EXTREME WIND AND WAVE LOADS
SITE-SPECIFIC COMBINATION OF DESIGN RULES

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Abstract. Background: The present paper regards the concerted action of wind and wave loads on offshore wind turbines in the extreme storm event. This load combination problem involves the definition of the characteristic loads and safety factors. In wind engineering and offshore engineering well established practices for the definition of characteristic values and safety factors for wind and wave loads exist separately. The aim is to investigate the possibility of making a merger of these existing practices into a site-specific design rule. Method of Approach: The paper applies a simplified probabilistic approach giving an understanding of how the merging can possibly be established and finally gives first guidance on the choice of characteristic values and safety factors. Results and conclusions: Under the assumptions made herein, it is demonstrated how a combination rule can be established for mildly site-specific conditions. This model assumes that one has just enough data to asses the marginal distributions of the storm peak values of mean wind speed and significant wave height, their correlation and the possible delay of the arrival of the wave storm relative to the wind storm.

Keywords: Offshore wind turbines, site-specific, extreme wind and wave loads, load combination, design rules, partial safety factors, structural reliability.

INTRODUCTION

The problem of combining extreme wind and wave loads on offshore wind turbines has special interest. For offshore wind turbines the extreme wind and wave loads will be comparable in magnitude; the balance between wind and wave loads will of course depend on site. In contrast to this, extreme wave loads generally govern design of conventional fixed platforms for offshore oil and gas exploitation. Therefore existing offshore standards place emphasis on the wave loading and pay little attention to the combination with wind loads. Generally wind loads are only considered important for structures on the deck, e.g. masts, cranes etc., and are not combined with wave loads.

A number of current standards for offshore wind turbines [1, 2, 3] address the problem of load combination of wind and wave loads. The guidelines in these standards are different with respect to degree of detail on how to perform combination of extreme wind and wave loads. In general guidance on how one could perform the load combination for site-specific conditions is needed.

The load combination problem involves the definition of the characteristic loads and safety factors. In wind engineering and offshore engineering well established, but separate, practices for the definition of characteristic values and safety factors for wind and wave loads exist. In [4] the aim was to investigate the possibility of making a simple merger of these existing practices into a possibly conservative design rule. This was done based on the simplifying assumption that wind and wave storms arrive simultaneously and have fully correlated peak values. Here the aim is to investigate the possible generalisations of the results from [4] to site-specific applications yielding less conservative results. As it will be seen this cannot be done without an extra effort in terms of acquiring more data and demonstrating their reliability. A simple model, based on the marginal distribution of storm peak values, their correlation and the possible time delay between wind and wave storm peaks is proposed for mildly site-specific conditions; however, one still needs to demonstrate its validity for a given site.

1 A SIMPLE CONSERVATIVE MERGER OF EXISTING DESIGN RULES

This section gives a summary of the findings in [4], which will be the starting point of the present paper. First the main result of the paper is repeated, and then assumptions on which it rests are presented.

1.1 RELIABILITY CONSIDERATIONS

A design rule consists in the specification of characteristic values, safety factors, and a limit-state equation to be used together with the aim of ensuring a desired level of structural reliability. In most civil engineering standards load factors are applied to the characteristic external loads prior to load effect calculations, i.e. calculations of deflections, cross-sectional forces and so forth. Such standards may be said to follow a load code format. Motivated by the dynamical sensitivity of wind turbine structures practise in wind turbine engineering has become to compute the load effect generated by the characteristic loads; these load effects are termed characteristic load effects. The load factors are then applied to the characteristic load effects instead of the characteristic external loads. Thus wind turbine standards may be said to follow a load effect code format. The advantage of the load code format is that different safety factors can be used for the different loads acting on the considered structure. On the contrary different load factors are not simple to implement in a load effect code format. Different attempts have been made to devise so-called weighted load factors. None of them are yet properly scientifically substantiated. Motivated by the convenience in having the same load factor on characteristic wind and wave load effects it was in [4] investigated if this would lead to the same level of structural reliability. If this would not be the case, it would be hard to apply a load effect code format to an offshore wind turbine. As described, applying the load factors to load effects is wind turbine engineering practice. When designing the support structures of offshore wind turbines one would make use of standards for design of offshore structures. To the author's knowledge all of these standards make use of a load code format. This problem may be solved by the introduction of an imaginary interface between tower and substructure at a suitable point over the maximum sea elevation. For the structure above the interface the load effect format is applied, and for the part below the interface a traditional load format is applied. For the structure below the interface the cross-sectional forces at the interface, which are obtained from the load effects from above the interface, are, when multiplied by the load effect partial safety factor, considered as external design loads. The drawback is that one then has to be a little careful about dynamic interaction, but it is possible. The author has failed to identify the origin of this proposal.

Whether loads or load effects are in the end or going to be considered, the detailing and accuracy of the load-response models considered herein imply that the reliability analysis will be equally valid for both cases, at least with respect to the load safety factors.

The work reported in [4] let to the recommendation that the same load factor, namely 1.35, could be used for both wind and wave loads. This requires that the characteristic load effects are determined from a certain recipe. Both the wind and wave loads must be determined from 50-yr return period envi-

ronmental parameters. The averaging period of the wind parameters shall be 10 min and so should the reference period. The reference period of the extreme sea state should be 3 hr. Thus, in accordance with common practice, the characteristic wind load effect can be established as the 10-min extreme response to the 50-yr 10-min mean wind speed with appropriate turbulence added, and the characteristic wave load effect can be the 3-hr extreme value obtained from 50-yr significant wave height. This result rests on a lot of assumptions out of which the most important are summarized below, but basically the conclusion was that it was not possible, with the given assumptions, to reject the use of the same load factors provided the characteristic values are determined as explained. The strength of this result is that it allows the use of well-established practices in wind turbine and offshore engineering, except that the typical return period for environmental parameters in offshore applications is 100 yr and not 50 yr. The result leaves an unanswered question: how should then the combined wind-wave load effect be determined? For instance the reference periods are different. This issue is taken up in Sec. 1.2. First follows details of the assumptions behind the result, as these will be needed in this paper.

The assumptions have to do with the limit state model and the so-called stochastic model. The stochastic model is an assembly of the various random variables that accounts for the uncertainties related to the assessment of external loads, load effects, and material strengths or component resistances.

1.1.1 LIMIT STATE MODEL

Models for purely aerodynamic load effects and purely hydrodynamic load effects in any cross-section in the support structure are needed. It is important to appreciate that simplified physical modelling of the load effect is used. This is acceptable because considerable uncertainty sources overrule the finer details in the physical modelling. The aim is that the physical models should reproduce relevant characteristics of load effect time series. Thus one should have a fair chance of understanding how the different uncertainty sources affect the computed reliabilities.

