REASONS FOR FOCUSING MORE ON PREDICTION OF THE VERY EXTREME SEA STATES

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1. INTRODUCTION AND MOTIVATION

Processes describing the environment are capricious fellows. They are a result of complex physical mechanisms often involving strong non-linearities. Occasionally, the resulting structural loads may involve an on-off type of mechanism, making the design load prediction problem even more complex. Although our understanding of the underlying mechanisms has increased considerably during the last century, most of these processes must still be considered as being of an inherent random nature. In sum these processes generate the occasionally rather hostile environment which mankind for all times have challenged with their manmade structures – land based as well as sea going.

Going far back in time, structural design was of an empirical nature and evolved gradually as experience was gained the hard way. A structure was build to serve some purpose and if for some reason the structure failed, a new and much stronger structure was typically replacing the failed one.

As of today, structural design codes encourage a more rational design approach. The time period of the foreseeable use of the structure is defined a priori. Recipes and guidance for how to determine foreseeable maximum loads towards which the structures are to be designed are provided by various rules and regulations. This ensures that low probability load events are not missed (at least not those being in agreement with the state-of-the art knowledge of the environment faced by the structure). Using modern codes, rather optimized (and therefore slightly cheaper) structures can be designed. However, in connection with such an optimization, there is a danger that the structure becomes less robust towards unforeseen load events. This because the optimizing approach may reduce the conservatism traditionally used regarding a given load quantity reflecting a belief that an increased insight regarding this load has been achieved. This does not deteriorate structural safety below what is acceptable with respect to that particular load quantity. However, it may well be that this traditional conservatism could represent an important contribution to an implicit barrier against load events not considered explicitly in the design process. The more optimized a structure is, the more important it is to control the structure against the very low probability events (i.e. an annual occurrence probability comparable to the maximum acceptable failure probability) of the governing load processes (e.g. the wave process) if they are dominated by inherent randomness (which is the case for the wave elevation process).

A modern design code ensures a certain robustness towards the unforeseen by requiring that both the predicted loads and the calculated structural capacities are multiplied and divided, respectively, by partial safety factors. A simplified review of the design framework will be included in the next chapter. The annual exceedance probabilities of the selected characteristic loads are typically from 1/20 – 1/100. It is tacitly assumed that an overall acceptable safety is obtained when designing the structure against these target loads multiplied by the rule specified partial load factors. However, this is under the condition that the statistical structure of the load populations considered when predicting the target characteristic values also are representative for what takes place with a much lower annual exceedance probability.

A typical time period for available data, measurements or hindcast, could cover 10-50 years.

This is possibly not too bad for estimating loads corresponding to the required target eceeedance probabilities for the characteristic loads. But one should bear in mind that most likely rather large uncertainties will be associated with the estimation of values corresponding to annual exceedance probabilities in the order of 10^{-5} – 10^{-3}. This is important because it is the aim of the design process to ensure that the structure shall resist loads at these low probability levels with at most some local structural damage.

When exposing a structure to a wave or a wave train with a 10^{-2} - probability wave height or significant wave height, respectively, a given picture of the maximum loads on the structure is obtained. (Throughout the paper a ***q-probability event*** means an event corresponding to *an annual exceedance probability of q*.) If the severity of the wave event is worsened by say 25% without dramatically changing the load picture (i.e. except for a general increase of say 25-50%, the load picture is the same), there are good reasons to believe that the structure has some robustness against the most severe weather conditions. However, if the design process merely is focusing on say 10^{-2} - probability wave conditions, a possible occurrence of say 10^{-4} - probability wave conditions can result in a quite different maximum load picture on the structure, e.g. if a wave-deck impact is experienced. This means that the total environmental load on the structure increases by a factor 2 or more instead of the implicitly assumed increase of 25-50%. For certain structural elements one may go from essentially no load to a very large load.

In summing up this introductory discussion, a properly designed structure shall correspond to a very low annual probability of structural collapse. Here we will not present any explicit target for the failure probability. Denoting the maximum permissible annual collapse probability by q_t, a structure should at least be demonstrated to withstand all individual environmental events corresponding to annual exceedance probabilities in the vicinity of q_t. In this connection, one should expose the structure to the most severe environmental conditions in order to verify that the design load picture used in connection with the characteristic loads (corresponding to an annual occurrence probability a couple of order of magnitudes higher than q_t) is representative for the q_t-probability load picture, i.e. the difference between the load pictures should essentially be a scaling of the load level.

It is often claimed that the prediction of events corresponding to annual exceedance probabilities well below say 10^{-2} is associated with very large uncertainties. This is true, no doubt, however, these uncertainties are not disappearing by not facing them! It is also important to remember that it is load events corresponding to these rather low annual occurrence probabilities that may represent a risk to the structural integrity.

In the paper, a simplified framework of structural design will be presented. In connection with this, the use of the accidental limit state (ALS) to capture very rare metocean loads will be discussed. It will be pointed out when the ALS control could be important to include. A method for predicting very rare load and response quantities for complex response problems will be presented. If a consistent estimate of a q-probability load/response shall be achieved, it is important to capture both the variability in the long term metocean conditions and the variability associated with the largest response given the metocean condition. It is demonstrated that if the median or mean largest response in a q-probability sea state is adopted as a characteristic response, the exceedance probability of the characteristic value is much larger than q. For a strongly non-linear problem it may possibly be an order of magnitude higher. Finally, the paper is closed by indicating the adequacy of the suggested approach for a generic example exposed to a harsh weather storm climate.

