Design of offshore structures: impact of the possible existence of freak waves

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Abstract. The importance of the possible existence of so-called freak waves regarding structural design is discussed. At first the framework for design of offshore structures on the Norwegian continental shelf is presented for the purpose of indicating the annual exceedance probabilities of concern regarding design. The adequacy of the second order random model for the sea surface process is briefly indicated. It is suggested that freak waves should be defined as wave events from a population not captured by the second order model. Evidences of the existence of freak waves are discussed and possible originating mechanisms for their existence are commented upon. Finally, a probabilistic framework for accounting for freak waves in design is presented. The approach is illustrated by assessing the total wave induced load on a generic jacket structure.

Introduction

Offshore structures are designed to withstand all foreseeable weather conditions. Foreseeable is in this connection defined as environmental events corresponding to an annual exceedance probability of q, where q could be of the order of 10^{-4}. In addition to environmental loading, including earthquake-induced loads, the structures are also to be designed against all accidental loads, e.g., collisions, fires and explosions, corresponding to annual exceedance probabilities of 10^{-4} or larger. In the following we will focus on wave-induced loads.

In order to calculate loads on structures due to waves, one has to introduce an adequate wave theory. The choice of a proper theory may depend on the problem under consideration. For a number of structural classes or structural problems, the loading on the structure will essentially be linearly related to the wave process. For such a problem, the sea surface process may be modeled as a piecewise stationary and homogeneous Gaussian random field. Furthermore, linear wave theory can be adopted for the purpose of calculating the corresponding wave kinematics.

For other structural problems, however, the governing wave-induced loading may be caused by the drag term of the Morison equation (*Sarpkaya and Isaacson,* 1981). This term is proportional to $u(t)|u(t)|$, where $u(t)$ is the horizontal water particle speed. The particle speed attains its maximum value under the wave crest. Accordingly, for this type of problem, the load has to be integrated to the exact sea surface, i.e., a wave theory reflecting the wave crest heights and wave steepness with a reasonable accuracy is required. For problems like slamming loads on platform columns and ship hulls, green water on ships, and wave-deck impact problems for fixed platforms, one will mainly be concerned about the height and steepness of the most extreme crest heights, wave heights, or possible group of extreme waves.

For problems being sensitive to the surface process, a deterministic theory is adequate if the response under consideration is of a quasi-static nature, i.e., there is a more or less one-to-one correspondence between crest height and the target response quantity. The 5th order Stokes theory is the most frequently adopted model for such cases. If dynamics are of concern, or the problem for other reasons requires that the temporal and spatial variability are accounted for, a second order randomized Stokes expansion is the most advanced model available for numerical routine engineering design work.

The aim of the present paper is to describe how waves are modeled for the purpose of estimating design loads on marine structures. For that purpose, the rule framework for design of offshore structures will first of all be briefly introduced. Thereafter we will present the most advanced wave modeling available for routine design. Based on this background, the possible existence of freak waves will be discussed. This discussion will be closed by defining freak waves in a way which will be convenient in order to discuss whether or not we in the future should account for this sort of waves in the design process. The last part of the paper will be devoted to a discussion of a probabilistic framework which may represent a possible way to account for freak waves in structural design.

Norwegian rule regime for design of offshore structures

Offshore structures at the Norwegian Continental Shelf are, with respect to overload, required to be checked against two limit states, the ultimate load limit state (ULS) and the accidental load limit state (ALS) (*Norsok*, 1999). For the illustrative purpose of this discussion, we may restrict the consideration to the environmental loads, i.e., permanent loads and operational loads are not included. Then the equation to be checked for both limit states is of the form

$$\gamma_f \, s_c < \frac{r_c}{\gamma_m} \tag{1}$$

s_c is the characteristic load in a structural member, r_c is the characteristic capacity of that member, γ_f is a safety factor accounting for uncertainties in the estimated characteristic load, and γ_m is a factor accounting for uncertainties in the estimated characteristic capacity.

The basic design limit state is the ULS control. For that control, the characteristic load is determined as the member load corresponding to an annual exceedance probability of 10^{-2}. The characteristic capacity is most often taken as a lower percentile, say 5%, of the distribution function representing the epistemic uncertainty of the elastic capacity of the member. The partial safety factors, γ_f and γ_m, are defined by the rules (*Norsok.* 2004). For the ULS control, the factors are in most cases 1.3 and 1.15, respectively. That means that the design load, $\gamma_f \, s_c$ is 30% larger than the load corresponding to an annual exceedance probability of 10^{-2}. For a linear response problem, the design load may therefore correspond to an annual exceedance probability in the order of 10^{-5}, while for a drag-dominated problem the corresponding probability may be between 10^{-4} and 10^{-3}.

Provided that the load versus exceedance probability is of a well-behaved nature, the structure designed in agreement with the ULS control is tacitly assumed to result in a reasonable safety against collapse, i.e., the nominal failure probability is in the order of 10^{-5} or lower.

However, this may not be the case if for some reason the load–exceedance probability relation changes abruptly in a worsening direction for exceedance probabilities between 10^{-2} and 10^{-4}. Such an abrupt change in load pattern could take place if the most extreme waves impact the deck of the structure. The load–exceedance probability relations for well-behaving and for bad-behaving response problems are shown in Figure 1. It is seen that in the case of a bad-behaving problem, $\gamma_f = 1.3$ may not produce a design load corresponding to a sufficiently low annual exceedance probability

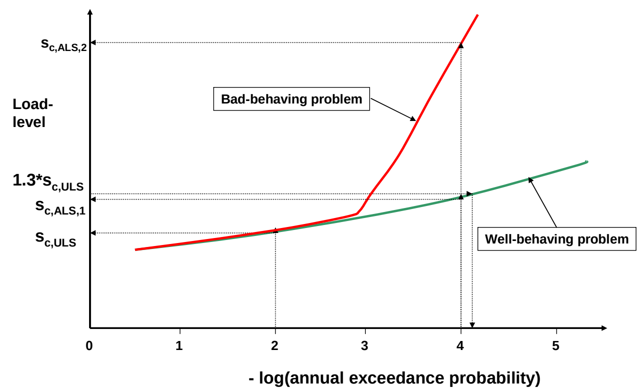


Figure 1. Illustration of well-behaving and bad-behaving response problems.

In order to ensure that a bad-behaving response problem is not slipping unnoticed through the design process, Norwegian Offshore Regulations (*Norsok*, 2004) also require that the structures shall withstand environmental loads corresponding to an annual exceedance probability of 10^{-4} with at most some local damage.

The introduction of the ALS control for environmental loads is not common beyond the Norwegian Continental Shelf. Within traditional shipping, wave-nduced loads of such a low annual exceedance probability will usually not be considered in the design process. The international offshore industry is also somewhat reluctant against introducing the ALS control for environmental loads. However, there are good reasons for considering this load check as an important step for improving the safety of structures against environmental loading.

For a well-behaving response problem, the ALS control will not have any effect on the design. However, properly implemented in the design process, it ensures that the designers do their best in order to identify the environmental loads occurring with an annual probability close to the target failure probability, i.e., there is a good chance for identifying bad-behaving problems at an early stage. The reluctance is often explained by claiming that predicted environmental loads corresponding to such low annual exceedance probabilities will be associated with very large uncertainties. Of course rather large uncertainties are associated with the prediction of such rare events, but these uncertainties will not disappear by skipping the ALS control for the environmental loads.

A storm or an individual wave corresponding to an annual exceedance probability of 10^{-2} should not represent any problem for an offshore structure. For such a weather condition, the loading should in principle be well within the elastic capacity of the structure. For a storm severity or an individual wave severity corresponding to annual exceedance probabilities lower than 10^{-4}, we approach weather conditions that could represent a threat to the structural integrity. The fact that we have experienced damage to structures for weather conditions less severe than the 10^{-2}-probability[1] weather, suggests that focus should be increased on making the structures more robust regarding the most severe foreseeable environmental loads, i.e., loads corresponding to an annual exceedance probability in the order of 10^{-4}.

