FATIGUE DESIGN OF SUPPORT STRUCTURES FOR OFFSHORE WIND ENERGY CONVERTERS

Prof. Dr.-Ing. Peter Schaumann; Dipl.-Ing. Patric Kleineidam
Institute for Steel Construction
University of Hannover
Appelstrasse 9A; 30167 Hannover; Germany
Tel: +49-511-762-3781; Fax: +49-511-762-2991
schaumann@stahl.uni-hannover.de; www.stahlbau.uni-hannover.de

Global WINDPOWER 2004, Chicago, Illinois, USA

SESSION: WIND TURBINE LOADING, CONDITIONS, PREDICTIONS AND MEASUREMENTS

TUESDAY, MARCH 30, 2004 FROM 3:30 PM - 5:00 PM

ABSTRACT

Support structures of Offshore Wind Energy Converters (OWECs) are exposed to combined loading from wind and waves. Therefore, to gain adequate design methods for OWECs the established design concepts for offshore platforms and onshore wind energy structures have to be improved. Traditional concepts for fatigue design vary from deterministic design methods, which can be used in special conditions for oil and gas platforms, to calculations in the time domain, which are applied to the actual generation of multi megawatt onshore wind energy converters.

For tripod structures the structural stress approach is likely to be adapted for fatigue assessment of tubular joints. This method requires the determination of stress concentration factors. Different analytical methods for determining the stress concentration factors have been compared to numerical methods.

Because of the great differences in the amount of computing time, it is important for the designing process to evaluate the quality of the fatigue damage prediction using different design methods. The investigations are focussed on wave loading. Results of comparative calculations in the frequency domain and with deterministic single waves show the influence of site specific environmental conditions. The fatigue damage is predicted using Dirliks-formula in the frequency domain.

KEYWORDS:

Offshore wind energy, support structure, fatigue, stress concentration, frequency domain

1 Introduction

Support structures of Offshore Wind Energy Converters (OWECs) are exposed to combined loading from wind and waves. Therefore, to gain adequate design methods for OWECs the established design concepts for offshore platforms and onshore wind energy structures have to be improved. Traditional concepts for the fatigue design vary from deterministic design methods, often used for oil and gas platforms, to calculations in the time domain which are applied to the actual generation of multi megawatt onshore wind energy converters. Currently available design tools for onshore wind energy converters like FLEX5 or Bladed have been extended to take into account offshore environment. But they are applicable to complex support structures like tripods, see [2] and [9]. For water depth of 30 m and more such complex types of foundations are expected. Since the available design tools do not provide an integrated design, suitable combinations of design tools for aerodynamic loading and wave loading have to be applied. The investigations presented here are focussed on wave loading. The purpose is to evaluate different methods for fatigue assessment of offshore structures considering the special structural and environmental conditions of OWECs.

For monopile structures the nominal stress approach may be used to determine the calculated damage values for the tower. More sophisticated analysis is necessary e.g. in ring flange joints. For tubular joints of tripod structures the nominal stress approach is not applicable. An alternative method is the structural stress approach which can be carried out using parametric formulas or numerical methods.

Based on an appropriate method for the detailed design different methods for the fatigue assessment of offshore structures are compared. Because of the great differences in computing time, it is interesting to evaluate the accuracy of the fatigue damage prediction using different design methods. Results of comparative calculations in the frequency domain and with deterministic single waves illustrate the influence of site-specific environmental conditions. The fatigue damage is predicted in the frequency domain using Dirliks-formula. The results are compared with respect to the environmental conditions in North and Baltic Seas.

The investigations presented here were part of the research program GIGAWIND at the University of Hannover, see http://www.gigawind.de for more information.

2 Fatigue Analysis of Complex Tubular Joints using the Structural Stress Approach

The fatigue assessment of tubular joints has been subject of different research projects in the 1980ies. The research was based mainly on the requirements of the offshore industry. Additional investigations concerning bridge building design were carried out recently, see e.g. [22]. The nominal stress approach with different detail categories and corresponding S-N-curves is not applicable for tubular joints. State-of-the-art for such joints is the use of structural stress approach for design calculations. More comprehensive local approaches like the “notch stress and strain

approach" and the "crack propagation approach" are reasonable for comparative tasks but the use for design calculations is limited because the local parameter data for most approaches lack statistical proof, see [16].