2. FRAMEWORK OF DESIGN

According to Norwegian Rules and Regulations see e.g. NORSOK(1999) and PSA(2001), an offshore structure is to be controlled against overload failures at two levels; ultimate limit state (ULS) control and accidental limit state control (ALS).

The ULS design control will most often govern the design against environmental loads. However, at the end of this section we will point to cases where there is a need for a design control beyond ULS. In connection with ULS, the characteristic environmental load effect, x_c, is defined as the load effect corresponding to an annual exceedance probability of 10^{-2}. The characteristic capacity, y_c, is taken as a lower percentile (often 5%) of the distribution of the elastic component capacity. It should be noted that in practical design work we will also have to account for load effects caused by permanent and functional loads. In view of the illustrative nature of this paper we will herein limit ourselves to wave induced loads. Uncertainties of various origins will be associated with both x_c and y_c and in order to ensure a sufficient margin against structural failure, partial safety factors, γ_f and γ_m, are introduced, i.e. the ULS control reads (when neglecting permanent and functional loads):

$$\gamma_f \, x_c \leq \frac{y_c}{\gamma_m} \qquad (1)$$

For steel structures on the Norwegian Continental Shelf, $\gamma_f = 1.3$ and $\gamma_m = 1.15$ will typically have to be used.

For a given load pattern, the variability in the capacity (which essentially is of an epistemic nature, i.e. it is caused by lack of knowledge) is typically rather small. Provided gross errors are avoided by an adequate quality assurance procedure, the distribution function reflecting the uncertainties in the capacity is typically well behaved, i.e. y_c/γ_m is expected to be a robust estimate of the design capacity.

For offshore structures, the nature of the load side of the problem is very different. The estimated characteristic load is of course also affected by epistemic type of uncertainties, but the dominating source of variability is the inherent randomness (aleatory variability) of the environmental processes. This means that with a very low annual probability, the structure can face loads significantly larger than the characteristic load - even if epistemic uncertainties where non-existing.

The values given above for γ_f and γ_m are meant to account for the typical levels of total variability associated with x_c and y_c excluding gross error effects. Regarding the load side, for a linear response problem (i.e. there is a linear relation between the response process and the wave process) the coefficient of variation for the annual maximum load (standard deviation over mean) is typically around 10% for Norwegian waters (somewhat larger for e.g. Gulf of Mexico conditions). If we multiply the value corresponding to an annual exceedance probability of 10^{-2} by the load factor of 1.3, the annual exceedance probability of $\gamma_f x_c$ is usually somewhat lower than 10^{-4}. For a non-linear response problem, say a quadratic problem, $\gamma_f x_c$ will typically correspond to an annual exceedance probability somewhat higher than 10^{-4}.

This illustration shows that (as far as γ_f is fixed) the annual exceedance probability of the design load is problem dependent. However, the annual exceedance probability will in most cases be within in a range around 10^{-4}. The width of the range could possibly be close to an order of magnitude. This, together with the fact that there is a rather low probability for the actual structural capacity to be lower than the predicted design capacity, ensures that the annual failure probability of the structure is sufficiently low. It is important to note that this is obtained provided the tails of the load and capacity distributions (upper tail of the load distribution and lower tail for the capacity) are well-behaved. The question thus becomes will the distribution tails always be well-behaved?

Restricting the consideration to the load side of the problem, it is rather easy to think of scenarios resulting in a bad-behaving upper tail, i.e. a tail with a shape parameter changing abruptly for an annual exceedance probability well above 10^{-4}. The difference between a well-behaving tail and a bad-behaving tail is illustrated in Fig. 1.

It is seen that for the well-behaving system, $\gamma_f x_c$ will give a design load level corresponding to an annual exceedance probability typically around 10^{-4}. For the bad-behaving problem, however, it is seen that this is far from the case. The product, $\gamma_f x_c$, corresponds to an annual exceedance probability much larger than 10^{-4}. For the bad-behaving case, it can be difficult to verify a sufficiently low annual failure probability, in particular if it is a manned platform.

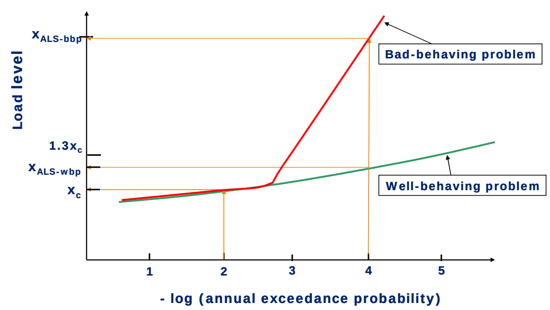

Fig. 1 Bad-behaved versus well-behaved response problem.

Design codes typically require offshore structures to be controlled against various accidental loads, i.e. collision loads, loads due to fires and loads due to explosions. According to Norwegian Rules, structures are required to withstand accidental loads corresponding to an annual occurrence probability of 10^{-4}. If the upper tail of the environmental load distribution is of a bad-behaving nature, Fig. 1 illustrates that one may well have a situation where the environmental load corresponding to an annual exceedance probability of 10^{-4} is much larger than the design load, $\gamma_f x_c$, predicted by the ULS design recipe. In view of what is required for accidental type loading, it is reasonable to consider this as an accidental load scenario. An excessive environmental load may be just as dangerous for the structure as a collision load.