Consider first the aerodynamic load effect. The extreme load effect $E_{\mathrm{max,aero}}$ in a given reference period, say 10 minutes, is assumed approximately Gumbel distributed with mean and standard deviation:

$$E[E_{\mathrm{max,aero}}] = c_{\mathrm{inf,aero}} \underbrace{\tfrac{1}{2}\rho V_{\mathrm{hub}}^2}_{p} \underbrace{(1+2k_p I_1 c_{\mathrm{amp}})}_{c_{\mathrm{dyn,aero}}}$$
$$D[E_{\mathrm{max,aero}}] = c_{\mathrm{inf,aero}} \underbrace{\rho V_{\mathrm{hub}}^2}_{2p} I_1 c_{\mathrm{amp}} \frac{\pi}{\eta\sqrt{6}} \qquad (1)$$

where

$$k_p = \eta + \frac{\gamma}{\eta} \quad , \quad \eta = \sqrt{2\log f_0 T} \qquad (2)$$

The factor $c_{\mathrm{inf,aero}}$ is an influence number relating the load effect $E_{\mathrm{max,aero}}$ to the product $pc_{\mathrm{dyn,aero}}$ of the mean wind pressure and the dynamic response factor. The influence number depends on the design of the turbine; among others $c_{\mathrm{inf,aero}}$ depends on the geometry of tower and the blades, and the lift and drag coefficients. The air density ρ and the mean wind speed V_{hub} at hub height give the mean wind pressure p, and the factor $c_{\mathrm{dyn,aero}}$ is the dynamic response factor given by the peak factor k_p, the along wind turbulence intensity I_1, and the factor c_{amp} which accounts for dynamic amplification and for admittance. Because the admittance depends on lift and drag coefficients, c_{amp} depends on the design of the turbine just like $c_{\mathrm{inf,aero}}$ does. Finally the peak factor depends on the 1st tower frequency f_0 implying that k_p depends on the design too. The expression in Eq. (1) is believed to represent for tower bottom response well but not necessarily well-suited for blades.

For the hydrodynamic load effect in a given reference period the following model is used:

$$E_{\mathrm{max,hydro}} = c_{\mathrm{inf,hydro}} H_{\mathrm{max}}^2 \underbrace{(1+D)}_{c_{\mathrm{dyn,hydro}}} \qquad (3)$$

The factor $c_{\mathrm{inf,hydro}}$ is an influence number relating the load effect $E_{\mathrm{max,hyro}}$ to the product $H_{\mathrm{max}}^2 c_{\mathrm{dyn,hydro}}$. The influence number depends on drag (and mass) coefficients and geometry, that is, $c_{\mathrm{inf,hydro}}$ depends on the design. The maximum wave height H_{max} is raised to the power 2 corresponding to drag-dominated loads in deep water. This limitation of the present case study is motivated by the fact that one of the latest studies on structural reliability calibrations for offshore structures is related to the ISO standard 19902 for steel offshore structures [5]. That study [8] considers drag wave load dominated structures because that is the case for steel offshore structures for oil drilling, which are typically situated at deep waters. Many of today's offshore wind turbine support structures experience a combination of drag and inertia wave loading. However, for the author, getting reliable information about uncertainty models for inertia-dominated wave loads has been impossible. In contrast to the model for the aerodynamic load effect, which had explicitly a long-term part (the mean wind pressure p) and a short-term part (covered by the dynamic response factor $c_{\mathrm{dyn,aero}}$) the model for the hydrodynamic load effect has the long-term and short-term variations united in H_{max}, which basically implies an assumption of quasi-static response. Minor dynamical amplification is covered by the deterministic dynamic amplification factor $c_{\mathrm{dyn,hydro}}$. This model is assumed appropriate for today's offshore turbines

that exhibit some dynamical amplification of wave loads, though not to the same extent as the turbulent wind load is amplified.

Finally a model for the resistance is required. Since load factors are the main subject here and because these are not that sensitive to a very accurate modelling of the resistance the expression

$$R_{\mathrm{b}} = c_R f_{\mathrm{y}} \tag{4}$$

is used. Here R_{b} stands for the bearing capacity of a structural member, f_{y} is yield strength, and c_R is a proportionality constant. This simple expression is representative for most cases. The capacity of columns is an exception and for concrete structures more complicated expressions for the resistance may be relevant. However, the level of detailing of (4) is compatible with the modelling of the response modelling used in this paper.

The limit states are now defined by

$$E_{\mathrm{max,aero}} = R_{\mathrm{b}} \qquad \text{and} \qquad E_{\mathrm{max,hydro}} = R_{\mathrm{b}} \tag{5}$$

In order to evaluate the reliability of the support structure the influence numbers $c_{\mathrm{inf,aero}}$ and $c_{\mathrm{inf,hydro}}$ which go into the expressions for $E_{\mathrm{max,aero}}$ and $E_{\mathrm{max,hydro}}$, respectively, must be known. Because the influence numbers are given by the design of the support structure equations that lead to $c_{\mathrm{inf,aero}}$ and $c_{\mathrm{inf,hydro}}$ are required. These will be the so-called design equations that determine the design in terms of safety factors and characteristic values. The design equations are obtained from the limit state equations by substituting the material strengths and loads by their design values. By tradition in wind engineering the characteristic value of $E_{\mathrm{max,aero}}$ is its expected value. Thus, see Eq. (1), the design equation for the aerodynamic load effect is defined by

$$\frac{c_R f_{\mathrm{y},c}}{\gamma_m} = \gamma_f c_{\mathrm{inf,aero}} p_c (1 + 2k_p I_c c_{\mathrm{amp}}) \tag{6}$$

in which γ_m and γ_f denote the material and the load effect safety factors respectively, and the subscript c stands for characteristic value. Using the same notation the design equation for the hydrodynamic load effect becomes

$$\frac{c_R f_{\mathrm{y},c}}{\gamma_m} = \gamma_f c_{\mathrm{inf,hydro}} H^2_{\mathrm{max},c} c_{\mathrm{dyn,hydro}} \tag{7}$$

In order not to over estimate the reliability the support structure is assumed designed to the limit. Thus

$$c_{\mathrm{inf,aero}} = \frac{c_R f_{\mathrm{y},c}}{\gamma_m \gamma_f p_c (1 + 2k_p I_c c_{\mathrm{amp}})} \tag{8}$$

and

$$c_{\mathrm{inf,hydro}} = \frac{c_R f_{\mathrm{y},c}}{\gamma_m \gamma_f H^2_{\mathrm{max},c} c_{\mathrm{dyn,hydro}}} \tag{9}$$

The scene is now set to lay down the expressions for the evaluation of the structural reliability. Because the obtained reliabilities are not directly interpretable as failure rates, see e.g. [15], the reliabilities will preferably be given in terms of reliability indices, denoted β, which, per definition, relates to the reliability via $\Phi(-\beta) = p_f$, where p_f is the probability of failure, i.e. $1-p_f$ is the reliability, and Φ denotes the standard normal cumulative distribution function (cdf).

