Dynamics and Design Optimisation of Offshore Wind Energy Conversion Systems
Stellingen
behorend bij het proefschrift

Dynamics and Design Optimisation of Offshore Wind Energy Conversion Systems

van
Martin Kühn

1. Voor grootschalige exploitatie van windenergie buitengangs is de ontwikkeling van een volwassen offshore windenergetechnologie van cruciaal belang. Basis voor een dergelijke ontwikkeling zijn de ervaringen met de windenergetechnologie op het land, de offshore technologie en de bedrijfsvoering van grote elektriciteitscentrales.

Dit proefschrift.

2. Het kruiven van de windturbine uit de windrichting in de golfrichting om de vermoeiingsbelastingen van een offshore windenergie-converter te verlagen, zoals voorgesteld door Sinclair en Clayton, zal het tegengestelde effect hebben.


3. Op overtrek geregelde windturbines met een vrijwel vast toerental zijn niet geschikt voor grootschalige toepassing buitengangs.

4. Op locatie moeten offshore windenergie-converters uit een minimaal aantal hoofdonderdelen worden opgebouwd. Over vijf à tien jaar zal, afhankelijk van de configuratie, dit aantal één tot drie bedragen.

5. Op langere termijn zullen de tweebladige offshore windenergie-converters herleven.

6. Kwaliteit is belangrijker dan kwantiteit bij de ontwikkeling van windparken buitengangs.

7. De snelle technologische ontwikkeling en groei van de internationale windenergie-industrie belemmert op dit moment de belangstelling voor zowel fundamenteel als toegepast onderzoek.

8. De suggestie van Bongers dat het optimaal gebruik van technologische kennis bepalend zou zijn voor het succes van de Nederlandse windturbinefabrikanten is noch in Nederland noch in verschillende andere landen juist gebleken.

Stelling 12 van Bongers, P.M.M., 1996, Modeling and Identification of Flexible Wind Turbines and Factorizational Approach to Robust Control. TU Delft.¹

9. Directe stimulering van studenten en jonge onderzoekers is belangrijker voor de toekomst van de Technische Universiteit Delft dan de aanstelling van toponderzoekers.

10. Internetten op school is niet cruciaal voor een succesvolle opvoeding.


¹‘De Nederlandse windturbinefabrikanten zullen alleen concurrerend kunnen produceren indien zij optimaal gebruik gaan maken van de technologische know-how.’
Propositions
belonging to the thesis

Dynamics and Design Optimisation of
Offshore Wind Energy Conversion Systems

by
Martin Kühn

1. For large-scale exploitation of offshore wind energy, the development of a mature offshore wind energy technology is of crucial importance. Such development should be based on the experiences with onshore wind energy technology, offshore technology and operation of large power plants.
   This thesis.

2. The proposal of Sinclair and Clayton, to reduce the fatigue loads of an offshore wind energy converter by yawing the turbine from the wind direction into the wave direction, will have the opposite effect.

3. Stall regulated wind turbines, with almost constant rotor speed, are not suitable for large-scale offshore application.

4. Offshore wind energy converters have to be installed onsite with a minimum number of pre-assembled sections. Within five to ten years this number will be between one and three, depending on the configuration.

5. In the long-term, two-bladed offshore wind energy converters will revive.

6. Quality is more important than quantity for the development of offshore wind farms.

7. The rapid technological progress and growth of the international wind energy industry hinders the interest for both fundamental and applied research today.

8. Bongers stated, that the optimum application of technological know-how would be decisive for the success of the Dutch wind turbine manufacturers. Neither in The Netherlands nor in other countries this turned out to be true.

9. The direct stimulation of students and young researchers will be more important for the future of the Delft University of Technology than the employment of top researchers.

10. Internet use at school is not crucial for a successful education.

11. The addition of 'funnies' to chocolate sprinkles ('hagelslag') stimulates sales. It is recommended to apply this sales approach to healthy food, in particular.
DYNAMICS AND DESIGN OPTIMISATION OF OFFSHORE WIND ENERGY CONVERSION SYSTEMS

PROEFSCHRIFT

ter verkrijging van de graad van doctor aan de Technische Universiteit Delft, op gezag van de Rector Magnificus prof. ir. K.F. Wakker, voorzitter van het College voor Promoties, in het openbaar te verdedigen op dinsdag 5 juni 2001 om 10.30 uur

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Dr. G.J.W. van Bussel, Technische Universiteit Delft

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Prof. dr. J.B. Dragt
Prof. Dr.-Ing. R. Gasch
Dr. G.J.W. van Bussel
Prof. dr. ir. G.A.M. van Kuijk
Prof. dr. ir. J.H. Vugts
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SUMMARY

Offshore wind energy will become an important renewable energy source for several countries in Northern Europe. Currently, rapid developments are taking place, with the technical and general economic feasibility having been demonstrated by small-scale prototype projects in sheltered waters during the 1990’s. In 2000, the first offshore wind energy converters in the megawatt class were installed, a technology which will also be employed for large-scale offshore wind farms with 100 to 160 MW capacity by the year 2002. For the medium and long term, multi-megawatt turbines and offshore wind farms in the range up to one gigawatt have been announced.

For exploitation of offshore wind energy on such scale, the development of a mature offshore wind energy technology is of crucial importance. Such development should be based on the experiences with onshore wind energy technology, offshore technology of the petroleum industry and operation of large power plants.

How can analysis of the structural dynamics and a specific design approach, improve the cost-efficiency and reliability of offshore wind energy conversion systems?

This question is the subject of the present Doctoral Thesis and is answered by four contributions:

- Development and demonstration of a design methodology for offshore wind farms
- Synthesis of analysis methods for the structural dynamics; the complexity of which is adjusted to the different stages of the design process
- Investigation of the particular dynamics of offshore wind energy converters and the interrelation with the design
- Demonstration of a design optimisation by means of tailored dynamics

In the following, the main results in these fields are sketched.

After a general introduction and a summary of the state of the art, the integrated design approach for offshore wind farms is introduced. This methodology considers all components as wind turbine, support structure and grid integration as well as installation and operation and maintenance aspects during the design decisions. Interactions between subsystems are considered as completely and practically as possible. Hence, the design solution is governed by overall criteria including levelised production costs, dynamics of the entire system, adaptation to the actual site conditions, installation effort and wind farm availability.

A reference approach for the analysis of bottom-mounted offshore wind energy converters is synthesised from the expertise in both onshore wind energy technology and offshore technology. This enables investigation of the response under simultaneous wind, wave, current and ice loading.

However, this accurate approach in the time domain is too cumbersome for fatigue analyses during the early design stages. Therefore, simplified methods are developed.
The damage equivalent stress ranges from time domain simulation of the aerodynamic response of an offshore wind energy converter in a calm sea are superimposed on the results of a linear spectral analysis of the hydrodynamic response of the support structure. Such separate calculations are acceptable if the aerodynamic damping of the support structure is accounted for. Standard design tools from the communities of wind energy and offshore technology can be applied conveniently for each task. For the certification calculations still integrated, non-linear time domain simulation of the simultaneous response is preferred, however, the scattering of wind and wave parameters results in too many load cases from a practical viewpoint. A low number of 'lumped' load cases can be constructed by another approach.

The support structure, loaded by wind and waves, is emphasised in the field of dynamics since the wind turbine is not significantly influenced by the hydrodynamic excitations. The interrelation of design and dynamics is studied for seven examples for distinct offshore wind energy converters. Three of them were actually constructed. Rated capacities range between 500 kW and 3 MW and environmental conditions span from a sheltered inland sea to exposed North Sea sites. The essentials of offshore specific dynamics include the environmental description, eigenmodes and frequencies, aerodynamic damping, uncertainty of the foundation behaviour and response under fatigue and extreme conditions. Recommendations for the dynamic analyses and design practice are compiled.

The design methodology, the analysis methods and the gained understanding of dynamics are demonstrated during the stepwise development of the design for a 300 MW offshore wind farm.

Application during the European Opti-OWECS study and the realisation of the Utgrunden offshore wind farm proved the benefits of an integrated consideration of both dynamics and design process.
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NOTATIONS AND COORDINATE SYSTEMS

Symbols

\( a \)  
slope parameter in the definition of design turbulence intensity  
[-]

\( a, a' \)  
axial, tangential induction factor  
[-]

\( a \)  
annuity factor in energy cost calculation  
[-]

\( a_n \)  
\( n^{th} \) vibration maxima  
[m]

\( A \)  
constant in the definition of the S-N curve  
[-]

\( A_i \)  
amplitude of \( i^{th} \) elementary wave  
[m]

\( A \)  
state matrix of state space model  

\( B_u, B_v \)  
input matrices of state space model

\( c \)  
local chord length of rotor blade  
[m]

\( c_d, c_l, c_m \)  
aerodynamic drag, lift and moment coefficient  
[-]

\( C_{D_r}, C_{l_m} \)  
hydrodynamic drag, inertia coefficient  
[-]

\( C \)  
output matrix of state space model

\( C_u \)  
undrained shear strength of clay  
[kPa]

\( d \)  
water depth  
[m]

\( dc_l/d\alpha \)  
derivative of the lift coefficient with respect to angle of attack  
[1/rad]

\( d_{gen} \)  
generalized damping matrix  
[Ns/m, Nms/rad]

\( d_{eq} \)  
equivalent viscous soil damping constant  
[Ns/m]

\( d_{horz}, d_{vert} \)  
horizontal, tilt, vertical foundation damping constant  
[Ns/m, Nms/rad]

\( D \)  
fatigue damage  
[-]

\( D \)  
outer diameter  
[m]

\( DC \)  
et net decommissioning cost  
[\( \mathcal{E} \)]

\( D_1, D_2, D_3 \)  
constants in the Dirlik expression  
[-]

\( D \)  
(physical) damping matrix  
[Ns/m, Nms/rad]

\( D_u, D_v \)  
feedthrough matrices of state space model

\( E[] \)  
expected value

\( E_r \)  
levelised annual energy yield  
[kWh/a]

\( f \)  
vector of the external forces  
[N, Nm]

\( f \)  
cyclic frequency, general  
[Hz]

\( f_0 \)  
first fore-aft bending eigenfrequency  
[Hz]

\( f_0 \)  
design value of first fore-aft bending eigenfrequency  
[Hz]

\( \Delta f, \Delta f_{min} \)  
frequency increment of spectral discretisation  
[Hz]

\( f_R, f_b \)  
rotor, blade passing frequency  
[Hz]

\( f_{flap}, f_{tower} \)  
blade flapwise, tower fore-aft eigenfrequency  
[Hz]

\( F \)  
vector function for state derivative of wind turbine model

\( dF \)  
hydrodynamic force per unit length  
[N/m]

\( \Delta F_x, F_x \)  
(discordance of) aerodynamic thrust force  
[N]

\( dF_x \)  
aerodynamic thrust force per unit length  
[N/m]

\( g \)  
gravitational acceleration  
[m/s^2]

\( G_{dyn} \)  
dynamic shear modulus of soil  
[MPa]

\( h_{hub} \)  
hub height with respect to terrain or mean sea level  
[m]
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H$</td>
<td>individual wave height</td>
<td>[m]</td>
</tr>
<tr>
<td>$H$</td>
<td>output vector function of wind turbine model</td>
<td></td>
</tr>
<tr>
<td>$H_{\text{max}}$</td>
<td>extreme wave height</td>
<td>[m]</td>
</tr>
<tr>
<td>$H_{\text{red}}$</td>
<td>reduced wave height according to Germanischer Lloyd</td>
<td>[m]</td>
</tr>
<tr>
<td>$H_s$</td>
<td>significant wave height</td>
<td>[m]</td>
</tr>
<tr>
<td>$l$</td>
<td>total investment cost</td>
<td>[€]</td>
</tr>
<tr>
<td>$I$</td>
<td>identity matrix</td>
<td></td>
</tr>
<tr>
<td>$l_{15}$</td>
<td>design turbulence intensity at 15 m/s mean wind speed</td>
<td>[-]</td>
</tr>
<tr>
<td>$l_G$</td>
<td>generator current</td>
<td>[A]</td>
</tr>
<tr>
<td>$k$</td>
<td>Weibull shape factor</td>
<td>[-]</td>
</tr>
<tr>
<td>$K$</td>
<td>modulus of initial subgrade reaction of soil</td>
<td>[MPa/m]</td>
</tr>
<tr>
<td>$k_{gen}$</td>
<td>generalized stiffness matrix</td>
<td>[N/m, Nm/rad]</td>
</tr>
<tr>
<td>$k_{\text{horz}}, k_{\text{tilt}}, k_{\text{vert}}$</td>
<td>horizontal, tilt, vertical foundation stiffness</td>
<td>[N/m, Nm/rad]</td>
</tr>
<tr>
<td>$K_C$</td>
<td>Keulegan-Carpenter number</td>
<td>[-]</td>
</tr>
<tr>
<td>$K$</td>
<td>(physical) stiffness matrix</td>
<td>[N/m, Nm/rad]</td>
</tr>
<tr>
<td>$K^1$</td>
<td>$K^1$-value, stress range resulting in $D = 1$ for $2\cdot10^6$ cycles</td>
<td>[MPa]</td>
</tr>
<tr>
<td>$L$</td>
<td>wave length</td>
<td>[m]</td>
</tr>
<tr>
<td>$m$</td>
<td>number of selected eigenmodes</td>
<td>[-]</td>
</tr>
<tr>
<td>$m_{\text{gen}}$</td>
<td>generalized mass matrix</td>
<td>[kg, kgm^2]</td>
</tr>
<tr>
<td>$m_n$</td>
<td>$n^{th}$ moment of the auto spectral density, $n \in {0, 1, 2, \ldots}$</td>
<td>[MPa^2/\text{s}^n]</td>
</tr>
<tr>
<td>$M$</td>
<td>(physical) mass matrix</td>
<td>[kg, kgm^2]</td>
</tr>
<tr>
<td>$M_0$</td>
<td>bending moment or torque</td>
<td>[Nm]</td>
</tr>
<tr>
<td>$n$</td>
<td>number of physical degrees of freedom</td>
<td>[-]</td>
</tr>
<tr>
<td>$n(\Delta \sigma), n_j$</td>
<td>number of predicted stress cycles in $j^{th}$ bin</td>
<td>[-]</td>
</tr>
<tr>
<td>$n_{\text{bio}}$</td>
<td>number of stress range bins</td>
<td>[-]</td>
</tr>
<tr>
<td>$n_e$</td>
<td>economic lifetime</td>
<td>[a]</td>
</tr>
<tr>
<td>$n_{\text{total}}$</td>
<td>total number of stress cycles during process</td>
<td>[-]</td>
</tr>
<tr>
<td>$N(\Delta \sigma)$</td>
<td>number of endured stress cycles, S-N curve</td>
<td>[-]</td>
</tr>
<tr>
<td>$N_b$</td>
<td>number of blades</td>
<td>[-]</td>
</tr>
<tr>
<td>$N_R$</td>
<td>number of endured cycles for the reference fatigue strength</td>
<td>[-]</td>
</tr>
<tr>
<td>of the considered S-N curve, i.e. $2\cdot10^6$ cycles</td>
<td>[-]</td>
<td></td>
</tr>
<tr>
<td>$P$</td>
<td>probability</td>
<td>[-]</td>
</tr>
<tr>
<td>$P$</td>
<td>lateral load per unit length of pile</td>
<td>[N/m]</td>
</tr>
<tr>
<td>$q$</td>
<td>generalized coordinate or modal degree of freedom</td>
<td>[m, 1/rad]</td>
</tr>
<tr>
<td>$OM$</td>
<td>annual operation and maintenance cost</td>
<td>[€]</td>
</tr>
<tr>
<td>$Q$</td>
<td>constant in the Dirlik expression</td>
<td>[-]</td>
</tr>
<tr>
<td>$r$</td>
<td>spanwise blade coordinate</td>
<td>[m]</td>
</tr>
<tr>
<td>$r$</td>
<td>test discount rate in energy cost calculation</td>
<td>[-]</td>
</tr>
<tr>
<td>$r$</td>
<td>reference signal of controller in state space model</td>
<td></td>
</tr>
<tr>
<td>$R$</td>
<td>radius of foundation plate</td>
<td>[m]</td>
</tr>
<tr>
<td>$R$</td>
<td>constant in the Dirlik expression</td>
<td>[-]</td>
</tr>
<tr>
<td>$R$, $R_{\text{root}}$</td>
<td>outer blade radius, blade root radius</td>
<td>[m]</td>
</tr>
<tr>
<td>$R_c$</td>
<td>controller matrix in state space model</td>
<td></td>
</tr>
<tr>
<td>$\text{std}$</td>
<td>standard deviation of stress time series</td>
<td>[MPa]</td>
</tr>
<tr>
<td>$S_{\text{so}}$</td>
<td>auto spectral density of stress response</td>
<td>[MPa^2/\text{s}]</td>
</tr>
<tr>
<td>$S_c$</td>
<td>controller matrix in state space model</td>
<td></td>
</tr>
<tr>
<td>$S_i$</td>
<td>$i^{th}$ set of elementary load cases for lumping</td>
<td>[s]</td>
</tr>
<tr>
<td>$t$</td>
<td>time</td>
<td></td>
</tr>
</tbody>
</table>
thickness of ice sheet \( t_{\text{ice}} \) [m]

individual wave period \( T \) [s]

total levelised annual 'downline cost' \( \text{TOM} \) [€]

transformation matrix for static reduction \( T \) [-]

peak period, average duration between peaks \( T_p \) [s]

duration of process or load case \( T_{\text{process}} \) [s]

zero (up-)crossing period \( T_z \) [s]

controlled input vector of state space model \( u \) [m, 1/rad]

displacement \( u \) [m/s]

structural velocity \( \dot{u} \) [m/s²]

horizontal water particle velocity \( \dot{u}_w \) [m/s]

structural acceleration \( \ddot{u} \) [m/s³]

horizontal water particle acceleration \( \ddot{u}_w \) [m/s³]

generator voltage \( U_G \) [V]

external input vector of state space model \( \mathbf{v} \) [m/s]

instantaneous wind speed, in general \( V \) [m/s]

mean wind speed, in general \( \bar{V} \) [m/s]

annual average wind speed \( V_{\text{ave}} \) [m/s]

extreme gust wind speed with return period 50 year \( V_{50} \) [m/s]

mean wind speed for cut-in, cut-out \( V_{\text{in}}, V_{\text{out}} \) [m/s]

hourly mean, three hourly mean wind speed \( V_{1h}, V_{3h} \) [m/s]

annual average wind speed \( V_{\text{ave}} \) [m/s]

apparent wind speed at airfoil section \( W \) [m/s]

dissipated energy during load cycle \( W_{\text{diss}} \) [Nm]

system state vector, vector of physical degrees of freedom \( \mathbf{x} \) [m]

lateral displacement of soil \( y \) [m]

output vector of state space model \( \mathbf{y} \) [-]

roughness length for wind shear model \( z_0 \) [m]

constant in the Dirlik expression \( Z \) [-]

incidence, angle of attack, seabed slope \( \alpha \) [rad]

seabed slope \( \alpha \) [rad]

wind shear exponent in power law \( \beta \) [-]

irregularity factor of broad banded signal, \( T_p / T_z \) [-]

Vanmarcke bandwidth parameter \( \delta \) [-]

strain at 50% of peak stress for clay \( \varepsilon_{SO} \) [%]

peak enhancement factor of JONSWAP wave spectrum \( \gamma \) [-]

parameter in the Dirlik expression \( \gamma \) [-]

safety factor for loads, safety factor for material \( \gamma_F, \gamma_M \) [-]

submerged unit weight of soil \( \gamma_{\text{sub}} \) [kN/m³]

saturated unit weight of soil \( \gamma_{\text{sat}} \) [kN/m³]

gamma function \( \Gamma \) [-]

non-dimensional slope of sea bottom \( \zeta \) [-]

exclusion range at rotor, blade passing frequency \( \zeta_{1P}, \zeta_{2N, P} \) [-]

lower, upper structural uncertainty range of \( f_0 \) [-]

pitch angle \( \Theta \) [rad]

rotational inertia of structure with respect to centre of gravity \( \Theta_{\text{cog}} \) [kgm²]

rotational inertia rotor and nacelle \( \Theta_{\text{top}} \) [kgm²]
\( \Lambda \)  logarithmic damping decrement  \\
\( \mu \)  inverse slope of the S-N curve  \\
\( \nu \)  Poisson ratio of soil  \\
\( \nu_D \)  correction factor for lumped sea state parameter  \\
\( \xi \)  damping ratio as fraction of critical damping  \\
\( \xi_o \)  damping ratio of fundamental mode  \\
\( \xi_{aero} \)  aerodynamic damping ratio  \\
\( \xi_{aero, prod} \)  effective aerodynamic damping ratio during production  \\
\( \pi \)  3.1416  \\
\( \rho_{air}, \rho_w, \rho_{soil} \)  air density, water density, undrained soil density  \\
\( \sigma \)  stress, general  \\
\( \sigma_{ip} \)  ice crushing strength  \\
\( \Delta \sigma \)  stress range  \\
\( \Delta \sigma(N) \)  endured stress range for \( N \) cycles, inverse S-N curve  \\
\( \Delta \sigma_R \)  reference fatigue strength, detail category  \\
\( \Delta \sigma_{eq} \)  damage equivalent stress range associated with \( N_R \)  \\
\( \phi, \phi_i \)  vector of the \( j \)th eigenmode, \( i \)th component of \( \phi \)  \\
\( \Phi \)  internal friction angle of soil  \\
\( \Phi \)  modal matrix  \\
\( \omega \)  circular frequency, general  \\
\( \Omega, \Omega_G \)  rotational speed of the rotor, generator  \\
\( \Omega \)  circular forcing frequency of soil

**Superscripts**

\( \text{red} \)  static reduction  \\
\( ^\wedge \)  amplitude  \\
\( ^\wedge \)  parameter of lumped load case

**Subscripts**

\( a \)  aerodynamic response  \\
\( ah \)  simultaneous or superimposed aerodynamic and hydrodynamic response  \\
\( c \)  controller  \\
\( dyn \)  dynamic degree of freedom  \\
\( eq \)  equivalent  \\
\( G \)  generator  \\
\( h \)  hydrodynamic response  \\
\( i, j, k, l \)  general running indices  \\
\( M \)  master degree of freedom  \\
\( m \)  related to fundamental eigenmode, e.g. damping, frequency  \\
\( o \)  associated with response during parking, idle or failure state  \\
\( prod \)  associated with response during production state  \\
\( R \)  rotor  \\
\( S \)  slave degree of freedom  \\
\( stat \)  static degree of freedom  \\
\( top \)  associated with tower top
**Abbreviations**

**API**  American Petroleum Institute  
**BEM**  blade element momentum theory  
**CD**  Chart datum  
**DAF**  dynamic amplification factor  
**DiBt**  Deutsches Institut für Bautechnik  
**GL**  Germanischer Lloyd  
**HAT**  highest astronomical tide  
**IEA**  International Energy Agency  
**IEC**  International Electrotechnical Commission  
**IFFT**  Inverse Fast Fourier Transform  
**JONSWAP**  Joint European North Sea Wave Project  
**LAT**  lowest astronomical tide  
**LPC**  levelised production cost  
**MIMO**  multi input - multi output system  
**MSL**  mean sea level  
**NESS**  North European Storm Study  
**O&M**  operation and maintenance  
**Opti-OWECS**  Structural and Economic Optimisation of Bottom-Mounted Offshore Wind Energy Converters  
**OWEC**  offshore wind energy converter  
**OWECS**  offshore wind energy converter system, offshore wind farm  
**1P**  rotor frequency  
**N_p**  blade passing frequency  
**pdf**  probability density function  
**PI**  proportional-integral controller  
**PID**  proportional-integral-differential controller  
**PM**  Pierson-Moskowitz  
**PSD**  auto power spectral density  
**RAMS**  reliability, availability, maintainability and serviceability  
**RP1, RP50**  annual return period, return period 50 years  
**SDOF**  single degree of freedom  
**SIMO**  single input - single output system  
**SWL**  still water level
**Coordinate systems**
Unless explicitly stated, the orientation of the coordinate systems are according to Figures 0.1 and 0.2.

![Blade Coordinate System](image)

- $X_B$: Perpendicular to $Z_B$, and pointing out-of-plane towards the tower
- $Y_B$: Perpendicular to blade axis and shaft axis, to give a right-handed co-ordinate system
- $Z_B$: Radially along blade axis
- $M_{XB}$: lead-lag (in-plane) bending moment
- $M_{YB}$: flapwise (out-of-plane) bending moment

**Figure 0.1: Blade coordinate system**

![Support Structure Coordinate System](image)

- $X_T$: pointing South
- $Y_T$: pointing East
- $Z_T$: vertically upwards

Origin at each support structure station. Note that for steady-state calculations the wind, wave, current and ice are deemed to approach from the North.

**Figure 0.2: Support structure coordinate system**
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CHAPTER 1

GENERAL INTRODUCTION

1.1 Design challenges for offshore wind farms

Rationale for offshore wind energy

With world attention focussed on the damaging impact of greenhouse gases, wind energy is emerging as a serious contender for large-scale, cost-effective and clean energy. The case for wind energy has been strengthened in recent years by the significant reduction of its energy cost owing to more mature technology, market forces and incentives. Currently the wind energy industry is subject to large growth rates comparable to mobile telecommunication and other information technologies (Figure 1.1).

However, there is an important and increasing problem constraining further exploitation of wind energy in parts of Europe. Limitations on land use in areas where the population density is high are slowing down the installation of new wind farms. In some countries in Northern Europe the public will no longer accept a significant increase of onshore wind power capacity. The exploitation of the huge offshore wind resource, with considerably less environmental impact than large onshore wind farms, will then become crucial in providing for future green energy needs.

Figure 1.1: Annual development of onshore and offshore wind energy development up to 1999 and predicted growth of world capacity [1.1]
Recent developments and perspectives

Between 1991 and 1997 the first small-scale offshore wind farms, rating two to five MW, were installed in sheltered Northern European waters. They have clearly demonstrated that offshore wind energy is feasible from a technological as well as an economic viewpoint. Already in 1995 the 'Study on Offshore Wind Energy in the EC' showed the large offshore wind resource. These experiences and the generally accepted need for a higher contribution of renewables, has significantly changed people's attitude towards offshore wind energy in the last few years. It sounds realistic when BTM Consult ApS predicts a 10 - 20% contribution of offshore wind energy to the total new installed capacity of 9,000 MW by the year 2004.

The year 2000 marks the start of the commercial development by the installation of first sea-based wind farms of intermediate scale and based upon the current generation of megawatt wind turbines (Figure 1.2). For the year 2002 the commissioning of the first large offshore wind farm with 160 MW capacity is planned in Denmark. For the more distant future, plans have been developed for offshore wind farms in the gigawatt range employing even larger multi-megawatt machines. It is imaginable that the installed capacity of offshore wind power plant may eventually amount to several times that installed on land.

Review of research

To date, research on offshore wind energy has concentrated on meteorological aspects and both the design and dynamic loading of offshore wind energy converters. Until recently most scientific publications have dealt mainly with national programmes for long-term development, resource assessment, design studies or prototypes and dynamics. Three conferences 'Offshore Wind Energy in Mediterranean and Other European Seas' OWEMES 1994, 1997 and 2000 have highlighted the situation. Inventories of the research requirements are presented in references [1.7] and [1.8].

It is mainly to the credit of the European Non-Nuclear Energy Programmes JOULE and THERMIE and the Danish and Dutch governments that first strategic projects have been realised.

Some understanding of design, dynamics and economics of offshore wind energy conversion systems (OWECS) has been gained from various past design studies. Several dedicated research projects are more important. For instance, Quarton and Wastling and Kühn have developed analytical tools for design calculations and highlighted the importance of the dynamics of the entire offshore wind energy converter. Also the compilation of first design guidelines resulted from EU funded research.

Nonetheless, we identified the following technical shortcomings; they form the background of this thesis.

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1 Giving a general overview of the state of the art of offshore wind energy is beyond the scope of this introduction. For such information the reader is referred to the second chapter.
The economic performance of the demonstration plant is often poor due to its small scale, its prototype nature and its lack of design optimisation. Recent design studies have demonstrated the prospects of economies of scale and more mature technology [1.9, 1.11].

The designs and design approaches applied have been governed by direct experience either in onshore wind energy or offshore technology [1.16, 1.17]. Only very few studies have developed solutions devoted particularly to offshore wind energy application [1.18 - 1.20].

Prototype installations were based upon a poor understanding of the dynamics of offshore wind energy converters and its relation to physical and economic performance. At the two Danish offshore wind farms Vindeby and Tønø Knob stiff, heavy and thus expensive foundations were chosen whilst recent studies suggest advantages by softer and lighter solutions and/or by tailored construction and installation techniques [1.19, 1.21]. At the first Dutch offshore wind farm Lely, simultaneous dynamic loading by wind and wave was neglected in the design calculations [1.22] and a too low allowance for uncertainties in soil stiffness led partly to unexpected dynamic characteristics [1.23].

Most of the previous design studies and research projects have examined the relation between dynamics and design insufficiently. Moreover, dynamics have been investigated at prescribed designs, rather than by demonstrating the opportunities of design improvements by tailoring the dynamics.
Some engineers have employed a quasi-static description of aerodynamic and hydrodynamic loads only [1.24]. Other studies have considered fatigue in an overly simplistic manner, i.e. either by taking into account only aerodynamic loads, [1.21, 1.25], or using rough approximations of combined aerodynamic and hydrodynamic fatigue [1.16, 1.26].

By contrast, analyses of integrated models of an offshore wind energy converter have gained a basic understanding of the simultaneous response under wind and wave excitation. In 1988, Oscar and Paez have shown the general effect of wind and wave excitation on the response by Finite Element Analyses [1.27].

The attention was directed at the aerodynamic damping of the support structure by Sinclair and Clayton who analysed a single degree of freedom model using a semi-empirical approach [1.28].

Quarton and Wastling have extended two design tools for onshore wind energy converters in the time domain and the frequency domain [1.13]. This facilitated the investigation of three designs tested both in fatigue and extreme loading conditions. It has been concluded that dynamic wave loading has little influence on the wind turbine but that the support structure suffers dynamic wind and wave loads depending on, among other parameters, its fundamental structural frequency.

Halfpenny [1.29] and Henderson [1.30] applied frequency domain methods to the analysis of floating multi-unit offshore wind farms.

During the year 2000 common onshore wind turbine design tools as PHATAS, TURBU, FLEX4 and HawC have been extended for offshore application; however, few results from their application have been published yet [1.31 - 1.33].

Until now, analytical models of the dynamic behaviour of offshore wind energy converters have not been validated by testing of actual plant. The few existing structural measurements have dealt with dynamic loading due to ambient and wake induced turbulence only [1.34], the eigenfrequencies [1.23] and loading of monopile support structures [1.35]. More results are expected from ongoing research projects under the Fourth European Framework [1.36].

1.2 Research objectives and methodology

Based upon the previous discussion we can state the main question of the present Doctoral Thesis as:

How can analysis of the structural dynamics and a specific design approach, improve the cost-efficiency and reliability of offshore wind energy conversion systems?

Considering the three interrelated and overlapping fields of the dynamics of offshore wind energy converters, the design and the optimisation of offshore wind farms, the specific objectives of this study include:

- Development and demonstration of a design methodology for offshore wind farms
- Synthesis of analysis methods for the structural dynamics; the complexity of which is adjusted to the different stages of the design process
- Investigation of the particular dynamics of offshore wind energy converters and the interrelation with the design
- Demonstration of a design optimisation by means of tailored dynamics.
Figure 1.3: Location of the research in the intersection of the three fields: dynamics, design and economic optimisation

Figure 1.3 illustrates both the overlap between the three scientific fields and how particular chapters of the thesis respond to the four objectives.

Development of the research project

The third of the above-mentioned goals, the investigation of the was stated at the beginning of the research in 1992/1993. However, the three other objectives evolved during the course as a consequence of the work itself and the rapid development of the field of offshore wind energy.

The need for simplified, but not simplistic methods, suitable for application during the early design stages of an actual design process was recognized after developing and using a scientific analysis approach for the dynamics of OWEC which fits only to the final design phase.

A broader consideration of the design problem followed from the coordination of the European research project Opti-OWECS ‘Structural and Economic Optimisation of Bottom-Mounted Offshore Wind Energy Converters’ (JOR3-CT95-0087) [1.11]. Offshore wind energy was identified as a techno-economic rather than as a primarily technical challenge; thus, design solutions require (economic) optimisation.

Furthermore, attention was directed to an integrated design approach considering offshore wind farms as a whole and to coordinate knowledge in order to utilize the valuable but partly contradictory experience from the parent technologies, i.e. (onshore) wind energy, offshore technology and power management.

The Utgrunden project (Figure 1.2) offered the opportunity for us to apply parts of the research to a real offshore wind farm during the years 1999 and 2000 [1.37].
**Applied methodology**

Obviously different methodologies are appropriate to each of the three areas. Dynamics were investigated by means of computational studies. Due to the lack of existing offshore wind farms at exposed sites and of suitable measurements this was the only possibility available. Nonetheless, studying two eigenfrequency measurements at the Dutch Lely plant revealed important insight on monopile foundations. A major part of the work concerned extending the onshore wind turbine design tool in the time domain DUWECS (Delft University Wind Turbine Simulation) [1.38] by introducing the features required for the description of the offshore environment and structural response. Semi-empirical methods laid the foundation for the development of simplified fatigue analysis approaches.

The integrated OWECS design approach was developed analytically and demonstrated during the Opti-OWECS project. Furthermore, it was given a firm base by applying a design methodology recently developed for large, complex civil engineering projects [1.39].

Optimisation of actual designs was demonstrated by applying methodologies developed in the parent technologies. It was beyond the scope of the study, and perhaps not possible at all, to establish a mathematical model of the optimisation problem appropriate to actual design practice. By contrast, emphasis was put on demonstrating the required steps during the different design stages and by introducing approaches that meet the requirements of these phases.

**1.3 Outline of the thesis**

Four chapters, each corresponding with one of the above-mentioned objectives (Figure 1.3), establish the core of this study. These major parts are embedded in a sequence of further chapters as shown in Figure 1.4.

The framework of the research is outlined by this general introduction (Chapter 1). Chapter 2 'State of the Art of Offshore Wind Energy' provides a comprehensive overview and can be read independently from the rest of the thesis. Readers who are not familiar with the general background might use it as a starter before consuming the main course. The integrated design approach is introduced in Chapter 3. It establishes a general philosophy for innovative designs but can also be considered as a rationale for the following elaboration.

The next chapters can be read in the actual sequence, however, they build up two parallel development lines which may be considered independently corresponding to personal taste and interest. Modelling of the environment and the OWEC itself and the interrelation of dynamics and design are investigated in the first branch. Secondly, appropriate approaches for the dynamic analysis are developed in Chapter 6 to 8.

Chapter 10 'Design Optimisation by Means of Tailored Dynamics' presents the synthesis of the two lines in the light of an integral treatment. Finally, the thesis rounds off with the conclusions in Chapter 11.

Appendices with supporting information may serve as desert.
Special reference is given to Appendix A, which explains the terminology of this thesis. It has been developed and used successfully during the Opti-OWECS project. In order to avoid misunderstandings there are two conventions that must be appreciated. Firstly, the acronym ‘OWECS’ (standing for offshore wind energy conversion system) and its synonym ‘offshore wind farm’ describes the entire system, that is the wind turbines, the support structures, the grid connection up to the public grid and the operation and maintenance aspects. Secondly, ‘OWEC’ (offshore wind energy converter) is used to refer to a single unit of an offshore wind farm comprising support structure (i.e. tower and foundation) and the wind turbine (i.e. aero-mechanical-electrical conversion unit on top of the tower).

![Diagram of Thesis Outline](image-url)
CHAPTER 2

STATE OF THE ART OF OFFSHORE WIND ENERGY

The development of renewable energies has been identified as a key issue of the new century and ambitious national and international plans for the reduction of greenhouse gases have been announced. Offshore wind energy will be important at least for several northern European countries since it has the potential for a considerable contribution to the energy supply (Figure 2.1). Generally the prospects are assessed very positively and the commercial development of a probably big market has commenced.

In this chapter it is tried to elaborate the state of the art of offshore wind energy with respect to the dimension of the subject. We are dealing with five key areas which have been defined with respect to scientific considerations: ¹

- Technology of offshore wind farms
- Integration and financing of large-scale wind farms in the international energy system
- Resource assessment and economic potential
- Social acceptance, environmental impact and politics
- Market activities, development targets and prospects

Together with the résumé we formulate a number of open questions. Their solution will be decisive for the future development.

![Diagram showing estimated offshore wind energy potential in the European Union (except Finland and Sweden)](image)

(1989 European annual energy consumption 1727 TWh)

**Figure 2.1:** Estimated offshore wind energy potential in the European Union (except Finland and Sweden) [2.1]

¹ A similar approach is currently applied within the European project 'Concerted Action on Offshore Wind Energy in Europe' (NNE5-1999-00562) which aims at a thorough assessment of all the critical issues (continued...)
2.1 Offshore wind farm technology

Since the early interest in offshore wind energy in the 1970's, there have been a number of significant advances in the understanding and the technology. After briefly introducing four general design requirements, the individual elements of the technology are presented. On this background we are able to review the development so far and outline one possible future evolution of a mature technology in four generations.

2.1.1 Offshore design requirements

The environmental impact and the demand of large power plants cause four distinct design requirements of offshore wind farms:

- Integrated design of offshore wind farms

More cost-efficient and reliable design can be achieved by an integrated design approach considering the entire offshore wind farm as one system (Chapter 3). Eventually optimum offshore design will differ from land-based design due to the different distribution of the investment costs on the major subsystems and the greater importance of operation and maintenance aspects.

- Design for RAMS (Reliability, Availability, Maintainability, Serviceability)

Operation and maintenance (O&M) aspects are a main design driver for offshore wind farms. Hindered access owing to bad weather after a fault results in longer down-time and loss of production. Therefore, at sea the reliability must be even better than the present good performance on land. Together with dedicated operation and maintenance strategies and hardware, acceptable availability and reasonable O&M cost can be achieved.

- Smooth grid integration and controllability

Future large-scale offshore wind farms will be operated as power plants in the international energy system. Fast controllability of the cluster as a whole as well as of the individual turbines, for instance a power reduction by 80% within 2 s, are demanded. Short power outages in the public high-voltage grid may not cause the entire wind farm to drop out of production [2.3]. Among other considerations this is ruling-out robust turbines with traditional stall control and almost constant rotor speed and to a lesser extent fixed-speed, active-stall concepts. Extra cost and design effort are needed to ensure high reliability of the more complex designs with variable-speed, variable-pitch which are favourable for power management and load control.

- Optimised offshore installation

Large costs but also a considerable potential for optimisation is related to installation of offshore wind farms. Eventually installation procedures will be optimized until the entire

\[1\] (...continued)

Concerning the exploitation of offshore wind energy in Europe. The 17 participants from 13 countries represent the utility sector, wind farm developers, engineering consultants, national agencies, research institutes and universities. Conducted by a team of experts in the field, the work addresses the technological, economic and socio-economic issues, and canvasses opinion from the financial, industrial, environmental and political communities. The beginning of the two years project was initially proposed for 1998 but took place in the summer 2000 [2.2].
offshore wind energy converter including foundation is installed by only one to three major offshore operations.

2.1.2 Components of the technology

Wind turbine

It is clear that the offshore environment poses particular requirements for wind turbines which are not relevant to onshore machines. It is, however, also evident that in the present state of offshore technology, the wind turbines above the height of wave impact are little different from onshore machines. Currently two approaches are followed to adapt the wind turbine design, i.e. marinisation of robust and proven onshore solutions and the more radical design consideration of the offshore requirements.

Figure 2.2 shows an example of the marinisation approach, the Enron Wind 1.5 offshore realised at the Utgrunden wind farm in 2000 (Figures 1.2 and 2.5). Contenders of the latter category are lightweight, high-tip-speed designs [2.4] and RAMS integrated designs which were investigated for instance by the European Opti-OWECS study [2.5].

Ongoing rapid growth of the machine size and the requirement of proven technology favour horizontal axis, upwind orientated wind turbines more or less similar to land-based solutions. Currently three-bladed designs are used while advanced concepts with two blades might been applied at a later stage when a more mature level of the offshore wind energy technology will be achieved. The simplified installation and maintenance and the opportunity for higher tip-speed ratios offshore and associated lower drive train loads favour two-bladed machines while the complicate dynamic behaviour and the high fluctuating loads require sophisticated solutions.

![Diagram of wind turbine components]

**Figure 2.2:** Example of some marinisation features of the *Enron Wind 1.5 offshore* used at the Utgrunden project

(Dimensions: length 8.8 m nacelle + 3.65 m hub, width 3.55 m, height 3.8 m)
To date the minimum size of a single offshore turbine is the so-called megawatt class with a rated power of 1.5 to 2 MW. However, large multi-megawatt units are under development (Section 2.1.3).

As above mentioned a strong tendency towards variable-speed, variable-pitch machines is observed and the further prospects of active stall turbines are questionable. With respect to the drive train and generator concept the diversity is still present and even increasing. The traditional concepts combine either a multi-stage gear box with a high speed generator or no gear box (‘direct-drive’) with a low speed, electromagnetic multi-pole generator. Innovative designs have been proposed which match a single stage gear with an intermediate speed generator (Figure 3.13) or use permanent magnet generators and DC power transmission.

We come back to the wind turbine technology in Section 2.1.3 when presenting a projection of the future development.

**Support structure**

With respect to bottom-mounted support structures Ferguson introduced a useful classification by distinguishing their features with respect to three basic properties: structural configuration, foundation type and installation principle [2.6]. Any structural concept including mono-tower, braced or lattice tower can be reasonably combined with either a piled and gravity foundation. From a logistic point of view it is, however, beneficial to match piled designs with lifted installation and gravity foundations with floated installation (Figure 2.3).²

Design optimisation with respect to local site conditions, batch size and the particularities of offshore wind energy converters offers large cost reduction (Chapter 10). The dynamic characteristics of the support structure is of particular importance and a compromise between the opposing requirements from the wind energy and offshore communities has to be found. Wind turbine towers in the megawatt class are slender and flexible structures with a first eigenfrequency below 0.45 Hz. Common offshore structures employ a significant higher natural frequency since dynamic wave loading is strongly reduced for structural frequencies above approximately 0.4 Hz.

Floating offshore wind farms have been proposed for water depths beyond, say, 60 m when bottom-mounted designs are not viable and the water depth is large enough to enable efficient moorings. The highest proportion of capital costs is absorbed by the moorings accounting, for instance, up to 28% in case of a proposed floating wind farm with 1.4 MW turbines mounted on spar buoys [2.7]. Therefore innovative concepts as the Multi Floating Unit Offshore Wind Farm (MUHOW) have been proposed which reduce the wave induced response and both mooring and cable costs (Figure 2.4) [2.8]. Nonetheless they are subject to extra material, construction and construction-site selection costs.

Recently Henderson drew the following sceptical résumé [2.9]. ‘All costing exercises into floating wind parks have concluded that the costs will be at least twice that of ground

² The floated installation of the tower and wind turbine on a lifted monopile is an interesting exception of this classification.

³ Three design solutions exist for wind turbine towers depending on the ratio between the fundamental eigenfrequency $f_o$ and either the rotor frequency $f_r$ or the blade passing frequency $f_b = N_b \cdot f_r$: soft-soft, i.e. $f_o < f_r$, soft-stiff, i.e. $f_b < f_o < f_r$ and stiff-stiff, i.e. $f_b < f_o$. In practice, stiff-stiff designs are no longer built for the current generation of wind turbines.
Figure 2.3: Classification of bottom-mounted support structure types by structural configuration, foundation and installation principle.

Figure 2.4: Multi Unit Floating Offshore Wind Farm (MUFOW)
based projects. Hence, if the concept is to progress beyond niche markets major conceptual advances must be found. Combining with alternative energy sources, such as wave or gas power is one possibility, as is generating an alternative higher value fuel, such as hydrogen.¹

Sometimes the re-use of decommissioned platforms of the oil and gas industry is discussed. The viability of such isolated and small-scale solutions, if any, will most likely be based upon the delay of the high dismantling cost of the original structure.

*Installation, decommissioning and dismantling*

Scarce equipment with high mobilisation costs, weather delays and safety requirements make offshore operations about 5 to 10 times more expensive as on land. The logistics required for such activities are illustrated by Figure 2.5 showing four of the seven vessels and barges used for the installation of the Utgrunden wind farm. Therefore especially installation and to a lesser extent decommissioning and dismantling are design drivers of offshore wind farms. Minimisation of in-situ work, e.g. by reduction of site preparation work or commissioning of the entirely equipped offshore wind energy converters at the construction site will be important for the future success. Dismantling and depositing of gravity foundations might reduce the prospects of such designs as observed recently in the oil and gas industry concerning European and North-Atlantic site. Complete removal of piles and submarine cables below the sea bottom, if environmentally meaningful and legally required, might cause considerable costs.
Electrical transmission and grid connection

There are various possible approaches to the design and installation of systems for power collection, transmission to shore and grid connection. Figure 2.6 gives the basic options for the grid connection. No real technical restrictions are foreseen because nowadays modular electronic components are available for a wide variety of applications. The challenge lies apart from the economics in the integration into the energy system which is discussed in Section 2.2.

The main choice which has to be made is between an AC or a DC connection to shore. AC transmission involves high dielectric losses (the isolation material acts as a capacitor); these losses are proportional to the cable length and the voltage. DC transmission requires expensive converters. For short distances AC transmission is the most cost effective option. The crossover point depends on the costs of the components involved and is situated typically around 60 km.

HVDC (High Voltage DC) transmission systems have been increasingly used in recent years to transport electricity from remote energy sources to the distribution grid. At present the maximum capacity is 600 MW; 1,000 MW is expected to be feasible at approximately the same cable cost per unit length by the year 2015 [2.10]. The progress in power electronics has brought up Medium Voltage DC systems, also denoted as 'DC-light', which might be interesting for medium size wind farms or the power collection within clusters.

<table>
<thead>
<tr>
<th>Connection to shore</th>
<th>AC</th>
<th>DC</th>
</tr>
</thead>
<tbody>
<tr>
<td>[A2]</td>
<td>![Diagram A2]</td>
<td>![Diagram C]</td>
</tr>
<tr>
<td>[B]</td>
<td>![Diagram B]</td>
<td>![Diagram D]</td>
</tr>
<tr>
<td>[C]</td>
<td>![Diagram C]</td>
<td>![Diagram C]</td>
</tr>
<tr>
<td>[D]</td>
<td>![Diagram D]</td>
<td>![Diagram D]</td>
</tr>
</tbody>
</table>

**Figure 2.6:** Basic grid connection options

For the power collection there also has to be made a choice between AC and DC. The first two options, A1 and A2, are the ones commonly used for onshore farms. The layout according to option B has been used at some few onshore farms. Option C, AC coupling of all wind turbines together with a DC connection to shore, may cause technical problems with respect to achieving stable operation in the 'AC island'. DC power collection and connection to shore, option D, is elegant but meaningful only if equipment for transformation of DC voltage for reasonable cost has become available.
ABB in cooperation with ScanWind have announced the development of a machine concept called 'WindFarmer' in this category.

Today generators operate with AC as the public grid does. This means that, where an intermediate DC link is used, both AC/DC rectifiers and DC/AC inverters are required. The converter stations consist, amongst other items, of thyristor switches. They have to be placed in series because they can only switch a limited voltage (8kV). With developments in semi-conductor technology, it is expected that the voltage which can be switched by one thyristor will grow gradually. This means lower cost at equal power and lower energy losses. IGBT (insulated gate bipolar transistor) switches can be used alternatively.

**Operation and maintenance (O&M)**

The operation and maintenance requirements of offshore wind farms are crucially important in the context of the overall cost effectiveness of offshore wind energy. Poor performance and resulting low availability can offset economies of scale of large offshore wind farms and multi-megawatt units. Two striking differences with onshore wind farms are the largely reduced accessibility during bad weather conditions (wave height, wind speed and visibility) and the costs of transport or lifting operations.

The demonstration offshore wind farms in sheltered waters show availabilities close to the onshore situation. Unfortunately this positive experience is not representative for real offshore conditions. Therefore in the scope of the Opti-OWECS project the operational and maintenance behaviour of large wind farms at remote offshore sites was analysed by Monte-Carlo simulations including random generation of failures and weather conditions [2.11]. Figure 2.7 gives one qualitative example. In the upper diagram the wind farm availability is shown as a function of both the fraction of time when of the wind turbines are accessible and the reliability or mean time between failure of the design. A site without restriction with respect to accessibility (i.e. onshore) reaches 97% availability for a state of the art design increasing up to 99% for an extremely reliable design under an adequate maintenance strategy. As weather conditions get worse availability may fall down to 53% to 81% depending upon the failure rate and maintenance approach.

The lower plot indicates the normalised energy cost for 20 years loan and 5% test discount rate as a function of accessibility and reliability. The Opti-OWECS design solution with 85% accessibility and improved reliability class is taken as 100%. The assumption is made that the energy cost at 100% accessibility increases with increasing reliability by approximately 9% due to the higher wind turbine investment. Harsher weather conditions with better annual average wind speed will simultaneously lead to a decreased accessibility and increased gross energy yield. With state of the art reliability, cost values below 106% are not reachable since the extra gross energy yield is more or less compensated by the lower availability. However, improved reliability will

---

4 Reliability data for the state of the art were derived from a large data base with turbines in the 500/600 kW class. The improved reliability level can be achieved with existing technology but extra expenses for higher specifications, partially redundant components and dedicated O&M strategies. Innovative technology and some years time for development and testing are required for the highly improved reliability class.

5 Gross energy yield for 100% availability, the actual energy yield results from multiplication with the real availability.
Figure 2.7 Wind farm availability (upper) and energy cost (lower) as function of the reliability of the wind turbine design for sites with different accessibility and wind resources.

(Energy cost for constant cost for support structure and grid connection)

find its pay back in terms of significantly reduced energy cost. Between 95% and 85% accessibility the improved reliability class shows the lowest energy cost whilst for more exposed sites the significant efforts for a highly improved reliability are cost effective.

Standards and regulations

The particularities of offshore wind energy limit the prospects for direct application of existing onshore wind energy and offshore oil and gas expertise. The commercial success of large international projects depends also upon sound technical criteria. Establishment and harmonisation of specific standards has not progressed very far, yet. In 1995 Germanischer Lloyd compiled first 'Design Recommendations for Offshore Wind Energy Conversion Systems' [2.12] which are based upon a combination of onshore
wind energy regulations, standards for fixed offshore installation and some generic offshore wind energy knowledge [2.1]. Revision and possible extensions are part of a recent European research project to be finished in 2002 [2.13]. In Denmark design bases have been established which orient at the specific requirements of the first national, large-scale projects [2.14]. The Working Group 3 of the Technical Committee 88 of the IEC has started the preparation of a separate offshore extension of the IEC 61400 series to be in force by the summer 2005.

2.1.3 Review and projection of technological development

The large success of wind energy during the last 15 years was possible due to the gradual and evolutionary development of the technology and both market forces and public incentives. The same is expected for offshore windpower, which stimulates us to project four illustrative technology generations.

Twenty years between initial ideas and first demonstration projects

First ideas on electricity generation from the wind on the oceans date back to the early 1970's [2.15]. Here we can give only a brief overview of the development. Matthies et al. [2.1] and Beurskens [2.16] provide more comprehensive information. The initial feasibility studies on large offshore wind farms were undertaken in the USA, the United Kingdom, Sweden, the Netherlands, Denmark and Germany between the mid 1970's and the mid 1980's. One general reason why offshore wind power plants were not installed in any of the countries concerned was that large-scale, commercial wind turbines were not available at that time [2.17]. Moreover, some early studies indicated too high cost in comparison with other energy sources as coal, gas and nuclear. Between the mid 1980's and early 1990's research was mainly focused on evaluating the resources and the economics of offshore wind energy. In parallel with these studies, onshore wind energy became of age evolving into a mature technology.

Prototype generation of the technology in the 500/600 kW class

Between 1990 and 1997 in Denmark, the Netherlands and Sweden the first (semi-)offshore wind farms were installed and behaved well, particularly in view of their prototype nature (Table 2.1). More or less standard machines of the 500/600 kW class were placed in sheltered waters for demonstration purposes. The projects and the converter units were too small to be competitive. The unexpectedly high energy yield of the second Danish farm at Túno Knob seems to have been something of a landmark highlighting offshore wind energy as a feasible and potentially cost effective resource. Research projects on Danish, Dutch and European level were initiated to start the development of the required technology and tools for accurate prediction of the wind climate. Utilities and other commercial enterprises from both within and outside the wind energy community began to seriously consider commercial offshore wind farms. It was also realised that in certain areas of Northern Europe the public acceptance of onshore wind power projects may reach a saturation point within the foreseeable future. In respond to this emerging barrier for further wind energy development, several national energy agencies, e.g. in Denmark and the Netherlands, as well as organisations such as Greenpeace drew up scenarios for the large scale exploitation of the offshore resource with projections of installed capacity in the range of 4 to 10 GW per country by the year 2020 to 2030 [2.18].
**First commercial generation: Marinised megawatt class**

The year 2000 marked the beginning of a new phase by installation of three small to medium scale projects (Table 2.1) which concepts will also be used at the first large-scale projects in the range of 100 to 160 MW to be realised in 2002 and 2003. The megawatt class rated 1.5 to 2 MW power and using diameters up to 80 m establishes the first generation of commercial offshore wind energy converters based upon marinised onshore designs. This is a reasonable choice since the current offshore market is too small for separate offshore designs but megawatt turbines are very well suited owing to their onshore track record and being the largest commercially available machines, yet.