In order to ensure that such cases are captured by the design process, the Norwegian Rules for offshore structures require that the ALS limit state also shall be applied to environmental loads. As for other accidental scenarios, the accidental loads are defined as the loads corresponding to an annual exceedance probability of 10^{-4}, i.e. $x_{c,ALS} > x_c$.

The capacity may in connection with the ALS control be taken as the system failure capacity where also effects of plasticity are utilized, i.e. $y_{c,ALS} > y_c$. The limit state formulation when neglecting permanent and functional loads is given by Eq. (1), but with the ALS values of x_c and y_c introduced, γ_f and γ_m are in most cases set equal to 1.0.

For old structures where the load pattern for one reason or the other is considerably changed, e.g. worsened wave conditions, reservoir subsidence, etc, one can very well foresee that a bad-behaving tail property is realized. Wave – deck impact is a mechanism that typically will result in a load – exceedance probability relation like the red curve in Fig. 1. A possible reason for experiencing a wave-in-deck problem could be that the wave conditions are more severe than what was predicted as the platform was designed – either in terms of storm severity or in terms of asymmetry of the surface elevation process.

As a conclusion, one may say that the ALS control with respect to environmental loads is a convenient way of ensuring a certain robustness against unforeseen environmental loads. Such a load scenario can be the case in connection with wave-deck impacts for fixed and floating platforms and green water loads on the deck structures of a ship. An unforeseen wave-deck impact may for example occur in connection if a storm much more severe than the ULS design storm hit the area or if a freak wave hit the structure. It is difficult to foresee all possible severe wave scenarios. However, by ensuring that the structure can withstand 10^{-4} – probability environmental loads predicted in view of best available knowledge, it is likely that the structure will have some robustness against the most extreme environmental loads.

3. ESTIMATING 10^{-4} - PROBABILITY LOADS

It is again important to note that according to Norwegian Rules and Regulations, the target annual exceedance probability refer to the load and not the environmental condition. This means that in connection with the ALS control against environmental loads, one should obtain reliable estimates for load corresponding to an annual exceedance probability of 10^{-4}.

In order to establish a consistent estimate for a load corresponding to a given annual exceedance probability, some sort of a long term response analysis is in principle required. A long term analysis can conveniently be done by selecting the 3-hour maximum load as our target response quantity, i.e. the weather development is approximated by a sequence of stationary 3-hour events. This sequence is a "continuous" sequence if all adjacent 3-hour weather conditions are included. This is fine if the weather in the area under consideration can be considered as some sort of a one population type of weather. In hurricane governed areas, it may be more adequate to merely consider the sequences of 3-hour events representing the hurricane episodes.

Denoting the conditional distribution of the 3-hour extreme value, X_{3h}, given the sea state characteristics, H_s and T_p, by $F_{X_{3h}|H_sT_p}(x|h,t)$, the long term distribution of the 3-hour maximum (or the marginal distribution of the 3-hour maximum) is given by:

$$F_{X_{3h}}(x) = \int_h \int_t F_{X_{\max}|H_s,T_p}(x|h,t)\, f_{H_sT_p}(h,t)\, dtdh \qquad (2)$$

$f_{H_sT_p}(h,t)$ is the long term joint distribution of H_s and T_p representing the governing sea state population regarding extreme loads.

As the long term distribution for the 3-hour maximum load is found, a consistent estimate for the value corresponding to an annual exceedance probability of q is found by solving:

$$1 - F_{X_{3h}}(x_{3h,q}) = q / m_{3h} \qquad (3)$$

m_{3h} is the expected annual number of 3-hour periods of the target population, i.e. if all 3-hour sea states is included in the target population, m_{3h}=2920, while m_{3h} is much lower if merely severe storms are considered.

In Eq. (2), $f_{H_sT_p}(h,t)$ denotes the joint long term distribution of the metocean characteristics. This quantity accounts for the variability in the metocean conditions. The challenge related to this quantity is primarily the availability of a sufficient amount of simultaneous data, measurements or hindcast. The physics of the response problem is baked into the conditional distribution of X_{3h} given the metocean conditions. For a very complex problem, this conditional distribution is by far the major challenge related with Eq. (2). In particular, if the problem is of an on-off nature. This will be the case if the problem includes rare wave-deck impacts.

For complex problems where an extensive model test program is required in order to identify the short term structure of the conditional distribution, the execution of a full long term analysis, Eq. (2), will be costly and time consuming. For such a case the environmental contour line approach, see e.g. Haver and Kleiven (2004), is a convenient approach for obtaining reasonable estimates for the target response quantity, i.e the value of X_{3h} corresponding to an annual exceedance probability of 10^{-4}. For more information on this approach reference is made to Haver and Kleiven (2004) or references included therein. Here we will merely summarize the basic steps of the method and, thereafter, indicate the adequacy of the method when applied to a generic response problem exposed to a storm population type of climate.

