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Technical paper

Hydrodynamic loading on offshore wind turbine support structures

Andrew R. Henderson [*], Michiel B. Zaaijer[†]
[*] Garrad Hassan & Partners Ltd., St. Vincent's Works, Silverthorne Lane, Bristol, BS2 0QD, UK.
[†] Wind Energy Research Group, Faculty of Aerospace Engineering, Delft University of Technology, Netherlands.

Summary

In the shallow seas that continue to be the favoured locations for offshore wind farms, the limited water depths can result in highly non-linear waves. Although existing offshore design methods can undoubtedly result in a durable structure, there may be excessive cost penalties.

Non-linear (higher order) and breaking waves experienced in shallow waters may mean that the design methods based on experience in deeper water are unconservative. The coastal engineering branch does have substantial experience in designing in shallow water conditions, albeit again to much more stringent durability criteria than are appropriate here due to the greater risk to life there.

The determination of the design wave loads involves the selection of appropriate models of wave kinematics as well as force and structural dynamics models. Each selection will involve a compromise between accuracy and usability (speed, ease of use and simplicity and reliability of evaluation).

This paper is based on the work undertaken within the OWTES R&D project and examines the following aspects:

- recommendations for slender (monopiles) and compact (GBS) structures,
- the effect of the shallow water on wave climate,
- evaluation of the uncertainties due to the selection of appropriate models and the associated parameters regarding: (i) wave kinematics models, (ii) wave load models, (iii) and structural models.

For the evaluation of proposed engineering models for slender structures, the paper draws prominently on data collected at the Blyth offshore windfarm, where one turbine is comprehensively instrumented and an extensive collection of measurements, including extreme waves, has been recorded. Regarding compact structures, an analysis of the effect the choice of wave kinematics and load models has on the preliminary design process is undertaken and correction functions are proposed for simple geometries (tower plus flat base) to account for the reduced accuracy of the simpler approaches. Further details are available in the reports [9] and [15] written as part of this R&D project.

1. Introduction

The calculation and determination of design wave loads on offshore structures is a complex undertaking involving different wave models, load-calculation methods and probability analyses. Both the extreme and fatigue load cases need to be considered and the chosen approach may differ for these two cases and for different support structures.

The key to the solution is to determine the nature of the waves: their distribution and their hydrodynamic properties.

The procedures necessary to calculate the critical wave loading, can be divided into three stages:

(i) determining the design wave (extreme) or wave climate (fatigue)
(ii) selecting an appropriate wave load calculation procedure
(iii) determining the effect on the structure

Each stage is of equal importance for achieving an appropriate design solution and cannot be considered in isolation, as they are interrelated: for instance, the design wave can depend on the structural response since a larger wave at a frequency away from the structure's natural frequency can be less critical than a smaller wave close to the natural frequency. Hence an important aspect in the prediction of extreme- and fatigue loading of the support structure of an offshore wind turbine can be its dynamic response. The predictability of this dynamic response differs in some important aspects from that of platforms for the offshore oil industry and of onshore wind energy converters. The natural frequency of an offshore turbine can be wedged between different excitation frequencies, whereas the natural frequency of a fixed platform for the offshore oil industry is usually designed to be well above the wave excitation frequencies. The geometry and dimensions of offshore foundations differ from typical onshore solutions, resulting particularly in a larger influence of soil characteristics on the monopile foundation design.

To date, the size of the offshore windenergy market has not warranted intensive research on developing new and bespoke methods. Hence, judgment of appropriateness and applicability of existing methods, which can easily be a very subjective process, is needed. Points of concern with the application of the existing offshore engineering methods

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include:

- uncertainties in the wave modelling, because of increased non-linearities that occur in shallow waters
- increased occurrence and importance of (near-) breaking waves
- inappropriate safety margins

The purpose of this paper is to provide recommendations to determine extreme and fatigue loads in the design stage of slender and compact structures. The paper starts with a review of the determination of wave kinematics and wave loads. This is followed by an analysis of the use of numerical models in the design process and finally the measurements at Blyth are analysed and discussed.

2. Review of Previous Research

2.1. Determination of the Wave Kinematics

When the windturbine is located in a sea, it will encounter a lifetime of waves of varying sizes and forms. How can this be distilled into a limited number of cases that can be dealt with in a timely and cost-effective manner and yet represent the full-life experience of the structure?

For small waves in deep waters, the simplest linear or Airy wave model [1] is sufficient for calculating the kinematics. However, as wave heights and lengths increase relative to the water depth, the boundary condition assumptions cease to be fulfilled. In that case, first modified linear waves and finally non-linear wave theories of increasing orders become necessary to model the wave kinematics with sufficient accuracy. Figure 1 compares an extreme wave profile calculated using four of the candidate models (for 10m 15s wave in 21m water; 7th order stream function recommended by [2]); all derivatives of the linear wave theory (i.e. Wheeler, constant and extrapolated crest) assume the same sinusoidal profile.