Marinisation is related to various gradual design modifications as maintenance aids enabling on-site repair of most components, improved nacelle climate and corrosion protection (Figure 2.2) [2.19]. Some manufacturers introduced more significant changes

<table>
<thead>
<tr>
<th>Location</th>
<th>Year</th>
<th>Power [MW]</th>
<th>Rotor diameter [m]</th>
<th>Hub height (MSL) [m]</th>
<th>Water depth, distance from shore</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nogersund, Baltic, SE</td>
<td>1990</td>
<td>1 x 0.22 = 0.22</td>
<td>23</td>
<td>37.5</td>
<td>= 7 m 0.25 km</td>
<td>hybrid tripod with gravity bases, abandoned in 1998 (Figure 3.9)</td>
</tr>
<tr>
<td>Vindeby, Baltic, DK</td>
<td>1991</td>
<td>11 x 0.45 = 4.95</td>
<td>35</td>
<td>37.5</td>
<td>3 - 5 m 1.5 km</td>
<td>concrete caisson</td>
</tr>
<tr>
<td>Lely, IJsselmeer, NL</td>
<td>1994</td>
<td>4 x 0.5 = 2</td>
<td>40.77</td>
<td>40</td>
<td>5 - 10 m 0.75 km</td>
<td>driven monopile, fresh water (Front cover)</td>
</tr>
<tr>
<td>Tunø Knob, Baltic, DK</td>
<td>1995</td>
<td>10 x 0.5 = 5</td>
<td>39</td>
<td>43</td>
<td>3.1 - 4.7 m 6 km</td>
<td>concrete caisson (Figure 3.6)</td>
</tr>
<tr>
<td>Dronten, IJsselmeer, NL</td>
<td>1996</td>
<td>28 x 0.6 = 16.8</td>
<td>43</td>
<td>50</td>
<td>= 5 m 0.02 km</td>
<td>driven monopile, fresh water</td>
</tr>
<tr>
<td>Bostigen Valar, Baltic, SE</td>
<td>1998</td>
<td>5 x 0.5 = 2.5</td>
<td>37</td>
<td>= 43</td>
<td>5.5 - 6.5 m 3 km</td>
<td>drilled monopile</td>
</tr>
<tr>
<td>Blyth Harbour, North Sea, UK</td>
<td>2000</td>
<td>2 x 2 = 4</td>
<td>66</td>
<td>58</td>
<td>8.5 m (MSL) 0.8 km</td>
<td>drilled monopile</td>
</tr>
<tr>
<td>Utgrunden, Baltic, SE</td>
<td>2000</td>
<td>7 x 1.425 = 10</td>
<td>70.5</td>
<td>65</td>
<td>7.2 - 10 m 8 -12.5 km</td>
<td>driven monopile (Figures 1.2 and 2.5)</td>
</tr>
<tr>
<td>Middelgrunden, Baltic, DK</td>
<td>2000/2001</td>
<td>20 x 2 = 40</td>
<td>76</td>
<td>64</td>
<td>3 - 6 m 3 km</td>
<td>concrete caisson</td>
</tr>
</tbody>
</table>
as introduction of full variable speed instead of limited speed variability or fixed speed.

Second commercial generation: Multi-megawatt designs for onshore and offshore siting

The trend to even larger machines, especially on the German market, still continues. The next turbine generation rated 2.5 to 3.5 MW with rotor diameters of 90 m and more is currently on the drawing boards. The target markets onshore and offshore will probably be served by site optimized variants. One 2.5 MW – 80 m onshore prototype was erected at the beginning of the year 2000 and other larger machines are expected in 2001 and early 2002 serving the second generation of offshore wind energy converters (OWEC).

For instance, Enron Wind is currently developing a new platform in the 3 MW class with different ratings for offshore and onshore siting. Many offshore requirements, e.g. higher reliability of components, exchange of all major components with build-in craneage, capsulation of the nacelle are considered already in the conceptual design. Many of these offshore triggered features will be beneficial on land as well.

Third commercial generation: Dedicated multi-megawatt offshore designs

In our opinion the third OWEC generation rated 5 MW and larger with rotor diameters of approximately 110 to 120 m will be applied more or less exclusively offshore. Siting on land, apart from prototype installation, will be difficult among other reasons due to the required infrastructure for transport and erection. The offshore wind farm proposals, to be materialized during and after the second half of the present decade, are so large that such dedicated designs will be fully viable and will be required. Furthermore particular machine concepts, e.g. very high tip speed designs or two-bladed rotors, might not be fully compatible with onshore requirements [2.4]. Also it might not be optimal to design such large rotor blades for the higher turbulence, wind shear and more pronounced directional changes onshore.

Some manufacturers claim to be able to jump directly from the megawatt class into the 4.5 - 5 MW range. However, time will show whether such an enormous step is technically and commercially feasible or an intermediate stage, as planned for instance in the Dutch ‘DOWEC’ project, is more realistic [2.20].

During the development of the second and third commercial OWEC generation an integrated design approach considering the offshore design requirements will be increasingly followed. Development of turbine size and technology will be most crucial but similar innovations are expected and required with respect to support structure design, installation and grid integration.

2.2 Grid integration, energy supply and financing

Integrating thousands of megawatt of offshore wind energy into the electricity system presents a great challenge to the transmission grid. The present power and heat production system in many countries is dominated by production from large-scale plants, centrally located in areas with great population density, and to a lesser extent by small-scale decentralized plants.

Massive development of offshore wind power means that electricity production from wind turbines will lead to considerably greater periodic imbalances between production
and consumption than today. In addition the expected capacity variations from wind farms are considerable. A balance must be struck between production, exchange with countries abroad and consumption at any point in the day or night for reasons of operational security in the electricity system. This means that other elements in the electricity system will have to regulate the capacity variations caused by the integration of large numbers of offshore wind farms.

Various types of countermoves to the periodic imbalance between production and consumption of electricity are possible and are under development, for example:

- Shifting times of electricity production/consumption
- Pumped storage power stations
- Geographically spread production
- Converting electricity consumption to other power sources
- Plant adjustments and changed operations
- Development of innovative operation patterns by existing and new utilities
- Savings in heating buildings
- Foreign trade.

Reliable prediction of expected wind energy generation within the next 24 to 36 hours will be important and can result in a significant price bonus [2.21, 2.22].

The financing of large offshore wind farms has to face different considerations as commonly valid for onshore wind energy. Investment costs are generally higher due to both the additional costs of offshore siting and the farm size. This together with the generally longer economic lifetime and the larger uncertainties in the plant performance results in a higher economic risk for the investor. The size of large onshore wind farms, e.g. in Germany, Spain or the USA, is growing rapidly. Banks and financing bodies seem to be prepared for large offshore projects in a due course.

2.3 Resources and Economics

Resources

It is clear from studies already undertaken that the offshore wind energy resource is vast. In 1995 the 'Study of Offshore Wind Energy in the EC' [2.1] quantified the total technically feasible European resources for bottom-mounted wind farms up to 40 km offshore and 30 m water depth. Maps of the annual average wind speed for the coastal European waters were computed and the potentially available areas for offshore wind energy were summed up. Locations with too large water depth, distance to shore and seabed slope were excluded as were areas for traffic lines, oil and gas exploitation, nature conservation and military usage. The theoretical potential was estimated to exceed the total annual electricity consumption (Figure 2.1). Some countries, namely the United Kingdom, Ireland, Denmark and France are much richer in available energy resources than others, e.g. Germany. Assuming a, say, 10% realisation of the total potential results in a supply of 170%, 31%, 18% and 13.5% of the electricity demand in the UK, Denmark, the Netherlands and France compared to 5% in Germany. The extent and cost of exploiting the resource available to a particular country will depend on the site conditions as well as the legal and economic constraints. During the Opti-OWECS project, European maps with indicative energy cost were produced and showed large resources at reasonable costs for the windy Northern European coastlines and water depth up to, say, 25 m and moderate distance to shore [2.23].
Economics

Various studies and offshore cost models suggest generally promising scenarios for offshore wind energy developments. This represents a major change from an earlier pessimistic view of offshore economics. It is now generally recognised that there is compensation for the added cost of an offshore infrastructure in better annual average wind speeds and lower wind turbulence offshore as compared with available land-based sites that may service the same population area. Offshore, typical capacity factors, i.e. the proportion of actual power generation to total potential generation, range 30 to 35% whilst currently 23% is achieved on land [2.24]. Comparison of studies and actual realised projects in Figure 2.8 shows a dramatic and continuous decrease in energy costs towards a level of other energy sources if equal financial conditions are applied and site conditions are good. For instance, typical energy costs based on 5% test discount rate and a repayment period of 20 years of coal-fired and gas-fired plant range in the order of 3.7 to 5.5 €ct/kWh and 3.1 to 4 €ct/kWh, respectively [2.25]. In reality the loans for present wind energy projects are shorter and conventional plants are amortised over a significantly longer period.

The cost breakdown of the energy cost of a 300 MW offshore wind farm proposed by the Opti-OWECS project is given in Figure 2.9. Operation and maintenance costs have a significant contribution. Optimum energy cost required high availability, which could in turn only be achieved with a continuous and high O&M effort and permanent disposal of heavy maintenance equipment. The Opti-OWECS study identified the average wind speed of the site as the major driver for the energy costs. While this may seem an unsurprising, indeed obvious conclusion, it is difficult to find it in the published literature that more often refers to water depth and distance from shore as design drivers. Other important environmental parameters are the distance from the shore, the distance from electricity grid infrastructure, and the water depth. Of the 'controllable' drivers, the most significant are the operation and maintenance strategy adopted for the farm, along with the rated capacity of both individual OWEC units and the whole wind farm. The design wave height and soil conditions can have a smaller, although still significant influence on the farm economics. This has a number of consequences. Excellent wind conditions are essential for the economic viability of offshore wind farms. Estimation of the long term wind conditions require great care if reliable energy cost predictions desired. Only if high availability is guaranteed, the full potential of exposed sites can be exploited. It would be economically worthwhile to invest a relatively large amount of capital in producing a reliable design with a high energy output. For instance, an increase in gross energy production or availability of 10% would bring an economic improvement even if it requires 30% higher investment for the turbine.

2.4 Public acceptance, local environmental impact and politics

The first offshore wind farms with 4 to 11 wind turbines in the 500 kW class have been regarded by the public as insignificant, test objects and the public interest has not been of any importance. Today the public interest has changed considerably as the results from the first wind farms are indicating a large potential for offshore wind energy. Typical
Figure 2.8: Comparison of energy costs between recent studies and projects

Figure 2.9: Contributions to energy costs of a 300 MW offshore wind farm in the Dutch North Sea
future farms are expected to comprise hundreds of wind turbines at least. This size has caused concerns to the public and the environmentalists.

**Public acceptance of onshore and offshore wind energy**

Implementation of wind energy on land becomes difficult in some countries due to scarce sites and/or poor acceptance. Under such conditions offshore siting is of particular interest but has an ambivalent function. Offshore windpower might be seen euphorically as the ultimate solution due to the large potential and the lower environmental impact. Such an attitude might lead to a frustration of the implementation onshore. For the manufacturers of the concerned countries, e.g. the Netherlands, this would restrict their market to siting offshore or abroad. Moreover it is questionable if the ambitious national goals for a sustainable development by, among others, wind energy utilisation could be achieved in magnitude and time 'only' by application of offshore wind energy.

Nonetheless, the public acceptance of offshore wind energy is not fully understood. In the pioneer countries as Denmark and the Netherlands the attitude is generally quite positive and the results of investigations of the environmental impact of the pilot plants do not contradict this. Also Greenpeace, as important participant to the political discussion, has formulated positive support.

At the same time however, environmentalists express their concern about the possible effects of large-scale offshore wind farms. Also the visual impact of turbine clusters on the 'open seaside' is sometimes feared and one is reminded on the NIMBY phenomenon ('Not-in-my-backyard'), well-known from many onshore projects.

**Local environmental impact**

A large-scale utilisation of offshore wind energy has very little known negative effect on the local environment. Nonetheless, natural conservation areas as the Wadden Sea National Park are excluded likewise as zones occupied by other uses.

Investigations at the small-scale pilot projects have not found significant influences on migrating and resident seabirds, fish, sea mammals.

Environmental Impact Assessments (EIA) were completed without finding major objections during the planning of the first four Danish 150 MW offshore projects and the Dutch 100 MW Nearshore project [2.26, 2.27]. Topics covered by the Danish studies included: hydrography, seabed conditions, water quality, marine biology, fish, birds and marine mammals, visual impact, natural resources, marine archaeology, recreational issues, planning issues, ships and navigation and fishery. The studies lasted about one year and a half and will be continued by 2 to 3 years lasting investigations during and after the construction phase. Similar investigations have been initiated in United Kingdom, Ireland and Germany [2.28].

Eventually the experience with the first large projects available in 2002/2003 will hopefully provide firm results which will be important for the future developments.

**Politics**

The necessary incentives to support a large utilisation of onshore and offshore wind energy in the EU are completely dependent of decisions from the politicians. Certainty about the conditions and duration of policy instruments, as a standard payment or a market based system, are of high importance [2.29]. Establishment of clear and fast national planning rules and regulations is essential to limit the financial risks involved in
the planning process of a large project in the range of a quarter to half a billion Euro
even long before the construction phase.
Still a significant exploitation of offshore wind energy is a political decision as wind
energy cannot compete with fossil fuels if these are not judged by their social costs.

2.5 Market activities, development targets and prospects

Commercial developments of the offshore resources have started in nearly all coastal
countries of the European Union. Here only a snapshot of some of the activities at the
time of writing can be given. Commercial interests and uncertainties about the outcome
and duration of the permission procedure complicate any prediction.
Most firm are five Danish projects of in total 750 MW to be realised before the year
2008. They are part of the long-term and well-coordinated process driven by decisions
of the government aiming at 4,000 MW of installed offshore capacity by the year 2030
corresponding together with the onshore generation to about 50% of the national
electricity consumption.
At least each one project of 100 MW is expected in Belgium and the Netherlands for
project 'Nearshore Wind Park' will be decisive for the further national development of
wind energy and the industrial opportunities. On the longer term 1,250 MW offshore wind
power are announced by the Dutch government for the year 2020. It is also imaginable
that a more rapid scenario is feasible [2.18].
A couple of projects of different size are under preparation in Sweden but low power
purchase prices are a potential problem.
In the United Kingdom two projects have received old NFFO 4 grants but realisation is
unclear at the moment. Up to ten other projects in territorial waters are in the planning
stage but permits are uncertain as learnt from the frustrating situation onshore. The
British Wind Energy Association considers it unlikely that any of these will be actually
be built until 2005 [2.30].
The situation in Germany has improved since the new Electricity Feed Law (EFL)
applies also within the territorial waters and for plants commissioned before the end of
the year 2005. Nonetheless, no prototype project is foreseen in a due course. So, wind
farm developers have announced big claims and started planning activities for more
than 15 projects in the range from 9 to 1,500 MW [2.31]. The size of the projects is
driven by large water depths of 20 to 35 m and long distances to shore of 15 to 60 km
which are required with respect to the conservation areas and the naval traffic lines.
Realisation will depend among other aspects on the complicated permit procedures,
financing and the availability of mature technology tested at foreign sites.
Ireland, France, Spain and Finland show activities as well. Some Mediterranean
countries, e.g. Italy, have started exploitation of their resources and feasibility studies.
Potential interest outside Europe is seen in Japan where for instance the feasibility of
a renewable energy island combining various energy sources was studied. On the
longer term also other countries with shallow waters and good wind resources, e.g. India
or Bermuda, could be interested.
Exploitation of offshore wind energy is in line with the EU White Paper 'Renewable
Sources of Energy', where the specific target for wind power is set at 40,000 MW
installed capacity by 2010, 25% of which will be delivered by 10,000 MW capacity of
large wind farms, most of them will be offshore wind farms [2.32]. The latest targets of the European Wind Energy Association (EWEA) formulated in October 2000 consider the ongoing rapid growth and exceed the old target which coincided with the EU White Paper. 60 GW installed wind power including 5 GW offshore are projected for the year 2010 increasing to 150 GW including 50 GW offshore by the year 2020 [2.24].

Benefits to the environment

Investigations from Denmark show that wind energy is one of the cheapest ways of CO$_2$ reduction compared to several other renewable energy sources [2.33]. The emissions and waste avoided by 1 kWh of wind generated electricity depends on the national energy mix and other factors. Taking Denmark as example each kWh wind electricity avoids 850 g of CO$_2$, 2.9 g of SO$_2$, 2.6 g of NO$_x$, 55 g of slag and ashes and 0.1 g dust [2.25]. According to the British Wind Energy Association each MW of installed onshore wind energy capacity supplies electricity for about 600 average households.

Benefits to the employment

There are recent signs of the established offshore engineering industry, energy conglomerates and industrial combines beginning to give serious attention to wind energy projects in particular offshore. The development of a market for large scale offshore wind farms will bring new business to the wind turbine and offshore engineering industries, shipyards and shipping branches of Europe, creating employment for the associated work forces. Under present European market conditions 1 MW of onshore wind power installed creates jobs for 15 to 19 people [2.25]. The number will likely be higher for offshore projects.

2.6 Résumé and open questions

Offshore wind energy, a benign exploitation of the natural resources of the continental shelf, undergoes a process of rapid development in several European countries. Encouraging experience with the first large-scale project to be realised within the next two years will decide upon the future of an annual market in the billion Euro range.

With respect to such prospects a number of questions, essential for the future development of (offshore) wind energy, arises:

• How can the economics and reliability of large offshore wind farms be improved?
• Which technology is required for such an enormous application at exposed sites?
• What kind of approaches are necessitated for design and project realisation?
• Are current certification rules and standards suited for offshore developments?
• How to integrate large wind farms in the international energy supply system?
• What is the attitude of the financial world and the decision makers?
• Will offshore wind energy be cost effective only with bottom-mounted concepts and mainly for a number of northern European countries?
• What are the technological and socio-economic conditions needed to encourage the future development?
• How to avoid conflicts with environmental and other interests?
• How to achieve a beneficial development for onshore and offshore wind energy?

The rest of this thesis is related to the first four of these questions.
CHAPTER 3

INTEGRATED DESIGN APPROACH

Optimal design is facilitated by considering an offshore wind farm on an integral system level (Figure 3.1). The so-called 'integrated design approach' is introduced to achieve more cost-efficient and reliable offshore wind energy conversion systems (OWECS). Other suitable design methodologies undoubtedly exist, but an integrated design approach will most likely be nearer to the optimum in both technical and economic sense.

Prior to the description of the novel approach, attention is focussed on the life cycle and the design objectives (Section 3.1). Next, the design practices in wind energy technology, offshore technology and power management (Section 3.2) and design approaches applied for offshore wind farms so far (Section 3.3) are analysed. After this preparation the integrated design approach is presented in Section 3.4. Sections 3.5 and 3.6 summarise a possible implementation and first experience on the application in a near-design situation of the Opti-OWECS study. More comprehensive information about the new approach can be found in the first volume of the final report of the Opti-OWECS study [3.1].

Figure 3.1: Sub-system and aspects of an offshore wind energy conversion system (OWECS)
3.1 Basic design considerations

Life cycle of offshore wind farms

Like most other products, the life cycle costs of an offshore wind farm are largely determined during the design process which accounts only for a relatively small proportion of the total investment. No fuel cost exist for this renewable energy source, therefore the life cycle costs are mainly built up of construction and installation costs initially and operation and maintenance cost during the operation phase, both closely dependent on the designer’s choices.

<table>
<thead>
<tr>
<th>Design: Project identification</th>
<th>Feasibility study, conceptual design</th>
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<td>Structural design, specification</td>
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<td>Site survey: desk top</td>
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<td>Site survey: in-situ</td>
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| Permissioning, financing, marketing | ??? ? ? | Tendering & Contracting

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<th>Implementation: Construction</th>
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<td>Commissioning</td>
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<td>Exploitation (operation &amp; maintenance)</td>
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<td>Post-Exploit.: Refurbishment (optional)</td>
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**Figure 3.2: Qualitative life cycle of an offshore wind farm**

Figure 3.2 shows a qualitative life cycle. Between the initial plan and the commissioning typically up to five or more years may elapse for the three main steps: initiation, planning (i.e. environmental investigation, permission procedure, design, financing and tendering) and building (i.e. construction, installation and commissioning). Gaining all required permits is a potential source of delays. However, also site surveys may require a considerable period because in-situ soil investigations have to be done for each turbine location and local wind speed measurements for at least one year are needed for a reasonable prediction of the expected energy yield.

The operation phase may typically continue for 15 to 20 years. From an economic point of view even longer periods would be advantageous. However, this could only be achieved after a refurbishment of the wind turbines and other components.

Dismantling of offshore wind energy converters, especially if founded on piled steel structures, is expected to be relatively unproblematic. Removing and depositing of ballast of gravity structures may require extra considerations.

All tasks involving heavy works on site such as site measurements, installation, commissioning and dismantling have a strong seasonal dependancy. If not completed within one summer period it is impossible to continue at most sites before late spring of the next year.

**Objectives for optimal design**

An offshore wind farm designed in an optimum manner will provide energy in the most reliable and cost-efficient way over the projected lifetime.
Four design objectives for the entire system can be stated:

- Tailored distribution of investment and operation and maintenance (O&M) costs over the entire offshore wind farm and its lifetime.

  The economics of the complete plant has to be balanced with respect to the overall operational goal, which could be the achievement of the minimum cost of energy, the delivery of a certain power quality and persistence or a combination of both. Such goals cannot be reached by optimisation of separate sub-systems alone.

- High reliability of the OWECS as a whole and of essential sub-systems.

  A failure of a major sub-system can result in a loss of production for a long period. For instance, many converter units could be out-of-operation due to a design mistake or power cut-off in the grid connection system. Such a failure together with unfavourably high repair costs might jeopardise the entire project.

- Adaptation to economy of scale and partial redundancy of single converter units.

  A typical large offshore wind farm, as regarded feasible this decade, comprises between 50 and 100 offshore wind energy converters rated approximately 2 to 5 MW each. For such a batch size, design should be optimised with respect to the particular environmental and economic conditions. Consideration of a partial redundancy of the single OWEC units with respect to the production of the entire wind farm might be worthwhile, for instance, within the operation and maintenance strategy or when determining a design probability of failure.

- Symbioses of experience from wind energy technology, offshore technology and power management.

  Optimal design is not achieved by the 'blind' application of common design practices from the involved disciplines and therefore requires a joint solution.

3.2 Design practices in the parent technologies

The design of offshore wind farms should be based, of course, on the accumulated experience of the wind energy technology, offshore technology and power management communities. However, one has to be aware of the different backgrounds and sometimes even contradictory principles in the disciplines which are summarised briefly.

Design of wind energy converters and wind farms

Wind turbine engineering has undergone a rapid development in the last ten years and is now 'coming of age' [3.2]. During this period the average rating of commercial wind energy converters increased by a factor of ten due to both better understanding of the technology and demands of the market. The price per installed capacity and the price per annual produced energy lowered significantly. The increasing dimension of large turbines lead to high structural flexibility and potential sources of coupled vibrations. Specific knowledge has been developed to achieve safe solutions under such conditions e.g. by increased material or active damping, machine supervision and advanced control. Soft tower designs are successfully applied for reduction of weight, dynamic loads and hence cost.
Figure 3.3: Indicative cost breakdown of onshore wind farms in Germany [3.3] and small offshore oil and gas platforms [3.4]

Figure 3.4: Comparison of fundamental eigenfrequency between wind energy converters and offshore oil and gas platforms
Most manufacturers produce between two and four different standard machines. Modifications of these base line machines, also denoted as 'platforms', to tailor them to specific location classes, take account of mainly two site parameters only. The annual average wind speed influences the fatigue loading and determines the rotor diameter and hub height while the extreme wind speed is an important factor in the strength analysis. The atmospheric turbulence intensity is in most cases prescribed by the applied standard. Sometimes site specific loads are calculated when the standardised conditions deviate too much from the actually expected. Differences in onshore soil conditions are compensated for by making minor variations in the foundation design rather than by using different towers.

Wind farm design is mainly driven by the selection of a suitable machine type and size from the range of standard designs, arrangement of the turbines under sufficient distance to each other to limit the wind farm induced turbulence, compatibility with the existing grid infrastructure and noise limitations. Operation and maintenance aspects are important in order to ensure a sufficient lifetime and minimise repair costs.

The cost breakdown of typical onshore wind farms (Figure 3.3) is dominated by the procurement cost. Transport and installation cost are moderate but become more important for large turbines and remote sites. The contribution of engineering costs is low owing to series effects.

Design of offshore structures

There is a long experience in offshore technology with the design of large and unique fixed structures for the petroleum industries which are built 'fit for purpose' with respect to their particular site and function. In contrast to wind turbine design, the influence of dynamic response under wave loading is generally limited by relatively high structural stiffness in relation to the total height of the installation (Figure 3.4). The wave excitation range extends between $f = 0.04$ Hz and $f = 0.5 \text{ to } 1.0$ Hz. As a rule-of-thumb dynamic wave loading becomes important for natural frequencies below 0.4 Hz in shallow waters. So, from the over 7000 fixed offshore installations world-wide only a small number of platforms in deeper water have eigenfrequencies below 0.3 Hz.

Although fatigue is important, generally it takes second place to the dominant extreme event loading conditions. Transportation and installation issues are often main design drivers since these costs can be even higher than those for the manufacturing of the structure onshore (Figure 3.3). Reduction, and where possible, elimination of underwater inspection and maintenance is essential because of the difficult access and the high cost associated with these operations offshore. Other important design aspects concern the safety of personnel working on or travelling to the platforms, environmental impact and removal/dismantling.

Design of conventional power plants

Large coal, oil, gas or nuclear fired power plants are built since the introduction of distribution lines for electrical power at the beginning of the last century. The power demand pattern (base load or peak load) and the power type (electricity or both electricity and heat) are the most important design requirements. Plants are controlled according to the instantaneous and the short-term predicted power demand and are integrated in the national and international network. A mixture of locally distributed
stations with different response times and a certain overcapacity ensures a reliable supply.
With the exception of hydropower plants, the site is chosen close to the consumption centre or where fuel and cooling water can easily be supplied. Also, environmental aspects are important.
Power plants are designed often in a modular way. So the total capacity is provided from a combination of a number of standard power units more or less out of series production.

Several reasons make the grid integration of large wind farms a new challenge to the power management and wind energy communities. Site selection is now governed more by the natural resources and legal constraints rather than the location of the consumption and the available infrastructure. The short-term and long-term variation of the supply of wind energy has to be matched with the fluctuation of the energy demand. This requires wind energy prediction tools and both accurate and fast controllability of the entire wind farms as well as of the individual converters [3.5]. Furthermore, the economical value of wind generated electricity can be significantly lower if spot market prices instead of fixed feed-in rates are considered [3.6].

3.3 Presently applied design approaches for offshore wind farms

The first prototype offshore wind farms and many of the studies and projects employed design approaches, which are denoted in this thesis as either the 'robust or traditional' or the 'parallel design approach'.

The organisation within the design approaches is explained by the terminology and hierarchical diagrams as developed recently for the design and construction of complex civil engineering systems [3.7]. Such a treatment has been successfully applied, for instance, on projects as the Storm Surge Barrier in the Nieuwe Waterweg near Rotterdam and the Ekofisk Protective Barrier in Norway. With some adjustments it seems also suitable for offshore wind farms.¹

On the other hand it is useful to decompose the system with respect to its variables or requirements in order to organise the design of such systems. The relation between the variables (upper left part of Figure 3.5) represents the total friction between solution and problem.

On the other hand structural control is simplified by a decomposition with respect to sub-systems or design clusters and their elements (upper right part of Figure 3.5). In theory the boundaries of the sub-systems should be governed by maximum internal and minimum external (inter-)relations. However, often practical considerations of involved disciplines, organisations or materials are of equal or even larger importance. Simultaneous goal and structural coordination is facilitated by merging of the two decompositions in an improved composite constructive diagram (lower part of Figure

¹ An offshore wind farm comprises several elements that are not typical for most civil engineering projects, e.g. a 'machinery part' supported by a 'building/structure part', strong interaction between both elements and a large number of identical units. Nonetheless, it seems worthwhile to apply here the experience gained in the design and construction of complex civil engineering projects.
3.5). The clustering of requirements yields aspect systems\(^2\) for goal control whilst the clustering of elements results in sub-systems for structural control. Without structural control on a sub-system level no goal control on the system level is possible. The different levels of the composite diagram are related to certain part systems of the system, i.e. system, goals, aspect systems, sub-systems and elements, noted at the left side of the diagram. At the right side certain members of the project team are assigned to the different levels.

### 3.3.1 Robust or traditional design approach

The second Danish offshore wind farm at Tune Knob (1995) [3.8] is a good example of the robust or traditional design approach. To a lesser extent, this holds also for the earlier farm at Vindeby (1991) [3.9]. In both cases the attribute ‘traditional’ refers to an onshore design approach applied to the offshore environment.

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\(^2\) An aspect system refers to issues or topics of a system which are a cluster of relations between elements and/or individuals. Examples of aspect systems of a civil engineering system are: weight, strength, stiffness, maintenance, costs, etc. A suitable set of aspect systems represents the behaviour of the system and can be used for goal control on effectiveness. Therefore aspect systems should be chosen as specific, as independent, as equally important and as quantifiable as possible.
The main objectives of both projects were the demonstration and the investigation of environmental effects. Therefore, and owing to the small project scale, more or less standard onshore wind turbines were used. At Tunø Knob, wind turbines and towers were installed by a large floating crane but in a similar manner as onshore, i.e. by separate lifting of tower, nacelle and rotor (Figure 3.6). Stiff gravity caissons are countering the design ice loads and are acting as artificial islands. Reduction of operation and maintenance costs was aimed for by well-proven onshore wind turbine design, marinated by features as improved corrosion protection, air-tight nacelle and built-in lifting facilities.

The design process (Figure 3.7) is organised in two main clusters according to the parent technologies. Wind turbine, tower, farm layout and grid connection design belong to the wind energy technology while the offshore technology deals with foundation, subsea cables and marine operations. Important aspects affecting the overall performance as installation or operation and maintenance are treated separately by the two technologies as elements of sub-systems rather than as separate sub-systems or as aspect systems. Clearly this approach fits the objectives of the prototype projects but is not suitable for the proposed large scale offshore wind farms.

3.3.2 Parallel design approach
The parallel design approach considers the design requirements of offshore wind farms separately for the main sub-systems wind turbine, support structure, grid connection and operation and maintenance (Figure 3.8).

The system goals are decomposed in aspect systems specific for the different sub-systems. For instance, costs as an important goal are controlled for every sub-system but not on the system level. Thus, the economic performance is limited to the summation of the optimised costs of the sub-systems. As is the case with the robust or traditional approach, installation aspects and operation and maintenance aspects are treated only on the sub-systems level.

The Dutch pilot project Lely in the IJsselmeer (1994) [3.10], the Swedish Nogersund onshore wind energy converter (1990) [3.11], the Danish Middelgrunden wind farm (2000/2001) [3.12] and the tenders for the first two 160 MW projects in Denmark (2000/2001) can be mentioned as examples. At Lely novel design solutions were applied by monopile foundations and a cable laying technique reducing the use of a cable laying ship. Both innovations were a success, in principle. Nonetheless, the system aspects have not been fully considered, for instance, in the investigation of the overall dynamics (Section 9.5.1) [3.13]. Again, this could be accepted because of the sheltered site and the demonstration nature of the project. The installation procedure at Nogersund was most spectacular due to the radical minimisation of in-situ work. The entire offshore wind energy converter was fully assembled and commissioned in a dry dock before it was floated with auxiliary ballast and towed to the final destination (Figure 3.9).

The tendering and construction of the first large Danish project at Middelgrunden project with 40 MW installed capacity was separated into three parts for wind turbine and tower, foundation and grid connection. Turbine and foundation were designed independently
Figure 3.6: Piecemeal installation of Tunø Knob wind farm as example of the robust or traditional design approach

(Pictures by W. Steche, Windkraft Journal 4/95)

Figure 3.7: Controlling of the robust or traditional design approach
Figure 3.8: Controlling of the parallel design approach
Figure 3.9: Commissioning of the complete Nogersund offshore wind energy converter in the dry dock as example of the parallel design approach

and after the selection of suppliers for both sub-systems, modifications of the foundation were required. Also, it was more difficult to optimise the installation process for foundations, cables and machines.

The main sub-systems of the first two 160 MW projects in Denmark will also be tendered separately, but currently procedures are sought to optimise the entire system.

The parallel approach is typical for demonstration projects and first commercial plants and might have some advantages if the client wishes to select separately the supplier for the sub-systems rather than to design or tender the project as a whole. Resources and extra time are required for the communication between the design clusters. Good matching of sub-system solutions and fulfilment of the system goals might be achieved only after a number of iterations.

As a consequence of the robust or traditional approach or the parallel approach it might happen that the certification is done with respect to both a wind turbine standard and standard for offshore structures. Such a cumbersome procedure involves conservatism in the wind turbine loads if for instance the lower atmospheric turbulence offshore is not considered.
3.4 Innovation by the integrated design approach

3.4.1 General principle
The integrated design approach is treating the offshore wind farm as an entire system, i.e. the offshore wind energy conversion system (OWECS). Interactions between sub-systems are considered, therefore, as complete and practical as possible. So the design solution is governed by overall criteria, the aspect systems. Examples for such criteria include levelised production costs, dynamics of the entire system and OWECS availability. Although such an approach will most likely produce an novel design solution, the evolutionary nature of technical progress and the commercial risks inherent to innovative solutions must be kept in mind. In particular it would be a serious mistake to interpret it as a justification for the application of unproven wind turbine design to the harsh offshore conditions.

The success of a design process is largely dominated by the organisation of the design team as the process and organisation can hardly be separated. Figure 3.10 proposes a scheme for the goal and structural coordination of the integrated design approach. The management is done by a ‘team of cluster leaders’ in which every partner is represented and all major decisions which affect the design of the system are taken by the team. Each person in this group manages a design team which is responsible for the development of a particular sub-system. It should be attempted to reduce the number of relations between the various sub-systems by suitable selection of the sub-systems. This will reduce potential design conflicts and will increase the flexibility of a design team. Decisions made by the ‘head design team’ should be carefully and consistently reported as they serve as ‘requirements’ for the sub-system designs of all teams involved.

Figure 3.10 illustrates how the sub-system design is controlled with respect to the project goals by means of five aspect systems: levelised production cost, adaptation on site conditions, dynamics and structural reliability consideration, installation and commissioning effort and OWECS availability. The aspect systems are resolved into qualified and controllable criteria. Procedures are available to evaluate the design. The engineers in the different disciplines involved need assistance and tools for judging intermediate results during the design process. Examples include an overall cost model [3.14], a simulation tool for the operation and maintenance behaviour [3.15] and codes for design calculation of the entire OWEC (Chapters 6 and 8). The complex system is split-up into sub-systems, each setting its design requirements. These requirements are then input for each design team to provide a satisfactory solution. If there are problems in the generation of such a solution the requirements should be re-evaluated by the ‘head design team’ as the changes will influence the design of all sub-systems (but probably not to the same extent).

Section 3.6 and Figure 3.11 summarize the first application of the integrated design approach during the design of a 300 MW offshore wind farm within the Opti-OWECS study. Probably the approach will also be considered in some proposed offshore projects in the Netherlands including the 100 MW Nearshore project, the Afsluitdijk project and the development project DOWEC [3.16]. Consortia covering the expertise of the different
Figure 3.10: Controlling of the integrated design approach
technical and financial fields are developing entire offshore wind farms or an offshore wind energy converter in these projects. Elements of the integrated design approach were also applied during the realisation of the Utgrunden offshore wind farm in the year 2000 (Section 9.1) [3.17].

Within the integrated design approach specific offshore wind energy standards, [3.18], are used and certification is done with respect to the particular site conditions.

The following sub-sections evolve four complementary variants of the integrated design approach each emphasising particular aspects including economics, operation and maintenance aspects, overall dynamics and radical design solutions. For the further elaboration of this thesis the part dealing with dynamics is most important. In reality, features of different variants will be combined according to the actual requirements.

3.4.2 Economic optimisation
The best economic performance of an offshore wind farm is achieved by a balance between capital cost, operation and maintenance (O&M) costs, availability and associated energy yield.

Important design parameters are interacting with certain sub-systems and their performance in different ways. For instance, an increased hub height results in a higher energy yield but also in higher support structure cost, installation and maintenance costs. More exposed sites offer more energy and potentially lower energy costs but have

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**Figure 3.11:** Innovative features of the design solution for a 300 MW offshore wind farm
to be paid for by larger investments, higher O&M cost, lower availability and greater transmission losses.

In an integrated design approach such relations are considered from the outset. The design effort can be directed towards crucial cost elements, e.g. operation and maintenance costs. During the course of the design process the sophistication of the cost evaluation should increase from simple estimates of the cost-breakdown and levelised production cost in the feasibility study, to an engineering cost model for the entire OWECS [3.14] during the conceptual design and the detailed evaluation in the final design phase. After each stage, solutions for an entire offshore wind farm at a particular site should be evaluated.

3.4.3 Design for reliability, availability, maintainability and serviceability
Challenging design targets for reliability, availability, maintainability and serviceability (RAMS) [3.19] of an optimised offshore wind farm can only be met by an integrated design approach.

The targets with respect to the price, quantity, quality and persistence of the delivered electrical power do not only guide the technical design but also the design for reliability and availability, i.e. failure states and rates, and the design for maintainability and serviceability, i.e. rate, ease and cost of repair and regular service. Since there is only a limited possibility to use experience for the development of an integrated maintenance approach for OWECS, a knowledge-based methodology must be applied.

Likewise for the structural design, the following specific targets are set and are effectively controlled in all phases of the design process.

- reliability target, i.e. the probability that the system is able to fulfil its functional design targets,
- availability target, i.e. the fraction of time that the system is able to operate as intended under given external conditions,
- maintainability target, i.e. the probability that a malfunctioning system can be brought back into operation within a given time,
- serviceability target, i.e. the ease and costs at which regular (scheduled) service can be carried out and specification of the fraction of time and the costs needed for service.

The principal relation and development of design targets and design specification during the different design phases are illustrated in Figure 3.12.

The specification of the functional design targets, the RAMS targets, and the design of the installation and maintenance concepts should first be addressed in an integrated way on the systems level during the feasibility study. Only then the best solution will be achieved. The targets on the systems level should then be translated in a consistent way into specifications on the sub-system level during the conceptual design. Next for the detailed design the specifications have to be determined also on the component levels. Finally the sub-components are treated in the same manner during the design specification phase.

After the assessment of the local consequences of the specifications on a certain level of the system, by a simplified calculation of reliability and availability, the results should be evaluated on the system level. Such a process of continuous evaluation and re-evaluation of the targets and specifications on all the design levels assures the optimal design of the OWECS.
Figure 3.12: Relation between design targets, specifications and evaluation in the design process for reliability, availability, maintainability and serviceability (RAMS) (R&A = reliability and availability)

With respect to the further assessment of the possibilities to achieve the RAMS targets it is worthwhile to apply design tools for reliability and availability calculations. Such design tools range from simple probabilistic techniques for simplified reliability and availability calculations based upon generic failure rates (small, inner iteration loops in Figure 3.12), to more advanced Markov chain modelling. For complex systems such as an offshore wind farm the most often applied technique for evaluation is Monte-Carlo simulation (large, outer iteration loops in Figure 3.12).

It is evident that design solutions of the OWEC unit design level will have consequences for the design of the OWECS maintenance concept and vice versa. But an integrated approach of the design of the installation process and the maintenance process will be beneficial for achieving the best solution. An example is the possibility of using a general purpose or purpose built/modified installation infrastructure, such as platforms, cranes and boats afterwards as maintenance infrastructure.
3.4.4 Overall dynamics
System dynamics and dynamic interactions between sub-systems are important and should be considered in an integrated manner during the different design phases. Examples of the treatment of dynamics of offshore wind energy converters (OWEC) include:

**Feasibility study**
- (qualitative) compatibility of support structure and wind turbine concepts
- (qualitative) compatibility of support structure concepts and sites,
- assessment of the order of magnitude of aerodynamic and hydrodynamic loading for extreme as well as fatigue conditions,

**Conceptual design**
- sensitivity analysis of dynamics with respect to soil properties,
- parameter studies on dynamic loading in frequency (or time) domain,
- simultaneous optimisation of wind turbine concepts (e.g. rotor speed and diameter, blade and drive train layout) and support structure concepts (e.g. stiffness and hub height),

**Detailed design**
- detailed dynamic analysis of OWEC with design tools in the time domain (especially if fatigue is governing),
- fine tuning of dynamics,
- investigation of effects due to the variability of site parameters within the wind farm.

In Chapter 10 'Design optimisation by means of tailored dynamics' demonstrates the consideration of dynamics during the design process of an offshore wind farm.

3.4.5 Radical design for offshore requirements
The use of unconventional designs might provide a major benefit for the entire OWECs under the condition that such radical designs are governed entirely by the offshore requirements rather than by adapting onshore experience for the offshore situation. For instance, the wind turbine would be designed consistently for offshore application from 'scratch' rather than 'marinising' an onshore design.

Without an assessment of their viability, some examples are listed here for illustration purposes only.
The Multibrid concept of the engineering consultant aerodyn Energiesysteme GmbH (Figure 3.13) introduces a novel wind turbine concept and several innovative features in order to meet the design requirements of large offshore wind energy converters. In contrast, most wind turbine manufacturers are following a less risky approach by up-scaling of a certain design concept particular to the company. The integrated drive train with a single stage gear and a medium speed generator of the Multibrid design facilitates reduction of failure rates by minimising the number of components, avoidance of high speed gears and bearings and complete capsulation of the entire nacelle. The compact design, innovations in the blade technology and yaw system lead to an extremely low tower head mass in the 5 MW power class [3.20].
Unconventional support structures are proposed by the multi turbine concept [3.21] and the Multi Unit Floating Offshore Wind Farm (MUFW) concept (Figure 2.4) [3.22].
Water cooling of all components

Single-stage planetary gearing

Only one sealing ring towards the outside

Slowly rotating generator

Integrated machine set

Planet carrier with integrated rotor bearing

Internal yaw bearing

Geometrical arrangement

Seawater heat-exchanger at foot of tower

Figure 3.13: Radical wind turbine design of the Multibrid concept

Ultimately, such a radical approach might be the most promising. However, unproven or just 'big' designs, do not lend themselves well for application in the demanding offshore environment and a different lesson can be taken from onshore development. The large success of wind energy technology during the last decade was achieved only by an evolutionary technical development. It required several years until the two major innovations, i.e. variable speed - variable pitch control and direct drive technology, showed commercial success. None of the radical designs of the 1980s and early 1990s, including prototypes of multi-megawatt turbines, single bladed machines, Vertical Axis Wind Turbines (VAWT), proved to be viable.

In summary, the radical approach is hardly considered feasible at the moment but may be required for very large offshore wind farms with a capacity in the gigawatt range.

3.5 Implementation

The previous section described the philosophy behind and the organisation of the integrated design approach. In the following one possible implementation during the different stages of the design process is presented.

The procedure is based upon the experience gained during the Opti-OWECS study. Emphasis is given to the sequence and interaction of the different parts of the work contents rather than to organisation, scheduling and resource management. Only the relations between the technical steps are considered here, no interaction with the 'customer' or the 'public' is taken into account. However, issues as public acceptance and permission can have a significant and sometimes unexpected impact.
Figure 3.14: Example design process of the integrated design approach
The process (Figure 3.14) starts with the project identification (Step 0) and the statement of the project objectives.
In the first step a feasibility study is performed which results in a selection of a number of concepts related to certain sites (Step 1.3) and the overall realisation plan. The choices are based on the separate pre-selecting of sites and sub-system concepts (Step 1.1) and the evaluation of the combination of sites and OWECs concepts (Step 1.2).
In the conceptual design (Step 2) the chosen combination of sites and OWECs concepts are further developed on a sub-system level (Step 2.1a). In parallel tools for the evaluation of the OWECs performance and the control of the aspect systems are developed or updated (Step 2.1b).
Again an evaluation (Step 2.2) and selection of the final concept at the final site (Step 2.3) takes place. Once the outcome of this phase is given, the client does not have opportunities to change concepts or scope of work without incurring substantial delay and extra costs. Therefore the outer right loop from the conceptual design to the problem identification is indicated only as a dashed line. During the detailed design in Step 3 the 'dimensions' of the design are fixed and the relation between the sub-systems is generated.
System specifications are worked out in the final, fourth step.
While during the conceptual design only major interactions between sub-systems are considered a more demanding treatment is required during the structural design phase. Now the interactions between the sub-systems are taken into account in a form as complete and practical as possible (Figure 3.15) and the solution is governed by quantified, overall criteria.
3.6 First application

The first application of the integrated design approach during the Opti-OWECS is summarised to demonstrate the feasibility and the prospects for future applications. Here only the procedure and the main experiences rather than details of the actual solution [3.23] are presented.

An offshore wind farm was developed against the background of three objectives. Firstly, the improved understanding of the principles underlying the design of OWECS gained during the course of the project was to be demonstrated by a practical solution. Application of promising innovations for large-scale utilisation, e.g. novel installation methods, consideration of operation and maintenance aspects, integrated design approach, was for this research project more important than achieving the absolute economic optimum. Secondly, during the design process, areas of poor understanding were to be identified and appropriate solutions were to be developed. Finally, the economic feasibility of large OWECS was to be investigated.

Project identification

The initial phase of the project identification comprised three aspects:
- establishment of the project group: an international cooperation of industrial engineers and researchers from the wind energy field, offshore technology and power management
- determination of project conditions (i.e. objectives and work programme)
- formulation of the particular design conditions, e.g. wind turbine size of 3 MW or larger, neglect of the details of the onshore grid connection.

Feasibility study

During the feasibility study, a broad inventory of all relevant aspects and concepts was made and pre-selections for the conceptual design were identified. Furthermore, a terminology appropriate to OWECS was established in order to promote smooth communications (Appendix A).

The identification of seven distinct reference sites in northern European waters was carried out in parallel with further investigation of the sub-system concepts and the other essential features of overall dynamics and operation and maintenance. The water depth of the considered sites ranged from 8 m to 25 m. The majority of the sites can be characterised as genuinely offshore rather than inshore or nearshore sites, as is the case for existing offshore wind farms.

Based upon a qualitative OWECS evaluation the following sub-system concepts were selected for further development:
- The 3 MW 80 m diameter Nässudden development line was chosen as reference turbine since it is the only one in the multi-megawatt league with a reasonable operational track record. In addition, the turbine rating of about 3 MW extended the state of the art of other offshore projects under development during the end of 1990s. Another justification for this choice was the availability of the entire set of design data and the industrial interests within the project consortium.
• two wind turbine concepts (geared - fixed speed, direct-drive - variable speed),
• rotor variants with diameters between 80 and 100 m and different rotor speeds,
• distinctly different combinations of support structure configuration and installation procedure, dynamic characteristics (stiff-stiff, soft-stiff and soft-soft 3) and site (North Sea and Baltic Sea),
• standard options for grid connection and wind farm layout.

Conceptual design

The conceptual design phase was carried out mainly in parallel with work on subsystems development and with the extension of the OWECs tools for cost modeling, simulation of the operation and maintenance behaviour, structural reliability considerations and overall dynamics.

Improved knowledge of particular OWEC aspects gained during this phase led to the consideration of a monopole support structure in addition to the gravity monotower and the gravity lattice tower (lower row in Figure 3.16). The extreme design wave together with a relatively low water depth resulted in a very significant heave force on the gravity monopole and required a large amount of ballast which is costly. Furthermore, both the soft-stiff monotower and the stiff-stiff lattice tower suffered considerable aerodynamic fatigue loads owing to their fundamental eigenfrequency being close to the blade

![Diagram of support structure concepts](Image)

**Figure 3.16:** Examples of support structure concepts

(GBS = Gravity-base System)

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3 Three design solutions exist for wind turbine towers depending on the ratio between the fundamental eigenfrequency \( f_0 \) and either the rotor frequency \( f_R \) or the blade passing frequency \( f_b = N_b \cdot f_R \). soft-soft, i.e. \( f_0 < f_R \), soft-stiff, i.e. \( f_R < f_0 < f_b \) and stiff-stiff, i.e. \( f_b < f_0 \). In practice, stiff-stiff designs are no longer built for the current generation of wind turbines.
passing excitation frequency and the constant speed - variable pitch concept. In particular, stress concentration at the tubular joints and the stiff characteristics of the lattice design, chosen well beyond the range of dynamic wave excitation at the exposed North Sea site, showed conflicts with the dynamic wind turbine loading. The dynamics of the three concepts are further discussed in Chapter 9.

Several results were directly related to the integrated approach:

• integrated development of support structure concepts and installation procedure
• simultaneous optimisation of wind turbine (e.g. rotor speed, blade layout) and support structure (e.g. stiffness) with the main goal of reduction of aerodynamic fatigue loads (Section 10.2.2)
• consideration of overall dynamics in the support structure design
• development of optimal O&M strategies based on Monte-Carlo simulations,
• development of structural reliability analysis for the support structure

Next, the cost model, developed elsewhere during the project, was used to evaluate different offshore wind farms assembled from the developed sub-system concepts at the seven pre-selected sites from the feasibility study. For the same OWECs concept the energy costs differed up to 60% between the seven compared sites, which initially were all considered as promising. Furthermore, a reduction of the energy cost of about 20% was found by application of a support structure of the monopile variety instead of a gravity monotower type. The economics together with some other criteria led to the selection of the final concept and the related site.

**Detailed design**

During the detailed design phase, the selected concept was developed further and interactions between the sub-systems were fully considered.

This integration facilitated several innovations, which represent significant steps towards mature offshore wind energy technology (Figure 3.11):

• An appreciably high offshore farm availability of 96.5% was achieved through a rational approach comprising a gradual improvement of the turbine’s reliability and maintainability with respect to the current onshore state of the art and an innovative operation and maintenance solution. The latter included both an optimised operation and maintenance strategy and a cost-efficient solution to the ‘craneage problem’ in the form of a permanently and quickly available self-propelled modified jack-up.

• Close cooperation between structural design and dynamic simulations of the OWEC facilitated a soft-soft monopile design even for 20 m water depth (LAT) in a demanding Southern North Sea environment with firm soil conditions. Balancing the combined aerodynamic and hydrodynamic fatigue loads, reduction of the water-piercing cross section and consideration of weight and installation issues were the main reasons for this choice. This is remarkable because the fundamental eigenfrequency of 0.29 Hz lies well below the rotor frequency of 0.37 Hz but just in the region of significant dynamic wave excitation extending between 0.04 and 0.5 to 1 Hz.

• Consideration of tower and foundation as one system resulted in a more appropriate and economic design than the separated treatment of tower and foundation.

• Significant cost reductions were possible by innovative installation methods for gravity systems which were floated and towed as entire OWEC unit to their final
destination. Conceptual solutions were developed for both floating and lifting installation of monopile structures. The variety of offshore equipment available, including jack-ups, crane vessels, barges, etc. opens several opportunities which have to be judged case by case. Also, open questions remain, for instance on the sea fastening for the vertical transportation of the tower and wind turbine unit.

• The aerodynamic efficiency of the wind farm, the cable costs of the grid connection and the space requirement of the OWECS were well balanced.

• Placement of the OWEC transformer in the nacelle was chosen after consideration of wind turbine, support structure, grid connection and maintenance aspects. Most important for this choice was that expensive modifications of either the tower diameter or of the access platform are avoided and that the dynamic characteristics remain nearly unchanged.

It is worth noting that neither during the detailed design phase, nor after the final evaluation of the design solution, major revisions of the design were required. The main reason for this was that the conceptual design had already been carefully examined with respect to technical feasibility and economic performance.

Beside the technical feasibility and economic prospects the Opti-OWECS study has also been demonstrated that such a methodology can be managed within an international cooperation of scientists and industry engineers from wind energy technology, offshore technology and power management.
Chapter 4

Description of the Offshore Environment

Various natural and man-made environmental conditions are affecting offshore wind farms (Figure 4.1). The most relevant for our purposes are the meteorological-oceanographic conditions, the so-called 'met-ocean' parameters, and the soil conditions. This chapter describes the environmental parameters while the next chapter deals with the modelling of offshore wind energy converters and the environmental impact. Only an overview can be provided here. Therefore the extensive usage of formulas has been avoided. For a more comprehensive discussion the interested reader may consult the references and particularly the design guidelines of the UK Department of Energy [4.1] and textbooks on meteorological boundary layer theory [4.2].

The different met-ocean parameters are discussed in Section 4.1 where special attention is given to wind and waves. The different needs of offshore wind energy applications compared to land-based wind energy converters and offshore platforms are highlighted. Section 4.2 is devoted to the correlation of wind and wave conditions, while Section 4.3 concerns the soil conditions and some auxiliary environmental effects such as corrosion, ship impact and influences arising from the energy consumer.

![Diagram of Environmental Impacts on an Offshore Wind Farm](image)

**Figure 4.1:** Environmental impacts on an offshore wind farm
Table 4.1: Met-ocean parameters relevant for the design of OWEC

**Wind**
- directional long-term distribution of mean wind speed at hub height
- extreme wind speeds (averaged over 10 min, 1 min, 3 s; RP1, RP50)
- vertical profile (roughness length or wind shear exponent)
- distribution of stability conditions
- ambient turbulence intensity at 15 m/s mean wind speed
- turbulence spectrum, coherence function and integral length scales

**Waves**
- extreme wave crest height
- extreme wave height, direction and range of associated periods (RP50)
- extreme sea state parameters (RP50)
- (directional) scatter diagram of significant wave height and zero crossing or peak period
- wave spectra and directional spreading functions

**Wind and wave correlation**
- long-term correlation of wave parameters and mean wind speed
- distribution of weather windows

**Currents**
- extreme current speed and direction
- vertical profile

**Water depth and sea level variations**
- water depth below lowest astronomical tide (LAT)
- tidal range
- extreme still water level variation, e.g. surge

**Icing and sea ice**
- design thickness of snow and ice on the structure
- sea ice design parameters (if relevant)

**Temperatures**
- extreme air temperatures
- extreme sea temperatures

**Marine growth**
- type and density of growth
- annual growth rate and terminal thickness profile

RP1 = annual return period, RP50 = return period 50 years
4.1 Met-ocean parameters

Harsh weather conditions, hindered access for inspection and unmanned operation for long periods, along with the need for cost-efficient design solutions result in great importance of the description of the environment (Table 4.1). The specific properties of offshore wind farms do not allow unproven application of the different experiences from both the offshore and the wind energy technology. For instance, offshore wind farms are generally located in more shallow waters and closer to shore than most offshore petroleum installations resulting in different load characteristics and more complex hydrodynamic behaviour. Wind loads and hydrodynamic loads are often of the same order of magnitude. Generally fatigue is more important. The highest loads do not necessarily occur during 50 year extreme storm conditions since extreme aerodynamic loads during production could be more demanding.

Onshore wind turbines are designed according to a few standardized wind classes appropriate for a variety of sites. The offshore wind climate, however, differs considerably from land and between locations. Hence, for large projects a site specific offshore design is needed from a technical and economical point of view.

4.1.1 Wind

Wind is continuously changing with time by nature. Figure 4.2 illustrates the frequency and energy contents of the different types of wind phenomena. A spectral gap opens between the macro-meteorological effects due to the gradient between pressure systems and semi-diurnal winds on the low frequency side and the micro-meteorological variations owing to thermal and roughness induced turbulence on the high frequency side. The low energy content between periods of some hours and about five minutes enables the separation of long-term and short-term statistics.

![Figure 4.2: Qualitative wind energy spectrum [4.3]](image-url)
In comparison to their counterparts on land, offshore winds are generally characterised by higher annual average wind speed, higher extreme mean wind speeds but lower turbulence and gust factors along with a smoother wind shear and more stable wind directionality.