Using the environmental contour line approach, a reasonable estimate for the q-probability value (i.e. the response value corresponding to an annual exceedance probability of q) can be obtained by the following steps:

1. Establish the q-probability contour or surface for the involved metocean characteristics, e.g. significant wave height and spectral peak period.
2. Identify the most unfavourable metocean condition along the q-probability contour/surface.
3. Establish the distribution function for the 3-hour maximum response for the unfavourable metocean condition.
4. An estimate for the q-probability response value is now obtained by the α-quantile of this extreme value distribution. If, say, two metocean characteristics are included, e.g. significant wave height and spectral peak period, an adequate value of α will typically be around. 0.90.

It is to be stressed that this is an approximate method, a full long term analysis is required if the estimate is to be verified. Experience with the method seems to suggest that it is rather robust for most structural problems. From Eq. (2) it is seen that two essentially different sources of inherent randomness is included in a full long term analysis; variability related to the environmental conditions (long term variability) and the variability of the 3-hour extreme value given the environment (short term variability). The basic idea by the environmental contour line approach is that the relative importance of these two sources is more or less the same for all structural problems. The long term variability is the dominating source, while the short term variability is more or less some sort of a perturbation of the long term results. Moderate changes in the relative importance can be compensated for by varying α, see step 4 above, around 0.90.

4. CONTOUR LINES FOR A STORM CLIMATE

In lack of access to data from a hurricane region, storms exceeding 8m significant wave heights for the Northern North Sea are selected as example data. For the years 1973 – 2006, the available data series includes 159 storm events with H_s > 8m where estimates both for H_s and T_p are available. Some few storms are obviously missing from this data base, but the series is considered acceptable for the present purposes. The observations include storm peaks up to 13m significant wave height. Mean and standard deviation of the storm sample are 9.18m and 0.96m, respectively.

In order to establish contour lines, a joint distribution is needed for the included characteristics. Here we will merely include the storm peak values, H_{sp} and T_{pp}, as short term characteristics. An alternative would be to include all 3-hour events exceeding 8m as the sample for which a joint distribution of H_s and T_p is established, i.e. a storm is typically characterized by several pairs of H_s and T_p. As the full long term response analysis is carried out in a later chapter, all 3-hour events exceeding 8m are accounted for. It is possible that the most consistent approach would be to establish contours for H_s and T_p including all data above 8m. But regarding the most interesting parts of the contours it is not expected to effect the location of the contours too much.

The joint probability density function for H_{sp} and T_{pp} are conveniently written:

$$f_{H_{sp}T_{pp}}(h,t) = f_{H_{sp}}(h)\, f_{T_{pp}|H_{sp}}(t|h) \qquad (4)$$

Since the actual storm sample is obtained by selecting all storm events above h_0 = 8m significant wave height, it is assumed that $(H_{sp} - h_0)^{\lambda}$ can be reasonably well modelled by an exponential distribution. A least square approach is selected as the fitting procedure. This gave as a result the following model:

$$F_{H_{sp}}(h) = 1 - \exp\left\{ -\frac{(h-8)^{1.2}}{1.31} \right\} \qquad (5)$$

Eq. (5) is compared to the sample distribution in Fig. 2.

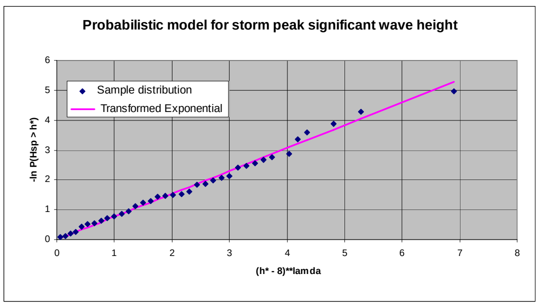

Fig. 2 Fitted probabilistic model for H_{sp}.

159 storm events during 33 years suggest that the expected no. of storms per year is 4.82. Accordingly, the storm peak significant wave height corresponding to an annual excedance probability of q is found by:

$$1 - F_{H_{sp}}(h_q) = \frac{q}{4.82} \qquad (6)$$

Storm peaks corresponding to q = 10^{-2} and 10^{-4} are found to be 13.7m and 17.1m. These values are slightly lower than the values obtained adopting all 3-hour data for the Northern North Sea.

The conditional distribution of the storm peak spectral peak period, T_{pp}, given the storm peak significant wave height, H_{sp}, is assumed to follow a Gaussian distribution:

$$f_{T_{pp}|H_{sp}}(t\,|\,h) = \frac{1}{\sqrt{2\pi}\,\sigma(h)} \exp\left\{ -\frac{1}{2}\left(\frac{t-\mu(h)}{\sigma(h)} \right)^2 \right\} \qquad (7)$$

The fitted Gaussian distribution is compared to the sample distribution for some few classes of H_{sp} in Fig. 3.

