Enhanced Structural Design for Offshore Wind Turbines

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Abstract: Support structures of Offshore Wind Energy Conversion Systems (OWECS) are exposed to combined loading from wind and waves. Therefore, the fatigue damage evaluation is an important part in the design phase of offshore wind farms.
Different methods for calculation of actions on offshore support structures at sites in North and Baltic Sea are presented. Traditional concepts for fatigue design vary from deterministic methods, which have been adopted from experiences in the design of oil and gas platforms, to calculations in the time domain, which are applied to the actual generation of multi megawatt onshore wind energy converters. As results show, time domain approaches should be used preferably.
For braced structures the fatigue strength of tubular joints is determined by local approaches. Besides the structural stress approach, sophisticated concepts like the notch stress approach can be used for fatigue assessment. A concluding fracture mechanics analysis supports both material selection and risk management.

Keywords: Offshore, Wind Energy, Support Structure, Fatigue

1 Introduction

As a result of increasing fossil fuel prices and political promotion of renewable energy within the last decade wind energy industry in Europe has developed fast and is nowadays an important branch of economy. More than 40000 Megawatts (MW) of capacity have been installed in Europe by end of 2005 with yearly growth rates near 20%. The German market and industry plays a significant role, since half of the total capacity has been installed there. Hence, actual developments in Germany are somewhat representative for future trends in other countries.
As economically profitable locations for land-based wind farms are becoming rare, the new challenge for wind energy – besides the "re-powering" of old wind farms - lies offshore. But experiences with offshore wind energy are limited to quite a few European countries and locations with rather shallow water.

2 Design Methodology

2.1 General Design Approach

Offshore projects require initially higher investments than onshore projects. According to the actual distribution of capital investment costs a stake of 20-25% has to be spent for the support structures and their installation.
Furthermore, support structures of Offshore Wind Energy Conversion Systems (OWECS) are exposed to combined loading from wind and waves. With increasing rotor diameter and turbine mass the wind loads tend to dominate, while for small turbines the wave loads outweigh. In terms of fatigue assessment the turbulent wind and the unsteady sea state lead to high dynamic loads with a number of cycles beyond 10^9. Various calculations by the authors show that for structural parts below water level the fatigue assessment, the issue this contribution will focus on, often governs the design.
The interaction between structure, turbine and its control system is making the design of offshore wind energy converters an interdisciplinary task for both structural and mechanical engineers. Along with the financial aspect mentioned above the role of structural engineers becomes more important. Considering that the proposed structures differ from those traditionally used by oil and

gas industry with decades of design and operation experience, safe design and detailed risk management becomes indispensable. A complete iterative design process therefore requires

- a time domain approach for calculation of actions on structures (Chapter 2.3)
- detailed fatigue design validated by local approaches (Chapter 2.4)
- the definition of inspection intervals based on fracture mechanics (Chapter 2.5).

Some special issues to be considered for an enhanced structural design will be outlined in the next chapters.

2.2 Support Structures

Many of the design requirements are directly related to the characteristics of the support structures. Therefore the possible foundation concepts of OWECS must be outlined prior to dealing with questions of an enhanced structural design.
Future OWECS will be provided with turbines of a capacity between 3 and 5 MW. These turbines are mounted on steel tube towers, which is the standard onshore solution. Due to the better wind conditions with higher mean wind speeds and a lower surface roughness the hub heights can be reduced to 70 - 80 m. Concerning the wind farms planned in the so-called Exclusive Economic Zone of Germany in North and Baltic Sea most of the support structures will be located in regions with water depths between 20 and 50 m (see Schaumann & Böker 2005). For these water depths, different types of support structures are currently under discussion.

