the location is arbitrary. Hence, the mean value of the direction is also arbitrary. This is somewhat unusual, as compared to other types of random variables, and needs some extra consideration when applying asymptotic reliability methods.

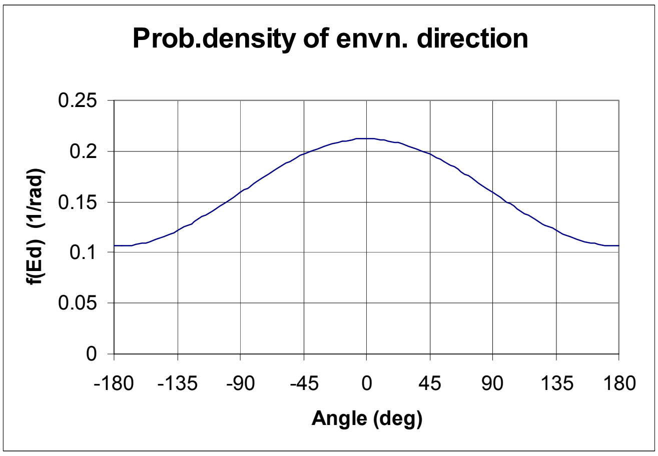

Figure 5 Probability density function for environmental direction (scaled w.r.t. radians, but with angle shown in degrees).

In the reliability analysis, the direction distribution can be obtained by a probability transformation from an auxillary variable with a uniform distribution. This was originally set up with the uniform variable defined on the range (-1,+1). The present symmetrical case is then conveniently handled by redefining the uniform variable on the range (0,1), without modifying the original probability transformation.

The short term environmental intensity k takes a long term Weibull distribution, written as

$$F_K(k;\theta) = 1 - \exp\left\{ -\left(\frac{k}{\alpha_K(\theta)} \right)^{\beta} \right\} \quad \textbf{(10)}$$

where the distribution parameters are $\beta = 1.0$ and

$$\alpha_K(\theta) = \left(k_0 + k_1 \cos(\theta - \theta_K)\right) \quad \textbf{(11)}$$

with coefficients $k_0 = 0.7,\ k_1 = 0.3$, and most severe direction $\theta_K = 0.0$. The parameter of the environmental intensity distribution is illustrated in Figure 6.

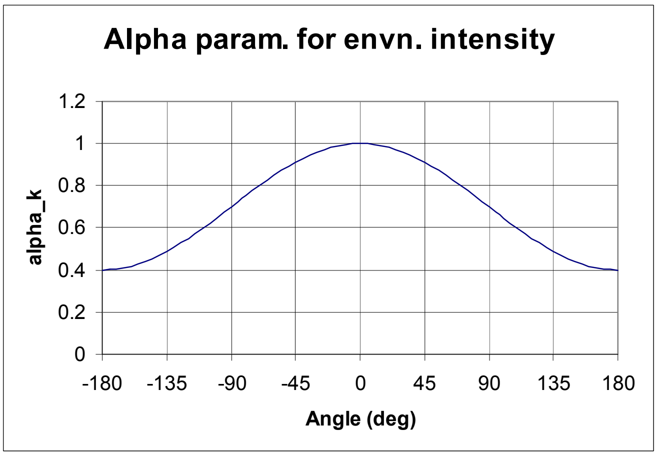

Figure 6 Variation in distribution parameter for environmental intensity with direction.

The simplified environmental loads are intended to represent the effects of wind, waves and current on a typical 8-legged jacket, with only one plane of symmetry. The distribution of environmental direction and intensity resembles the directional distributions found for these effects, and the distributions commonly applied for significant wave height or wind speed.