It can be seen that non-linear theories exhibit (i) sharper wave crest and flatter troughs and (ii) higher crest and trough elevations. This is also clearly visible in the measurements at Blyth, Figure 2, evidence of the need to utilise higher order wave theories in spite of the additional effort.

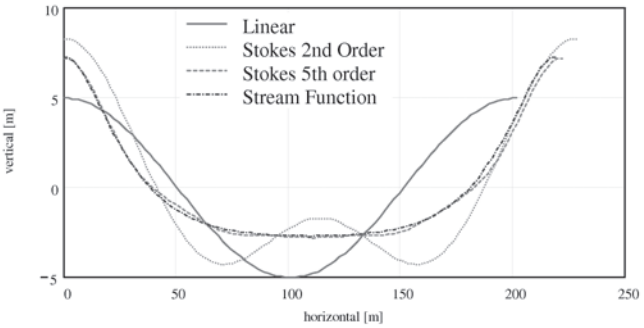


Figure 1: Wave Surface Profile

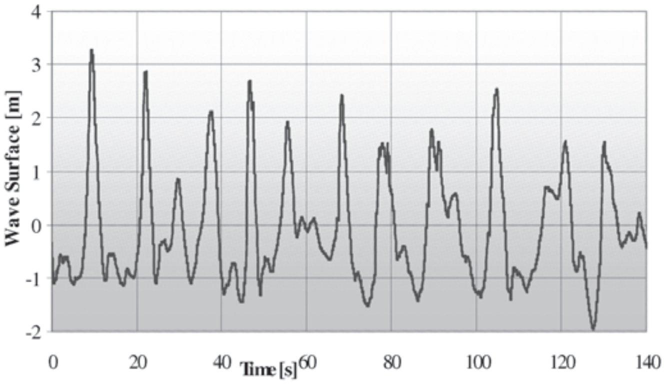


Figure 2: Recorded Sea Surface (Blyth)

A weakness of non-linear wave theories of an insufficiently high order is also apparent in Figure 1 in that the profile of the second order Stokes theory includes an erroneous higher order harmonic and that there was no solution for the wave length for the fifth order Stokes theory.

2.2. Calculation of the Wave Loads

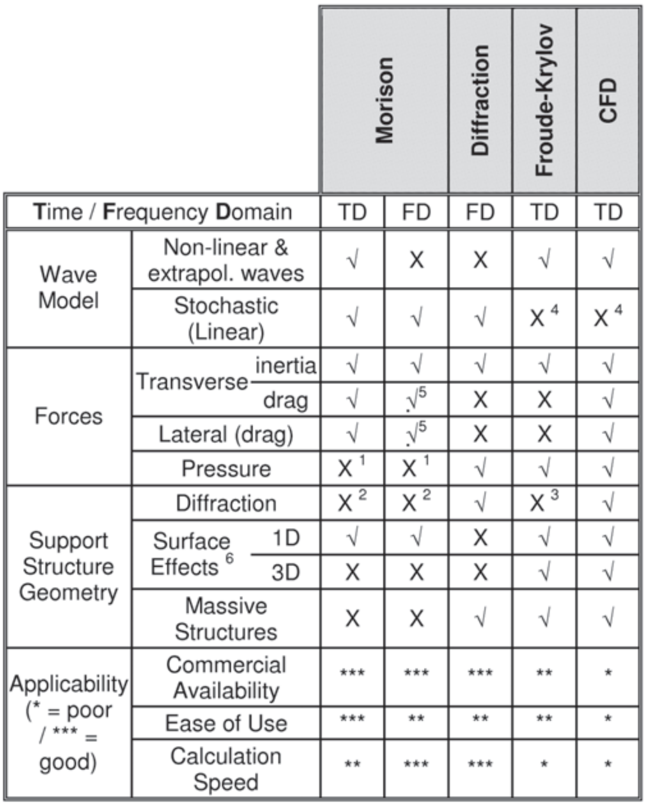
Of the available methods for calculating the wave loads listed in Table I, the two most widely used are:

- Morison's method, usually in the time domain, used for slender structures, such as monopiles and tripods,
- Diffraction theory, used for massive or compact structures, such as gravity base structure (GBS)

In addition the Froude-Krylov (or pressure integration) method offers the advantage of being able to model massive and complex structural geometries with any wave model [4], however diffraction has to be estimated in a similar manner as for Morison's method, but the more complicated geometries make this harder to perform. In the situations where this method could offer the most beneficial results, i.e. gravity base structures in shallow water, the wind loads on the turbine tend to dominate the design process [13]. In the longer term, CFD offers promising benefits of being able to model all aspects, though at undoubted penalties of time and clarity.

Table I summarises the capabilities of the candidate wave load theories in terms of which forces can be calculated, for which support structure geometries and which wave kinematic models can be utilised.