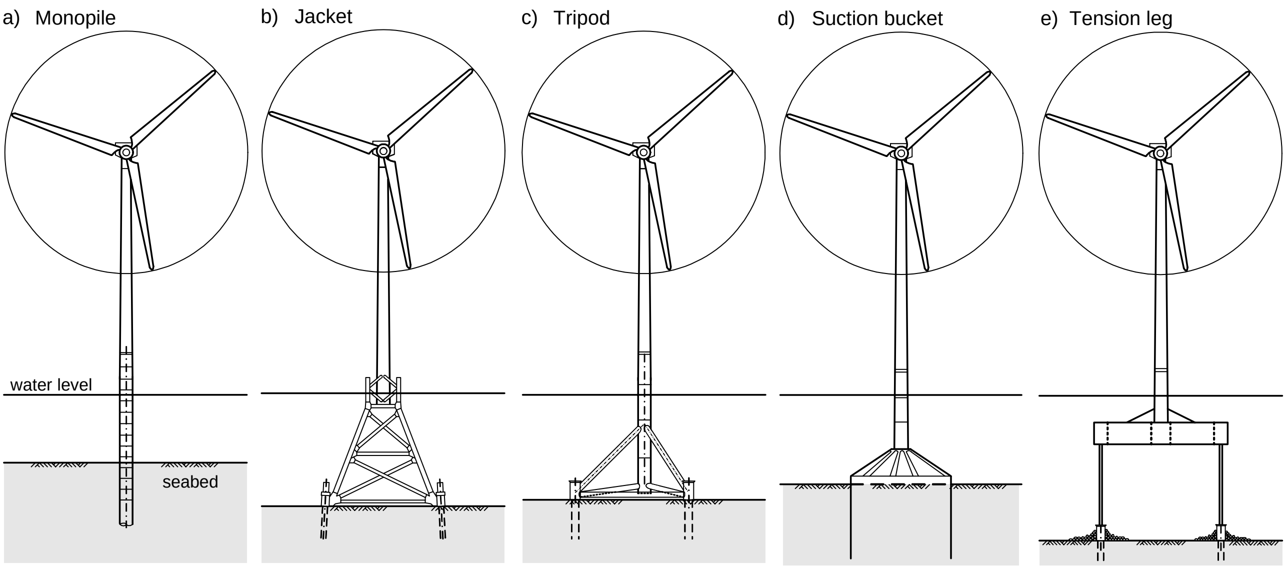


Figure 1: Concepts for the support structures of offshore wind energy converters acc. to (Schaumann et. al 2004)

The so-called monopiles (Figure 1a) are effectively an extension of the steel tower, driven or drilled into the seabed. They are used extensively in the off- and nearshore environment for supporting oil and gas platforms and other coastal structures. To some extent the monopile can be considered as the state of technical knowledge, being the preferred design concept for medium water depths as an actual ranking with comparison matrix has underlined (Schaumann & Böker 2005). Although relatively simple, the monopile structure includes a number of special details like the "grouted joint", a hybrid connection type which is currently under research at the Institute for Steel Construction (Schaumann & Wilke 2006a). Gravity-based foundations, not shown in Figure 1, are designed either as blocks or caissons with a flat base to resist the overturning forces. Caisson types, typically made of steel or concrete, can be ballasted with water, iron or various grouted materials. Both types have shown to be cost-effective only for shallow water.
For future projects with higher water depths other structures are proposed, mostly adopted from designs already used in other offshore sectors. Possible concepts are shown in Figure 1b-e. Braced towers can be realised as tripods or lattice towers like the jacket solution, both discussed by

Schaumann et al. (2005). The tripod supports a central tube which extends into the tower, with each corner of the tripod support piled into the seabed. The jacket can be any of a variety of arrangements whereby a central tube is surrounded by numerous piled supports. In case both monopile and tripod or jacket are technically possible it should be mentioned that alternatives to monopile foundations have to offer significant weight savings in order to be competitive. Thus the monopile should also be taken into consideration for higher water depths. Figure 2 shows a comparison of monopile and tripod masses. The pile weights required for fixing the tripod structure on the sea floor are not included, whereas the monopile masses in Figure 2 include both the sub-structure and the foundation. The required masses for the foundation piles of the tripods are typically in the range of 100 to 300 tons. Taking this into account, it can be seen that monopile and tripod designs yield comparable overall weights for sub-structure and foundation.