Probabilistic model

No time-independent random variables are included in this analysis, in order to allow more scope for computational investigation of directional effects; i.e. there is no model uncertainty on load or resistance and although the resistance is dependent on direction, it is otherwise deterministic. The marginal probability of failure in a single, random short term state is obtained from the limit state equation (4) and integration with respect to the distributions of environmental direction, environmental intensity and short term extreme load.

$$P_f(3hr) = \int_{g<0} f_{\Theta}(\theta) f_{K|\Theta}(k|\theta) f_{L^*|K,\Theta}(l|k,\theta;3hr)\, d\theta\, dk\, dl \quad \textbf{(12)}$$

The annual probability of failure is subsequently obtained by taking account of the number of short term states in a year N_{yr}, as

$$P_f(1yr) = 1 - \left[1 - P_f(3hr)\right]^{N_{yr}} \quad \textbf{(13)}$$

The PROBAN program is used for the reliability calculations [4].

Directional results

The model is scaled to a typical probability level by inserting a characteristic resistance of $r_C = 69.0$ to be multiplied by the resistance coefficient in (5). Results from a parameter study on the mean resistance parameter are shown in Figure 7. Four types of reliability methods are applied in the calculation. The second order reliability method (SORM) and directional simulation agree closely, while the first order reliability method overestimates the probability of failure by a factor of from 2 to 5. These directional simulation results are unbiased and have been computed with a coefficient of variation of 6% to 7%. One point has been checked by Monte Carlo simulation of 10^8 sample points, yielding a coefficient of variation of 12%, and showing good agreement with directional simulation and SORM. The results seem fairly typical of directional problems, which tend to show sufficient curvature in the limit state function (in the transformed standard normal space) such that the FORM results are inaccurate.

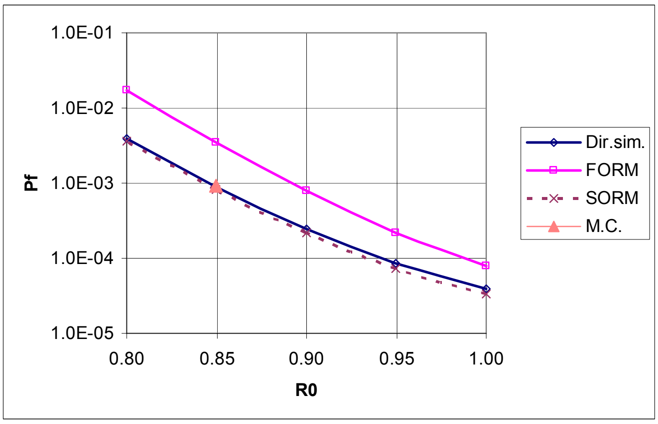

Figure 7 Annual probability of failure as a function of mean resistance parameter for 4 types of calculation.

Similar problems with FORM have previously been observed by the authors in another reliability problem with strong directional content; viz. the ultimate limit state for a single mooring line in a spread mooring. A simple explanation of the different performance of FORM with respect to directional random variables as opposed to other types of random variables may perhaps be provided by remembering that FORM provides a linear approximation to the failure surface, such that:

- more effective approximation is provided to effects that behave monotonically; e.g. loads tend to increase with random wave height and lateral resistance tends to increase with soil strength,
- whereas less effective approximation is provided to effects that do not behave monotonically; e.g. the load coefficient tends to decrease on both sides of the peak angles in Figure 4 and the resistance tends to increase on both sides of the troughs in Figure 3.

SORM may be expected to be more effective, since it is based on a second order approximation to the failure surface.