Different from the nominal stress approach the structural stress approach takes into account local effects. This method emphasises the stress concentrations caused by the macrogeometry, see [16]. For this reason it is no longer necessary to provide a special S-N-curve for every joint geometry, but a representative curve which includes the effect of the weld geometry, see [15]. To get the structural stress, also called hot-spot-stress, the nominal stress is combined with the stress concentration factor within the design process, see equation 1.

$$\sigma_{\text{hotspot}} = \text{SCF} \cdot \sigma_{\text{nom}} \qquad (1)$$

The stress concentration factors differ from joint to joint and can be determined by experiments, numerical investigations or parametric equations.

2.1 Parametric Equations for SCFs

Based on a huge number of experiments and numerical investigations which have been carried out in the past different parametric equations have been developed to predict the stress concentration factor of tubular joints. With these formulas the structural stress approach is easily accessible for practical design tasks. The formulas depend on dimensionless parameters (α, β, γ, τ and θ) characterising the behavior of the joint, see Fig. 1 for explanations.

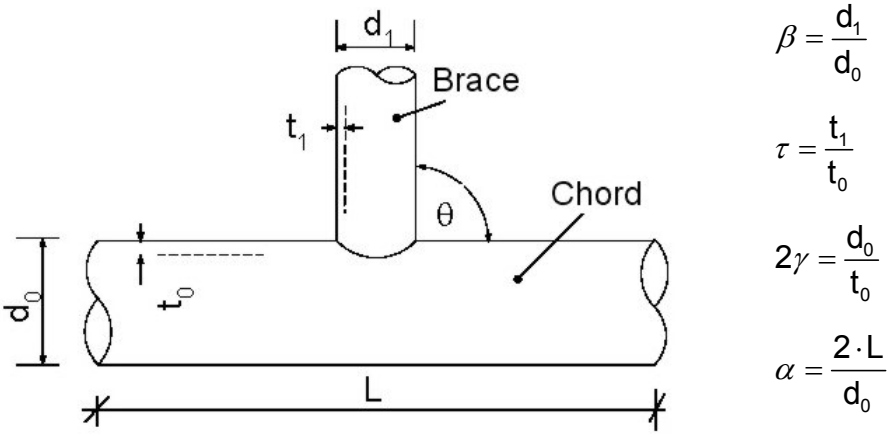

Fig. 1: Y-joint with dimensionless parameters

In [12] an extensive comparison is documented. It shows that the formulas of Efthymiou [5] and Lloyd's Register [12] correlate well with experimental and numerical results. Additionally the Norwegian guideline N-004 [14] recommends the application of Efthymiou's formulas. The use of Efthymiou's approach is emphasized by the fact, that he had provided a formalistic possibility to cover even complex joints by the superposition of stresses resulting of different braces. The method is based on influence functions. The resulting stresses do not only depend on the geometry of

the joint but also on the relation between the stresses in the braces. Therefore the resulting SCFs have to be determined for every single step of the calculation, as it has been shown exemplarily by the authors in [20]. This approach was implemented into the Software Tool *Han-Off* which has been developed at the Institute for Steel Construction and is capable of doing structural calculations for offshore structures. Han-Off uses the FE-program ANSYS® for the structural calculations.

2.2 Numerical Studies

A parameter study was done, to evaluate the different methods of determining SCF-values. For simple T- and Y-joints the parametric formulas of Efthymiou and Lloyd's Register were analyzed. Additionally numerical calculations with the FE-program ANSYS® were performed. The FE-model used for these comparisons were validated with respect to experimental results which were taken from [13]. The validation of the FE-model is described in [20] in more detail. The results of the calculations are displayed in Fig. 2 for the chord saddle location and axial forces in the brace. The correlation between the numerical results and the parametric formulas is good, even if there is some scatter between the investigated parametric formulas and the numerical calculations. For practical cases, where SCFs are expected to be lower than 15 the numerical results are nearly in all cases smaller than the results of the parametric formulas. The conclusion is that the use of the parametric formulations is reasonable and that the effort of the numerical calculations can pay out, especially for the serial production of offshore wind energy converters for large wind farms. For complex joints which are part of the discussed structures like tripods or jackets the combined loads can be assessed by the method described by Efthymiou in [5].