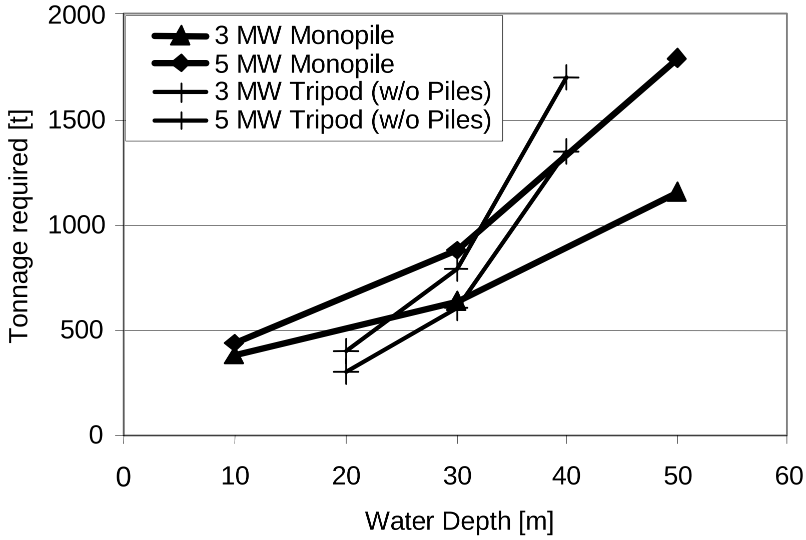


Figure 2: Water depth dependency of monopile and tripod weights for a 3 and 5 MW turbine (Schaumann & Böker 2005)

Suction-based foundations have also been proposed, replacing the part of the monopile driven into the seabed. An inverted 'bucket' forms the foundation to which suction is applied until it penetrates to the desired depth. The tension leg system is a submerged floater with tensioned vertical anchors. Advantage of this new concept is its simple installation – the structure can easily be towed to the site – and its applicability for a wide range of water depths.
An approximate classification of the different types of support structures regarding the water depth is done by Schaumann et al. (2004). Nevertheless, it should be pointed out that there are other parameters with great impact on design and optimisation of the support structures, as Figure 3 shows, with the three ones in shaded grey being the major design drivers.

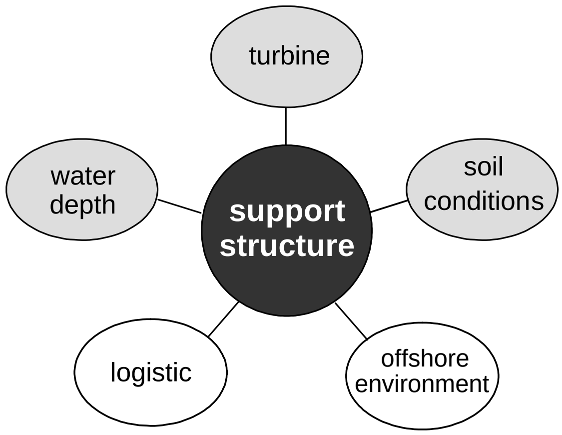


Figure 3: Design drivers for OWEC's support structures

Based on experiences with the already completed wind farms and preliminary design calculations done by the authors, the required amount of steel for an entire structure (foundation and tower) is in the region of 1000 tons (see also Figure 2). Considering only the future German plans with predicted 8000 OWECS to be installed until 2030 it will lead to a noteworthy steel consumption by offshore wind energy industry.

2.3 Calculation of Actions on OWECS

Actions for the fatigue assessment of offshore structures subjected to wave loading may be calculated by different approaches (Figure 4). The deterministic approach, widespread in offshore industry, uses a discrete wave analysis in combination with site specific wave height exceedance diagrams. Inaccuracies are mainly originating from the definition of the relationship of wave height and period and the simplification of dynamic effects assuming a quasi-harmonic excitation. Time domain simulations in contrast use a number of sea state dependent time series which are used as input for time history calculation. The third method calculates the structural response in the frequency domain, self-evident as wave spectra are naturally described in frequency domain, leading to the problem that damage evaluations also have to been done in the frequency domain. Connection of time and frequency domain results in a so called hybrid approach, which combines the accuracies of the time history analysis with the savings in computing time of frequency based calculations. A detailed description of the different approaches is given by Kleineidam (2004).