WAVE PERIOD

Longuet-Higgins [5] provides an expression for the short term distribution of the period of an individual wave, conditioned on the wave height. This distribution may also be found in Massel's textbook [6]. It is more amenable for use in reliability analysis than the distributions given by Cavanie et al. [7] and by Lindgren and Rychlik [8]. Retaining most of the notation from the paper [5], this conditional probability density may be written as

$$f_{T|R}(t|r) = \frac{1}{\sqrt{\pi}\nu F(r/\nu)} \frac{r}{t^2} \exp\left\{-\frac{r^2}{\nu^2}\left(1-\frac{1}{t}\right)^2\right\} \quad \textbf{(14)}$$

where $t = \tau/\bar{\tau}$ is a normalised wave period, with τ as the wave period and $\bar{\tau}$ as the mean wave period. $r = \rho/\sqrt{2m_0}$ is a normalised wave amplitude, with ρ as the wave amplitude and m_0 as the zero order moment of the wave spectrum. The wave amplitude is taken to be half the wave height. $\nu = \sqrt{m_0 m_2 / m_1{}^2 - 1}$ is a spectral width parameter, with m_1 and m_2 as the first and second moments of the wave spectrum. Now the function $F(r/\nu)$ is a well-known error function, that may be neglected for our purposes, in relatively large waves, when $r > 1$. This distribution is applicable to the period of the individual wave applied to load a jacket platform, when the wave height is drawn from the annual extreme wave period.

When implementing this distribution in a reliability analysis, it is convenient to employ a transformation to an auxiliary variable u_T, with a standard normal distribution function, as defined by

$$\phi(u_T)\cdot du_T = f_{T|R}(t|r)\cdot dt \quad \textbf{(15)}$$

$$\frac{u_T}{\sqrt{2}} = \frac{r}{\nu}\left(1-\frac{1}{t}\right)$$

where $\phi(.)$ is the normal density function. The auxiliary variable is modelled as an input variable in the reliability analysis, and a realisation of the individual wave period can be obtained as follows, in terms of more convenient parameters

$$\tau = \frac{T_Z\sqrt{\nu^2+1}}{1-\dfrac{u_T \nu H_S}{2h}} \quad \textbf{(16)}$$

where h is the individual wave height, H_S is the associated significant wave height, and T_Z is the zero-up-crossing period. There is a possibility that the transformation may lead to negative wave periods for large values of u_T. This possibility is related to the correction made by the error function in the original distribution. The transformation is not allowed at the transition between positive and negative values, at $u_T = 2h/\nu H_S$. Some bound needs to be set to avoid these values in the reliability computation, because they may give rise to numerical difficulties. Another bound should be set to avoid wave height and period combinations that imply breaking waves. Both bounds can easily be implemented using a truncated normal distribution for the auxiliary variable u_T.

An example of the conditional wave period distribution is shown in Figure 8. The distribution tends to become more narrow and centered on the mean period as the individual wave height increases. The upper bound on u_T has no real influence in this example, but the bound for breaking waves obviously has appreciable influence. The example applies to a severe sea state, and to an extreme individual wave height in that state $(h/H_S = 2.0)$. The bound for breaking waves would have less effect in milder conditions. This wave period distribution is primarily based on mathematical theory for narrow-banded processes, without making direct use of the physical properties of ocean waves. The bound for breaking waves takes some of these properties into account, but it also seems that there may be further scope for improvement of the distribution model for individual wave periods under extreme conditions.

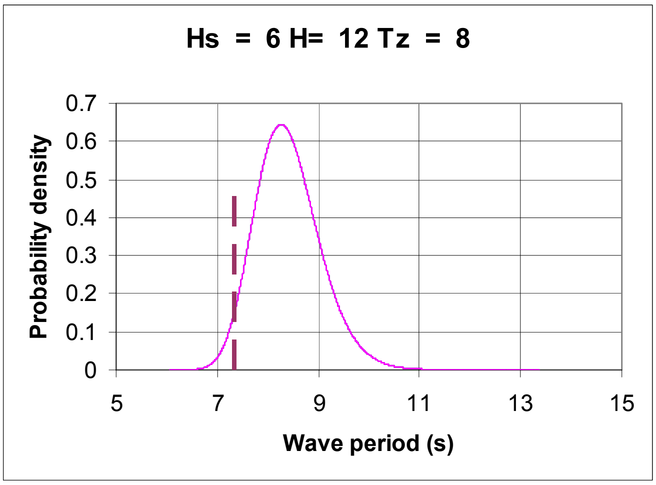

Figure 8 Conditional distribution of periods of individual waves for $h = 12m, H_s = 6m, T_Z = 8s, \nu = 0.3$.