It should be remembered, that safety against environmental loads is merely a small part of the overall structural safety aspect. On a day-to-day basis, more traditional accidental load cases, i.e., loads caused by collisions, fires, and explosions are most probably of greater concern regarding structural safety. However, an adequate safety against environmental loads is very important since this safety level defines the base line safety of the structure.

Wave description for design purposes

For a general structural response problem, there is not a perfect one-to-one relation between, say, wave crest height and platform loading. However, in most cases there is a rather large positive correlation. In particular, in connection with an on-off mechanism, there will often be a strong positive correlation between the response and environmental parameter turning on the on-off phenomenon. This means that if we shall be able to predict reasonable estimates for the *q*-probability loads, $q = 10^{-2}$ and 10^{-4}, the wave models used for design should accurately reflect wave events with annual occurrence probabilities of these orders of magnitudes.

Regarding the description of wave conditions for design purposes, the modeling can be separated in two more or less separate problems: i) The long term modeling of sea state conditions, and ii) The short term description of the surface process given the sea state conditions. These two sources of randomness are conveniently combined by a convolution type integral as shown below:

$$F_{C_{3h}}(c) = \int_h \int_t F_{C_{3h}|H_sT_p}(c|h,t)\ f_{H_sT_p}(h,t)\ dt\,dh \qquad (2)$$

where $F_{C_{3h}|H_sT_p}(c|h,t)$ is the conditional distribution for the 3-hour maximum crest height given the sea state characteristics, and $f_{H_sT_p}(h,t)$ represent the long term description of the wave characteristics. A proper long-term description can be obtained by various approaches, but for any matter of approach a consistent estimation process requires that both the short-term and the long-term variability are properly accounted for.

The sea state modeling can either be formulated as modeling all possible short term sea states, see, e.g., *Haver and Nyhus* (1986) or by modeling merely storm sea states, i.e. a peak over threshold formulation, see e.g., *Tromans and Vanderschuren* (1995). For any matter of long-term modeling, one needs the short-term distribution of crest heights. Crest heights are preferred instead of wave heights since crests are more sensitive to a deviation from the Gaussian hypothesis. Furthermore, a broad range of response problems are more strongly correlated to crest height than the wave height.

For design purposes, the most advanced approach that is available for routine purposes is a second order random model, see e.g., *Marthinsen and Winterstein* (1992). An example of a simulated second order process is shown in Figure 2. The figure shows a 400-second window of a realization of a sea state with the significant wave height, H_s, equal to 18.2 m and the spectral peak period, T_p, equal to 17 s. This sea state is a sea state located on the 10^{-4}–probability contour line in the northern North Sea (see Figure 11). The second order correction to the Gaussian process involves both a difference frequency term and a sum frequency term, but in the figure merely the sum frequency correction is shown.

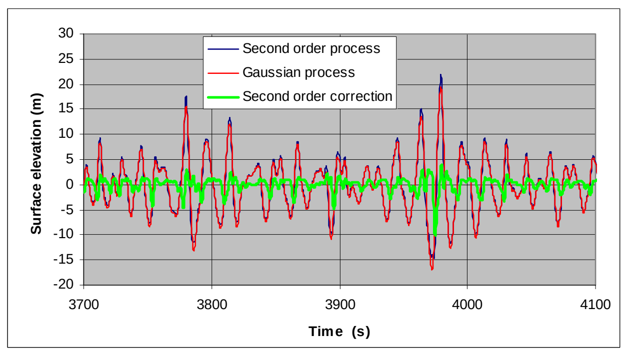


Figure 2. Realizations of second order component processes for h=18.2 m and t=17 s.

Based on a large number of simulated realizations of a second order process, a parameterized probabilistic model for the crest heights is proposed by *Forristall* (2000). The model is a 2-parameter Weibull model and is given by:

$$F_{C|H_sT_p}(c|h,t) = 1 - \exp\left\{ -\left(\frac{c}{\alpha_F\, h} \right)^{\beta_F} \right\} \qquad (3)$$

The parameters, α_F and β_F, are parameterized in terms of two parameters, a measure of steepness, s_1, and the Ursell number, u, which is a measure of the impact of water depth on the nonlinearity of waves. These quantities are given by:

[1] *q*-probability event means an event corresponding to an annual exceedance probability of *q*.

$$s_1 = \frac{2\pi\, h}{g\, t_1^2}, \qquad u = \frac{h}{k_1^2\, d^3} \tag{4}$$

h is the significant wave height, t_1 is the mean wave period calculated from the two first moments of the wave spectrum, k_1 is the corresponding wave number, g is acceleration of gravity, and d is the water depth.

For long-crested waves, the expressions for α_F and β_F read (*Forristall*, 2000):

$$\alpha_F = 0.3536 + 0.2892\, s_1 + 0.1060\, u \tag{5}$$

$$\beta_F = 2 - 2.1597\, s_1 + 0.0968\, u^2 \tag{6}$$

Instead of working with all individual crest heights, it is, regarding extreme value predictions, more convenient to consider the 3-hour maximum crest height, which is the quantity involved in Eq. (2). Denoting the zero-up-crossing wave frequency by $\nu_0^+(h,t)$ and assuming that all global crest heights of the short term (3-hour) sea states are statistically independent, the distribution function for the 3-hour maximum crest height reads:

$$F_{C_{3h}|H_sT_p}(c|h,t) = \left[F_{C|H_sT_p}(c|h,t)\right]^{10800\,\nu_0^+(h,t)} \tag{7}$$

The zero-up-crossing frequency can be calculated from the 0th and 2nd spectral moment of the wave spectrum and the 10800 showing up in Eq. (7) is 3 hours in seconds. Equation (7) may often be very well approximated by a Gumbel model. In that case, close form expressions for the Gumbel parameters can be given in terms α_F, β_F and $n_c = 10800\,\nu_0^+$ (see *Bury*, 1975).

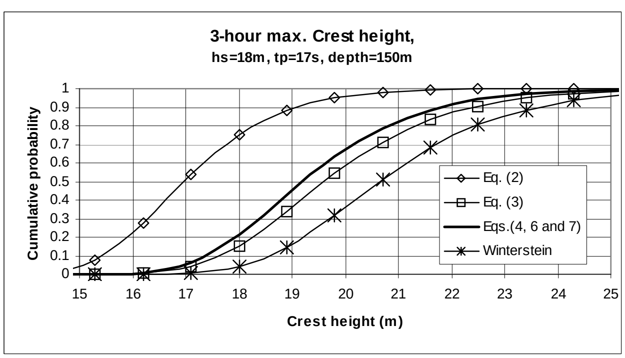


Figure 3. Distribution functions for 3-hour maximum crest height (*Haver*, 2002). Equation numbers refer to *Haver* (2002) and the meanings are as follows:
Eq. (2): Rayleigh distributed crest heights,
Eq. (3): Jahns and Wheeler distributed crest heights,
Eqs. (4,6, and 7): Forristall distributed crest heights,
Winterstein: The surface process is written as a Hermite expansion of a standard Gaussian process where the coefficients are determined such that certain global moments are obtained (*Winterstein*, 1988).

Figure 3 compares the distribution function for the 3-hour maximum crest height for some suggested crest height models (see figure caption). It is seen that the 3-hour largest crest height at a fixed site will show a considerable variation from one 3-hour period to another 3-hour period. For the Forristall model, we may experience a 3-hour maximum crest height in the range from 16 m to 25 m if a sea state with h=18 m and t=17 s is realized, while a typical value would be between, say, 18 and 21 m.

Adequacy of the second order model

A second order model for the surface elevation process is expected to yield an adequate approximation to the real sea surface in most cases. This is indicated by the results shown by *Forristall* (2000) and *Prevosto and Forristall* (2002).