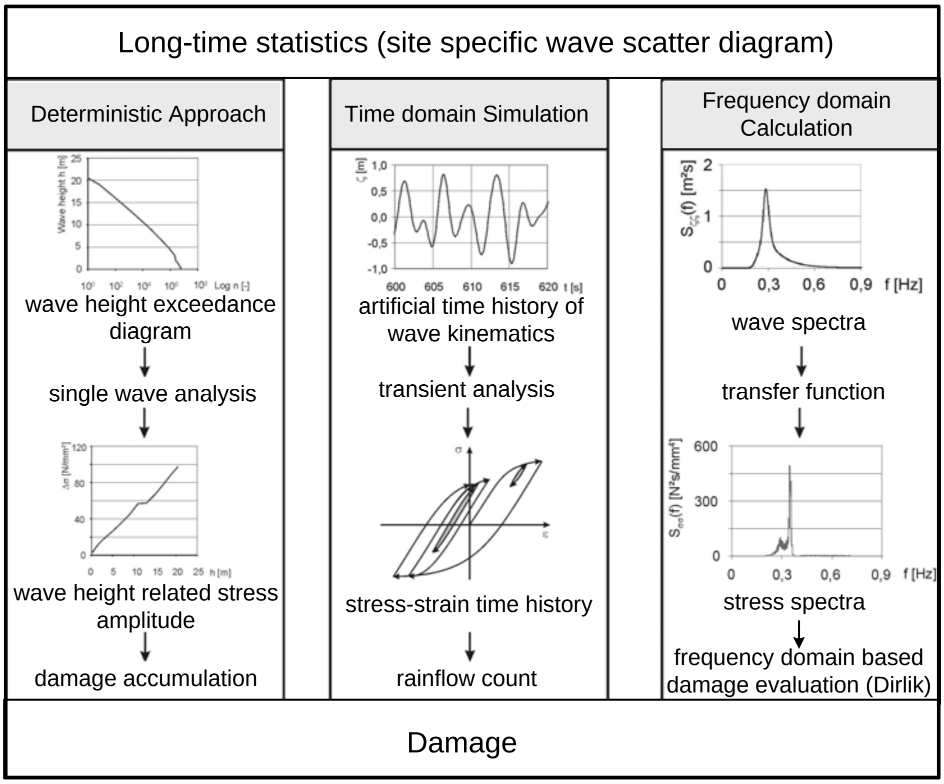


Figure 4: Fatigue design approaches acc. to Kleineidam (2004)

The different approaches have been compared for monopile systems, using identical numerical models and water depths. Wave load calculations are based on linear wave theory. The generation of the wave load time series has been done using the PIERSON-MOSKOWITZ-spectrum. Two different sites have been considered with different long time wave statistics as shown in Figure 5. For Baltic Sea conditions it should be noted that the peak of probability lies within the range of the first eigenfrequency of the structure (which usually is adjusted between 0.3 and 0.4 Hz for three-bladed turbines).

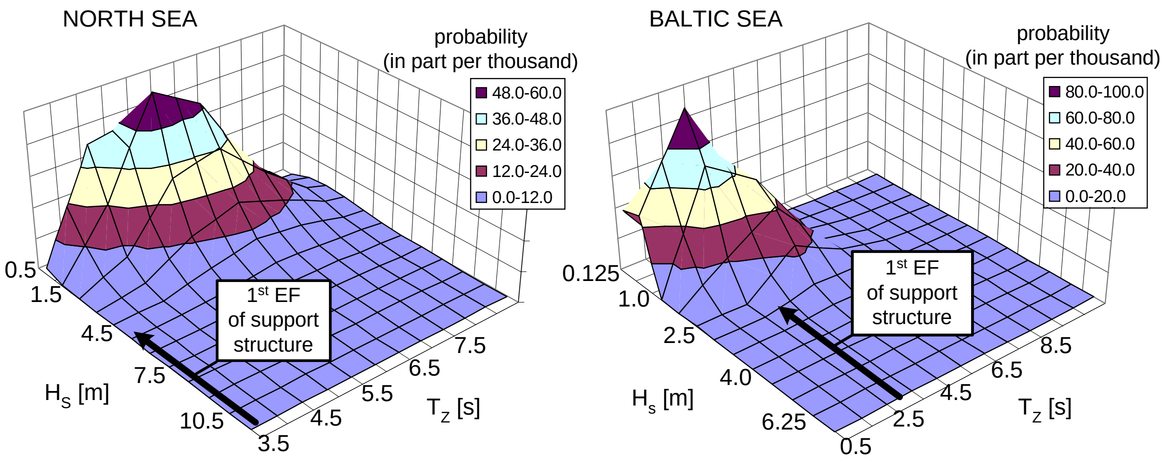
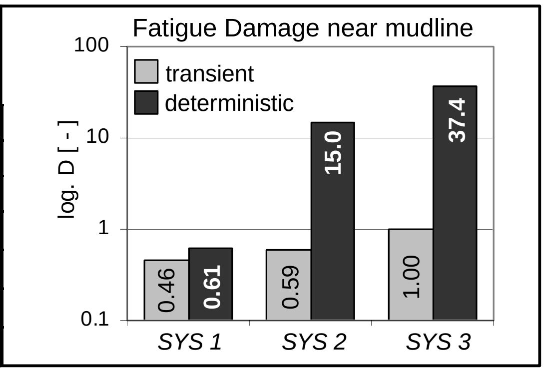