Table I: Wave load calculation Methods



		Morison		Diffraction	Froude-Krylov	CFD
Time / Frequency Domain		TD	FD	FD	TD	TD
Wave Model	Non-linear & extrapol. waves	√	X	X	√	√
	Stochastic (Linear)	√	√	√	X[4]	X[4]
Forces	Transverse inertia	√	√	√	√	√
	Transverse drag	√	√[5]	X	X	√
	Lateral (drag)	√	√[5]	X	X	√
	Pressure	X[1]	X[1]	√	√	√
Support Structure Geometry	Diffraction	X[2]	X[2]	√	X[3]	√
	Surface Effects[6] 1D	√	√	X	√	√
	Surface Effects[6] 3D	X	X	X	√	√
	Massive Structures	X	X	√	√	√
Applicability (* = poor / *** = good)	Commercial Availability	***	***	***	**	*
	Ease of Use	***	**	**	**	*
	Calculation Speed	**	***	***	*	*

[1] = can be modelled relatively easily by adding an extra term
[2] = can be modelled using MacCamy-Fuchs [11] correction for simple shapes
[3] = must be estimated
[4] = high demands on computation power
[5] = linearised

[6] = non-linear surface effects between the structure and the wave-field:
1D = in vertical direction only (i.e. wave height considered only at the vertical-axis of the structure)
3D = full geometric field (i.e. ave height at each surface element of the structure)

3. Hydrodynamic Loading – Examination of Theory

3.1. *Slender Support Structures*

The wave loads experienced by a typical slender offshore wind turbine support structure, a 4 m diameter monopile in 21 m, were examined. Using wave theory selection charts (such as in [2]) it is found that non-linear theory is recommended even for the smallest 1 m waves since otherwise the wave kinematics would be calculated insufficiently accurately. Similar charts have been developed to aid the selection of the load-model [4], which would show that diffraction effects are straight-forward and that both drag and inertia are important, hence the Morison method should be utilised. For linear theory, the overturning moment can be less than 25% of the more accurate value determined using stream function theory, in particular situations [9]. Other situations will result in different values however the overall conclusions will remain similar, being more extreme in shallower waters and less so as the water depths increase.

In shallower water depths, such as the 6 m at Blyth, wave-induced fatigue damage estimated using the Wheeler wave model is 43% of the value calculated using the more accurate stream function value [9] although turbine rotor fatigue will probably exceed that due to waves over the structures life time. The corresponding figures for the deeper water (21 m depth) case is 97% and 95% assuming the same wave distribution, confirming the supposition that using a non-linear wave model is important for the shallower waters only. This error does not include other inaccuracies due to ignoring surface effects for example.

3.2. *Compact Support Structures*

The determination of hydrodynamic loads on a gravity base structure, a support structure resting through its own weight on the seabed, is more complicated than on slender monopiles, due to the irregular geometry and the complicated effect that the structure has on the wave field (termed diffraction).

The traditional approach to wave load calculation for gravity base structures in the offshore industry has been to use diffraction analysis [2]. In the deep waters, in which such structures are located, the wave height is relatively low compared with the water depth. Therefore the use of linear wave theory, upon which the most commonly implemented form of diffraction theory is based, is applicable. On the other hand, offshore windfarms are located in much shallower seas, where highly non-linear waves are a more frequent phenomenon.

Both diffraction and Froude-Krylov (pressure integration) methods suffer from substantial but different weaknesses when calculating the wave loads on massive structures, see Table I, and hence the obvious approach is to use both methods together: diffraction theory to estimate the effect of the structure on the flow field and Froude-Krylov to calculate the wave loads using non-linear waves. Since GBS structures tend to be fairly simple, i.e. consisting of a round base, a tower section and possibly an ice-cone at the water surface, in many cases, it should also be possible to estimate the diffraction coefficients by comparing with other similar structures. In the early stages of the design process, it may be necessary for reasons of practicality to use the Morison method to determine the wave loads, for example in the procedures utilised here (a pressure term is added to the usual form of the equation to account for the forces on the base slab).

Table I identifies the main weaknesses of the three models; for the *diffraction* model, these are that it:

- Does not calculate drag loads (transverse or lateral)

- Ignores surface effects (i.e. the effect of the sea surface rising and falling around the column) hence also the full effects of complex geometries at the water surface
- Cannot model non-linear waves

Considering the *Froude-Krylov* approach, the main limitations identified are:

- Does not calculate drag loads (transverse or lateral)
- Does not calculate diffraction effects
- In addition, utilising non-linear wave theories is very demanding on computational resources, that being a disadvantage in industry rather than in research.

Finally turning to the *Morison* method, the major deficiencies are that it:

- Ignores surface effects (i.e. the sea surface rising and falling around the column)
- Ignores three dimensional effects of loads on the column, (i.e. more complicated variations of the wave field through the column's volume; only the first differential as calculated at the centre-line is considered)

and shortcomings that can and should be addressed include:

- two dimensional effects on end-loads (i.e. variations of the wave field over the base-slab surface).