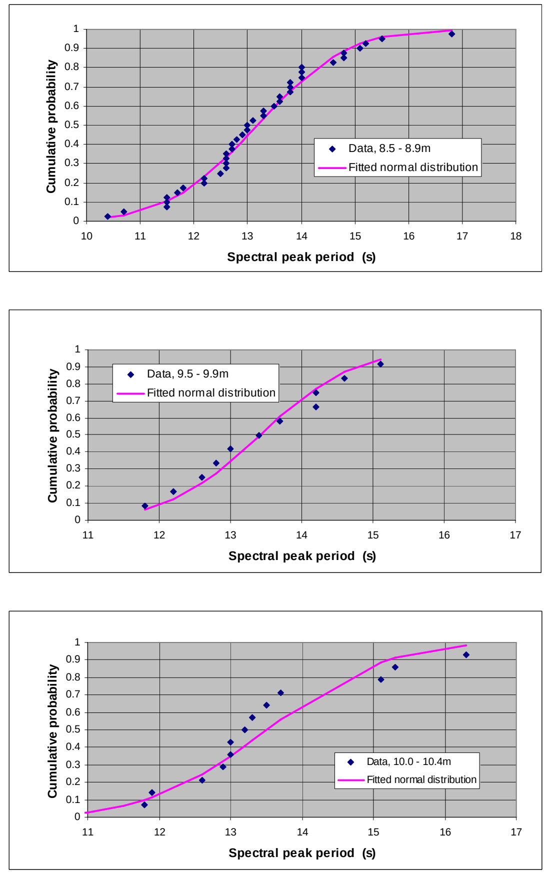

Fig. 3 Conditional distribution of T_{pp} given H_{sp}.

In order to estimate the conditional mean and standard deviation, $\mu(h)$ and $\sigma(h)$, beyond the range of observations, smoothed functions for the parameters are needed. The point estimates for the conditional mean and standard deviation are shown in Figs. 4 and 5. For the mean a linear regression seems reasonable, while for the standard deviation the average of the point estimates is used in the further modelling. The adopted values for $\mu(h)$ and $\sigma(h)$ are given in Figs. 4 and 5, respectively.

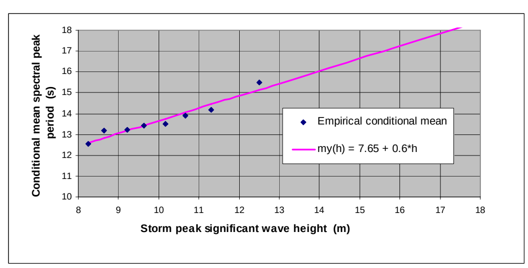

Fig. 4 Conditional mean storm peak spectral peak period.

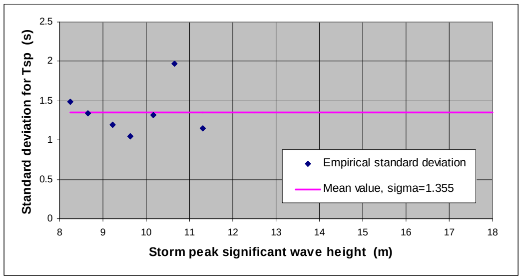

Fig. 5 Conditional standard deviation of storm peak spectral peak period.

As the joint distribution of H_{sp} and T_{pp}, $f_{H_{sp}T_{pp}}(h,t)$, is known, the joint distribution can be transformed over to a variable space consisting of independent standard Gaussian variables, U_1 and U_2. Associating U_1 with H_{sp} and U_2 with T_{pp}, this can be done using the Rosenblatt transformation, see e.g. Madsen et al. (1986):

$$\Phi(u_1) = F_{H_{sp}}(h) \tag{8}$$

$$\Phi(u_2) = F_{T_{pp}|H_{sp}}(t|h) \tag{9}$$

where Φ( ) is the standard Gaussian distribution function.

From Eqs. (8 and 9) one obtains a unique transformation from one point in the physical parameter space to a specific corresponding point in the standard Gaussian space (u-space) and vice versa. The reason for transforming the problem over to a standard Gaussian space is that contour lines corresponding to given exceedance probabilities are circles (or spheres) in the space defined by independent, standard Gaussian variables. The radius of the contour line corresponding to an exceedance probability per storm of q/4.82 is given by:

$$r_q = -\Phi^{-1}\left(-\frac{q}{4.82}\right) \tag{10}$$

Regarding explanation of q and 4.82, reference is given to the text paragraph associated with Eq.(6). For further details regarding the determination of contour lines, reference is made to Winterstein et el. (1993) and Kleiven and Haver(2004).

All combinations of u_1 and u_2 located on the contour line with radius r_q, do correspond to an annual exceedance probability of q. By transforming these points back to the physical parameter space by means of Eqs. (8 and 9), the q-probability contour lines for H_{sp} and T_{pp} are obtained. The contour lines obtained for the selected storm model are shown in Fig. 6 for q = *0.63*, *q=0.1*, *q=0.01*, *q=0.001* and *q=0.0001*. (q=0.63 is defined to represent the 1-year contour line.)

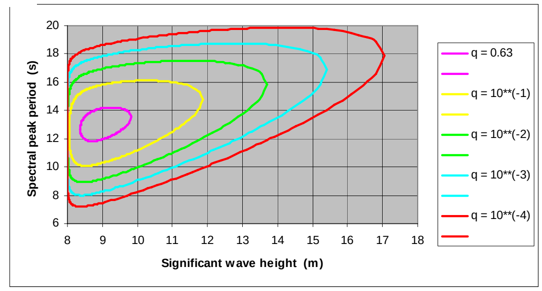

Fig. 6 q-probability contour lines for the example storm climate.