Take the aerodynamic case first:

$$\Phi(\beta) = \Pr\{c_R F_{\mathrm{y}} X_m \geq c_{\mathrm{storm}} c_{\mathrm{inf,aero}} P X_{Ea} (1 + X_{\mathrm{dyn}} T)\} \tag{10}$$

In Eq. (10) F_{y} and P denote the random variables corresponding the physical variables f_{y} and p. The random variables X_m and X_{Ea} are material and aerodynamic load effect model uncertainties, respectively. The factor c_{storm} has been introduced as a correction factor for the non-conservatism of the characteristic value resulting from neglecting most of the storm profile, i.e. neglecting the fact that the duration of a storm is more than 10 min (and 3 hr). In [4] it is demonstrated that $c_{\mathrm{storm}} \approx 1.05$ for wind loads. Strictly speaking a model uncertainty factor X_{storm} should have been introduced, however data has not yet been sufficient to allow for such detailed modelling, and it is assumed that the main influence of neglecting the storm profile is well captured by the mean bias. The random variable T accounts for the Gumbel distributed 10-minute maximum load effect conditional on the 10-minute mean wind speed. The expectation of T is $2k_p I_c c_{\mathrm{amp}}$. As an approximation this expectation is set to 1, implying that the expected 10-minute maximum response is twice the 10-minute mean response. Finally X_{dyn} accounts for uncertainties in the modelling of the dynamical response to the turbulent wind load. Substituting Eq. (8) into Eq. (10) leads to the following expression for the reliability against aerodynamic loads

$$\Phi(\beta) = \Pr\{\gamma_m \gamma_f \tilde{F}_{\mathrm{y}} X_m \geq c_{\mathrm{storm}} \tilde{P} X_{Ea} \frac{1 + X_{\mathrm{dyn}} T}{2}\} \tag{11}$$

where the tildes denote that the variables have been normalized by their characteristic values. Similarly one obtains for the reliability against hydrodynamic loads

$$\Phi(\beta) = \Pr\{\gamma_m \gamma_f \tilde{F}_{\mathrm{y}} X_m \geq c_{\mathrm{storm}} \frac{\tilde{H}^2_{\mathrm{max}}}{X_{Eh}}\} \tag{12}$$

Table 1: inherent, model and statistical uncertainties for hydrodynamic and aerodynamic loads and resistance.

Name	Description	Type	Bias	CoV	Char. Value
$\tilde{H}^2_{\max}$	Annual maximum hydrodynamic load effect, pure drag, normalised by characteristic value	$\text{Min}_3(k=1)$	1.00	31%	98%/ 99%
		Gumbel	1.00	*20%/ 30%*	
X_{Eh}	Model uncertainty, hydrodynamic loads	Gaussian, truncated at $\pm 1.5\sigma$	1.09/ *1.00*	10%	Mean
		Lognormal	*1.00*	*10%/ 15%*	
$\tilde{P}$	Annual maximum 10-minute mean wind load effect normalised by charac. value	Gumbel	1.00	32%	98%
X_{Ea}	Model uncertainty, aerodynamic loads	Lognormal	1.00	10%	Mean
T	Normalised extreme turbulent response	Gumbel	1.00	10%	Mean
X_{dyn}	Model uncertainty	Lognormal	1.00	5%	Mean
$\tilde{F}_{\text{y}}$	Normalised yield strength	Lognormal	1.13	5%	5%
X_m	Model uncertainty	Lognormal	1.11	8.5%	Mean

In [4] it is demonstrated that for drag dominated wave loads $c_{\text{storm}} \approx 1.10$. In Eq. (12) X_{Eh} accounts for model uncertainties associated with the load modelling. The different position of the model uncertainty variables in Eq. (11) and Eq. (12) merely reflects differences in arbitrary choices made in [6] and [8], respectively.

1.1.2 STOCHASTIC MODEL

The uncertainty models for aerodynamic and hydrodynamic load effects are adopted from [6] and [8], respectively. The uncertainty model for the resistance is adopted from [8]. The models are recapitulated in Table 1. All distributions used are two-parameter distributions, implying that they are fully defined by their coefficient of variation and their characteristic value and/or bias, which, inhere, is defined as the ratio of the actual characteristic value to assumed characteristic value. These quantities are specified in the columns entitled 'CoV', 'Char Value', and 'Bias'. The type of the distribution is given in the 'Type' column. In the 'Char Value' column the numbers specify percentiles.

The uncertainty model used in [8] is based on many detailed and thorough studies, e.g. [10], however its application in the current context deserves some clarifying comments. Since [8] (and [10]) deals with drag-dominated loads in deep waters it is the distribution of $\tilde{H}^2_{\max}$ that is given in the first row of Table 1. The text in italics states alternatives to the distributions used in [8]. The alternatives are proposed for parametric studies. The distribution Min(k=1) is a shifted exponential obtained by tail fit to generic long-term response distributions produced according to the Storm Sea Method, see e.g. [9]. The distribution includes model uncertainties and statistical uncertainties associated with the assessment of the distribution of the extreme wave climate. Because [10] considers UK waters the distribution used for $\tilde{H}^2_{\max}$ represents storms at North Sea sites with deep water. At other sites the hydrodynamic loads can follow other distributions why one must bear in mind that the results obtained below are North Sea-specific. The associated model uncertainty X_{Eh} has a conservative bias of 1.09 reflecting the anticipation that the ISO 19902-CD scheme (recipe) for assessment of extreme wave loads is followed. If this recipe is not followed, a skilled engineer could potentially do better and the conservative bias should be neglected, as proposed in the alternative. In [8] the model uncertainty has CoV 18%. Because [8] deals with larger and more complex support structures than those generally used for offshore wind turbines it is anticipated that assuming less model uncertainty will be fair. Thus a Cov for X_{Eh} of 10% is used. The truncation of X_{Eh} is introduced because it is assumed that any value beyond the truncation limit is captured during the course of the design process. The alternative distribution model for the drag load is assumed Gumbel distributed (the Gumbel distribution results from modelling the sea-elevation process as a step-wise narrow-band stationary Gaussian process). The proposed CoVs are taken from the low and high end of values that can be found in the literature, which exhibits a range from 15% to 35% [11]. The alternative model uncertainty is chosen as a log-normal distribution, which is in accordance with

common practice, and with CoVs from the low end to the centre of proposed values.