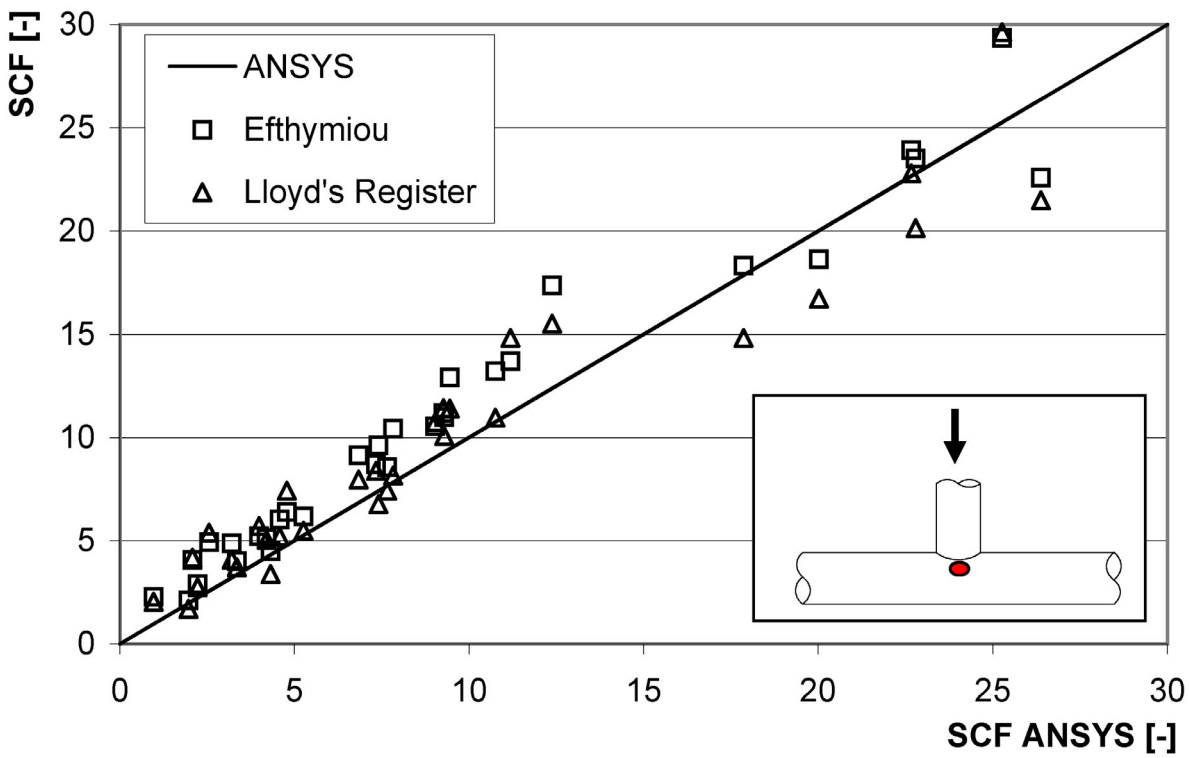

Fig. 2: Analytic stress concentration factors compared to numerical results, chord-saddle location of Y-joints under axial loading

3 Fatigue Design Approaches for Offshore Structures

For the fatigue assessment of offshore structures under wave loadings several approaches exist. The approaches differ with respect to the required numerical effort. The usefulness of each approach depends on the structural dynamic properties against the long term description of the sea environment. This is deduced by comparison of various calculations. For the described examples the calculations were carried out using the program Han-Off which was developed at the Institute for Steel Construction of the University of Hannover within the last years, see e.g. [20] for more details.

3.1 Deterministic Approach

The deterministic method used in this paper is based on the descriptions of Hapel [8]. It is furthermore applied in commercial engineering program systems for offshore structures like for example STAAD.Pro with the Offshore Loading Modul, see [18] and [17]. The first part of this method is the determination of the wave height exceedance diagram from the site-specific wave scatter diagram. In the next step the stress wave height diagram has to be identified under consideration of the dynamic structural properties. In the last step the fatigue analysis is carried out using the results of the first two steps, determining the number of waves within several classes. The corresponding stress range for every class is derived from the stress wave height diagram. With this information the damage for every wave height class is calculated with Miner's rule. The usage of this method for offshore wind energy converters has been described by the authors in [19].

3.2 Simulations in Time Domain

For simulations in time domain a sufficient number of sea states of the site specific scatter diagram must be evaluated. The wave scatter diagram contains information about the occurrence frequency of sea states. The sea states are in most cases characterised by the significant wave height and the zero crossing period. The statistical properties of a sea state are determined using standard wave spectra like the JONSWAP or PIERSSON-MOSKOWIETZ spectrum $S_{\zeta\zeta}(f)$. A real sea state can be described by the superimposition of a large number of regular waves with a random phase shift:

$$\eta(t) = \sum a_i \cdot \cos(\omega_i \cdot t + \alpha_i) \qquad (2)$$

where a is the amplitude of partial wave i, ω its circular frequency and α_i an equally distributed random phase angle between 0 and 2π.