5. EXAMPLE APPLICATION

In order to illustrate the application of environmental contour lines, an rather idealized response problem is selected. It is assumed that the 3-hour maximum, X_{3h}, of a given response quantity is described by the Gumbel model:

$$F_{X_{3h}|H_{sp}T_{pp}}(x|h,t) = \exp\left\{-\exp\left[-\left(\frac{x-\alpha(h,t)}{\beta(h,t)}\right)\right]\right\} \tag{11}$$

The selected model will typically be a very good probabilistic model for the 3-hour maximum for most practical response problems. The nature of the response problem is reflected in the estimated response surfaces $a(h,t)$ and $\beta(h,t)$.

For the purpose of this illustration it is assumed that the response process is proportional with the square of the surface elevation process. Assuming that the wave process is close to a Gaussian process, i.e. Rayleigh distributed amplitudes (global maxima), the global maxima of the response process will be exponentially distributed. (Global maximum: Largest maximum between adjacent zero-up-crossings.) Under these assumptions we will have:

$$\alpha(h,t) = \beta(h,t)\ln\left(\frac{10800}{0.75t}\right) \tag{12}$$

10800 is simply the duration of the 3-hour sea state in seconds and 0.75 is taken as the ratio between the zero up-crossing wave period and the spectral peak period. The number of response maxima in 3 hours is for the purpose of this study assumed equal to the number of waves during the sea state. The duration (or number of maxima) is not a very important quantity so in view of the purpose of this paper this approximation is of a sufficient accuracy.

As a generic scale parameter, the following function is selected:

$$\beta(h,t) = 0.1h^2\left[1+\cos^{40}\left(\frac{2\pi(t-11.5)}{80}\right)\right] \quad (13)$$

It is seen that the selected function represents a system where the scale parameter is proportional to the square of the significant wave height, h. Furthermore, it is seen that the system is particularly sensitive to a period band around 11.5s spectral peak period. The scale parameter is shown versus spectral peak period for two values of significant wave height in Fig. 7.

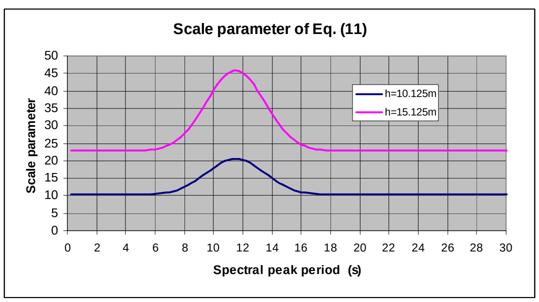

Fig. 7 Scale parameter of example 3-hour extreme value.

Provided response surfaces for α and β are known, it is rather straight forward to carry out a full long term analysis. The long term distribution of X_{3h} is given by:

$$F_{X_{3h}}(x) = \int_h\int_t F_{X_{3h}|H_sT_p}(x|h,t)\, f_{H_sT_p}(h,t)\,dtdh \quad (14)$$

In order to be in consistence with the storm climate considered herein, $f_{H_sT_p}(h,t)$ is the joint probability density of all sea states exceeding 8m significant wave height. A joint model for H_s and T_p for all sea states is presented in Haver and Nyhus (1986). This model is representative for the offshore area covered by the present storm data population, but of course it is deduced based on data from a much shorter date period, 1973-1983. By simply truncating this model at H_s = *8m*, a joint model for all sea states above 8m is obtained. For the range of concern, H_s > 8m, the marginal density function of H_s of this model reads:

$$f_{H_s}(h) = k\frac{\lambda}{\rho^\lambda}h^{\lambda-1}\exp\left\{-\left(\frac{h}{\rho}\right)^\lambda\right\},\ h > 8\text{m} \quad (15)$$

The scaling constant, *k*, reads:

$$k = \frac{1}{1-F_{H_s}(h_0)} \quad (16)$$

h_0 is the truncation threshold and is here 8m. The parameters of the distribution are $\rho = 2.822$ and $\lambda = 1.547$, Haver and Nyhus (1986). Thus *k*=1/0.0067 is found.

The truncated model for H_s is compared to the empirical distribution in Fig. 8. The empirical distribution is obtained from all observations above 8m H_s for the selected date series from the Northern North Sea 1973 – 2006. The model looks nice up to about 11m significant wave height. Above this level the model seems to slightly conservative, however, in view of the rather limited number of observations above 11m, the model is herein accepted. A simplified least square fit of the same model was carried out for the present sample and the resulting model was rather close to the model shown in Fig. 8 (in fact it was apparently slightly more conservative regarding the upper tail as suggested by the storm data).

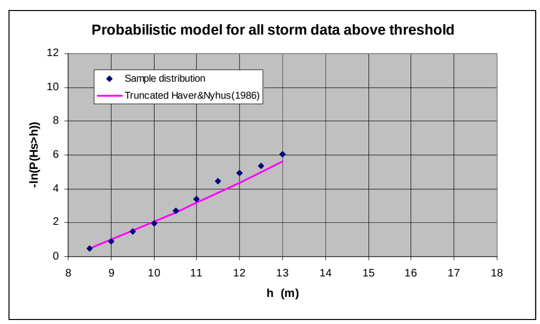

Fig. 8 Truncated Haver and Nyhus model for Hs > 8m compared to the empirical model.