The uncertainty model for the aerodynamic load effect has been adopted from [6] (of which a short presentation is given in [7]). The reader is referred to these references on the details of the model. The only difference between the model applied here and the model in [6] is that a simplification has been introduced in order to have similarity in the degree of detailing with the uncertainty model for the hydrodynamic loads. Thus $\tilde{P}$ accounts both for the physical variation of the extreme wind climate, for statistical uncertainties associated with the estimation of its distribution and for model uncertainties.

The materials uncertainty model also rests on [8]. Except from the biases the model is very similar to that usually applied in calibration studies. The bias of $\tilde{F}_{\mathrm{y}}$ is non-conservative corresponding to an assumption that the ISO practice is followed. The model uncertainty X_m assumes the ISO recipe for bending capacities of tubular members is used. The general validity of these biases is debatable, though valid for support structures designed according to ISO. The details of this part of the entire uncertainty model are however less important for the conclusions of this paper, since the same materials uncertainty model is employed for both wind and wave loads. Thus the absolute values of reliability levels computed herein may be discussed but the load effect safety factors are not very sensitive to changes of the material uncertainty model.

Though the reliabilities evaluated as described here cannot be given a strict frequentistic interpretation they can however still be compared. The validity of such a comparison rests on the assumption that the underlying uncertainty models for the wind and wave load effects are comparable – i.e. equally good (or bad). Due to the similarity in detail of the models presented above it is judged that the reliability indices obtained in the present work for wind and wave loads separately should be considered equal up to differences in β of about 0.25.

1.2 THE COMBINED LOAD EFFECT

This section discusses how the result presented above may be used to establish a conservative design rule. An important difference in the definition of the characteristic load effects is that the wind load effects are determined with a 10-min reference period, whereas the wave load effects are determined with a 3-hr reference period. This makes sense because wave storms are typically more persistent than wind storms but how should one combine the extreme responses? In a given time period the extreme wave load effect does not occur at the same time as the extreme wind load effect. Further the extreme 10-min wind and 3-hr wave load effects are peak values in a storm. Does the 3-hr peak value occur before, or after the 10-min peak, or simultaneously?

The approach in [4] is based on some simplifying assumptions about the nature of storm climates. Depending on site the wind and wave storm peaks will occur out of phase, i.e. the peak of the wind climate will typically appear earlier in time than the peak of the wave climate. Further the peak values may be more or less correlated. In [4] any time delay was neglected and the peak values were assumed fully correlated. This leads to a conservative estimate of the combined load because it effectively maximises wind and wave climate intensities and the respective load effects separately and subsequently join them. Using such a conservative approach it was demonstrated that the structural reliability against combined loads is higher than against the individual loads.

Typically one will need both to make quasi-static evaluation of response in order to properly account for non-linear wave kinematics of extreme waves and to perform pseudo random response simulations in order to examine possible dynamic amplification of the wave loads. Both these cases are addressed in [4], which for the deterministic case propose a moderate wave height to be combined with extreme wind speed, and vice versa. For the response simulation it is proposed to transform the 10-min and 3-hr environmental reference period parameters into parameters that, when applied in a 1-hr simulation, will reproduce the 10-min and 3-hr extreme load effects to wind and wave loads, respectively. Dynamic response simulations with simultaneous wind and wave loads using the transformed values are then performed.

Another problem about the difference in reference periods is that measured or hindcast metocean data are often available as 1-hr data. This means that for the wind transformation from 1-hr data to 10-min data is needed. After this transformation the above mentioned transformation to 1-hr values for simulation purposes may be relevant. It would be convenient if the measured/hindcast 1-hr data could be directly applied in the simulation. For the significant wave heights there is no dependence on reference period. The reference period merely states the duration of stationary sea states. Therefore there is no need to transform between significant wave heights except if simulation periods shorter than the reference period (3 hr) are applied the significant wave height shall be inflated, i.e. up-scaled, as explained in the previous paragraph. Investigations carried to support Working Group WG3 under the IEC (International Electrotechnical Committee) Technical Committee TC88 [12, 13] suggest that for the wind loads it is slightly conservative to use the measured/hindcast 1-hr data directly for simulation.

2 SITE-SPECIFIC LOAD COMBINATION: IFORM

The strength of the conservative combination rule described in Sec. 1.2 is that it requires only the individual 50-yr wind and wave parameters. Often this is all the information one has and then one cannot expect to do better. If one has more detailed information the question is how to use it. As explained in the previous section there may be time delays between peak values and peak values may be only partially correlated. So, one could combine the peak value of wind speed with a companion value of the significant wave height, which would be less than the peak value – and vice versa. It might even be that two non-peak simultaneous values of wind and wave parameters generate more extreme load effects than the peak+companion combinations do. This depends on the specific load effect one considers. For instance wind loads typically govern overturning moment whereas base shear is highly influenced by the wave loads. Thus for the overturning moment peak wind and companion wave parameters may be the most critical while combinations of non-peak values might be most critical for the base shear. In fact these considerations apply to any cross-section of the support structure. Therefore the proper combination of wind and wave loads depend on actual design. The influence of design could be neglected in [4] because of the assumption of full correlation and no time delay makes this issue irrelevant.

So the situation is that a methodology that accounts for: 1) geometry, and 2) peak parameter correlation plus time delay is needed. The peak parameter correlation and time delay may be described in terms of the joint probability distribution function (jpdf) of the mean wind speed and the significant wave height. It should either be the jpdf of all values of V and H_s – not just the peak values, or it should be a more cleverly devised jpdf of peak values and V and H_s values conditional on the peaks. We return to this in Sec. 3. An appealing idea would be to search along appropriately defined contours of the jpdf for the parameter combinations that make a specific load effect for a specific design become the most extreme. That is, one searches along a contour of – in a certain sense – combined 50-yr return period values. Such a method has been developed in [14] and is known as the Inverse First Order Reliability Method (IFORM). This approach has the advantage that one may establish the contour, known as the environmental contour, once and for all for a given site. For any design one can then determine the proper load combinations without reconsidering the climate. In this section the procedure to establish the so-called environmental contour is briefly presented. For further details refer to [14].

Before going into details it should be said that the methodology is not limited to only two climate parameters. Current and wind-wave misalignment may be included too. The latter may be important with regard to damping of transverse oscillations. The application of the method requires that a joint distribution model is available. This requires a sufficient amount of data, which is the main drawback of the method. However for large offshore wind farms it is feasible to perform hindcast simulations. From these one will have sufficient data to fit a joint distribution model. For reasonably highly correlated environmental parameters a multivariate distribution model may even be established solely from the marginal distributions and covariances by use of the so-called Nataf's distribution, see the appendix. This distribution will be considered in the following paragraph that describes the procedure.