A GBS structure may fail through the combination of (vertical) heave, which reduces the apparent weight of the structure, and (horizontal) surge, which then moves or flips it. Linear wave theory can lead to a conservative conclusion as can be seen from the calculated wave loads on an example GBS structure in Figure 3 and Figure 4. If the less accurate linear theories are used, both the maximum surge and heave forces are over-predicted and the peak surge force, which would cause the structure to start to slide, is incorrectly predicted to occur simultaneously with large upwards lifting force, which would reduce the frictional resistance and hence encourage slippage.

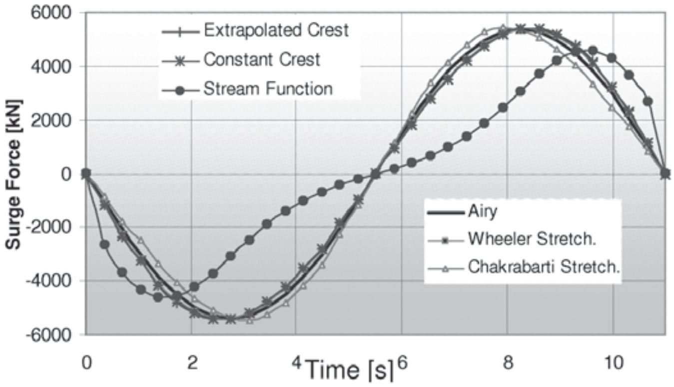


Figure 3: Surge Force

As would be expected, the *utilisation* is lower for stream function, Figure 6, than for airy wave, Figure 5, calculated loads, leading to an expanded feasible-design boundary, Figure 7.

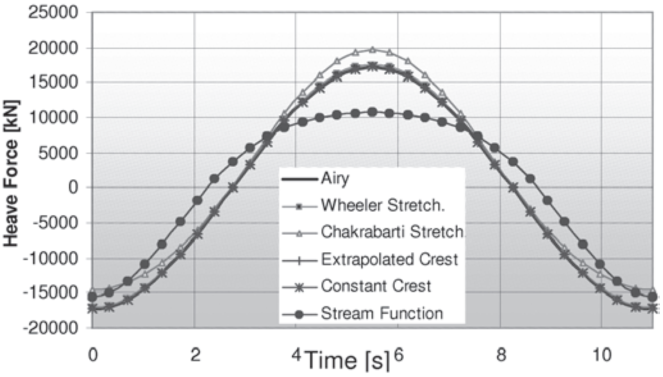


Figure 4: Heave Force

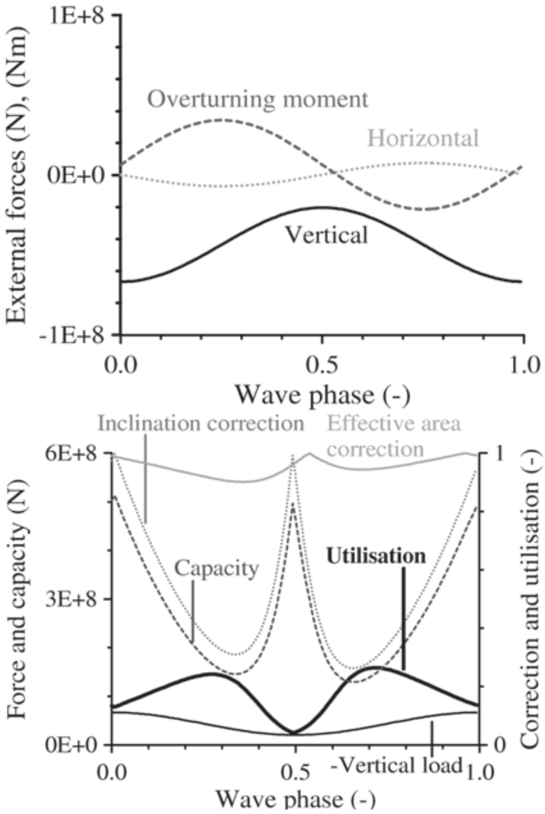


Figure 5: Design Forces using Airy Wave Theory

Here in Figure 7 the reference is airy theory with inertia coefficient of 2. The Stable region to the upper right represents an acceptable GBS design. The boundary represents the limit beyond which the structure will fail at the design conditions. Using a lower inertia coefficient and applying stream function theory both expand the limit for stable design and hence allow smaller and hence more cost effective foundations to be built.

