Hydrodynamic Loading on Offshore Wind Turbines

Andrew R. Henderson\(^1\), Michiel B. Zaaijer\(^1\), Tim R. Camp\(^2\)

(1) Section Wind Energy, Delft University of Technology, PO Box 5048, 2600 GA Delft, Netherlands, tel: +31-15-278-5170, henderson@offshorewindenergy.co.uk / M.B.Zaaijer@CiTG.TUDelft.nl

(2) Garrad Hassan & Partners Ltd., St. Vincent’s Works, Silverthorne Lane, Bristol, BS2 0QD, UK. tel: +44-117-972-9929, camp@garradhassan.co.uk

Abstract

In the shallow seas that are the favoured locations for offshore wind farms, the limited water depths can result in highly non-linear waves. 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 of evaluation). Currently, the offshore oil and gas industry is focusing on ever deeper waters as much of the available hydrocarbon resource in accessible medium depth waters is already being exploited. In contrast, offshore wind energy is being developed in shallower waters, often at sites exposed to extreme weather such as the European North Sea. The approach needed here will be subtly different but equally demanding, with economic reasons being more prominent. Electricity is a low-value commodity in a highly competitive market and the costs of generation using offshore wind farms are approaching the costs of conventional generation. It is vitally important that inappropriate and excessively conservative design approaches do not sabotage this trend.

Although existing offshore design methods can undoubtedly result in a durable structure, there may be excessive cost penalties. On the other hand, non-linear 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.

This paper gives a final report on the work undertaken on this subject within the OWTES project and examines the following aspects:
- different recommendations for slender (monopiles) and compact (GBS) structures,
- the affect 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, have been recorded. Regarding compact structures, an analysis of the differences 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 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. It is however of vital importance if a cost-effective and durable structure is to be designed. Both the extreme and fatigue load cases need to be considered and the actual approach may differ for these two cases and for different support structures. The key to the problem is to determine the nature of the waves: their distribution and their hydrodynamic properties.

The procedures necessary to calculate the critical wave loading, for either the fatigue or extreme cases, can be divided into three stages:

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

Each stage is of equal importance for achieving an appropriately design solution and cannot be considered in isolation, as they are interrelated: for instance, the design wave can depend on the structural response when 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 energy converter (OWEC) 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 OWEC 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 for the slender monopile foundation.

At this moment, the size of the offshore windenergy market does not warrant intensive research on developing new and bespoke methods, and 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 include:

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

2 - Review including Relevant Previous Research

When considering the hydrodynamic loading aspects of the design process, both the extreme and the fatigue cases need to be considered, each of which can require a different approach. Defining the design procedure consists of selecting an appropriate wave or wave climate, kinematic model, loading model and structural model. Only a limited number of permutations of these models are possible (an example being that
diffraction cannot be applied to high order wave theories) and the more sophisticated or accurate the model, the greater the demands on the engineer’s and the computer’s time.

2.1 - Determination of the Wave Kinematics

First the external loads need to be defined and for OWEC support structures the hydrodynamic wave forces are of great interest (In contrast, hydrodynamic loads have limited impact on the design of the rotor and nacelle). When the wind turbine 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?

Several wave kinematics models have been compared with the measurements and these are briefly described below:

(i) Airy; or linear wave theory’s [1] relative simplicity is both its strength and weakness, in that what it lacks in accuracy itself, it can compensate by being integrated with other aspects of the wave-load calculation process, such as corrections, stochastic waves, diffraction etc. Its primary weakness is that it cuts off wave peaks and troughs

(ii) Wheeler Stretching [14] is an example of the flexibility of the Airy wave in that the kinematics as calculated at the mean-water level are applied up to the true surface with the distribution down to the seabed being stretched accordingly; this gives an improved prediction of the wave kinematics and this correction can also be applied to simulations of stochastic seas

(iii) Deans Stream Function [6]; has largely superseded all other wave-theories for regular waves where Airy is insufficient. It is a numerical solution, which can be applied up to near breaking wave height.

Stokes and Cnoidal theories found favour in the past however are now superseded by stream function theory. Boussinesq [3] is receiving renewed research attention [10] but within the engineering profession its use is still limited. Other corrections to linear theory include constant and extrapolated crest; these can be excessively conservative in deeper waters (but are satisfactory in the water depths of interest here) and so are in limited use by engineers.