For most sites and with a reasonable turbine spacing, this results in lower aerodynamic loads and offers opportunities for design optimisation. At exposed sites the benign effects are offset by both the more frequent high wind speeds, which dominate the fatigue, and stronger extreme gusts governing limit state. Section 9.2 quantifies site specific loading in more detail.

The following discussion focuses on wind aspects relevant for load calculations. Other specific meteorological effects are of importance for reliable predictions of the wind resources and for extrapolating measured wind speeds to larger heights. References [4.4 - 4.8] provide further information on daily and seasonal variations, the effect of the internal boundary layer thickness, atmospheric stability corrections, vertical wind profiles, wind speed dependent roughness, ambient turbulence and low level jets.

**Wind speed distribution and wind profile**

Figure 4.3 compares the wind speed distribution between two fictitious but typical sites onshore and offshore. At sea the annual average wind speed, $V_{\text{ave}}$, is greater and the shape of the probability density function of the mean wind speed is shifted to higher wind speeds corresponding to a larger Weibull shape factor, $k$. Just for comparison, the Rayleigh distributions with $k = 2$ are plotted as solid lines without markers for both sites. Generally on sea the annual average wind speed and the shape factor are larger (see solid lines with markers). Both properties result in a larger probability of high wind speeds and favour a high specific rating for OWEC in terms of rated power per unit swept area. This can be realised for instance by a greater generator capacity at the same rotor diameter compared to onshore siting.

The reduced surface roughness at sea results in a steeper vertical wind profile and a lower turbulence intensity. Small wavelets or ripples on the water surface rather than the wave height itself determines the roughness. Charnock proposed a constant ratio between skin friction velocity and roughness, valid only for the equilibrium state between wind and wind generated waves [4.9]. Often short fetch, i.e. the distance the wind is acting over the sea, or short duration of the wind condition result in circumstances which are not fully developed, in which the Charnock constant is no longer applicable. However, in most cases a roughness length of 0.2 mm [4.10] is a reasonable assumption compared to 0.01 - 0.25 m for sites on land [4.11].

For engineering purposes the wind speed profile is expressed by a logarithmic relation between height over ground and roughness, $z_0$, [4.11, 4.12] or by a power law as function of non-dimensional height with the wind shear exponent, $\alpha$, as a parameter [4.13 - 4.15]. Both relations are valid only for neutral atmospheric stability conditions.

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1 The Danish wind turbine standard DS 472 recommends $z_0 = 0.001$ under normal and 0.004 m under extreme offshore wind conditions [4.12]
**Figure 4.3:** Wind speed distributions for typical sites onshore \( V_{ave} = 7 \text{ m/s} \) and offshore \( V_{ave} = 8.5 \text{ m/s} \) (solid: Rayleigh distribution, solid with markers: Weibull distribution)

**Figure 4.4:** Logarithmic wind profiles for typical onshore and offshore sites \((z_0 = \text{roughness length})\)
stability conditions when thermally induced turbulence is negligible compared to mechanically induced turbulence. Neutral conditions are less frequent offshore but they are still dominating in the range of high wind speeds which are most relevant for the mechanical loads. Figure 4.4 illustrates how the wind profile develops at two typical sites. Offshore the hub height is commonly chosen lower (Table 2.1) since relatively little extra energy can be gained by a taller structure, which is relatively more expensive than on land.

Atmospheric turbulence and wind field simulation

Turbulent gusts are caused by eddies in the wind flow. For the wind field simulation it is assumed that they travel with the mean wind speed according to the commonly applied frozen turbulence hypothesis (Figure 4.5). The eddies have different frequencies, amplitudes and phases and may intersect each other. The characteristic lengths of the turbulence structures, denoted as integral length scales, are of the same magnitude as the rotor diameter of large wind turbines. The rotational period is commonly smaller than the time for a turbulence bubble to pass the swept area. Consequently most of the gusts affect only parts of the rotor but are seen several times by blades. This ‘rotational sampling’ or ‘eddy slicing’ transforms the wind spectrum seen by a rotating observer leading to a lumping of excitation energy at the rotor frequency 1P and its multiples (Figure 4.6).

The temporal and spatial turbulence properties can be represented by power spectra associated with the three velocity components and a coherence function accounting for the spatial correlation. Based upon the work of Shinozuka [4.16] and Veers [4.17], the auto and cross spectral density functions are determined at discrete points of the swept area [4.18] or on the rotor blades [4.19] from which random representations in the time domain are generated by Inverse Fast Fourier Transform (IFFT).

In this thesis the longitudinal turbulence is described by a von-Karman spectrum and an exponential coherence function with aid of the SWING 3 code [4.20]. Recently it has become common to fully account for all three components of the turbulent wind field [4.15]. Here only the longitudinal component is simulated stochastically. The effect of the other components is modelled in a simplified manner by a sinusoidal variation of the yaw misalignment of ± 30° during the duration of the dynamic simulations [4.21].

Ambient and wake turbulence

The lower roughness at sea reduces the atmospheric turbulence intensity, i.e. the ratio of the standard variation of the instantaneous wind speed and the mean wind speed, and stretches the integral length scales of turbulence. Measured values of the turbulence intensity, range around 7 - 9% at 15 m/s for neutral conditions offshore [4.22, 4.23]. Larson and Jørgensen proposed an empirical relation for the design turbulence intensity based upon evaluation of two Danish shallow water sites and the fatigue behaviour of different materials [4.24]. At 15 m/s mean wind speed this results in a figure of 9.7%; however, above 20 m/s unrealistic high values are obtained.

Generally, the same power spectra and coherence functions are considered onshore and offshore but with different design turbulence intensities. Figure 4.7 compares the design turbulence intensity as function of mean wind speed at the two sites onshore and offshore. At low wind speed thermal turbulence and non-neutral conditions are prevailing and higher turbulence is observed. For higher wind speeds, the mechanical turbulence owing to roughness dominates and the turbulence intensity decreases.
The dynamic loading in a wind farm increases significantly due to the wake of upstream turbines. Especially when the swept area of a turbine is partly covered by a wake, high load fluctuations are observed. Measurements at the Vindeby offshore wind farm indicate that the relative increase in wake turbulence is considerably higher offshore and is more persistent downstream due to the lower ambient turbulence. The observations showed a machine to be affected only by the directly adjacent turbines rather than by all upstream turbines [4.25].

For engineering purposes it is convenient to account for the different effects of the wake on the wind turbine components by an increase of the ambient turbulence intensity which results in approximately the same fatigue damage as in the actual operation at the most unpleasant position within the wind farm.

Frandsen et al. [4.25] introduced a relatively simple correction which considers the wind turbine thrust coefficient, mean wind speed and spacing ratio, only. This model is evaluated for the pitch-regulated wind turbine used within the Opti-OWECS study in Figure 4.7. The wake effect is relatively small offshore if a large spacing of 8 to 10 diameters is applied.

Other research pointed out an effect of the slope of the S-N curve of the material under investigation [4.26, 4.27]. For instance, rotor blades made from glass fibre reinforced plastic are more prone to high cycle fatigue and are consequently more sensitive to wake effects than steel components, e.g. the tower.

Recent investigation suggest that the wind turbine of large wind farms are acting as roughness obstacles and are inducing a background turbulence that at least partly compensate low ambient turbulence [4.27].

**Extreme wind gusts and directional changes**

Extreme wind conditions relevant for design are distinguished by the operational mode of the wind turbine to be either parking, i.e. standing still, idling or failure state, or power production.

The governing load for parking results from extreme storm conditions with a 50 year return period. Here the traditional approach of a quasi-static calculation of the extreme dynamic pressure corrected with a dynamic amplification factor [4.28, 4.29], is now replaced by more accurate and less conservative time domain simulations of the response under a turbulent wind field.

The practice with respect to extreme wind conditions during production is in contrast to the sophisticated description of the stochastic behaviour under fatigue conditions. All wind turbine standards still assume simplified and deterministic situations, e.g. a given variation of the mean wind speed by a gust with a ‘one-minus-cosine’ or ‘Mexican-hat’ shape. Such gusts are coherent over the swept area and combined with or without a prescribed change of the wind direction. Other examples include an extreme vertical and/or horizontal wind shear or extreme direction change [4.15]

Application of the above-mentioned standard simulation techniques for turbulent wind requires unreasonable computations to find time series which includes an extreme gust with a certain magnitude. Recently, a more adequate probabilistic description has been demonstrated that embeds a prescribed gust amplitude in a background turbulence in such a manner that the stochastic properties of the synthesised signal cannot be distinguished from real turbulence [4.30]. The new approach applies methods which are common in advanced offshore engineering [4.31] and is an example of a cross-fertilisation between wind energy and offshore technology.
Figure 4.5: Wind turbine rotor slicing though the temporal and spatial structure of a turbulent wind field

Figure 4.6: Rotational sampling of gusts illustrated at the transformation of the Kaimal turbulence spectrum from a fixed to a rotating observer
Figure 4.7: Onshore and offshore turbulence intensities (Functional relation of ambient turbulence acc. to IEC 61400-1 ed.2, wake turbulence acc. to [4.25]: $I_{15} = \text{turbulence intensity at } 15 \text{ m/s mean wind speed, } a = \text{slope parameter}$)

Table 4.2: Wind conditions according to Germanischer Lloyd [4.13, 4.14]

<table>
<thead>
<tr>
<th>Wind condition</th>
<th>Onshore</th>
<th>Offshore</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reference wind speed (at hub height) $V_{ref}$</td>
<td>Class I: 50 m/s  Class III: 37.5 m/s</td>
<td>Class II: 42.5 m/s  Class IV: 30 m/s</td>
</tr>
<tr>
<td>Extreme gust $V_{e50} \dagger$</td>
<td>$1.4 \cdot V_{ref}$</td>
<td>$1.2 \cdot V_{ref}$</td>
</tr>
<tr>
<td>Extreme operating gust</td>
<td>identical 13 m/s</td>
<td></td>
</tr>
<tr>
<td>Extreme change in wind direction</td>
<td>$V \leq 5 \text{ m/s } : 180^\circ$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$V = 15 \text{ m/s } : 35^\circ$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$V = V_{ref} : 15^\circ$</td>
<td></td>
</tr>
<tr>
<td>Partial safety factor</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• environmental loads</td>
<td>$\gamma_F = 1.5$</td>
<td>$\gamma_F = 1.35$</td>
</tr>
<tr>
<td>• other loads</td>
<td>identical ($\gamma_F = 1.1 \text{ - } 1.5$)</td>
<td></td>
</tr>
</tbody>
</table>

$\dagger$ Germanischer Lloyd originally stated 5 s as averaging period while commonly 3 s are used in accordance with IEC 61400-1 ed.2
Owing to the lower turbulence offshore the Germanischer Lloyd has proposed less severe gust factors offshore (Table 4.2). Nonetheless, the magnitude of the extreme operational gust is maintained which is an important extreme load case for most pitch regulated turbines.

4.1.2 Waves and currents
From the various hydrodynamic effects mentioned in the macro-scale energy spectrum in Figure 4.8 the wind induced waves and tidal currents are the most important in European coastal waters. Generally one has to distinguish between wind seas generated by local winds and swells originating from waves travelling over a long distance into the area of interest. The topography of the North Sea and the Baltic Sea lead to lower significance of swell.

The period range of energy rich waves lies between 2 s and 20 s. Extreme waves have long periods, typically 7 - 13 s for our purposes, since the energy of a sea state is proportional to the fourth to fifth power of the dominant period and the square of the wave height. Nonetheless, waves of moderate height are most important for the dynamic wave loading since their frequency is closer to the eigenfrequencies of the structure and their probability of occurrence is high.

The energy distribution of the wind generated waves corresponds to the macro-scale wind spectrum in Figure 4.2. Low energy is observed between one and six hours. Commonly a sea state is regarded stationary within a period of 3 hours. So, again short-term and long-term statistics can be separated.

Figure 4.8: Qualitative energy spectrum of ocean waves [4.32]

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2 The concentration of wave energy with periods of approximately 0.1 hour does not correspond directly with the wind energy spectrum because seiches result from local resonance phenomenon of waves in bays and harbours.
Wave spectra

The disturbance of the water surface elevation by random ocean waves is described by semi-empirical energy spectra. In distinction to a turbulent wind field no coherence function is required as the entire flow field is derived from a single point on the still water surface only. The surface profile in the vicinity of the structure is composed from wave trains and the kinematics of the water particles is determined entirely from the motion of the water surface vertically above it. Various parametric forms of spectra for wind generated waves can be found in the literature. A comprehensive overview is provided by [4.33, 4.34].

In order to understand some basic characteristics of wave energy spectra a simple wave generation model is elaborated (Figure 4.9). Suppose an initially calm ocean and a constant mean speed. The shear force and the differences of the dynamic pressure over a crest and trough are generating perturbations of the free surface. Now waves with a low phase velocity and corresponding high frequency and short wave length are excited because of the large difference between the wind and wave phase velocity. The height of a wave grows until it reaches the deep water breaking limit. Then kinetic energy is transferred to other waves with lower frequency. The new waves are again enlarged by the wind. This process of shifting energy down the frequency scale is continued until a certain limit where wind speed and wave phase velocity are in equilibrium. Therefore wave spectra (Figure 4.10) have a long trail towards the high frequency end but a steep drop in energy below which wave energy can only come from breaking high frequent waves. The wind has to blow for a certain duration and waves need a sufficiently long distance of propagation to generate a fully developed sea. For high wind speeds, both conditions are relatively rare, hence most of the related sea states are either duration limited or fetch limited.

Assuming a Gaussian distribution of the water surface elevation, the random waves can be resolved in a superposition of an infinite number of regular wave trains with various frequencies and wave lengths. Each elementary wave train is of sinusoidal form with an infinitesimally small amplitude. In most cases it is assumed that all waves are propagating in the same direction. This is denoted as long-crestedness owing to the uniform lateral wave field. In reality waves from different directions are interfering and any sea state is short-crested to a certain extent.

Wave spectra are separated in an omnidirectional frequency spectrum and a commonly frequency-invariant function for the directional spreading if short-crestedness is considered. The most relevant spectra for engineering purposes are the Pierson-Moskowitz (PM) spectrum and the JONSWAP (Joint European North Sea Wave Project) spectrum.

The Pierson-Moskowitz spectrum in its original form is based on one parameter only, i.e. the hourly mean wind speed at 19.5 m height [4.36]. Assuming fully developed sea

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3 In literature at least four different spectral periods are distinguished referring to the mean zero-up crossing period, the spectral peak, the average period of the highest third of the waves and the mean wave period. Particular care is required to identify them in a given formula for a certain spectrum. Reference [4.33] provides accurate general definitions. For Pierson-Moskowitz spectra also conversion formulas are given.
states with infinite water depth, fetch and duration only, the spectrum is much too conservative. Frequently a modified, two-parametric form [4.33] is considered for which energy content and frequency range can be controlled by prescribed values of the significant wave height $H_s$, i.e. the average of the 1/3 highest waves in a record, and the zero up-crossing wave period $T_z$.

Fetch and duration limited sea states are better represented by the JONSWAP spectrum [4.37, 4.38]. A third parameter, the peak enhancement factor $\gamma$, controls the amplification of the spectral peak and governs the Pierson-Moskowitz spectrum for $\gamma$ equal to one. The peak enhancement ranges from one to seven with a mean of 3.3. Just for comparison, Figure 4.10 shows different spectra for the same values of $H_s$ and $T_z$. For larger $\gamma$ the energy is more concentrated and shifted to higher frequencies.

The choice of the wave spectrum depends also on the available set of sea state parameters. In practice often only the scatter diagram for wave height and period exist.

![Figure 4.9: Sea state development [4.35]](image)

![Figure 4.10: Comparison of Pierson-Moskowitz spectrum and JONSWAP wave spectra for different peak enhancement $\gamma$ but same value of $H_s$ and $T_z$]
and the Pierson-Moskowitz spectrum is used for fatigue calculations and the JONSWAP spectrum with a mean value of the peak enhancement of $\gamma = 3.3$ is applied for extreme conditions. This thesis emphasizes fatigue aspects and the Pierson-Moskowitz spectrum is used unless stated otherwise.\footnote{Actually, this treatment is likely not conservative for typical OWEC support structures since the physically more appropriate JONSWAP spectra would yield relatively more excitation energy at the fundamental frequency of the structure than the used Pierson-Moskowitz spectra.}

**Directionality, directional spreading and wave misalignment of sea states**

The sea state distribution included in a scatter diagram varies for different mean wave directions owing to the associated differences of fetch, water depth and wind speed distribution. In offshore engineering this so-called 'directionality' of sea states is either neglected by averaging the wave characteristics of all directions, so-called omnidirectional waves, or by analysing the response in 8 to 12 directional sectors. Independent of the directionality the wind and wave direction are misaligned to a certain extent and the sea states are showing a 'directional spreading' of energy or short-crestedness. The various forms proposed for the directional distribution of wave energy all consider a bell-shape spreading with respect to the dominant direction. One well-known formula distributes the wave energy over $\pm 90^\circ$ symmetrical to the main wave direction by weighting the spectral density by a power of a cosine function. Without further data a quite narrow distribution should be assumed which obtains 46% and 78% of the total wave energy within a directional sector of $\pm 15^\circ$ and $\pm 30^\circ$, respectively [4.1]. In most circumstances it is conservative to neglect short-crestedness. However, during production offshore wind energy converters experience aerodynamic damping of the response in the direction of the rotor axis, which generally coincides well with both wind and wave direction. At the same time only structural damping of the lateral response exists. Consequently, the combined wind and wave response might be more onerous if wave misalignment and/or directional spreading is accounted for (Section 9.6.1).

**Extreme waves**

In most cases establishment of extreme sea state conditions and associated extreme design waves for, say, 50 year return period has to be done by extrapolation from a few years of measurements. Such extreme value statistics are highly dependent on the assumed shape of the low probability tails of the parent distributions, which are not well known. The three parameter Weibull or the Fisher-Tippet I distribution are used with reasonable success for extrapolation of extreme significant wave heights [4.38]. For instance, Germanischer Lloyd [4.13] states two procedures to extrapolate the distribution of wave heights and to establish an extreme design wave. The standard treatment for common fixed offshore structures includes analyses of single design waves with different associated wave periods and directions in conjunction with a non-linear wave theory. Alternatively, the extreme sea state loading is investigated by a stochastic approach.
Water kinematics and wave theories

The structural members of most offshore installations are small compared to the wave length. Such structures are considered to be hydrodynamically transparent, i.e. the flow field is not altered by the presence of the structure. Only when the diameter of a local element exceeds 20% of the wave length the structure is called 'hydrodynamically compact' and the flow is corrected for diffraction and refraction (Section 5.2.5).  

Assuming a periodic or regular wave is helpful to understand some fundamentals of irregular waves. More importantly, regular waves are considered in the analysis of the extreme loading of offshore structures with low dynamic response. A regular wave is approximately sinusoidal if the wave height, $H$, is small compared to the water depth, $d$. The shape of the wave progresses with the celerity, i.e. wave length, $L$, divided by wave period, $T$. The water particles normally move with a lower velocity on elliptical orbits so that the travelling wave shape is caused partly by a horizontal and partly by a vertical particle motion. In deep water the orbits are circular and the maximum horizontal particle velocity at the water surface equals $\pi H/T$ while in more shallow waters the orbits are more and more stretched in the direction of wave propagation leading to considerably higher horizontal velocities than $\pi H/T$. The diameter of the orbits and consequently the associated velocities and wave forces are decreasing exponentially with depth, $d$. At the sea bottom of shallow waters significant particle velocities in the horizontal plane are observed (Figure 4.11).

Analytical wave theories are dealing only with a single, regular wave and are based upon solutions of the potential flow problem and the associated boundary conditions. Their validity is limited to a certain range of non-dimensional water depth, $d/g \cdot T^2$, and non-dimensional wave steepness, $H/g \cdot T^2$ (Figure 4.12). Linearisation of the boundary condition at the free surface is the base for the Airy or linear wave theory [4.33]. It can be applied in a wide region, i.e. waves with small or intermediate steepness in deep water and for waves with small steepness in intermediate water. Linear theory is also the common base for modelling of irregular waves by superposition of many elementary waves with small amplitudes. Free surface effects and finite wave height are accounted for by a non-linear correction of the flow field. Quite popular is the approach after Wheeler which considers the kinematics at the instantaneous free surface to be identical to those originally calculated at the still water level [4.39].

Stokes wave theories [4.33] are commonly used for extreme waves in deep water and for intermediate depth. In shallow waters and for steep waves with a height up to about 90% of the breaking wave height the Stream function wave theory of different orders are appropriate [4.40]. Within this thesis linear wave theory with Wheeler stretching is applied for sea state analysis. Extreme regular waves are described by higher order Stream function theory.

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5 For engineering purposes the consideration of diffraction effects is restricted to the use of the linear wave theory.

6 Non-linear, irregular wave theories have been proposed but are difficult to apply and are not frequently used for engineering purposes, yet.
Figure 4.11: Particle orbit of regular waves [4.38]

Figure 4.12: Ranges of suitability of regular wave theories [4.38]

(d = water depth, g = gravity acceleration, H = wave height,
L = wave length, T = wave period)
Spilling breaker \( \zeta \leq 0.5 \)  
Plunging breaker \( 0.5 \leq \zeta < 3 \)  
Surging breaker \( \zeta \geq 3 \)  

**Figure 4.13:** Breaking wave types as function of non-dimensional slope \( \zeta = \tan \alpha \sqrt{g T^2 / 2 \pi H} \) \[4.41\]

*Shallow water effects and breaking waves*

Due to the restricted water depth the generation of high waves is hindered or high waves propagating from the open sea towards the shore become steeper. The wave height increases while the wave period remains constant and the wave length decreases. The more shallow the water the more non-linear such waves behave. The crest height increases while the through decreases and water particle velocities are significantly higher in the wave top than in the through (Figure 5.15).

Compared to most offshore structures, except lighthouses and coastal engineering installations, wave breaking is much more relevant for offshore wind energy applications located typically in shallow waters. Breaking takes place when the water particle velocity in the crest exceeds the celerity, the propagation speed of the wave. According to their appearance, breakers are classified as spilling, plunging or surging (Figure 4.13). Spilling breakers occur at a wave height of approximately 0.78 times the water depth in shallow water or for 0.14 times the wave length in deep water. Seabed slopes and strong winds in the direction of the waves reduce the breaking wave height further. According to these conditions wave breaking begins to be of importance for water depths below 2.5 times the significant wave height. In the southern North Sea this is the case for depths below approximately 22.5 m which are typical for near future offshore wind farms. The kinematics of spilling waves are well modelled by non-linear regular wave theories for finite wave heights. Especially Stream function wave theory according to Dean [4.40] is suited.

A nearly horizontal water surface and a spout form are observed at plunging breakers resulting from sudden breaking due to a seabed slope. The description of the kinematics and the pressure field is difficult and numerical models based upon computational fluid dynamics rather than regular wave theories are required [4.42, 4.43]. As an alternative to a modelling of the velocities in the crest other literature, e.g. DS 449, proposes a distribution function for the impulsive wave forces in conjunction with a wave slamming formula [4.44, 4.45]. A general overview is provided by [4.38]. For application to offshore wind energy converters the reader is referred to [4.46].

Surging breakers are found when long waves of moderate height approach the shore in very shallow water and if the sea bed slope is steep. They are not relevant for our purposes.
Equidistant discretisation

\[ \Delta f = \text{const.} \quad A_i \neq A_j \neq A_k \]

Equi-amplitude discretisation

\[ A_i = A = \text{const.} \quad \Delta f \neq \Delta f \neq \Delta f \]

Enhanced discretisation

- Range 1: equi-amplitude discretisation
- Range 2: as range 1, but refined spacing around eigenfrequency \( f_o \)
- Range 3: linearly increasing spacing

**Figure 4.14:** Spectral discretisation techniques for simulation of random waves

**Simulation of irregular waves**

The wave energy spectra can be directly used for a linear spectral analysis of the sea state loading as commonly done in the offshore technology. In many cases the non-linear drag loading can be linearised and only if the system's response is significantly non-linear, a time domain approach is preferable.

Generation of time series with a random representations can be done by different approaches. Generally the spectrum is discretised in equidistant frequency slices each with the same width, \( \Delta f \), but with a different portion of the total energy and thus each resulting in an elementary wave with different amplitude, \( A_i \), (Figure 4.14 top). A number of frequencies equal to a power of two is compatible with the Inverse Fast Fourier Transform (IFFT). Usually sinusoidal elementary waves with random phase angles are
superimposed. The summation of sine and cosine components with random amplitudes is another technique, which is discussed with some other useful aspects in [4.33, 4.47, 4.48]. The time series from any of these techniques repeat with a period of $1/\Delta f$. For instance, $16384 = 2^{14}$ frequencies are required to generate a three hours time series without periodicity representing the spectrum up to a reasonable cut-off frequency of 0.628 Hz.\(^7\)

Matthies et al. [4.46] used a spectral discretisation in slices of same energy but with non-equidistant frequency increments (Figure 4.14 middle) leading to an accurate representation of the spectrum and some advantages for the simulation of extreme waves. Furthermore, the time series is non-periodic, even with a relatively low number of elementary waves of, say, 100 to 200.

The resulting coarse frequency spacing in the high-frequent tail of the wave spectrum is, however, problematical if the fundamental eigenfrequency is situated in this range, which is typical for support structures of offshore wind energy converters. In the DUWECs simulation code, extended during the present research, we overcame this shortcoming by a specific discretisation of three spectral ranges (Figure 4.14 bottom). First, in the vicinity of the eigenfrequency the spectrum is discretised by equi-energy waves with a minimum frequency increment, $\Delta f_{\text{min}} = f_{\omega} \cdot \xi_o$, related to both the eigenfrequency, $f_{\omega}$, and its damping ratio, $\xi_o$. [4.49]. This ensures a proper description of the dynamic response behaviour. Next, the high-frequency tail of the spectrum is divided up with a linearly increasing frequency spacing. Finally, the low-frequency part of the wave spectrum, governing mainly the quasi-static response, is modelled by equi-energy waves as in the original approach. Further details of the approach are documented in [4.50].

**Currents**

Sea currents are characterised as near-surface currents generated by wind and waves, sub-surface currents due to tides or thermosaline currents and near shore wave induced surf currents. The UK Department of Energy and Germanischer Lloyd provide further guidance [4.1, 4.13].

Due to the relatively shallow water depth, typical for a bottom-mounted OWEC, the current loading is of lower significance but should be considered in the extreme load calculations. Indirect effects such as erosion of the seabed in the vicinity of the structure or wave-breaking due to an opposing current can also be relevant.

### 4.1.3 Other parameters

#### Variation of the sea level

The overall water depth at a location consists of a mean depth and a fluctuating component which varies with the still water level (SWL) due to tides and storm surges (Figure 4.15). The highest astronomical tide (HAT) and lowest astronomical tide (LAT) are regular and predictable. Meteorologically generated storm surges are irregular and are superimposed on the tidal variations. Total still water levels above HAT and below LAT may occur. In addition, long-term changes of the sea level due to natural or man-made influences and motion of the sea bottom might occur. Again the UK Department of Energy provides comprehensive guidance on the establishment of design values

---

\(^7\) This technique corresponds to the generation of the longitudinal wind turbulence in a single point of the rotor plane (Section 4.1.1).
Water depth shown in Admiralty charts are commonly related to LAT or a Chart Datum (CD), a reference level close to LAT.

In shallow waters it is unclear beforehand whether maximum or minimum water depth generates the most onerous loading. A lower depth reduces the lever arm of the hydrodynamic forces but increases the wave steepness leading to more non-linear behaviour and possibly wave breaking. The more shallow the water the higher heave forces are acting on support structures of the caisson type due to the pressure head of passing waves.

In a fatigue analysis the mean sea level (MSL) is often assumed as constant still water level over the entire lifetime.

Icing and sea ice

Icing of the wind turbine and the superstructure from freezing snow, rain or fog can occur but has lower importance than on land. More relevant seems the possible accumulation of freezing sea spray, which was observed on platforms in the Dutch North Sea and the Baltic. Norwegian standards state up to 80 mm ice thickness from wave spray and a maximum of 10 mm from rain and snow for locations up to 56° N latitude.

Sea ice is a very significant extreme loading for Baltic sites due to a low salinity and cold climate, whilst it is negligible for the North Sea or the Atlantic. Proper design requires accurate investigations on the local ice properties, especially thickness and

<table>
<thead>
<tr>
<th>Table 4.3: Indicative design properties of sea ice</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Site</strong></td>
</tr>
<tr>
<td>Northern Golf of Bothnia [4.51]</td>
</tr>
<tr>
<td>Rødsand, Denmark ‡ [4.52]</td>
</tr>
<tr>
<td>Øresund, Denmark † [4.53]</td>
</tr>
</tbody>
</table>

design ice density 900 kg/m³ for all sites
† 3.0 MPa intact moving ice at the coldest time of the winter
2.5 MPa intact slowly moving ice (e.g. thermal expansion), coldest time of the winter
1.5 MPa intact moving ice in the spring, temperature close to melting point
1.0 MPa partially weakened melting ice moving to melting temperature
‡ design value for return period 50 years
strength, and the ice motion. Predictions are complicated due to variation of salinity, ice
temperature, ice history, size and velocity of ice floes and possible ice ride-up. Indicative
design ice properties are provided by Table 4.3 while ice force formulas are discussed
in Section 5.2.6.

Temperature

The large thermal capacity of the sea stabilizes the temperature variation known from
land. Differences between water and air temperature and temperature gradients are
important for determining atmospheric stability conditions, wind shear and turbulence
but are not directly relevant for structural design. Extreme minimum temperature values
for the selection of the steel and concrete grade range between -20° C in the southern
Baltic, -12° C in the Dutch North Sea to -4° C around the western British islands.
Maximum values relevant for the dimensioning of the wind turbine cooling system do not
exceed 25° C in the mentioned regions [4.1, 4.54, 4.55].

Marine growth

Fouling generates extra structural mass and additional hydrodynamic mass. The
thickness can grow up to 100 to 300 mm near the water surface. The modified surface
roughness increases the hydrodynamic drag coefficient (Table 5.4). For our type of
structures, mainly loaded by the hydrodynamic inertia force, the enlarged structural
diameter is of importance. Typical design properties are given by [4.1, 4.56].
Structures of the lattice or tripod type with slender members and greater surface are
more sensitive to marine growth than compact monopile or caisson designs.

4.2 Correlation of wind and wave parameters

So far we have addressed the met-ocean parameters separately, now we will draw our
attention to the correlation of wind and wave conditions, in particular.

Met-ocean databases and long-term statistics

The local site conditions such as fetch, water depth, sea bottom topography, etc. affect
also the correlation of the met-ocean parameters. Simple empirical closed-form
expressions are useful only for the early design stages (Section 10.1). Figure 4.16
compares the Neumann-Pierson relation [4.57] (10.1) valid for fully developed sea states
with the actual relation between the mean significant wave height and the mean wind
speed for different directions in the Dutch North Sea close to the site NL-1 (Figure 9.1).

The only alternative to site measurements is the application of numerical hindcasting
techniques that derive magnitudes and directions of simultaneous wind, wave, current,
tidal and surge parameters from meteorological data as atmospheric pressure
distributions and related wind fields. Only research institutes and meteorological offices
are capable of such complex models that process the data simultaneously at many grid
points of a large geographical region and for a long period covering the last 30 - 35
years. The results are validated against field measurements and are stored in
Figure 4.16: Relation between mean significant wave height and hourly mean wind speed at a Dutch North Sea site [4.59]

Figure 4.17: Distribution of mean wind and wave direction at a Dutch North Sea site [4.59]

Figure 4.18: Misalignment of mean wave direction with respect to mean wind direction at a Dutch North Sea site [4.59]
high-quality met-ocean databases [4.58]. Beyond a water depth of about 20 m such environmental data can be derived without too much disturbance by the local bottom topography. Application to sites with lower depth and considerable changes of the depth and bottom topography relative to the nearest grid point, should be carried out with care.

For Northwestern Europe the NESS database (North European Storm Study) is of particular importance [4.59, 4.60]. Within the Opti-OWECS study, data for two grid points in the Dutch North Sea were made available though Shell International Exploitation and Production B.V. A continuous record of nine years enabled reliable fatigue calculations and extreme parameters could be extrapolated from all storms during a period of 30 years. In Chapters 9 and 10 these grid points are matched with the sites NL-1 and NL-5 (Figure 9.1).

As an example, Figure 4.17 compares the distribution of the mean wind and mean wave direction showing prevailing southwesterly winds. Two main wave directions are observed which correspond to the main wind direction and the northern direction from the Central North Sea with associated long fetch. In Section 9.6.1 the wave misalignment is further investigated and collinearity of wind and waves is found a reasonable assumption for omnidirectional waves as 65% of the waves are within a ± 22.5% sector around the wind direction (Figure 4.18). The three-dimensional scatter of wind, wave height and wave period for the same site is shown in Figure 6.7 and Table 8.1.

Short-term statistics

It is common practice to assume the met-ocean conditions as statistically constant over an averaging period of three hours. In wind engineering, often a period of 10 minutes or one hour is used instead. Both periods are situated in the spectral gap of winds and waves (Figures 4.2 and 4.8). In the following the entire set of environmental conditions, rather than only the wave parameters, is denoted as a sea state.

Fatigue analysis is an in principle straight-forward but cumbersome task if a suitable model of the entire offshore wind energy converter (Chapters 5 and 8) and a scatter diagram with all sea state parameters is available. Both time and frequency domain approaches rest upon the independence of wind and wave loading on short time scales.

Establishment of the simultaneous extreme environmental parameters for a certain sea state is not easy for several reasons. Wind and wave loads can have a similar order of magnitude. In contrast, the design of most bottom-mounted offshore platforms is governed by extreme wave loads, whilst the wind contribution accounts for typically around 10% of the total loading. Secondly, dynamic response is more pronounced than for most other structures of the petroleum industry. Finally, the operational conditions of the OWEC require attention. The extreme response can occur during the power production mode associated with sea states far below the extreme 50 years conditions.

Joint probabilities for extreme gust and extreme design waves can be derived assuming Gaussian wind turbulence and Rayleigh distributed waves. On this rationale Germanischer Lloyd recommends extreme load cases associated with either the most probable extreme wave or wind gust for a given sea state (Table 4.4). Such treatment is suited to engineering purposes and is followed within this thesis but it has shortcomings. Two thirds of the maxima are higher than the most probable extreme value and only a probabilistic description is accurate. The most onerous environmental
Table 4.4: Wind and wave correlation for extreme load cases according to Germanischer Lloyd [4.13]

<table>
<thead>
<tr>
<th>OWEC condition</th>
<th>Identifier</th>
<th>Wind conditions †</th>
<th>Hydrodynamic conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>production</td>
<td>E1.1</td>
<td>mean wind within production range: 13 m/s</td>
<td>sea state corresponding to mean wind: - reduced wave $H_{red} = 1.32 H_s$</td>
</tr>
<tr>
<td></td>
<td>E1.5 ‡</td>
<td>- extreme operating gust 13 m/s - reduced 1 min gust</td>
<td>- extreme wave $H_{max} = 1.86 H_s$</td>
</tr>
<tr>
<td>parked</td>
<td>E2.1</td>
<td>50-year mean wind: - extreme 3 s gust (1.2 times mean wind)</td>
<td>50-year extreme sea state: - reduced wave $H_{red} = 1.32 H_{150}$</td>
</tr>
<tr>
<td></td>
<td>E2.2</td>
<td>- reduced 1 min gust (1.09 times mean wind)</td>
<td>- extreme wave $H_{max} = 1.86 H_{150}$</td>
</tr>
<tr>
<td></td>
<td>E2.4</td>
<td>50-year mean wind</td>
<td>50-year sea ice</td>
</tr>
</tbody>
</table>

† collinear mean wind and wave direction, but prescribed change in instantaneous wind direction, ‡ generally the E1.5 load case is not governing

parameters should be derived from the parent distribution of the extreme response rather than the joint-probabilities of the environmental conditions. Recently the application of structural reliability methods, developed in advanced offshore engineering to offshore wind energy converters proposes solutions for this problem [4.61].

Safety factors

Germanischer Lloyd proposed a lower partial safety factor on extreme environmental loads of $\gamma_F = 1.35$ for offshore wind energy application instead of the value of $\gamma_F = 1.5$ for land-based installations (Table 4.2). This is due to the loose correlation between short-term extreme wind and waves loads and the reduced risk of human life due to a major failure of an OWEC compared to a wind turbine on land or an offshore structure of the petroleum industry.

4.3 Soil conditions and auxiliary external effects

Soil conditions

The demands of OWEC foundations differ from those of more common offshore structures. Governing design criteria for the foundation of an oil platform are often the ultimate bearing capacity and sliding resistance, as well as overall stability against overturning. Therefore the minimum strength under static (or dynamic) loading is one of the most important soil parameters. Mainly the lower bound of the foundation stiffness is crucial since it corresponds with the minimum fundamental eigenfrequency and maximum dynamic amplification of the wave loading. Any variation of the soil properties leading to a stiffer behaviour of the structure and hence less onerous condition is welcome.
In contrast, both minimum and maximum soil stiffness are relevant for OWEC since the structural eigenfrequencies have to be tailored with respect to the three main excitation ranges, the dynamic wave loading and both rotor and blade excitations. Chapter 10 proposes a rational treatment of the uncertainties in the foundation behaviour during the course of the design process. The values of the soil parameters for strength and stiffness vary significantly depending on soil type, loading history and location. Tables 4.5 and 4.6 give indicative soil properties that correspond to the models described in Sections 5.2.2 and 5.2.3.

**Table 4.5: Indicative soil properties for gravity foundations**

<table>
<thead>
<tr>
<th>Soil</th>
<th>Submerged unit weight $\gamma_{sub}$ [kN/m³] [4.62]</th>
<th>Dynamic shear modulus $G_{dyn}$ [MPa] [4.63]</th>
<th>Poisson ratio $\nu$ [-] [4.63] [†]</th>
</tr>
</thead>
<tbody>
<tr>
<td>sand (loose)</td>
<td>10</td>
<td>50 - 120</td>
<td>0.25 - 0.35</td>
</tr>
<tr>
<td>sand (medium dense)</td>
<td>11</td>
<td>70 - 170</td>
<td></td>
</tr>
<tr>
<td>gravel (sandy, dense)</td>
<td>10</td>
<td>100 - 300</td>
<td></td>
</tr>
<tr>
<td>mud, clay</td>
<td>9</td>
<td>3 - 10</td>
<td>0.35 - 0.45 [††]</td>
</tr>
<tr>
<td>clay (soft to firm)</td>
<td>10</td>
<td>20 - 50</td>
<td>0.45 - 0.49 [††]</td>
</tr>
<tr>
<td>clay (stiff to very stiff)</td>
<td>11</td>
<td>80 - 300</td>
<td></td>
</tr>
<tr>
<td>rock (layered, cracked)</td>
<td></td>
<td>1,000 - 5,000</td>
<td></td>
</tr>
<tr>
<td>rock (solid)</td>
<td></td>
<td>4,000 - 20,000</td>
<td></td>
</tr>
</tbody>
</table>

† [4.63] recommends calculation with ± 50% variations to account for uncertainties
‡ [4.38] recommends the undrained Poisson ratio $\nu = 0.5$ for cyclic loading
†† silt, depending on sand and clay content; ‡‡ clay, depending on water content

**Table 4.6: Indicative soil properties for piled foundations [4.38, 4.64]**

<table>
<thead>
<tr>
<th>Sand / gravel</th>
<th>Submerged unit weight $\gamma_{sub}$ [kN/m³]</th>
<th>Friction angle $\phi$ [deg]</th>
<th>Initial subgrade reaction $K$ [MPa/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>silty, clayed</td>
<td>7 - 13</td>
<td>25 - 35</td>
<td></td>
</tr>
<tr>
<td>loose</td>
<td>7 - 9</td>
<td>28 - 30</td>
<td>2 - 7</td>
</tr>
<tr>
<td>medium dense</td>
<td>8 - 10</td>
<td>30 - 36</td>
<td>7 - 28</td>
</tr>
<tr>
<td>dense</td>
<td>10 - 11</td>
<td>36 - 40</td>
<td>28 - 41</td>
</tr>
<tr>
<td>dense, very dense</td>
<td>11 - 13</td>
<td>40 - 45</td>
<td>41 - 59</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Clay</th>
<th>Submerged unit weight $\gamma_{sub}$ [kN/m³]</th>
<th>Undrained shear strength $C_u$ [kPa]</th>
<th>Strain at 50% of peak stress $\varepsilon_{50}$ [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>clay (soft to firm)</td>
<td>4 - 7</td>
<td>10 - 50</td>
<td>5 - 20</td>
</tr>
<tr>
<td>clay (stiff to very stiff)</td>
<td>9 - 10</td>
<td>50 - 200</td>
<td>5 - 20</td>
</tr>
</tbody>
</table>
Scour

Loose sand and soft clay are sensitive to seabed currents around the base of the structure. Strong tidal currents and wave induced currents in the breaking wave zone may lead to sediment scour, which will affect the generic foundation types differently. Piled foundations will suffer from a localised reduction in overburden pressure and a loss of lateral resistance. The longer lever arm of the environmental forces directly increases the bending loads in the pile and a softening of the dynamic characteristics results in more pronounced dynamic response on wave excitation and possible resonance with the rotor excitation for soft-stiff designs. The magnitude of scour for large monopiles in shallow water is difficult to predict but values of three to six metres seem realistic for silicious soils. In the design of offshore jacket structures commonly 1.5 times the leg diameter of local scour and one metre of global scour is allowed for [4.38]. Germanischer Lloyd recommends a value of 2.5 times the pile diameter if no other data are available [4.13]. This figure seems, however, too conservative for large monopiles. Gravity structures may undergo erosion of soil from underneath the base of the structure if skirts do not penetrate until sufficient depth. In shallow waters scour is likely to exceed the maximum value of 2 m observed at structures in the central and northern North Sea [4.38].

There are two main design options to deal with sea bed erosion; allow for scour in the design or to monitor the actual scour and replace the material if necessary. The latter is also needed if scour protection was already provided during installation.

Auxiliary external effects

Corrosion is an important consideration in the design of both steel and concrete marine structures. Sites in the North Sea and Atlantic Ocean are more sensitive than in the Baltic Sea owing to the harsh environment and the 4 to 6 times higher content of aggressive ions [4.65].

Little evidence exists about the frequency of lightning strikes at offshore wind energy converters compared to their land-based counterparts. Generally a higher degree of lightning protection is recommended due to greater exposure of the structure, the longer downtime after a severe incident and the potentially higher repair cost.

Depending on the site and design conditions, other loadings such as ship impact, earthquake, mobile sand features, etc. might require particular consideration. Initial inclination or settlement during exploitation can induce significant gravity loads in the slender and tall support structure and might affect proper function of the yaw system of the wind turbine.

Influences arising from the energy consumers need attention during design of an offshore wind farm, as well. Grid failure and/or loss of load are to be considered as normal external conditions while major frequency, voltage and load fluctuations, interfering voltages and line short-circuit are examples of extreme influences which can affect not only the electrical system and the drive train but also the rotor and support structure.
CHAPTER 5

MODELLING OF OFFSHORE WIND ENERGY CONVERTERS

Design calculations for offshore wind energy converters (OWEC) require a mathematical model which is based upon the state of the art in both wind energy and offshore technology. In order to limit the complexity to a level appropriate to engineering application a particular approach was developed emphasising aspects most relevant to bottom-mounted offshore wind energy converters.

In this chapter the modelling of the two main sub-systems, the wind turbine and the support structure is elaborated in Sections 5.1 and 5.2. There are however several sub-system aspects that affect the entire system. So, Section 5.3 discusses the time invariant properties of the wind turbine rotor, the aerodynamic damping and the variation of the system dynamics from an integral viewpoint.

Obviously the choice of a suitable analysis approach for the entire system must be compatible with the properties and modelling of all sub-systems. Therefore this chapter describes only the treatment of the sub-systems, while Chapter 6 introduces the so-called integrated, modular, non-linear time domain simulation approach for the entire offshore wind energy converter.

Aiming at a compact description, most attention is given to those aspects of offshore wind energy converters which differ from their land based counterparts.

5.1 Wind turbine modelling

General remarks on the applied model

For this thesis the onshore wind turbine design tool DUWECs (Delft University Wind Energy Converter Simulation) [5.1, 5.2] was extended by many features required to analyse offshore wind energy converters. Emphasis was put on an integrated model including wind turbine and support structure. So most enhancements were required with respect to the support structure and its loading (Section 5.2) and the computational efficiency of simulations of the simultaneous aerodynamic and hydrodynamic loading (Sections 6.3 and 6.4). The wind field representation and the structural dynamics of the wind turbine model [5.3] was extended but the implementation of the rotor aerodynamics remained nearly unchanged [5.4].

As a matter of fact the applied DUWECs model includes (Figure 5.1) all major features required for wind turbine design calculations but suffers some limitations with respect to the recent developments of onshore design calculations. Shortcomings include the
three-dimensional turbulence, state of the art rotor aerodynamics and multiple blade bending modes. Table 5.1 compares the used program to a popular, commercial simulation code [5.5]. Due to the consistency with the work within the Opti-OWEC study most simulations in this thesis are computed with the ROTOR2 module of DUWECs, a two-bladed rotor model with fore-aft translation degree of freedom. For some particular analyses the more recent module ROTOR3 with two or three rotor blades, both fore-aft and lateral translation degree of freedom and some other improvements are used. The overall suitability of the chosen modelling was confirmed with a benchmark experiment carried out by comparison to another commercial design tool [5.6, 5.7].

The rest of this sections presents the essentials of wind turbine modelling for design calculations of OWEC more from a general point of view. Comments on the specific implementation in this thesis are given where appropriate. More comprehensive information on the varieties of wind turbine modelling is available from overview papers [5.8, 5.9] and textbooks [5.10 - 5.14].

5.1.1 Non-homogeneities of the wind field

The inflow comprises two contributions; a stochastic part from the ambient and wind farm induced turbulence and a deterministic portion owing to wind shear, yawed wind direction and tower influence. The former aspect was discussed in detail in Section 4.1.1 where the importance of a sophisticated representation of spatial and temporal structure of the wind was highlighted.

Now we will have a closer look at the stationary composition of the wind field. Modelling of the stationary flow field is straightforward. A geometric transformation of the flow accounts for yawed flow due to a misalignment or delay of the yaw tracking system and inclined stream owing to the tilt angle of the rotor shaft, unstable atmospheric conditions or complex terrain conditions. For design calculations a power law for the wind shear rather than a logarithmic wind profile is used.

Similar to the rotational sampling of turbulence, also the deterministic disturbances of the spatial flow field are observed as temporal variations of the local wind speed and
Figure 5.2: Effect of yawed flow, wind shear and rotor tilt on Fourier Series decomposition of the blade bending and nacelle tilt moment (Rigid model of a two- or three-bladed, stall controlled 500 kW turbine with constant solidity and tip speed)

Figure 5.3 Effect of tower shadow and rotor tilt on Fourier Series decomposition of the blade bending and nacelle tilt moment (Rigid model of a two- or three-bladed, stall controlled 500 kW turbine with constant solidity and tip speed)
### Table 5.1: Comparison of applied wind turbine modelling to a state-of-the-art simulation code

<table>
<thead>
<tr>
<th>Feature</th>
<th>DUWECs</th>
<th>FLEX4</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Wind description</strong></td>
<td>• wind shear (power law)</td>
<td>• wind shear (power law)</td>
</tr>
<tr>
<td>• deterministic</td>
<td>• tower shadow (dipole)</td>
<td>• tower shadow (dipole)</td>
</tr>
<tr>
<td></td>
<td>• yawed and inclined flow</td>
<td>• yawed and inclined flow</td>
</tr>
<tr>
<td>• stochastic</td>
<td>• axial turbulence (SWING 3 code, rotating</td>
<td>• 3D turbulence (Veers model)</td>
</tr>
<tr>
<td></td>
<td>observer) with deterministic variation of lateral component by sine law</td>
<td></td>
</tr>
<tr>
<td><strong>Rotor aerodynamics</strong></td>
<td>• uniform axial induction</td>
<td>• axial and tangential induction per annuli</td>
</tr>
<tr>
<td>• blade element momentum theory</td>
<td>• $c_l$ and $c_d$ from tables</td>
<td>• $c_{\mu}$ and $c_m$ interpolated from tables</td>
</tr>
<tr>
<td>• turbulent wake state</td>
<td>• Johnson correction</td>
<td>• Glauert correction</td>
</tr>
<tr>
<td>• blade root and tip effects</td>
<td>• lift loss factor</td>
<td>• Prandtl tip loss factor</td>
</tr>
<tr>
<td>• 3D correction of profile</td>
<td>• depending on airfoil tables, e.g. Viterna &amp;</td>
<td>• depending on airfoil tables</td>
</tr>
<tr>
<td>aerodynamics</td>
<td>Corrigan, Snel, etc.</td>
<td></td>
</tr>
<tr>
<td>• dynamic inflow</td>
<td>• 1st order differential equation on induction factor</td>
<td>• dynamic wake model for induced velocity per annuli</td>
</tr>
<tr>
<td>• unsteady profile</td>
<td>• ONERA (not used)</td>
<td>• Stig Øye dynamic stall model</td>
</tr>
<tr>
<td>aerodynamics</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Structural dynamics</strong></td>
<td>• flap and lead-lag hinge model</td>
<td>• 2 flapwise, 2 edgewise modes</td>
</tr>
<tr>
<td>• blade</td>
<td>• rotation, torsion</td>
<td>• rotation, x- &amp; y-bending, torsion</td>
</tr>
<tr>
<td>• drive train</td>
<td></td>
<td>• tilt</td>
</tr>
<tr>
<td>• nacelle</td>
<td></td>
<td>• fore-aft, lateral &amp; torsion</td>
</tr>
<tr>
<td>• tower top</td>
<td>• fore-aft (ROTOR2),</td>
<td></td>
</tr>
<tr>
<td></td>
<td>fore-aft and lateral (ROTOR3)</td>
<td></td>
</tr>
<tr>
<td>• tower &amp; foundation</td>
<td>• Finite Element model ($n$ modes)</td>
<td>• 1st fore-aft, lateral &amp; torsion mode</td>
</tr>
</tbody>
</table>
## Table 5.1 (cont.): Comparison of applied wind turbine modelling to a state-of-the-art simulation code

<table>
<thead>
<tr>
<th>Feature</th>
<th>DUWECS</th>
<th>FLEX4</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Transmission &amp; Generator</strong></td>
<td>• gear box flexibility (optional)</td>
<td>• induction or synchronous generator</td>
</tr>
<tr>
<td></td>
<td>• induction or synchronous generator</td>
<td>• fixed or variable speed</td>
</tr>
<tr>
<td></td>
<td>• fixed or variable speed</td>
<td>• link of user built generator routines</td>
</tr>
<tr>
<td></td>
<td>• link of user built transmission and generator routines</td>
<td></td>
</tr>
<tr>
<td><strong>Control</strong></td>
<td>• state space analog controller</td>
<td>• link of user built routine for transducer and actuator dynamics</td>
</tr>
<tr>
<td></td>
<td>• digital controller and filters</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• link of user built control routines incl. transducer and actuator</td>
<td>• link of user built control routines</td>
</tr>
<tr>
<td></td>
<td>dynamics</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• linearisation option</td>
<td></td>
</tr>
</tbody>
</table>
direction in the rotating reference frame. Yaw and wind shear are experienced by the rotor blades as nearly perfect sinusoidal excitation with the rotor frequency (1P).

Figure 5.2 illustrates a Fourier decomposition of the quasi-static blade root and nacelle tilt moments of two fictitious turbines with same solidity. The blade root loading of a blade of a three-bladed rotor is about 67% of a two-bladed design owing to the reduced blade area. In the fixed reference system of the nacelle the excitations are seen mainly as multiples of the blade passing frequency. While a significant averaging between the three blades is observed, the opposite holds for the two-bladed case where the tilt contributions from both blades are acting in phase.

For upwind orientated turbines the blocking effect of the tower and nacelle is commonly modelled by a solution of the potential flow problem. The blade suffers the sharp velocity deficit and deformed streamlines in front of the tower mainly as a 1P excitation but a large number of higher harmonics contribute to a slowly decreasing but low level (Figure 5.3). In the fixed reference frame of the nacelle the forcing by the blade passing frequency is rather high in the two blades case. This is the reason why sometimes two-bladed rotors are combined with a teeter hub which is compensating the 2P loading by a counteracting out-of-plane motion of the blades. The three-bladed rotor is behaving again similar to a disk which smoothen the inflow. It is amazing to see that the three blades are sampling also a low 2P excitation.

Two-bladed concepts are appealing for offshore siting due to various aspects. On sea they can operate with an optimal and high tip speed ratio. Installation is simplified and aesthetics are of lower concern. Nonetheless, as seen at the above mentioned example, the dynamic loads of two-bladed turbines are considerably higher (at least for constant tip speed). Moreover the periodic variation of the system properties results in a more complex dynamic behaviour.

5.1.2 Rotor aerodynamics

Blade element momentum theory

All wind turbine design tools calculate the aerodynamic loads with a combination of blade element and momentum theory. In order to overcome the shortcomings of this rather crude method several engineering rules are applied as corrections. Despite the rapidly increasing computer power, more advanced methods applying Computational Fluid Dynamics [5.9, 5.15] are still under development and will most likely only be able to support and validate much simpler design tools for years.

The flow effects and associated models in rotor aerodynamics are often distinguished by the different spatial and temporal scales involved (Figure 5.4). The global flow field has dimensions in the order of the rotor diameter and a time scale of several seconds while the blade flow field is related to the chord length and periods of 10 to 50 milliseconds which a particle needs to cross the blade.

The global flow field is described by Froude's actuator disk theory originating from 1889 [5.16] and was adopted to wind turbine applications by Lancaster [5.17] and Belz [5.18]. Commonly it is corrected for conditions with high axial loading (turbulent wake state), for instance with empirical relations according to Glauer [5.19]. Blade element or wind theory with empirically determined airfoil characteristics expresses the local lift and drag forces [5.20].

Both flow fields are linked by the axial (out-of-plane) and tangential (in-plane) 'braking action' at the rotor disk, denoted as induction. On the one hand the induction determines
Figure 5.4: Blade Element Momentum theory linking of the global rotor flow field (left) to the local blade field (right) by the induction force \( dF_x \) of an annulus

The velocities at the rotor disk by the global momentum balance. On the other hand the local aerodynamic forces are depending on the magnitude and angle of incidence, \( \alpha \), of the apparent wind velocity composed from four contributions: undisturbed wind speed, \( V \), including turbulent and deterministic variations; blade azimuthal velocity, \( \Omega \cdot r \), blade motion due to deformation of the blade, nacelle and support structure and the axial and tangential induced velocities\(^1\) (Figure 5.4).