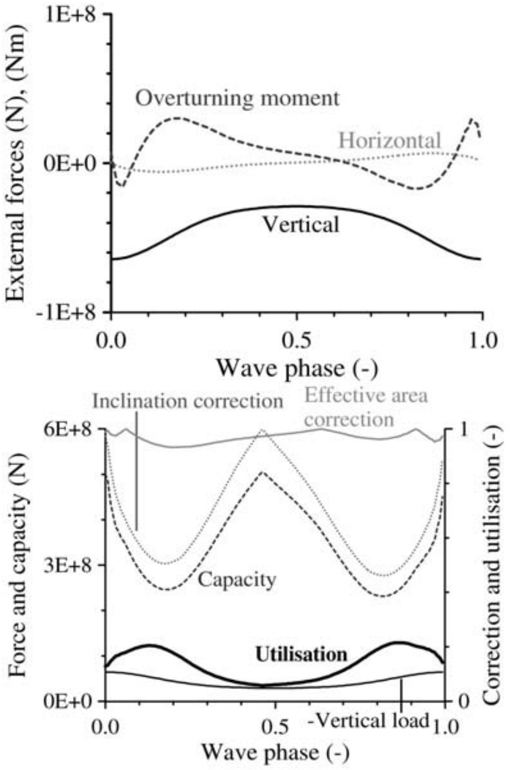


Figure 6: Design Forces using Stream Function

In attempting to evaluate the uncertainties associated with the different wave theories in Table II, the extreme load case for a deepwater GBS structure has been. The *error* values are with reference to the most accurate case.

The conclusion is that Morison is the least appropriate, as would be expected, but it is sufficient for initial concept evaluation and preliminary optimisation if the identified steps are followed. For the design stage, the choice is between diffraction and Froude-Krylov method, with both having important omissions in their scope. For deepwater structures, the weaknesses associated with diffraction analysis become smaller; on the other hand in shallow waters, where waves become less linear, the weaknesses in the Froude-Krylov theory become minor, assuming that care has been taken in selecting appropriate force coefficients.

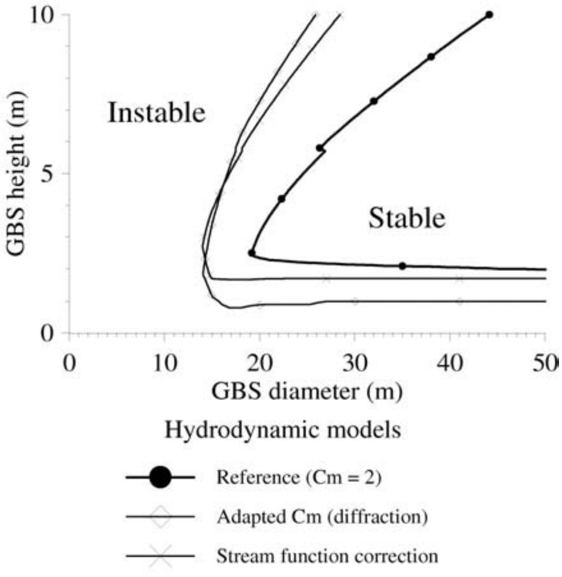


Figure 7: Boundary of Feasible Design

Table II: Evaluation of Errors in Wave-load Methods

Parameter	Wave Force Model		
	Morison	Diffraction	Froude-Krylov
Transverse Drag	Accurate	5%	5%
Lateral Drag	Error	Error	Error
Vertical Pressure	250% [1]	Accurate	Accurate
Diffraction [2]	20%	Accurate	10%
Surface Effects	5%	5%	Accurate
Non-Linear Waves	Accurate	10%	Accurate
Total Error	**30%**	**20%**	**15%**

1 = for the total error, it is assumed that the more accurate method of calculating the pressure forces (i.e. calculating the lift force at several points over the surface and not just at the centre) is used
2 = the diffraction error depends on the wave force model and is higher with Morison than with the Froude-Krylov method

The principal weakness of the Froude-Krylov model is the selection of appropriate force coefficients. Table III provides preliminary guidance based on the base slab diameter, D, and height, B, though a separate diffraction analysis would also always be recommended.

Table III: Inertia Coefficients for Use with Froude-Krylov Method

	Froude-Krylov		Morison	
	Horizontal	Vertical	Horizontal	Vertical
Inclined Slab	1	0.85	-	-
Rectangular Slab	$1+1.75\frac{B}{D}$	1	$1+2\frac{B}{D}$	1
Column	2	-	2 or see [11]	-
Ice-Cone	1.5	1	1.5	1

4. Hydrodynamic Loading – Evaluation of Measurements

This section compares calculated wave loads against the measurements taken at one of the Blyth monopile mounted offshore wind-turbine during severe weather. The campaign was recorded on 9 Nov '01 at 02:33, when the significant wave height was 4.63 m, the tide level was 1.53 m LAT and the wind speed was 13.9 m/s. The turbine was not generating power, which has implications for the damping but the loads induced by individual waves can be clearly identified.

4.1. Individual Waves

The surface elevation, Figure 8, and the mud-line bending of the monopile, Figure 9, of a large wave from this measurement campaign examined.