Marginal measures of the possible deviation from the Gaussian hypothesis for the surface elevation process are the coefficient of skewness, γ_1, and coefficient of kurtosis, γ_2:

$$\gamma_1 = \frac{\overline{\mu}_3}{\sigma^3}, \quad \gamma_2 = \frac{\overline{\mu}_4}{\sigma^4} \tag{8}$$

where $\overline{\mu}_n = E\left[(\Xi(t) - \mu_\Xi)^n\right]$ is the central moment of order n of the surface elevation process, $\Xi(t)$, with mean value μ_Ξ, and $\sigma = \sqrt{\overline{\mu}_2}$ is the standard deviation of the surface process.

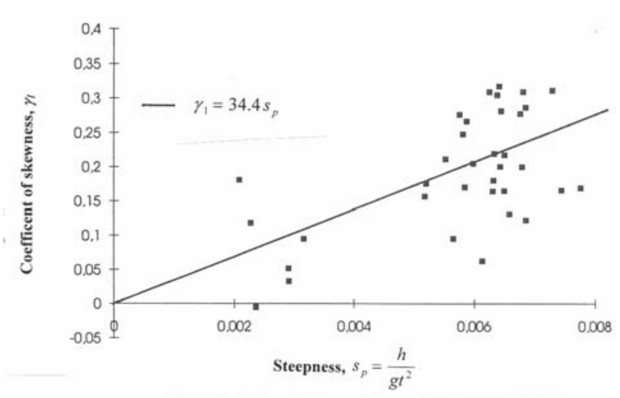
By introducing a perturbation expansion of the surface process, the coefficient of skewness is found to leading order by including second order terms, while third order terms are required in order to obtain the coefficient of kurtosis to leading order (*Vinje and Haver,* 1994). The coefficient of skewness deduced *from a second order expansion reads approximately* (*Vinje and Haver,* 1994):

$$\gamma_1 \approx 34.4 \frac{h}{gt^2} \tag{9}$$

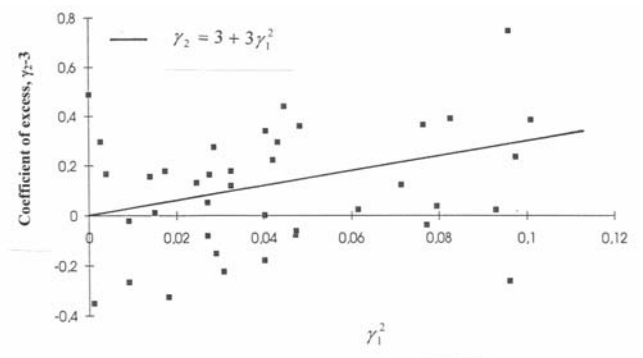
Equation (9) is compared with values estimated directly from full scale wave measurements in Figure 4a. In an average sense, a reasonably good agreement is seen. The measurements are made from a fixed platform using a down-looking laser device and the significant wave height, h, varies from 5 m to 13 m. Accounting approximately for terms up to third order, the following expression is suggested for the coefficient of kurtosis (*Vinje and Haver,* 1994):

$$\gamma_2 = 3 + 3\gamma_1^2 \tag{10}$$

Equation (10) is compared with values estimated from the wave measurements in Figure 4b. Again it is seen that the simple formula seems to agree with the average trend in the data.



a) Skewness versus steepness



b) Excess versus skewness squared

Figure 4. Calculated coefficients of skewness and kurtosis versus estimates from data (*Vinje and Haver*, 1994).

In order to indicate qualitatively the effects of skewness and the various contributions to the kurtosis, the surface elevation process can be modeled as a Hermite expansion of a standard Gaussian process (*Winterstein*, (1988). The coefficients of the expansion ensure that target values for the global statistical moments, mean, standard deviation, coefficient of skewness, and coefficient of kurtosis, are achieved. Utilizing this transformation, one may transform the distribution function of the normalized (= c/σ) 3-hour maximum crest obtained under the Gaussian assumption to distributions corresponding to various levels of deviations from the Gaussian assumption. The distribution functions are shown in a Gumbel probability paper in Figure 5.

Introducing a coefficient of skewness in agreement with a second order process, while (inconsistently) maintaining a Gaussian value for the coefficient of kurtosis, an increase of the 3-hour maximum crest height by 15-20% is seen from Figure 5. Utilizing the second order contribution to the kurtosis increases the crest height by 5%, while the third order contribution to the kurtosis results in a further increase of about 2%. These figures should be taken as somewhat approximate, but they give a certain idea of the relative importance of the various contributions. In Figure 3 it is indicated that the Winterstein expansion yields significantly larger crest heights than what is obtained by the parameterized results of a second order process.

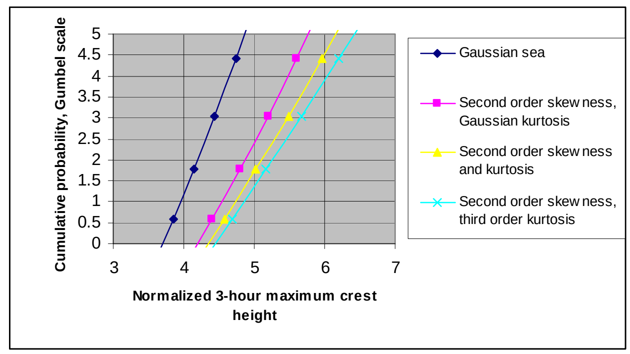


Figure 5. Distribution functions for the normalized 3-hour maximum crest height.

In summing up the adequacy of the second order model for the sea surface elevation, it will be concluded that the model most likely will represent an adequate model for most of the time. However, regarding extreme crest heights, a slight underestimation is expected to take place due to a sustained or steady state effect of higher order terms. This is also indicated when comparing second order theory with generated surface realizations in model tests (*Stansberg*, 1999).

Freak waves and the second order wave population

Over the years some few observations of rather extreme wave events have been made, see e.g., *Haver and Andersen* (2000). An example of such an event is the Draupner wave measured at a Statoil-operated jacket platform January 1, 1995 (*Haver*, 2004a). The 20-minute time history including the majestic crest height is shown in Figure 6. The normalized crest height experienced 270 seconds into the record is about 6.2. This is considerable larger than what is typically realized in 20-minute time series of a second order model. The probability of observing this value in a 20-minute window is about 10^{-4}, while the probability increases with roughly an order of magnitude if the window is extended to 3 hours.

A second-3order model can be forced to agree with the observed Draupner wave (*Jensen*, 2004), but the probability for this to be occurring for real waves is very small.

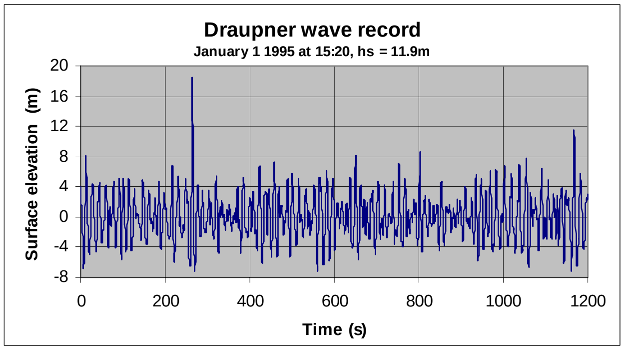


Figure 6. The Draupner wave record

Basic questions in connection with events of the type mentioned above are these:

i) Is the observed event a very rare realization from the typical, slightly non-Gaussian surface process population?

ii) Is the observed event a more typical realization of a very rare and strongly non-Gaussian surface process population?

The answers to these questions are what matter when it comes to consider whether or not phenomena like freak waves exist and, consequently, whether or not they may represent a problem for practical design work.