The swept area is divided into 10 to 20 annuli under the rough assumption that no spanwise interference occurs and the induction is distributed uniformly over all annuli. The axial and tangential inductions are determined from the static or dynamic equilibrium depending on the used wake model. Obviously the induction is non-homogeneous if only two or three blades operate. So a correction according to Prandtl [5.21] is applied, denoted with the misleading name 'tip correction'. The Blade Element Momentum (BEM) method was applied for wind turbine rotor design by Schmitz [5.22] and has been synthesised for modern computational use by Wilson and Lissaman et al. [5.23] in the 1970's. Since then it is a daily tool for engineers using it successfully but also a topic for researchers discussing it frequently.

Semi-empirical corrections

The Blade Element Momentum approach has been continuously improved by various engineering corrections. Reference [5.24] provides a comprehensive overview.

The aerodynamic coefficients of the thick airfoils at the blade root sections determined from two dimensional wind tunnel experiments are now corrected for three dimensional and rotational effects of the separated flow where stall is delayed and up to 50% higher lift is observed than predicted by the elementary theory (Figure 5.5 left) [5.25].

In the previous section we got an impression of the high load fluctuations that are

\(^1\) The axial and tangential induced velocities, \( a \cdot V \) and \( a' \cdot V \), are related to the undisturbed (axial) velocity, \( V \), and circumferential velocity, \( \Omega \cdot r \), by the non-dimensional induction factors \( a \) and \( a' \). The tangential induction of a modern wind turbine with high tip speed ratio is very small.
Figure 5.5: Extensions of elementary rotor aerodynamics:

- Left: Three-dimensional and rotating effects delay the stall.
- Right: An oscillations of the angle of attack, $\alpha$, result in a hysteresis of the lift coefficient, $C_L$. (A: Flow reversals within boundary layer, vortex formation. B: Vortex detaches and moves over surface. C: Vortex passes trailing edge, full stall. D: Flow re-attachment.)

The flow around wind turbines operating in the atmospheric boundary layer is inherently unsteady. Dynamic inflow denotes all unsteady aerodynamics associated with the rotor flow field. A common engineering approach changes the static equilibrium of the induction into a radially dependent, dynamic relation with first order delay characteristics [5.26].

On the blade scale the fluctuating external conditions result in hysteresis effects of the aerodynamic coefficients in the stall region (dynamic stall), depending on the blade geometry, the frequency and the amplitude of the flow variation. Figure 5.5 (right) presents a typical behaviour. Common models from helicopter aerodynamics, as for instance reviewed and extended by Björck et al. [5.28], express the time derivative of the lift coefficient as a function of the instantaneous value, the velocity and the acceleration of the angle of incidence.

Analyses of the aero-elastic stability, especially important for large, flexible rotors operating in stall, require both a sophisticated model of the structural dynamics and the unsteady profile aerodynamics [5.29].
5.1.3 Structural dynamics
It is now state of the art to consider dynamic loads in the design process of all load carrying major components of a wind turbine. Therefore design tools comprise models of the structural dynamics of rotor blades, hub, drive train including support and brakes, generator, yaw system, tower, foundation and controller. Due to the specific combination of a rotating structure undergoing gross overall motion with a fixed support structure, particular mathematical models are required which were until quite recently not available in general purpose codes. Therefore most tools were developed over a long period by research institutes particularly with respect to the specific needs of the wind turbine community.
Such models are using either the Finite Element representation or the modal analysis. While the former requires extensive computer power during the actual time domain simulation process the latter approach involves the cumbersome derivation of the equations of motion during the establishment of the model. The majority of tools follow the modal analysis with eigenmodes and frequencies extracted from a multi-body model (Figure 5.1) or a Finite Element model. The typical number of degrees of freedom of the dynamic models varies between 20 and 40 describing the relevant dynamics up to 5 or 10 Hz. Due to the non-linear kinematics and the non-linear rotor aerodynamics all commercial tools rest on time domain Monte-Carlo simulations.
In Sections 6.1.2 and 6.2 we introduce a suitable analysis approach in more detail since the support structure of an offshore wind energy converter features distinctly different modelling requirements than the wind turbine part (Table 6.1).

5.1.4 Mechanical-electrical conversion system
The importance of a transmission and generator system for the system dynamics is explained by two examples of modern turbine designs.
The massive primary shaft of a common 1.5 MW wind turbine has a diameter of approximately 600 mm and a length of some 2.5 to 3.5 metres. Nonetheless it is beside the slender blades and tower one of the most flexible and oscillatory components owing to the enormous torque and the inertia of the rotor and generator.
Variable speed designs require a frequency converter with power electronics for at least a portion of the generator power. Such generator - converter systems are highly active and can be used for tailoring energy yield, fatigue loads, braking and damping the turbine. They have less in common with the traditional, robust induction generators modelled simply with a linear torque - rotor speed characteristic.

Only the first one or two torsion modes of the drive train are of importance. Therefore relatively simple models are surviving. Drive train models with one or two degrees of freedom, for the low speed or both low speed and high speed shaft flexibility, and user defined generator routines are implemented in design tools. The three or four stage gear box (if any) is commonly represented solely by a rigid, massless transmission, the dynamic properties of which are often unknown to both the turbine and gear designer.
In contrast to the generator manufacturer, only the low-frequency dynamics of the generator or generator - converter system are relevant for the turbine designer. Models for the various used types, e.g. induction generator, double-fed asynchronous generator, synchronous generator with DC-link, are described in references [5.1, 5.30 - 5.32].
5.1.5 Supervisory and power control

Control principles applied at wind turbines

The technical and economic performance depends on proper understanding of the control system and its interaction with the rest of the turbine. Actions of the supervisory control as start, stop, brake, yaw, synchronisation manoeuvres have to be considered in design calculations because they contribute to the accumulated fatigue and have an important impact on extreme loads related to design gusts or emergency actions. Commonly the reaction of the simulation model is observed with respect to synthesized signals, of for instance pitch variation, rotor speed ramp or brake torque, or triggered occurrences as emergency shut-down and power loss.

Dynamic power control has to meet three basic requirements: safe limitation of the elementary machine loads of power, rotor speed and thrust, high energy yield at partial load conditions and moderate fatigue loads by limitation of load variations and control actions. Stability, good disturbance rejection and robustness must be met under the variable operational conditions and with autonomous operation. Different answers are provided by the three currently popular wind turbine concepts of either traditional stall - fixed speed, active stall - fixed speed or variable pitch - variable speed.

At traditional stall controlled turbines these tasks rest entirely on the aerodynamic behaviour of which we still have an incomplete understanding. An induction generator with a high turn-over torque ensures fixed rotor speed which results in large angle of incidence and stall when the wind speed increases. The reduction of lift and increase of the drag are limiting the rotor torque and power. Apart from the rotor aerodynamics and possible aero-elastic effects modelling is straightforward.

Active stall designs follow the same principle because the stall effect itself is faster and theoretically simpler than any mechanical control. Relatively slow variations of the blade pitch angle in the rotor plane (to larger angles of attack) assist the stall effect and adjust it to the mean wind speed and local site conditions. Furthermore, start and stop action and parking is simplified and extreme loads are reduced. Simple single-input - single-output (SISO) controllers of PI or PID type with moderate time constants are sufficient. An opposite pitch variation out of the rotor plane is used at the variable pitch (or pitch controlled) concepts to reduce the angle of attack and accordingly both lift and drag forces. During the early times of the industry, when power electronics were not present or too costly, the principle was combined with almost constant rotor speed. Later it was realised that pure pitch control with pitch rates up to 10 °/s and higher was not fast enough to counter the turbulence induced fatigue and power variations. Therefore turbines of intermediate size are using a successor of the classical pitch principle. A limited variability of the rotor speed is facilitated by increased slip of the asynchronous generator. Optimum performance by high technological effort is achieved by the variable-speed, variable-pitch concept that gains increasingly more contenders within the community in recent years.

Variable speed enables fast load control through the entire range between cut-in and cut-out wind speed as the rotor is acting as flywheel flattening the fluctuating power in the wind. In the partial load regime noise and energy yield are optimised as well. Pitch control with moderate pitch rates of about 2 to 5 °/s is commonly activated only above the rated wind speed to limit the power. High pitch rates are required only at emergency stops. Obviously, control of variable pitch - variable speed machines is complex and
interrelated with the dynamics of the entire system. At partial load a single-input - single-output controller of PID or higher order can be applied. Both speed and power serve inputs at rated power while pitch and a converter - generator quantity are controlled (multi-input - multi-output system, MIMO). Balancing the above mentioned control goals requires a combination of different controllers which are switched close to the rated wind speed [5.33] and the parameters of which are adjusted gradually to the operating conditions [5.1, 5.34].

Implementation of control

The DUWECs code used within this thesis was designed especially for analysis of controlled wind turbines. Analog or digital controller and filter modules can be easily implemented in the modular structure of the program. Figure 5.6 shows a model with a simple SISO controller of PI type as used later for the pitch controlled WTS 80 turbine. A first order response behaviour of the actuator was included in the controller as well. More advanced modelling of pitch systems is described in reference [5.35].

The simulation code DUWECs is able to derive a linear state space model of the entire OWEC (5.1) with respect to deviations from a point of equilibrium. This linearisation option facilitates application of classical linear control theory during the design of the controller. In addition the system eigenfrequencies and damping can be determined from the complex eigenvalues of the state matrix, A.

\[
\begin{align*}
\dot{x} &= A \cdot x + B_u \cdot u + B_v \cdot v \\
y &= C \cdot x + D_u \cdot u + D_v \cdot v
\end{align*}
\]  

(5.1)

where:

- \( x \) state vector
- \( u \) controlled input, e.g. pitch angle
- \( v \) external input, e.g. wind speed
- \( y \) output, e.g. rotor speed
- \( A \) state matrix
- \( B_u, B_v \) input matrices
- \( C \) output matrix
- \( D_u, D_v \) feed-through matrices

\( (x, u, v \text{ and } y) \) are defined with respect to their equilibrium values. \( D_v \) vanishes for most real systems.)
The state space model itself is derived by treating the wind turbine as a black box and
observing the variation of the outputs and the derivatives of the states on successive
perturbations of the state and the input variables.
Suppose the non-linear wind turbine model at an operating point \((\ddot{x}, \ddot{u}, \ddot{v})\) is given by
the vector functions \(\dot{x} = F(\ddot{x}, \ddot{u}, \ddot{v})\) and \(\ddot{y} = H(\ddot{x}, \ddot{u}, \ddot{v})\) then an element
\(A(i,j)\) of the state matrix \(A\) is determined by the finite central difference.

\[
A_{ij}(\ddot{x}, \ddot{u}, \ddot{v}) = \frac{F_i(\ddot{x}, \ddot{u}, \ddot{v})|_{x_j = \ddot{x}_j + \Delta x_j} - F_i(\ddot{x}, \ddot{u}, \ddot{v})|_{x_j = \ddot{x}_j - \Delta x_j}}{2\Delta x_j}
\] (5.2)

If an analog controller of the wind turbine is defined in state space form (5.3) is applied,
an extended state matrix, \(A_{\text{closed-loop}}\) according to (5.4) can be established.

\[
\begin{align*}
\dot{x}_c &= A_c x_c + B_c y + R_c r \\
u &= C_c x_c + D_c y + S_c r
\end{align*}
\] (5.3)

where:
\(x_c\) controller state vector
\(y\) controller input, i.e. turbine output
\(u\) controller output, i.e. turbine input
\(r\) reference signal
\(A_c, B_c, C_c, D_c, R_c, S_c\) state space matrices of the controller

\[
A_{\text{closed-loop}} = \begin{bmatrix}
A + B_u D_c & B_u (C_c + D_c D_u C_c) \\
B_c & A_c
\end{bmatrix}
\] (5.4)

In Section 7.1.3, Eqn. (5.4) is used to calculate the aerodynamic damping of the support
structure including the important dynamic effects of the control system.

5.2 Support structure

Tower and foundation should be considered as one support structure sub-system. This
is meaningful with respect to the modelling but also offers a potentially cheaper design
solution. In most aspects the treatment of the support structure is comparable to
common offshore platforms from the oil and gas industry. The main differences lie in the
relatively small size, the lower water depth and the more dominant aerodynamic loading.
For further details not covered in this section one is referred to textbooks [5.36, 5.37].

5.2.1 General aspects of structural modelling

Choice of Finite Element representation

The diversity of generic types of support structures, e.g. monotorower, braced or lattice
tower configurations either piled or gravity (Figure 3.16), requires equally diverse
modelling features that can be achieved only by Finite Element analysis. For our
purposes beam element models are sufficient.

Within this thesis and in contrast to both the previous work of the author [5.38] and other
recent OWEC design tools, the Finite Element representation is not directly implemented
in the design tool. Instead the general purpose code ANSYS™ 5.3 [5.39] with a consistent formulation of mass and stiffness matrices is used as a post-processor for the generation of the support structure model. This choice, further justified in Section 6.1.2, enables the treatment of support structures of 'arbitrary' complexity rather than only monopile or monotorre structures. In addition, specific aspects are available as submerged and hydrodynamically loaded elements, stress stiffening, non-linear structural characteristics, various dynamic analyses capabilities and techniques for the reduction of degrees of freedom
Prior to the separate consideration of piled and gravity structures some general remarks on mass and damping aspects relevant for both generic types are given.

Mass aspects common to all support structure types

A structure moving in a calm sea or in a sea state experiences a hydrodynamic inertia force which is proportional to the structural acceleration. This force can be considered as a part of the fluid loading but it is more conveniently computed by superimposing the so-called 'water added mass' on the structural mass (Section 5.2.5). The interior of offshore structures below the sea surface is often filled up with water to counter the buoyancy force and the external hydrostatic pressure. In a dynamic analysis this mass has to be added to the structural mass analogous to the hydrodynamic added mass.
The mass of the marine growth of space frame structures can be considerable and should be taken into account.

Damping aspects common to all support structure types

Four sources of damping are important for an OWEC, which are given in the order of decreasing relevance:
- aerodynamic damping
- structural damping
- soil damping
- hydrodynamic damping

The aerodynamic damping is a system rather than sub-system property. It is related to the interaction between the support structure and the wind turbine and is discussed with the modelling of other system properties in Section 5.3. Internal material friction and bolted or welded joints generate structural damping which is conveniently considered as modal damping. Typical values recommended for dynamic response analysis of wind loaded structures are given in Table 5.2. In this thesis generally 0.5% of material damping is considered for steel structures. Combined structural and soil damping of wind turbine towers in the range of 0.2 to 0.5% has been measured by Schaumann and Seidel [5.40].
The different types of soil damping are elaborated in conjunction with the gravity and piled foundations.
Finally the structural motion in the water leads to hydrodynamic damping. Radiation of vibration energy is very low, but damping due to the viscous drag force on flexible, submerged members as cables and risers is well known in offshore engineering. For our type of structures, which are relatively stiff below the water surface, the effect is, however, negligible (Sections 5.2.5 and 9.4).
**Table 5.2: Recommended structural damping ratios [5.36]**

<table>
<thead>
<tr>
<th>Structure</th>
<th>Modal damping ratio (1\textsuperscript{st} bending mode, as percentage of critical damping)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel lattice tower</td>
<td></td>
</tr>
<tr>
<td>- all welded</td>
<td>0.2 %</td>
</tr>
<tr>
<td>- welded bracings</td>
<td>0.2 - 1. %</td>
</tr>
<tr>
<td>- bolted bracings</td>
<td>0.3 - 1.2 %</td>
</tr>
<tr>
<td>Steel chimneys (unlined)</td>
<td></td>
</tr>
<tr>
<td>- welded</td>
<td>0.4 - 0.7 %</td>
</tr>
<tr>
<td>- bolted</td>
<td>0.6 - 1. %</td>
</tr>
<tr>
<td>Concrete chimneys</td>
<td></td>
</tr>
<tr>
<td>- without internal flues</td>
<td>0.5 - 1.2 %</td>
</tr>
<tr>
<td>- with internal flues</td>
<td>1.2 - 2.5 %</td>
</tr>
<tr>
<td>Steel tubes and pipes</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.5 - 1 %</td>
</tr>
</tbody>
</table>

**Table 5.3: Indicative values of apparent fixity length for piles of oil and gas jacket platforms [5.36]**

<table>
<thead>
<tr>
<th>Fixity length</th>
<th>Soil condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.5 $D$ - 4.5 $D$</td>
<td>stiff clays</td>
</tr>
<tr>
<td>7 $D$ - 8 $D$</td>
<td>very soft silts</td>
</tr>
<tr>
<td>6 $D$</td>
<td>general calculations</td>
</tr>
</tbody>
</table>

where: $D$ outer pile diameter

5.2.2 **Piled structures**

The load transfer mechanism and consequently the modelling of piled foundations differs between onshore and offshore wind energy converters. On land a cross-shaped foundation plate of several hundred tonnes of concrete is connected to 12 to 16 piles. So, the overturning moment and the base shear are transferred into both axial and lateral pile loads. The piles are designed mainly for axial capacity. For a dynamic analysis of the entire wind energy converter equivalent foundation springs for the horizontal and rotational motion of the foundation plate are introduced [5.41].

Piled foundation of offshore wind energy converters are either of the monopile or tripod type. In the former case the foundation dynamics are entirely governed by the lateral pile resistance while in the latter case also the axial resistance has a certain contribution. A model of the actual pile or piles and of the surrounding soil along the entire penetration depth is required. Attention has to be given to the properties of the different soil layers, the inherent uncertainties of the estimated soil parameters and scour degrading the foundation stiffness and strength.
Node with degrees of freedom

Figure 5.7: Modelling of monopile support structures
Left: Simplified model with apparent fixity length
Middle: Simplified model with foundation springs at mudline
Right: Detailed model of lateral pile behaviour by P-y curves

Figure 5.8: Deflection, soil resistance and bending moment of a pile with 3.5 m diameter in medium dense sand loaded by a thrust of 300 kN at the tower top
Models of distinct sophistication are appropriate for the different stages of the design process. Figure 5.7 illustrates such treatments which are demonstrated for the Opti-OWECS monopile design (Figure 9.1).

During the preliminary design only crude estimates are possible. Most convenient is the concept of the 'apparent fixity length', the length below the seabed under which the pile is assumed to be rigidly clamped in order to achieve the same fundamental eigenfrequency than the flexible supported pile (Figure 5.7 left). In the offshore technology rough approximations are often used (Table 5.3). For large piles with diameters of several metres probably lower values are more appropriate. For instance, fixity lengths of 3.3 and 3.7 diameters are calculated by analysing one converter of the first Dutch offshore wind farm Lely founded on sand and clay and the Opti-OWECS design solution for medium dense sand, respectively (Section 9.3.1).

As an alternative, independent foundation springs, \( k_{\text{rot}} \) and \( k_{\text{trans}} \), for the rotational and translational flexibility could be used (Figure 5.7 middle). However, in distinction to the gravity structures treated in the following section, it is difficult to find meaningful stiffness values without assistance by a more sophisticated model. Randolph [5.42] has derived analytical expressions for the foundation stiffness matrix, the deflection shape and the distribution of pile bending moment by Finite Element analyses of soils with stiffness proportional to the depth.

For both model types the material and soil damping and optionally also the aerodynamic damping is included in the generalized damping of the different modes, \( d_{\text{gen}} \).

The conceptual and certainly the final design stages require proper modelling of the subsoil behaviour (Figure 5.7 right). In accordance with common practice the following simplified assumptions are made. The forces occurring between the soil and the pile at any level are independent of the deflections at any other level. The effect of the mass of the soil acting with the pile has a negligible effect on the structural dynamics. 

The soil is represented by a finite number of lateral springs. The required grid spacing can be related to the natural deflection wave length of the beam on an elastic foundation [5.36]. For convenience one may select one metre between the springs for our purposes.

Pile group effects do not need consideration if the distance between the piles is greater than eight diameters. So, each pile of a tripod with three single piles can be modelled analogously to a monopile except that also the axial deflection should be taken into account.

The lateral load - deflection characteristics of the individual springs, denoted as P-y curves, are derived by semi-empirical rules from the soil parameters and as a function of the penetration depth. Although dedicated geotechnical codes are available for complex conditions, the establishment involves always a certain degree of arbitrariness and uncertainty. For more normal offshore structures in the petroleum industry the P-y curves according to the API Recommendations [5.43] are quite popular and are accepted as conservative estimates for the minimum bearing capacity. For an OWECS both the lower and the upper bound of the soil stiffness are relevant and the API Recommendations should be applied with care.

Figure 5.8 presents the lateral pile deflection, lateral soil resistance per unit length and the pile bending moment for the Opti-OWECS monopile in medium dense sand.
Figure 5.9: P-y curves for a pile with 3.5 m diameter and 25 m penetration into medium dense sand
(Markers show operating points for 300 kN thrust loading at tower top.)

Figure 5.10: Hysteresis loop of deflection at mudline for load cycles with different magnitude
The deflection of the pile remains relatively small. About 4 m below the sea bed the soil shows maximum resistance. The maximum bending moment associated with the loading by the rated aerodynamic thrust occurs 5.5 m below the soil surface. For pure hydrodynamic loading with a much smaller effective lever arm of the applied load the increase of the bending moment with depth along the first few metres is much steeper and the critical cross section is located at a larger depth of up to 7 m.

Common offshore structures suffer very significant cyclic wave loads during extreme conditions. Therefore API recommends different curves for static and cyclic loading conditions. For an OWEC the case is more difficult since the fluctuating aerodynamic and hydrodynamic loads are superimposed on the stationary mean wind loading. Some of the P-y curves applied at the example are plotted in Figure 5.9. A difference between the resistance for static or cyclic loading is observed only in the upper soil layers, where however the maximum bending moment occurs. Markers on the curves indicate the mean deflection associated with the rated thrust loading. Generally the deflections for large monopiles are relatively small, i.e. in the range of millimetres. The validity of the P-y curves according to API for such small strain magnitudes is more qualitative. Therefore the differences between static and cyclic loading and whether a pre-stress level is considered should not be overstressed. Other soil models can generate quite different curves in the low strain region. Instead one should consider such effects as part of a systematic variation of the structural uncertainties including also the wide spreading of the soil parameters (Section 5.3).

Under the condition that the load variation does not alter significantly the soil stiffness, the linearisation of the P-y curves for a certain pre-stress level is possible. This is required for both frequency domain analysis and time domain simulation using modal decomposition.

**Soil damping of piled structures**

Soil damping comprises two important parts: internal and radiation. Internal soil friction resulting in hysteresis or plastic damping is more important for piles. Nonetheless, it is often quite low for OWEC foundations with sufficient pile stiffness and penetration depth. Under these conditions the load-displacement behaviour under operational conditions is almost linear and therefore only small plastic deformations and associated damping occur. Figure 5.10 shows the hysteresis loop of the displacement at the mudline for the above mentioned monopile. The area enclosed by the loop corresponds to the dissipated energy. The damping is negligible for a typical operational load variation applied at a height of about 40% of the overall height. Only for extreme conditions, e.g. an extreme operational gust with emergency shut-down, significant damping occurs. Closed form solutions for the hysteretic damping of piles exist only for completely plastic and uniform soils [5.44]. Other conditions require a non-linear, static Finite Element analysis of a load cycle with load amplitude and resulting deflection amplitude at mudline, $\dot{y}$ (Figure 5.10). The energy dissipation, $W_{\text{diss}}$, during the load cycle is represented by the area enclosed by the load path. An equivalent viscous damping at mudline, $d_{eq}(\Omega)$, can be derived by equating the dissipated energy with the work of a massless, spring-dashpot system during a corresponding load cycle with circular frequency, $\Omega$, (5.5).

$$d_{eq}(\Omega) = \frac{W_{\text{diss}}}{\pi \Omega \dot{y}^2} \quad (5.5)$$
Such a viscous damper can be included in the equations of motion but complicates the modal decomposition. The damping is neither stiffness nor mass proportional and results in either complex but uncoupled equations of motion or a non-diagonalised generalised damping matrix and real, coupled equations. Here the latter is accepted together with the assumption that the excitation frequency, $\Omega$, equals the eigenfrequency, $2 \pi f$, of the associated modal degrees of freedom, $q_i$.

In Section 9.4 this exercise is performed for two offshore wind energy converters to quantify the damping of different eigenmodes. In practice the low magnitude of the hysteretic damping can be either neglected or it is considered directly as a small addition to the material damping.

### 5.2.3 Gravity structures

The modelling of gravity support structures is similar to those of onshore wind energy converters with a raft foundation. The caisson or foundation plate is considered as a rigid body supported by a linear elastic, homogenous and isotrope half-space characterised by the undrained density, $\rho_{\text{soil}}$, the dynamic shear modulus, $G_{\text{dyn}}$, and the Poisson ratio, $\nu$, (Figure 5.11 left). Finite Element models for the layered soil are generally applied only for large offshore structures, nuclear power plants, high speed railways or similar buildings.

The behaviour of the elastic half-space can be described by frequency dependent springs and viscous dampers associated with the rigid body degrees of freedom of the foundation (Figure 5.11 right). The springs are representing the elastic properties whilst the damper elements model the radiation of vibration energy away from the caisson. Material damping of the soil is generally neglected.

A variety of different and partially contradictory models are found in the literature to handle the frequency dependency. Three categories can be identified.

- Frequency independent spring or spring and damper models (Figure 5.12 left)
  Common agreement exists on the values of equivalent, frequency invariant foundation springs as function of the soil parameters and the radius, $R$, of the foundation plate, (5.6) [5.36, 5.45, 5.46]. For wind turbines on land normally only the tilt rotation and horizontal translation are considered. Such models are widely used because of their simplicity and the sensitivity study on the soil parameters which is carried out anyway. For our purposes this fits to the preliminary and conceptual design stages.

\[
\begin{align*}
  k_{\text{horz}} &= \frac{8 \ G_{\text{dyn}} \ R}{2 - \nu} \\
  k_{\text{vert}} &= \frac{4 \ G_{\text{dyn}} \ R}{1 - \nu} \\
  k_{\text{tilt}} &= \frac{8 \ G_{\text{dyn}} \ R^3}{3 \ (1 - \nu)}
\end{align*}
\]  

(5.6)

Frequency independent damping values are obtained by a dynamic analysis of the entire system at resonance in the associated degree of freedom, (5.7, 5.8).

\[
\begin{align*}
  d_{\text{horz}} &= \frac{4.6 \ R^2}{2 - \nu} \sqrt{\rho_{\text{soil}} \ G_{\text{dyn}}} \\
  d_{\text{vert}} &= \frac{3.4 \ R^2}{1 - \nu} \sqrt{\rho_{\text{soil}} \ G_{\text{dyn}}}
\end{align*}
\]  

(5.7)

Establishment of a frequency invariant parameter for the damping of the tilt rotation is problematical since in principle a parabolic dependency on the excitation frequency and no damping for the quasi-static case exist. Here we apply an approximative relation including the inertia of the entire structure with respect to the centre of gravity, $\Theta_{\text{c.o.g.}}$. 

\[
\begin{align*}
  d_{\text{tilt}} &= \frac{1.9 \ R^2}{2 - \nu} \sqrt{\rho_{\text{soil}} \ G_{\text{dyn}}} \\
  d_{\text{tilt}} &= \frac{3.4 \ R^2}{1 - \nu} \sqrt{\rho_{\text{soil}} \ G_{\text{dyn}}}
\end{align*}
\]  

(5.8)
Figure 5.11: Principle of modelling of gravity support structures

Figure 5.12: Actual modelling of gravity support structures
- Left: Frequency independent spring or spring and damper model
- Middle: Frequency dependent spring and damper model
- Right: Frequency independent mass, spring and damper elements with internal degrees of freedom, $x_{1,\text{trans}}$ and $x_{1,\text{rot}}$
\[ d_{\text{tilt}} = \frac{0.8 R^4 \sqrt{q_{\text{soil}} g_{\text{dyn}}}}{(1 - v) \left( 1 + \frac{3 (1 - v) \omega_{\text{tilt}}^{C,O,G}}{8 q_{\text{soil}} R^5} \right)} \]  

(5.8)

- Frequency dependent spring and damper models (Figure 5.12 middle)
  More adequate is the consideration of the frequency dependent properties. Values for the stiffness and damping coefficient can be obtained from diagrams or formulas [5.46, 5.47]. Such models are suitable for harmonic analysis of structures in machine dynamics, but they are difficult to use for transient analysis which is aimed for, here. In addition, mode superposition techniques can hardly be applied [5.48].

- Simple physical models with frequency independent mass, spring and damper elements (Figure 5.12 right)
  More suited for the design practice are simple physical models with frequency invariant mass, spring and dampers and with a few additional internal degrees of freedom [5.49]. This treatment approximates directly the transfer functions of the structure - soil system derived by more complex models and can be included conveniently in the Finite Element analysis of the structure. The approach is not further applied here because the non-proportional damping is not compatible with the standard mode superposition technique and the damping effects are not so significant to justify the use of complex eigenmodes to decouple the equations of motion.

In this thesis the frequency invariant spring and damper models according to the recommendations of the German Geotechnical Society [5.45, 5.46] are applied for the final design stage or detailed analyses. The relevant excitation frequencies are much lower than in other applications, e.g. foundation of fast spinning turbines or foundations with shock impact. In Section 9.4 the choice is justified by a comparison of different models for a gravity monopole.

5.2.4 Direct aerodynamic loading of the support structure

During power production the magnitude of the aerodynamic rotor loads exceed the direct wind loading of the support structure by an order of magnitude. Only beyond the cut-off wind speed, when the turbine is parked or idling, the direct wind loading becomes significant and has been considered in this thesis. The loading is applied as coherent gusts based upon drag coefficients and quasi-static gust response factors in accordance with DIN 4133 and DIBt Guidelines [5.50, 5.51]. Recently it became common practice to simulate directly the dynamic loading by turbulent wind which leads to less conservative results compared to the application of a gust response factor.²

5.2.5 Hydrodynamic loading

Classification of loading and analysis approach

Section 4.1.2 has discussed the first two steps of a hydrodynamic analysis dealing with the determination of the environmental conditions and of the water particle kinematics,

² In Section 4.1.1 it is mentioned that within this thesis the turbulent wind field is expressed in a rotating reference frame. This has advantages for the rotor part, but makes it impossible to derive directly the wind field affecting the fixed support structure.
i.e. surface elevation, water particle velocities and accelerations. Here we address the third step, the calculation of the hydrodynamic forces and response.

The fluid loading on structures may be classified as:

- *Drag loading* caused by vortices generated in the flow as it passes the structure. The associated forces are proportional to the square of the relative particle velocity and occur both in steady currents and wave fields.
- *Inertia loading* generated by the pressure gradient in an accelerating fluid moving around a member. The inertia force is proportional to the acceleration of the water particles owing to waves.
- *Diffraction loading*, a part of the inertia force on a structure which is so large that the wave kinematics are altered significantly by its presence.
- *Water added mass*, an inertia force experienced by a member accelerated relative to the fluid.
- *Slam and slap loading*. An inertia force proportional to the square of the relative velocity when a member passes though the water surface. Here it is mentioned as one approach for breaking wave loading (Section 4.1.2).
- *Vortex shedding induced oscillating loading* associated with the drag loading. Forces with the frequency of vortex separation are generated lateral to the flow direction and with the double frequency longitudinal to it. For our purposes with relatively large pile diameters it is without relevance.

The wave force approach to be applied is determined by two non-dimensional quantities. The non-dimensional Keulegan-Carpenter number, KC, increases with the ratio between the drag and inertia force, \( F_D / F_M \). The relative size of the structure, expressed by the diameter of the structural member, \( D \), divided by the wave length, \( L \), determines the degree of wave diffraction.

Figure 5.13 locates two common methods in relation to the Keulegan-Carpenter number and relative size of the structure. The diagram is limited on both axes by the slope of the deep water breaking wave curve, \( H/L = 1/7 \), where \( H \) refers to the wave height.

The Morison equation obtains the total force from an inertia and a drag component. Hydrodynamic drag becomes significant when the diameter of the structure is small compared to the wave length, \( D/L < 0.2 \) and the wave height, \( H \), is large with respect to the water depth (upper left corner of Figure 5.13). For deep water the Keulegan-Carpenter number is proportional to the ratio of wave height and structural diameter, (5.9), whilst for lower water depth it is driven by the amplitude of the water particle velocity.

\[
KC = \frac{\pi H}{D} \quad \text{for water depth } d > L/2 \tag{5.9}
\]

Taking the example of a monopile with 3 m diameter and waves up to 1 m height, typical for fatigue conditions, KC is equal or smaller than 1, corresponding to a drag force below five percent of the inertia force (Figure 5.14). The drag force should however not be underestimated for extreme conditions. High waves and near-breaking waves in shallow water are associated with high velocities in the wave crest and the non-linear drag force becomes of similar magnitude than the inertia loading, even for compact monopiles (Figure 5.15).
Figure 5.13: Ranges of application of wave force formulas [5.36]

If the structure is large in relation to the wave length, for example for gravity structures, the effects of diffraction and scattering have to be considered by the diffraction theory. It is based on a potential flow theory and requires in most cases a numerical solution of the Laplace equation and the boundary conditions [5.52].

For our purposes, dealing mainly with monopile and tripod configurations, the Morison equation is more relevant and diffraction can be accounted for by suitable corrections. Implementation in a design tool, as for instance the DUWECS code, is straightforward.

**Morison equation**

According to Morison et al. [5.53] the hydrodynamic force per unit length, \( dF \), for a fluid with density, \( \rho_w \), is represented by an inertia term, which is linear in the water particle accelerations, \( \dot{u}_w \), and quadratic in the structural diameter, \( D \), and a drag component, that is quadratic in the water particle velocities, \( u_w \), and linear in \( D \), (5.10).\(^3\) The expression is empirical owing to the drag and inertia coefficients, \( C_M \) and \( C_D \), which are determined from experiments.

\(^3\) In accordance to the notation in structural dynamics we denote the displacement, velocity and acceleration as \( u, \dot{u} \) and \( \ddot{u} \). In contrast, \( u \) and \( \dot{u} \) are commonly used for the velocity and acceleration in hydrodynamics.
$$dF = C_M \rho_w \frac{\pi D^2}{4} \ddot{u}_w + C_D \rho_w \frac{D}{2} |\dot{u}_w| \dot{u}_w$$  \hspace{1cm} (5.10)

The equation is valid for most bottom-mounted OWE C support structures loaded by higher waves. Exceptions are the caissons of gravity support structures and tower structures with a large diameter in small waves when diffraction becomes important. In both cases drag is negligible but diffraction has to be accounted for.

Initially the Morison equation was proposed only for the horizontal wave force on a vertical cylinder. It shown, however, also to give reasonable results for the more general case of inclined cylinders when applied to the flow and force components resolved normal to the member axis.

The drag force, proportional to the square of the water particle velocity, is non-linear even if the water particle velocity is sinusoidal according to the linear wave theory. This generates difficulties in linear frequency domain and probabilistic analyses. From a dynamic viewpoint the non-linearity is important since a part of the wave energy is transferred to the region of the structural eigenfrequencies. The amplitude of the third and fifth harmonics of the wave frequency account for 1/5 and 1/45 of the drag force.

For illustration the contribution of the drag and inertia loading to the total base shear was calculated for the monopile of the Utgrunden wind farm with 3 m diameter in 10 m water depth. In the fatigue regime with moderate but frequent waves (Figure 5.14) the inertia forces dominate and the behaviour is linear. In contrast, both the water kinematics and the loading becomes non-linear at extreme conditions. Figure 5.15 shows kinematics and forces associated with a wave of 6.4 m height. The wave crest of 4.95 m is more than three times higher than the trough which results in high particle velocities in the crest, resulting in a drag force of almost the same order of magnitude as the inertia force. The bending moment due to the wave impact (not shown) contains several higher harmonics with significant excitation energy.

**Structural motion, hydrodynamic added mass and damping**

The Morison equation can be applied also to a structure moving in a calm fluid or in a wave and current field with structural acceleration, \( \dddot{u} \), and velocity, \( \dot{u} \), when it is formulated in the relative water particle kinematics.

$$dF = C_M \rho_w \frac{\pi D^2}{4} \ddot{u}_w - (C_M - 1) \rho_w \frac{\pi D^2}{4} \dddot{u} + C_D \rho_w \frac{D}{2} |\dot{u}_w - \dot{\ddot{u}}| (\dot{u}_w - \dot{\dddot{u}})$$  \hspace{1cm} (5.11)

For numerical purposes it is convenient to split up the expression of the inertia force in terms of the water particle acceleration and structural acceleration, (5.11). Successively, the product of the so-called 'water added mass' and the acceleration of the cylinder can be written to the left hand side of the equations of motion. The dry structural mass of the submerged elements is increased by this portion which equals approximately the amount of gross displaced water since the value of \( C_M \) is around two. The additional mass is considered only up to the constant still water line in order to keep the mass matrix time invariant.
Figure 5.14: Kinematics and forces of an inertia dominated fatigue wave calculated with linear wave theory (Utgrunden monopile Ø 3 m, water depth d = 10 m)

Figure 5.15: Non-linear kinematics and forces of an extreme wave in shallow water calculated with 10th order stream function wave theory (Utgrunden monopile Ø 3 m, water depth d = 10 m)
Figure 5.16: Effect of wave diffraction on the inertia coefficient $C_M$ of a circular vertical cylinder

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Fatigue conditions</th>
<th>Extreme conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>inertia coefficient $C_M$</td>
<td>2.0</td>
<td>1.7</td>
</tr>
<tr>
<td>drag coefficient $C_D$</td>
<td>0.7</td>
<td>0.6 $^\dagger$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.7 $^\dagger$</td>
</tr>
</tbody>
</table>

$^\dagger$ smooth without marine growth, $^\ddagger$ rough with marine growth

The drag force on the moving structure generates hydrodynamic damping. This viscous force depends on the square of the relative particle velocity. In a time domain analysis this is handled by an iteration procedure which is started with the values of the structural velocity from the last time step. In the frequency domain the wave drag force and the hydrodynamic damping are linearised.

As mentioned-above, Section 9.4.2 proves this effect only theoretically relevant for bottom-mounted OWEC since the water depth is low and the structures are relatively stiff in the submerged part which results in negligible structural velocities in this region.

Selection of hydrodynamic coefficients and diffraction correction

The hydrodynamic coefficients $C_M$ and $C_D$ are obtained empirically. They depend mainly on the Reynolds number, the Keulegan-Carpenter number $KC$, the shape of the cross section and the specific roughness due to marine growth. Moreover the flow situation, e.g. oscillating fluid or structure in waves, has some influence. Establishment of appropriate values requires experience and/or laboratory testing.

In design practice often fixed values, not corresponding directly to these complex influences are chosen. They are valid only in a certain context as they are compensating
other, conservative assumptions. Table 5.4 lists the recommendations of the UK Department of Energy [5,37], which are followed here under the conditions that independent (extreme) values of wave and current are combined and that unidirectional (long-crested) waves, regular wave theories and no shielding effects are considered.

For hydrodynamically compact designs the disturbance of the wave field by the structure and the reflection of wave energy becomes important and requires specific computational tools based on solution of the potential flow problem. For a single column structure the Morison equation may still be applied in the diffraction regime by consideration of a modified, frequency dependent inertia coefficient and a phase shift of the wave kinematics according to MacCamy and Fuchs [5.54] (Figure 5.16).

A further documentation of the hydrodynamic models applied in this thesis can be found in reference [5.55].

5.2.6 Sea ice loading
Prediction of static or dynamic ice loads is difficult due to physical, statistical and model uncertainties. Beside the spreading of the environmental properties there is a variety of empirical approaches. Common ice pressure models include up to four empirical factors. The actual ice loading is caused by current and wind induced ice motion and the failure of the ice floe at the structure. At low failure frequencies of up to 1 Hz the ice sheets are broken by the bending mode and an ice cone can reduce the loading very effectively. For higher frequency of 0.5 to 10 Hz the ice is crushing and dynamic ice-structure interaction occurs.

Figure 5.17 illustrated four ice failure modes distinguished by increasing ice velocity. Examples for static or quasi-static load equations appropriate for mode I and II are provides by references [5.57 - 5.59]. At modes III and partly mode II a steady state interlocking of the ice failure frequency and the structural frequencies can occur and very significant dynamic amplification factors are the result if the structure is poorly damped. Some researchers propose a sinusoidal force history [5.59, 5.56] while other recommend a saw-tooth like function which includes higher frequencies as well [5.60].

![Mode I: almost steady ice](image1)
![Mode II: periodic transient response](image2)
![Mode III: steady vibration due to interlocking](image3)
![Mode IV: small response due to continuous ice crushing](image4)

**Figure 5.17:** Structural displacement \( u \) as response on different ice failure modes distinguished by increasing ice velocities [5.56]
5.3 Particular sub-system aspects that affect the entire system

For the understanding of the dynamic behaviour of an OWEC it is essential to mention some particular sub-system aspects that have a major effect on the overall dynamics.

*Time invariance of the rotor in an inertial reference frame*

The variation of the rotor stiffness and inertia in the inertial reference frame introduces some difficulties in the treatment of the dynamics. The effect, well known for wind energy converters, is of particular importance for (one and) two bladed rotors and to a lesser extent for flexible three-bladed rotors in general. Different established methods are available within the time domain approach. In the frequency domain the treatment of a linear model with periodic coefficients is possible as well but not yet used for design purposes [5.61]. For modal analysis, frequently a description a rotational invariant, isotopic rotor or a fictitiously fixed or ‘frozen’ azimuth position is applied. The latter is done within this thesis and all mentioned eigenfrequencies are so-called ‘snap-shot’ natural frequencies.

*Aerodynamic damping*

Any oscillating structure located in the air, for instance a chimney, experiences some amount of damping which is generally lower than the material damping. For wind turbines a different effect is of paramount importance, i.e. the aerodynamic damping induced by the operating rotor.

Figure 5.18 explains the effect for one oscillation cycle of the structure. A motion of the tower top in the wind direction (second figure) results in a smaller angle of attack, $\alpha$, at the rotor blades because the apparent out-of-plane velocity component, $V$ (1-a), is reduced by the tower top velocity, $\dot{u}_{\text{top}}$. A smaller angle of attack corresponds, for attached flow conditions, to lower aerodynamic lift and drag forces and to a reduction of the thrust force, $dF_x$, by an increment, $\Delta dF_x$. Likewise a movement of the nacelle against the wind direction increases both the angle of attack and the thrust force. In both situations the alternation of the thrust is oriented opposite to the disturbing tower top motion and is experienced as aerodynamic damping. Similar but less pronounced interaction occurs for the other tower top degrees of freedom. For higher angles of attack, stall occurs and the aerodynamic damping is lower or may even become negative because the slope of the lift curve is reduced. Under such conditions aero-elastic instability can occur if insufficient damping exists.

Equation (5.12) provides an instructive, analytical expression for the damping ratio as fraction of critical damping. The relation is based on stationary rotor aerodynamics and some simplifications valid for a wind turbine operating with a high tip speed ratio and near the rated wind speed. A similar form including the effect of the aerodynamic drag can be derived and is more appropriate for stall regulated wind turbines above the rated wind speed.

---

4 Garrad has described the background of the formula in reference [5.62].
\[ \xi_{\text{aero}} = \frac{N_b \cdot q \cdot \Omega}{8 \pi f_o M_o} \int_{R_{\text{root}}}^{R} \frac{d c(r)}{d \alpha} c(r) r \, dr \]  

where:
- \( \xi_{\text{aero}} \): aerodynamic damping ratio as fraction of critical damping
- \( N_b \): number of blades
- \( q \): air density
- \( \Omega \): rotational speed of the rotor [rad/s]
- \( f_o \): frequency of the first fore-aft mode [Hz]
- \( M_o \): modal mass of the single degree of freedom system
- \( R_{\text{root}}, R \): blade root rotor radius, outer rotor radius
- \( dc(r)/d\alpha \): slope of the lift coefficient with respect to angle of attack
- \( c(r) \): chord length at radius \( r \)
- \( r \): spanwise coordinate

The closed-form estimate discloses two main influences on the damping. Firstly the slope of the lift coefficient versus angle of attack can vary from approximately 2 \( \pi \) for attached flow to zero or even negative values in case of separated flow. An effect that cannot be covered by the simple model is the critical dependency of the aerodynamic damping in separated flow on non-linear unsteady stall hysteresis effects where it is not simply related to the slope of the lift coefficient. Secondly, the aerodynamic damping ratio depends on the eigenfrequency and modal mass of the support structure. It can change from less than 1\% of critical damping for a stiff and heavy tower to above 5\%, several times the structural damping, in case of a soft and light support structure.

Considerable aerodynamic damping takes place only during the operation of the rotor and is active on both aerodynamic and hydrodynamic excitation of the system. Apart from the fore-aft motion of the support structure motion also the flapwise, i.e. out-of-plane, motion of the flexible blades and to a lesser extent for the tilt and yaw movement of the nacelle experience aerodynamic damping.

Any dynamic OWEC model must account for aerodynamic damping in a reasonable way. Integrated, non-linear time domain simulation are most accurate (Chapter 6). During the early design stages simplified models are advantageous (Chapter 8) which substitute the aerodynamic damping by additional structural damping.

**Variation of system properties**

Most of the system properties discussed in the last three sections can only be estimated with a considerable uncertainty; moreover, they are to a certain extent time variant due to effects of operating conditions, loading, tide, marine growth, aging, settlement, etc. Therefore either design calculations have to account for these variations or should assume at least the most onerous combination(s). Tables 5.5 and 5.6 list conditions resulting in either maximum or minimum values of the eigenfrequencies and damping. A priori it is not always clear for which conditions the highest extreme response will occur. For fatigue loads which accumulate over a long time it might be appropriate to assume average conditions, e.g. mean sea level, moderate marine growth, low lateral pile loading, etc. So different structural models of the OWEC should be established for fatigue and extreme load calculations assuming the most onerous conditions.
Figure 5.18: Illustration of aerodynamic damping effect

- Fore-aft motion $\dot{u}_{\text{top}}$ at wind speed $V$ during production
- Motion $\dot{u}_{\text{top}}$ changes angle of attack $\alpha$ at a blade element
- Variation of lift coefficient $c_L$ and drag $c_D$ (not shown)
- Variation of thrust $\Delta F_x$ opposite to perturbation
### Table 5.5: Variation of system properties for calculation of eigenfrequencies

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Reduction of eigenfrequency</th>
<th>Increase of eigenfrequency</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>mass effects</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>still water level (SWL)</td>
<td>HAT + surge + settlement</td>
<td>LAT - surge</td>
</tr>
<tr>
<td>hydrodyn. added mass</td>
<td>max $C_m$ associated with max SWL</td>
<td>min $C_m$ associated with min SWL</td>
</tr>
<tr>
<td>internal fluid</td>
<td>max</td>
<td>none</td>
</tr>
<tr>
<td>marine growth</td>
<td>max assuming max density</td>
<td>assuming min density</td>
</tr>
<tr>
<td>soil mass</td>
<td>max</td>
<td>min</td>
</tr>
<tr>
<td>structural mass</td>
<td>max</td>
<td>horizontal</td>
</tr>
<tr>
<td>rotor position (2-bladed)†</td>
<td>vertical</td>
<td>min or none †</td>
</tr>
<tr>
<td>wind turbine mass / inertia</td>
<td>yes</td>
<td></td>
</tr>
</tbody>
</table>

### stiffness effects

<table>
<thead>
<tr>
<th></th>
<th>Reduction of eigenfrequency</th>
<th>Increase of eigenfrequency</th>
</tr>
</thead>
<tbody>
<tr>
<td>soil stiffness</td>
<td>min, P-y curves for static loading</td>
<td>max, P-y curves for cyclic loading</td>
</tr>
<tr>
<td>mean lateral pile loading</td>
<td>max</td>
<td>none</td>
</tr>
<tr>
<td>scour</td>
<td>max</td>
<td>none</td>
</tr>
</tbody>
</table>

† resulting in minimum or maximum eigenfrequency of the fore-aft bending modes
‡ during installation and major overhaul condition

### Table 5.6: Variation of system properties and operational conditions for calculation of damping

<table>
<thead>
<tr>
<th>Damping type</th>
<th>Minimum damping</th>
<th>Maximum damping</th>
</tr>
</thead>
<tbody>
<tr>
<td>aerodynamic damping</td>
<td>parked or idle rotor or operating rotor with separated flow, lower bound of $\Omega$, maximum bending eigenfrequency</td>
<td>operating rotor with attached flow, upper bound of $\Omega$, minimum bending eigenfrequency</td>
</tr>
<tr>
<td>radiation damping of gravity foundation</td>
<td>lower bound of soil shear strength and density, low excitation frequency</td>
<td>upper bound of soil shear strength and density, high excitation frequency</td>
</tr>
<tr>
<td>hysteretic damping of pile foundation</td>
<td>upper bound of static soil stiffness, no mean lateral pile loading</td>
<td>lower bound of cyclic soil stiffness, upper bound of mean lateral pile loading</td>
</tr>
<tr>
<td>structural damping</td>
<td>lower bound</td>
<td>upper bound (e.g. cracked concrete)</td>
</tr>
<tr>
<td>hydrodynamic damping (if relevant)</td>
<td>none or related to min $C_{Dp}$, min still water level, low waves</td>
<td>related to max $C_{Dp}$, max still water level, high waves</td>
</tr>
</tbody>
</table>
CHAPTER 6

INTEGRATED, MODULAR, NON-LINEAR TIME DOMAIN SIMULATION

Non-linear time domain simulation of the overall dynamics of an entire offshore wind energy converter is a very powerful and potentially very accurate analysis method. This chapter describes both the general principles and the particular implementation in this thesis, denoted here as integrated, modular, non-linear time domain simulation.

In order to achieve its full potential, however, sufficient experience and high computational effort in terms of dedicated mathematical models and computational time is required. Therefore in engineering practise the approach is best suited for in-depth investigations and certification calculations at the end of the design process when detailed design data are available and a good understanding of the principles underlying the design has already been gained.

During parameter studies and in the early stage of the design process a simplistic application with an inadequate structural model or random simulations which are not representative for the actual stochastic loading can be dangerous. The accuracy might then be lower than that of simplified and faster approaches as introduced in Chapter 8.

Figure 6.1: Integrated dynamic model including short-term dynamic interactions within offshore wind energy converter (OWEC) and between OWEC and both natural environment and grid
The time domain simulation approach will be elaborated with respect to a number of general aspects of structural dynamics and practical requirements. Section 6.1 explains the importance of an integrated model of an entire offshore wind energy converter and how it is implemented in a modular manner in this research. So, the convenient combination of distinct, but dedicated structural models in the wind turbine and support structure part is possible. The decision between time domain and frequency domain approach is discussed in Section 6.2.
Section 6.3 investigates techniques for the reduction of degrees of freedom and of load cases with long duration of the simulations, while Section 6.4 provides an overview on the implementation in the DUWECS (Delft University Wind Energy Converter Simulation) code.

6.1. Integrated but modular dynamic model

6.1.1 Integrated model

*Physical background*

A simple model of an offshore wind energy converter considers a system with two independent, random vectorial inputs for the aerodynamic and hydrodynamic excitation.

Figure 6.1 illustrates a more detailed block model showing the simultaneous, short-term interactions between the aerodynamic, hydrodynamic and geotechnical environment, the offshore wind energy converter with its major components and the consumption of electrical energy.

\[
\begin{bmatrix}
M_{WT}^{SS} + M_h^{SS} \\
D_{D}^{SS} + D_{a}^{SS} \\
K_{K}^{SS} + K_h^{SS}
\end{bmatrix}
\begin{bmatrix}
\ddot{x} \\
\dot{x} \\
x
\end{bmatrix}
= \begin{bmatrix}
\dot{f}_{a}^{SS} + f_{h}^{SS}\end{bmatrix}
\]

*Figure 6.2: Structure of the equations of motion of an integrated OWEC model*

- \(M^{WT}\): mass matrix
- \(D^{SS}\): damping matrix
- \(K^{SS}\): stiffness matrix
- \(x\): degrees of freedom
- \(f\): load vector

Occupied portions of the matrices are highlighted. The wind turbine related portions of the structural mass, damping and stiffness matrices are generally time variant and depend upon the rotor speed, azimuth angle and deformation of both rotor and structure. The aerodynamic damping and stiffness matrices depend also on the wind speed. The hydrodynamic mass matrix is constant.
Although the random aerodynamic and hydrodynamic loading can be regarded as independent under fatigue conditions it is not straightforward to calculate the aerodynamic and hydrodynamic response separately. We will discuss this further in Section 7.1. Considering the constitution of the linearised equations of motion in Figure 6.2, we find aerodynamic damping and stiffness terms and a hydrodynamic inertia term in the system matrices. So, the responses to any of the two independent load inputs will depend on both aerodynamic and hydrodynamic conditions.

Design practice

For some years, integrated dynamic models in the time domain are the only accepted approach for design calculations of onshore wind energy converters. Section 5.1 explained how such models integrate aerodynamics, structural dynamics of rotor, drive train and tower as well as the dynamics of the generator and control system. From the discussion above it is consequent that offshore wind energy converters are modelled in an integral way by considering the wind turbine, with all above-mentioned features, and the support structure, their simultaneous response under aerodynamic, hydrodynamic and electrical loading and their mutual interactions (Figure 6.1). This is denoted as an integrated model within this thesis.

However, is an integrated model really required and does it fit the needs of an iterative design process?

A separate treatment of the main sub-systems, the wind turbine and the support structure, would be convenient with respect to the complexity of the design problem and the expertise distributed on the two disciplines of wind energy and offshore technology. There is no single answer to this question for all applications. From a scientific point of view the integrated model is the obvious choice and it should be used for any investigation of OWEC dynamics.

During the design process, resources are scarce and must be used efficiently. Evidence exists that the wind turbine part is not significantly affected by wave induced response. The support structure, however, is suffering simultaneous wind and wave response and incorporation of the aero-elastic interactions, namely the aerodynamic damping, is essential [6.1, 6.2].

For analysis limited to the wind turbine part of the OWEC, application of an onshore wind turbine design tool might be sufficient if the dynamics of the tower and foundation are properly represented. Furthermore, it has to be ensured that such a reduced model is consistent with the wind turbine representation within the integrated OWEC model used for the support structure analysis.

During the early design stages or if fatigue is not governing the design, the support structure fatigue might be analysed with one of the simplified methods based upon separated wind and wave responses (Section 8.1 and 8.2). The integrated approach is clearly required for certification calculations of structures dominated by combined wind and wave fatigue.