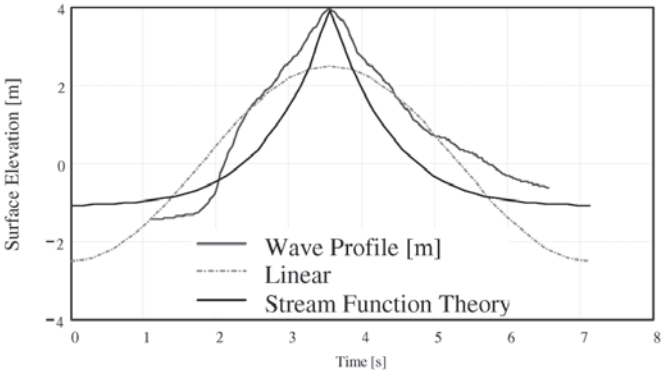


Figure 8: Wave Profile

The stream function theory predicts the correct crest elevation, but linear theory does not, Figure 8, as would be expected. All theories under predict the bending moment Figure 9, with stream function being closest. It can also be seen that the peaks in the loading of both the recorded wave and the stream function solution occur where the wave surface is steepest for each profile, at approximately $2^{1/4}$ s and $3^{1/4}$ s respectively.

Note that in Figure 9 measured internal bending stresses are being compared against theoretical external wave loads, i.e. the dynamics are not taken into account in the theoretical traces. Inclusion of dynamics in the theoretical trace will change the profile (by adding high frequency oscillations due to modal response) but would probably not change the maximum value significantly for this example; where dynamics is of particular importance is when the structure is already oscillating when the wave impacts onto it.

This is shown in Figure 10 and Figure 11, which illustrate examples of amplification and cancellation respectively. The ~2s first mode natural frequency of the structure is clearly apparent.

Dynamic amplification and cancellation are random events and hence cannot be assessed individually in a rigorous manner.

4.2. Campaign (30 minute sea state)

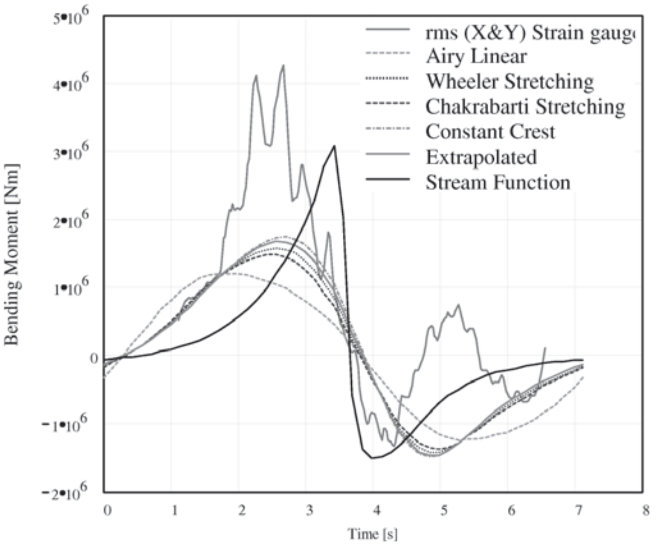


Figure 9: Pile Mudline Moment

Comparing calculated and measured wave loads across a complete 30 minutes measurement campaign, shows that the wave load calculation procedures are unconservative, Figure 12. The majority of the points lie to the right of the diagonal equality line, indicating that the measured values are higher than those calculated.

Three regions are identified in Figure 12:

region (a) where ringing induced by a previous large wave continues through a subsequent smaller wave, obscuring its impact,

region (b) dynamic amplification resulting in high measured loads in comparison with theory, Figure 10, (i.e. underestimation by theory)

region (c) dynamic cancellation resulting in low measured loads in comparison with the theory, Figure 11.

Note that, as expected, the underestimation is more pronounced for linear and stretched-linear theories than stream function (not shown).