Assume one has access to a joint distribution model for the mean wind speed V and the significant wave height H_s. To construct the environmental contour a probability transformation from two uncorrelated standard normally distributed variables, U_1 and U_2, to the jointly distributed pair (V, H_s) is required:

$$(V, H_s) = \varphi(U_1, U_2) \tag{13}$$

Applying the Nataf's distribution the transformation may be reduced to (see the appendix):

$$\begin{aligned} V &= \varphi_1(u_1) = F_V^{-1}[\Phi(u_1)] \\ H_s &= \varphi_2(u_1, u_2) = F_{H_s}^{-1}[\Phi(\rho u_1 + \sqrt{1-\rho^2}\, u_2)] \end{aligned} \tag{14}$$

where $F_V(V)$ is the marginal CDF of the mean wind speed, $F_{Hs}(H_s)$ is the marginal distribution of the significant wave height, and $\rho = \rho(r)$ is a function of the correlation r between V and H_s. By use of the transformation in Eq. (14), the environmental contour is now obtained by the following procedure. A circle of radius β in the U_1-U_2 plane, i.e. points that satisfy the equation $U_1^2+U_2^2=\beta^2$, is transformed into a curve in the V-H_s plane, which is then the environmental contour. The radius β is defined by

$$\Phi(\beta) = 1 - \frac{1}{N} \tag{15}$$

where N is the number of independent observation time periods in, say, 50 years. If $F_V(V)$ and $F_{Hs}(H_s)$ are the distributions of the annual extremes then simply N = 50 leading to $\beta \approx 2.05$. If $F_V(V)$ and $F_{Hs}(H_s)$ are the distributions of the storm peaks, of which there may be more than one per yr, then N and β are larger. Generally one does not need to determine the entire environmental contour. Only the part of the environmental contour in the neighbourhood of the 50-yr return period mean wind speed V_{50} is needed. The entire contour may be determined by Eq. (16).

$$V = F_V^{-1}[\Phi(u_1)]$$
$$H_s = F_{H_s}^{-1}[\Phi(\rho u_1 \pm \sqrt{1-\rho^2}\sqrt{\beta^2 - u_1^2})] \quad (16)$$

In Eq. (16) $-\beta \le u_1 \le \beta$. The part near V_{50} is obtained for u_1 close to β.

In this presentation of IFORM the jpdf of V and H_s have been considered. This will be the typical situation. However, on could have considered variables derived from V and H_s as well, for instance P and $H_{\max}^2$ as defined in the previous section. In the remainder of this paper these derived variables are considered because this allows the direct application of the distribution models from Table 1.

3 APPLICATION OF IFORM

IFORM described in previous section gives a general tool to the determination of the combined environmental conditions. As mentioned one uses in the general case the jpdf of V and H_s. Because IFORM in this case requires much data for the assessment of the jpdf alternatives are desirable. The aim here is to provide a simple load combination rule based on a few characteristics of the wind and wave storms at a given site. Using IFORM such a combination rule is derived assuming that one has just data enough to asses the marginal distributions of the storm peak values of mean wind speed and significant wave height, their correlation and the possible delay of the arrival of the wave storm relative to the wind storm. For this purpose the Nataf's distribution is well suited as long as the correlation is not too small. Otherwise one must apply IFORM in its general form, unless the necessary combination rules are so obvious from data that one needs not search among a selection of candidates. So, this section demonstrates the application of IFORM in conjunction with the Nataf's distribution, as derived in the previous section, based on the stochastic model presented in Sec. 1.1.2. That is, IFORM is applied to the jpdf of the peak values rather than the jpdf of all values V and H_s.

First of all, a limit-state that models the combined response is required. It is assumed, as a reasonable first approximation, that the maximum load effect generated by the combined loading in a given 10-min period is

$$E_{\max} = \mu_{E_{\text{aero}}} + \sqrt{(E_{\max,\text{aero}} - \mu_{E_{\text{aero}}})^2 + E_{\max,\text{hydro}}^2} \quad (17)$$

where $\mu_{E_{\text{aero}}}$ and $E_{\max,\text{aero}}$ denote the 10-min mean and max, respectively, of the aerodynamic load effect under given conditions, whilst $E_{\max,\text{hydro}}^2$ is 3-hr max hydrodynamic load. Thus it is conservatively assumed that the max wave load in the 3-hr sea-state that surrounds the considered 10-min period occurs in these 10 min. This conservatism is accepted as this allows the use of well-established offshore engineering practices for wave load calculations. The assumptions behind Eq. (17) are:

1. Limited dynamical interaction of wind and wave loads, e.g. stiff foundation is assumed or damping is neglected
2. Approximately the same zero-crossing frequency of wind and wave load response
3. Not too different distribution of response to wind and waves

Of these assumptions the first is believed to be the most severe, but for this application, where there are many other inaccuracies it is believed to be sufficiently accurate to apply Eq. (17).

Eq. (17) may be rewritten into the following response model to be used in the IFORM:

$$E_{\max} = c_{\text{inf,aero}} P_c \left[\tilde{P} + \sqrt{\tilde{P}^2 + (\varepsilon \tilde{H}_{\max}^2)^2} \right] \quad (18)$$

Note that the constant $c_{\text{inf,aero}} P_c$ need not be known for the search of maximum combined load effect on the environmental contour. The ratio ε between aerodynamic and hydrodynamic load effects generated by the individual characteristic values has been introduced:

$$\varepsilon = \frac{c_{\text{inf,hydro}} c_{\text{dyn,hydro}}}{c_{\text{inf,aero}}} \frac{H_{\max,c}^2}{P_c} \quad (19)$$

this ratio will be used as governing parameter in the following parameter studies, thus, roughly, a low value corresponds to overturning moment, whereas a medium value correspond to base shear. It is emphasized that $H_{\max,c}^2$ and P_c are the individual 50-yr return period values, not the combined values that result from the application of the IFORM procedure. These combined values are denoted $H_{\max,c,\text{comp}}^2$ and $P_{c,\text{comp}}$. Thus ε is measure of the ratio of the load effects in the conservative combination rule described in Sec. 1.2. The normalisation with the individual characteristic values is convenient as the marginal distributions of the annual extreme values normalised with their characteristic values are known, see Table 1.