---

1 Section 9.3.2 explains why the wind turbine rotor is insensitive to the wave excitation. The extent of interactions and the conditions under which separation of the simultaneous response is possible for simplified analyses is investigated in Section 7.1.
### Table 6.1: Comparison of wind turbine and support structure with respect to modelling requirements

<table>
<thead>
<tr>
<th>Aspect</th>
<th>Wind turbine (horizontal axis)</th>
<th>Support structure (bottom-mounted)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Geometrical configuration</td>
<td>(flexible) multi-body system with open chains, <em>limited</em> set of generic types (e.g. 2- or 3-bladed, fixed or flexible yaw, etc.)</td>
<td>fixed structure of cantilever or truss type, <em>various</em> configurations especially for braced and lattice type</td>
</tr>
<tr>
<td>Structural kinematics</td>
<td>flexible deflections superimposed on gross motion (e.g. azimuthal rotation, teeter, tower top)</td>
<td>(small) flexible deflection about a mean position</td>
</tr>
<tr>
<td>Stress-strain material behaviour</td>
<td>linear</td>
<td>linear, partly non-linear for piled foundations and/or extreme conditions</td>
</tr>
<tr>
<td>Material damping</td>
<td>proportional</td>
<td>proportional</td>
</tr>
<tr>
<td>Other damping</td>
<td>aerodynamic damping</td>
<td>aerodynamic damping, equivalent viscous soil damping</td>
</tr>
<tr>
<td>Structure-flow interaction</td>
<td>important</td>
<td>less important</td>
</tr>
<tr>
<td>Equations of motion</td>
<td>non-linear, time-variant</td>
<td>linear, time-invariant</td>
</tr>
<tr>
<td>Typical number of degrees of freedom</td>
<td>6 - 30 (modal plus discrete) †</td>
<td>20 - 600 (physical), 5 - 30 (modal) †</td>
</tr>
<tr>
<td>Dynamic loading</td>
<td>aerodynamics, gravity, manoeuvre</td>
<td>wind turbine, gravity, hydrodynamics, ice</td>
</tr>
<tr>
<td>origin</td>
<td>mainly non-linear</td>
<td>linear†† or non-linear††</td>
</tr>
<tr>
<td>nature</td>
<td>- fatigue conditions - random</td>
<td>- random</td>
</tr>
<tr>
<td>- extreme conditions</td>
<td>- deterministic</td>
<td>- deterministic or random</td>
</tr>
</tbody>
</table>

† valid for modal representation of the wind turbine (Section 5.1),
†† valid for Finite Element representation of the support structure (Section 5.2),
† † fatigue conditions and/or inertia dominated structures,
† † † extreme conditions and/or slender, drag dominated member
In any case an integrated OWEC model is an important tool that can be used in three ways:

- Investigation of the dynamics of the entire OWEC and of the support structure response to simultaneous aerodynamic and hydrodynamic loading during the final design stages when simplified methods are not applicable.
- Studying the wind turbine response to pure wind excitation. For convenience all hydrodynamic features are switched off and the integrated model behaves similarly to an onshore design tool.
- Investigation of the hydrodynamic response of the support structure for an extreme load analysis and parameter studies. Now the model acts as an offshore technology design tool.

If properly applied, an integrated model does not only provide a more accurate description, it also enables less conservative and thus most probably more cost-efficient design solutions. This might explains the growing commercial interest in the approach.

**Overview of integrated design tools**

Today, several integrated OWEC design tools in the time domain exist. The early model of Oscar and Paez based upon the general purpose Finite Element code NASTRAN™ [6.3] and the extension of an offshore technology design tool by Kortlever [6.4] are no longer in use. All other programs are extensions of onshore wind turbine design tools. Continuation of the work of Wastling and Quarton [6.1] led to the first commercial OWEC design tool Bladed for Windows [6.5]. Approximately at the same time the DUWECs code was further developed within the work summarised in this thesis and was applied during different research and industrial projects at Delft University of Technology. Quite recently, other commercial onshore wind turbine design tools as PHATAS, FLEX5 and HawC have been extended [6.6 - 6.8], but only illustrative results have been published so far.

Non-linear, periodic and unsteady effects in the wind turbine part (Section 6.2.1) and the interaction between wind turbine and support structure complicate the derivation of frequency domain models. Such tools have been proposed, e.g. [6.1, 6.6, 6.9], but these linear models are not fully integrated because spectral superposition of separate wind and wave responses is assumed. Their application is limited to research or the early stage of the design process.

**6.1.2 Modular model**

*Distinct modelling requirements*

The wind turbine and support structure sub-systems are subject to quite different modelling requirements (Table 6.1). For the wind turbine, only very few geometrical configurations are relevant which all result in geometrically non-linear, time-variant equations of motion owing to the gross rotation and flexible deformation of the rotor. The support structure properties are just opposite. Figure 3.16 shows a variety of structural configurations, e.g. monopile, tripod, lattice tower. Nonetheless, all equations of motion are linear and time-invariant. Likewise, the non-linearity of the external forces is less pronounced for the majority of the hydrodynamic load cases than for the aerodynamic conditions.
There are two principal approaches to integrate such different requirements in one entire model:

- consistent formulation of the equations of motion for the entire system

  Typically this could be done by a sophisticated Finite Element representation capable of gross motion superimposed on small deflections or by a multi-body system code with automated generation of the equations of motion. This option offers a more consistent description, but requires extensive computational power. A number of recently extended onshore wind turbine tools provide a compromise in this respect. They are always dealing with the same generic set of equations of motion by consideration of only one support structure type, namely the simple monopile/monotower configuration. Sometimes other structures can be modelled by importing externally generated mass and stiffness matrices.

- modular formulation of the equations of motion for the wind turbine and support structure

  Obviously a multi-body or modal approach with specific treatment of the aero-elastics fits the wind turbine part whilst a linear Finite Element model is ideally suited for the support structure. For both aspects established models exist in the form of onshore wind turbine design tools and multi-purpose structural codes. The approach is more elegant and efficient, but requires a solution of the interface problem between the two sub-systems and associated models. The greater computational efficiency during the actual analyses is of particular value for time domain Monte-Carlo simulations.

Within this research we followed the second option and extended the (onshore) wind turbine design tool DUWEC (Section 6.4) to offshore wind energy application. Among other reasons this code was chosen due to its suitability for the modular approach.

The equations of motion of the used rotor module [6.10] are derived with the so-called ‘method of generalised momenta’ after van Holten [6.11]. This avoids a cumbersome second time derivative known from Lagrange equations, the principle of virtual displacements or most other approaches. Previous work by the author adopted the method of generalized momenta to the modular approach within DUWEC [6.12].

Coupling of equations of motion of wind turbine and support structure

Several established methods exist for the treatment of the crucial interface between the rotating and non-rotating structure [6.13 - 6.15]. The DUWEC code applies a different approach, the stepwise integration of the equations of motion of the separated modules, which is possible only in the time domain.²

The complete system is divided into a number of modules under the assumption that the interaction between the modules takes place by prescribed variables (Figure 5.6). Typically such interaction variables are either kinetic variables, e.g. generalised forces and momenta, or kinematic quantities, e.g. generalised coordinates and velocities (Figure 6.3). The equations of motions are established separately for every module taking into account global inputs and outputs, e.g. wind speed or grid voltage, and the

² This treatment has been applied also in other technical fields, e.g. the contact dynamics of the current collector - overhead line system in railway engineering [6.16]
interaction variables from the other modules. This feature provides a large degree of freedom to the code developer as long as the module interfaces are compatible. In particular it enables use of the above-mentioned combination of a non-linear multi-body system approach for the rotor and of linear Finite Element methods for the support structure.

Instead of assembling the coupled set of equations of motion of the entire system, the equations of the modules are integrated separately and subsequently for every module. Depending of the module sequence, it is required to approximate the instantaneous value of some interaction variable by the value of the previous integration time step. For instance, the rotor behaviour is calculated based upon the tower top velocities from the last time step while the tower motion is considering the tower top forces from the current time step, which were calculated just before in the rotor module.

**Figure 6.4:** Error of aerodynamic damping of first fore-aft mode by using only certain tower top degrees of freedom at the example of a stall regulated wind turbine (x: in wind direction, y: lateral to the left, z: upwards, see Figure 0.2)
Experience shows that for the typical length of the integration time step between 5 and 25 ms no iteration of the interaction variables is required to simulate the overall behaviour in a relevant frequency range of up to approximately 5 Hz.

The incorporation of the generalised tower top forces in the dynamics of the support structure is straightforward; however, consideration of the tower top motion involves a lot of bookkeeping in the derivation of the equations of motion in the rotor module. Therefore several wind turbine design tools currently applied by the industry suffer shortcomings in modelling the tower top motion.

The rotor module of the DUWECS code generates the entire set of six generalised tower top forces but, depending on the used rotor module, either only the kinematics of the fore-aft translation or only the fore-aft and lateral translation are considered in the dynamics (Figure 6.3).

In a simplified manner we attempt to judge the error in the aerodynamic damping of the first fore-aft bending mode which results from the incomplete modelling of the rotor-tower interaction. From Kaiser [6.17] and Peinel [6.18] we take the aerodynamic damping matrix for the entire set of six degrees of freedom at the tower top of a stall controlled 270 kW wind turbine. Such a damping matrix can easily be incorporated into a general purpose Finite Element model of the wind turbine - support structure system. By switching on and off different degrees of freedom we can now observe the consequences for the damping of the fore-aft mode.

For this particular case, the error is conservative and in an acceptable range between 6% and 15% if only the tower top fore-aft translation, $u_x$, or both fore-aft and lateral translation, $u_x$ and $u_y$, are considered (Figure 6.4). Activating the tilt degree of freedom, $rot_y$, and especially of both the rolling and tilt degrees of freedom, $rot_x$ and $rot_y$, further reduces the error with respect to the complete modelling. The simplified modelling will likely cause larger errors in the description of the aerodynamic damping of the second fore-aft mode which is however less important for the hydrodynamic response.

6.2. Time domain versus frequency domain approach

6.2.1 Non-linearities
The three different media, air, water and soil, all behave non-linearly with respect to the loading and response of the offshore wind energy converter. The strongest non-linearity occurs in the rotor aerodynamics due to flow separation at higher wind speeds. The stall effect and the associated variation in the slope of the lift coefficient discard linearisation of the aerodynamics (Figure 5.5). Kinematic non-linearities of the hydrodynamics are significant for high waves and in shallow waters. The finite wave height and the asymmetry of crest and trough are incompatible with linear wave theory and lead to higher harmonics in the water particle velocities and accelerations. For our purposes the non-linearity of the hydrodynamic drag is of lesser importance, since most OWEC support structures are dominated by the hydrodynamic inertia force. Under such conditions the offshore experience with the linearisation of the drag force is applicable and linear spectral analysis is possible. The drag becomes, however,
significant for extreme, (near-)breaking waves as OWEC structures are located in small water depths. The hydrodynamic non-linearities can result in excitation of structural frequencies by higher order harmonics of the wave frequency, the so-called ringing effect [6.19].

In principle, the pile-soil interaction under static or cyclic loading is non-linear and the soil behaviour of a gravity foundation depends on the excitation frequency. In Section 9.4.3 such effects are judged to be less significant.

As a reminder to the discussion of Section 6.1.2, the geometrical non-linearities caused by the gross overall motion of the rotor and the considerable elastic deflections of the slender blades are mentioned again.

The long term fatigue behaviour of the structure establishes another strong non-linearity, which is however independent of the structural model. The fatigue deterioration is a function of the third or higher power of the stress range. In contrast with the material of the structure itself can be treated according to classical theory of linear elasticity.

To summarize, significant non-linearities and transient effects are present, especially in the wind turbine part, which explains the preference of the wind energy community for a non-linear analysis approach for this sub-system. Linear treatment of the support structure is reasonable for most fatigue load cases, but is limited for extreme conditions in shallow waters.

6.2.2 Fatigue analysis in time domain or frequency domain

Different methods are established for each of the four steps included in a fatigue analysis, i.e. stochastic environmental modelling, structural response calculation, establishment of stress range distribution and damage accumulation. There are only two basic approaches commonly used (Figure 6.5):

- time domain approach, i.e. generation of random time series from wind and wave spectra, non-linear, unsteady system simulation by time step integration techniques and Rainflow counting of stress ranges

- frequency domain approach, i.e. direct application of the environmental spectra, linear spectral analysis and determination of stress range distribution from the spectral moments

The applied damage accumulation hypothesis is largely independent from time or frequency domain approach. Both wind energy and offshore technology commonly use the well-known linear damage accumulation rule after Palmgren-Miner together with standard S-N curves. More sophisticated approaches based upon fracture mechanics are sometimes applied in offshore engineering [6.20].

Indicated by dashed lines, Figure 6.5 shows some alternative paths, which are uncommon due to the required effort or the lack of certain information.

**Time domain approach**

The rationale of the time domain approach is to simulate the physical behaviour of the system by modelling the state of every component in great detail. The physical variables of the entire trajectory from the environmental impact to the stress range cycles are followed for numerous subsequent time steps until sufficient statistical convergence is achieved.
**Figure 6.5**: Fatigue analysis by either time domain approach (thick arrows) or frequency domain approach (double arrows)

(Dashed lines indicate alternative paths between time and frequency domain. pdf: probability density function, RFC: Rainflow Counting)

The approach is very popular in wind energy engineering because it is widely proven by measurements and capable of treating non-linear and unsteady systems. Even non-Gaussian input can be considered, which is however not normally done. Fatigue analyses in the time domain are observed in offshore technology only for systems with strongly non-linear behaviour, e.g. floating platforms and riser systems.
We are using a 4th order Runge-Kutta explicit integration schemes with constant time step. As a consequence the highest eigenfrequency dictates the length of the integration time step leading often to a much finer temporal discretisation than required with respect to the frequency content of the applied input signals.³

Linear spectral analysis

In contrast with the above, the frequency domain approach deals directly with statistics of loading and response. Transfer functions can be established in different ways by analytical or numerical linearisation, by evaluation of the response of a 'black-box' model to test signals or by processing the response on normalised excitations with different frequencies. The statistics of the stress spectra are transformed by algebraic relations into a probability density function of the stress ranges. The entire process is between 10 and 100 times faster than Monte-Carlo simulations.

The frequency domain approach is the general method for assessment of the fatigue of offshore structures and much experience exists with its application [6.21]. Application in the field of wind turbine engineering is not new [6.22] but encounters a number of problems including time variance of the equations of motion, non-linearities, unsteady phenomena and periodic effects. Furthermore the bandwidth of the aerodynamic response is so wide that most frequency domain methods generate far too conservative results for the fatigue damage. Only the Dirlik method is suited in this respect, which is further discussed in Section 7.2.2.

There is another practical argument against the frequency domain approach. The price/performance ratio of current computer hardware is so low that fast and accurate design calculations in the time domain are viable for onshore wind energy converters.

Preferred approach

For accurate analysis of the entire OWEC or the wind turbine part the time domain approach is currently regarded as the only suitable choice. However, the frequency domain approach is certainly appropriate for the support structure under pure hydrodynamic loading.

In Chapters 7 and 8, simplified analysis methods for the early design stage are developed that combine the advantages of both approaches by the combination of time domain analysis of the aerodynamic response and frequency domain analysis of the hydrodynamic response of the support structure.

6.3. Model reduction

6.3.1 Reduction of degrees of freedom

Most time domain wind turbine design tools apply a relatively low number of degrees of freedom only, typically around 20. Only with this number the complexity of the non-linear equations of motion and the computational time remains reasonably limited. Computation on typical PC-based platforms results in a duration between several hours to approximately one day for one entire design calculation of an onshore wind energy

³ For numerical stability the integration time step of the Runge-Kutta scheme must be smaller than 2.8 divided by the maximum eigenfrequency of the system. For accurate results a much smaller time step is required.
converter comprising between 12 and 25 fatigue load cases and 20 to 200 extreme load cases.\textsuperscript{4} Offshore design calculations involve more complex structural and loading models and a larger number of fatigue load cases. Commonly long random realisations per load case, of for instance 3 hours, are applied in offshore engineering if time domain analyses are carried out. In contrast, in the wind energy community only a few number of simulations of 10 minutes each are used in practice.\textsuperscript{5} Therefore it is evident that the computational effort for the analysis of offshore wind energy converters has to be limited.

In Section 6.1.2 the need for a Finite Element model of the support structure was explained. The resulting number of physical degrees of freedom, typically between 20 and 600 if beam elements are used, is too high. However, various methods exist for the reduction of degrees of freedom of structures with stiffness or mass proportional damping, linear kinematics and linear constitutive relations. The most important for our purposes are the modal reduction or mode superposition and the static or Guyan reduction.

Prior to the application of any of the two methods, the complexity of the model should be chosen appropriate to the support structure type, the features of the simulation code and the analysis type. Relatively few beam elements generally provide a quite reasonable model and node spacing should reflect changes in the structural properties, distribution of external forces and hot spot locations. Higher degrees of freedom can generally be locked. They are not critical but result in unduly high eigenfrequencies. Consideration of the torsional behaviour is meaningful only if the aero-elastics and the structural dynamics of the yaw dynamics are modelled in a sophisticated manner.

\textit{Mode superposition}

The mode superposition is based on the modal decomposition of the system which transforms the set of \( n \) coupled differential equations of the \( n \) physical degrees of freedom, \( \mathbf{x} \), into a set of \( n \) uncoupled equations for the same number of generalized or modal coordinates, \( \mathbf{q} \). The transposed modal matrix, \( \mathbf{\Phi}^T \), with the eigenmodes, \( \varphi_j \), \( j = 1 \ldots m \), constitutes the transformation from the physical to the modal coordinates, \( (6.1) \). Consideration of only \( m \) modes, with \( m \) smaller than \( n \), generates a reduced set of generalized coordinates

\[
\begin{bmatrix}
q_1 \\
\vdots \\
q_m
\end{bmatrix}
= \begin{bmatrix}
\varphi_{1,1} & \cdots & \varphi_{1,n} \\
\vdots & \ddots & \vdots \\
\varphi_{m,1} & \cdots & \varphi_{m,n}
\end{bmatrix}
\begin{bmatrix}
x_1 \\
\vdots \\
x_m
\end{bmatrix}
\]

\[ m < n \quad (6.1) \]

\textsuperscript{4} Different combinations of mean wind speed, mean wind direction, initial rotor position and system properties generate a high number of extreme load cases each with relatively short duration.

\textsuperscript{5} Taking the Opti-OWECS monopile configuration and one severe sea state as example, Hofland [6.23] investigated the influence of simulation length and number of averaged simulations on different response properties. Quite similar convergence of the damage equivalent loads was found between separate simulations of the wave and wind response. For waves the 95\% confidence intervals accounted for \( \pm 10\% \) and \( \pm 4\% \) when one time series of 10 min and 3 time series of 20 min each were used, respectively. Slightly lower values of \( \pm 7\% \) and \( \pm 3\% \) were observed for the wind case. Further research is required to establish more general conclusions on the required simulation technique.
and modal equations of motion with diagonalized matrices for the generalised mass, \( m_{gen} \), damping, \( d_{gen} \), and stiffness, \( k_{gen} \) and the generalized forces, \( \Phi' \cdot f \), (6.2).

\[
\begin{bmatrix}
\vdots & \vdots & \vdots & \vdots & \vdots \\
\vdots & \ddots & \ddots & \ddots & \ddots \\
\vdots & \ddots & \ddots & \ddots & \ddots \\
\vdots & \ddots & \ddots & \ddots & \ddots \\
\end{bmatrix}
\begin{bmatrix}
\varepsilon \\
\varepsilon \\
\varepsilon \\
\varepsilon \\
\end{bmatrix}
+ \begin{bmatrix}
\vdots & \vdots & \vdots & \vdots & \vdots \\
\vdots & \ddots & \ddots & \ddots & \ddots \\
\vdots & \ddots & \ddots & \ddots & \ddots \\
\vdots & \ddots & \ddots & \ddots & \ddots \\
\end{bmatrix}
\begin{bmatrix}
\varepsilon \\
\varepsilon \\
\varepsilon \\
\varepsilon \\
\end{bmatrix}
+ \begin{bmatrix}
\vdots & \vdots & \vdots & \vdots & \vdots \\
\vdots & \ddots & \ddots & \ddots & \ddots \\
\vdots & \ddots & \ddots & \ddots & \ddots \\
\vdots & \ddots & \ddots & \ddots & \ddots \\
\end{bmatrix}
\begin{bmatrix}
\varepsilon \\
\varepsilon \\
\varepsilon \\
\varepsilon \\
\end{bmatrix}
= \begin{bmatrix}
\varphi_{11} & \cdots & \varphi_{n,1} \\
\vdots & \ddots & \vdots \\
\varphi_{1m} & \cdots & \varphi_{n,m} \\
\end{bmatrix}
\begin{bmatrix}
f \\
\end{bmatrix}
\]

\( i=1..m, \ j=1..n \)

This treatment is possible if proportional damping is assumed which is a reasonable assumption for the material damping of the structure. In most cases one decomposes the conservative system (without damping) and considers the structural damping as modal damping by prescribing the generalized damping.

The models for soil damping of piled or gravity foundation, described in Sections 5.2.2 and 5.2.3, result in non-proportional damping that is applied at the mudline. So, the physical damping matrix is empty except of the few diagonal elements associated with the concerned degrees of freedom at mudline. Modal decomposition of such a system results in complex eigenmodes [6.24]. We are interested in the low-frequency dynamics only on which the soil damping has a rather small influence (Section 9.4.3). Therefore we avoid such a complication and use the real eigenmodes of the conservative system for the modal decomposition. As a consequence the few elements of the physical damping matrix are spread over the entire generalized damping matrix which is still real but no longer diagonalized. Nonetheless, still a simple and efficient numerical implementation is possible.

In a certain variant, the so-called 'mode superposition with static improvement', the dynamics of only a low number of modal degrees of freedom, \( q_{dy} \), with eigenfrequencies in the range of the excitation are considered, (6.3). The inertia and damping forces from the remaining higher modes are neglected but the static deflection of these degrees of freedom, \( q_{stat} \), improves the overall solution.

\[
\begin{bmatrix}
\vdots & \vdots & \vdots & \vdots & \vdots \\
\vdots & \ddots & \ddots & \ddots & \ddots \\
\vdots & \ddots & \ddots & \ddots & \ddots \\
\vdots & \ddots & \ddots & \ddots & \ddots \\
\end{bmatrix}
\begin{bmatrix}
\varepsilon \\
\varepsilon \\
\varepsilon \\
\varepsilon \\
\end{bmatrix}
+ \begin{bmatrix}
\vdots & \vdots & \vdots & \vdots & \vdots \\
\vdots & \ddots & \ddots & \ddots & \ddots \\
\vdots & \ddots & \ddots & \ddots & \ddots \\
\vdots & \ddots & \ddots & \ddots & \ddots \\
\end{bmatrix}
\begin{bmatrix}
\varepsilon \\
\varepsilon \\
\varepsilon \\
\varepsilon \\
\end{bmatrix}
+ \begin{bmatrix}
\vdots & \vdots & \vdots & \vdots & \vdots \\
\vdots & \ddots & \ddots & \ddots & \ddots \\
\vdots & \ddots & \ddots & \ddots & \ddots \\
\vdots & \ddots & \ddots & \ddots & \ddots \\
\end{bmatrix}
\begin{bmatrix}
\varepsilon \\
\varepsilon \\
\varepsilon \\
\varepsilon \\
\end{bmatrix}
= \begin{bmatrix}
F_{dy}^T \\
F_{sta}^T \\
\end{bmatrix}
\begin{bmatrix}
f \\
\end{bmatrix}
\]

Three criteria govern the selection of modes. Firstly, the range of the associated eigenfrequencies should cover the frequency content of the input signal. Too high eigenfrequencies would require an uneconomically small integration time step because of the stability requirement of the explicit integration scheme with a constant time step. Secondly, only such modes can be excited effectively whose shapes show significant deflection at the application point and in the direction of the loading. Finally, if the static response and the internal member forces are important, a relatively high number of modes might be worthwhile.

Sometimes, difficulties occur owing to the mode superposition method during the calculation of member forces in complex structures, e.g. of the lattice type. The required superposition of the 3rd and 4th order derivatives of the displacement field converges only poorly or not at all, even for a very high number of modes. Therefore the common choice in the offshore industry for the analysis of jacket platforms is the direct solution in the physical coordinates rather than mode superposition [6.25].
Static reduction

For the static reduction after Guyan [6.26] the equations of motion are partitioned according to a reduced number of master degrees of freedom, $x_m$, and the remaining slave degrees of freedom, $x_S$. (6.4)

$$\begin{bmatrix} M_{MM} & M_{MS} \\ M_{SM} & M_{SS} \end{bmatrix} \begin{bmatrix} \ddot{x}_M \\ \ddot{x}_S \end{bmatrix} + \begin{bmatrix} D_{MM} & D_{MS} \\ D_{SM} & D_{SS} \end{bmatrix} \begin{bmatrix} \dot{x}_M \\ \dot{x}_S \end{bmatrix} + \begin{bmatrix} K_{MM} & K_{MS} \\ K_{SM} & K_{SS} \end{bmatrix} \begin{bmatrix} x_M \\ x_S \end{bmatrix} = \begin{bmatrix} f_M \\ f_S \end{bmatrix}$$  (6.4)

The identity relation and the static equilibrium between master and slaves establish the transformation matrix, $T$, (6.5).

$$\begin{bmatrix} x_M \\ x_S \end{bmatrix} = T \begin{bmatrix} x_M \end{bmatrix} \quad \text{where:} \quad T = \begin{bmatrix} I \\ -K_{SS}^{-1}K_{SM} \end{bmatrix}$$  (6.5)

Multiplication with the transposed transformation matrix from the left and the transformation matrix from the right obtained the reduced equations of motion which are not uncoupled but have considerably smaller the size.

$$M^{red} \ddot{x}_M + D^{red} \dot{x}_M + K^{red} x_M = T^T f$$  (6.6)

For illustration we write out the reduced mass matrix, (6.7).

$$M^{red} = M_{MM} - M_{MS}K_{SS}^{-1}K_{SM} - K_{MS}K_{SS}^{-1}M_{SS} - M_{SS}K_{SS}^{-1}K_{SM}$$  (6.7)

Opposite to the modal reduction, this approach is exact for static loading but approximative for the dynamic case. Master degrees of freedom should be selected in such a way that the shapes of important eigenmodes can be reproduced reasonably and that master degrees of freedom correspond to the location and direction of the loading. General purpose Finite Element codes provide also an opportunity for the automatic master selection based upon the associated mass - stiffness ratio. Good results are achieved by a combined approach of manual selection of the majority of master degrees of freedom by the analyst and automatic selection of the remaining portion of, say, 10 - 20%. 6

Implementation in the DUWECS code

The support structure module within the DUWECS simulation tool always applies mode superposition. However, several variants are possible according to the particular needs:

- mode superposition without modal reduction
  This unusual choice uses all eigenmodes that are extracted by the preceding Finite Element analysis. It is suited only for very simple models without high eigenfrequencies or after a static reduction.

- modal reduction
  Eigenmodes are chosen according to the three criteria mentioned above. Different numbers of modes should be tested in order to check the desired accuracy.

---

6 For the present case of coupling a linear elastic model of the support structure to a non-linear model of the rotor, a mixed static and modal reduction of the degrees of freedom [6.27] would be an ideal approach. It was not applied here due to the cumbersome implementation.
• mode superposition with static improvement
This recommended option integrates several advantages. The dynamics are well represented by a low number of dynamic degrees of freedom. A coarse time integration step is applicable because high eigenfrequencies are not relevant. The static solution and especially the member forces are improved by the contribution of a larger number of static modal degrees of freedom for which the inertia and damping forces are neglected.

• static reduction prior to any of the mode superposition variants
The support structure model itself is derived with aid of the general purpose Finite Element code ANSYS 5.3™ which enables a static reduction of the model prior to the DUWECS input. Thus, a static reduction can be combined with any of the above-mentioned methods.

Calculation of member forces by mode superposition
The suitability of the mode superposition technique was investigated for two typical support structures limiting the entire range of possible solutions, i.e. a tubular monotower and a fully braced lattice tower (Figure 9.4). In the case of the monotower, classical mode superposition was found sufficiently accurate by selection of the ten lowest bending modes per direction (Figure 6.6). The associated eigenfrequencies up to 70 Hz require, however, an uneconomically small time integration step. So one has to accept a lower accuracy by using only the first five modes up to 10 Hz if classical mode superposition is desired. As above-mentioned, mode superposition with static improvement is of great advantage since only the first three to five modes are needed for the descriptions of the dynamic response whilst the

![Diagram](image)

**Gravity monotower:**
excitation due to harmonic thrust force at $0.9 f_0$, 1.5% damping

**number of considered fore-aft modes as dynamic degrees of freedom [-]**

**Figure 6.6:** Convergence of member forces at a tubular support structure calculated by mode superposition compared to mode superposition with static improvement
rest are static degrees of freedom without negative consequences for the time integration step.
Proper modelling of lattice type structures required more effort. A static reduction with about 50 - 100 master degrees of freedom is recommended. Subsequent mode superposition with static improvement showed similar results as for the monotower; however, a considerably higher number of modes was required to achieve a reasonable accuracy for the member forces.

6.3.2 Lumping of load cases for wind and wave fatigue
For an offshore wind energy converter at least an order of magnitude more fatigue load cases exist than for an onshore wind energy converter. Consideration of directional effects, typically done by 12 sections, increases the number of load cases by another order of magnitude. Evidently this is incompatible with integrated time domain simulations and establishment of a low number of characteristic load cases is required. A so-called 'lumping of load cases' is well known in the offshore technology for pure hydrodynamic loading.\(^7\) We have to deal with significant wind and wave fatigue as well as the dependency of the response behaviour on the aerodynamic damping for different wind speeds and operational conditions.
In Chapter 8 a simplified analysis approach for support structure fatigue owing to wind and wave loading is introduced. This approach enables a very fast calculation of hundreds of load cases which provides the basis for the lumping of sea states. The environmental parameters of a lumped load case are selected in such a manner that they will result in approximately the same damage that accumulates considering all elementary load cases with spread environmental parameters.

![Graph showing scatter of sea states](image)

**Figure 6.7:** Scatter of 58,333 sea states at the NL-1 site during 9 years. (Boxes include such sea states that are considered in the 21 lumping load cases from Table 8.3. The damage centres indicate the parameters of the lumped load cases.)

---

\(^7\) In structural dynamics, 'lumping' has a quite different meaning and denotes the fictitious concentration of a property, for instance the structural mass, in the nodes as an approximation of the continuous distribution in reality.
Each dot in Figure 6.7 represents one sea states occurring during nine years according to the NESS database and was grouped into a three-dimensional scatter diagram with a minimum probability of 0.5 \text{%}. From the 235 elementary load cases of the scatter diagram 21 lumping load cases were constructed which parameters or ‘damage centres’ are indicated by the markers.

The details of the lumping approach are explained in Section 8.3 where the approach is developed in the context of the other simplified methods.

**Pre-Processing**

- Design Data
- Offshore Wind Energy Converter
- Environmental Parameters
- ANSYS (Finite Element Code)
- Support Structure and Soil Model
- Rotor, Transmission, Generator and Controller Model
- SWING (Stochastic Wind Field Generator)

**Simulation**

- DUWECs (Integrated, Non-Linear, Time-Domain Simulation of Aero-Elastic, Structural and Hydrodynamic Behaviour)
- Loads, Displacements, Member Forces

**Post-Processing**

- FAROB (Strength and Fatigue Analysis)
- Extreme Stresses, Fatigue Damage

**Figure 6.8:** Implementation of simulation approach in the time domain

### 6.4. Implementation

In this thesis a particular implementation of the integrated, modular, non-linear time domain approach has been developed and implemented by major extension of the basis design tool DUWECs (Delft University Wind Energy Conversion Simulation) which originally included only a single degree of freedom model of onshore towers [6.28]. Details of the modelling are discussed in Chapter 5.

Figure 6.8 explains the general analysis process. The total simulation process is divided into three parts with respect to the chronological order and the software used. The basic
idea of this approach is to minimize the computational effort in the actual simulation by treating as many as possible tasks prior or after the main process.

- **Pre-Processing** (model generation and reduction, wind generation)
  The general purpose Finite Element code ANSYS 5.3™ is used for the generation and reduction of the model of the support structure including the non-linear foundation. A modal analysis with or without pre-stress effects establishes the model. Geometric data, the eigenmodes and eigenfrequencies are transferred in an object orientated format [6.29] by an interface program [6.30] which also generates modal unit load vectors and modal unit member forces. The spatial and temporal structure of the wind field is calculated by the SWING code (Stochastic WINd Generator) [6.31]. Furthermore, input data for other modules have to be prepared by the user. Various utilities are available for generation of input files and processing of batch calculations.

- **Simulation** (response calculation)
  Time histories of displacements, loads and member forces are computed with the simulation code. Aerodynamics is based upon standard blade element momentum theory. Wave kinematics of regular or random waves according to non-linear Stream function wave theory or linear wave theory with Wheeler correction are determined. The hydrodynamic forces on structural members with arbitrary orientation are calculated with the Morison equation optionally corrected for wave diffraction. In addition, DUWECs is capable of linearizing the system at the equilibrium state. The calculated state-space model can be used for frequency domain analysis, controller design, calculation of the coupled eigenfrequencies and determination of the aerodynamic or soil damping. Hydrodynamic transfer functions can be generated by steady state simulations of the response to elementary waves.

- **Post-Processing** (response analysis and interpretation)
  Signal analysis and graphics are performed by another software package, e.g. MATLAB™. The fatigue analysis with Rainflow-Counting and linear Palmgren-Miner rule is done in the FAROB code which has been developed at the Stevin Laboratory of the Delft University of Technology [6.32]. Again utilities are provided for batch processing.
CHAPTER 7

BASES FOR SIMPLIFIED FATIGUE ANALYSIS APPROACHES

Application of the integrated, modular, non-linear time domain simulation approach, described in the previous chapter, can potentially lead to more cost-effective and reliable design solutions for offshore wind energy converters. The high computational effort associated with the fatigue analysis of the simultaneous aerodynamic and hydrodynamic response of the support structure in the time domain is, however, not compatible with the iterative nature of the early stages in the design process. In such situations it would be convenient to separate wind and wave response since well-established and fast approaches exist in both wind energy and offshore technology to deal with either wind or wave response.

Indeed it is possible to establish such simplified (but not simplistic) approaches. This chapter demonstrates how the results of separate analyses of wind and wave induced fatigue can be combined in a convenient way which accounts for the two important effects of partial cancellation of wind and wave responses and aerodynamic damping of the wave response. Figure 7.1 compares a simplistic superposition neglecting both aspects to one of the new simplified approaches for the equivalent bending moment at the mudline of the Opti-OWECS monopile.

Before the two novel methods are demonstrated in a cookbook manner and on an

![Diagram](image)

**Figure 7.1:** Simplistic superposition of separate wind and wave fatigue responses versus simplified analysis approach
empirical basis for the long-term fatigue loads in Chapter 8, we have to establish a more theoretical basis here. Therefore, this chapter deals only with the short-term statistics of stationary processes.

The problem is split up into two major aspects: Separation of the simultaneous response in Section 7.1 and superposition of separated responses in Section 7.2. Evaluation of the interactions within an offshore wind energy converter (OWEC) in Section 7.1.1 reveals the aerodynamic damping of the fore-aft motion as essential for our purpose. This insight enables the separation of the simultaneous response into an aerodynamic and hydrodynamic contribution (Section 7.1.2). Different estimates of the aerodynamic damping are evaluated in Section 7.1.3 while the substitution by additional structural damping is demonstrated in Section 7.1.4.

Next, the superposition of fatigue loads is elaborated. After comparing different superposition strategies in Section 7.2.1, the chosen combination of damage equivalent fatigue loads is derived and validated in Sections 7.2.2 and 7.2.3, respectively.

The reference case for the numerical investigations in this chapter is the design solution of a 3 MW, two-bladed, pitch regulated offshore wind energy converter with constant rotor speed and a soft-soft monopile developed during the Opti-OWECs study (Figure 9.4, Section 9.1). In contrast to Chapter 9, here only a simplified model of the support structure (Figure 7.2) is taken from reference [7.1]. The lowest five fore-aft bending modes are used. Wave loads rest on the linear wave theory in its original form. Hydrodynamic drag is not significant under fatigue conditions and is neglected in the frequency domain analyses. The wind turbine model is identical to that from Chapter 9.

Further data can be found in Appendix B.

In this chapter we restrict us to collinear and omnidirectional waves which results for the reference design in conservative fatigue loads compared to situations with a misalignment between wind and wave direction (Section 9.6.1) and a directionality of wind and wave loading. So, the influence of the wave excitation on the lateral response is not significant, pure aerodynamic response dominates in the side direction and the computational effort is limited by considering only the fore-aft response.

Moreover we restrict the following derivation to fatigue caused by production load cases. Parked and transitional situations, where either wave or wind loads dominate, are discussed in Chapters 8 and 9.

7.1 Separation of simultaneous response under wind and wave loading

7.1.1 Independence of wind and waves and dynamic interaction of responses

The underlying idea for the separation of the OWEC response is that wind and wind generated wave loading can be regarded as independent and stationary on a short time scale, i.e. within a period of ten minutes to three hours. Figure 7.3 plots two time series as an example. The process of wave generation by the wind is driven only by the mean wind speed rather than by atmospheric turbulence.  

---

1 The wave phase velocity (celerity) is slower than the mean wind speed, so wave crests and troughs are not synchronised with atmospheric eddy structures. Local influences of the instantaneous water surface elevation on the wind field are relevant only in the splash zone but not at the swept rotor area, located at least 10 - 30 m above the sea level.
Non-linear constitutive relations and geometric non-linearities are not significant for fatigue conditions. Consequently time series and spectra of the quasi-static responses of independent, random aerodynamic and hydrodynamic loading processes can be superimposed from separate analyses.

As the aerodynamic forces depend on the structure’s kinematics, superposition of the dynamic responses must account for the mutual interaction and superposition is not straightforward.

Figure 7.4 shows the associated bending moments at mudline as an example for the interaction of the dynamic responses. Superposition of the separate responses calculated without aerodynamic damping of the fore-aft motion would significantly overestimate the combined response.

**Figure 7.2:** Opti-OWECS monopile support structure (left) and simplified model used within this chapter (right)

The various dynamic interactions within the OWEC and between the converter and both environment and grid are illustrated in Figure 6.1. Not considered are stationary effects with respect to the duration of the random loading process, for instance, wind generated...
waves or the influence of the roughness of the sea surface on the atmospheric turbulence and wind shear.

For the present discussion the following four aspects are relevant:
- interactions between the dynamics of the OWEC components,
- aero-elastics, especially aerodynamic damping,
- interaction between the hydrodynamic forces and structural response,
- soil-structure interaction.

*Interactions between the dynamics of the OWEC components*

The dynamic behaviour of rotor, conversion system including the drive train, support structure and controller is strongly interrelated with the aerodynamic loading. Since a couple of years it is common practice to take into account the structural dynamics of the entire system during the *calculation of the aerodynamic response*. As an additional requirement, a time domain simulation approach is preferable with respect to the non-linear rotor aerodynamics.

*Aero-elastic interactions, especially aerodynamic damping*

Several aero-elastic effects can be observed at wind turbine rotors and their supporting structures. Common to all is the feed-back loop between the dynamic behaviour of the structure, including elastic and inertia forces, the aerodynamic loads and the active control system (if any). The most important issue for this investigation is the aerodynamic damping of the fore-aft motion of the support structure motion experienced during power production of the wind turbine which was already discussed in Section 5.3.

*Interaction of the hydrodynamic forces and structural response*

Significant interaction between hydrodynamic forces and structural kinematics occurs only for slender and flexible members, such as cables, moorings and risers. The alternation of the hydrodynamic drag force owing to the moving structure is experienced as hydrodynamic damping. For bottom-mounted OWEC support structures the structural velocities of the submerged part are very small compared to the water particle velocities and the viscous and radiation damping is much smaller than the structural damping (Section 9.4.2). So, *wave loading*, i.e. the external forces, can be calculated separately from the structural response and the wind loading. From the previous discussion of the aerodynamic damping, it is obvious that *wave response*, i.e. the internal member forces due to wave loading, will be significantly affected by the response of the entire OWEC.

*Soil-structure interaction*

The magnitude of the stress ranges in the soil due to either aerodynamic or hydrodynamic fatigue loading is small compared to the ultimate strength. Hence soil behaviour can be regarded as linear or can be linearized with respect to the stationary aerodynamic and gravitational pre-loading. Separate analyses of the wind and wave responses with the same soil model are therefore suitable.
Figure 7.3: Example time histories showing wind and wave loading to be uncorrelated on short time scale

Figure 7.4: Example time series of bending moment at mudline for separate wind and wave responses (upper) and combined responses with and without aerodynamic damping (lower)
7.1.2 Contributions of wind and waves to the simultaneous response
Separation of the contributions of wind and waves to the simultaneous OWEC response is far from simple since the aerodynamic loading is affected by the hydrodynamic response and vice versa.
Here a practical solution is proposed for the development of a simplified analysis approach. It is shown that the aerodynamic contribution is well approximated by simulation of the response under wind loading in a calm sea. Substitution of the aerodynamic damping by an addition to the modal structural damping of the fundamental fore-aft mode (Section 7.1.4) is a reasonable approach determining the hydrodynamic contribution.

Approximation of the wind contribution
Even in the case of significant hydrodynamic excitation the magnitude of the tower top velocity is relatively small compared to the variation of the longitudinal wind velocity. Likewise the difference of the thrust loading between the simultaneous wind and wave response and the fictitious case of pure wind loading in a calm sea is small in relation to the magnitude of the thrust itself. The observation is illustrated by time series of two load cases in Figures 7.5 and 7.6.

Figure 7.5 compares the response for simultaneous wind and wave loading and pure wind loading in a calm sea at the partial load conditions from Figure 7.3. The upper plot shows the thrust loadings at the hub for pure wind loading and simultaneous wind and wave loading. Both signals are very similar, so, the their difference is given by the time series in the lower diagram. In addition, the difference of the associated fore-aft velocity of the tower top is included. Assuming linear behaviour the difference of the thrust forces can be regarded as the aerodynamic damping force related to the wave contribution of the simultaneous response. Apparently, this force is negative and proportional to the difference of the tower top velocities.

The employment of the pitch controller at full load is demonstrated at a higher mean wind speed of 21 m/s and a corresponding severe sea state (Figure 7.6). Above the rated wind speed the analog PI-controller regulates the blade pitch angle to minimize generator speed variation (Figure 5.6, Table B.7). The asynchronous generator with rated 3% slip is directly connected to the grid which ensures a stiff speed-torque characteristics. The maximum pitch rate accounts for 5 °/s.
The upper two subplots and present the wind and wave loading and responses of the pitch controller and the aerodynamic thrust for simultaneous wind and wave excitation. The two lower subplots and show the difference of the time series associated with the simultaneous response and the pure aerodynamic response. The relation between tower top velocity and thrust force variation in the third subplot is similar to the above-mentioned partial load situation. The pitch controller is, however, sensitive to the wave excitation. Waves induce a tower top response which affects the aerodynamic forces and thus also rotor torque and rotor speed. The deviation of the rotor angular velocity triggers the controller. Comparison of the difference of the pitch angle variation and the difference of the tower top velocity in fourth subplot reveals an undesired effect of this rotor speed controller. Instead of showing synchronous behaviour compared to the wave induced tower top velocity, the pitch difference has a phase delay of approximately 90°, which results in a reduced aerodynamic damping.
Figure 7.5: Responses at partial load without pitch control:
Aerodynamic thrust of pure wind and simultaneous wind and wave response (upper),
Thrust difference opposite to difference of tower top velocity (lower)

Figure 7.6: Responses with pitch control at full load: Phase shift of differential
pitch response with respect to differential tower top velocity in plot (d)
For 17 production load cases similar to Table 8.3 the thrust loading is compared
between pure wind response and simultaneous wind and wave response. The difference
of the aerodynamic forces was found large enough to employ effectively the
aerodynamic damping influence on the wave induced response, but it was too small to
significantly affect the wind induced fatigue loads. The aerodynamic thrust of the pure
aerodynamic response underestimated the thrust associated with the simultaneous wind
and wave response by 1.5% ±0.4% in terms of damage equivalent fatigue loads with
respect to an S-N curve with inverse slope of four.

In summary, the wind contribution to the combined response can be approximated by
the wind response in a calm sea, taking water added mass as the only hydrodynamic
effect. This corresponds to the application of an onshore wind turbine design tool where
extra mass at the tower base is considered.

**Approximation of the wave contribution**

The establishment of the wave contribution to the combined response is more involved
due to the aerodynamic damping. Three possible approaches are identified.
One may define the wave contribution as the remaining share after subtracting the wind
contribution from the simultaneous response [7.1]. This leads to difficulties because the
wind contribution has only been approximated and the combined response is available
only from a time-consuming analysis with the integrated OWEC model.
Secondly, the wave contribution is simulated by the wave response of the OWEC
operating in a steady wind field. Despite the good approximation (Section 7.1.4) and
previous work of the author this approach is not further recommended owing to the
requirement of an integrated dynamic model of the OWEC.

Following an engineering approach, aerodynamic damping is substituted by increasing
the modal damping ratio of the structure’s fore-aft mode. No integrated model is needed
and any design tool for offshore platforms should be suitable to calculate the wave
response. The difficulty lies, however, in the consideration of the parameters and
operational conditions that influence the aerodynamic damping. Nonetheless this
approach is preferred and investigated in the next two sub-sections.

### 7.1.3 Estimation of the aerodynamic damping

Determination of the aerodynamic damping for certain values of the mean wind speed
(or non-dimensional tip speed ratio) and pitch angle is far from new. Kaiser, for instance,
provides a comprehensive overview [7.2]. In the design practice such investigations are
rare since common time domain design tools are not capable of a linearisation of the
wind turbine model. Therefore we analyse the suitability of three methods for the
approximation of the aerodynamic damping of the fore-aft translation in a separate
analysis of the wave response.

**Closed-form linearisation**

The simple, closed-form model of the linearized aero-elastic behaviour from Eqn. (5.12)
enables physical insight and the identification of important parameters. According to
Prandtl the slope of the lift coefficient equals approximately 2π for attached flow and an
upper bound of the damping can be established easily.
Information on the stationary flow conditions along the rotor blades is required for more
realistic conditions with partially separated flow. Any standard wind turbine code or even
Table 7.1: Example of stationary flow conditions used as input for the closed-form estimate of aerodynamic damping (5.12)

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<th>( \alpha )</th>
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A spreadsheet based model of the elementary blade element momentum theory can be applied for this purpose. For illustration, we evaluate a condition just below the rated wind speed with the DUWECs simulation code (Table 7.1). The angle of attack (fourth column) at the inner four blade sections is large enough that partial flow separation and reduction of the slope of the lift coefficient (sixth column) occurs. Eqn. (5.12) estimates 2.9% of the critical damping. In comparison 5% are obtained for fully attached flow at wind speeds below 9 m/s.

**Numerical linearisation**

A numerical linearisation is capable of more comprehensive rotor aerodynamics, unsteady effects, a larger number of degrees of freedom and the dynamics of the control system. In general such an option is not implemented in standard wind turbine design tools used for dynamic load calculations. So, one has to use specific aero-elastic codes [7.2] or extended design tools.

In Section 5.1.5 it was explained how the simulation code DUWECs is able to derive a linear state space model of the entire OWEC (5.1). The aerodynamic damping can be calculated conveniently from the complex eigenvalues of the state matrix of the uncontrolled system, \( A \) (5.2), or from the state matrix, \( A_{\text{closed-loop}} \) (5.4), if the controller dynamics are included.

**Non-linear time domain simulation**

A more cumbersome, but easier available approach applies non-linear time domain simulations. Different methods are available, e.g. evaluation of the steady state response under harmonic excitation, analysis of the transient decay of free vibrations...
after a step or pulse loading or response spectra analysis. In principle such treatments should be possible with any standard wind turbine design tool and some manual post-processing.

Here transient vibrations were analysed to study the influence of the vibration amplitude and inflow. Two alternatives are discussed in more detail for a mean wind speed of 12 m/s. In the first case a thrust pre-loading of 10% of the rated loading was released after 30 seconds and free vibrations in steady inflow were observed (Figure 7.7). The free vibrations were also excited in the presence of a turbulent wind field (Figure 7.8). The deterministic vibrations and the stochastic response are of the same order of magnitude. The contribution of the free vibrations is clear but the damping cannot be derived unless the stochastic part of the response is filtered out. Assuming the validity of linear superposition of both contributions, the tower top displacement from a stochastic simulation with the same random seed but without pre-loading is subtracted from the combined response. The remaining signal in the fourth plot is regarded as the deterministic part of the free vibrations in a turbulent wind field.

The logarithmic decrement of the total structural and aerodynamic damping, \( \Lambda \), is easily read from the semi-logarithmic plot of the sequence of the \( n \) successive maxima \( a_r \) to \( a_n \) (Figure 7.9).

\[
\Lambda = \frac{1}{n-1} \ln \frac{a_1}{a_n}
\]

\[
\xi = \frac{\Lambda}{\sqrt{4\pi^2 + \Lambda^2}} \quad \xi \ll 1 \quad \frac{\Lambda}{2\pi}
\]

The damping ratio, \( \xi \), was not affected significantly by the vibration amplitude. Both below and above the rated wind speed slightly lower damping was observed for small vibration amplitudes in a turbulent wind field.

**Comparison of different approaches**

The three methods, closed-form expression, numerical linearisation and non-linear simulations, were compared for the reference design. In the transient approach six turbulence representations for each of the 20 wind speed classes were averaged. Figure 7.10 shows estimates of the aerodynamic damping of the first fore-aft mode against mean wind speed. Only the stationary but not the dynamic behaviour of the controller is dealt with. The aerodynamic damping remains constant at a high level up to 10 m/s since the flow is fully attached. Damping is decreasing until the rated wind speed of 13.7 m/s caused by partial flow separation at the inner blade sections. Beyond the rated wind speed the pitch controller decreases the angle of attack to limit the power. Flow becomes more attached and the aerodynamic damping increases again. All three methods show quite similar results. The slightly lower damping estimates of the closed-form expression (5.12) are probably caused by neglecting the contribution of the drag force.

Figures 7.11 and 7.12 highlight the importance of the dynamics of the control system. In the range just above the rated wind speed the aerodynamic damping increases compared to the case with stationary controller behaviour. However, for higher wind speeds the damping lowers dramatically down to only 0.5% at the cut-out wind speed. The time series in Figure 7.6 illustrate this effect in the time domain.
Figure 7.7: Determination of aerodynamic damping from transient vibration in a steady wind field

Figure 7.8: Determination of aerodynamic damping from transient vibration in a turbulent wind field
Figure 7.9: Determination of the logarithmic damping decrement of the first mode from free vibrations at a mean wind speed of 12 m/s

The controller is switched on and off at the rated wind speed. Transition phenomena cause lower damping in the more realistic turbulent case than for both numerical linearisation and transient analysis with steady inflow. Above 16 m/s the damping derived in turbulence is approximately 0.5% higher than in a steady wind field. The actual behaviour may be specific to the reference case. For instance, introduction of a threshold for activating the pitch variation might reduce the undesired feed-back from the wave excitation to the pitch system (Figure 7.6). Nonetheless, the inclusion of the controller dynamics is essential for the damping of controlled turbines. The simple PI controller was designed as single input - single output system to limit the variation of the rotor speed and corresponding generator power. In fact in some advanced wind turbines the control system fulfills multiple control goals, e.g. reduction of both power variations and fatigue loads [7.3]. Application to OWEC should be possible in principle, as well.

Recommendations

A number of conclusions are drawn from the investigations. It should be kept in mind that only one particular design was analysed. Pitch regulated turbines with variable speed, for instance, might show different behaviour. The application of either the numerical linearisation or the transient vibration in a steady wind field is recommended for the determination of the additional structural damping. Evaluation of transients should be possible with any wind turbine design tool.

The accuracy of the estimates by the closed-form expression depends strongly on those of the assumed flow conditions along the rotor blades. If input data from a standard aerodynamic code are applied, the method might be suited well for the wind speed range without controller activity. Stall regulated machines should be treated with care since the effects of aerodynamic drag and unsteady aerodynamics are not included in the formula.

The numerical linearisation and the free vibrations analysis in steady inflow are preferred with regard to the more cost and cumbersome free vibrations in turbulence. The differences between the approaches are not significant with respect to other inaccuracies of the entire simplified fatigue analysis approach. Generally the accuracy of any theoretical estimate of the aerodynamic damping is at least in the order of ±0.5%
**Figure 7.10:** Aerodynamic damping derived without controller dynamics

**Figure 7.11:** Aerodynamic damping derived by numerical linearisation or free vibration analysis

**Figure 7.12:** Aerodynamic damping derived from free vibration analysis with and without controller dynamics
to 1%. It seems more appropriate to apply a simpler approach and add some conservatism than to take the risk of introducing modelling errors or larger deviations by imperfect execution of a more involved approach.

In the transition region of the controller it is recommended to consider the lower damping estimate from two separate estimates with and without controller dynamics.

7.1.4 Empirical validation of separation of responses

An empirical verification of the separation of the response is carried out in the time domain. Figure 7.13 illustrates the different approaches used.

First the damage equivalent fatigue loads are calculated after Rainflow Counting (RFC) of time domain simulations of the simultaneous wind and wave response (top).

In the two separate analysis approaches the time series of the wind response in calm water are superimposed to those from the wave response which accounts one or another way for the aerodynamic damping.

In the first case the modal damping ratio of the first fore-aft mode is increased by the estimated aerodynamic damping. Again damage equivalent fatigue loads are computed (middle). This methods is applied once with full substitution of the aerodynamic damping estimated by DUWECS linearisation and once only with substitution of 80% of aerodynamic damping.

Secondly the wave response in presence of steady wind is simulated with an integrated model of the OWEC including the dynamic behaviour of the controller. The mean wind speed was adjusted according to the wave height. The case is considered only for comparison.

For each load case the same seeds are used for the generation of the stochastic wind and wave loading regardless whether the simultaneous or separate responses are simulated. In total four representations each with a duration of 40 minutes were considered for 17 production load cases similar to Table 8.3. Equivalent fatigue loads were derived by Rainflow Counting of 4000 to 18500 cycles and for an S-N curve with an inverse slope of four.

Figures 7.14 and 7.15 show the error of the equivalent fatigue loads at two cross sections derived from superimposed time series relative to the simultaneous response. The markers and trendlines present the results for the individual load cases whilst the bar charts at the right-hand side illustrate the extrapolation to the entire lifetime. The latter is dominated by the load cases associated with high wind speeds. The two different cross sections correspond to either a wave or a wind dominated loading regime.

The scatter between different load cases, probably owing to the random representation and finite sampling frequency, is significant. Compared to this and the spreading in the fatigue strength determined by experiments there is only a small difference between the three discussed approaches. Consideration of the full amount of the estimated aerodynamic damping appears to underestimate the combined response between 1.5% and 3.5% with respect to the entire lifetime. Much larger errors occur for the individual load cases. If only 80% of the aerodynamic damping is accounted for, the combined responses are approximated conservatively by a small overestimate of the long-term fatigue loads between 0.5% and 2%.