A deepwater bound for breaking waves is indicated at 7.3 s.

Prior to introducing this conditional wave period distribution it might typically be assumed that the period corresponding to the extreme individual waves would be close to the peak period of the wave spectrum. The conditional distribution tends to lead to a shorter wave period, closer to the average period of the spectrum. This change in wave period was found to lead to an appreciable increase in wave loads in the reliability analysis of a jacket platform.

GEOTECHNICAL ISSUES

The axial capacities of the piles in a jacket foundation provide the resistance against the overturning moment on the jacket, while the lateral capacities provide the resistance against the base shear.

For evaluation of the overall stability of a jacket foundation under extreme loading conditions, the spatially averaged soil strength properties over the extent of the foundation are of interest. Soil strength properties exhibit spatial connectivity vertically as well as laterally, i.e. there is correlation between the soil strengths from one point to another within the soil volume. The horizontal correlation length of the soil strength field is usually much larger than the vertical correlation length.

Axial pile capacity

The axial capacities of the piles in the foundation come about as the skin friction integrated over the respective fairly long pile lengths. For the axial capacities, it can therefore be assumed that the effects of the local fluctuations of the skin friction from point to point along each pile will average out over the length of the pile, and the axial capacities of all piles can thus be represented by capacities calculated from average skin friction properties only, without considering any local variability

Lateral pile capacity

The lateral capacities of the piles come about from a much more localized soil strength, i.e. it arises from the soil strength in a limited zone near the soil surface. The vertical extent of this zone is so limited that for practical purposes it will not be reasonable to count on any effect of spatial averaging vertically. This leaves to consider spatial averaging horizontally for its influence on the lateral capacities of the piles in the foundation.

The lateral pile capacity is proportional to the undrained shear strength of the soil. The spatial average of the lateral pile capacities over the lateral extent of the foundation will therefore come about in the same manner as the spatial average of the undrained shear strength over this extent. In the structural reliability analysis, it will therefore suffice to represent the lateral capacity of each pile as the lateral capacity that comes about from a calculation on the basis of a spatially averaged undrained shear strength.

In a reliability analysis, the spatially averaged undrained shear strength can be expressed as

$$S_{u,spatial} = E[S_u] + U_S \cdot \sqrt{\lambda} \cdot \sigma \tag{17}$$

in which $E[S_u]$ and σ denote the mean value and the standard deviation, respectively, of the local undrained shear strength. U_S denotes a standard normally distributed variable, and λ is a variance reduction factor associated with the spatial averaging in the horizontal plane and whose value is less than 1.0. This representation is based on an assumption of a Gaussian strength field.

For a jacket foundation of N piles, the value of λ can be established from Monte-Carlo simulations of joint outcomes of an N-dimensional standard normal variable **X** whose correlation matrix is an NxN matrix with entries calculated according to an expression for the correlation coefficient $\rho(\Delta r)$ between two piles located a horizontal distance Δr apart. The ith element X_i in **X** represents the local variability in the strength at the ith pile among the N piles in the foundation. For the entry in the ith row and jth column of the correlation matrix, Δr comes about as the distance between the ith pile and the jth pile in the foundation. When a joint outcome of $\mathbf{X}=(X_1,...X_N)^T$ is simulated, this gives one outcome of the derived variable

$$Y = \frac{1}{N}\sum_{i=1}^{N} X_i \tag{18}$$

The distribution of Y can be established from an adequate number of Monte-Carlo simulations of **X** (and thus of Y). The variance reduction factor λ can be interpreted as the variance of Y that results from the simulations and will be a function of the model for the horizontal correlation structure for the soil strength and of the horizontal correlation length in this model. The commonly assumed quadratic exponential decay model,

$$\rho(\Delta r) = \exp(-(\frac{\Delta r}{R})^2) \quad (19)$$

in which the correlation length R represents the horizontal scale of fluctuation, will produce a variance reduction factor λ as indicated in Figure 9 for an example foundation.