3.1 NO TIME DELAY BETWEEN STORM PEAKS

As a first study, the case where there is no time delay between peaks is treated. After some manipulations similar to those in Sec. 1.1.1 one finds that:

$$\Phi(\beta) = \Pr\left\{ \tilde{F}_y X_m \gamma_m \gamma_f \ge \frac{c_{\text{storm}}}{\eta_{\text{a}} + \sqrt{\eta_{\text{a}}^2 + (\varepsilon \eta_{\text{h}})^2}} \times \left[\tilde{P} X_{Ea} + \sqrt{(\tilde{P} X_{Ea} X_{\text{dyn}} T)^2 + (\varepsilon \frac{\tilde{H}_{\max}^2}{X_{Eh}})^2} \right] \right\} \quad (20)$$

where the ratios of the combined characteristic values to the individual characteristic values have been introduced:

$$\eta_{\mathrm{a}} = \frac{P_{c,\mathrm{comp}}}{P_c}, \quad \eta_{\mathrm{h}} = \frac{H^2_{\max,c,\mathrm{comp}}}{H^2_{\max,c}} \tag{21}$$

In Sec. 1.1.1 the storm profile correction factor c_{storm} was introduce to compensate for the fact that storms have longer duration than the reference period used for the load effect computation. The correction factor was different for wind and wave loads. Here those individual factors have been replaced by a common factor, which will be put equal 1.07. This has only limited effect on the results.

Figure 1 and Figure 2 show the contours of the peak values. In Figure 1 P and $H_{\max}^2$ are Gumbel and shifted exponentially distributed, respectively. In Figure 2 P and $H_{\max}^2$ are both Gumbel distributed. Thus the figures correspond to the two alternatives proposed in Table 1. Five values of correlation are included: $r = 0.60, 0.75, 0.85, 0.90, 0.95$. A correlation of 0.60 is a low and it may be debatable if the Nataf's model is applicable in that case, however that case is included for the purpose of illustration. The highest correlation gives the narrowest environmental contour. Three ratios of ε are regarded: 0,1, and ∞. The corresponding combined characteristic values are plotted with circles, encircled crosses, and crosses, respectively. The precise numbers are listed in Table 2 and Table 3 where they have been expressed in terms of percentiles of the marginal distributions of the peak P and $H_{\max}^2$. to be able to appreciate the figures in those tables it is noted that the implication of the return periods in terms of reduced loads is that a return period of 15, 25, 35, and 45 yrs correspond roughly to a 20%, 10%, 5% and 2% reduction of loads, respectively. This holds for both wind and wave loads.

It is seen that the assumption of no time delay implies the one must reach correlation 0.85 before a

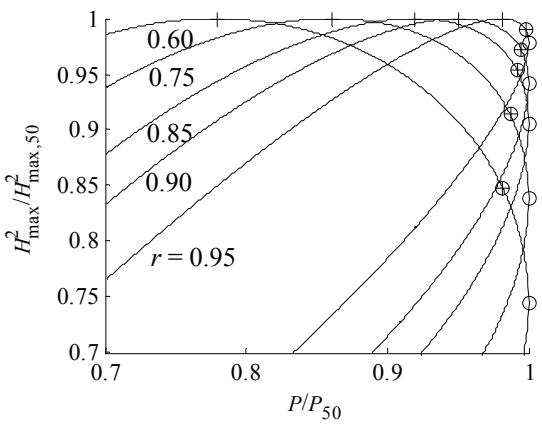

Figure 1: environmental contour for P and $H_{\max}^2$ Gumbel and shifted exponentially distributed, respectively.

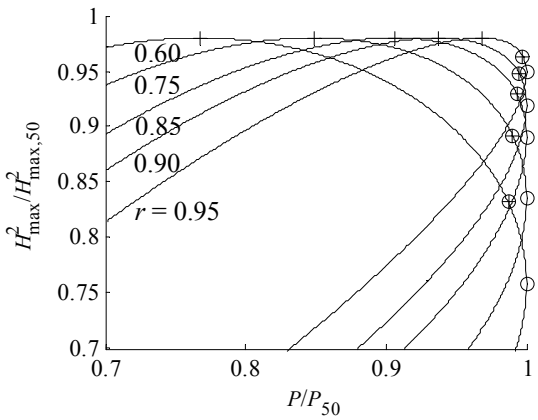

Figure 2: like Figure 1, except that P and $H_{\max}^2$ are both Gumbel distributed.

significant drop in combined characteristic values relative to the individual characteristic values is obtained. As a rough average of Table 2 and Table 3 one may propose the numbers listed in Table 4.

Table 4 gives a simple relation between correlation and reduction of characteristic value for two end point values and a central value of ε. For other values one may do as follows: For any value of ε between 0 and ½ the return periods may be as-

r	$\varepsilon = 0$		$\varepsilon = 1$		$\varepsilon = \infty$	
	P	$H_{\max}^2$	P	$H_{\max}^2$	P	$H_{\max}^2$
0.60	50	10	44	20	10	50
0.75	50	19	46	29	19	50
0.85	50	28	47	38	28	50
0.90	50	35	48	42	35	50
0.95	50	44	49	47	44	50

Table 2: characteristic values corresponding to the plot in Figure 1 expressed in terms of return period in years of the marginal distributions.

r	$\varepsilon = 0$		$\varepsilon = 1$		$\varepsilon = \infty$	
	P	$H_{\max}^2$	P	$H_{\max}^2$	P	$H_{\max}^2$
0.60	50	10	46	17	10	50
0.75	50	17	47	26	17	50
0.85	50	26	48	34	26	50
0.90	50	32	48	39	32	50
0.95	50	40	49	44	40	50

Table 3: like, but for Figure 2.

r	$\varepsilon = 0$		$\varepsilon = 1$		$\varepsilon = \infty$	
	P	$H_{\max}^2$	P	$H_{\max}^2$	P	$H_{\max}^2$
0.60	50	10	45	20	10	50
0.75	50	18	46	25	18	50
0.85	50	26	50	35	26	50
0.90	50	34	50	40	34	50
0.95	50	42	50	45	42	50
1.00	50	50	50	50	50	50

Table 4: simple relation between correlation and reduction of return period.

sumed constant and equal the return periods for $\varepsilon = 0$. Likewise for any value of ε between 10 and ∞ the return periods may be assumed constant and equal the return periods for $\varepsilon = \infty$. For values of ε between ½ and 10 logarithmic interpolation with respect to ε gives an acceptable approximation. Though this is a simple relation it cannot be used without proper substantiation of the choice of correlation coefficient. Especially for lower correlations, expected to be characteristic of distinct site-specific conditions, one must be careful. Having determined the combined return periods the response to the reduced characteristic 10-min values and 3-hr values may be obtained as explained in Sec. 1.2.