No doubt the accuracy of the long-term fatigue estimates seems reasonable for a simplified analysis with any of the two damping cases. Other designs with lower damping than this pitch controlled OWEC with a soft support structure might be considerably
more sensitive to variations of the assumed damping. As a conservative measure and with respect to other conditions it is recommended to account only for 80% of the estimated aerodynamic damping.

The consideration of the aerodynamic damping from simulations of the wave response in a steady wind field obtained a 1.5% to 3% too conservative estimate of the long-term fatigue loads. The scatter in the individual results was smaller than for the other two damping cases. However, this does not justify the considerably higher computational effort associated with the application of an integrated model.
Figure 7.14: Error of equivalent fatigue loads from superimposed time series of separate wind and wave responses
(Cross section 1 below mudline: Relative contribution of wave loading 71 - 82%)

Figure 7.15: Error of equivalent fatigue loads from superimposed time series of separate wind and wave responses
(Cross section 7 near tower top: Relative contribution of wave loading 19 - 48%)
7.2 Superposition of fatigue loading

After establishing the wind and wave contributions to the simultaneous response, the question arises how to combine the results of separate fatigue analyses. In Section 7.2.1 we analyse different techniques for combining the results of two separate fatigue analyses. The superposition of damage equivalent fatigue loads is identified as most suitable. Section 7.2.2 derives two methods on either an analytical or semi-empirical basis which are numerically validated in Section 7.2.3.

7.2.1 Evaluation of different superposition strategies

Simplistic but conservative approach

A simplistic, too conservative method of combining the results of two separate fatigue analyses is the in-phase superposition of damage equivalent loads due to wind and waves (Figure 7.1). In the previous section we have learnt how to consider the aerodynamic damping of the wave response. Now we are looking for an alternative superposition technique that accounts for the partial cancellation owing to the random phase relation between wind and wave response. A solution was sought systematically in two directions: literature survey and evaluation of possible combinations during different stages of the fatigue analysis process.

Literature survey

Fatigue due to two independent processes has been addressed by researchers in different fields, e.g. dynamics of moored floating devices, aerospace engineering, railway dynamics.

Combined wind and wave fatigue is not significant for bottom-mounted offshore structures in the oil and gas industry. Support structures are dominated by hydrodynamic loading. While wind induced fatigue is only important for exposed and flexible elements such as masts, towers and flare booms. Relevant experience for our investigation is, however, found from the mooring dynamics of floating devices. The environmentally induced response can be divided into three frequency ranges, i.e. steady state response due to current forces, mean wind and mean wave drift forces, low-frequency vessel motion due to wind and waves excitation of the fundamental mooring modes and wave-frequency vessel motion in a higher frequency range. Both types of dynamics introduce fatigue [7.4]. The effect can be approximated well by two independent narrow banded processes with zero-crossing frequencies differing by a factor between four and 20.

Lie and Fylling [7.5] compared five approaches which were proposed as alternative to accurate but expensive time domain simulation and successive Rainflow Counting of the combined response.

The two authors prefer the combination of two narrow banded Gaussian responses after Jiao and Moan [7.6]. The fatigue damage due to a slowly varying process is added to the damage due to the narrow banded wave-frequency process. Where the slowly varying process consists of the low frequency resonance of the mooring tension and the envelope of the high frequency wave induced tension.

The approach is founded on a firm theoretical basis but unfortunately it leads to fairly conservative results in our case since two basic assumptions are not valid. The wind response of a support structure of an offshore wind energy converter is broad banded while the wave response is reasonably narrow banded in most cases. Furthermore,
the responses occur in partially overlapping frequency ranges if the fundamental eigenfrequency is excited by both wind and waves. The so-called 'simple summation' of the damage of the individual responses produces highly nonconservative results because of the non-linear dependency of the total damage on the stress magnitude of the two responses. In the 'combined spectrum method' the fatigue damage is derived from the superimposed spectra. The broad bandwidth of the combined response produces conservative results if the damage is calculated under the assumption of a narrow bandwidth. This is also the case for the bandwidth corrections according to Wirsching and Light [7.7] or Jiao [7.8]. A better alternative, not considered by Lie and Fylling, would be the treatment of the combined spectra by the semi-empirical approach after Dirlik, (7.10) [7.9]. For us it is however difficult to properly estimate spectra from the signals of the wind response.

In the field of aerospace engineering Lambert [7.12] proposed an analytical approach for the fatigue analysis of a single degree of freedom system with combined sinusoidal and random responses. A typical example includes the gunfire vibration of a jet aircraft. The method is not considered here because of the restriction on two narrow banded component responses with the same zero-crossing frequency.

Sakai and Okumara [7.16] developed a semi-empirical combination of the Rayleigh stress range distributions associated with two narrow banded processes in distinct frequency ranges. The method was discarded for our purposes because of the broad banded wind response.

In different technical disciplines sometimes damage equivalent, constant amplitude fatigue loads with the same number of cycles are combined. If nothing is known on the phase relation, an in-phase superposition or arithmetic summation of the load magnitudes provides a conservative estimate for the combined fatigue loading (Figure 7.16 upper). For spatially uncorrelated loads, e.g. the axial and bending loads in a member of an offshore platform, the $90^\circ$ out-of-phase superposition or square root of the

![Diagram](image)

**Figure 7.16**: In-phase versus $90^\circ$ out-of-phase superposition of harmonic signals

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2 The Dirlik method is summarized for instance in references [7.10] and [7.11] which are easier available than the original work.
sum of the squares (quadratic summation) of the load magnitudes has been proposed [7.14] (Figure 7.16 lower). Unfortunately no theoretical or empirical basis for the latter case was found in the literature.

Superposition during different stages of the fatigue analysis

Considering the fatigue analyses approaches in the time domain and the frequency domain as parallel processes, as illustrated by Figure 6.5, a combination of results is possible at four stages:

- superposition of time series or response power spectra (after transformation to one domain)
- superposition of spectral moments (after derivation of 'pseudo-moments' from the standard deviation and counted zero crossings and peaks)
- combination of stress range distributions
- combination of damage equivalent stress ranges.

As previously mentioned, any superposition of damages from the separate responses is senseless. Highly nonconservative results can be the effect, even in case of one dominant component response.

A systematic investigation of the four strategies for the superposition of wind response calculated in the time domain and wave response computed in the frequency domain was carried out in cooperation with Hofland [7.1].

The first option, the superposition of response time series or response power spectra is a straightforward solution for independent signals. However, inherent inaccuracies are caused by the spectral estimate of time series of finite length and owing to the randomness of the time domain representations of the spectra. Considering also the large amount of data associated with time series, this option is not suited for early design purposes. Later in this section the superposition of sufficiently long time series is used as reference solution during the verification of other superposition approaches.

Another possibility is the superposition of the spectral moments and successive application of a frequency domain relation for the fatigue damage. Under fatigue conditions it is reasonable to assume the wind and wave responses as Gaussian. Referring to Rice's fundamental work on Gaussian processes [7.15] the zeroth, second and fourth spectral moment, \( m_0 \), \( m_2 \) and \( m_4 \), can be estimated from the standard deviation, \( \text{std} \), the counted zero-crossing period, \( T_z \), and the peak period, \( T_p \), of stress time series, \( \sigma(t) \), which are sufficiently long from a statistical point of view, (7.2).

\[
\begin{align*}
m_0 & = \text{std}(\sigma(t))^2 \\
m_2 & = \left( \frac{\text{std}(\sigma(t))}{T_z(\sigma(t))} \right)^2 \\
m_4 & = \left( \frac{\text{std}(\sigma(t))}{T_z(\sigma(t)) T_p(\sigma(t))} \right)^2
\end{align*}
\]

The superposition of the spectral moments encounters a practical problem owing to the generally broad bandwidth of the combined OWEC response and the given set of spectral moments. A reasonable estimate of the fatigue loading would be possible only with the Dirlik approach which requires also the first spectral moment. However, an accurate estimate of the first spectral moment can hardly be gained from finite time
series. Application of other, less sophisticated frequency domain methods tends to be too conservative.

As mentioned above the method after Sakai and Okumara for the superposition of stress range distribution is discarded since at least the wind response is nonnarrow. In-phase superposition of two stress range distributions is possible after both fatigue load spectra are portioned in sets with constant stress ranges and corresponding number of stress cycles [7.17]. However, application as arithmetic superposition would be too conservative here. After establishing the so-called 'direct quadratic superposition' of the equivalent stress ranges (7.17) in the next section one may propose to combine the stress range distributions accordingly. This approach is certainly worth of further investigation, e.g. when dealing with S-N curves with bi-linear slopes, which is beyond the scope of the present research.

The problem directly leads to the consideration of the superposition of damage equivalent, constant amplitude fatigue loads with the same reference number of cycles. Such quantities are a compact and characteristic representation of the fatigue loads which are obtained conveniently in both time and frequency domain.

For our problem a clear preference we give to the combination of damage equivalent fatigue loads which is further developed in the next sub-section.

7.2.2 Derivation of the superposition of equivalent fatigue loads
The purpose of this section is to derive a relation for the superposition of the aerodynamic and hydrodynamic fatigue loads of with two stationary, random processes.

Basic fatigue theory
In this thesis the material resistance to fatigue failure is represented by S-N curves in combination with the linear damage accumulation after Palmgren and Miner [7.18]. An alternative, better suited for the assessment of in-service cracks and for the selection of inspection methodology, is to use fracture mechanics techniques, which are based on an analysis of the behaviour of the cracks themselves [7.19]. In contrast to the offshore or aircraft technology such methods are not frequently used in wind energy applications to date.

\[ N(\Delta \sigma) = A \Delta \sigma^\mu \] (7.3)

For steel, the influence of the mean stress is negligible and the number of endured cycles, \( N \), of a periodic loading depends only on the stress range, \( \Delta \sigma \), and the two empirical constants \( A \) and \( \mu \). In order to derive analytical expressions it is convenient to assume S-N curves with constant inverse slope \( \mu \).

Different S-N curve are used depending on the loading, environment, local geometrical detail and last not least the applied standard. For instance, Figure 7.17 illustrates three

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3 The first spectral moment, defined by (7.11), can be approximated from the integration of the estimated response spectrum or from the bandwidth parameter after Vanmarcke (7.9), [7.16]. For narrow banded processes the Vanmarcke bandwidth parameter, \( \delta \), is given by the ratio of the standard deviations of the slopes of the envelope process and the process itself. Another indirect way is to back iterate the first spectral moment with the aid of the Dirlik approach from equivalent stress range (7.6) after Rainflow Counting of the signal.
Figure 7.17: Examples of S-N curves according to Germanischer Lloyd
(The curve with inverse slope of $\mu = 4$ is used as a linear approximation of the bi-linear curve of Type 'A' with $\mu = 3$ and $\mu = 5$.)

curve types proposed by the guidelines of Germanischer Lloyd. Type 'A' with bi-linear inverse slopes of $\mu = 3$ and $\mu = 5$ refers to a mildly corrosive environment or a corrosive environment where an adequate protection is provided. For a constant stress range magnitude in a non-corrosive situation a fatigue limit may be assumed by using type 'C' curves, whilst for a corrosive environment without protection the inverse slope of $\mu = 3$ is extended according to type 'B'.

The Type 'A' - curve is most appropriate for us and we are using it for analyses on the certification level (Sections 9.6.1 and 10.3.2) where a numerical evaluation of the S-N curve is done anyway. In the preliminary design stage and when an analytical evaluation is desired or when damage equivalent stress ranges are dealt with (Chapters 7 and 8, Sections 10.1.2 and 10.2.2) the assumption of a constant slope is convenient. In our case the bi-linear Type 'A' - curve is approximated by an S-N curve with the same reference fatigue strength but constant inverse slope of $\mu = 4$. Figure 7.17 proves this to be conservative for a wide range of endured number of stress cycles with the exception between $2 \cdot 10^6$ and $2 \cdot 10^7$ cycles. For stochastic loading where many different stress range magnitude are occurring this simplification is conservative in most cases.  

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4 Taking the stress range distributions from Figures 9.17 and 9.18 as examples the damage is -4% to 14% higher at the monopile and 51% to 72% higher at the tower if a constant inverse slope of 4 is assumed. The too low estimation of -4% occurs for loading case No. 2 at the monopile where almost half of the total damage of $D = 5.3$ is associated with stress ranges between 50% and 100% of the reference fatigue strength.
The ratio between the duration of the process, i.e. in most cases the design lifetime, and the expected time until a failure is denoted as fatigue deterioration or (fractional) damage, $D$. Depending on the applied safety philosophy the design damage is set between 1 and, say, 0.1 corresponding to a time to failure ranging between the service life and a 10 times longer period.

After Palmgren and Miner the damage equals the summation of the ratio between the predicted, $n(\Delta \sigma_j)$, and the endured number of stress cycles, $N(\Delta \sigma_j)$, for all stress range classes, $\Delta \sigma_j$:

$$D = E \left[ \sum_j \frac{n(\Delta \sigma_j)}{N(\Delta \sigma_j)} \right] \quad (7.4)$$

The stress range distribution is gained either from a histogram, $n(\Delta \sigma_j)$, resulting from Rainflow Counting of the time series or from a probability density function of stress ranges, $\rho(\Delta \sigma)$, derived from the spectral moments.

**Equivalent stress range**

Often it is convenient to introduce damage equivalent, constant amplitude loads. For S-N curves with a constant slope such an equivalent stress range, $\Delta \sigma_{eq}$, depends only on the slope parameter, $\mu$, the range distribution of cycles, $n(\Delta \sigma_j)$, and an arbitrary reference number of cycles. In wind energy engineering, for instance, the total number of rotor revolutions during the design life is used to define the so-called ‘equivalent 1P stress range’ where 1P denotes the revolution frequency $[7.20]$.

Likewise the total number of stress cycles, $n_{total}$, during the duration, $T_{process}$, of the process with is used sometimes as reference number of cycles $[7.10]$.

$$n_{total} = \frac{T_{process}}{T_p} = \sum_j n_j \quad (7.5)$$

For us it is convenient to define the equivalent stress range with respect to the number of endured cycles, $N_{eq}$, associated with the reference fatigue strength or detail category, $\Delta \sigma_{eq}$, of the considered S-N curve, i.e. $2 \cdot 10^6$ according to EUROCODE or Germanischer Lloyd (Figure 7.17). $^5$

---

$^5$ Another load parameter is useful in the particular case of a bi-linear S-N curve. In a design situation the structure’s geometry has to be adjusted until the design stress is below the design strength. Therefore the stress reserve with respect to a unity damage rather than the actual damage is of interest. For this purpose van Delft at al. $[7.21]$ introduced the so-called $K'$ - value. For a given stress range distribution the reference fatigue strength, $\Delta \sigma_{ref}$, of a fictitious S-N curve is varied iteratively until the damage approximates one. The gained reference fatigue strength is denoted as the ‘$K'$ - value’ where the superscript 1 refers to unity damage. The $K'$ - value, also written as $\Delta \sigma_{ref}'$, can be regarded as the required reference fatigue strength, however, this is meaningful only in a design optimisation process and not if a measure for the load magnitude is required.

The $K'$ - value, rather than the equivalent stress range, is linear in any load factor of the stress range distribution. Hence the ratio of reference fatigue strength and $K'$ - value represents the design's reserve in fatigue strength which is also denoted as 'stress reserve factor'.

Our investigation is restricted to S-N curve with constant slope, so the equivalent stress range, $\Delta \sigma_{eq}$, and the $K'$ - value are identical. During a possible future extension on bi-linear S-N curves the $K'$ - value should be considered. It provides a kind of natural weighting of the shape of the stress range distribution which is useful if fatigue loads with different magnitudes are combined.
\[ \Delta \sigma_{eq} = \mu \sqrt{\frac{N_{total}}{N_R} \int_0^\infty \rho(\Delta \sigma) \Delta \sigma^\mu \, d\Delta \sigma} = \mu \sqrt{\frac{1}{N_R} \sum_j \Delta \sigma_j^\mu n_j(\Delta \sigma_j)} \] (7.6)

For a bi-linear S-N curve the equivalent stress range is calculated from the inverse S-N curve \( \Delta \sigma(N) \) as function of the damage, \( D \), from (7.4).

\[ \Delta \sigma_{eq} = \Delta \sigma \left( \frac{N_R}{D} \right) \] (7.7)

**Bandwidth effects**

The wide frequency range of the wind loading and partially the different frequency content of the wind and waves result in a broad banded response of OWEC support structures. Figures 7.18 and 7.19 illustrate typical spectra and signals with different bandwidths.

Different parameters are used to measure the bandwidth of a signal. We apply the regularity factor, \( \beta \), and the more theoretical bandwidth parameter after Vanmarcke, \( \delta \), [7.16].

\[ \beta = \frac{m_2}{\sqrt{m_0 m_4}} = \frac{T_p}{T_z} \] (7.8)

\[ \delta = \sqrt{1 - \frac{m_1^2}{m_0 m_2}} \] (7.9)

For Gaussian processes the regularity factor represents the ratio between peak period \( T_p \), i.e. the mean time between peaks, and zero up-crossing period \( T_z \), i.e. the mean time between up-crossings of the mean value. Narrow bandwidth is treated only as the specific case with \( \beta = 1 \) and \( \delta = 0 \) of a generally broad banded process characterised by \( 0 < \beta < 1 \) and \( 0 < \delta < 1 \).

The probability density function of stress ranges of a narrow banded signal equals the Rayleigh distribution of the stress peaks and an analytical expression for the fatigue damage exists [7.22].

However, all positive troughs and negative peaks are ignored and all positive peaks define ranges of double amplitude regardless whether they actually form stress cycles. For broad banded signals the approach therefore overestimates the probability of large stress ranges and so any damage calculation tends to be conservative. Various researchers have proposed semi-empirical or analytical relations to overcome this shortcoming, e.g. [7.10], [7.23 - 7.27]. Independent comparisons [7.5, 7.9, 7.11, 7.28], proved Dirlik's method (7.10) to be currently the best frequency domain approach for broad banded processes while Rainflow Counting of time series of sufficient length is preferred in the time domain.
\[
\rho(\Delta \sigma) = \frac{D_1 e^{\frac{Z}{Q}} + \frac{D_2 Z}{R^2} e^{\frac{Z^2}{2R^2}} + D_3 Z e^{\frac{Z^2}{2}}}{2 \sqrt{m_0}} \\
\text{Dirlik expression for distribution of Rainflow ranges}
\]

where:

\[
\beta = \sqrt{\frac{m_2}{m_0 m_4}} \quad \gamma = \frac{m_1}{m_0} \sqrt{\frac{m_2}{m_4}} \quad D_1 = \frac{2 (\gamma - \beta^2)}{1 + \beta^2} \quad D_2 = \frac{1 - \beta - D_1 + D_1^2}{1 - R} \quad D_3 = 1 - D_1 - D_2
\]

\[
Q = \frac{1.25 (\beta - D_3 - D_2 R)}{D_1} \quad R = \frac{\beta - \gamma - D_1^2}{1 - \beta - D_1 + D_1^2} \quad Z = \frac{\Delta \sigma}{2 \sqrt{m_0}}
\]

\text{(7.10)}

**Weighted quadratic superposition of equivalent stress ranges**

Section 7.1 has examined conditions under which the simultaneous response can be approximated by superposition of separate responses under wind and wave loading. Hence, the auto spectral density, \(S_{\sigma, \sigma}^{nh}\) of the simultaneous response is composed of the linear superposition of the auto spectral densities, \(S_{\sigma, \sigma}^{a}\) and \(S_{\sigma, \sigma}^{n}\) of the separate responses and likewise the superposition is valid for the \(n^{th}\) spectral moment, \(m_n\).

Here index 'a' denotes all aerodynamic related loading and response, comprising direct effects due to wind loading of the structure as well as any indirect effects due to rotor operation, functional forces and structural response. Likewise, all hydrodynamic effects are marked by index 'h', whilst 'ah' refers to the combined response.

\[
m_{n, \text{eh}} = \int_0^\infty S_{\sigma, \sigma}^{\text{eh}}(f) f^n \, df = \int_0^\infty \left( S_{\sigma, \sigma}^a(f) + S_{\sigma, \sigma}^h(f) \right) f^n \, df
\]

\[
= m_{n,a} + m_{n,h}
\]

\text{(7.11)}

Note that the spectral moments are defined here as functions of the cyclic frequency, \(f\), in units of Hertz.\(^6\)

The superposition of the energy contents, i.e. the variances or zeroth moments, of the separate responses determines the basis for the superposition of aerodynamic and hydrodynamic fatigue loads. The peaks of a narrow banded process are Rayleigh

\[\text{Using the relation between the spectral densities in either cyclic or circular frequency } S(f) = 2 \pi S(\omega) \text{ the spectral moments, } m_n, \text{ expressed in the circular frequency } m_n(\omega) = (2\pi)^n m_n(f), n \geq 0 \text{ equal } 2 \pi \text{ to the power of } n, \text{ times the moments related to cyclic frequency.}\]
distributed. Applying (7.6) we find a relation between the variance and the equivalent stress range which can be easily resolved with respect to the variances.  

\[ \Delta \sigma_{eq} = \sqrt{8 m_0} \mu \sqrt{\frac{n_{total}}{N_R}} \Gamma \left( \frac{2 + \mu}{2} \right) \]  

(7.12)

Unfortunately, both the aerodynamic and the simultaneous response are not narrow banded. Dirlik’s expression (7.10) is capable of such cases but cannot be resolved with respect to the variance. In the current situation we are looking, however, only for a formula to link the equivalent stress ranges rather than for the magnitude of the separate fatigue loads. In this respect the less accurate, empirical correction of the analytical narrow banded solution after Hancock and Gall, [7.10, 7.24] is more convenient. We rewrite the original form in terms of the reference number of cycles, \( N_{R} \), instead of the total number of stress cycles, \( n_{total} \), (7.13).

\[ \Delta \sigma_{eq} - \sqrt{8 m_0} \sqrt{\beta \frac{n_{total}}{N_R}} \Gamma \left( \frac{2 + \mu}{2} \right) \]  

Hancock expression (7.14)

Substitution of the variances in (7.11) leads to:

\[ \Delta \sigma_{eq, ah} = \sqrt{\Delta \sigma_{eq, a}^2 \left( \frac{\beta_{ah}}{\beta_{a}} \frac{n_{total, ah}}{n_{total, a}} \right)^2 \frac{2}{\mu} + \Delta \sigma_{eq, h}^2 \left( \frac{\beta_{ah}}{\beta_{h}} \frac{n_{total, ah}}{n_{total, h}} \right)^2 \frac{2}{\mu}} \]  

(7.15)

After exchange of the total number of cycles and the regularity factor from Eqn. (7.5) and (7.8) one obtains a compact relation. The equivalent stress ranges of the two responses are weighted by the \( \mu \)th root of the ratio between the zero-crossing periods, \( T_z \), of the component processes and the combined response. Successively the contributions are added as the root of the sum of the squares.

\[ \Delta \sigma_{eq, ah} = \sqrt{\Delta \sigma_{eq, a}^2 \left( \frac{T_{z, a}}{T_{z, ah}} \right)^2 \frac{2}{\mu} + \Delta \sigma_{eq, h}^2 \left( \frac{T_{z, h}}{T_{z, ah}} \right)^2 \frac{2}{\mu}} \]  

(7.16)

The weighting factors are substituted in terms of the zeroth and second spectral moment, \( m_0 \) and \( m_2 \), since the zero-crossing period of the combined response, \( T_{z, ah} \), is not directly available.

After this final arrangement we find the desired expression for the so-called ‘weighted quadratic superposition of equivalent stress ranges’ (7.17).

---

7 Neglecting the bandwidth one might derive a superposition formula also from the analytical solution under the rough assumption that the total number of cycles of the simultaneous response equals the sum of the total number of cycles of the individual responses.

\[ \Delta \sigma_{eq, ah} = \sqrt{\Delta \sigma_{eq, a}^2 \left( \frac{n_{total, a}}{n_{total, h}} \right)^2 \frac{2}{\mu} + \Delta \sigma_{eq, h}^2 \left( \frac{n_{total, a}}{n_{total, h}} \right)^2 \frac{2}{\mu}} \]  

(7.13)

Due to the good results later in this section, the expression was, however, not further investigated.
Weighted quadratic superposition of equivalent stress ranges

\[
\Delta \sigma_{eq,ah} = \mu \sqrt{\frac{m_{2,a} + m_{2,h}}{m_{0,a} + m_{0,h}}} \left( \Delta \sigma_{eq,a}^2 \sqrt{\frac{m_{0,a}}{m_{2,a}}} + \Delta \sigma_{eq,h}^2 \sqrt{\frac{m_{0,h}}{m_{2,h}}} \right) \tag{7.17}
\]

All required quantities are available in the frequency domain or can be estimated with Eqn. (7.2) in the time domain.

**Direct quadratic superposition of equivalent fatigue loads**

In Sections 7.2.3 and 7.3 a comprehensive numerical investigation of the validity of Eqn. (7.17) will be carried out. Prior to this, however, a further simplification of the relation is proposed from a practical rather than a theoretical point of view.

A more convenient relation is obtained if the weighting factors are omitted, assuming approximately equal zero-crossing periods of the individual responses. The Pythagoras law is recovered, which can also be interpreted as 90° out-of-phase superposition of the equivalent stress ranges (7.18).

**Direct quadratic superposition of equivalent stress ranges**

\[
\Delta \sigma_{eq,ah} = \sqrt{\Delta \sigma_{eq,a}^2 + \Delta \sigma_{eq,h}^2} \quad \text{for } T_{z,a} = T_{z,h} \tag{7.18}
\]

In most cases the above assumption is not valid in a strict sense because the aerodynamic response is generally associated with a shorter zero-crossing period than the hydrodynamic response. Therefore, particular care is required during the following numerical verification of the simplified expression.

### 7.2.3 Empirical validation of superposition approaches

**Outline of the simulation study**

The validity of the two derived formulas for the superposition of the equivalent stress ranges, Eqn. (7.17) and (7.18), was investigated by an extensive numerical study including parametric variation of the shape and the bandwidth of the individual response spectra. In addition, the influence of the ratio of the variances and ratio of the zero-crossing frequencies of the component processes was analysed.

Because of the great bandwidth of some of the combined responses only a comparison in the time domain was meaningful. The superposition of the equivalent stress ranges derived from the time series of the separate responses was compared to the result of Rainflow Counting of the superimposed time series. So, any influence of the methods described in Section 7.1, as separation of wind and wave contribution and substitution of aerodynamic damping was not included in this investigation. See Section 8.4 for such an overall validation.

The complexity of the investigation was increased successively for four different types of responses.
• block bimodal spectra with \( \delta = 0.1 \)
Guided by Jiao and Moan [7.6] block bimodal spectra were used corresponding to two narrow banded processes with a Vanmarcke bandwidth, \( \delta \), of 0.1. The ratio of variances ranged from 0.1, 0.3, etc. to 0.9 whilst the zero-crossing frequencies differed by factors of 0.8, 1.2, 1.6, 3.2 and 7.9.

• block bimodal spectra with \( \delta_1 = 0.1 \) and \( \delta_2 = 0.3 \)
As above, but with distinct Vanmarcke bandwidths. Figure 7.18 presents typical power spectra and associated time series.

• simplified response spectra
Analytical wind and wave response spectra were derived from a single-degree-of-freedom system with an eigenfrequency of 1 Hz and von-Karman turbulence and Pierson-Moskowitz wave spectra. The damping constant was varied artificially to construct different bandwidth values and different contributions of the quasi-static and dynamic part of the response (Figure 7.19). In particular the low-frequency quasi-static wind response resulted in a broad band. Again the ratio of the component variances varied from 0.1 to 0.9.

---

**Figure 7.18:** Bimodal spectra and corresponding time series for different ratios between the zero-crossing frequencies of the component processes (Vanmarcke bandwidth of components \( \delta_1 = 0.1 \) and \( \delta_2 = 0.3 \), variances 0.5 MPa²)

---

\(^6\) For the block spectra the Vanmarcke bandwidths \( \delta_1 = 0.1 \) and \( \delta_2 = 0.3 \) correspond to regularities of \( \beta_1 = 0.981 \) and \( \beta_2 = 0.87 \).
• OWEC response spectra

Finally time series for three load cases of the wind and wave response of the reference
design were evaluated. The ratio between the variances ranged from 0.4 to 13
depending on the considered cross section and load case. Figure 7.20 presents the
different frequency contents of the wind and wave responses for one example. In
distinction to the simplified response spectra, several peaks, corresponding to the fore-
aftr modes and the multiples of the blade passing frequency 2P, are observed in the wind
response spectrum and the bandwidth is further increased. The wave response has
moderate bandwidth and is only significant at the fundamental support structure mode
of 0.29 Hz and the quasi-static wave excitation.

Time series were generated by Inverse Fast Fourier Transform (IFFT) and randomly
chosen phase angles in the first three cases. The DUWECs simulation code was
applied for the OWEC spectra. The minimum number of stress cycles in any response
was 2300 for the block bimodal spectra, 7100 for the simplified and 7700 for the OWEC
response spectra. Generally each stress cycle was represented by at least 10 time
steps. The time series of the OWEC responses were sampled with 13 Hz. For the high
frequency wind response this corresponded on average only to 5.5 points per cycle,
however, in such cases more than 80000 cycles were evaluated.

Discussion of results

At the beginning the overall accuracy of the four methods was studied. The analytical
method after Jiao and Moan [7.6], the evaluation of the superimposed response spectra
with the Dirlik approach (7.10) and the two formulas for the superposition of the
equivalent stress ranges were compared for block bimodal spectra with Vanmarcke
bandwidths of the components of $\delta_1 = 0.1$ and $\delta_2 = 0.3$.

Figure 7.21 plots the relative error of the equivalent fatigue loading against the regularity
of the combined response $\beta$. Generally the error increased for greater bandwidth, i.e.
lower $\beta$. Only the approach after Jiao and Moan behaved in the opposite manner
because it requires two narrow banded component processes in distinctly different
frequency ranges.

Both the direct quadratic superposition of the equivalent stress ranges (7.18) and the
Dirlik approach exhibit a tendency for slightly nonconservative results for a greater
bandwidth. Larger absolute errors but with positive, i.e. conservative, sign were found
for the weighted quadratic superposition of the equivalent stress ranges (7.17).
Generally the accuracy of the two new formulas was at least as good as any of the
alternatives.

Comparison of the error of the weighted quadratic superposition of the equivalent stress
ranges (7.17) for all four types of response spectra in Figure 7.22 was surprising. The
best approximations were observed for the OWEC spectra rather than for any simplified
spectrum. Typical errors accounted for -3% to 5% of the equivalent fatigue loading,
which is regarded as a quite reasonable accuracy for a simplified approach.
The direct quadratic superposition (7.18) showed a smaller scatter and smaller
magnitude of errors than the weighted quadratic superposition (Figure 7.23). However,
the clear tendency to nonconservative results of up to -5% is unfavourable.
Figure 7.19: Simplified spectra: Response of a single degree of freedom system to wind excitation (left) and wave excitation (right) (Variances normalised to unity.)

Figure 7.20: OWEC response spectra for bending of soft-soft monopile below mudline ($V_w = 14$ m/s, $H_s = 1.67$ m, $T_s = 3.34$ s): pure wind response (left), simultaneous wind and wave response (right, solid line), wave response with aerodynamic damping (right, dashed line)
7.3 Empirical validation of superposition of separate fatigue analyses

7.3.1 Short-term fatigue loads

So far we have learnt to analyse wind and wave responses of an OWEC separately (Section 7.1) and to combine the resulting equivalent fatigue loads (Section 7.2). The individual methods showed reasonable accuracy for a simplified approach, however, the accumulation of errors through the entire trajectory has not been investigated, yet. For this purpose the equivalent fatigue loads of the reference design at seven hot spots were analysed for 17 lumped production load cases (Table 8.3). Aerodynamic loading was calculated by time domain simulations with a commonly used duration of 10 minutes per wind speed class. Spectral fatigue analysis with the Dirlik approach (7.10) obtained the hydrodynamic fatigue loads.

95% confidence intervals for the estimated fatigue loading were derived by statistical evaluation of 12 representations of 10 minutes. Per load case the average of 9 simulations of 40 minutes each was used as reference of either pure aerodynamic or simultaneous wind and wave loading.

In order to avoid bias by application of different models and tools, all analyses except the operations in the frequency domain were performed with the DUWECs code. For instance, the transfer functions were derived from time domain simulations of the steady state wave response while in practice an offshore design tool might establish them directly in the frequency domain. Likewise the aerodynamic damping was accounted for in steps of 0.5% after DUWECs linearisation.

Matching of the hydrodynamic model in the frequency domain and the corresponding model used for the integrated time domain simulation of the simultaneous response was checked by comparing 20 wave load cases without aerodynamic damping. Figure 7.24
Figure 7.22: Error of weighted quadratic superposition of equivalent reference stress ranges (7.17) for different response spectra

Figure 7.23: Error of direct quadratic superposition of equivalent reference stress ranges (7.18) for different response spectra
plots the relative difference as function of wave height for the lowest and the highest hot spot considered. Despite the long simulations with each 9 time series of 40 minutes, still some scattering is observed due to the random representations. The general agreement is very good which can be seen from the interpolated trend lines and the extrapolation to the entire lifetime. The frequency domain approach shows slightly higher loads especially in the top of the structure.

Four combinations resulting from the magnitude of considered aerodynamic damping, either 80% or 100%, and of either the weighted or the direct quadratic superposition were analysed. Figure 7.25 presents the relative error of the combined loading and the 95% confidence intervals against the mean wind speed.

Both methods show similar accuracy. Depending on the cross section and load case the mean error of an individual load case differs ±10% while the spreading due to only one simulation of 10 minutes accounts for additional ±3% to ±5%. The direct quadratic superposition suffers slightly smaller errors but a general conclusion can hardly been drawn from the short-term behaviour.

7.3.2 Long-term fatigue loads

Comparison of the long-term fatigue loads accumulated from all the load cases on the right hand side of the diagram shows much better accuracy. Which is shown in Figure 7.26 presenting the long-term error along the entire structure. Accounting for 100% of aerodynamic damping nearly all results are slightly non-conservative. So the recommendation from Section 7.1.3 is confirmed to consider only 80% of the estimated aerodynamic damping as additional structural damping.

The direct quadratic superposition has a spreading of the mean error of about ±2% in contrast to ±5% for the weighted quadratic superposition. Likewise the confidence intervals of ±1.8% around the mean error are smaller in relation to the value of ±2% for the other approach.

The larger errors of the weighted superposition probably accumulate from the error in the estimation of the zeroth and especially of the second spectral moment of the aerodynamic response (7.2). Hofland [7.1] estimated confidence intervals for the standard deviation between 3% and 8% and for the zero crossing period between 8% to 10% depending on the simulation length.

Overall evaluation

A benchmark exercise of one particular design proved both formulas for the superposition of separate fatigue analyses robust and accurate enough for simplified approaches. In practice, errors are likely to be significantly higher, say in the order of ±10% due to application of different tools for the aerodynamic and hydrodynamic fatigue analyses, rougher approximation of the aerodynamic damping and execution as an engineering approach. Therefore a conservative damping assumption should be used.

The relatively low sensitivity of the fatigue loads might be specific to designs with a high degree of aerodynamic damping such as the pitch regulated reference design. The weighted quadratic superposition, (7.17), is founded on a more stable basis by application of the Hancock formula, (7.14), which at least empirically accounts for a broad bandwidth. The numerical validation showed, however, lower accuracy compared to the very simple direct quadratic superposition which was derived by ignoring the considerable bandwidth of the simultaneous wind and wave response. Further investigations of both formulas for instance with different reference designs should be carried out. For the time being, the direct quadratic superposition is preferred.
Figure 7.24: Relative difference between wave induced equivalent fatigue loads computed in frequency domain and time domain

Figure 7.25: Relative error of superposition of short-term equivalent fatigue loads from separate analyses with respect to integrated time domain simulation of the simultaneous response

Solid: error when using the average of 9 x 40 min of aerodynamic response
Dashed: 95% confidence interval for using only 1 x 10 min per load case
Stars: Individual error from using 1 x 10 min per load case
Figure 7.26: Relative error of long-term equivalent fatigue loads along the structure (accumulated from superposition of separate short-term fatigue loads) with respect to integrated time domain simulations of the simultaneous response:

Solid: Error when using the average of 9 x 40 min simulations per load case
Dashed: 95% confidence interval for using 1 x 10 min simulation per load case

All required quantities of the two expressions are available in the frequency domain or can be estimated with Eqn. (7.2) in the time domain. This allows application of the preferred approaches for fatigue analysis in offshore technology and wind energy, respectively.

In the next chapter the two expressions are implemented the simplified analyses approaches for the simultaneous wind and wave fatigue covering the entire range from the environmental and structural modelling to response analysis.
CHAPTER 8

SIMPLIFIED ANALYSIS OF AERODYNAMIC AND HYDRODYNAMIC FATIGUE

The previous chapter has established the theoretical and empirical basis for both the separation of the simultaneous response of an offshore wind energy converter into an aerodynamic and hydrodynamic contribution and the successive superposition of the aerodynamic and hydrodynamic damage equivalent fatigue loads. We have restricted the discussion to stationary Gaussian processes, representing short-term load cases, because only under this condition spectral superposition is valid. In the present chapter the previous findings are applied to propose simplified analysis approaches for the long-term fatigue due to many load cases.

In Sections 8.1 and 8.2 we introduce each one approach for the simplified fatigue analysis. The difference lies in the superposition based on either the short-term or the long-term fatigue loads. The treatments fit well to the requirements of the early design process since they are relatively simple and are based on the application of standard design tools for the time-domain simulation of the wind response and the linear spectral analysis of the wave response (Figure 8.1). For the certification level still integrated, non-linear time domain simulations of the simultaneous response are preferred, however, the scattering of wind and wave parameters results in too many load cases from a practical point of view. Therefore

- non-linear time domain simulation of wind response
- wind turbine and support structure in a calm sea
- linear spectral analysis of wave response
- support structure with top mass
- additional structural damping = 80% aerodynamic damping

Figure 8.1: Simplified analysis approach for aerodynamic and hydrodynamic fatigue of the support structure
Section 8.3, by using the method from Section 8.1, establishes an approach for constructing a low number of so-called ‘lumped’ load cases (Section 6.3.2) which approximate the fatigue deterioration due to the complete environmental characteristics. In addition to the random, stationary load cases, Section 8.3 constitutes also transition and fault load cases which should be included in a certification calculation. The simplified approaches introduced in Section 8.1 and 8.2 skip such occurrences because in most cases their contribution to the fatigue damage is moderate and integrated time domain analyses are required for instationary situations. Finally, Section 8.4 compares the three approaches introduced in the previous sections for a reference example.

8.1 Simplified analysis based on superposition of short-term fatigue loads

A straightforward and well-based solution of the long-term fatigue is to superimpose the aerodynamic and hydrodynamic fatigue loads associated with the same load case and to accumulate successively the long-term fatigue from the superimposed short-term loads.

Figure 8.2 shows the five steps of this approach which are illustrated at the example of the soft-soft Opti-OWECS monopile configuration described in Section 9.1.

Step 1.: Establishment of environmental conditions

For a simplified analysis it is generally sufficient to consider collinear and omnidirectional wind and waves. Consequently the environmental conditions can be described by a three-dimensional scatter diagram with classes for the significant wave height $H_s$, zero-crossing period $T_z$ and mean wind speed at hub height $V$. In many situations of an early design stage no correlated wind and wave data or not even a $H_s - T_z$ scatter diagram of the site might be available and one has to accept one of the simplified environmental models introduced in Sections 10.1.1 and 10.2.1.

In the present example a three-dimensional scatter diagram (Table 8.1) is constructed for classes of $\Delta H_s = 0.5 \text{ m}$, $\Delta T_z = 0.5 \text{ s}$ and $\Delta V = 2 \text{ m/s}$ by evaluation of the long-term NESS database [8.1]. From the theoretically possible 990 combinations of the 10 wave height, 9 wave period and 11 wind speed classes only 87 load cases have a probability of 0.5 % and higher.

In the following a distinction between long-term and short-term quantities is introduced by marking the latter with the indices $i$, $j$ and $k$ of the corresponding wave height, wave period and wind speed class.

The consideration of the operational modes of the OWEC is important because they determine whether or not aerodynamic damping has to be accounted for in the calculation of the hydrodynamic response associated with winds between the cut-in and the cut-out wind speed. So in principle two scatter diagrams are required for the production wind speed range: one with the load cases probabilities during actual production and another for failure and repair state. Establishment of such load case probabilities is far from easy. A non-uniform distribution of the down-time can be expected since the access for maintenance is hindered during periods with strong winds, high waves or sea ice. Evaluation of Monte-Carlo simulations of the operation and maintenance behaviour of the entire wind farm [8.2] can derive a proper estimation. In this chapter we consider a uniform distribution.
A conservative treatment must consider the worst result of two analyses with different assumed availability. The assumption of the minimum expected availability within the wind farm is necessary for design configurations and hot spots, for which the response during shut-down, i.e. mainly hydrodynamic loading without aerodynamic damping, exceeds the combined aerodynamic and hydrodynamic response with aerodynamic damping during power production at the same environmental conditions. In case of a dominating aerodynamic loading and for locations closer to the tower top, the maximum availability, say for convenience 100%, results in the highest loading.

**Figure 8.2:** Simplified analysis based upon superposition of short-term equivalent fatigue loads
Table 8.1: Step 1. of simplified analysis: Three dimensional scatter diagram for the NL-1 site and 100% availability

Cell notation: wind speed class at hub height [m/s] : probability [parts per thousands]

<table>
<thead>
<tr>
<th>( H_z )</th>
<th>( T_z )</th>
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<tbody>
<tr>
<td>4.75 m</td>
<td>&gt;23: 1</td>
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<tr>
<td>4.25 m</td>
<td>22: 1</td>
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<tr>
<td>2.5 m ≤ ( H_z ) &lt; 3 m</td>
<td>&gt;23: 2</td>
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<td>5 s ≤ ( T_z ) &lt; 5.5 s</td>
<td>13 m/s ≤ ( V ) &lt; 15 m/s; ( p = 0.2% )</td>
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<td></td>
<td>15 m/s ≤ ( V ) &lt; 17 m/s; ( p = 1.0% )</td>
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<td></td>
<td>17 m/s ≤ ( V ) &lt; 19 m/s; ( p = 1.3% )</td>
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<td>19 m/s ≤ ( V ) &lt; 21 m/s; ( p = 0.5% )</td>
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<td>21 m/s ≤ ( V ) &lt; 23 m/s; ( p = 0.1% )</td>
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<td>8: 8</td>
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<tr>
<td>0.25 m</td>
<td>Σ = 986</td>
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</table>

\( H_z \) \( T_z \) | 2.75 s | 3.25 s | 3.75 s | 4.25 s | 4.75 s | 5.25 s | 5.75 s | 6.25 s | 6.75 s
Figure 8.3: Step 2. of simplified analysis: Results of the aerodynamic fatigue analysis against wind speed class with unit probability

Step 2.: Aerodynamic fatigue analysis for each wind speed load case
The aerodynamic fatigue response is analysed separately from the hydrodynamics by means of a standard wind turbine design tool. The aerodynamic response of the OWEC in a calm sea is computed by non-linear time domain simulations (Figures 8.1 and 8.2, left). After Rainflow counting of the signals the equivalent stress ranges are determined according to Eqn. (7.6) for a unity probability of each load case. Only if one applies the weighted quadratic superposition, Eqn (7.17), the zeroth and the second spectral moments, $m_0$ and $m_2$, are estimated from the standard deviation and the counted zero-crossing period with respect to the signal mean by means of Eqn. (7.2). Figure 8.3 shows a nearly proportional increase of the equivalent stress range and the spectral moments against the wind speed. In the vicinity of the rated wind speed some irregularities occur due to the switching of the pitch controller. Aerodynamic loading outside the production wind speed range and direct wind loading of the support structure do not give a significant contribution and are therefore neglected (Section 9.6.1).

Step 3.a: Determination of hydrodynamic transfer functions
The aerodynamic damping during the operation of the wind turbine is considered as additional structural damping of the fundamental fore-aft mode of the support structure during the calculation of the hydrodynamic transfer functions. In Section 7.3 it has been recommended to reduce the estimated aerodynamic damping by 20% to achieve more conservative results.
First the dependency of the aerodynamic damping from the wind speed classes, $V_w$, established in Step 1., is investigated. Either numerical linearisation or transient vibration analysis in steady inflow are recommended (Section 7.1.3). Depending on the observed variation of the damping, which is a consequence of the employed wind
turbine and support structure, damping classes, $\xi$, are established. At least two classes are needed for the situation without any aerodynamic damping and with an average aerodynamic damping. Generally damping classes of 0.5% to 1% of critical damping provide a more accurate model. For each damping class a specific set of hydrodynamic transfer functions is derived. In the example case the aerodynamic damping is rounded in steps of 1% and 0.5% for values greater to 2% and below 2%, respectively. Successively only 80% of the rounded values are considered (Table 8.2). By this the 11 wind speed classes are associated with six damping classes for which the hydrodynamic stress transfer functions are

<table>
<thead>
<tr>
<th>Wind speed class</th>
<th>Estimated aerodynamic damping $\xi (V_p)$ [-]</th>
<th>Additional structural damping class $\xi$, [-]</th>
<th>index $k$</th>
</tr>
</thead>
<tbody>
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<td>$V_p$ [m/s]</td>
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<td>&lt; cut-in or unav.</td>
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<td>0.0%</td>
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<td>22</td>
<td>10</td>
<td>1.0%</td>
<td>0.8%</td>
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</tbody>
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> cut-out or unav. | 11                                          | 0.0%                                       | 0.0%    | 1       |

**Table 8.2: Step 3.a of simplified analysis: Establishment of damping classes**

![Diagram](image)

**Figure 8.4: Step 3.a of simplified analysis: Hydrodynamic transfer functions from wave height to stress range with different values of additional structural damping (Damping without aerodynamic damping equals 0.5%).**
derived by time domain simulation of the steady state response (Figure 8.4). Depending on the available tools also a frequency domain analysis is suitable.

**Step 3.b:** Hydrodynamic fatigue analysis for each wave load case
The equivalent stress range and if required the zeroth and second spectral moments are calculated for each load case by linear spectral analysis (Section 6.2.2). Again unit probabilities are considered. The correct transfer functions are chosen according to the relation between mean wind speed and damping class from Table 8.2. This treatment results in the repetition of some identical calculations if for a certain set of $H_s$ and $T_z$ the same damping class is associated with different wind speed classes. Nonetheless, the frequency domain approach is so rapid that this hardly affects the computational time.

Figures 8.5 and 8.6 illustrate the entire trajectory of the fatigue analysis process for two sea states and different operation conditions at the critical cross section 5.5 m below the mudline.

The spectral analysis is shown in three left subplots. For the light sea state (Figure 8.5) the spectral peak of the Pierson-Moskowitz spectrum (top-left) lies close to the resonance of the transfer function (middle-left) and the response spectrum (bottom-left) is dominated by the dynamic response.

The right three subplots illustrate the fatigue calculation. The histogram of the stress ranges (top-right) is computed with the Dirlik formula, (7.10), from the spectral moments of the response power spectrum. Applying the Palmgren-Miner rule, (7.4), the damage can be determined from the stress range histogram and the S-N curve. The horizontal distance between the graph of the histogram and the S-N curve is a qualitative illustration for the lifetime. Cumulative range distributions (middle-right) are more commonly used. Apparently only the stress magnitude rather than the number of stress cycles is affected by the damping. The equivalent stress range (bottom-right), (7.6), equals a rectangular cumulative distribution owing to the constant range with $2 \cdot 10^6$ cycles. Without aerodynamic damping the equivalent stress range is 2.8 times higher corresponding to a 98% shorter lifetime.

The severe sea states in Figure 8.6 includes 19 times more energy which is proportional to the square of the significant wave height. The lower frequency contents results, however, in a lower dynamic amplification and a larger contribution of the quasi-static response especially in case of presence of aerodynamic damping. The difference between production and parking is much lower than in the previous example because for this particular wind turbine design only few aerodynamic damping is observed at high wind speeds.

**Step 4:** Superposition of short-term fatigue loads
The equivalent stress range of the combined fatigue for each load case, $\Delta \sigma_{eq,an,i,k}$, is composed from the direct quadratic superposition, (7.18), or the weighted quadratic superposition, (7.17). Application of the direct quadratic superposition is simpler and probably less scatter of the results occur. The equivalent stress ranges are related to a unit probability and are marked with a corresponding subscript, $p = 1$. 

Figure 8.5: Spectral fatigue analysis of a light sea state with unity probability
(Dashed: production at 8 m/s with 4% of additional structural damping
Solid: parked with only 0.5% actual structural damping)

Step 5.: Calculation of long-term fatigue loads
The actual probability of the individual load cases, $p_{i,j,k}$, is considered during the summation of the long-term aerodynamic and hydrodynamic fatigue response, $\Delta \sigma_{eq,ah}$.

$$\Delta \sigma_{eq,ah} = \frac{\mu}{\sum_i \sum_j \sum_k \Delta \sigma_{eq,ah,i,j,k}^{\mu} p_{i,j,k} |^{p-1}} \quad (8.1)$$

The total damage from all load cases, $D_{ah}$, can be calculated from the equivalent stress range associated with the long-term fatigue, (8.2).

$$D_{ah} = \frac{N_R}{N(\Delta \sigma_{eq,ah})} \quad (8.2)$$

Alternatively the damages of the individual load cases are summed up, (8.3).

$$D_{ah} = \sum_i \sum_j \sum_k \frac{N_R p_{i,j,k}}{N(\Delta \sigma_{eq,ah,i,j,k}^{\mu} |^{p-1})} \quad (8.3)$$
Figure 8.6: Spectral fatigue analysis of a severe sea state with unity probability
(Dashed: production at 16 m/s with 2.4% of additional structural damping
Solid: parked with only 0.5% actual structural damping)

In both cases the number of endured cycles $N(\Delta \sigma)$ is obtained from the S-N curve, (7.3). In the example case the hydrodynamic response without aerodynamic damping is more severe for cross sections below the mudline than the combined aerodynamic and hydrodynamic response during power production. Consequently the fatigue damage below the mudline is higher if the OWEC availability is reduced. Figure 8.7 presents the distribution of the relative damage summed over the considered wind speed classes for the two cases of 100% and 85% availability. In the former case the damages from the categories ‘parking below cut-in and above cut-out wind’ and ‘production at 100% availability’ are summed up. In the latter case a 50% higher total damage is suffered due to the conditions ‘parking below cut-in and above cut-out wind’, ‘parking at 15% of production time (85% availability)’ and ‘production at 85% of production time (85% availability)’. For most wind speed classes in the production range the damage during the 15% of time without aerodynamic damping is higher than during the remaining fraction of actual production. The importance of a proper description of the availability is emphasized.
8.2 Simplified analysis based on superposition of long-term fatigue loads

In an early design situation and for parametric studies it would be advantageous to use a further simplified fatigue approach than introduced in the previous section. Empirical studies have proven that this can be achieved with a reasonable accuracy by the direct superposition of the long-term equivalent fatigue loads associated with aerodynamic and hydrodynamic response.

Figure 8.8 explains the four steps. The main difference with the previous approach lies in the reverse order of the summation of the long-term fatigue, done now separately for wind and waves in Step 2. and 3.b, and the superposition of aerodynamic and hydrodynamic fatigue, carried out in Step 4. Consequently, a fifth step is not necessary.

Step 1.: Establishment of environmental conditions
The simplified description of the aerodynamic damping is another distinction. Instead of several damping classes, now only two situations are considered: the idle, shut-down or failure state without any aerodynamic damping, hereafter called 'parking', and the production state with assumption of an average value of aerodynamic damping.

Therefore only wave criteria for the parking state rather than a detailed wind-wave correlation are required. If only separate wind and wave data are available, the cut-in and cut-out wind speeds can be transformed into corresponding wave heights under the
Figure 8.8: Simplified analysis based upon superposition of long-term equivalent fatigue loads by direct quadratic superposition assumption of a monotonic wind speed - wave height relation. Correlating wind speeds and wave heights with the same cumulative probabilities can derive such a relation.

As long as it is unknown whether aerodynamic or hydrodynamic response dominates for all members of the structure one has to evaluate both a minimum and a maximum availability (see first step of the previous method).

Next the probabilities of the separated aerodynamic and hydrodynamic load cases for the two operational modes are established by evaluating the Weibull wind speed distribution and a two-dimensional wave scatter diagram.

Again both a minimum and a maximum availability has to be taken into account for designs with potentially dominant hydrodynamic response.
Step 2: Analysis of long-term aerodynamic fatigue
In most cases it is reasonable to consider fatigue only when the rotor is operating in the production wind speed range. Below the cut-in wind, fatigue is absolutely negligible and above the cut-out wind speed the hydrodynamic loads are dominating. Therefore we calculate only the aerodynamic fatigue response during production. A standard wind turbine design tool is used for the computation of the equivalent stress range of the aerodynamic fatigue. We do not need the spectral moments because Step 4 applies the direct quadratic superposition of the fatigue loads, (7.18). The equivalent stress range, \( \Delta \sigma_{eq, a, prod} \), is summed from the contributions of the individual load cases, (8.4).

\[
\Delta \sigma_{eq, a, prod} = \sqrt{\sum_k \Delta \sigma_{eq, a, prod, k}^{\mu} p_k}
\]

(8.4)

It can also be calculated directly from the long-term stress range distribution, (8.5).

\[
\Delta \sigma_{eq, a, prod} = \sqrt{\frac{1}{N_R} \sum_j \Delta \sigma_{a, prod, j}^{\mu} n_j(\Delta \sigma_j)}
\]

(8.5)

Step 3.a: Estimation of the effective aerodynamic damping and determination of hydrodynamic transfer functions
Estimation of an average or effective aerodynamic damping for the production load cases is no straightforward task; even if the variation of the damping with the wind speed is known. Generally sea states with higher waves but relatively low probability of occurrence contribute significantly to the long-term fatigue damage. Such conditions correspond to high wind speeds above the rated wind speed for which the aerodynamic damping might be lower. Figure 8.9 correlates the additional structural damping and the damage as functions of the wind speed. In this case the damage distribution was available from the previous section. Once such a distribution of the hydrodynamic damage on the wind speed classes is known, at least approximately, a reasonable estimate of the effective aerodynamic damping \( \xi_{aero, prod} \) is gained by weighting the aerodynamic damping of the different wind speed classes \( \xi_{aero}(V_j) \) by the relative contribution of the damage \( D(V_k) \) of the associated wind speed classes.

\[
\xi_{aero, prod} = \frac{\sum_k \xi_{aero}(V_k) D(V_k)}{\sum_k D(V_k)}
\]

(8.6)

The aerodynamic damping may be estimated from the closed-form expression, Eqn. (5.12). However, care is required for the influence of the controller dynamics and for stall regulated design where the aerodynamic drag is significant at higher winds. Only two sets of hydrodynamic stress transfer functions corresponding to both damping classes are required.
Figure 8.9: Establishment of effective damping during production mode:
Additional structural damping against wind speed (upper),
Relative hydrodynamic damage against wind speed (lower)
(Effective additional damping equals 2%).