A time series of sea state dependent water waves with corresponding wave loads is obtained, simulating this superimposition of partial waves. For more information about the calculation of wave loadings for simulated random sea states the authors refer to [19]. The structure is analysed under this time series of wave loading by a transient analysis in time domain. This calculation results in a time series of structural stresses. The rainflow-counting method as described in [3] is used for the

fatigue evaluation of this time series of stresses. The duration of the simulation has to fulfil minimum criteria to reduce the possible scatter in calculated damage values as it is shown in [21].

3.3 Calculations in Frequency Domain

As the description of sea state is done by the wave spectra in frequency domain it would be promising to get the structural response also in frequency domain. In that case two problems are of great interest. Firstly it must be clarified how the structural response can be derived and secondly, a damage evaluation has to be done for the stress spectra in the frequency domain.

Stating that the structure is the connection between the stochastic wave process and the stochastic stress process in the structure as a consequence of wave loading, every detail of the structure can be represented by a transfer-function in frequency domain. For simple structures like monopiles analytical solutions can be found in [8]. For deepwater offshore platforms Kan has presented a so called “hybrid time-frequency domain fatigue analysis”, see [10]. The concept contains the simulation of representative time series of wave loadings and furthermore the calculation of the structural response in time domain under consideration of nonlinear effects. For this representative time series of the stresses the according transfer function can be calculated using equation 2. For this the time series of the stresses must be transferred into the frequency domain using e.g. the fast fourier transformation.

$$H(f) = \sqrt{\frac{S_{\sigma\sigma}(f)}{S_{\zeta\zeta}(f)}} \qquad (3)$$

3.3.1 Damage Evaluation in Frequency Domain

While it is state-of-the-art to evaluate time series of stresses using the rainflow counting method, for the evaluation of stress spectra other techniques must be applied. Bishop has compared different possibilities in [1]. His conclusion is that the formula of Dirlik, see [4], leads to the best results. In accordance with [11] this method is used for the damage evaluation of stress spectra described in this paper. The method is based upon the following principles:

If the probability of distinct classes of stress ranges is known, which are expected in a certain period of time, it is possible to evaluate the underlying process using the Miner’s rule. S-N-curves describe the theoretical number of stress cycles before failure occurs for a distinct stress range $\Delta\sigma$. Here the indications are taken from Eurocode 3.

$$N = \frac{a}{\Delta\sigma^{m}} \qquad (4)$$

The stress ranges are derived from the stress peaks, considering processes with a mean value of zero, see equation 5. If the fatigue assessment is independent of the

median, as it is supposed for the described concept, any stationary stress processes can be modified in a way that their mean value is zero.

$$\Delta\sigma_i = 2 \cdot s_i \qquad (5)$$

The number of stress cycles in a certain time period T is derived from the expected number of stress peaks per unit time E[P]. This value can be calculated from the moments of the stress spectrum.

$$n = E[P] \cdot T = \sqrt{\frac{m_4}{m_2}} \cdot T \qquad (6)$$

The moments of the stress spectra can be calculated using the following equation:

$$m_n = \int_{-\infty}^{\infty} \omega^n \cdot S_{uu}(\omega) d\omega \qquad (7)$$

The expected damage in this time period is derived from these values.

$$E[D] = \frac{n}{N} = \sqrt{\frac{m_4}{m_2}} \cdot T \cdot \frac{\Delta\sigma^m}{a} \qquad (8)$$

Normally an equivalent stress range is used in equation 8 while a constant slope is considered for the S-N-curve. This equivalent stress range should reflect the properties of the stress spectrum.

$$\Delta\sigma_{equi}{}^m = \int_0^{\infty} \left(\Delta\sigma^m \cdot p(\Delta\sigma) \right) d\Delta\sigma \qquad (9)$$

For practical cases the use of S-N-curves with changing slopes is required by certain design codes like Eurocode 3 [22] or GL [7] and must therefore be taken into account for support structures of OWECs. An equivalent number of allowable stress cycles is introduced here instead of an equivalent stress range to integrate such S-N-curves in this concept. In equation 10 the reciprocal of this value is shown.

$$\frac{1}{N_{equi}} = \int_0^{\infty} \left(\frac{\Delta\sigma^{m(\Delta\sigma)}}{a(\Delta\sigma)} \cdot p(\Delta\sigma) \right) d\Delta\sigma \qquad (10)$$

For the integration in equation 10 the probability density function of the stress ranges has to be known. This function can be derived using the above mentioned formula of Dirlik, equation 11, which is based on a huge number of comparative numerical calculations.