Figure 5: Wave scatter diagrams for two different German locations

The main features of the monopile support structures are shown in Table 1. The towers are equipped with a small turbine, thus wave loads outweigh the fatigue assessment. Calculation of fatigue damage from sea state both for time domain and deterministic approach shows two characteristics. According to the underlying wave scatter diagrams the fatigue damage in North Sea condition is expected to be smaller, as only few waves are in the range of the eigenfrequency of the structure. Both approaches lead to almost identical damages. For the Baltic Sea locations the deterministic approach gives much higher damages compared to the transient analysis. Here the simplified calculation of the dynamic response has a great influence, which is confirmed by the enormous increase in damage caused by the small shift in eigenfrequency between System 2 and 3. If there is a significant number of sea states with mean periods near the natural period of the structure, the deterministic approach is too conservative and should be used with caution.

Table 1: Features and results of the analysed monopile structures

Monopile Support Structure			
water depth = 25 m			
hub height = 70 m			
m_{top} = 100 t			
-	Location	Soil type	1st Eigenfreq.
SYS 1	North Sea	non cohesive	0.36 Hz
SYS 2	Baltic Sea	non cohesive	0.36 Hz
SYS 3	Baltic Sea	cohesive	0.31 Hz



Simplified approaches for combination of wind and waves exist, but integrated models are effectively state-of-the-art. As long as transient effects from turbine control as well as other nonlinearities (e.g. due to soil-structure-interaction) have to be considered in a detail design, there is no alternative to the time-domain approach. This leads to requirements which are - in terms of classic civil engineering - only known from earthquake or flutter analysis of large span bridges: 12-14 artificial wind time histories with a duration of minimum 600 sec have to be combined with the corresponding sea states from the scatter diagram. Additionally the different wave directions have to be considered, usually in steps of 10° over a sector of 180°. If spatial wave fields are introduced, as shown by Schaumann et al. (2004), a too penalising design can be avoided. Multiplied with the number of time histories to be considered, apparently very effective design tools have to be used, developed over the last years (Schaumann & Wilke 2006a). Integrated simulation software, which is capable of including complex structures like tripods, is rare and its development is part of actual research.