If the answer to the first question is yes, then there is no reason to be concerned about these events. Provided the wave modeling adopted for design accounts properly for deviations from the Gaussian assumption, i.e. being at least as severe as reflected by a second order model, this sort of wave event is implicitly accounted for by the standard design recipe. This, however, will not eliminate the chance of having a structure damaged in connection with a severe wave event, but the annual occurrence probability of such an event should be lower than say 10^{-4} – 10^{-5}.

If the answer to the first question is no, while the answer to the second question is yes, this sort of wave events may represent a greater challenge. In this case they are not explicitly accounted for by the present design practice beyond the robustness ensured by the partial load factors and the (hopefully) inherent conservatism of the design process. If such a strongly non-Gaussian population exists, it should in principle be accounted for if it affects the annual extreme value distribution of the target variable for exceedance probabilities larger than say 10^{-5}. Qualitatively, the effects on the annual extreme crest height of the various models for the sea surface process are indicated by Figure 7

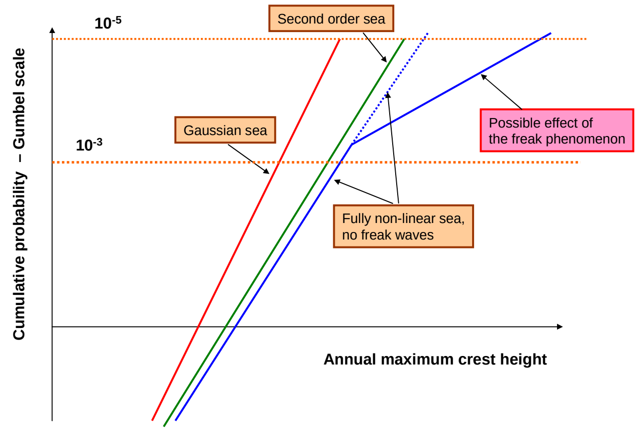


Figure 7. Qualitative indication of effects of non-linearities on the annual maximum crest height distribution.

If such a rare population of wave events shall be accounted for in design, we first of all have to agree on what is the definition of the waves belonging to such a population. There is presently no consensus regarding this definition. The most popular definition is to define freak events as events being larger than some limit, e.g., a freak wave height is often defined as a wave height larger than twice the significant wave height and a wave crest is defined as freak if it is larger than 1.25 times the significant wave height. Such a definition is rather incomplete if it is not followed by a specification of the observation window, i.e., 20 min., 3 hours or the full storm. It should also be pointed out that even under the second order model these factors are expected to be exceeded by a certain fraction of the wave events. The best use of these factors would be to adopt them as indicators of possible freak waves. They should also be limited to cases where the observation window refers to a point observation. If one extends the observations to cover the maximum over an area, one will even under the Gaussian assumption produce a considerable number of events well in the excess of the factors above (*Krogstad et al.*, 2004; *Forristall*, this volume).

In view of the basic questions, from a structural design point of view a more convenient definition is as follows (*Haver*, 2000, 2004b):

A freak wave event is an event (crest height, wave height, steepness or group of waves) that represents an outlier when seen in view of the population of events generated by a piecewise stationary and homogeneous second order model of the surface process.

The definition is anchored to the second order modeling since that is the most sophisticated numerical model available for routine design purposes. As more sophisticated models become available, the *"classical" extreme wave* population will grow on the cost of the *freak wave* population (*Haver,* 2004b).

From a structural design point of view, the freak wave challenge can be summarized as follows (*Haver*, 2004b):

a) Does a separate freak wave population exist?

One needs to identify the underlying physical mechanisms that can make a freak wave development possible. If these are beyond what is baked into a second order formulation, a separate population exists.

b) If a separate population exists, what is the conditional probability, say per 3-hour duration of a sea state, for a freak wave development to take place given some engineering sea state characteristics?

From a design point of view, a freak wave does not represent a problem unless it at a given site occurs sufficiently frequently to affect our predictions of 10^{-2}- and 10^{-4}-probability wave events.

c) *If a freak wave occurs in a 3-hour sea state with given characteristics, what is the conditional distribution for the freak wave amplification factor?*

The freak wave amplification factor is defined as the ratio $c_{3hr,freak}/c_{3hr,nonfreak}$. In some cases a freak wave will not be larger than the largest non-freak wave in another group of the sea state. On the other hand, if the bulk of the energy of the largest group of the sea state is focused into a single majestic wave, the amplification factor may be considerable.

Some evidence of freak waves and their possible threat to structural integrity

Over the centurie,s stories, myths and rumors have told about majestic waves experienced by sea men. During the last couple of decades, wave measurements and structural damage reports have indicated the presence of rather unexpected (in view of the overall sea conditions at the time) large crest heights. During the same periods, mathematical solutions of the governing equations and carefully designed model tests have suggested the existence of mechanisms that could cause individual waves to become much larger than what is accounted for in design. In sum, this suggests that further work should be done for the purpose of documenting whether or not freak waves represent a problem for practical design work.

If a freak wave should occur, the overall structural load will not by default represent a problem. Although the crest height of the Draupner wave is well in excess of the 10^{-2}-probability crest height, the global loading on the platform was less than 50% of the design wave load on the platform, i.e., 10^{-2}-probability load times 1.3 (*Hansteen et al.* , 2003). Of course this comforting observation could be the result of the inherent conservatism in the design process, but it may also suggest that the overall loading is not as extreme as the crest height of such waves. This is provided the massive crest height does not impact structural parts not designed for wave loading, e.g., deck structure of fixed platforms. The Draupner wave gave a considerable increase in the platform response (see Figure 8). The wave was possibly touching parts of a temporary working deck. But no damage to the deck was reported, so a massive wave-deck impact did not take place. A major wave-deck impact is possibly one major problem of these unexpected large crest heights if they reach well above the structural freeboard (distance from still water level to deck level). This is a load scenario which most fixed platforms are not designed against. The impulse type loading caused by a massive wave-deck impact could represent a threat to the structural integrity, even if the loading on the sub-structure is not that large in connection with the passage of the freak wave.

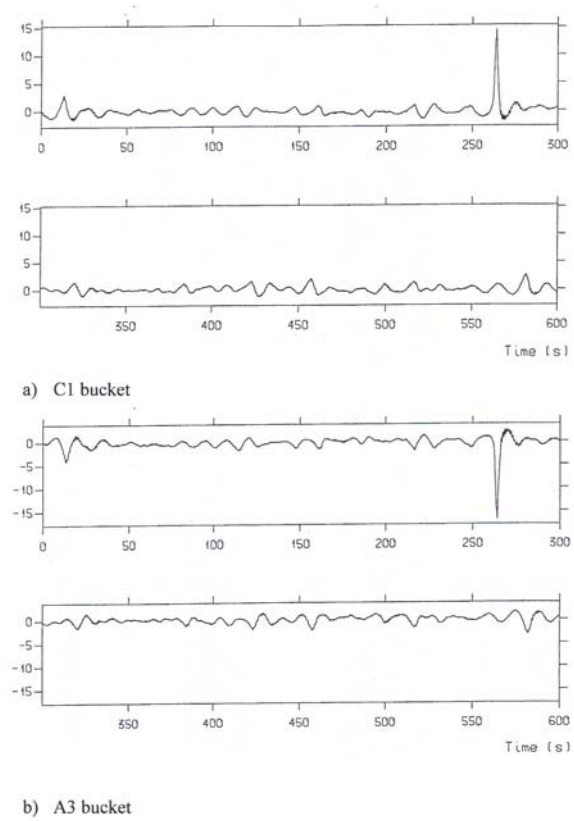


Figure 8. Vertical forces (MN) on the Draupner foundation buckets in connection with the Draupner wave (*Hansteen et al.*, 2003).