$$p(\Delta\sigma) = \frac{\frac{D_1}{Q} \cdot e^{\frac{-Z}{Q}} + \frac{D_2 \cdot Z}{R^2} \cdot e^{\frac{-Z^2}{2 \cdot R^2}} + D_3 \cdot Z \cdot e^{\frac{-Z^2}{2}}}{2 \cdot (m_0)^{0.5}} \qquad (11)$$

with

$$Z = \frac{\Delta\sigma}{2 \cdot (m_0)^{0.5}} \qquad x_m = \frac{m_1}{m_0} \cdot \left[\frac{m_2}{m_4}\right]^{0.5}$$

$$R = \frac{\alpha - x_m - D_1^2}{1 - \alpha - D_1 + D_1^2} \qquad D_1 = \frac{2 \cdot (x_m - \alpha^2)}{1 + \alpha^2}$$

$$D_2 = \frac{1 - \alpha - D_1 + D_1^2}{1 - R} \qquad D_3 = 1 - D_1 - D_2$$

$$Q = \frac{1.25 \cdot (\alpha - D_3 - (D_2 \cdot R))}{D_1} \qquad \alpha = \frac{m_2}{\sqrt{m_0 \cdot m_4}}$$

3.3.2 Example calculation

The evaluation in frequency domain is compared for an example monopile to the rainflow counting method. For the structure shown in Fig. 6 a time series of wave loading were simulated with the duration of 2200 s, while the loadings were calculated with time steps of 0.25 s. The sea state is considered with a JONSWAP spectrum, with the parameters H_S = 1.5 m and T_Z = 5.5 s. The structural response was calculated with a time step size of 0.125 s. The collective of the stress ranges which results from the rainflow counting method, see Fig. 3, is transferred to the probability density of stress ranges for a direct comparison with the results of Dirlik's formula.

Additionally the time series of the stresses was shifted in the frequency domain by using the fast fourier transformation. The wave spectrum, the transfer function, and the stress spectrum can be found for this example in Fig. 4. The probability density function of the stress ranges can be calculated using Dirlik's formula, after determining the moments of the stress spectrum. The probability density functions for the rainflow counting method and Dirlik's formula are displayed in Fig. 5. They show a very small scatter. The damage evaluation leads to the following values.

$$D_{\text{Rainflow, 1 year}} = 7.081 \cdot 10^{-6}$$

$$D_{\text{Dirlik, 1 year}} = 6.522 \cdot 10^{-6}$$

They are calculated for a time period of 1 year under consideration of a detail category with a nominal value of 125 N/mm² at 2,000,000 stress cycles. The difference between the two fatigue evaluation methods is small.

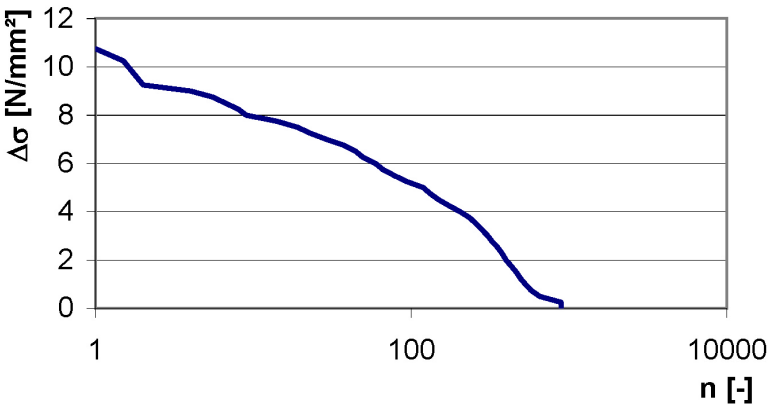

Fig. 3: Totalized collective of stress ranges

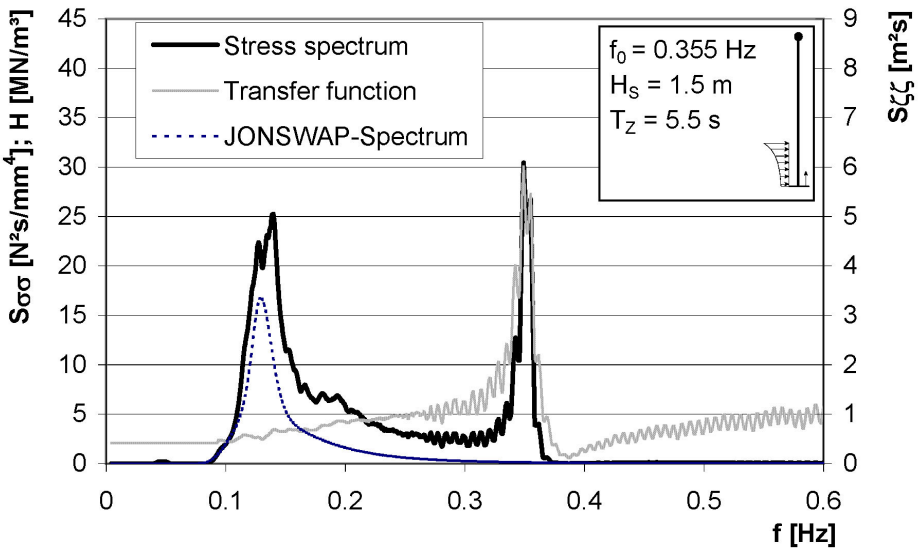

Fig. 4: Stress spectrum, transfer function and JONSWAP spectrum for example calculation