2.4 Fatigue Assessment by Local Concepts

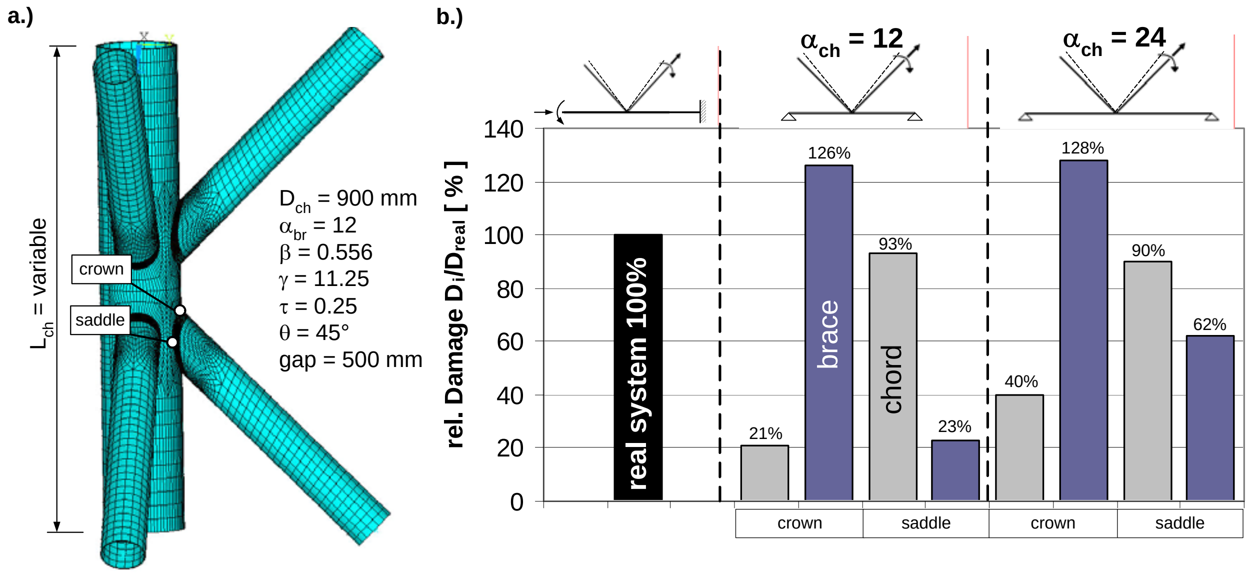


Figure 6: a.) Model of 90°-DK-joint; b.) Related damage at real and simplified system

For joints of braced or lattice structures with its large variety of potential geometries the nominal stress approach can hardly be applied. Local concepts, described by Radaj & Sonsino (1998) in detail, must be applied. The use of the structural stress approach for tubular joints is state-of-the-art and part of all actual offshore standards (see e.g. DNV 2004). Research has shown that stress concentration factors (SCF) for the hot-spot method should be determined by finite element analysis with volume elements and adequate weld modelling, e.g. according to AWS (2000). Furthermore complex joints, their three-dimensional loadings and special boundary conditions, for example in tripod or jacket structures acc. to Figure 1, can hardly be compared to simple tubular joints. Nevertheless it is common practice in offshore engineering to reduce the chord's boundary conditions to a hinged single-span system with the braces attached as loaded members. The lack of this approach is shown in Figure 6. The results of comparative damage calculations of a typical 90°-Double-K-joint (DK) used in a jacket type foundation are presented, applying the simplification with values of α_{ch} = 12 and 24 ($\alpha_{ch} = 2 \cdot L_{ch}/D_{ch}$ with the index 'ch' denoting the chord's dimension) and for the real system with braces <u>and</u> chord directly loaded. It shows that chord bending moments of the structure do not correlate with the bending moments of the hinged system induced by brace's action forces. Hence, as it is demonstrated by comparison of damage along the inner part of the circumferential weld (Figure 6b), an assessment by use of simplified systems can lead to severe underestimation of damage.

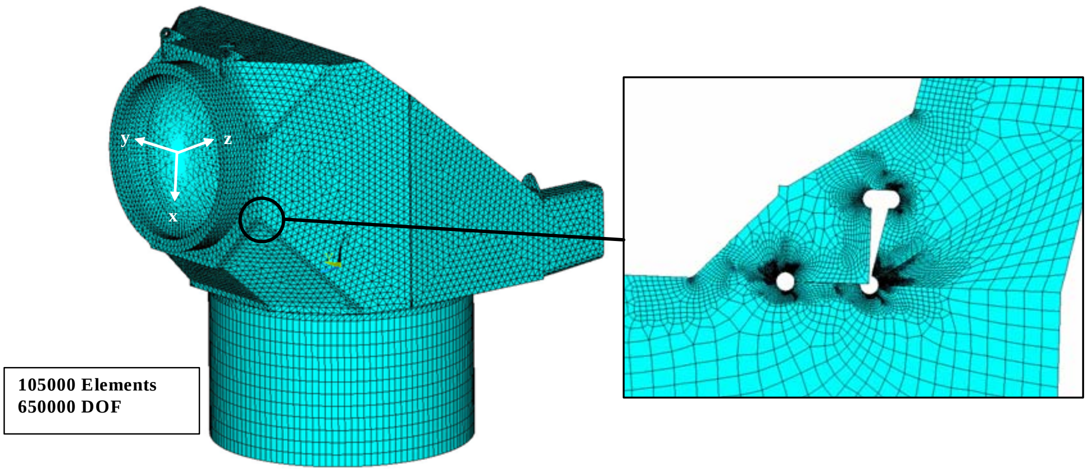


Figure 7: Analysed part of the welded machine carrier

Although not part of current offshore standards, more sophisticated local approaches, the notch stress approach (verified by Schaumann & Wilke 2005a) or the notch strain approach (Schaumann & Wilke 2006b), can be adopted successfully. These concepts are applied subsidiary, if