![Wave Surface Profile](image1.png)

![Recorded Sea Surface (Blyth)](image2.png)

Figure 1: Wave Surface Profile

Figure 2: Recorded Sea Surface (Blyth)

Figure 1 compares the wave profile for the above wave models for an extreme wave \(^1\) (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

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\(^1\) for 10m 15s wave in 21m water
profile. It can be seen that, relative to linear theory, the non-linear theories predict that waves have (i) sharper crest and flatter trough profiles and (ii) higher crest and trough elevations. This is also clearly visible in stochastic seas as the surface recording from Blyth in Figure 2 shows. A weakness of low order theories when applied to extreme waves is apparent in Figure 1 in that the profile of the second order Stokes theory includes an erroneous higher order harmonic and in that the exact wave length for the fifth order Stokes theory could not be calculated because the algorithm used here failed to converge.

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 structures, such as gravity base supports

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 makes 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: Wave load calculation Methods

<table>
<thead>
<tr>
<th></th>
<th>Morison</th>
<th>Diffraction</th>
<th>Froude-Krylov</th>
<th>CFD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time / Frequency Domain</td>
<td>TD</td>
<td>TD</td>
<td>TD</td>
<td>TD</td>
</tr>
<tr>
<td>Forces</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transverse inertia</td>
<td>√</td>
<td>√</td>
<td>√</td>
<td>√</td>
</tr>
<tr>
<td>Transverse drag</td>
<td>√</td>
<td>5</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Lateral (drag)</td>
<td>√</td>
<td>5</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Pressure</td>
<td>X^1</td>
<td>X^1</td>
<td>√</td>
<td>√</td>
</tr>
<tr>
<td>Geometry</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Diffraction</td>
<td>X^2</td>
<td>X^2</td>
<td>X^3</td>
<td>√</td>
</tr>
<tr>
<td>Surface Effects 1D</td>
<td>√</td>
<td>X</td>
<td>√</td>
<td>√</td>
</tr>
<tr>
<td>Surface Effects 3D</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>√</td>
</tr>
<tr>
<td>Massive Structures</td>
<td>X</td>
<td>X</td>
<td>√</td>
<td>√</td>
</tr>
<tr>
<td>Wave Model</td>
<td>Non-linear &amp; extrapol. waves</td>
<td>√</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td>Stochastic (Linear)</td>
<td>√</td>
<td>√</td>
<td>X^4</td>
</tr>
<tr>
<td>Applicability</td>
<td>Commercial Availability</td>
<td>***</td>
<td>***</td>
<td>***</td>
</tr>
<tr>
<td></td>
<td>Ease of Use</td>
<td>***</td>
<td>**</td>
<td>**</td>
</tr>
<tr>
<td></td>
<td>Calculation Speed</td>
<td>**</td>
<td>***</td>
<td>***</td>
</tr>
</tbody>
</table>
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. wave height at each surface element of the structure)

3 - Hydrodynamic Loading – Examination of Theory

3.1 - Slender Support Structures

In this section, the wave loads calculated using different kinematic models is examined. In each case, Morison theory is used, since this is most appropriate for the geometry under consideration. A table of waves with period ranging between 5 s and 15 s at 1 s intervals and the height ranging between 1 m and 7 m at 1 m intervals is applied. The structure was assumed to be a 4 m diameter monopile, with very high roughness due to age, hence $C_M = 1.79$ and $C_D = 1.55$ as recommended by DNV [7]. The water depth was taken as 21 m. (Note these parameters are different from those at Blyth since these are likely to be more representative of future offshore windfarms). 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. 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.

Figure 3: Airy (left) & Wheeler (right) Overturning Moment

This group of charts shows the calculated maximum values of the wave-induced overturning moment. Figure 3 plots the values calculated from using Airy linear (left) and Wheeler theories (right) and Figure 4 (left) shows the results from stream function theory, which gives the highest predictions for the loads. For small wave heights of up to perhaps 2 m, the differences are generally low but the errors increase rapidly with height, Figure 4 (right). These charts reflect a particular case and 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. For linear theory, the overturning moment can be less than 25% of the more accurate value determined using stream function theory.
Figure 4: Stream Function Overturning Moment (left) and %-Error in Airy (right)

Turning to shallower 6 m water depths, such as at Blyth, if the total fatigue damage is estimated by applying the Blyth sea climate, use of the Wheeler wave model results in a lifetime fatigue of 43% of the more accurate stream function value (linear theory returns 35%). 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 but it will not matter if the fatigue damage is dominated by the rotor loads.