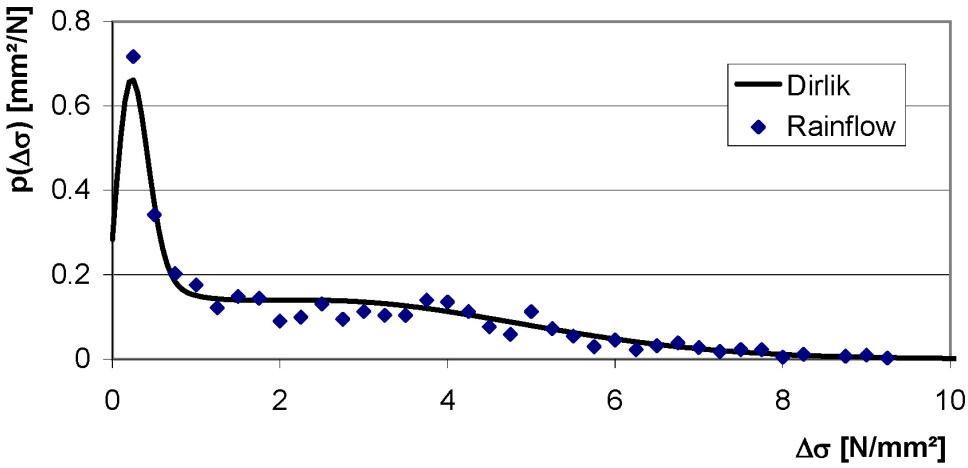

Fig. 5: Probability density for stress ranges, calculated with Dirlik's formula and from collective resulted from rainflow-counting

3.4 Comparison of deterministic and frequency domain approach

The above described methods for the fatigue assessment of offshore structures were investigated by the authors in various calculations which are described in more detail in [19] and [21]. In this paper typical results are presented for the structural model displayed in Fig. 6. The transfer function which was numerically derived can be seen in Fig. 7. In the presented calculations aerodynamic damping was not included. This is of no influence on the general results. The expected damage has been calculated, using the above described method in the frequency domain and considering all sea states of the wave scatter diagrams which are displayed in Fig. 8. The left scatter diagram is representative for North Sea environments while the right one describes an exemplary environment in the Baltic Sea. It is remarkable that the probability of sea states with zero crossing periods in the order of the natural period of the structure (2.8 s) is very low for the North Sea environment, while for the Baltic Sea environment many sea states have to be expected within this range.