To close this presentation an investigation of whether or not the load combination proposed ensures an acceptable level of reliability. For this purpose Eq. (20) is used, which can be seen to reproduce Eqs. (11) and (12) for pure aerodynamic loading, $\varepsilon = 0$, and hydrodynamic loading, $\varepsilon = \infty$,

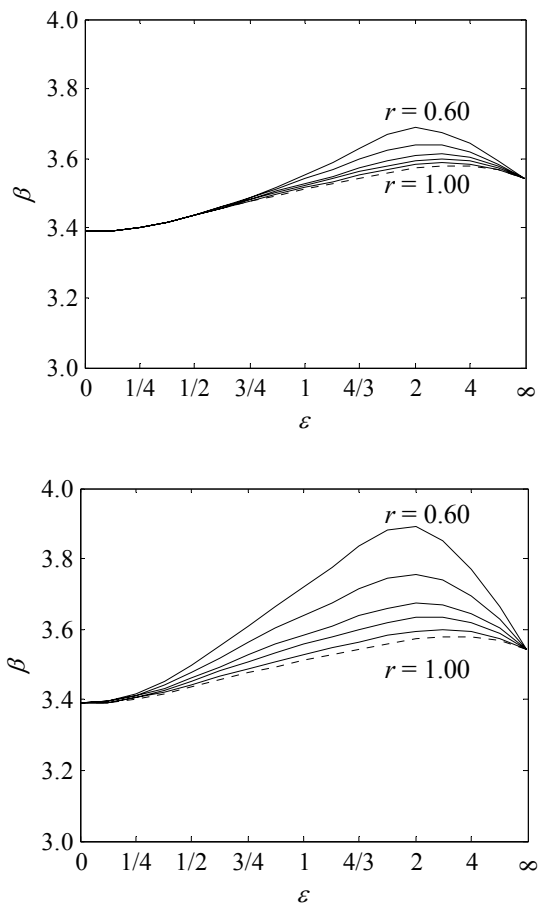

Figure 3 Top plot: the dependency of β against ε. When $\varepsilon = 0$ there is now wave loads, and when $\varepsilon = \infty$ there is no aerodynamic load. In the present plot P and H_{max}^2 are Gumbel and shifted Exponentially distributed, respectively, and r ranges from 0.60 to 1.00. Bottom plot: the same situations as in the top plot, except peak values are assumed fully correlated as in [4]. This is obtained by setting $\eta_a = \eta_h = 1$ in Eq. (20).

respectively. The top plot in Figure 3 shows the reliability index as function of ε for $\gamma_f = 1.35$ in the case P and H_{max}^2 are Gumbel and shifted Exponentially distributed, respectively. The endpoint values at $\varepsilon = 0$ and $\varepsilon = \infty$ correspond to the results obtained in [4]. Between the endpoints the reliability is higher than for the endpoints, which shows that sufficient reliability is achieved. Similar plots are obtained for the wave load being Gumbel distributed. The bottom plot in Figure 3 shows the increase in reliability obtained by assuming the storm peak values of P and H_{max}^2 fully correlated. As an example it is mentioned that an increase of β from 3.5 to 3.6 corresponds to an increase of notional reliability of order 1.5.

3.2 TIME DELAY BETWEEN STORM PEAKS

This section presents the results of some amount of work that has been made to investigate the influence of time delay between peaks of the wind and wave storm. To pursue this two generic storm profiles for wind and waves have been assumed. These are shown in Figure 4. The wind storm profile has been determined from an analysis of the Sprogø data, and the wave storm profile comes from the NORSOK standards, see [4] for further deltails.

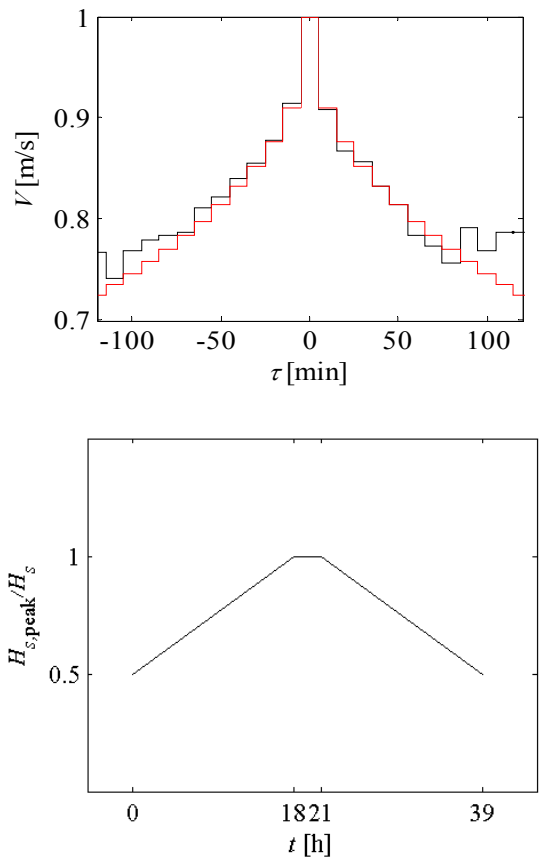

Figure 4: generic storm profiles for wind (top) and waves (bottom).

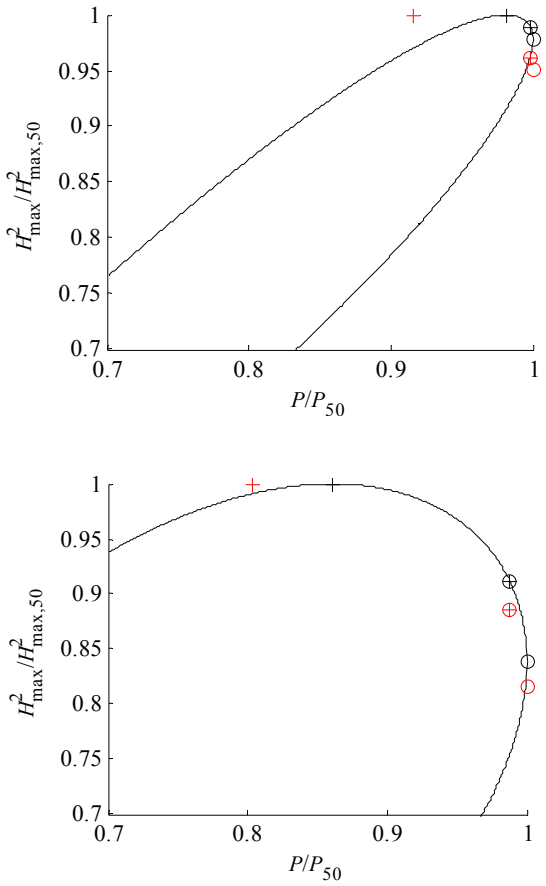

Figure 5 environmental contour for P and H_{max}^2 Gumbel and shifted exponentially distributed, respectively. The time delay is 2 hr and the correlation is 0.95 in the top plot and 0.75 in the bottom.