The extreme and fatigue response stresses depend strongly on the dynamic behaviour of the wind turbine structure. The peaked surface elevation of the non-linear waves shown in Figure 1 results in harmonic loads at integer multiples of the wave frequency, a feature that is less pronounced if present at all in linear waves. When harmonics of the wave frequency coincide with the structural natural frequency, resonance of the structure results in amplification of the response, Figure 5. The presented amplification is valid for a infinite sequence of regular and identical periodic waves. For a single incoming wave, the relation between the phase of the wave and the initial structural motion determines the effective amplification, as is shown below in Figure 12 and Figure 13 from the measurements taken at the Blyth turbine.

Figure 5: Dynamic Amplification of Non-Linear Waves

Figure 5 shows that there is significant dynamic amplification at wave periods of around three ($\approx 6.4$ s) and four ($\approx 8.5$ s) times the tower's first natural period and that this is only apparent for the extreme wave (the undulations in the small wave curve are due to
insufficient run times in the simulations) and also that no cancellation can be identified at the mid-natural period point (i.e. $3 \frac{1}{2} \times$ natural period) but instead being present immediately prior to the multiple value (i.e. $2.9$ and $3.9 \times$ natural period). Examining the dynamic amplification in terms of kinematics, if the ratio is an integer, the structure will be moving in phase with the wave when each rising crest reaches the monopile and severe dynamic amplifications will be stimulated, while if it is an integer plus a half, it is expected that the structure would be moving into the wave when it impacts.

### 3.2 - Compact Support Structures

Gravity base foundations have been a popular foundation concept for offshore wind turbines in sheltered and ice-infested waters. As wind turbines increase in size and move to deeper and more exposed sites, the hydrodynamic loading on the large gravity base structures becomes an ever more important contribution. The determination of hydrodynamic loads on a gravity base structure 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). Furthermore, the design of gravity base structures with a large contribution of hydrodynamic loading will be more susceptible to inaccuracies in the determination of the hydrodynamic loading. The hydrodynamic loading on the gravity base structure itself is part of the design loop with respect to its load bearing function. On the other hand, the geo-technical design of a pile foundation can be performed directly using the hydrodynamic loading on the structure above the seabed as a priori knowledge.

The traditional approach to wave load calculation for gravity base structures in the offshore industry has been to use diffraction analysis. 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 (also called Airy 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 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 possible 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 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 conservative conclusion as can be seen from Figure 6 and Figure 7. If the more accurate stream function theory is used, both the maximum surge and heave forces are reduced and the maximum surge force does not occur simultaneously with the significant heave forces. Note also that when non-linear theory is used, the erratic shape of the time-trace means that the time-resolution needs to be higher to ensure that peak loads are detected. As would be expected, the *utilisation* shown in Figure 8 is lower for stream function than for airy wave calculated loads, leading to an expanded feasible-design boundary, Figure 9. Here the reference is airy theory with inertia coefficient of 2. Using a lower inertia coefficient and applying stream function theory both allow smaller and hence more cost effective foundations to be built.
In attempting to evaluate the uncertainties associated with the different wave theories in Table II, the extreme load case for the deepwater GBS structure has been used and the values quoted for the error are derived from analyses of that structure. (The error values are not meant to be illustrative only and hence the total error for each method is found buy addition, the whole process being too approximate to warrant more rigorous mathematics such as r.m.s.). 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.

### Table II: Evaluation of Errors in Wave-load Methods

<table>
<thead>
<tr>
<th></th>
<th>Error</th>
<th>Morison</th>
<th>Diffraction</th>
<th>Froude-Krylov</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse Drag</td>
<td>5%</td>
<td>-</td>
<td>√</td>
<td>√</td>
</tr>
<tr>
<td>Lateral Drag</td>
<td>0</td>
<td>√</td>
<td>√</td>
<td>√</td>
</tr>
<tr>
<td>Vertical Pressure</td>
<td>250%</td>
<td>√¹</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Diffraction²</td>
<td>10%</td>
<td>20%</td>
<td>-</td>
<td>√</td>
</tr>
<tr>
<td>Surface Effects</td>
<td>5%</td>
<td>√</td>
<td>√</td>
<td>-</td>
</tr>
<tr>
<td>Non-Linear Waves</td>
<td>10%</td>
<td>-</td>
<td>√</td>
<td>-</td>
</tr>
<tr>
<td><strong>Total Error</strong></td>
<td>30%</td>
<td>20%</td>
<td>15%</td>
<td></td>
</tr>
</tbody>
</table>