- uncertainties in the structural stress approach exist (e.g. required mesh refinement, extrapolation method)
- qualitative studies regarding the local weld geometry and plate thickness effects (Schaumann & Wilke 2005a) have to be done or
- fatigue classification of special details without performing tests is necessary.

Figure 7 shows the machine carrier of a multi-megawatt turbine were a FAT-classification was realised successfully (Schaumann & Wilke 2005b).

The question which local concept to apply often can be answered by analysing the actions. In wind energy high-cycle-fatigue dominates. Thus most of the stresses remain elastic and stress based concepts should be preferred which is demonstrated by Figure 8. Here the critical plane P_{SWT}-damage parameters (Smith et al. 1970) over their occurrence for the stress ranges of a typical time history, using stabilized cyclic material data according to the uniform material law (UML, for further details see Bäumel & Seeger 1990), are shown. In the same diagram the P_{SWT}-Woehler-Curve split up in elastic and plastic part is plot. As mentioned above the cycles predominantly stay in the elastic range what finally led to the application of the notch strain approach.

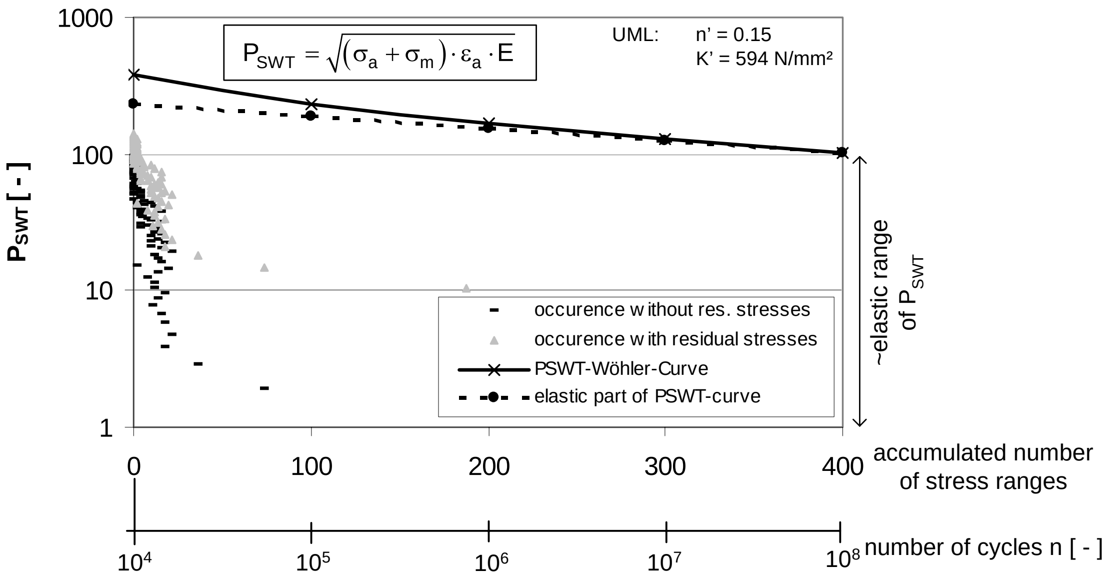


Figure 8: P_{SWT} damage parameter for the stress ranges of an exemplary time-history near the cut-out wind speed for the detail in Figure 6

2.5 Fracture Mechanics

Fatigue lives predicted by the fracture mechanics (FM) approach are sensitive to some parameters which are difficult to control, but which are implicit in S-N data. For this reason it is often seen to be advantageous to ensure consistency between the FM and S-N approach both in a deterministic and probabilistic sense. This has led to a correlation of initial crack sizes and crack growth parameters with the S-N-curves, resulting in fictitious initial crack sizes that often cannot be detected. The main use of the FM approach is for the assessment of the growth of through-thickness cracks. In this original form it can be used to define

- necessary inspection intervals
- material properties to prevent brittle failure.