Mechanisms for making a freak wave development possible

A variety of possible mechanisms have been proposed as explanation of the observed possible freak wave events. Herein we will not enter into a detailed discussion of this subject. Various mechanisms are discussed by other authors at this workshop and the reader is referred to their corresponding papers. Various explanations are listed below, including some subjective views of the author to the various points. The background for this list is that it seems as if consensus is reached regarding the existence of unexpected severe wave events, but consensus is not reached regarding the mechanisms causing them and whether the observation should be classified as a freak event or not. Of course the latter is difficult since there is no general agreement to what is a freak wave event. In the discussion below, freak waves are defined in agreement with the definition given above.

A: The expectation of the observer regarding severity of wave events is too optimistic.

This may well be the explanation for a large number of freak waves reported over the years. The merchant shipping tradition has often been to adopt a 5×10^{-2}–probability wave event as the design wave event. The design wave is furthermore often modeled as a sine wave. If irregular seas are adopted, they are usually modeled as realizations from a Gaussian surface process. Consequently, the Rayleigh distribution is adopted for the maxima in a given sea state. It is not unlikely that this is the background for introducing all waves with a wave height exceeding twice the significant wave height as freak waves per default.

B: Nature has to fill the upper tail of the distribution functions.

This is another very likely reason for a number of freak wave or rogue wave reports. Even within assumptions underlying routine design, very large values may occur with low probability. Anchoring the understanding within extreme value theory, one may, in a somewhat popularized way, say; “The “impossible” is likely to happen some day” (*Gumbel*, 1958). The inherent variability of extremes is important to recognize if a factor definition is introduced as the definition of freak waves. The limitations of the suggested factor should be stated, i.e., it is questionable to use the factor defined based on point data to spatial observations. Regarding structural design, one should focus on a point (or nearly a point) when it comes to assessing annual exceedance probabilities of severe wave events. A structure may cover an area of 100 m × 100 m, but it cannot be at several distinct positions at the same time. The structure should be able to withstand all wave events occurring at the structural site (may be an area of 100 m × 100 m) with an annual exceedance probability of 10^{-4}–10^{-5} or higher. It is not designed to withstand the 10^{-4}–10^{-5}-probability wave event of an area of say 200 km × 200 km.

Bearing in mind the inherent randomness of the extremes, all observed so-called freak wave events can most probably be considered as low probability realizations of, say, a second order population, in particular if the effects of the spatial variability also are considered. However, sound engineering suggests that one should aim for eliminating the other mechanisms before falling back to this explanation.

C: Wave-current interaction mechanisms.

A direct effect of this mechanism is the increase in wave steepness that will be experienced as high waves propagate into a region of strong opposing current. This would of course make the surface process more exposed to wave breaking. Due to the increased steepness, ships sailing in the region could be more exposed to major slamming events. However, a rather strong current is needed for this effect to become strong and wave entering into an opposing current is not likely to explain freak waves as defined herein. It may well contribute to explaining damage to ships sailing along the east coast of South Africa due to heavy sea from the Southern Ocean entering into the strong southerly Agulhas current (*Mallory*, 1975).

A more indirect effect of wave-current interaction regarding huge wave events, could be wave refraction due to the presence of current, see e.g., *White and Fornberg* (1998). If waves propagate over a horizontally sheared current, a focus point can occur somewhere in a down-wave direction. It is not expected that this phenomenon can explain the Draupner wave, however. If such a focusing took place, it would be expected to result in a general worsening of the wave conditions close to the focus point. This phenomenon is not expected to change the shape of the extreme value distribution of the 20-minute maximum dramatically. Assuming the observed time history to be a realization of a second order process, the second order component process can be estimated through an iteration scheme. Under the hypothesis that the total process is second order, the remaining process (total process – second order component process) should be a Gaussian process.

For three 20-minute histories at Draupner, January 1, 1995, this has been done. The component processes are shown for parts of the time histories in Figures 9 a-c, while the distribution functions for the global maxima of the first order component processes are shown in Figures

10 a-c. It is seen that for both the 14:20 and the 16:20 time history, the estimated distribution functions are not too far off the Rayleigh distributions. However, it is seen from both cases that the upper tail is slightly fatter than the Rayleigh model. A possible explanation for this is that higher order terms (higher than 2) increase the severe crest heights slightly, say by about 5%. This is in reasonable agreement with the discussion accompanying Figure 5.

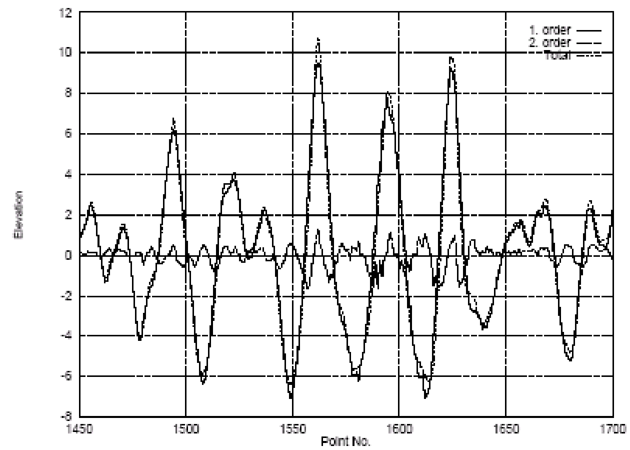
For the 15:20 series (the series with the Draupner wave, see Figure 6), however, the situation is quite different. The bulk of the distribution is in very good agreement with the hypothesis of the total process being a realization of a second order process. The upper tail, on the other hand, deviates significantly from this pattern. It clearly suggests that the underlying process is locally very much influenced by higher order phenomena. It should be noted, however, that the upper tail shape is defined by merely 2 out of about 180 crests, i.e., inherent randomness could be the reason. If current refraction had been the mechanism, one would not expect this to be materialized merely for 2 individual waves. It is therefore expected that the tail pattern observed at 15:20, could be due to inherent randomness or some other phenomenon very localized in time and space.

D: Sea state steepness and wave breaking mechanisms.

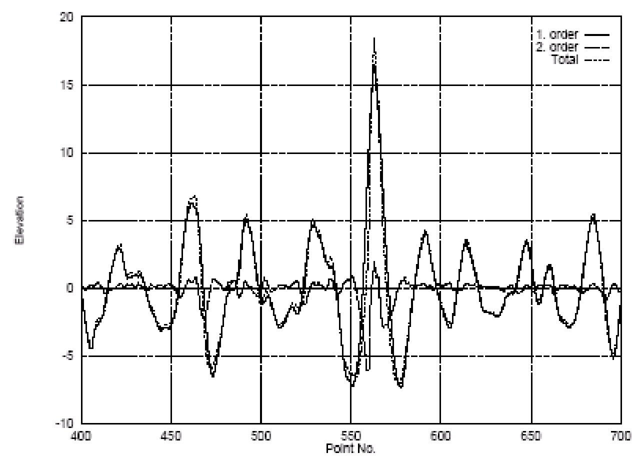
As the sea state steepness increases (or rather the steepness of energetic groups increases), the probability of experiencing instabilities in the surface process increases. During the early phase of the development towards breaking, the crest height will increase and it may reach a height well in the excess of the initial crest height. The onset of breaking seems to be rather sensitive to the local wave steepness, see e.g., *Banner and Tian* (1998). It may well be that it is a development towards wave breaking that is the explanation for the Draupner wave. Such a phenomenon could generate a global maximum distribution as indicated by Figure 10b.