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 load method and is higher with Morison than with the Froude-Krylov method

As mentioned in the previous paragraph, the main weakness of the Froude-Krylov design method is the selection of appropriate force coefficients. Table III attempts to provide 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

<table>
<thead>
<tr>
<th></th>
<th>Froude-Krylov</th>
<th>Morison</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Horizontal</td>
<td>Vertical</td>
</tr>
<tr>
<td>Inclined Slab</td>
<td>1</td>
<td>0.85</td>
</tr>
<tr>
<td>Rectangular Slab</td>
<td>$1 + 1.75 \frac{B}{D}$</td>
<td>1</td>
</tr>
<tr>
<td>Column</td>
<td>2</td>
<td>-</td>
</tr>
<tr>
<td>Ice-Cone</td>
<td>1.5</td>
<td>1</td>
</tr>
</tbody>
</table>

4 - Hydrodynamic Loading – Evaluation of Measurements

This chapter evaluates wave load theory against measurements taken at the instrumented offshore wind-turbine at Blyth. The campaign datasets used to illustrate within this paper was campaign reference 158, recorded on 9 Nov ‘01 at 02:33, when the significant wave height was 4.63 m, the tide level was 1.53 m and the windspeed was 13.92 m/s. The turbine was off. The initial strain gauges calibrations were performed using the method described in Appendix A - and this has since been confirmed by external calibrations.

In all figures in this paper, the wave height is defined as the difference between the crest and the mean of the previous and following troughs. This provides a generally close match for the stream function wave, Figure 10 being a particularly good example.

4.1 - Individual Waves

A high wave selected from the dataset is examined in Figure 10 (surface elevation) and Figure 11 (pile mudline bending moment). It can be seen that for this example the stream function theory predicts the same crest elevation but that, as would be expected, linear theory does not. This is also reflected in the bending moment traces in Figure 11, which show that all theories underestimate the maximum bending moment but that the stream function is 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, at approximately 2¼ s and 3¼ s respectively.
Note that in Figure 11 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 that much for this example; where dynamics is of particular importance is when the structure is already oscillating when the wave impacts onto it. This is clearly shown in Figure 12 and Figure 13, which illustrate amplification and cancellation respectively.

**Figure 12: Dynamics Amplification**

**Figure 13: Dynamics Cancellation**

### 4.2 - Campaign (30 minute sea state)

If a comparison is made between the measurements and theory for all individual waves in the campaign (30 minutes), Figure 14, it can be seen that all theories underestimate the maximum bending moment, with linear and stretched-linear theories being lower than stream function. A correct prediction would place the points on to the diagonal line. There is a large amount of scatter, due to several reasons (identified in Figure 14): region (a) ringing induced by the previous wave resulting in an apparent underestimation by theory of the loads, region (b) dynamic amplification resulting in high measured loads in comparison with theory, Figure 12, (i.e. underestimation by theory) region (c) dynamic cancellation resulting in low measured loads in comparison with the theory, Figure 13.

**Figure 14: Maximum Bending Moment - Calculated verses Measured**

### 5 - Discussion and Conclusions

This paper has focused on understanding and predicting the hydrodynamic loads and hence the structural response of offshore windturbine support-structures. The principal
The problem identified is the waves in shallow water are less linear than those that the currently used methods were developed for (the methods being developed by the offshore industry which focuses on deeper waters). 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 will be CFD however we await further development of theory as well as necessary increases in computer power, both of which should be available in perhaps a decades time.

Currently, the design process for slender offshore windturbine support structures takes two approaches: regular non-linear waves and stochastic linear seas. The problem with using regular non-linear wave approach is that the structural motion at incidence of the wave determine 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.