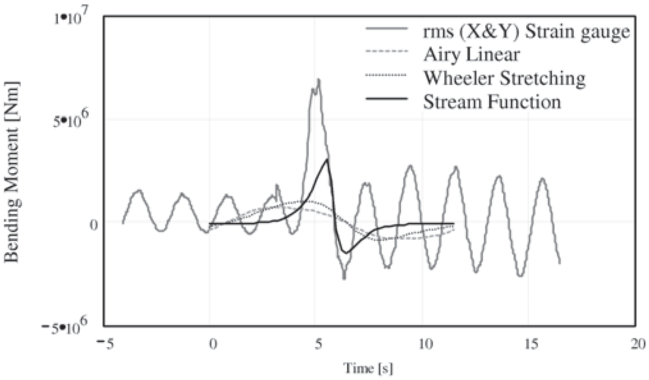


Figure 10: Dynamic Amplification

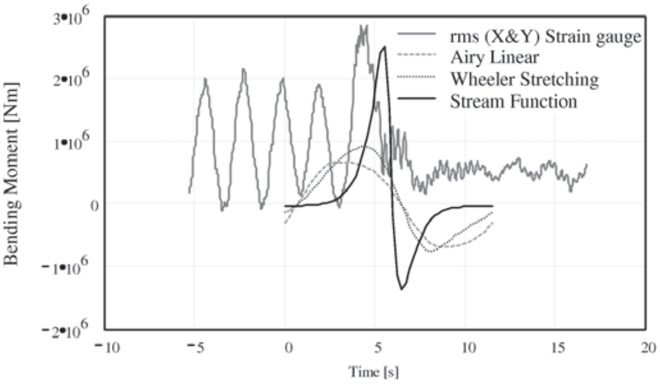


Figure 11: Dynamic Cancellation

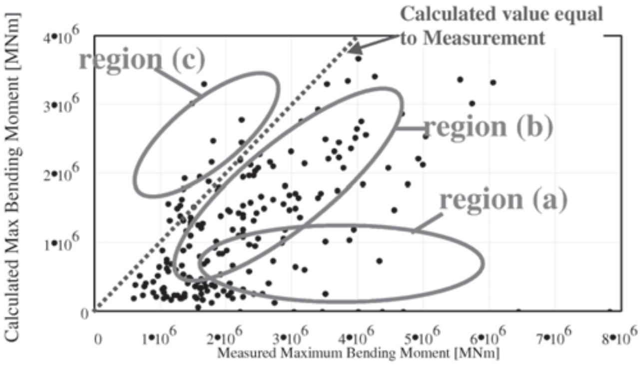


Figure 12: Maximum Bending Moment - Calculated verses Measured

5. Discussion and Conclusions

The principal issue identified is that methods developed by the offshore industry for deeper waters waves are unconservative in shallow water. This affects the analysis of slender and compact structures in different ways:

(i) slender structures respond dynamically to the loads, however no design approach is currently able to include this structural response together with stochastic non-linear waves of an appropriately high order,
(ii) compact structures exhibit little structural response however none of the available design methods are able to include both diffraction effects, non-linear waves and complex geometries simultaneously

A long term solution to both these dilemmas may well be CFD however we await further development of theory as well as necessary increases in computer power, neither of which will be available in the immediate future.

For this reason, the design process for *slender offshore windturbine support structures* takes two compromise approaches:

- regular non-linear waves
- and stochastic linear seas.

Since the concern with utilising *regular non-linear* wave approach is that the structural motion at incidence of the wave determines the dynamic response, (either amplification or cancellation), a potential solution, which was beyond the scope of this present work, would be to use linear stochastic models to determine a preliminary estimation of the motion response distribution and to apply the initial conditions to regular non-linear wave analysis.

The concerns regarding *linear stochastic* or probabilistic approach relate to underestimation of structural response [8]. These excluded aspects may cause damage disproportionate to their size because they potentially act near to the structure's natural frequencies resulting in both extreme but also fatigue loading being underestimated.

Conventional design approaches suggest that wind turbine loads will dominate the design process sufficiently to permit latitude in the accuracy of the wave load modelling. However, if wave loads are under represented to the extent suggested for these examples, the convenient assumption may not be true.

Examining this issue from an alternative perspective, Figure 13 and Figure 14 show the fatigue damage calculated from the measured strain histories for selected measurements campaigns, including operating, idling (labelled "ON" and "OFF" respectively and 30 minutes in length) and start/stop events (labelled "SWITCHING" and 4 minute in length. The fatigue damage was calculated using rainflow analysis of the time-series to determine the stress cycle history and Palmgren-Miner's hypothesis assuming a Wöhler curve with a slope of m=3 to determine the cumulative fatigue damage. For reasons of confidentiality, the damage is normalised in terms of arbitrary units.

Figure 13 shows that when the turbine is off, fatigue damage correlates well with significant wave height, while Figure 14 shows that when the turbine is on, fatigue damage correlates well with wind speed. Apart from the worst fatigue occurring when the turbine is started or stopped (especially in high winds), both wind and wave loads appear to be important.

Regarding *Compact Support Structures*, it can be concluded that:

1. diffraction is necessary to determine the added mass or diffraction coefficient of the support structure, in particular of the base.
2. For simple structures, such as the deepwater GBS examined here, a simple relationship can be determined using a handful of diffraction analyses.
3. The loads should then be checked using the Froude-Krylov method, to allow the implementation of non-linear wave theory, utilising the added mass coefficients calculated using diffraction analysis previously.