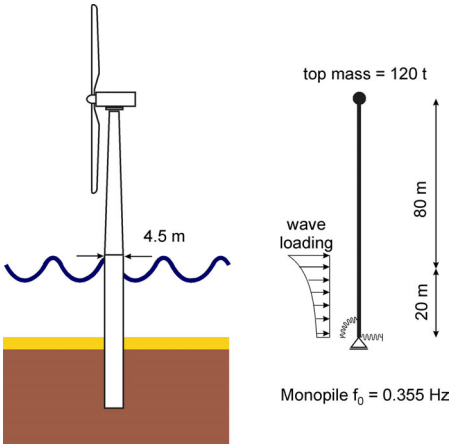

Fig. 6: Structural model for calculations

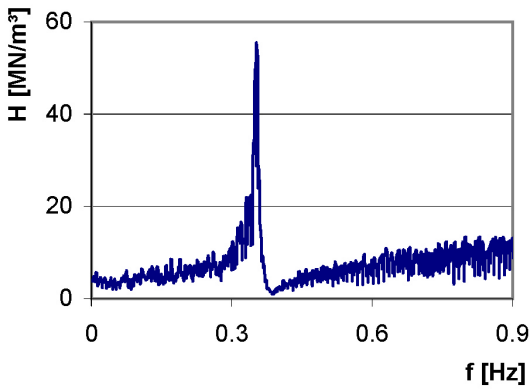

Fig. 7: Transfer function for fatigue assessment

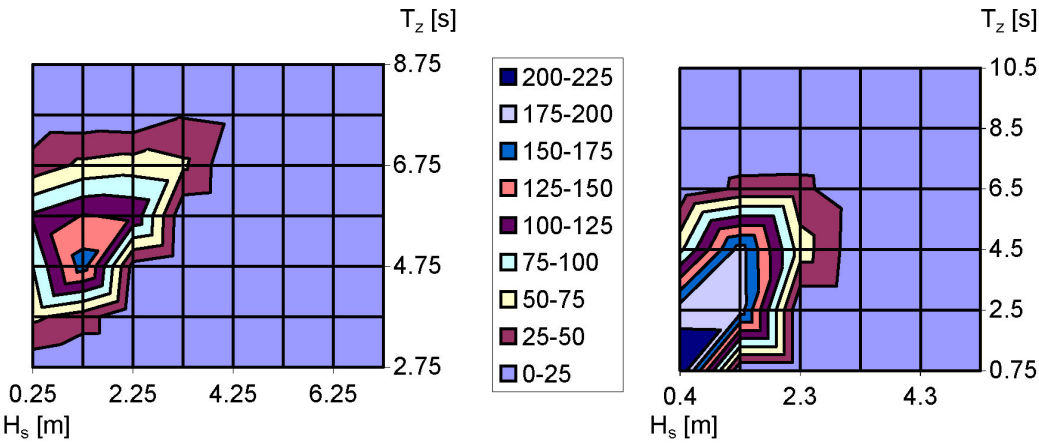

Fig. 8: Wave scatter diagrams used for calculations, probability in part per thousand, related to class range of wave height respectively wave period, denomination:
Scatter North Sea[1] *Scatter Baltic Sea[2]*

[1] The data are based on simulations carried out by the Institute for Fluid Mechanics, University of Hannover.

[2] The data have been provided to the authors within a development project by OTP GmbH MV, Source: DWD.

Damage evaluation method	Deterministic approach	Frequency domain approach
North Sea	1.00	1.71
Baltic Sea	46.64	1.74

Table 1: Relative damage for different environmental conditions and approaches

In Table 1 the results of the damage evaluation in frequency domain are compared to the deterministic approach. The enormous difference between North and Baltic Sea using the deterministic approach has not been found for the calculations in the frequency domain. The huge difference for the deterministic approach is caused by high values of the dynamic amplification factors for small waves with high numbers of occurrence, see [20] for more details. Although the resulting damage is very similar for both situations using the frequency domain approach, there are some significant differences. In Figures 9 and 10 the relative damage of the considered sea states is cumulated in descending order of the damage ratio. That means that the highest damage values per probability of sea state occurrence are considered firstly. Additionally the corresponding zero crossing periods of the sea states are displayed. For North Sea environment 80% of the damage is caused by 20% of the sea states while for the Baltic Sea scatter diagram 80% of the damage are caused by only approx. 15% of the sea states. The highest damage ratios are combined for North Sea environment with high values of corresponding zero crossing periods. This indicates that the dynamic behaviour of the structure plays no significant role for the damage calculation. The highest damage ratios for the Baltic Sea scatter diagram are combined with comparatively small values of corresponding zero crossing periods.