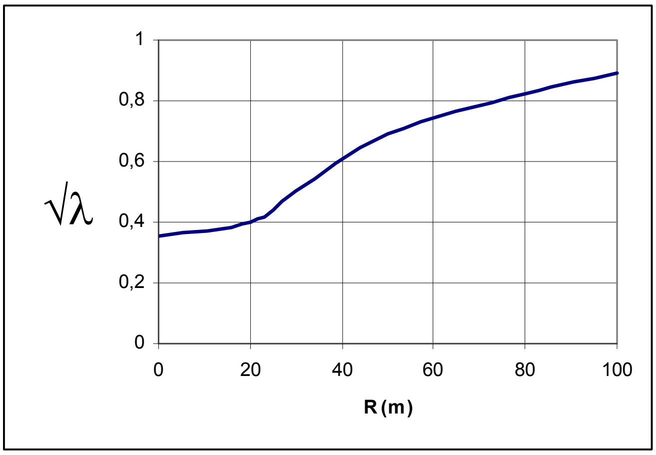

Figure 9 Example of standard deviation reduction factor vs. horizontal correlation length for an 8-legged jacket.

Note that the emphasis in the above considerations has been placed on the total lateral capacity of the entire pile foundation, as there is no reason to expect that a variation in lateral capacity between the individual piles will lead to failure of one pile and thereby initiate a progressive failure scenario. This is so, because if one (weak) pile would tend to fail, then part of the load on this pile would immediately become redistributed through the jacket structure to the other (stronger) piles. The failure mode for the jacket foundation in lateral loading is thus a global failure mode, governed by the soil-structure interaction and the total lateral capacity of the piles.

Model uncertainty

Another geotechnical issue of great importance for a reliability analysis of a jacket foundation is the model uncertainty associated with prediction of the axial pile capacities. In a reliability analysis, the model uncertainty can be represented by a random model uncertainty factor F. The factor is defined as the ratio between the true axial pile capacity and the predicted axial pile capacity and is applied as a factor on the capacity as predicted by the chosen capacity model in the limit state function. The distribution of the stochastic model uncertainty factor F can be assessed based on data from full-scale tests on piles, from which the observed pile capacity, measured as the axial pile load at failure, can be interpreted as a measure of the true pile capacity. For establishing the distribution of F, data bases of results from full-scale tests on piles need to be consulted. Several such data bases exist, however, they cover a wide range of soil types and in many cases a very limited number of tests are available from each location. This puts a limit on which data can be of use for the piles of a particular jacket foundation. A mean value of F different from 1.0 indicates a biased prediction model. A data base, which is commonly referred to, is the so-called API data base presented in ref.[9].

CONCLUSIONS

Some features of reliability analysis of jackets have been discussed and may be summarised as follows:

- Detailed reliability analysis with respect to random directions of environmental effects is demanding, and can usefully be explored with simplified models.
- FORM analysis tends to provide inaccurate results for the effects of random directions, while SORM appears to be reasonably accurate, when the conditional probability of failure with respect to direction has a single, dominant peak.
- The mean direction is arbitrary and should be chosen such that the dominant peak is kept well away from the ends of the direction range, when SORM (or FORM) is applied.
- The individual wave period associated with an extreme wave height may be modelled using a distribution function due to Longuet-Higgins [5]. This distribution appears to need a bound against breaking waves. Shorter wave periods than the spectral peak period tend to be associated with the highest waves when using this distribution.
- Average skin friction properties may be applied to establish the axial capacity of piles, without considering local variability.
- Spatial averaging of the undrained shear strength of the soil is required to establish the lateral capacity of a pile foundation.

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