The conditional distribution is modelled by a log-normal model, but the model parameters are such that the distribution is close to a Gaussian model for H_s > 8m. The model reads:

$$f_{T_p|H_s}(t|h) = \frac{1}{\sqrt{2\pi}\,\phi t}\exp\left\{-\frac{1}{2}\left(\frac{\ln t-\mu(h)}{\phi(h)}\right)^2\right\} \quad (17)$$

Where $\mu(h)$ and $\phi^2(h)$ are the conditional mean and variance of $\ln T_p$ given H_s, respectively.

Smoothed functions are fitted to the point estimates for the various significant wave height classes and the recommendations in Haver and Nyhus (1986) are:

$$\mu(h) = 1.59 + 0.42\ln(h+2) \qquad (18)$$

$$\phi^2(h) = 0.005 + 0.085\exp\left\{-0.13h^{1.34}\right\} \qquad (19)$$

In view of the present application, Eqs.(18 and 19) could have been simplified, but this will not be done herein.

Utilizing Eq. (14) for combining the short term variability of the response extreme value and the long term wave climate variation described by the truncated Haver and Nyhus joint model, the long term (or marginal) distribution of X_{3h} is obtained. The long term distribution in terms of the exceedance probability distribution is shown in Fig. 9.

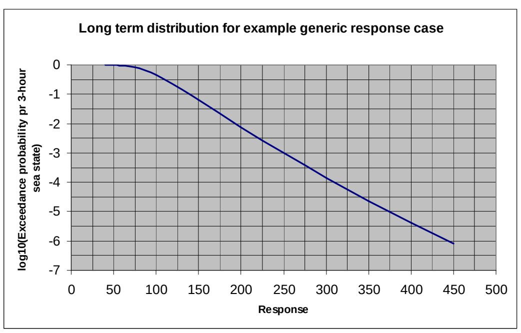

Fig. 9 Long term exceedance probability for an arbitrary 3-hour sea state above 8m.

According to the original Haver and Nyhus joint model, the probability of exceeding 8m significant wave height by an arbitrary 3-hour sea state is 0.0067. The expected (equivalent) annual number of 3-hour sea states above 8m is therefore 19.56. When considering all sea states above 8m, the exceedance probability corresponding to an annual exceedance probability of q then reads:

$$q_{All\ Sea>8m} = \frac{q}{19.56} \qquad (20)$$

Using this relation, values corresponding to various annual exceedance probabilities are given in Table 1.

If the response problem under consideration is of a very complex nature, it is a challenge to establish the short term probabilistic model, e.g. a model like Eq.(11). In such cases extensive model tests or time domain computer simulations may be required. For such cases, it is convenient if the analysis could be narrowed to merely consider some few short term sea states. For this purpose, the environmental contour line approach could represent a possibility.

Table 1 q-probability response extremes (return period in years in parenthesis) using the full long term approach

Annual exceedance probability	**Response value**
0.63 (1 year)	155
0.1 (10 years)	209
0.01 (100 years)	266
0.001 (1000 years)	327
0.0001 (10000 years)	393

The contour lines are determined in an earlier section of the paper. What remains to be done is to find which quantile of the short term extreme value analysis yields an adequate estimate of the target long term extreme value.

A priori, in view of how the contours are prepared, we know that if we could neglect the short term variability, i.e. the short term extreme value distribution was extremely narrow, we could estimate the q-probability response by the median response for the most unfavourable sea state along the q-probability contour line.

For practical problems we cannot neglect the short term variability. However, after we have identified the most unfavourable sea state along the contour line, *the design sea state*, we can compensate for the short term variability of this sea state and of all neighbouring (in the scatter diagram) sea states by selecting a higher quantile. The question is which quantile is adequate. To address this question for the storm climate case and the selected response example, certain quantiles are estimated for a number of sea states along the q-probability contours. Results are shown for $q = 10^{-1}$, $q = 10^{-2}$, $q = 10^{-3}$ and $q=10^{-4}$ in Tables 2- 5, respectively.

It is seen from Tables 2 and 3 that regarding an estimation of the 10^{-1} - and 10^{-2} – probability responses, the environmental contour approach in combination with the choice of the 90% quantile as the short term characteristic yield reasonable estimates. For most problems the relative importance of the short term variability increases with decreasing annual exceedance probability.

This is observed for the 10^{-1} - and 10^{-2} – probability estimates. The 90% value is slightly conservative for the 10^{-1} – probability value, while it is slightly non-conservative for the 10^{-2} – probability value. This tendency is strengthen as we consider predictions of more rare extremes. For the 10^{-3} – probability value, a quantile close to 95% is needed to obtain an adequate estimate, while for the 10^{-4} – probability value the an adequate quantile is slightly above 97.5%.

Some few observations of interest can be made from this idealized example. It is clearly demonstrated that if a response quantity corresponding to a given exceedance probability is to be adequately estimated, the short variability of the response needs to be accounted for in addition to the slowly varying weather variability. If the median of the q-probability sea state is selected, a considerable under-prediction is the case. The magnitude of the under-prediction is problem dependent, but for a non linear problem similar to this example, the under-prediction may be from 20% for the 10^{-1} – probability value to about 40% for the 10^{-4} – probability response. If the median response of the worst sea state along the 10^{-4} – probability contour line is adopted as the accidental environmental load, the annual exceedance probability for this load is about 0.0053, i.e. a number far above the typical accidental load occurrence probability level of 10^{-4}.

One should also note the observed tendency of a higher required quantile if target extremes corresponding to a decreasing annual exceedance probability are to be accurately estimated.