Step 3.b: Analysis of long-term hydrodynamic fatigue

Hydrodynamic fatigue can be significant during production as well as parking. In Step 2, we have assumed that aerodynamic fatigue is negligible during parking. Therefore the total fatigue for this operating state, $\Delta \sigma_{eq,ah,parking}$, equals the hydrodynamic fatigue response during parking, $\Delta \sigma_{eq,h,parking}$:

$$\Delta \sigma_{eq,ah,parking} := \Delta \sigma_{eq,h,parking} \quad (8.7)$$

The long-term equivalent stress ranges for the parking and the production state are calculated separately from spectral fatigue analyses of the concerned wave load cases. The probability of the production load cases it adjusted by the assumed maximum or minimum probability. Either the long-term, equivalent stress ranges are summed from the individual load cases or they are calculated directly from the long-term stress range distribution analogous to the aerodynamic fatigue.

Step 4: Superposition of aerodynamic and hydrodynamic fatigue

Now we can superimpose the long-term fatigue loading during production by the direct quadratic superposition of the aerodynamic and hydrodynamic equivalent stress range, Eqn. (7.18).

$$\Delta \sigma_{eq,ah,prod} = \sqrt{\Delta \sigma_{eq,a,prod}^2 + \Delta \sigma_{eq,h,prod}^2} \quad (8.8)$$
A straightforward combination of the fatigue during production and during parking would be the summation of the associated damages. Dealing with the damage equivalent stress ranges from (8.7) and (8.8) we have to exponentiate them with the inverse slope of the S-N curve before summing up.

\[
\Delta\sigma_{eq,an} = \sqrt[\mu]{\Delta\sigma_{eq,ah,parking}^{\mu} + \Delta\sigma_{eq,ah,prod}^{\mu}} \quad (8.9)
\]

The total damage can be obtained by (8.2).

### 8.3 Establishment of load cases for integrated time domain simulations

For an offshore wind energy converter at least an order of magnitude more fatigue load cases exist than for an onshore wind energy converter. The scatter diagram in Table 8.1, for instance, comprises 87 sea states. Later in this chapter we show that it is more accurate to deal also with of some rare but severe load cases which are not included in this scatter diagram. They rise the total number of sea states to 145 or 235 depending on the width of the wind speed classes. Consideration of the different operational states of the turbine results in approximately the double number and taking directional effects into account would increases the number of load cases by another order of magnitude. Evidently hundreds of fatigue load cases are incompatible with integrated time domain simulations and establishment of a low number of characteristic load cases for certification calculations (Section 10.3) is required.

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**Figure 8.10:** Lumpung of load cases by superposition of short-term fatigue loads

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1 In contrast to the discussion on the combination of the fatigue of two simultaneous processes in Section 7.2.1, a summation of damages is valid here because the processes are occurring sequentially.
A lumping of load cases is well known in offshore technology and is done most times under the assumption of quasi-static wave response [8.3]. In the present case the problem is more involved since dynamic response is important and offshore wind energy converters suffer both significant wind and wave fatigue. The simplified method introduced in Section 8.1 offers a possibility for an effective and accurate solution. The approach has already been mentioned briefly in Section 6.3.2 but is now explained in full detail (Figure 8.10).

In addition to the lumped load cases, which consider only random, stationary production and parking conditions, we establish a number of transition and fault situations at the end of this section.

Step 1: Establishment of environmental conditions
A three-dimensional scatter diagram for significant wave height, $H_s$, zero-crossing period, $T_z$, and mean wind speed at hub height, $V$, is required as starting point. The corresponding load cases are denoted here as 'elementary load cases' and the parameters, $H_{s,i}$, $T_{z,j}$, $V_k$, are marked by indices $i$, $j$ and $k$. If directional effects are significant, the entire procedure should be applied for each directional sector (Section 9.6.1, Table B.2).

Step 2: Damage calculation of the elementary load cases
The damage of every elementary load case due to wind and wave loading is determined with the approach from Section 8.1.

Step 3: Preliminary lumping of sea states
Next a first lumping of load cases is carried out. Criteria for lumping of sea states can be to achieve approximately the same damage for all load cases or lumping of all sea states associated with a certain wind speed class or a combination of both.

No exact procedure for the lumping process is known. Instead here a practical treatment is proposed which includes some simplifications and arbitrary elements. The number of lumped load cases, typically about 20, should not be chosen too small. Good experience was gained by maintaining most of the wind speed classes with class width of 1 m/s within the production range but by using broader wind speed classes near the cut-in and cut-out wind speeds.

The process of lumping itself consists of a weighting of the environmental parameters of the elementary load cases by their relative damage. Ideally the $H_s-T_z-V$ set of a lumped load case lies on the centre of damage of the considered elementary load cases. In fact only the damage at certain values of the environmental parameters rather than the quite non-linear damage distribution between the discrete parameters is known. So, an iterative procedure has to be followed. A reasonable first estimate or prediction step of the lumped parameters can be gained under the simplified assumptions of Eqn. (8.10), which are derived from the quasi-static response.

\[
D \propto \Delta \sigma^\mu \propto H_s^\mu \\
D \propto n_{\text{total}} \propto \frac{1}{T_z} \\
D \propto \Delta \sigma^\mu \propto V^\mu
\]  

(8.10)
Assuming a linear response, the stress ranges are considered proportional to the significant wave height as well as the standard deviation of the wind speed and thus mean wind speed, presuming constant turbulence intensity. The number of stress cycles is of lower influence on the damage than the stress range. Therefore we suppose the number of stress cycles to vary with the inverse zero-crossing period which holds, strictly only for a quasi-static response. The damage itself is proportional to the \( \mu \)th power of the stress ranges and the total number of cycles, \( n_{\text{total}} \).

Now the first estimate for the lumped sea state parameters, \( \hat{H}_{s,i}^{(n)}, \hat{f}_z^{(n)}, \hat{V}_l^{(n)} \) where \( n = 1 \), is calculated as the centre of damage within the set of the elementary load cases, \( S_i \). The new index \( l \) refers to the number of the lumped load case while the superscript, \( n \), of the load case parameters denotes the iteration stage.

\[
\hat{H}_{s,i}^{(1)} = \mu \left[ \frac{\sum_i \sum_j \sum_k H_{s,i}^\mu P_{ijk}}{\sum_i \sum_j \sum_k P_{ijk}} \right]_{(i,j,k) \in S_i} \equiv \frac{1}{\hat{f}_z^{(1)}} = \frac{\sum_i \sum_j \sum_k P_{ijk}}{\sum_i \sum_j \sum_k \hat{P}_{ijk}} \left[ \frac{\sum_i \sum_j \sum_k P_{ijk}}{\sum_i \sum_j \sum_k \hat{P}_{ijk}} \right]_{(i,j,k) \in S_i} \tag{8.11}
\]

\[
\hat{V}_l^{(1)} = \sqrt{\mu \left[ \frac{\sum_i \sum_j \sum_k \hat{V}_l^\mu P_{ijk}}{\sum_i \sum_j \sum_k P_{ijk}} \right]_{(i,j,k) \in S_i}}
\]

**Step 4.** **Damage calculation of the lumped load cases**

The aerodynamic and hydrodynamic fatigue associated with the lumped load cases is calculated separately and the combined fatigue is superimposed for each load case. The required parameters of the aerodynamic fatigue can be interpolated from the results of Step 2. if the mean wind speed of some load case has been altered. This avoids a cumbersome new time domain simulations.

**Step 5.** **Refined lumping of load cases**

The damage of the lumped load cases will not match exactly the summed damage associated with the elementary load cases owing to the simplifications in Eqn. (8.10). The corrections, (8.12) and (8.13), of the lumped significant wave height and inverse zero-crossing period with the same factor, \( v_D, (8.14) \), should be suitable if the wind speed of the load cases is not modified.

\[
\hat{H}_{s}^{(n+1)} = v_D \hat{H}_{s}^{(n)} \tag{8.12}
\]

\[
\hat{f}_z^{(n+1)} = \frac{\hat{f}_z^{(n)}}{v_D} \tag{8.13}
\]

The correction, \( v_D \), is also based upon the simplified, quasi-static relations from Eqn. (8.10). Approximately the \( \mu(\mu+1) \)th fraction of the error of the damage of the lumped load
cases is corrected by changing the significant wave height while the remaining \(1/(\mu+1)\)th part of the error is compensated by the variation of the zero crossing period.\(^2\)

\[
\nu_{D} = \sqrt{\sum_i \sum_j \sum_k D(H_s,i, T_z,j, V_k) \left|_{(i,j,k) \in S_i} \right. D\left(\tilde{H}_s^{(n)}, \tilde{T}_z^{(n)}, \tilde{V}^{(n)}\right)}
\]

(8.14)

In some cases with dominant dynamic response owing to a zero-crossing period, \(T_z\), the vicinity of the fundamental eigenfrequency, the iteration may not converge. Then only the significant wave height should be corrected by (8.12). Consequently \(\nu_p\) is calculated by applying the \(\mu\)th root in (8.14).

**Step 6.: Re-calculation of combined fatigue and check for different cross sections**

Similar to Step 4., the combined fatigue is re-computed for the refined lumped load cases. The error of the damage should be checked at cross sections with different ratio between the aerodynamic and the hydrodynamic fatigue. The procedure is repeated from Step 3. or Step 5. if the deviation between the damage associated with the elementary load cases and the lumped load cases is not acceptable.

**Example of load case lumping**

The NESS database enables us to consider a minimum probability of 0.5\(^9\)/100 and classes of \(\Delta H_s = 0.5\) m, \(\Delta T_z = 0.5\) s and \(\Delta V = 1\) m/s which results in 235 sea states. This number enlarges to 440 elementary load cases when parking of the turbine within the production wind speed range is taken into account. As illustration Figure 6.7 presents the scattering of all 3-hour sea states included in the NESS database for a period of 9 years.

By following Step 2. to 6. we can established 21 lumped sea states according to Table 8.3. From Figure 6.7 is can be seen that these load cases are covering very well the spreading of the actual conditions. If a reduced availability is considered, the load cases within the production wind speed range, i.e. No. 3 to 19, have to be simulated for production as well as parking and a total number of 37 load cases exists.

Table B.2 in Appendix B presents lumped load cases for directional sectors with a width of 30°. Opposite sectors are combined, so, only six sectors must be evaluated.

The approximately 10 times lower number of integrated time domain simulations, which remain after the sea state lumping, reduces the computational time accordingly. The effort for the entire lumping procedure depends mainly on the manual operations conveniently done with a spreadsheet during Step 1., 3. and 5. These operations may take one hour in total. The damage calculations in Step 2., 4. and 6. require only some minutes if the aerodynamic fatigue loads are provided and tools for linear spectral analysis and the superposition of fatigue loads exist.

**Establishment of transition and fault load cases**

So far we have only considered fatigue due to stationary, random aerodynamic and hydrodynamic loading during normal production or parking. Fatigue is also caused by transition between operating conditions, e.g. start-up, shut-down, switching between the

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\(^2\) Assuming an inverse slope of \(\mu = 4\) and an error of 10% then a correction of both \(H_s\) and \(T_z\) by approximately 2%, i.e. \(\nu_p = 1.019\), is sufficient. The damage is adjusted by a factor of 1.019\(^4\) = 1.079 due to the modification of \(H_s\) and by a factor of 1.019 owing to the new zero-crossing period.
### Table 8.3: Lumped load cases for the Opti-OWECS design at the NL-1 site for 100% availability

<table>
<thead>
<tr>
<th>No.</th>
<th>$H_s$ [m]</th>
<th>$T_z$ [s]</th>
<th>$V_{th}$ [m/s]</th>
<th>$I_f$ [%]</th>
<th>Probability [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.48</td>
<td>3.57</td>
<td>&lt; 5</td>
<td>none</td>
<td>66</td>
</tr>
<tr>
<td>2</td>
<td>0.80</td>
<td>4.50</td>
<td>&lt; 5</td>
<td>none</td>
<td>63</td>
</tr>
<tr>
<td>3</td>
<td>0.72</td>
<td>3.88</td>
<td>6.1</td>
<td>11.8</td>
<td>158</td>
</tr>
<tr>
<td>4</td>
<td>0.83</td>
<td>3.88</td>
<td>7.5</td>
<td>11.4</td>
<td>88</td>
</tr>
<tr>
<td>5</td>
<td>0.99</td>
<td>4.16</td>
<td>8.5</td>
<td>11.2</td>
<td>95</td>
</tr>
<tr>
<td>6</td>
<td>1.10</td>
<td>4.23</td>
<td>9.5</td>
<td>10.9</td>
<td>90</td>
</tr>
<tr>
<td>7</td>
<td>1.27</td>
<td>4.39</td>
<td>10.5</td>
<td>10.7</td>
<td>91</td>
</tr>
<tr>
<td>8</td>
<td>1.45</td>
<td>4.55</td>
<td>11.5</td>
<td>10.6</td>
<td>71</td>
</tr>
<tr>
<td>9</td>
<td>1.62</td>
<td>4.68</td>
<td>12.5</td>
<td>10.4</td>
<td>58</td>
</tr>
<tr>
<td>10</td>
<td>1.83</td>
<td>4.82</td>
<td>13.5</td>
<td>10.2</td>
<td>47</td>
</tr>
<tr>
<td>11</td>
<td>2.03</td>
<td>4.94</td>
<td>14.5</td>
<td>9.9</td>
<td>41</td>
</tr>
<tr>
<td>12</td>
<td>2.23</td>
<td>5.05</td>
<td>15.5</td>
<td>9.6</td>
<td>35</td>
</tr>
<tr>
<td>13</td>
<td>2.47</td>
<td>5.22</td>
<td>16.5</td>
<td>9.4</td>
<td>25</td>
</tr>
<tr>
<td>14</td>
<td>2.68</td>
<td>5.34</td>
<td>17.5</td>
<td>9.2</td>
<td>21</td>
</tr>
<tr>
<td>15</td>
<td>2.89</td>
<td>5.48</td>
<td>18.5</td>
<td>9.1</td>
<td>14</td>
</tr>
<tr>
<td>16</td>
<td>3.15</td>
<td>5.66</td>
<td>19.5</td>
<td>9.0</td>
<td>11</td>
</tr>
<tr>
<td>17</td>
<td>3.40</td>
<td>5.84</td>
<td>20.5</td>
<td>8.9</td>
<td>9</td>
</tr>
<tr>
<td>18</td>
<td>3.64</td>
<td>5.97</td>
<td>21.5</td>
<td>8.8</td>
<td>6</td>
</tr>
<tr>
<td>19</td>
<td>3.84</td>
<td>6.11</td>
<td>22.5</td>
<td>8.7</td>
<td>4</td>
</tr>
<tr>
<td>20</td>
<td>3.87</td>
<td>6.03</td>
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<td>4</td>
</tr>
<tr>
<td>21</td>
<td>5.19</td>
<td>6.98</td>
<td>&gt; 23 †</td>
<td>none</td>
<td>2</td>
</tr>
<tr>
<td>Sum</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>999</td>
</tr>
</tbody>
</table>

$V_{th} =$ hourly mean wind speed at 60 m height  
$I_f =$ turbulence intensity for a turbine spacing of 10 diameters (Figure 4.7)  
† The assumed shut-down condition for the WTS 80M is either a 3 s gust of 30 m/s or higher or a 10 minute mean wind speed greater than 25 m/s. From a statistical point of view both cases correspond to a hourly mean wind speed of 23 m/s at hub height if hysteresis effects are accounted for. If turbulent wind loads are considered during parking one may assume for these load cases a mean wind speed of 0.7 $V_{th}$ in accordance with IEC 61400-1 ed.2.

### Table 8.4: Transition load cases for the Opti-OWECS design at the NL-1 site

<table>
<thead>
<tr>
<th>Load case</th>
<th>$H_s$ [m]</th>
<th>$T_z$ [s]</th>
<th>$V_{10\text{min}}$ [m/s]</th>
<th>Probability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Start-up &amp; shut-down at cut-in wind</td>
<td>0.66</td>
<td>3.44</td>
<td>5</td>
<td>1000 times/year</td>
</tr>
<tr>
<td>Start-up &amp; shut-down at cut-out wind</td>
<td>4.79</td>
<td>6.38</td>
<td>25</td>
<td>50 times/year</td>
</tr>
</tbody>
</table>
rotor speeds of double speed designs and passing of the tower resonance at soft-soft design with variable rotor speed. Likewise fault conditions could be of importance which do not directly initiate a shut-down of the turbine. A typical example includes a malfunction of the wind vane and associated longer operation with a significant misalignment between the wind direction and the rotor axis. Manoeuvres with short duration as emergency shut-down, extreme gusts or wind shears are commonly only considered in the ultimate limit state rather than fatigue analysis [8.4].

Establishment of any of such load cases depends on the particular design. The sea state parameters can be derived from the mean conditions taken from the NESS database. Germanischer Lloyd recommends in its OWECS guidelines [8.5] to take into account start-up and shut-down procedure with a frequency of 1000 times per year at cut-in wind and 50 times per year at cut-out wind both combined with a corresponding sea state (Table 8.4). Section 9.6.1 demonstrate only a minor contribution of these load cases to the support structure fatigue.

The situation of power production with a large yaw misalignment is very unlikely for the WTS 80M turbine which is equipped with two independent wind vanes and an extensive remote monitoring and control system. Therefore the load case is not further considered here.

### 8.4 Parameter study

Some practical recommendations and an impression on the accuracy of the three proposed approaches are gained by a parameter study for soft-soft Opti-OWECS monopile configuration. In accordance with Section 9.6.1 the critical cross section of the monopile 5.5 m below mudline and a location 10.6 m underneath of tower top flange are analysed. As reference we consider the long-term fatigue for 85% availability which is extrapolated from integrated, non-linear time domain simulations of 440 elementary load cases with a duration of 40 minutes each. Again a inverse slope of the S-N curve of 4 is applied.

Figure 8.11 presents the error of the damage equivalent stress ranges from the short-term superposition of fatigue from Section 8.1 for four cases:

- **Short-term, direct quadratic superposition of 160 load cases** corresponding to a minimum load case probability of 0.5\%\textsubscript{100} and classes of $\Delta H_s = 0.5$ m, $\Delta T_z = 0.5$ s and $\Delta V = 2$ m/s (see scatter diagram in Table 8.1). The total probability account for 98.6% of the conditions within the NESS database.
- **Short-term, direct quadratic superposition of 260 load cases** as above but the minimum probability equals 0.5\%\textsubscript{100} which covers 99.9% of all sea states included in the database
- **Short-term, weighted quadratic superposition of the 260 load cases**
- **Short-term, direct quadratic superposition of 440 load cases** corresponding to a minimum load case probability of 0.5\%\textsubscript{100} and classes of $\Delta H_s = 0.5$ m, $\Delta T_z = 0.5$ s and $\Delta V = 1$ m/s

In any case of this particular example the accuracy of the simplified method is good since the error is below 5%. Fatigue of the monopile is underestimated while the
Figure 8.11: Error of equivalent stress range from short-term superposition of fatigue loads (Section 8.1) for 85% availability with respect to integrated time domain simulation of 440 elementary load cases.

Figure 8.12: Error of equivalent stress range from long-term superposition of fatigue loads (Section 8.2) for 85% availability with respect to integrated time domain simulation of 440 elementary load cases.

Figure 8.13: Error of equivalent stress range from integrated time domain simulation of 38 lumped load cases (Section 8.3, Table 8.3) for 85% availability with respect to 440 elementary load cases.
opposite holds for the tower section. The influence of the class width for the wind speed is less significant than that of the minimum load case probability. If possible with respect to the available environmental data a minimum load case probability of 0.5% is recommended while a wind speed bin of 2 m/s seems to be sufficient. The difference between the direct quadratic superposition, (7.18), and the weighted quadratic superposition, (7.17), is small and the preference for the simpler and more robust direct quadratic superposition, stated in Section 7.3.2, is confirmed.

The influence of the estimated effective aerodynamic damping during production is studied for the long-term superposition approach from Section 8.2. Again the errors of this even simpler method are small and range between -6% and -1%. Now at both cross sections a too low estimate of the equivalent stress range is observed. The reference design is quite insensitive with respect to a ±20% variation of the assumed effective aerodynamic damping which is a typical accuracy of the damping estimation. In this example the equivalent stress range is affected only by ±2%.

Finally the load case lumping from Table 8.3 in Section 8.3 is verified. Very good agreement of ± 0.6% with the respect to the result from 12 times more time domain simulations is found in Figure 8.13. This observation is amazing because the method rests upon the simplified approach from Section 8.1 which itself is less accurate (Figure 8.11). The most likely explanation is that the simplified approach is used only to weight the relative contribution of the different elementary load cases to a certain lumped load case.

The results of the parameter study are promising and clearly prove the suitability of the new methods for engineering applications during the different stages of the design process.

In the daily design practice larger errors are expected due to application of different computational tools and less consistent models for both the environment and the mechanical system. Consideration of only one particular reference design limits, however, a general judgement of the accuracy and further investigation of different offshore wind energy converters are recommended.
CHAPTER 9

INTERRELATION OF DYNAMICS AND DESIGN

The dynamic behaviour of an offshore wind energy converter is influenced by the design properties and site conditions. Vice-versa, the design depends (among other aspects) on the dynamics. In this chapter the interrelation is investigated for a broad spectrum of concepts and conditions.

After introducing the reference designs (Figure 9.1, Table 9.1), categorized by the applied wind turbine concept and size in Section 9.1, five essential aspects of dynamics and design of offshore wind energy converters (OWEC) are discussed in Section 9.2 to 9.6. Successively we are dealing with the siting and the environmental conditions, the

![Map of the six reference sites](Image)

**Figure 9.1:** Location of the six reference sites

<table>
<thead>
<tr>
<th>Wind turbine</th>
<th>Support structure</th>
<th>Site</th>
</tr>
</thead>
<tbody>
<tr>
<td>NedWind 40†</td>
<td>soft-stiff monopile A3†</td>
<td>IJsselmeer: Lely A3</td>
</tr>
<tr>
<td></td>
<td>soft-soft monopile A2†</td>
<td>IJsselmeer: Lely A2</td>
</tr>
<tr>
<td></td>
<td>soft-soft monopile A2-mod†</td>
<td>IJsselmeer: Lely A2</td>
</tr>
<tr>
<td>Enron Wind 1.5o†</td>
<td>soft-soft monopile†</td>
<td>Baltic: Utgrunden</td>
</tr>
<tr>
<td>Kvaermer Turbin</td>
<td>stiff-stiff lattice tower†</td>
<td>North Sea: NL-5</td>
</tr>
<tr>
<td>WTS 80†</td>
<td>soft-stiff gravity monopile†</td>
<td>Baltic: Blekinge</td>
</tr>
<tr>
<td>WTS 80M †</td>
<td>soft-soft monopile†</td>
<td>North Sea: NL-1</td>
</tr>
</tbody>
</table>

† actually built, ‡ proposed
dynamic characteristics, the aerodynamic and other sources of damping, the foundation uncertainty and the response behaviour under fatigue and extreme conditions. It is beyond the scope and not necessary to analyse every item for every design and site. So only those designs are considered which are suitable to stress important effects and to derive design recommendations. Section 9.7 provides a more general overview by comparing the design drivers and complementary results for the different support structures and sites.

9.1 Reference designs and sites

Overview

A variety of proposed or actually realised designs and sites is chosen for investigation. Looking at the wind turbine we see three different ratings of 500 kW, 1.5 MW and 3 MW which reflect the rapid development of offshore wind energy. The 500/600 kW class was applied for the first demonstration plants in Denmark, the Netherlands and Sweden between 1991 and 1998. Beginning in the year 2000 the megawatt class is employed for the first large-scale offshore wind farms. Even larger machines in the multi-megawatt class have been proposed for very large wind farms on a medium time scale. Each turbine is of a different design concept being the two-bladed, active-stall controlled, 500 kW NedWind 40, the three-bladed variable-pitch, variable-speed Enron Wind 1.5 offshore and the two-bladed, pitch regulated Kvaerner Turbin WTS 80 / WTS80M with constant speed. In fact, both two-bladed machines are representing uncommon design concepts. Especially the combination of pitch regulation and constant speed of the machines of Kvaerner Turbin is aged and not representative for machines in the multi-megawatt class which are currently under development.

The NedWind 40 design was realized at the first Dutch offshore wind farm Lely in 1994. Although it is an example for the parallel design approach (Section 3.3.2); important lessons about the sensitivity and response of monopile concepts were gained [9.1, 9.2]. Experiences that could be used in a more integral manner during the design and installation of the Utgrunden offshore wind farm with the Enron Wind 1.5 offshore in the year 2000. A prototype of the WTS 80, known as 'Näsudden II', is operating since the beginning of the 1990's on the island of Gotland. The sister machine with variable speed 'Aeolus II' was erected in Wilhelmshaven, Germany.

The diversity of designs is completed by the consideration of three generic support structure types, monopile, gravity monotor and gravity lattice tower. Particular emphasis is given to the monopile configuration. It is investigated for each of the three wind turbine classes because of its superior economic performance for many sites and turbine types. Another, not directly visible, difference lies in the dynamic characteristics which cover the entire span of opportunities from soft-soft, soft-stiff to stiff-stiff.

Sub-megawatt class of the first Dutch offshore wind farm Lely

The first Dutch offshore wind farm Lely (see front cover) comprises four NedWind 40 turbines at the western side of the IJsselmeer. Installed in 1994, steel monopiles were chosen for the very first time as OWEC foundations. Figure 9.2 shows the three investigated configurations, denoted Lely A3, A2 and A2-mod. The first two are based on the data specified in the original design calculations [9.3] and some updated data [9.4]. The design A3 has soft-stiff characteristics thanks
to a pile with a large diameter of 3.7 m and a long penetration depth of 20.5 m at 5 m water depth. Soft-soft characteristics were proposed for design A2 at a greater water depth of 10 m. So a lighter pile of 3.2 m diameter and only 13.5 m penetration was chosen. Eventually measurements proved soft-stiff characteristics, most likely caused by unexpected soil conditions or a modification of the design after the design calculation. For us, however, investigation of the original design configuration is more interesting.

By contrast to its two companions, the design A2-mod only exists within this thesis as an example for a design improvement of case A2 (Section 9.5.3). Although both concepts show the same fundamental eigenfrequency, the design A2-mod uses a larger pile penetration depth, a reduction in pile diameter close to the water surface and a stretched tower.

**Megawatt class at the Utgrunden wind farm**

With commissioning in autumn 2000 the wind farm on the Utgrunden reef in the Kalmarsund between the Swedish mainland and the Öland Island was one of the first projects which employed wind turbines of the megawatt class (Figures 1.2 and 2.5). Located 12.5 km from the mainland and 8 km from the island, the water depth ranges between 7.2 and 10 m.

The project with 7 units of the Enron Wind 1.5 offshore type was a turn-key project developed by Enron Wind. This enabled an integrated design of the wind farm and a first direct application of results of this thesis.

Utgrunden is also the first project with soft-soft monopile foundations. The foundation and tower were designed together as one structure, considering loads and frequencies
from machine/rotor, wind, waves, ice, etc, into an efficient and optimal structure. This could use a lighter and thus cheaper pile and tower. The relatively small pile diameter of 3 m reduces the installation efforts and results in lower ice loads. By this no ice cone was required which itself is sensitive to ice damage and which cannot be applied at sites with larger variations in the sea level e.g. due to tides and storm surges. Monopile foundations are very interesting from an economical point of view. However, attention is needed for the inclination of driven monopiles after installation and the possible damage of the pile top owing to the hammer impact. Therefore an innovative grouted joint between the pile and the tower is used (Figure 9.3). It proved easy and fast installation and capabilities to correct pile inclinations and compensate imperfections of the pile top.

In this thesis the location T4 with the maximum water depth of 10 m is considered as a reference example. The monopiles were designed for a large range of soil conditions which were difficult to estimate for this site on a glacial moraine with complex geotechnical constitution. We assume soil parameters in the middle of the design range which correspond to measurements of the natural frequencies. The fundamental eigenfrequency of the seven monopiles of approximately 0.27 to 0.29 Hz lies in the operational range of the mean rotor speed from 0.183 to 0.33 Hz. Therefore for each OWEC an individual operational window of the rotor speed is excluded from stationary operation. Above rated wind speed the turbines run at a mean rotor frequency of 0.33 Hz.

**Multi-megawatt concepts of the Opti-OWECs**

During the Opti-OWECs project three distinct OWEC concepts all using a proposed 3 MW wind turbine design of Kvaerner Turbin AB were subject to detailed dynamic analyses (Figure 9.4) [9.5]:

- **Lattice tower - NL-5 site**
  A gravity, steel lattice tower with stiff-stiff characteristics is combined with the WTS 80 wind turbine at a site 50 km off the Dutch North Sea coast. The stiff framework with three legs is based on experience in the design of offshore structures for demanding environments.

- **Gravity monotower - Blekinge site**
  A steel monotower on a gravity base with soft-stiff characteristics carries the WTS 80 wind turbine at a Swedish Baltic site. The considerable ice loading justifies a robust, single column structure.

- **Monopile - NL-1 site**
  The final design solution of the project includes a steel monopile with soft-soft characteristics and the WTS 80 M wind turbine at a Dutch North Sea site.
Figure 9.4: Opti-OWECS support structures: Left: Gravity lattice tower with three legs at 25 m water depth (MSL), Middle: Gravity monotower at 15 m water depth (MSL), Right: Steel monopile at 21 m water depth (MSL)
approximately 15 km offshore. This concept was developed with an integrated
design approach considering the techno-economics of the entire wind farm (Section
3.6, Chapter 10).

The WTS 80 M (marine) machine with a 16% higher rotor speed is the offshore variant
of the WTS 80 with a two-bladed 80 m pitch-regulated, constant-speed rotor, developed
originally for onshore siting [9.6].

The oscillating motion of the flexible, hydraulic yaw system is used to lower the inherent
high, dynamic loads of two-bladed rotor with a rigid hub and constant rotor speed. In
addition, damping is introduced into the yaw motion and other vibration modes of the
system. Within the scope of the thesis it was not possible to accurately model the
characteristics of the yaw system, partly due to restrictions of the used computational
tools and partly due to incomplete design specifications. The assumption of a rigid yaw
system is, however, likely to be conservative. A proper representation would have
required a cumbersome new derivation of the equations of motion in the rotor module
and an advanced model for unsteady aerodynamics and yawed flow. In the modelling
of the control system some simplifications have been made which were not necessary
if more detailed wind turbine data was available.

Further details of the designs and sites are presented in Appendix B.

9.2 Siting

Site characteristics

The six reference sites (Figure 9.1) are representing a broad band of environmental
conditions ranging with increasing water depth and exposure from the two places in the
sheltered inland sea of the IJsselmeer and two ice loaded locations in the Baltic to the
two North Sea sites. The growing severity of the sites is typical for the evolution of
offshore wind energy over time. While sheltered waters were chosen for the
demonstration plants about a decade ago, hostile North Sea locations will probably be
exploited on a medium time scale.

Table B.1 in Appendix B provides the main characteristics of the sites.

Offshore versus onshore wind climate

In Section 4.1.1, we have discussed the differences of the offshore wind climate to the
situation on land, being higher average and extreme wind speeds but lower turbulence,
wind shear and gust factors. Now we will use the soft-soft Opti-OWECS monopile design
to quantify the fatigue effect of different site characteristics.

Figure 9.5 plots contour lines for the same equivalent stress range at the critical cross
section of the sub-soil pile for different annual average wind speeds (x-axis) and design
turbulence intensities at 15 m/s mean wind speed (y-axis). For illustration we have
marked the offshore site NL-1 ($V_{ave} = 9$ m/s, $I_{15} = 9\%$) and two onshore site classes
defined by the IEC 61400-1 ed. 2 standard. The IEC TC III-A (type class III: $V_{ave} = 7.5$
m/s, turbulence class A: $I_{15} = 18\%$) represents a site with moderate wind speed but high
turbulence while IEC TC II-B could be a windy coastal location with low turbulence ($V_{ave}$
$= 8.5$ m/s, turbulence class B: $I_{15} = 16\%$). The aerodynamic fatigue loading is normalised
by the value for the NL-1 site.

Obviously the fatigue loading is driven by both annual average wind speed and
turbulence intensity, however, the latter has the stronger influence. The equivalent
Figure 9.5: Contour plot of the normalised equivalent stress range at the monopile of the Opti-OWECS design due to pure wind loading as function of the annual average wind speed (x-axis) and the turbulence intensity at 15 m/s (y-axis).

(Loading at the NL-1 site equals 100%.)

Fatigue loading at the land-based site is approximately 40% to 50% higher than at the NL-1 location. If the same high turbulence would be assumed at both onshore sites, a 23% higher loading could be expected at the more windy location. In fact, the increase account only for half of this, thanks to the lower assumed turbulence class. Large offshore wind farms will take relatively less profit from the low (natural) ambient turbulence since the wakes of upstream turbines are more persistent and induce a background turbulence level. For the Opti-OWECS design this effect is not so significant since a simpler and older wake model (Figure 4.7) and a wide turbine spacing of 10 rotor diameters is chosen. In practice, half of the distance is more realistic.

Likewise to the turbulence, the relative gust amplitude at extreme wind conditions is lower offshore. Measured evidence is however scarce. So, standards use different safety philosophies here. If one follows the Germanischer Lloyd Guidelines the gust wind speed offshore is not necessarily higher than on land because offshore the extreme gust is assumed to be only 20% higher than the extreme mean wind speed in comparison to an increase of 40% onshore (Table 4.2). IEC 61400-1 ed. 2 has been developed for onshore applications only and uses the same gust factor than Germanischer Lloyd onshore while DS 472 applies the identical gust factors onshore and offshore. For pitch regulated wind turbines the extreme operating gust is often an important extreme load case. Again different models exist in the various standards. Germanischer
Lloyd and DS 472 apply site independent gust magnitudes while the IEC 61400-1 ed.2 relates the amplitude to the turbulence intensity and rotor size.

High average wind speeds over 8.5 m/s classify many offshore sites in the demanding Type Class I according to IEC 61400-1. However, if the lower turbulence is considered machines which were originally designed for Type Class II might still be accepted by a site specific certification. Likewise the extreme mean wind speeds of most of such Class I sites is considerably lower than the 50 m/s assumed by the international standard.

**Hydrodynamic and ice conditions**

The hydrodynamic conditions vary largely between the sites (Figure 9.6). In addition, the greater water depth at exposed sites further increases the overturning moment. Roughly wave loads with double magnitude occur at the NL-5 site compared to the Baltic sites. The fatigue loading depends also on the dynamics of the structure and the wave steepness. Dynamic loading of short waves with low height can be significant even in the IJsselmeer if the design is sensitive and excited close to resonance. At many Baltic sites the governing extreme load takes place as a combination of the extreme sea ice and an aerodynamic loading.

**9.3 Dynamic characteristics**

Analysis of the eigenfrequencies and eigenmodes can provide important, more qualitative insight in the dynamic behaviour which itself can only be quantified by more cumbersome response analyses.
In this section we give an overview on the natural frequencies of the reference designs and see how far they are affected by the offshore foundation (Section 9.3.1). Simple one-degree-of-freedom models can explain the influence of the eigenfrequencies on the aerodynamic excitation of the tower top and the hydrodynamic forcing of both the support structure and flapwise blade bending (Section 9.3.2). Finally, in Section 9.3.3, the relevance of the mode shapes for the sensitivity against variations of the soil stiffness and the effectiveness of the wave excitation is investigated.

### Table 9.2: Eigenfrequencies of the reference designs in Hertz

<table>
<thead>
<tr>
<th>Eigenmode</th>
<th>Design</th>
<th>Lely, ( f_n = 0.54 )</th>
<th>Utgrunden, ( f_n = 0.32 )</th>
<th>WTS 80, ( f_n = 0.63 )</th>
<th>WTS80M, ( f_n = 0.73 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1(^{st}) fore-aft</td>
<td>A3</td>
<td>0.70</td>
<td>0.28</td>
<td>0.76</td>
<td>0.51</td>
</tr>
<tr>
<td></td>
<td>A2</td>
<td>0.44</td>
<td>0.44</td>
<td>0.76</td>
<td>0.51</td>
</tr>
<tr>
<td></td>
<td>A2-mod</td>
<td></td>
<td></td>
<td>0.76</td>
<td>0.51</td>
</tr>
<tr>
<td>2(^{nd}) fore-aft</td>
<td></td>
<td>3.03</td>
<td>1.46</td>
<td>2.69</td>
<td>1.27</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.63</td>
<td></td>
<td>2.69</td>
<td>1.27</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2.00</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1(^{st}) blade flap</td>
<td></td>
<td>2.17</td>
<td>1.03</td>
<td>1.40</td>
<td>1.41</td>
</tr>
<tr>
<td>1(^{st}) blade lead-lag</td>
<td></td>
<td>3.26</td>
<td>1.83</td>
<td>2.11</td>
<td>2.11</td>
</tr>
</tbody>
</table>

\( f_n \) = rotor frequency, \( f_b = N_b \cdot f_n \) blade passing frequency

\( f_R \) = 0.183 - 0.33 Hz and \( f_b = 0.55 - 1 \) Hz for operational range of mean rotor speed

\( \Omega = 11 - 20 \) rpm, mean rotor speed above rated wind speed 20 rpm

#### 9.3.1 Natural frequencies and foundation influence

**Natural frequencies**

The major eigenfrequencies of the reference designs and the rotor excitation frequencies are listed in Table 9.2. The blade frequencies are not significantly affected by the support structure stiffness, thus, no difference is made between the Lely and WTS 80 designs.

**Foundation influence and fixity length of monopiles**

Obviously, the design of offshore wind energy converters has to bypass resonances with the rotor frequency or its multiples and excessive wave excitation. Different considerations exist for the foundation design compared to land-based buildings or offshore platforms.\(^1\)

---

\(^1\) The foundation design of most land-based buildings is often governed by the bearing capacity under limit state conditions. Stiffness or other dynamic criteria generally are of lower importance. For onshore wind energy converters both the bearing capacity and the avoidance of a rotor or blade resonance is important. The latter requirement is often matched by tailoring the tower because the foundation flexibility reduces the fundamental eigenfrequency only up to 5 to 10% compared to a rigid foundation. Thus the design is fit for purpose for a variety of sites if the foundation stiffness is located somewhere between the minimum value assumed and infinity. The design of most foundations of offshore platforms is governed by limit state loads as well. The foundation influence on the first natural frequency is between 0% and almost 20 to 40% at small offshore structures. This considerable spread is caused by the foundation type, the variable soil properties and (continued...)
This problem is accentuated by the comparison of the foundation influence between the seven reference designs in Table 9.3. The soil flexibility decreases the fundamental eigenfrequencies of the two gravity designs, lattice tower and monotower, by about 7%. In contrast, the monopiles are subject to an at least two times higher reduction. The Lely A2 case shows an exceptionally high effect of 40%.

Another, more illustrative measure for the flexibility of a piled foundation is the apparent fixity length introduced in Section 5.2.2. Table 9.3 presents this quantity in units of the pile diameter. An appreciably low variation between 3.1 and 3.7 is discovered between the values derived from the Lely measurements and the design values of the Utgrunden and Opti-OWECS design. For a preliminary design an indicative figure between 3 and 4 pile diameters can be recommended.

The illustration of the fictitious clamping points at the fixity depth of the Lely designs in Figure 9.2 is useful to explain the dynamic behaviour. Whilst the A3 and A2-mod monopiles are regarded to be fixed at about 3/4 of the penetration, the fixity point of the A2 design is located at about the double penetration depth. This seems to be strange but is confirmed when considering the almost rigid body shape of the first eigenmode in Figure 9.10.

The sensitivity of the eigenfrequencies is further studied in Section 9.5.2 and Section 10.3.2 where the effect of uncertain soil properties and other influences is studied.

In general, the foundation influence is so large that individual foundation design is necessary for each wind farm site and each OWEC type. Within a wind farm site, modifications of the support structure design including the foundation might be required owing to variation of the water depth and possibly the local soil conditions.

<table>
<thead>
<tr>
<th>Table 9.3: Quantification of foundation influence</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lely</td>
</tr>
<tr>
<td>A3</td>
</tr>
<tr>
<td>decrease of 1st eigenfrequency compared to rigid foundation</td>
</tr>
<tr>
<td>length [pile diameter]</td>
</tr>
</tbody>
</table>

* derived from measurements, * derived from Lely A2 measurements

(...continued)

the non-linear soil behaviour under different load levels. This and other considerations favour relatively stiff offshore structures with natural frequencies well beyond the wave excitation range extending between $f = 0.04$ Hz and $f = 0.5$ to $1.0$ Hz. Dynamic wave loading is strongly reduced for structural frequencies above approximately $0.4$ Hz.
9.3.2 Excitation of tower and flapwise blade bending

From previous investigations it is well-known that the rotor blades of a (bottom-mounted) OWECS are affected predominantly by aerodynamic loads whilst the support structure is subject to both significant aerodynamic and hydrodynamic excitation [9.7, 9.8]. One may argue that this is an obvious fact since waves may never touch the rotor whereas the wind and wave loads are transferred through the structure into the soil. But does this also hold for very soft designs, as the Opti-OWECS monopile, where considerable vibration amplitudes occur at the tower top?

Application of two models each with a single degree of freedom (SDOF) for the translation of the tower top and for the flapwise blade motion can explain the basic response characteristics (Figure 9.7).

Both aerodynamic and hydrodynamic loads are directly exciting the support structure which corresponds to an excitation of the SDOF system at the spring. The response is first quasi-static, i.e. the dynamic amplification factor (DAF) is close to one. With increasing forcing frequency it passes high dynamic amplifications in the resonance region and decreases finally for high excitation frequencies below the static response. Figure 9.9 plots the DAF for two damping values associated with the response of a pitch regulated design in the wind direction, where an aerodynamic damping ratio of 3% is active, and perpendicular to the wind direction with only 0.5% material damping. The excitation frequency is made non-dimensional by the fundamental eigenfrequency.

Most important for the aerodynamic response are the rotor frequency (1P) and blade passing frequency ($N_b$P) which are shown with markers for the three Opti-OWECS designs. The stiff-stiff lattice tower design exhibits a DAF of 3.7 of the strong 2P excitation whilst the soft-stiff monotower and soft-soft monopile show lower amplifications of 1.8 and 0.2, respectively. This explains qualitatively, what we will see during the aerodynamic and hydrodynamic fatigue analyses in Sections 9.6.1 and 10.2.2. For the two-bladed WTS 80 turbine it is quite beneficial to have a soft support structure even if higher dynamic wave loading has to be accounted for.

Due to the non-dimensional frequency scale the wave excitation range has different extensions for the three concepts. Nearly quasi-static response occurs for the stiff-stiff lattice tower. The natural frequency of the soft-stiff monotower lies on the edge of the wave energy band, however, the soft-soft monopile is located in the middle of it.

Wave excitation of the rotor blades can only take place through inertia forces generated by the tower top acceleration which is most significant at the structural eigenfrequencies. Here the SDOF system is forced through a prescribed motion of the frame. The dynamic amplification in Figure 9.8 has a reverse dependency on the non-dimensional excitation frequency $f_{\text{lower}}/f_{\text{flap}}$ given by the ratio between support structure and flap eigenfrequency [9.9]. The blade follows the low-frequency excitation and does not feel any significant inertia loads. Beyond the resonance peak the excitation is too fast and the blade rests more and more in an inertial reference frame.

If we take the Opti-OWECS monopile for an example, the ratio of the first tower and blade eigenfrequency accounts for 0.2 governing a dynamic amplification of only 0.04.

---

2 The ratio between blade mass and equivalent tower top mass is so small that for this purpose the interaction between the support structure with nacelle and the rotor is described as a forced vibration of the rotor due to a kinematic excitation at the hub.
Figure 9.7: Single degree-of-freedom models for the support structure excitation and flapwise blade excitation

Figure 9.8: Dynamic amplification of flapwise blade motion due to tower top acceleration with frequency equal to the tower eigenfrequencies

Figure 9.9: Dynamic amplification of the support structure of the Opti-OWECS concepts due to aerodynamic and hydrodynamic loads
Although a DAF of 1.6 is related to the second support structure eigenfrequency with $f_{\text{tower}}/f_{\text{bag}} = 0.85$ the wave excitation energy is much lower for this mode than for the fundamental mode. So both modes do not result in a significant blade response either due to a low DAF or low excitation energy.

### 9.3.3 Relevance of mode shapes

A visual inspection of the shape of the eigenmodes can be used to judge the foundation’s sensitivity to the variable soil conditions and to the hydrodynamic excitation. As example the modes of the three Lely monopile designs and the gravity monotower are plotted in Figure 9.10.

**Foundation sensitivity**

Comparison of the subsoil displacements of the Lely designs reveals distinct behaviour. Designs A3 and A2-mod are properly clamped in the soil whilst design A2 undergoes large deflection in this area. The first eigenmode of Lely A2 behaves similar to a rigid body mode, having minor curvature along the structure but large rotational motion in the soil. Significant lateral displacements occur over the entire penetration depth. In contrast, only the upper quarter of the soil in case of the first mode and the upper half of the penetration for the higher modes are affected for Lely A3 and especially A2-mod. A large contribution of the foundation translation to the entire mode is also observed for the second mode of the gravity monotower.

Obviously, modes with considerable soil deformation are more sensitive to the soil conditions. In Section 9.5.2 (Figure 9.13) this is confirmed also quantitatively by a numerical analysis, which proves high sensitivity in particular for the first modes of Lely A2 and the second mode of the gravity monotower.

**Hydrodynamic sensitivity**

In the previous Section 9.3.2 we have studied the dynamic amplification as the result of the ratio between excitation frequency and natural frequency and of the excitation type. The distribution of the excitation forces, as wave or wind loads, in relation to the mode shapes is of importance, as well. For instance, a certain mode of a rod can barely be excited in the vicinity of a vibration node even if the forcing frequency is close to the resonance frequency. This effect can be quantified by the magnitude of the generalised external forces, $\Phi^T \cdot f$, on the right hand side of the equations of motion, (6.2). The modal transformation corresponds to the spatial integration of the product of the concerned mode shape and the external load.

We are mainly interested in the wave excitation of the support structure and introduce a hydrodynamic participation factor, $\chi_{\text{hyd},i}$, of the $i^{\text{th}}$ mode shape, $\varphi_i(z)$. Normalising the maximum displacement of the mode shape to unity and division by the base shear obtains a non-dimensional expression for the generalized force, (9.1), which has only a relative meaning.\(^3\)

---

\(^3\) For the fundamental mode, which has the maximum deflection at the tower top, the hydrodynamic participation factor can be interpreted as the relative effectiveness of the wave excitation compared to the forcing by the rotor thrust.
\[
    x_{\text{hyd},i} = \frac{\int_{-d}^{0} \varphi_i(z) \ dF(z) \ dz}{\max \{ \varphi_i(z) \} \ \int_{-d}^{0} dF(z) \ dz}
\]

(9.1)

The wave force per unit length, \(dF(z)\), shows an exponential decay from the instantaneous water surface down to the sea bottom. So, small waves are only effective near the water surface while larger waves (in shallow waters) affect the entire submerged structure.\(^4\)

Table 9.4 compares the hydrodynamic participation factors for the six monopile and monotower designs.\(^5\) For the Lely A2 design a two to three times higher hydrodynamic excitation can be expected than for the other Lely and Opti-OWECS designs if the same fundamental eigenfrequency is assumed. The Utgrunden monopile has an extraordinary low participation factor due to the rather stiff monopile in comparison to the slender, double conical tower. Therefore, mainly the tower flexibility contributes to the first two bending modes.

<table>
<thead>
<tr>
<th>Bending mode</th>
<th>Lely</th>
<th>Opti-OWECS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A3</td>
<td>A2</td>
</tr>
<tr>
<td>1(^{st}) mode</td>
<td>5.9 %</td>
<td>20 %</td>
</tr>
<tr>
<td>2(^{nd}) mode</td>
<td>57 %</td>
<td>80 %</td>
</tr>
<tr>
<td>3(^{rd}) mode</td>
<td>14 %</td>
<td>5 %</td>
</tr>
</tbody>
</table>

9.4 Aerodynamic and other sources of damping and inertia

9.4.1 Aerodynamic damping

The most significant source of damping of an offshore wind energy converter is the aerodynamic damping of the flapwise blade bending modes and the fundamental support structure fore-aft mode. Other modes show none or much lower damping. In Section 5.3 we discussed the principles of this effect and we found two major influences by analysing the simple closed-form expression (5.12): The local derivative of the lift coefficient with respect to the angle of incidence and the structural dynamics of the wind turbine and the support structure. Both aspects can be studied at the reference designs.

Section 7.1.3 and Figures 7.10 to 7.12 addressed the aerodynamic damping ratio as fraction of critical damping of the fundamental fore-aft mode for the pitch regulated WTS

\(^4\) In fact the influence of the wave height and wave length is moderate and it is sufficient to apply linear wave theory which yields an integration between \(z = 0\) at the still water line and \(z = -d\) at depth, \(d\), on the sea bottom. We assume a deterministic wave with the parameters of the most probable sea state.

\(^5\) The participation factor, (9.1), is not defined for space frame structures where horizontal and vertical wave forces are acting simultaneously.
Figure 9.10: Bending mode shapes of the three Lely designs and the Opti-OWECS gravity monotower

- 1st mode,
- 2nd mode,
- 3rd mode
Figure 9.11: Aerodynamic damping of the fundamental fore-aft mode of the Lely designs

80M wind turbine on a soft-soft monopile. The behaviour for an active-stall regulated design can be studied at the Lely examples in Figure 9.11. The difference between stall and pitch regulation is obvious. In both cases the high damping at low wind speeds reduces towards the rated wind speed. For a stall regulated design the flow separation reduces the damping ratio dramatically in the vicinity of the rated wind speed. The increasing drag loading leads to some recovery of the damping on a low level in deep stall close to the cut-out wind speed. Where in case of pitch control it recovers beyond the rated wind speed since the reduction of the angle of incidence results again in more attached flow (Figure 7.11). The fast dynamics of the pitch control system influence the damping. In this particular case the damping ratio decreases towards the cut-out wind speed.

Softer support structures with a lower modal mass experience larger aerodynamic damping ratios. The lower natural frequency of the Lely A2 and A2-mod configurations result in a higher damping ratio than for the A3 design. The more slender tower of A2-mod features a lower modal mass and affect more damping compared to Lely A2 with the first same eigenfrequency. Similar behaviour is observed at the Opti-OWECS designs where the monopile is about 50% better damped than the gravity monotower and lattice tower. In the latter case the low modal mass of the stiff framework results in an appreciably high damping ratio for such a stiff structure of approximately 3% at low wind speeds where the monotower shows about 3.5%.

In summary, the variation of the damping ratio between different designs and different operational conditions are so large, that proper modelling is certainly required. Time domain simulations with an integrated OWEC model consider the aerodynamic damping inherently. For other approaches, e.g. in the frequency domain, a set of models covering the full range of operational conditions and associated aerodynamic damping values is required.
9.4.2 Hydrodynamic mass and damping
An effect of the fluid loading of offshore structures, present also in a calm sea, is the mass of the internal water and the effective mass of the surrounding fluid, (5.11). Neglecting both of them, would result in an increase of the fundamental eigenfrequency of the reference designs of less than 2%. Only the Lely A2 design shows a greater sensitivity of 5%. By contrast, the second bending frequency is significantly affected for all designs. While the effect accounts for 15% for the stiff lattice tower, it is up to 60% for the soft Lely design A2. The larger influence can be explained qualitatively by the greater value of the hydrodynamic participation factor of the second eigenmode in comparison to the first mode in Table 9.4.

Two general reasons make hydrodynamic damping and vortex shedding not significant for bottom-mounted OWEC support structures. The submerged part of such structures is relatively stiff in comparison to the dry section and structural velocities of the wet members are low. So the hydrodynamic lift and drag forces, which depend quadratically on the velocities, are also low and any viscous hydrodynamic damping is negligible. Consequently, the hydrodynamic drag forces are calculated based upon the Morison equation written in absolute water particle velocities.

There will be a very small amount of damping owing to the radiation of energy from the submerged surface, which is however not further considered.

9.4.3 Soil damping
Sections 5.2.2. and 5.2.3 described distinct models for both the hysteretic or plastic damping, more significant for piled foundations and the radiation damping, important for gravity foundations. It has been attempted to provide damping estimates for both foundation types.

Analysis of the hysteretic soil damping of the piled foundation is more complex due to the influence of the lateral loading and applied soil model. Table 9.5 compares the modal damping ratios of the bending modes of the Lely A2 and the Opti-OWECs monopile derived from an equivalent viscous damping (5.5). Due to the restrictions of the mathematical model the values itself are more qualitative and the most important conclusion is that the fundamental modes show little soil damping while the higher modes experience damping ratios between 0.15% and 0.6%.

The lateral loading is relatively small compared to the ultimate bearing capacity. So almost elastic rather than elasto-plastic soil behaviour occurs, which results in low damping. Furthermore, it should be noted that the prediction of hysteretic soil damping is strongly dependent on the P-y curve used and the modelling of the soil behaviour.

<p>| Table 9.5: Estimates of the soil damping of two monopile support structures |
|-----------------------------|-----------------------------|-----------------------------|</p>
<table>
<thead>
<tr>
<th>Bending mode No.</th>
<th>Eigen-frequency</th>
<th>Damping ratio</th>
<th>Bending mode No.</th>
<th>Eigen-frequency</th>
<th>Damping ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (fore-aft)</td>
<td>0.444 Hz</td>
<td>≈ 0.26 %</td>
<td>1 (fore-aft)</td>
<td>0.288 Hz</td>
<td>≈ 0.07 %</td>
</tr>
<tr>
<td>1 (lateral)</td>
<td>0.455 Hz</td>
<td>≈ 0.23 %</td>
<td>1 (lateral)</td>
<td>0.292 Hz</td>
<td>≈ 0.04 %</td>
</tr>
<tr>
<td>2 - 4</td>
<td>1.6 - 14 Hz</td>
<td>0.15 - 0.5 %</td>
<td>2 - 6</td>
<td>1.2 - 15 Hz</td>
<td>0.4 - 0.6 %</td>
</tr>
</tbody>
</table>

(Level of cyclic loading corresponds to operation during the sea state associated with the annual average wind speed.)
under low strains. Small differences in the mathematical modelling can have a large effect.

Indication on the total structural and soil damping can be gained from measurements during shut-down and employment of the mechanical brake at the Lely A2 and A3 turbines. From the evaluation of the time series of the lateral tower top acceleration with (7.1) we find damping ratios of 0.85% and 0.7% for both designs (Table 9.6). These values are larger than the range of 0.2 to 0.5% measured at common onshore wind turbine towers (Section 5.2.1) [9.10]. For the time being the 0.5% damping ratio used in the numerical simulations seems to be an adequate number and further investigations at other monopile structures are recommended before this might been altered.