Regarding the stochastic or probabilistic approach, this appears to underestimate the structural response since it does not model the highest waves accurately (exclude harmonics) and does not take account of impact loading [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. Together with other aspects identified within this paper, this may lead to the hydrodynamic loading being relatively more important in the overall design of the support structure than expected. Figure 15 shows the estimated fatigue damage derived from a selection of measurements campaigns (both 4 and 30 minute), plotted against significant wave height (left) and wind speed (right). The turbine status (off, switching or on) is identified by the marking used in the charts. From the left chart, it can be seen that when the turbine is off, fatigue damage correlates well with significant wave height, while from the right chart, it can be seen that when the turbine is on, fatigue damage

Figure 15: Campaign Tower Mudline Fatigue Damage

Offshore Windenergy in Mediterranean and Other European Seas (OWEMES conference), Naples, Italy, April 2003.
correlates well with windspeed. Apart from the worst fatigue occurring when the turbine is switched on and off, both wind and wave loads appear to be important.

Examination of the results from the above hydrodynamic analyses for Compact Support Structures suggests that diffraction is necessary to determine the added mass coefficient of the support structure, in particular of the base. For simple structures, such as the deepwater GBS examined here, a simple relationship can be determined using a handful of diffraction analysis. 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. For the deepwater 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 correction of the bearing capacity due to the inclination of the combined loading is a dominant factor in the bearing utilisation. Therefore, hydrodynamic load calculations must not only lead to correct prediction of load amplitudes, but the shape of the variation during passing of the wave must also be accurate. The results point in the direction that both modelling of non-linearity and of (linear) diffraction are important to obtain a safe lightweight design solution.

6 - Recommendations

Future work on hydrodynamic loading of offshore windturbines should focus on achieving the following in an efficient and effective manner:

- To examine how stochastic non-linear wave loading of a higher order than at present can be applied to slender support structures
- To examine how both diffraction and non-linear surface effects can be applied to compact support structures

In the meantime, for slender support structures, it is recommended that analyses involving a combination of linear stochastic seas and non-linear regular waves be performed.

For compact structures, it is recommended that diffraction analyses are performed as well as checks using the Froude-Krylov method. During conceptual evaluation, Morison may be used but care must be taken how the forces on the base slab are determined.

The measured wave loads at Blyth are higher than any of the applied theories predict. It is expected that this be partly due to breaking wave impact loads but the fact that waves in the non-breaking seas will be of an extreme form, which cannot be adequately modelled, must also be important.

Acknowledgements

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**Appendix A - Calibration of Strain Gauges**

This appendix presents a description of how preliminary calibrations of the strain gauges in the tower and pile were made against the periodic first mode oscillations during short periods (of a few seconds) of relative calm in between the waves that were generating the motion response. This method depends on the fact that extreme non-linear waves have a particular shape: they consist of short and sharp peaks in between long regions of relatively smooth water where the wave loadings are minimal, see Figure 1 and Figure 2.

Since the external calibrations of the strain gauges were undertaken at a late stage in the project, in order to start analysing the data earlier on and hence gain maximum benefit from the measurement programme, it was necessary to perform a preliminary calibration. Examining the data for the nacelle accelerometers and the strain gauges, it could be seen that at certain times, when the turbine was turned off, there was good correlation between the signals, Figure 16.

![Figure 16: Correlation of Nacelle Accelerations and Strain Gauge Readings](image)

When all such cycles within a 30 minute campaign are evaluated, and the most appropriate selected (i.e. with little higher order noise) a calibration of the strain gauge voltage reading against the nacelle acceleration can be made, Figure 17. Making an assumption for the mode shape allows an estimation of the calibration against the

![Figure 17: Preliminary Calibration of the Strain Gauges [m/s^2 per V]](image)

![Figure 18: Calibration Factors^2](image)

^2 (Moment per Volt output) for Tower and Pile Strain Gauges (shows ± standard deviation)
moments at the strain gauge to be made in both X and Y axes, Figure 18. These calibration factors were in good agreement with the externally calibrated values, in particular the differences for the mudline values were in fact significantly less than the accepted uncertainty of the external calibration process, see Table IV.

Table IV: Evaluation of Strain Gauge Calibration Method

<table>
<thead>
<tr>
<th>Axis</th>
<th>Calibration Factor [Nm per V]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Calculated [8]</td>
</tr>
<tr>
<td>X</td>
<td>1.27 × 10^10</td>
</tr>
<tr>
<td>Y</td>
<td>1.21 × 10^10</td>
</tr>
</tbody>
</table>

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