E: Non-linear focusing within wave groups

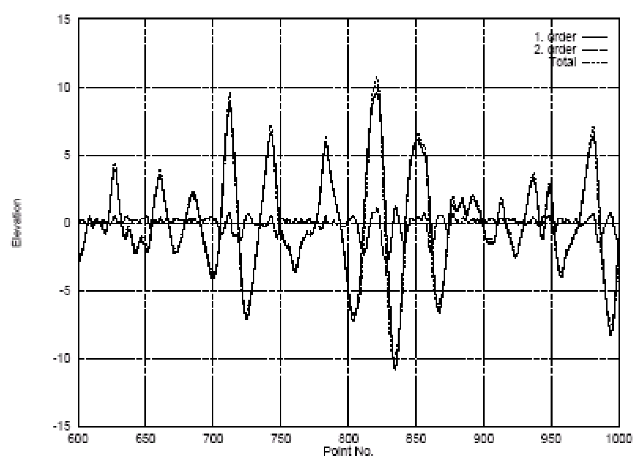
This phenomenon is addressed by a number of authors both in the literature and at this workshop, see e.g., *Dysthe and Trulsen* (1999) and *Osborne* (1999). The mechanism has also been demonstrated in model tests in the towing tank at Marintek, Onorato et al. (2004). This mechanism seems to produce solutions that agree qualitatively with what this author has expected to visually take place in connection with an event like the Draupner wave: "*One is looking at a stormy sea surface, rather large waves occur every now and then, but nothing beyond the typical is observed. Then suddenly something starts to evolve in a local area and after some few wave lengths this "something" has evolved into a majestic individual wave event, which some few wave lengths downwave has disappeared.*"



a) 14:20

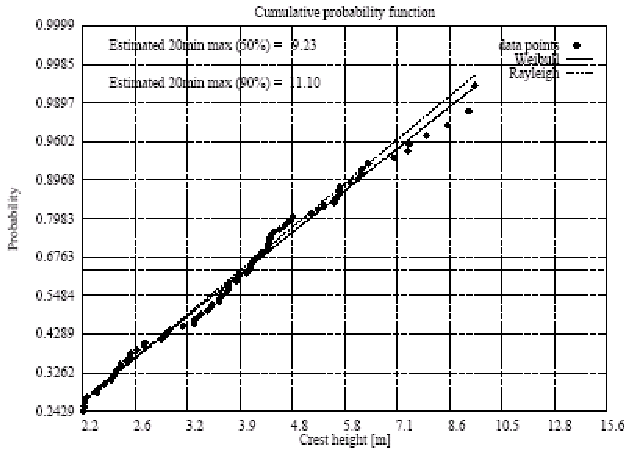


b) 15:20

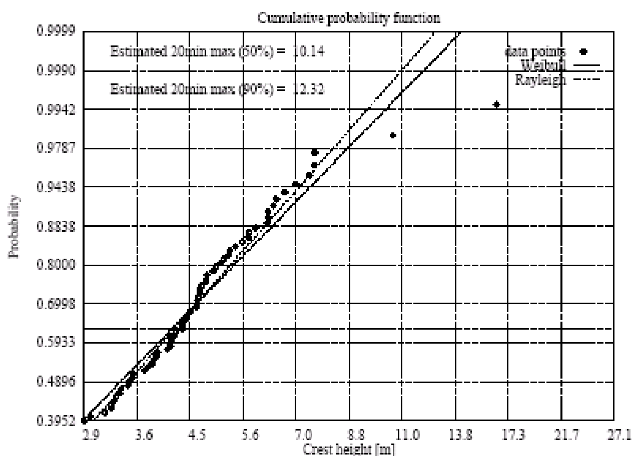


c) 16:20

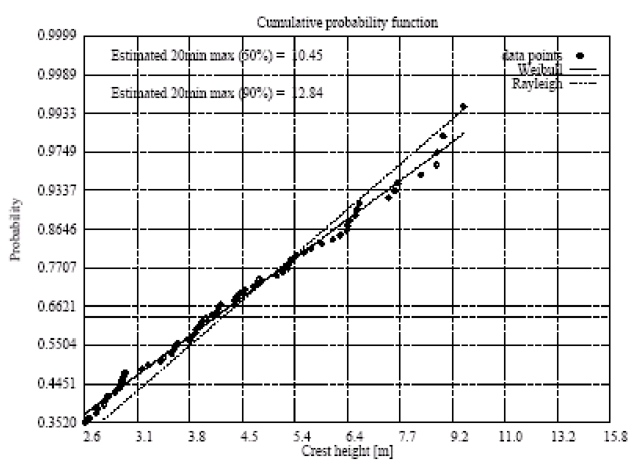
Figure 9. Measured surface process at Draupner January 1, 1995.



a) 14:20



b) 15:20



c) 16:20

Figure 10. Distribution of global maxima for the estimated first order component processes.

A mechanism like this is likely to produce a distribution function of global maxima like the distribution shown in Figure 10b. A challenge related to this mechanism is to anchor its onset to the underlying sea state characteristics. The mechanism is thoroughly discussed at this workshop and the reader is referred to other workshop papers for this subject.

F: Sea state directionality

A number of years ago, sea state directionality was proposed as a possible important property regarding the development of freak waves and their kinematics (e.g., *Sand et al.*, 1990). In this connection, freak waves were defined as waves with a height and/or crest height well beyond the typical values. During the last year, the directionality of sea states seems to have received increased attention again. With directionality we will think both of the short crestedness of a given sea system and the superposition of two (or more) sea systems with different direction of propagation. Linear superposition of waves with different direction of propagation is proposed as a possible explanation for observed freak wave events by *Donelan and Magnusson* (this volume), while sea state directionality in combination with the nonlinear self focusing mechanism is proposed as an important mechanism by *Osborne* (this volume). This author will be surprised if it is a linear superposition that explains wave events like the Draupner event, but it is a fact that the sea state in the Northern North Sea January 1, 1995, consisted of two sea systems (*Haver,* 2004a). The effect of sea state directionality should in combination with other mechanisms be further investigated in the future.

G: Other mechanisms

In most design work, short-term sea states are considered as realizations from a stationary and homogeneous random field. The duration of stationary conditions is usually taken to be 3 hours. By introducing stationarity and homogeneity, we have in some sense introduced strong restrictions on the wave models used for design. All model testing is based on these assumptions. In view of the possible existence of freak waves and the onset of mechanisms being able to produce these waves, one should review if the assumptions of stationarity and homogeneity in connection with numerical and physical modeling may erroneously suppress the onset of a freak wave development.

Concluding remarks regarding possible mechanisms

From a structural point of view it is not vital to know which mechanism or combination of mechanisms can result in severe wave events being more frequent than anticipated in design. The important thing is to learn if there exists such a mechanism or family of mechanisms not captured by standard design recipes. If it can be concluded that all observations considered as freak waves can be explained by the points A and B, everything is fine. There is no reason to be concerned about such waves for practical design. However, in order to reach that conclusion, we have to eliminate the other points above. This will require that the conditional probability of the onset of the various mechanisms given the sea state characteristics is calculated. This also goes for the conditional amplification of the, say, 3-hour maximum crest height given that the phenomenon is taking place. As these conditional probabilities become available, freak waves, if they exist as a separate population, can be accounted for in the design.

A possible framework for accounting for freak waves in design of offshore structures

Response prediction approach.

If the aim is to establish proper estimates of the q-probability values, e.g., crest heights, wave heights, or response amplitudes, one needs, in principle, to solve an integral of the form as indicated by Eq. (2). If this is to be done for wave crest heights accounting for freak waves, this means that we must know the 3-hour extreme value distribution for crest height for a large number of sea states. This can be difficult and/or time consuming for complicated problems. The freak wave problem does definitely qualify for being a complex problem. In order to determine $F_{C_{3h}|H_sT_p}(c|h,t)$ one will either have to base the assessment on a huge number of time domain simulations of the governing equations or execute a huge number of carefully designed model tests. Whether or when these approaches are available for reliably estimating the short-term probabilistic structure of C_{3h} accounting for the possible existence of freak waves is not quite clear yet. However, during the last decade, considerable work has been undertaken regarding solving the underlying mathematical models so, hopefully, reliable time domain simulations accounting for all possible freak mechanisms will be available within the next decade. Whether model tests at all can represent a valid approach for investigating the short term structure of C_{3h} accounting for possible freak events is not quite clear for this author. Model tests will be useful for deterministically verifying the existence of certain mechanisms. It is more questionable if a standard ocean basin can be used for the purpose of adequately reflecting the inherent randomness of extremes under influence of freak mechanisms, since such effects may need a number of wave lengths to develop. However, in the future, it may well be that this can be accounted for in an artificial way by the input signal to the wave maker.