Figure 5 shows the outcome of two computations with parameters as stated in the caption. The cross, the encircled cross, and the circle have the same meaning as before. The points on the contour are the characteristic values of the peak values. The points outside the contour result from the reduced value on the storm profile, which is shifted 2 hr relative to the other. These plots suggest that the impact of delay on reduction loads is independent of correlation and that the impact can be relatively significant for large correlations, why the effect of delay should not be neglected.

4 CONCLUSIONS

Combining existing design rules for offshore engineering and wind turbine engineering to site-specific conditions is highly relevant. Unlike to classification approach used for onshore turbines (e.g. in the IEC 614100-1 standard) the seabed implies that offshore conditions, i.e. wave loading and soil, are highly site-specific. The so-called IFORM, which is an advanced method, has been presented in order to demonstrate how site-specific characteristic values can be determined. Unless one accepts to be rather conservative site-specific design loads cannot be established without an extra effort in terms of acquiring more data and demonstrating their reliability.

However a simple approach for sites with relatively high correlation between mean wind speeds and significant wave heights, that is sites with only mildly site-specific conditions is proposed. This approach assumes that one has just enough data to asses: the marginal distributions of the storm peak values of mean wind speed and significant wave height, their correlation, and the possible delay of the arrival of the wave storm relative to the wind storm. Low correlations have been considered and this demonstrates that low correlation is required to reach considerable gain in terms of reduced loads. Especially for lower correlations the Nataf's distribution used here requires validation before use, though not as much as a more complete model for the joint probability distribution may do. Finally it is noted that for the wave loads it has been assumed that they are drag dominated, and that the site is a not shallow North Sea site. Not only has a procedure to establish combined characteristic values for environmental parameters been presented, it has also been demonstrated that if these are used together with 10-min max response to the wind in combination with the 3-hr. max response to the waves and a safety factor of 1.35 an acceptable level of reliability is obtained. As the influence of time delay has only been lightly touched upon a natural extension of this work will be to investigate the influence of time delay in more detail. There are indications that the delay is more important than the correlation.

5 ACKNOWLEDGEMENTS

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APPENDIX: ON THE NATAF DISTRIBUTION MODEL

The Nataf distribution model is a simple way to establish a joint distribution model for a number of correlated variables from a limited amount of information. Say one regards the variables $X_1, \ldots, X_n$. The Nataf model requires marginal distribution models for all X_i and a their mutual covariances. Generally this is not sufficient information to fully define the joint distribution of $X_1, \ldots, X_n$. Nevertheless the so-called Nataf's distribution provides a joint distribution model from this information alone. Considering that this information is all that one may have in practice the Nataf model may be sufficiently accurate. The Nataf is described in brief in [15] and in detail in [16]. Here the basic formulas are repeated for easy reference.

Since only the covariances are known it is reasonable to derive, by use of a suitable transformation, a distribution model from a zero-mean multivariate normal distribution, which is by nature completely defined by its covariances. This is in fact how the Nataf's distribution is defined. Denoting the marginal cdf of X_i by F_i, and letting Y_i denote the i'th normal variable the following transformations are applied:

$$\Phi(y_i) = F_i(x_i) \tag{1}$$

where Φ denotes the standard normal cdf By this transformation it is ensured that $X_1, \ldots, X_n$ will have the given marginal distributions. The next step is to ensure that the given covariances are reproduced. This is done by requiring that the correlation matrix of $Y_1, \ldots, Y_n$ through the transformation in Eq. (1) is converted into the correlation matrix of $X_1, \ldots, X_n$. This requirement gives n^2 equations, which may not always have a solution and which in most cases will have to be solved numerically. For a number of selected distributions this has been done in [16], which provides the solutions in terms of approximating polynomials to the ratio of the correlation r_{ij} between X_i and X_j to the correlation ρ_{ij} between Y_i and Y_j. These approximating polynomials are expanded in powers of r_{ij} and the COV, V_i, of X_i, and cross-terms hereof. In this paper only the case of two correlated variables is consider, namely one for wind and one for wave loads. Further a Gumbel distribution is used for the wind loads and a Gumbel or a shifted Exponential distribution is used for the wave loads. The Nataf's distribution for the Gumbel-Gumbel model and for the Gumbel-Exponential model are treated in [16] and repeated here; for other distribution models refer to [16]. Since only two variables are considered here r_{ij} and ρ_{ij} are replaced by r and ρ, respectively. For the Gumbel-Gumbel case one has the approximation

$$\frac{\rho}{r} \approx 1.064 - 0.069\,r + 0.005\,r^2 \qquad r \in [-0.886, 1.000] \tag{2}$$

and for the shifted-Exponential-Gumbel case one has

$$\frac{\rho}{r} \approx 1.142 - 0.154\,r + 0.031\,r^2 \qquad r \in [-0.780, 0.981] \tag{3}$$

where the maximum error in Eq. (2) is 0.0% whereas in Eq. (3) it is 0.2%, which for any practical purpose is equivalent to zero. In both cases the CoVs are in the range from 0.1 to 0.5. It is noted that there are limitations to the value of r. For r outside the specified intervals the Nataf's distribution is not defined. In the present case only positive correlations are of interest. It is seen that the limitations are not of practical relevance because a correlation of 0.981, or higher, for the practical purposes considered here is equivalent to full correlation.

The Nataf's distribution is obtained by transformation of correlated standardised Gaussian variables, but for the use in IFORM one needs a transform from uncorrelated standardised Gaussian variables. This is easily obtained. If U_1 and U_2 denote such uncorrelated variables then one may apply

$$\begin{aligned} Y_1 &= U_1 \\ Y_2 &= \rho U_1 + \sqrt{1-\rho^2}\,U_2 \end{aligned} \tag{4}$$

Combining Eqs. (1) and (4) finally gives

$$\begin{aligned} x_1 &= \varphi_1(u_1, u_2) = F_1^{-1}[\Phi(u_1)] \\ x_2 &= \varphi_2(u_1, u_2) = F_2^{-1}[\Phi(\rho u_1 + \sqrt{1-\rho^2}\,u_2)] \end{aligned} \tag{5}$$

which is the formulae that are needed. The correlation $\rho = \rho(r)$ is obtained from Eqs. (2) and (3) or similar from [16]. The inverse transform of (5) is

$$\begin{aligned} u_1 &= \Phi^{-1}[F_1(x_1)] \\ u_2 &= \frac{\Phi^{-1}[F_2(x_2)] - \rho\Phi^{-1}[F_1(x_1)]}{\sqrt{1-\rho^2}} \end{aligned} \tag{6}$$