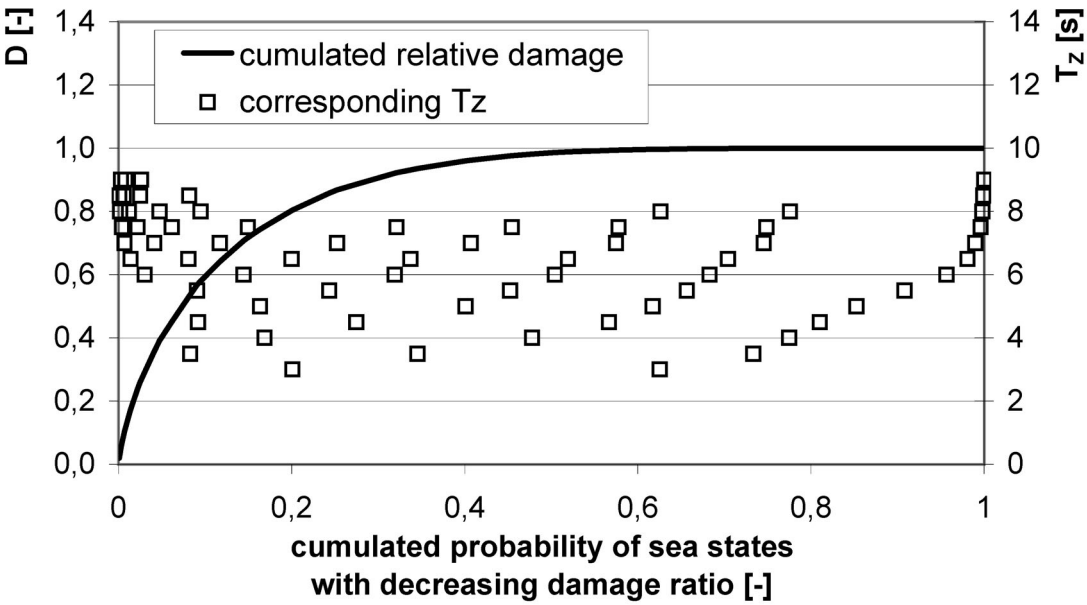

Fig. 9: Cumulated relative damage for North Sea environment in dependency of the cumulated probability of sea states with decreasing damage ratio with corresponding zero-crossing periods

The authors expect that the description of the dynamic behaviour is more realistic in the frequency domain analysis. Although the results of the deterministic approach were conservative in most cases, reasonable damage values are expected for the deterministic approach only, if the probability of sea states which have significant energy contents near to the first natural mode of the structure is small.

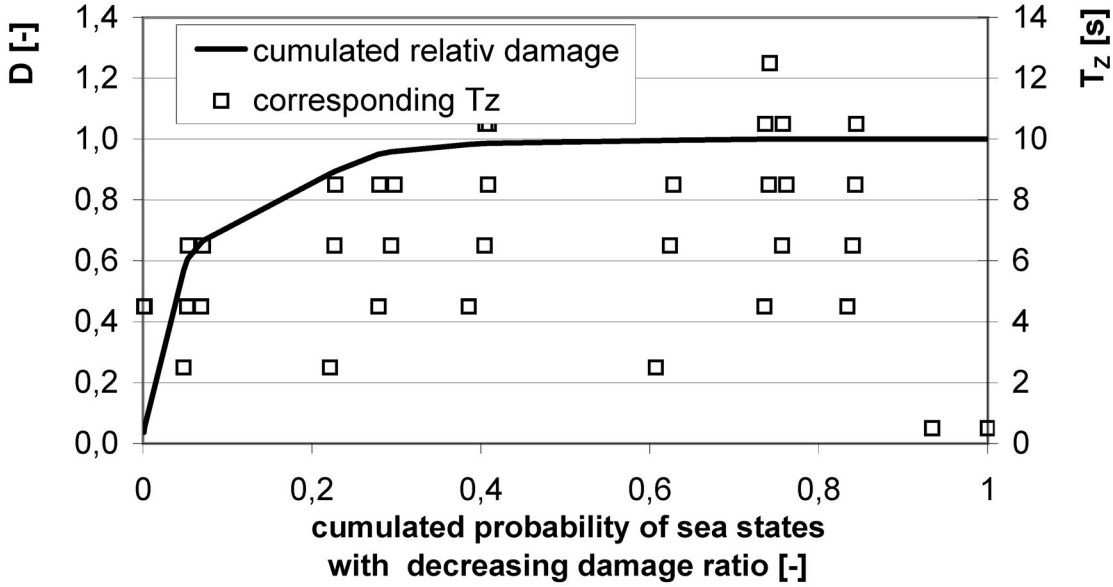

Fig. 10: Cumulated relative damage for Baltic Sea environment in dependency of the cumulated probability of sea states with decreasing damage ratio with corresponding zero-crossing periods

4 Conclusion

The damage evaluation of support structures of offshore wind energy converters is an important part of the design phase of offshore wind farms. For support structures in a water depth of 30 m or more the use of tripod structures is currently discussed. For the fatigue assessment of tubular joints which are part of tripod structures the use of the structural stress approach is state-of-the art. The determination of stress concentration factors is an essential part of this approach. A comparison between parametric formulas and FE-calculations showed that the use of numerical calculations can in certain cases reduce the calculated damage values.

For the fatigue assessment of offshore structures under wave loading different approaches are described and comparative calculations have been presented using the deterministic and the frequency domain approach for typical environmental conditions in the North Sea, respectively in the Baltic Sea. It has been shown that the deterministic approach should be used with caution, if there are a significant number of sea states with mean periods near to the natural periods of the structure. For example this is expected for typical support structures of OWECs in the Baltic Sea. The deterministic approach should be used with caution in those cases, since the presented examples proofed this approach to be very conservative. It is expected that the frequency domain approach gives a more reasonable description of the dynamic behaviour of the system.

Acknowledgement:
The research program GIGAWIND (www.gigawind.de) at the faculty for civil engineering at the University of Hannover has been funded from 2000-2003 by the Federal Ministry of Economics and Technology of Germany.

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