In this manner an enhanced fatigue assessment can be completed. A linear elastic FM approach acc. to equation (1) (see also DNV 2004) modified with the Foreman (1967) relationship is used:

$$\frac{da}{dN} = \frac{C \cdot \left(\Delta K^{m} - \Delta K_{th}^{m} \right)}{\left(1 - R\right) - \frac{\Delta K}{K_c}} \tag{1}$$

For consideration of possible brittle cracking both the toughness master-curve according to Wallin (1998), which combines temperature and toughness, and the modified SANZ correlation (included e.g. in DASt 2005) is applied. This leads to the crack propagation vs. life curve shown in Figure 9, which has been calculated with the HANRIWA®-tool based on the weight function approach with inclusion of welding residual stresses. For demonstration purposes the threshold value of the stress intensity ΔK_{th} has been set to zero. Additionally, the required toughness $K_{C,req}$ has been plotted. In this example it would lead to an inspection interval of about 7.5 years, without exceeding the material requirements of the offshore standard, giving at least an idea of the risks coming along with cracks undetected by monitoring.

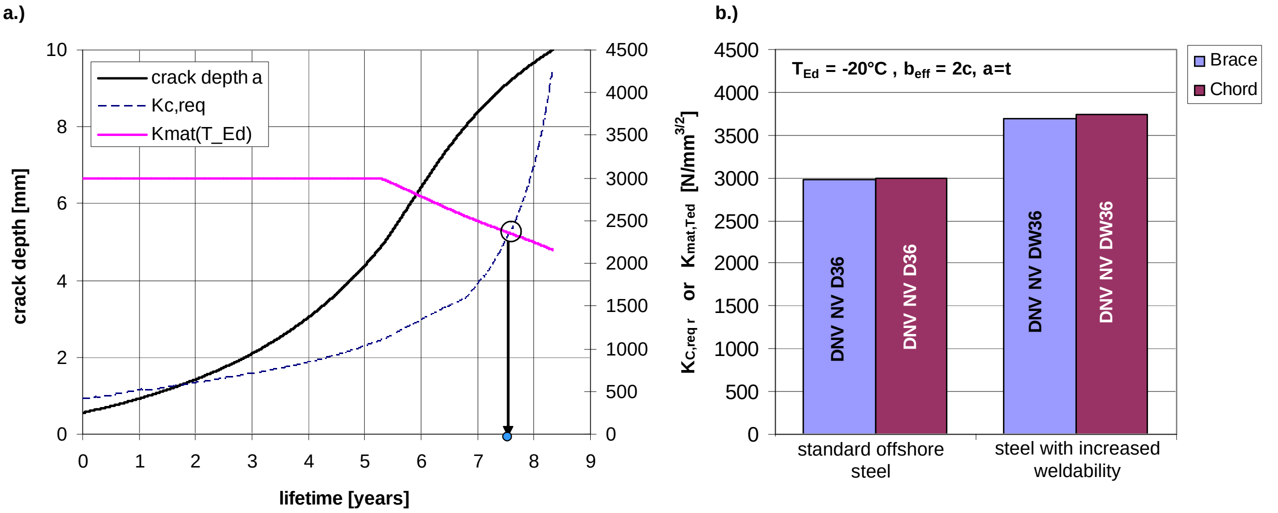


Figure 9: a.) crack depth/required fracture toughness vs. lifetime for the brace saddle location of the DK-joint Figure 6; b.) required fracture toughness of two different steel materials acc. to DNV (2004) for a joint in the splash zone (T_{Ed} = -20°C)

3 Conclusion

The fatigue damage of support structures for offshore wind energy converters often governs the design. It has been shown that time-history calculations necessarily have to be used.
Fatigue assessment for the presented braced types of support structures must be done by local approaches. The complexity of the problem usually requires the application of finite element methods. More sophisticated approaches than the structural stress approach can be applied beneficially. The definition of acceptable dimensions of undetected cracks at welds, which correlate to certain inspection intervals, are to be done by using approved methods of FM.
To summarise, an enhanced structural design should combine all concepts to reduce uncertainties of new foundation concepts used in offshore wind energy.

4 Acknowledgement

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