Finally, it should be pointed out that further work is recommended for practical response applications. It should also be further considered whether the contour lines should be established from a joint model reflecting all observations above the storm threshold and not merely the storm peak. Good quality data series including a number of rather severe storms are required if reliable probabilistic models shall be obtained. The present study lacks full consistence since the contour lines are established from storm peak data, while a truncated version of an existing joint model is adopted for the full long term analysis. In the future a consistent joint model of all sea states above threshold should be established. This will remove some uncertainties related to the contour line estimates and the estimates obtained using a full long term analysis.

6 CONCLUSIONS

Reasons for using the accidental load principle also with respect to natural loads are discussed. It is suggested that accidental environmental loads should be taken as loads corresponding to an annual exceedance probability of 10^{-4}, which is the requirement baked into the Norwegian Rules and Regulations. It is indicated that provided the load – annual exceedance probability relation does not show any abrupt changes in a worsening sense for annual probabilities between 10^{-4} and 10^{-2}, the accidental environmental loads will not effect the structural design. Introducing a requirement that the structure shall withstand environmental loads corresponding to an annual exceedance probability of 10^{-4} will ensure a certain robustness against unforeseen large environmental loads, e.g. due an unexpected severe storm event, an unexpected large wave event in a storm, or a larger reservoir subsidence than foreseen at the design stage.

Typical scenarios where abrupt changes can be expected are cases where wave–deck impacts or freeboard exceedances can take place for low annual probabilities.

In order to predict loads and load effects corresponding to an annual exceedance probability, a full long term analysis is in principle required. In connection with accidental load scenarios this can be complicated since extensive model tests may be required in order to reveal the short term structure of the response. It is suggested that the environmental contour line principle can be used as an approximate method. The adequacy of this approach is indicated for a generic response problem exposed to a peak over threshold type of wave climate. Establishing q-probability contour lines for the storm peak significant wave height and spectral peak period, adequate estimates for the long term extremes can be obtained by selecting a high quantile value for the 3-hour extreme value distributions. For accidental loads (10^{-4} – probability loads) a quantile around 97.5% seems to be adequate for the example case. The results clearly demonstrates that if the median 3-hour extreme value of the worst sea state along the 10^{-4} – probability contour line is selected as the accidental load, the annual probability of exceeding this load is more than an order of magnitude higher than the target accidental occurrence probabilities of the Norwegian Rules and Regulations.

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Table 2 Various quantiles for the worst range of the 0.1- probability contour line

0.1-probability contour sea state		**Selected quantiles (%)**				
h (m)	**t (s)**	**50**	**85**	**90**	**95**	**97.5**
9.82	11.00	143.4	170.9	179.2	192.8	206.3
10.05	11.25	151.2	180.4	189.1	203.6	217.8
10.35	11.60	160.3	191.4	200.6	216.0	231.2
10.67	12.02	167.0	199.5	209.2	225.4	241.2
10.95	**12.42**	169.2	202.2	**212.1**	228.5	244.6
11.31	13.02	165.2	197.7	207.5	223.6	239.4
11.62	13.68	153.9	184.4	193.5	208.6	223.5
Full long term analysis		**209**				

Table 3 Various quantiles for the worst range of the 0.01- probability contour line

0.01 – probability contour sea state		**Selected quantiles (%)**				
h (m)	**t (m)**	**50**	**85**	**90**	**95**	**97.5**
10.95	10.94	177.4	211.5	221.7	238.6	255.2
11.36	11.41	193.6	231.0	242.2	260.7	279.0
11.80	11.95	205.2	245.1	257.1	276.9	296.3
12.15	**12.41**	208.6	249.3	**261.5**	281.7	301.5
12.51	12.93	205.5	245.8	257.9	277.9	297.6
12.89	13.53	195.0	233.5	245.1	264.2	282.9
Full long term analysis		**266**				

Table 4 Various quantiles for the worst range of the 0.001- probability contour line

0.001 – probability contour sea state		**Selected quantiles (%)**				
h (m)	**t (m)**	**50**	**85**	**90**	**95**	**97.5**
12.02	11.02	215.0	256.4	268.7	289.3	309.4
12.47	11.52	233.1	278.2	291.6	314.0	336.0
12.94	12.08	244.3	291.8	306.0	329.6	352.8
13.26	**12.49**	246.1	294.2	308.6	**332.5**	355.9
13.59	12.93	242.5	290.2	304.4	328.1	351.3
14.12	13.67	227.4	272.5	285.9	308.3	330.2
Full long term analysis		**327**				

Table 5 Various quantiles for the worst range of the 0.0001- probability contour line

0.0001 – probability contour sea state		**Selected quantiles (%)**				
h (m)	**t (m)**	**50**	**85**	**90**	**95**	**97.5**
12.75	10.84	239.4	285.3	299.0	321.8	325.1
13.12	11.24	257.9	307.7	322.5	347.2	352.0
13.70	11.89	277.8	331.7	347.9	374.7	381.9
14.11	**12.37**	282.6	337.8	354.3	381.7	**390.5**
14.53	12.88	279.1	334.0	350.3	377.5	387.9
14.96	13.43	267.7	320.6	336.4	362.6	374.2
17.06	17.89	206.5	249.0	261.6	282.7	303.4
Full long term analysis		**393**				