For complex problems the environmental contour line approach is a convenient tool for obtaining proper estimates of the *q*-probability events (i.e., long term extremes) by means of short-term methods. This method is described into some detail in *Winterstein et al.* (1993), *Kleiven and Have*r (2004), and *Haver and Kleiven* (2004). Here we will mainly present the main steps of the method:

(a) Assuming that the joint probability density for H_s and T_p, $f_{H_sT_p}(h,t)$, is known, *q*-probability "contour"-lines can be established. In this connection the contour lines follow lines of constant exceedance probability. An example of a set of contour lines for a northern North Sea location is shown in Figure 11.

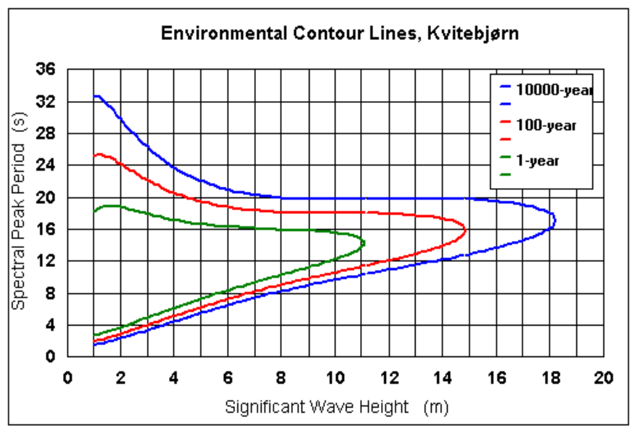


Figure 11. *q*-probability contour lines for the northern North Sea.

(b) The next step is to identify which sea state along the contour line is the worst sea state in view of the problem under consideration. For the crest height problem, it will typically be close to the part of the contour corresponding to the highest significant wave height.

(c) If the conditional probability density function of C_{3h} given the sea state characteristics is very narrow, the short term variability can be neglected and the variable C_{3h} can be replaced by, e.g., its median value, $c_{3h,0.5}$. This means that one can calculate the median 3-hour maximum crest height for sea states along the *q*-probability contour line. The maximum out of these will then be a proper estimate of the *q*-probability crest height.

(d) If the short term variability cannot be neglected, which will be the case in practice, one can obtain an approximate estimate for the *q*-probability value as follows. Establish the distribution function for C_{3h} for the most unfavourable sea state along the q-probability contour line. A reasonable estimate for the q-probability crest height is then obtained by the α-percentile of this distribution. A proper choice of α depends on the number of slowly varying characteristics (here 2, H_s and T_p), the nonlinearity of the problem (i.e., the extent of nonlinearity between the 3-hour crest height under the Gaussian assumption and the "exact" 3-hour crest height) and the target exceedance probability (i.e., value of q). For the present problem, we will assume that α=0.9 and α=0.95 are proper for $q=10^{-2}$ and $q=10^{-4}$, respectively.

Probabilistic description of 3-hour maximum crest height accounting for possible freak events

In the following illustration, we will adopt the contour line approach and mainly illustrate the suggested framework on the prediction of the 10^{-4}-probability structural load accounting for possible freak waves.

Prediction of 10^{-4} – probability crest heights accounting for possible freak waves

Without considering the underlying physics, the 3-hour maximum crest height accounting for possible effects of freak wave developments can be written:

$$C_{3h-F} = C_{3h-NF} + K\,\Delta C_{3h-F} = C_{3h-NF}\,(1 + K\,\Lambda) \quad (12)$$

C_{3h-NF} is the 3-hour maximum crest height under the hypothesis that the sea surface elevation is given as a second order random process, while C_{3h-F} is the 3-hour maximum crest height when the sea surface process may be exposed to a freak wave development. K is a geometric random factor that attains the value 1 if the surface conditions are such that a freak wave development can occur, otherwise, $K=0$. ΔC_{3h-F} is a measure of the effective increase in crest height due to the freak wave development. It is seen from Eq. (12) that the increase is conveniently modeled as a fraction variable, Λ, of the C_{3h-NF}. Λ is a random variable being larger or equal to 0.

Under the second order hypothesis, the distribution function for C_{3h-NF} is given by Eq. (7). The challenge is to establish the probabilistic structure of K and Λ being in agreement with the underlying physics of freak wave developments. So far this is out of reach and the present formulation is primarily made for illustrative purposes. However, it may still be used for indicating whether or not a freak wave is likely to represent a problem for the present design recipes for offshore structures. The probabilistic structure of K and Λ is discussed in *Haver et al.* (2004). $P(K=1|h,t)$ is modeled proportional to a two-dimensional Gaussian function, while Λ is modeled by a shifted and truncated 2-parameter Weibull model. The reason for using $F_{\Lambda|K}(\lambda|K=1)= F_0 > 0$ for $\lambda = 0$ is that a freak wave development will not always result in the 3-hour maximum crest height. The chosen models are shown in Figures 12 and 13.

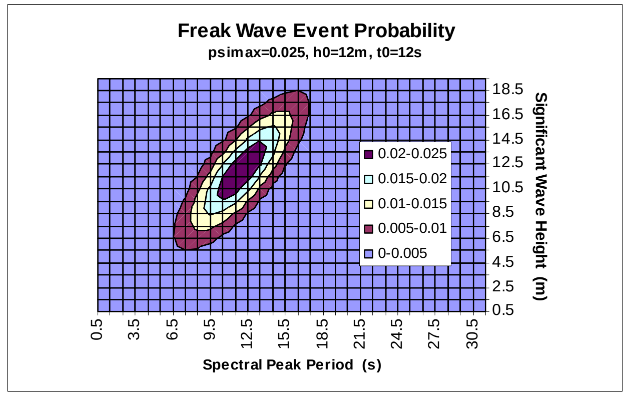


Figure 12. Probability mass function, $P(K=1|h,t) = \psi(h,t)$, for freak wave sea state conditions.

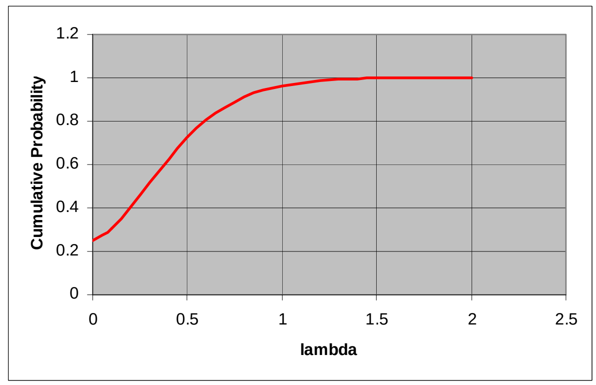


Figure 13. Conditional distribution function, $F_{\Lambda|K}(\lambda|K=1)$, for the freak wave amplification factor.

The centre position of $\psi(h,t)$, the maximum value, and the spreading around the maximum are some sort of a best guess based on present experience. However, it is of course somewhat arbitrarily chosen within the best guess domain. The hope is that in a foreseeable future, a more documented choice can be made for $\psi(h,t)$. The same is to be said regarding the probabilistic modeling of Λ.

Example: Failure of a generic jacket structure

A possible practical consequence will be illustrated by considering the annual probability of failure of a generic jacket. Neglecting wind, current, and water level variations, the total horizontal load, b (MN), on the generic jacket is assumed to be given by

$$b = 0.0228\,c^{2.4251} \quad (13)$$

where c is the crest height (m). For the field under consideration, see Figure 11, 10^{-2}- and 10^{-4}-probability crest heights under the second order assumption read 17.1 m and 22.0 m, respectively. Thus the 10^{-2}-probability load is found to be 22.3 MN.