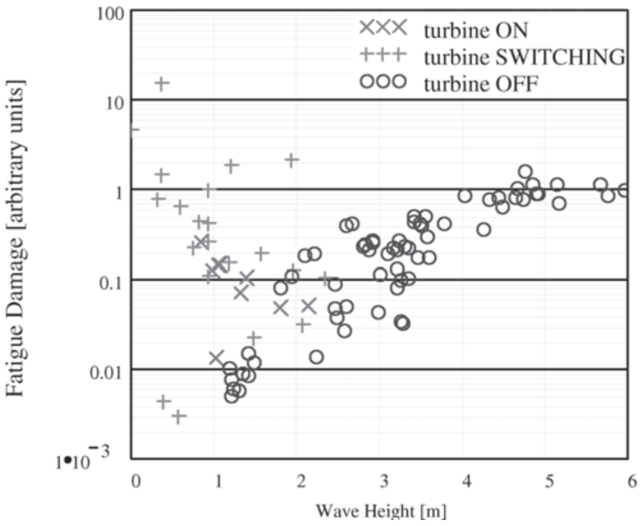


Figure 13: Campaign Tower Mudline Fatigue Damage, plotted against Wave Height

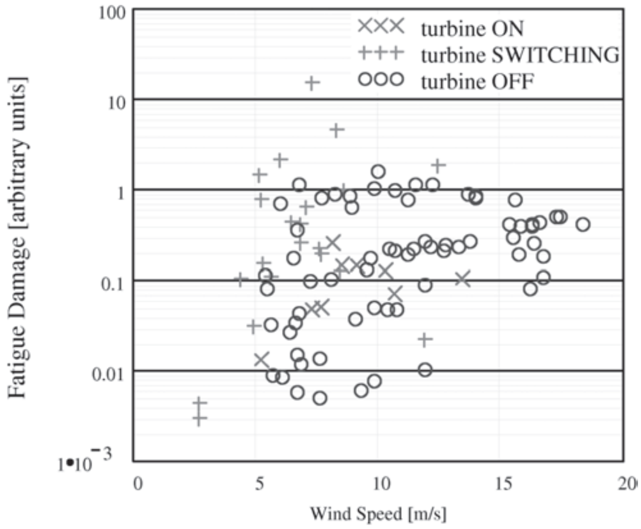


Figure 14: Campaign Tower Mudline Fatigue Damage, plotted against Wind Speed

For the example geometry examined here, it was found that linear theory was conservative, since using linear theory gives both a higher maximum lifting force, and a higher base shear (surge) force at that critical moment in the phase of the wave.

The inclination of the combined loading is a dominant factor in the GBS bearing utilisation hence to obtain a safe lightweight design solution, the time trace of the wave loads needs to be considered and not just the peak values.

Acknowledgements

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References

1 Airy, Sir G. B., Tides and waves, *Encycl. Metrop., Art* **192**, 1845.

2 Barltrop, N.D.P., Adams A.J., *Dynamics of Fixed Offshore Structures*, Butterworth-Heinemann, 1991.

3 Boussinesq, J., Théorie des ondes et des ramous qui se propagent le long dún canal rectangulaire horizontal, en communiquant au liquide contenu dans ce canal des vitesses sensiblement pareilles de la surface au fond, *J. Math. Pures Appl.*, Vol. **17**, pp 55-108, 1872.

4 Chakrabarti, S.K., Hydrodynamics of Offshore Structures, *WIT Press*, ISBN 0-905451-66-X, 1987.

5 Cheng, P.W., A reliability based design methodology for extreme responses of offshore wind turbines, *PhD Thesis, Delft University of Technology*, 2002.

6 Dean, R.G., Stream Function Representation of Nonlinear Ocean Waves, *J. Geophys. Res.,* Vol **70**, No. 18, pp 4561-4572, 1965.

7 Det Norske Veritas, Rules for the classification of fixed offshore installations. DNV, 2000.

8 Henderson, A.R. & Cheng, P.W., Wave loads on slender offshore structures: Comparison of Theory and Measurement, *German Wind Energy Conference [DEWEK]*, Wilhelmshaven, Germany, October 2002.

9 Henderson, A.R. et al., Hydrodynamic Loading on Offshore Wind Turbines, Report SW0218x, Section Wind Energy, Technical University of Delft, February 2003.

10 Madsen, P.A., Bingham, H., Liu, H., The ultimate Boussinesq formulation for highly dispersive and highly nonlinear water waves, *Coastal Engineering Conference*, pp.176-189, 2000.

11 MacCamy, R.C. and Fuchs, R.A., *Wave Forces on Piles: a Diffraction Theory*, Tech. Memo No. 69, US Army Corps of Engineers, Beach Erosion Board, 1954.

12 Neumann, G. and Pierson W. J., *Principles of physical oceanography*, Prentice-Hall. 1963.

13 Sørensen, H. C. et al, Middelgrunden 40 MW offshore wind farm, a prestudy for the Danish offshore 750 mw wind program, ISOPE 2000.

14 Wheeler, J.D. Method for calculating forces produced by irregular waves, *First Offshore Technology Conference, Houston, Texas*, OTC 1006, 1969.

15 Zaaijer, M.B. et al, Sensitivity Analysis for Foundations of Offshore Wind Turbines, *Section Wind Energy, TUDelft,* Report WE 02181, March 2002.