Due to safety factors and the extra capacity due to system behaviour and capacity beyond the elastic capacity, the load representing a threat to the structural integrity will be much larger. Here we will assume that failure will take place if the total load on the structure exceeds 45MN, i.e., $b_{failure}$ = 45 MN. A major load problem for a jacket design is if extreme wave crests result in massive wave-deck impacts, because the structure is typically not designed for such events. To ensure that the annual probability for such a scenario is sufficiently small, the platform freeboard (height between deck level and still water surface) is presently often taken to be larger than the 10^{-4}-probability crest height. Based on this we will select the platform freeboard, h_D, equal to 23 m. If the crest height exceeds 23 m, a significant impact load will be experienced since a rather large area will be exposed to the impact load. The total load on the structure accounting for possible wave-deck impacts is given by:

$$B = 0.0228 \cdot \{\min(C_{3h}, h_D)\}^{2.4251} + 16 \cdot \max(0, (C_{3h} - h_D) \qquad (14)$$

Failure occurs as $B > b_{failure}$. In a full long-term analysis, the failure probability would be calculated for all sorts of 3-hour sea states. The marginal failure probability of an arbitrary 3-hour sea state is thereafter obtained by weighting these conditional probabilities with the respective probabilities for the various sea states, i.e., solving an integral analogue to Eq. (2). Here we will adopt the contour line approach, i.e., we will calculate the conditional failure probability for 3-hour sea states along the 10^{-4}-probability contour line. If the conditional failure probability is less than 0.05–0.1, we can at least conclude that is likely that the structure can withstand the 10^{-4}-probability wave-induced load accounting for possible freak events.

In this example it is assumed that the most unfavourable sea state is a sea state with h=17.9 m and t=17 s. The deck height is kept the same, but of course varying deck height could be an interesting parameter variation since a number of jackets worldwide will have a deck freeboard less than the 10^{-4}-probability crest height. The only thing that is varied herein is the probability of adequate freak wave conditions (whatever that means) for the adopted most unfavourable 10^{-4}-probability sea state.

The results in terms of the 3-hour extreme value distributions for the load on the jacket are shown in Figures 14-16. In Gumbel scale, 0.90 corresponds to 2.25 while 0.95 corresponds to close to 3. This means that if we shall be concerned about freak waves, they must show a significant effect on the 3-hour extreme value distribution for a cumulative probability in Gumbel scale in the range 2.25–3 or lower (larger exceedance probability). If the effect is seen well above 3, we would tend to conclude that freak waves will not affect the present design recipes.

It is seen from the figures that if the conditional probability of sea state conditions being adequate for the freak wave phenomenon is 0.5%, freak waves do not represent a problem. If the conditional problem is increased by a factor 10, i.e. every 20th realization of the sea state class is expected to have conditions adequate for freak wave developments, the freak wave phenomenon could be of concern. Finally, if the conditional probability is further increased by a factor 4, i.e., about every 5th realization will show conditions being favourable for freak wave developments, freak waves will be very important to consider. Although, these results will be somewhat sensitive to the modeling of the amplification factor, Λ, it is believed that these results represent a reasonable first guess regarding how frequent a freak wave phenomenon would have to be for the critical sea state in order to be of concern.

Are freak waves a problem for design of offshore structures?

Based on experience with analyzing wave time histories for 2-3 decades, it does not seem likely that a freak wave development probability should be larger than, say, 1% for a given sea state. This suggests that freaks are not a problem. Of course this figure is based on sea states where we have a considerable amount of data. The remaining challenge is therefore to consider whether what we have observed is valid for sea states beyond the observed severity, i.e. in the direction of the 10^{-4}–probability contour line in Figure 11. Collecting data is most probably not the way to go regarding solving this challenge. Further efforts on solving the mathematics and physics of freak waves are therefore recommended.

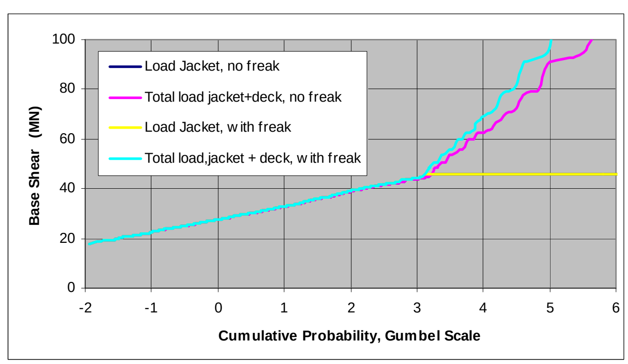


Figure 14. The 3-hour extreme value distribution for wave-induced load, ψ(17.9 m, 17 s) = 0.0055.

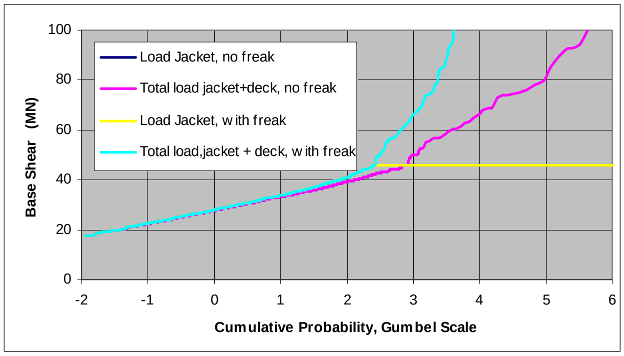


Figure 15. The 3-hour extreme value distribution for wave-induced load, ψ(17.9 m, 17 s) = 0.055

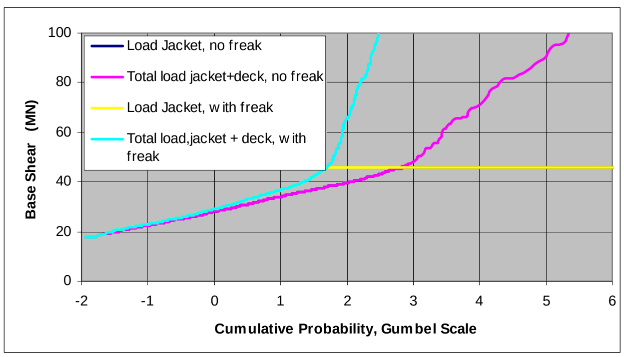


Figure 16. The 3-hour extreme value distribution for wave-induced load, ψ(17.9 m, 17 s) = 0.22

Conclusions

The effects of possible freak waves regarding present design recipes are discussed. Rules applied for offshore structures at the Norwegian Continental Shelf are briefly reviewed with respect to target annual exceedance probabilities of the loads and responses involved in design. Based on this, it is concluded that freak waves should be considered if they effect present predictions of wave extremes corresponding to annual exceedance probabilities of 10^{-5}-10^{-4} or higher. It is indicated that the surface elevation in most cases is reasonably well modeled by a second-order process. Freak waves are therefore recommended to be defined as events that do not belong to the population defined by the second-order model.

It is pointed out that there are numerous evidences of the existence of unexpected large waves. Based on a present best guess on the conditional probability of adequate freak wave conditions, freak waves are not expected to represent a problem for practical design work. This conclusion is based on the assumption that the probability of freak wave onset is not increasing with increasing sea severity beyond what is observed. In order to verify this assumption, further work regarding the underlying mechanisms is recommended.

A probabilistic framework which can, if necessary, be used for accounting for freak waves in design is suggested. The procedure is illustrated by applying it for estimating the failure probability of a generic jacket accounting for freak waves. Further work is recommended regarding the onset probability of the various mechanisms that can cause freak waves. It is also recommended that further work should also be undertaken regarding the effective amplification of the 3-hour extreme crest height given that a freak wave phenomenon occurs.

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