Closed-form solution for the soil stiffness, (5.6), and soil damping, (5.7) and (5.8), of gravity bases on an elastic half-space were discussed in Section 5.2.3. For the gravity monopile the eigenfrequencies and associated damping ratios of a frequency invariant model are compared to those from the iterative solution of a frequency dependent modelling in Figure 9.12 [9.11]. In the relevant frequency range up to 5 Hz the natural frequencies are not affected by the model choice. Small effect is seen on the damping except for the important fundamental mode where the frequency dependent approach obtains 0.4% damping which is only half of the value from the simpler method. The frequency dependent model is only compatible with a harmonic analysis rather than with transient time domain simulations which is our intention. Thus, neglecting of the frequency dependency and a conservative assumption of the calculated soil damping is recommended for design purposes.

Further experimental investigations of both piled and gravity support structure types are recommended to reduce the uncertainty about the magnitude of soil damping.
9.5 Foundation uncertainty

Even after in-situ tests the soil properties are inherently uncertain and soil dynamics of offshore structures remain a difficult affair. Despite the various mathematical models mentioned in Sections 5.2.2 and 5.2.3 the most rational treatment is to consider a much larger safety margin than for the environmental loading and material strength. Load calculations should consider the most onerous soil conditions which will be in most cases the lower assumed bound for soil stiffness and strength (Section 10.3.2).

The particularities of OWEC foundations are highlighted by evaluation of eigenfrequency measurements at the wind farm Lely with an unexpected large deviation from the proposed design for the Lely A2 converter (Section 9.5.1). Taking this experience we analyse the foundation sensitivity for the reference designs and are able to derive some design recommendations in Section 9.5.2 which are used in Section 9.5.3 to propose a fictitious design improvement for the problematic Lely A2 case.

9.5.1 Evaluation measurements at the Lely wind farm
The Lely project was the first of its kind employing monopile foundations. Therefore measurements of the eigenfrequencies and the shut-down behaviour of the A2 and A3 designs were carried out in early 1995, six months after installation [9.12]. Both measurements, listed in Table 9.6, confirmed stiffer response behaviour than predicted by the design calculation [9.3]. For the A3 location the prediction by the original design calculations was approximately 6\% too low. Despite the mismatch this is no problem for a safe plant operation. An even larger margin to both the rotor excitation at 0.54 Hz and the wave excitation is provided.

By contrast, the error of 35\% at Lely A2 can neither be accepted nor explained completely. Only the fact that the deviation is large enough to change the characteristics from the proposed soft-soft to the actual soft-stiff type prevents considerable rotor excitations and likely fatigue problems. As a side effect the significant wave excitation of the proposed design is reduced.

A parameter study was performed to explain possible reasons for the discrepancy between predictions and measurements [9.4]. The design calculations were repeated with more realistic assumptions, which include consideration of water mass, a lower lateral prestress level of the piles, slightly revised design data for nacelle mass, tower wall thickness and mass of the tower interior. The P-y curves of the soil were maintained, since the derivation from Cone Penetration Tests (CPT) and partly laboratory tests could be reproduced [9.13]. The errors were lowered by this exercise (Table 9.6) but do not resolve the wrongdoing behaviour.

So far only speculative explanations can be given. Firstly, the site conditions could be different than assumed. A lower water depth owing to back-filling of the local sink or stiffer soil might be the case. Indeed the design is extraordinarily sensitive to the soil conditions, as will be concluded below. No single reason is sufficient since, for instance, the soil stiffness must be increased by a factor of 10 (!) to fit the measurement. Secondly, the design might not correspond to the specifications, e.g. the pile penetration or pile stiffness are larger or the wind turbine mass is lower. Further queries did not answer this question.

Thirdly, the measurement at A2 but not at A3 might be wrong or the similar design A1 at a lower water depth was measured instead.
To clarify the situation a repetition of the measurements has been initiated which also should study possible aging effects [9.14]. At the time of writing the investigation was ongoing.

**Table 9.6: Measured versus predicted first eigenfrequencies at Lely**

<table>
<thead>
<tr>
<th>Lely</th>
<th>Measurement</th>
<th>Original design calculation</th>
<th>Calculation with revised OWEC data</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>eigen-freq.</td>
<td>damping ratio †</td>
<td>eigen-freq.</td>
</tr>
<tr>
<td>A2</td>
<td>0.63 Hz</td>
<td>≈0.85% Hz</td>
<td>0.41 Hz</td>
</tr>
<tr>
<td>A3</td>
<td>0.72 Hz</td>
<td>≈0.7% Hz</td>
<td>0.68 Hz</td>
</tr>
</tbody>
</table>

† total structural and soil damping measured during shut-down

### 9.5.2 Sensitivity of eigenfrequencies

_Sensitivity study for the reference designs_

Design calculations have to account for the uncertainty of the soil parameters and models. In Chapter 10 distinct approaches are introduced that fit the requirements of the design process.

One of them proposed for the conceptual design phase is applied here to study the sensitivity of the reference designs. The soil stiffness is multiplied by a partial safety factor γ_m. A priori it is not clear whether a lower or a higher resistance is more onerous. Existing design guidelines, for instance Germanischer Lloyd or American Petroleum Institute provide only safety factors on the soil strength ranging between 1.5 and 2. Without more specific knowledge about the particular conditions, we may apply indicative values of γ_m = 0.5 and γ_m = 3.0.

The upper tolerance is greater than the reciprocal lower tolerance because soil parameters are often estimated at the lower side, which is conservative from a strength point of view. Moreover, the fundamental eigenfrequency is depending decreasingly on the soil stiffness, so the asymmetrical stiffness variation corresponds to an approximately symmetrical frequency variation.

Figure 9.13 compares the sensitivities of the first two fore-aft natural frequencies of the reference designs.

The fundamental eigenfrequency of all designs with the exception of Lely A2 is affected by ±5% or less which is reasonable. The two gravity foundations show larger sensitivity than the monopiles. This holds particularly for the comparison with the Utgrunden and Opti-OWECS monopiles. Lely A2 suffers unacceptable vulnerable and unsafe behaviour. The risk of even more extensive wave excitation or a rotor resonance is enormous.

In all cases the sensitivity of the second bending mode is greater than for the first mode. Again and even more pronounced the gravity foundations perform less favourable than
Figure 9.13: Sensitivity of first (left) and second (right) fore-aft natural frequency of the reference designs

the monopiles. Although a resonance of the fundamental mode is more critical, the second mode could easily interfere with higher harmonics of the rotor frequency or a blade bending mode.

For completeness it is mentioned that the possibility of scour (Section 4.3) induces another soil uncertainty that increases the lever arm of the environmental forces and can lower the eigenfrequencies significantly.

‘Tuning’ of eigenfrequencies

Sometimes it is proposed to ‘tune’ the fundamental eigenfrequency by placing ballast near the tower top. However, one should be aware of the considerable weight required and the practical implications of the installation. For instance, 20% extra nacelle weight decreases the fundamental frequencies of the reference designs between 6.5% and 7.5% only. The second bending mode is hardly affected since the associated translation of the tower top is small.

Wind turbines with variable rotor speed as the Enron Wind 1.5 offshore at Utgrunden can tackle eigenfrequencies located ‘somewhere’ within the rotor speed range by intelligent control and exclusion of an operational window. Nonetheless, difficulties occur if the eigenfrequency lies in the vicinity of the nominal rotor speed which is commonly used for operation between the rated and the cut-out wind speed. Alternation of the nominal rotor speed would have a larger impact on the control system and the aerodynamic efficiency.

9.5.3 Improved design for wind farm Lely

A too low penetration depth of the Lely A2 design can be concluded from the evaluation of the eigenmodes in Section 9.3.3.3 and the sensitivity study in the previous section.
Figure 9.14 illustrates the effect of the penetration depth on the nominal first eigenfrequency and its indicative sensitivity range for the three Lely designs. When the penetration depth of A2 is increased beyond that of Lely A3, the frequency tolerance becomes acceptable but a rotor resonance occurs. At the A2-mod design the pile penetration is increased from 13.5 m to 20 m, a level beyond which no further significant increase of stiffness can be achieved and the uncertainty range of the first eigenfrequency is reduced to ±5%. The soft-soft behaviour is maintained by employing a pile extension of 1.5 m above the sea level and a 8.5 m longer tower. The shift of the flexibility from the foundation to the taller tower also reduces the sensitivity to the soil stiffness compared to both A2 and A3.

![Graph showing first eigenfrequency vs pile penetration depth](image)

**Figure 9.14:** First eigenfrequency of the three Lely designs and uncertainty ranges as function of the pile penetration depth

### 9.6 Fatigue and extreme response

#### 9.6.1 Aerodynamic and hydrodynamic fatigue

Different sections of the thesis discuss aerodynamic and hydrodynamic fatigue. Therefore only some complementary aspects are dealt with here. We address again the soft-soft monopile of the Opti-OWECS study since it is most challenging in this respect. In contrast to other sections here only the method of the integrated, non-linear time domain simulation is applied.

The design is subject to considerable aerodynamic fatigue owing its size and the concept of a two-bladed rotor with constant speed and pitch regulation. With the fundamental eigenfrequency in the range of significant wave excitation the concept suffers also high dynamic amplification of the loading and considerable hydrodynamic fatigue. In Section 10.2.2 (Figures 10.8 and 10.9) it is demonstrated how the overall loading has been tailored by a higher rotor speed and a softer support structure.

*Start-up and shut-down behaviour*

Table 8.4 in Section 8.3 has established load cases for the transition between the production and parking state and vice versa. In many cases such load conditions are of
Figure 9.15: Start-up procedure of the soft-soft monopile configuration and passing of the blade resonance at 8.35 rpm and rotor resonance at 16.7 rpm lower importance for the total fatigue loading of the support structure but they are demanded for certification calculations. The WTS 80M with the soft-soft monopile is an interesting configuration where during start-up and shut-down the rotor and the blade excitation frequencies have to pass the fundamental support structure eigenfrequency.

Figure 9.15 illustrates the start-up behaviour at the cut-in wind speed. If a critical wind speed or rotor idling speed is detected by the supervisory control system the blades are pitched into the rotor plane (1\textsuperscript{st} signal). In the example a start from the resting position at 85° is assumed but idling at a low rotor speed with a pitch angle of approximately 60° - 65° is possible as well. During the pitching action the rotor is accelerated aerodynamically (2\textsuperscript{nd} signal) and the pitch rate is reduced. In case the acceleration is not fast enough to reach the rated speed or the rotor speed remains too long in the vicinity of the resonance frequency the start-up manoeuvre is cancelled and the blades are feathered again. In the present case with stationary wind the operation is successful and the generator is connected after synchronisation to the grid (3\textsuperscript{rd} signal). Owing to the negligible aerodynamic damping of the lateral motion the two resonances are clearly visible in the lateral bending moment (6\textsuperscript{th} signal). Tower shadow, wind shear, yawed flow and nacelle tilt (Figures 5.2 and 5.3) cause the blade passing (2P) excitation of the structure. The aerodynamic damping leads to a much lower excitation of the fore-aft response (5\textsuperscript{th} signal) than for the lateral motion (6\textsuperscript{th} signal). The rotor resonance (1P) is
triggered by the mass and aerodynamic imbalances of the rotor. Now both fore-aft and lateral response are significant. For such a fast passing of the resonance ranges the vibration amplitudes at the tower top stay below 8 cm and the associated stress variations are low.

During shut-down at low wind speeds a short employment of the mechanical brake with a brake torque of 40% of the rated torque is required to pass the blade resonance quickly enough. Pure aerodynamic braking results in a large overshoot of the tower top motion into the wind direction since the rotor acts as a propeller when the pitch angle is too large with respect to the rotor speed.

Starting and stopping at high wind speeds have to be combined with the power control but enable easy crossing the resonances. The power in the wind is large and enables quick up-speeding and slowing-down of the rotor in combination with only moderate pitch angle variations.

The transition load cases from Table 8.4 have been analysed in both directions and at the cut-in and cut-out wind speeds for stationary wind combined with stochastic waves.

A total damage below 0.003 occurs for pure wind loading. For combined aerodynamic and hydrodynamic loading the damage in the monopile is still below 0.01 and it is dominated by the wave response at the cut-in wind speed. In principle such situations are covered also by the parking load cases No. 2 and 20 in Table 8.3. The probability of occurrence of these stationary, stochastic conditions has been estimated conservatively by assuming a high cut-in and low cut-out criterion. It is therefore concluded that for the present OWEC configuration the transition load cases do not contribute significantly to the fatigue loading.

**Dynamic amplification of wave response**

In contrast to the preceding aspect the dynamic amplification of the wave loading is a remarkable issue. Figure 9.16 presents the dynamic amplification of the damage equivalent overturning moment at the mudline for pure wave loading according to lumped load cases similar to those from Table 8.3. When the turbine is parked, no aerodynamic damping is present and the dynamic amplification factor (DAF) grows.

![Figure 9.16: Dynamic amplification of the damage equivalent bending moment at mudline of the Opti-OWECS monopile due to pure sea state loading](Image)

The graph shows the dynamic amplification factor (DAF) against spectral peak frequency [Hz]. The cases include:

- **turbine parked:**
  - $H_s = 4.5$ m
  - $T_z = 6.4$ s
  - $p = 27$ h/year
  - 0.5% structural damping

- **production:**
  - **structural and aerodynamic damping**
  - $H_s = 1.7$ m
  - $T_z = 4.4$ s
  - $p = 440$ h/year

- **turbine parked:**
  - $H_s = 0.5$ m
  - $T_z = 3.4$ s
  - $p = 1300$ h/year
proportionally to the peak frequency of the Pierson-Moskowitz spectrum. Excessive DAF values up to 7 are observed since the spectral peak frequency is close to the fundamental natural frequency of 0.29 Hz.

During production when aerodynamic damping is active the DAF varies only between 2.15 and 3. Obviously two opposing effects are present. The greater aerodynamic damping ratio occurring at lower wind speeds (Figure 7.11) compensates the larger dynamic amplification for higher peak frequencies as seen without aerodynamic damping. For illustration the parameters of three sea states are given. The high dynamic amplification at parking is associated with small but very frequent waves whilst the sea states during the few days with severe storms exhibit a lower DAF.

**Stress range distributions and effect of availability on fatigue response**

Figures 9.17 and 9.18 plot the cumulative Rainflow counts of the bending stresses at two locations, close to the tower top and below the mudline. In addition, the damage equivalent stress ranges for a S-N curve with inverse slope of 4 are shown.

Five different combination of omnidirectional and collinear loading are investigated:

No. 1: Response on pure wind excitation during production at 100% availability

No. 2: Pure wave response without aerodynamic damping

No. 3: Wind and wave response at 100% availability, i.e. combined wind and wave response in the production wind speed range and pure wave response below cut-in and above cut-out wind speed

No. 4: Wind and wave response at 85% availability, i.e. as No. 3 but pure wave response without aerodynamic damping when turbine unavailable

No. 5: Wave response in the production wind speed range with aerodynamic damping generated by operation of the rotor in a steady wind field

No. 3 and No. 4 represent actual fatigue conditions with different availability whilst the other cases are fictitious. A comparison of the cases 1 and 5 reveals the relative contribution of the aerodynamic and the hydrodynamic loading. At the monopile the wave loads dominate while the opposite holds for the upper tower part. It should be noted that the wave loading of the tower is purely caused by the inertia loads of the dynamic response.

The difference of conditions 2 and 5 is an indication for the dynamic amplification of the wave response in the production wind speed range if no aerodynamic damping is present. Both cases are not fully compatible since case 2 includes also the wave response below cut-in and above cut-out winds. The ratio of total number of cycles and total duration corresponds to 95% of the fundamental eigenfrequency of the monopile structure. This is again a consequence of the large dynamic response.

The total number of cycles of the cases with wind contribution, i.e. No. 1, 3, and 4, is driven by the excitation with the rotor frequency 1P and its multiples. The figures correspond to approximately 4P and 2P at the upper tower part and the monopile, respectively.

As mentioned in Section 8.1 both an **upper and a lower availability** should be compared. For the present case the contribution of the wave response without aerodynamic damping is so significant that for both cross sections a lower availability (No.4) results in a shorter lifetime than full availability (No.3).
Figure 9.17: Cumulative Rainflow counting and damage equivalent stress ranges of bending stresses at a location near the tower top of the Opti-OWECS monopile design.

Figure 9.18: Cumulative Rainflow counting and damage equivalent stress range of bending stresses at a location below the mudline of the Opti-OWECS monopile design.
Lateral response and misalignment of wind and waves

So far we have shown only results for the fore-aft response and for collinear wind and wave loading. Lateral wind speed variations, variation of the aerodynamic loading of the different rotor blades, mass imbalance of the rotor and especially the negligible aerodynamic damping of the lateral motion cause a significant aerodynamic response in the lateral direction. Specific to offshore wind energy converters is the lateral excitation by waves that are misaligned to the rotor orientation and the spreading of wave energy with respect to the mean wave direction (short-crestedness).

Figure 9.19 presents the distribution of the damage equivalent stress range ($\mu = 4$) along the circumference of the critical cross section of the monopile below the mudline. In all considered cases the maximum response occurs in the fore-aft direction. At the pile the lateral aerodynamic response, No. 1, accounts to approximately one third of the fore-aft response. In the upper tower part (not shown) this ratio increases approximately to one half. If long-crested waves, No. 2, approach always from the same direction, no lateral response occurs. Therefore the lateral response of the case No. 4 with collinear wind and wave loading at 85% availability is equal to the lateral response for pure wind loading. Most realistic is the case No. 4a where a misalignment of the mean wind and wave direction according to the distribution from Figure 4.18 is taken into account. The lateral response is more than double compared to the collinear condition but the fore-aft response is not altered significantly.

As explanation of this phenomenon the distribution of the damage equivalent stress range is shown for five different angles between the wind and the wave direction (Figure 9.20). Up to a misalignment of $\pm 30^\circ$, which represents 82% of the time, the total response in the wave direction is not altered significantly but the distribution along the circumference flattens. For large misalignments of 60$^\circ$ and 90$^\circ$ the absence of the aerodynamic damping increases the wave response and it exceeds the maximum total loading for collinear conditions.

Short-crestedness has not been investigated, yet. The above results indicate however similar behaviour for typical conditions where the wave energy is distributed relatively narrow around mean direction (Section 4.1.2).

**Directionality**

Obviously, the magnitude and frequency of the wind and wave loading at a particular site depends on the direction and the associated fetch length and water depth. Figures 4.16 and 4.17 show the variation of the significant wave heights for different directions and prevailing wind and wave directions close to the NL-1 site. Consideration of directionality in time domain simulations is cumbersome and care is required to achieve conservative results. Figure 9.21 compares the damage distribution along the circumference of the pile for four cases which are discussed in order of increasing sophistication. Firstly, omnidirectional loading is applied. Independently from their actual direction wind and waves are assumed collinear and they approach always from the prevailing wind direction. This method is clearly conservative with respect to the maximum damage and

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6 Due to practical considerations of the computation here the wave direction is maintained and the wind direction and turbine yaw angle are varied.

7 In contrast to the two previous figures we are dealing with a damage distribution because this enables the convenient re-distribution of the fatigue results from the different load sectors.
Figure 9.19: Distribution of damage equivalent stress range on the pile circumference for different loadings ($\mu = 4$) (See Figure 4.18 for misalignment of wind and waves for loading No. 4a.)

Figure 9.20: Distribution of damage equivalent stress range on the pile circumference for different misalignment of wind and wave direction ($\mu = 4$)
Figure 9.21: Distribution of damage on circumference of monopile for four different models for the directionality of the wind and wave loading

should be considered as the default analysis. Secondly, the damage from the previous case is distributed over the perimeter according to the distribution of the mean wind direction. This approach results in the most regular damage distribution but also in the minimum damage in the prevailing direction. Next, each 21 lumped load cases are generated for 6 wind direction sectors of 30° ± 180° (Table B.2 in Appendix B) and 224 cases with collinear loading are simulated for 40 minutes each. Figure 9.19 demonstrated an increase of the lateral response when wind and wave direction are considerably misaligned. Therefore in the final approach the maximum damage is calculated from the lumped load cases for each of the six load sectors and subsequently the relative distribution of the damage on the circumference corresponding to case No. 4a from Figure 9.19 is accounted for. This procedure yields the most realistic damage distribution as long as short-crestedness is not considered additionally. The maximum damage is only half in comparison to the omnidirectional loading but the required quality of the correlated wind and wave data and computational effort are high.

9.6.2 Extreme response
It is beyond the scope of the thesis to discuss the extreme response of offshore wind energy converters in general and for all reference designs. Instead examples are given for the extreme loading during power production and during 50 years survival conditions.

Extreme loading during power production

The establishment of the extreme aerodynamic and hydrodynamic conditions during power production has to consider in principle the dynamic response rather than the

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8 The directional wind distribution is considered because it is more pronounced than the wave distribution in Figure 4.17. The fatigue of the steel pile does not depend on the sign of the mean loading hence opposite sectors are treated together.
extreme environmental parameters. So far, existing standards, as for instance the
Germanischer Lloyd guidelines, propose the combination of an extreme aerodynamic
event with a moderate hydrodynamic condition and vice versa. This is appropriate only
if one of the two effects is dominating which is indeed the case at the following example
of the Opti-OWECs monopole design. If wind and wave induced response are of similar
magnitude, various circumstances are imaginable which may lead to a higher response
than the cases with one dominating load component. Further development of structural
reliability methods is required for safe design of multi-megawatt OWEC at demanding
sites.

Germanischer Lloyd proposes the offshore load case E1.1 as 'Extreme operating gust
of 13 m/s with directional change and a reduced wave loading 1.32 \cdot H_s at a sea state
associated with the mean wind speed'.

The characteristic bending stresses at the pile section below the mudline for different
loadings are shown in Figure 9.22. The stress levels are far lower than the yield and
buckling strength, so, fatigue and the dynamic requirements are the design drivers for
the support structure (Figures 10.11 and 10.13). The total response is driven by the
aerodynamic contribution owing to the five times greater magnitude of the wind turbine
induced response compared to the wave case. Moreover, above the rated wind speed
the responses show a reverse dependency on the mean wind speed.

For this pitch regulated turbine the highest wind response occurs at a mean wind speed
at or below the rated wind speed. The controller is not fast enough to increase the pitch
angle according to the instantaneous wind speed and a high overshoot of the tower top
motion takes place. Furthermore the magnitude of the directional change, as specified
by Germanischer Lloyd, is very large in this range (Table 4.2).

The negative overshoot of the tower motion at low mean wind has several reasons. During
the first few seconds of the gust the axial thrust loading increases but is followed
then by a rapid decease below zero due to the large yaw misalignment of the flow and
the initiated shut-down of the turbine. The flow conditions during pitch action and the
employment of the mechanical brake can cause a transition from the turbine to the
propeller stage (Figure 9.23). After comparison of different settings, the combination of
a moderate pitch rate of 5°/s and braking with twice the nominal torque was chosen.

The wave loading increases almost linearly with the mean wind speed. This could be
expected for the quasi-static loading since it is approximately proportional to the wave
height. The dynamic wave loading behaves similarly owing to the almost constant DAF
shown in Figure 9.16.

The operational load case E1.1 produces the highest extreme loading for the design. In
comparison the maximum characteristic stresses associated with the extreme gust load
case E2.1 and the extreme wave load case E2.2 at 50 years survival conditions are only
in the order of 52 and 60 MPa, respectively.

*Extreme loading during 50 years storm conditions*

In comparison to the other reference designs the Utgrunden case is most interesting
with respect to the extreme wave loading in shallow waters and the ice loading. The
design wave height of 6.4 m corresponds to the breaking wave height for a water depth
of approximately 8.25 m which is within the range of water depths at the site. Actually
the wind farm is erected on a subsea ridge and wave breaking may occur. Also the
monopile with its soft-soft characteristics is sensitive to dynamic wave loading.
Figure 9.22 Extreme stresses of the Opti-OWECS monopile below mudline for the load case E1.1: 'Extreme operating gust and reduced wave' at different wind speed

Figure 9.23: Time series for load case E1.1: Extreme operating gust at a mean wind speed of 12 m/s (no hydrodynamic loading)
Figure 9.24: Comparison of load magnitudes at the monopile of the extreme load cases at the Utgrunden wind farm in 10 m water depth (MSL)

As a matter of fact both conditions are not fully compatible with common engineering analysis methods. On one hand the calculation of the dynamic wave loading should consider the randomness of the extreme sea state which is generally only possible by the use of the linear wave theory in its pure form or corrected by an empirical modification, e.g. the Wheeler stretching (Section 4.1.2).

On the other hand non-linear wave theories are commonly capable only of a single deterministic wave with a prescribed height and period rather than an entire sea state. Of course, it is possible to calculate the dynamic response on a non-linear wave in the time domain. However, the response is very sensitive to the chosen wave period and it is unclear whether the transient or the steady-state response is more realistic. Calculations for the present monopile design showed only a moderate dynamic amplification of the transient response in the order of 1.5 during the first wave cycles of long waves but unrealistic high dynamic amplification factors of up to 16 (!) for the steady state response of shorter waves when the second harmonics coincides with the fundamental natural frequency.

During the preliminary design phase of the Utgrunden project a pragmatic and conservative way was followed to estimate the extreme wave loading [9.15].

A generalized dynamic amplification factor (DAF) of the extreme wave loading of 3.4 was determined from the comparison of simulations with and without structural dynamics. The DAF was related to the median value of the extreme overturning moment of 10 simulations of the extreme sea state with 3 hour duration each. In parallel, a non-linearity operator of 2.1 was derived from the comparison of the quasi-static overturning moment according to a 10th order Stream function wave (Figure 5.15) and from application of linear wave theory with Wheeler stretching. By this two new load cases were synthesised. The so-called 'E2.2 Sea state - pseudo non-linear' case comprises the nearly linear response from the sea state loading multiplied by the non-linearity operator. Secondly, load case 'E2.2 Design wave - pseudo dynamic' combines the quasi-static, non-linear wave loading multiplied by the dynamic amplification factor from the extreme sea state analysis.
Figure 9.24 compares the relative magnitudes of the design loads of the different load cases and load origins. The static sea ice loading for 40 cm of ice with a crushing strength of 2 MPa dominates the monopile bending below the mudline. In contrast to the Opti-OWECS monopile the loading at extreme wind speed conditions is more severe than during operation. The magnitude of the extreme sea state response is similar to that of the design wave. Multiplication of the design wave loading with the large value of the above-mentioned dynamic amplification factor results in the most onerous wave loading of the two constructed load cases.

The above-mentioned treatment is conservative since distinct loading effects and frequency ranges are associated with the non-linearity and the dynamic amplification of the response.

The non-linear contribution of the response is mainly related to the quasi-static loading by the non-linear drag at the low frequency, high energy part of the wave spectrum. In contrast, the dynamic response mainly occurs in the vicinity of the fundamental eigenfrequency located in the high frequency, low energy tail of the wave spectrum. In this frequency range the wave loading is inertia dominated and almost linear. Figure 8.6 shows typical wave energy and response spectra for such a case.

Instead of the multiplication of the non-linear, quasi-static response with the dynamic amplification factor it would be more appropriate and less conservative to add the additional dynamic response to the quasi-static, non-linear response. Reference [9.16] discusses this problem in more detail and compares two suitable engineering methods.

9.7 Comparison of design drivers for the support structure

At the end of the chapter an overview of the design drivers and some complementary results for the support structures of the seven reference designs is given. Further information can be found in previously published material [9.1, 9.5, 9.15, 9.17 - 9.19].

Soft-soft and soft-stiff monopiles at the wind farm Lely in the IJsselmeer

The Lely monopiles A2 and A3 were designed with respect to the dynamic requirements. The highest extreme loading with a utilisation of approximately 50% is related to static ice loading at this fresh water site in combination with an extreme wind speed. Only wind turbine induced fatigue was checked in the original design calculations [9.3].

Dynamic analysis of the combined wind and wave response at the soft-stiff design A3 showed two times higher equivalent fatigue stresses than for pure wind loading. The fatigue safety factor is however still large. The monopile A2 with an indented soft-soft characteristics is subject to an extensive wave induced fatigue if the soft-soft behaviour would actually exist. The total aerodynamic and hydrodynamic fatigue loading is about four times higher than for the stiffer companion A3. This amazing result is caused by the low eigenfrequency, very high hydrodynamic participation factors due to a too small penetration depth (Table 9.4) and the short waves on the IJsselmeer.

The fatigue performance of the fictitious design A2-mod is significantly improved; the equivalent loading is only half of design A2.

The foundation sensitivity of design A2 is unacceptable while A3 and A2-mod show moderate sensitivity. The reasons for the actually much stiffer behaviour of the A2 design are still unclear and are subject to further investigations.
Soft-soft monopile at the Utgrunden reef

The weight per unit length of the monopiles at Utgrunden and at Lely A3 is equal, nonetheless, a wind turbine with three times higher rating and an approximately 50% greater hub height (MSL) is employed 6 years later. This was possible since the monopile was better balanced with respect to the dynamic and strength requirements and was designed as one unit together with the tower. The variable-speed, variable-pitch regulated, three-bladed turbine shows lower loads in relative terms than the active-stall regulated, two-bladed Lely design.

The Utgrunden support structure is governed by the extreme sea ice and wind speed condition. The aerodynamic contribution dominates the fatigue loading because the wave regime is moderate and the foundation is relatively stiff and shows very low hydrodynamic participation factors (Table 9.4). The design is appreciably insensitive to variation in the soil properties.

Gravity lattice tower at NL-5 site with WTS 80 turbine

Experience from the offshore technology was used for the design of the lattice tower at the most demanding site. The transparent structure attracts only low wave loads and little dynamic amplification is observed, thanks to the stiff characteristics. The dynamics are driven by the high amplification of the aerodynamic loads at the strong 2P excitation of the blade passing and the stress concentration at the tubular joints. A fatigue analysis showed the need for major structural reinforcements in the upper truss.

At extreme conditions the aerodynamic base shear account for only 13% of the wave and current loading. The size of the caissons at the three legs is determined by the sliding resistance and the overturning moment. The foundation sensitivity is moderate for the first fore-aft mode but quite high for the second mode.

Soft-stiff gravity monotower at the Baltic Blekinge site with the WTS 80 turbine

The gravity monotower is, likewise to the lattice tower, dominated by the dynamic response at the blade passing frequency of the turbine. The damage equivalent overturning moment for combined wind and wave loading is only 4% higher than for pure wind loading. The gravity base experiences a very considerable heave force during the passage of the extreme design wave which can be countered only by 3,000 t of ballast. Like the lattice tower, the foundation sensitivity is moderate for the first fore-aft mode but quite high for the second mode.

Soft-soft monopile at NL-1 site with the WTS 80M turbine

The final design solution of the Opti-OWECS project was found after an integrated design process (Chapter 10) and is dominated by wind and wave induced fatigue. The soft-soft characteristics is a compromise between the wind turbine and support structure requirements and is possible only with a higher rotor speed of the turbine. A tapered monopile with a small water-piercing cross section is needed to keep the wave loading within acceptable bounds. The original design from the Opti-OWECS project was found to require some small reinforcement after considering a poor availability of 85% of the turbine and lower structural damping of only 0.5%. The increased loading is purely caused by a higher contribution of dynamic wave response at parking. A water depth of 20 m (MSL) at good soil conditions clearly limits the viability of the concept.

The highest extreme loads occur during turbine operation at partial load conditions when wave loading is low. The foundation sensitivity is low for medium dense to dense sand.
CHAPTER 10

DESIGN OPTIMISATION BY MEANS OF TAILORED DYNAMICS

Tailoring the dynamics of an offshore wind energy converter can offer an effective design optimisation. The present chapter demonstrates this statement by following the design of a large-scale offshore wind farm (Figure 10.1) through three successive stages of the design process: feasibility study, conceptual design and detailed design (Sections 10.1 to 10.3).

The chapter integrates results of previous chapters. Consideration is given to the integrated design approach from Chapter 3, modelling appropriate to the stages of the design process (Chapters 4 to 6), the analysis methods for aerodynamic and hydrodynamic fatigue introduced in Chapters 8 and the interrelation of dynamics and design as discussed in Chapter 9.

In addition to the top-down or vertical approach following the design through its development process, also a thematic or horizontal structure is given to the chapter. In each design phase we will present and demonstrate a suitable treatment of the essentials of dynamics and design:

• environmental description
• dynamic characteristics
• aerodynamic damping
• foundation uncertainty
• fatigue and extreme response
• economic evaluation

Figure 10.1: Artist impression of the Opti-OWECS design solution
<table>
<thead>
<tr>
<th>Table 10.1: Modelling and analyses of OWEC dynamics and design within the integrated design approach</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Feasibility study</strong></td>
</tr>
<tr>
<td><strong>Environmental description</strong></td>
</tr>
<tr>
<td><strong>Structural model</strong></td>
</tr>
<tr>
<td><strong>Dynamic characteristics</strong></td>
</tr>
<tr>
<td><strong>Aerodynamic damping</strong></td>
</tr>
<tr>
<td><strong>Foundation uncertainty</strong></td>
</tr>
<tr>
<td></td>
</tr>
</tbody>
</table>
Table 10.1 (cont.): Modelling and analyses of OWEC dynamics and design within the integrated design approach

<table>
<thead>
<tr>
<th>Feasibility study</th>
<th>Conceptual design</th>
<th>Detailed design</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Fatigue</strong></td>
<td>short-term superposition of separate wind and wave fatigue analyses in time and frequency domain, respectively</td>
<td>short-term superposition of separate wind and wave fatigue analyses in time and frequency domain, respectively</td>
</tr>
<tr>
<td><strong>Extreme response</strong></td>
<td>spreadsheet like analyses or scaling from similar condition by rule of thumb</td>
<td>standard design tools: quasi-static analysis of wave response plus Dynamic Amplification Factor, dynamic analysis of wind response</td>
</tr>
<tr>
<td><strong>Economics</strong></td>
<td>global cost model, screening of possible sites with a Graphical Information System</td>
<td>detailed engineering cost model</td>
</tr>
</tbody>
</table>
Table 10.1 provides an overview on the proposed approaches. While the first five aspects have also been addressed in the previous chapter, the economic evaluation is introduced here for the first time. For completeness also the structural modelling is mentioned in the table. The design solution is very similar to that of the Opti-OWECS project, an offshore wind farm with 100 units of 3 MW with the WTS 80M wind turbine and a soft-soft monopole at the Dutch North Sea site NL-1 [10.1]. In the scope of this thesis the methods used and some assumptions are updated and the design process is repeated. Therefore slightly different results are established.

10.1 Feasibility study

The purpose of the feasibility study is a rough estimate of investment and energy costs for a number of potential sites and associated wind farm designs. In our case one site and two preliminary design concepts are the outcome which are further developed during the conceptual design phase.

10.1.1 Treatment of dynamics and design

Environmental description

During the early design stage correlated wind and wave data or even any specific data for the particular site might not be available and one has to use one of the following simplified environmental models listed in order of increasing complexity.

- Weibull distribution of the mean wind speed, Neumann-Pierson relation, \((10.1)\), giving significant wave height, \(H_s\), and zero-crossing period, \(T_z\), for fully developed sea states in deep water as function of the hourly mean wind speed at 10 m height, \(V_{1h}\), \((\text{Figure 4.16})\) [10.2, 10.3]. One should be aware of the conservatism of the formulas when they are applied to fetch limited sites in shallow waters.

\[
H_s = \frac{0.243}{g} V_{1h}^2 \quad T_z = \frac{5.48}{g} V_{1h} \quad (10.1)
\]

where: \(g\) gravity acceleration

- Weibull distribution of the mean wind speed, Weibull distribution of significant wave height in European waters derived from voluntary ship observers [10.2] or similar data from the Baltic Sea Handbook [10.4] in combination with an empirical relation between significant wave height and zero-up crossing period (wave steepness)

- Weibull distribution of the mean wind speed, two-dimensional \(H_s - T_z\) scatter diagram

In the last two cases a monotonic wave height - wind speed relation can be derived by equating parameters with the same cumulative probability of occurrence. Indicative values for extreme environmental conditions can be gained, e.g. from the Guidelines of the UK Department of Energy [10.5] or the Baltic Sea Handbook [10.4]. Both for fatigue and extreme conditions the shallow water breaking wave limit (Section 4.1.2) should be accounted for in order to avoid unrealistic large wave heights and too
long wave periods. In all cases wind and waves are considered as collinear and omnidirectional.

**Dynamic characteristics**

Various textbooks provide closed-form estimates for the fundamental eigenfrequency of the support structure. Approximative numerical methods like the Rayleigh quotient can be implemented easily within a spreadsheet that can also be used for a simplified strength analysis.

Consideration of the soil characteristics can be more important than the detailed mass and stiffness distribution along the structure. For gravity systems the foundation stiffness can be estimated by Eqn. (5.6) whilst for piled foundations the concept of the apparent fixity length (Section 5.2.2 and 9.3.1) is extremely useful.

The hub height is a design parameter with strong influence on the dynamic characteristics. At offshore sites a lower height than on land is advantageous. During the Opti-OWECS project the minimum hub height was determined as:

\[
\text{tidal amplitude} + \text{surge} + \text{extreme crest height (approximately } \% \text{ of extreme wave height)} + 1.5 \text{ m air gap according to Germanischer Lloyd} + \text{platform height (0.5 m)} + 4.5 \text{ m minimum blade clearance} + \text{rotor radius} \quad (10.2) \]

minimum hub height above MSL

The minimum blade clearance was established after consultation with Germanischer Lloyd and in reference to a local German building rule [10.6]

**Aerodynamic damping**

As discussed in Section 8.2 the assumption of only two values of the aerodynamic damping ratio, corresponding to the idle condition and the average behaviour during power production of the OWEC, is reasonable at this stage of the design process. The closed-form relation (5.12) can be evaluated. However, the dynamics of the pitch control system (if any) and the effect of the aerodynamic drag on the damping of stall regulated turbines is not included in this way.

**Foundation uncertainty**

During the preliminary design only rather general soil condition data are available. Instead of detailed parameter studies on the foundation behaviour it is more meaningful to assume directly an uncertainty range for the fundamental eigenfrequencies based upon experience. Without further knowledge of the actual conditions a tolerance of 15% relative to the design eigenfrequency is recommended.\(^1\) Such a large uncertainty margin is actually required with respect to the rough assumption and limited data of the early design stage. This conservatism could, however, hinder to pinpoint the really interesting design concepts.

\(^1\) Positive and negative eigenfrequency tolerance are stated with equal magnitude since the eigenfrequency depends decreasingly on the foundation stiffness. For instance, a variation from 50% to 300% of the reference soil stiffness may result in approximately ±5% change in the eigenfrequency (Figure 9.13).
\[
\begin{align*}
\frac{f_D}{f_{R,\text{min}}} & \leq \frac{1}{(1 + \zeta_{1P})(1 + \eta)} & \text{soft-soft design} \\
\frac{f_D}{f_{R,\text{max}}} & \geq \frac{1}{(1 - \zeta_{1P})(1 - \eta')} & \text{soft-stiff design} \\
\frac{f_D}{f_{R,\text{min}}} & \leq \frac{N_b}{(1 + \zeta_{N_bP})(1 + \eta')} & \text{stiff-stiff design} \\
\frac{f_D}{f_{R,\text{max}}} & \geq \frac{N_b}{(1 - \zeta_{N_bP})(1 - \eta')} & \text{(10.3)}
\end{align*}
\]

where: 
- \(f_D\) design value of eigenfrequency
- \(f_{R,\text{max}}, f_{R,\text{min}}\) upper, lower rotor frequency
- \(N_b\) number of blades
- \(\zeta_{1P}, \zeta_{N_bP}\) exclusion range around \(f_R\) and \(N_b f_R\)
- \(\eta, \eta'\) structural uncertainty of eigenfrequency

We denote these dimensionless, positive tolerances as \(\eta\) and \(\eta'\). Even under bad conditions the eigenfrequency should not fall into the exclusion ranges, \(\zeta_{1P}\) and \(\zeta_{N_bP}\), around the rotor frequency, \(1P\), and blade passing frequency, \(N_b P\). Figure 10.2 illustrates the three ranges for the design value \(f_D\) of the fundamental eigenfrequency as well as the exclusion and uncertainty ranges.

Once the first eigenfrequency calculations for a preliminary design are available, the tolerance could be related to the structural behaviour. The difference of the eigenfrequencies between a flexible and a rigid foundation gives a good indication of the
sensitivity of the design. For a specific site, a foundation tolerance $\eta^*$ of approximately half of this distance should be a safe estimate, substituting the previously mentioned general value of 15%. For individual tolerances the design ranges of the fundamental eigenfrequency, $f_D$, can be determined by Eqn. (10.3). Default design ranges for two and three-bladed rotor concepts can be read from Table 10.2.

Table 10.2: Indicative design ranges for the first eigenfrequency, $f_D$, relative to the rotor frequency, $f_R$ ($\zeta_{IP} = 5\%, \zeta_{NIP} = 10\%, \eta = \eta^* = 15\%$)

<table>
<thead>
<tr>
<th>Rotor</th>
<th>Dynamic characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>soft-soft</td>
</tr>
<tr>
<td>two-bladed</td>
<td>$f_D \leq 0.83$</td>
</tr>
<tr>
<td>three-bladed</td>
<td>$1.24 \leq \frac{f_D}{f_R}$</td>
</tr>
<tr>
<td></td>
<td>stiff-stiff</td>
</tr>
<tr>
<td></td>
<td>$\frac{f_D}{f_{R,\min}} \leq 1.58$</td>
</tr>
<tr>
<td></td>
<td>$2.61 \leq \frac{f_D}{f_{R,max}}$</td>
</tr>
<tr>
<td></td>
<td>stiff-stiff</td>
</tr>
<tr>
<td></td>
<td>$\frac{f_D}{f_{R,\min}} \leq 2.37$</td>
</tr>
<tr>
<td></td>
<td>$3.92 \leq \frac{f_D}{f_{R,max}}$</td>
</tr>
</tbody>
</table>

Fatigue and extreme response

The ratio between wind, wave and ice loads should be studied for a couple of extreme load cases.² Concerning the fatigue loading the simplified analysis based upon the superposition of equivalent long-term fatigue loads associated with the wind and wave response is most suitable (Section 8.2). In such an early design stage the required design data for time domain simulations of the aerodynamic response might not be available or the level of detail involved in such computations might not be appropriate. In such cases it could be worthwhile to derive the loads by a rule-of-thumb from previous designs or different site conditions. Distinct support structure types and different hub heights can have a large impact on the fundamental eigenfrequency and consequently on both extreme and fatigue loading. This should be considered when estimating the preliminary load assumptions.

Economic evaluation

For a commercial development, economic criteria related to the entire offshore wind farm must be qualitatively evaluated right from the beginning. Various established approaches exist in the financing field for the determination of the profitability of an investment. Classical methods for Discounted Cash Flow Analyses include the Net Present Value Method ³ and the Yield Method ⁴ [10.7].

---

² For instance, referring to Table 4.4 the load cases E1.1, E2.1, E2.2 and if relevant E2.4 according to Germanischer Lloyd could be analysed.

³ The expected annual income from an asset is discounted, i.e. reduced, to allow for the delay in receiving that income, using a prescribed rate of interest. If the total of these discounted annual amounts exceeds the capital sum needed to buy the asset now, the investment may be considered profitable.

⁴ One determines at what rate of interest the total discounted annual amounts of income would be exactly equal to the capital sum needed to buy the asset now. If the rate of interest, or yield, is greater than the market rate, the investment may be regarded as profitable.
In this thesis the economic question is stated with a slightly different motivation. Economics, or more precisely investment and energy costs, are considered as important criteria in the evaluation of the performance of a particular design. Moreover, these economic quantities are used for the choice between different design options and the (global) comparison between the cost of offshore wind energy and other energy sources. For such engineering applications the method of the levelised production cost has been established by the International Energy Agency (IEA) [10.8].

The levelised production cost (LPC) is defined as the average cost per produced energy unit over the economic lifetime of the plant. The total net energy output and the total cost over the economic lifetime, i.e. capital, running and decommissioning costs, are discounted by a chosen test discount rate to the start of the operation. Then the LPC is obtained as ratio of discounted annual cost and net annual energy output. This approach is based upon a constant amount of capital, the annuity, that has to be returned to the lender each year. Energy yield and all other costs except the initial investment can vary in principle during the lifetime; however, their effect on the energy cost is levelised over the lifetime. Assuming constant net annual energy yield and constant annual operation and maintenance cost results in Eqn. (10.4) and (10.5).

\[
LPC = \frac{I}{a \ E_y} + \frac{TOM}{E_y}
\]

where: 
- \(LPC\) levelised production cost
- \(I\) investment incl. interest during construction
- \(a\) annuity factor \(a = \left(1 - \frac{1}{(1 + r)^{n_e}}\right) \frac{1}{r}\) \(\text{(10.4)}\)
- \(r\) test discount rate
- \(n_e\) economic lifetime in years
- \(TOM\) total levelised annual 'downtime cost'
- \(E_y\) annual energy yield

\[
TOM = OM + \frac{DC}{a (1 + r)^{n_e}}
\]

where: 
- \(OM\) annual operation & maintenance cost
- \(DC\) net decommissioning cost

Typical values for the economic parameters used at commercial wind farms can be found in Table 10.3 [10.9]. In accordance with the first large-scale project in Denmark we apply a test discount rate of 5% and 20 years loan.
Table 10.3: Typical test discount rates and repayment periods

<table>
<thead>
<tr>
<th>Country</th>
<th>Test discount rate</th>
<th>Repayment period</th>
<th>Annuity factor a</th>
</tr>
</thead>
<tbody>
<tr>
<td>Denmark</td>
<td>7%</td>
<td>20 years</td>
<td>10.6</td>
</tr>
<tr>
<td>Germany</td>
<td>varies, 5% upwards</td>
<td>10 years</td>
<td>7.7 or lower</td>
</tr>
<tr>
<td>The Netherlands</td>
<td>4 to 5%</td>
<td>10 years</td>
<td>7.7 to 8.1</td>
</tr>
<tr>
<td>United Kingdom</td>
<td>developer's choice</td>
<td>15 years</td>
<td></td>
</tr>
<tr>
<td>IEA Recommendation</td>
<td>5%</td>
<td>20 years</td>
<td>12.5</td>
</tr>
</tbody>
</table>

† rate at which the nominal rate exceeds the inflation rate
‡ for comparison of different energy sources

Cost models

The LPC can be evaluated even by hand if the different quantities are known. An estimate of the investment cost is however a difficult task which can be eased by general cost models.
During this research three of such models were used. In 1995/96 Köhler and Pauling developed the SCOpiM (Simple Cost Optimisation Model) code [10.10 - 10.12]. During Opti-OWECS project a more advanced cost model was applied [10.13].
For this thesis the simplified design tool MonOpt (Monopile Optimisation) was developed. Features of the MonOpt code include:
- Finite Element like model of cantilever structures
- calculation of arbitrary number of eigenfrequencies
- wave force calculation with linear wave theory and Morison equation
- spectral analysis of hydrodynamic fatigue
- direct quadratic superposition of long-term wind and wave fatigue
- extreme wind loading of the tower
- static ice loads according to [10.14]
- linear strength and buckling analysis according to DIN 18800
- support structure cost estimate according to Opti-OWECS cost model
- energy cost evaluation according to IEC recommendations.
Implementation has taken place in MATLAB™ 5.3.

10.1.2 Preliminary design of the Opti-OWECS wind farm

Design objectives

As illustration of the optimisation approach the development of a 300 MW offshore wind farm is taken from the Opti-OWECS study [10.1]. It would be beyond the scope to repeat here a treatment of similar extension as in the European project.
We focus on the dynamics of the support structure and the wind turbine and take the solutions for the grid connection and operation and maintenance from the study. Also, the same design concepts for the support structure, installation and turbine are used. Due to practical reasons we restrict us to the WTS 80 / WTS 80M machine, rated 3 MW with a two-bladed, 80m rotor and variable-pitch, constant speed control.\(^5\) As starting point the WTS 80 with a rated rotor speed of 19 rpm is considered. In the meantime, larger turbines with a lower specific rating and lower dynamic loading due to variable-speed are under development and might be better suited. Such data was however not available for this thesis.

The design goal is to achieve lowest levelised production cost based upon a test discount rate of 5% and 20 years loan. All given energy cost are purely indicative, are related to these financial parameters and neglect grid connection cost on land.

**Pre-selected sites**

Six distinctly different sites spread over northern Europe are selected as candidate sites. Data of Table 10.4 are collected and harmonized from various sources given in the project report [10.1]. The NL-1, NL-5 and SE-1 location are identical to those considered in Chapter 9. The site UK-1 is located in the Wash bay on the British east coast at about 53° N. The German location DE-1 is near the city of Rostock and is chosen because of its low water depth and close distance to shore. In fact, a nearshore site it would not be suited for such a large offshore wind farm.

**Compatibility of concepts and sites - Pre-selection of concept**

Four generic support structure types are initially considered which are qualitatively matched with the sites in Table 10.5.\(^6\)

The *monopile* is suited good to excellent to all sites except of NL-5 where the large water depth and demanding environment would require uneconomic dimensions. The possible presence of boulders or rock covered by glacial sediment at the Swedish site could complicate both driven and drilled installation. At present mainly a soft-stiff solution is of interest. According to Table 10.2 a soft-soft monopile should have a design eigenfrequency below 0.26 Hz if the WTS80 with a rotor speed of 19 rpm is used. High aerodynamic and hydrodynamic fatigue makes such a design less attractive. Different sites are compatible with the *gravity monotower*. At NL-5 the hydrodynamic loads would require a heavy structure. During the Opti-OWEC project it was realised that high waves in shallow waters generate very considerable heave forces which can only be compensated by a large amount of ballast. Thus the suitability for DE-1 and DK-1 is lower, as well.

Section 9.3.2 highlighted the high aerodynamic fatigue loads of designs with natural frequencies in the vicinity of blade-passing frequency. In addition, the *lattice tower* is subject to stress concentrations at the joints. Consequently, the lattice tower might only have some chances at the most demanding NL-5 site. The Baltic sites neither fit to the lattice tower nor to the tripod because the framework of smaller structural members is too sensitive to ice loads.

The *tripod or braced monopile* corresponds ideally to the NL-5 site where a 'stiff' foundation with a 'soft' tower would be beneficial for the dynamics. Previously such a

\(^5\) See Section 9.1 for a further discuss of the particularities of this wind turbine concept.

\(^6\) Table 3.4-1 in [10.1] presents a comprehensive discussion of the pros and cons of various options.
design was analysed for such a location [10.2, 10.15]. At DK-1 the water depth is too low for this concept. During the Opti-OWECS project the gravity monotower showed significant economic disadvantages compared to the monopile. We maintain this specific finding, knowing that recent Danish research with own specific conditions indicates similar performance [10.16]. The monopile is the best concept for all sites except NL-5 where the tripod is more suited. Here we restrict ourselves to the possibilities offered by the monopile since an optimisation of a tripod structure is beyond the scope of this chapter.

**Evaluation of preliminary designs: Modelling**

A parametric monopile design based upon the Opti-OWECS design solution together with the WTS 80 machine with 19 rpm is analysed at the sites. The applied modelling is briefly reviewed.

Wave scatter diagrams are not available for all sites. So the indicative Weibull distributions of the significant wave height derived from voluntary fleet observations [10.2] and data from the Blekinge feasibility study [10.17] are used. The Weibull distributions for the significant wave height and the mean wind speed are correlated according to the methodology described in Sections 8.2 and 10.1.1. A minimum availability of 85% is assumed.

The structure was modelled Finite Element like with aids of the MonOpt code. A constant fixity length of four pile diameters and a total penetration of 25 m are assumed. The maximum bending moment was assumed to occur at a depth of two pile diameters. The effective aerodynamic damping ratio during production is estimated at 2% of critical damping for a first eigenfrequency of 0.3 Hz decreasing to 1% at 0.6 Hz.

Table 10.2 provides the ranges for the design fundamental eigenfrequency, $f_0$:

- soft-soft: $f_0 \leq 0.26$ Hz
- soft-stiff: $0.39$ Hz $\leq f_0 \leq 0.50$ Hz

Damage equivalent tower top fatigue loads are simulated in the time domain for four values of the support structure stiffness and typical offshore turbulence intensities. Interpolation gives the loads for other eigenfrequencies and other annual average wind speeds. Five extreme load cases from Germanischer Lloyd are considered, for which the wind loads are derived from simulations with a soft support structure. The MonOpt code generates the hydrodynamic loads by spectral fatigue analysis or deterministic analysis with linear wave theory and corrections for non-linear waves and dynamic amplification.

Cost assumptions are based on the final Opti-OWECS design with a price level of winter 1997. Wind farm availability is corrected for both annual average wind speed and distance from shore. Likewise electrical transmission losses are accounted for.

**Evaluation of preliminary designs: Analysis**

Comparison of the capital and energy cost in Figure 10.3 reveals the DK-1 and NL-1 site as the favourites combining good wind conditions with moderate distance to shore and environmental exposure. The Dutch site has slightly lower net energy yield (Figure 10.4) and higher support structure cost owing to the greater water depth and higher waves. Both disadvantages have to be paid by approximately 5% higher cost of energy. The Baltic sites SE-1 and DE-1 range on intermediate positions whilst the British site shows the highest cost due to three less favourable site parameters: low annual average wind speed, considerable water depth and large distance to shore.
### Table 10.4: Site data for the feasibility study

<table>
<thead>
<tr>
<th>Quantity</th>
<th>UK-1</th>
<th>NL-1</th>
<th>NL-5</th>
<th>DK-1</th>
<th>DE-1</th>
<th>SE-1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Annual average wind speed at 60m</td>
<td>8.2 m/s</td>
<td>9 m/s</td>
<td>9.5 m/s</td>
<td>9.2 m/s</td>
<td>7.8 m/s</td>
<td>8.4 m/s</td>
</tr>
<tr>
<td>Water depth (MSL)</td>
<td>20 m</td>
<td>17.5 m</td>
<td>25 m</td>
<td>11 m</td>
<td>8 m</td>
<td>15 m</td>
</tr>
<tr>
<td>Distance from shore</td>
<td>30 km</td>
<td>15 km</td>
<td>50 km</td>
<td>20 km</td>
<td>5 km</td>
<td>7 km</td>
</tr>
<tr>
<td>Onshore distance to HV grid point</td>
<td>5 km</td>
<td>5 km</td>
<td>54 - 70 km</td>
<td>5 km</td>
<td>5 km</td>
<td>10 km</td>
</tr>
<tr>
<td>Extreme mean wind speed at 60 m</td>
<td>41.5 m/s</td>
<td>41.5 m/s</td>
<td>41.5 m/s</td>
<td>43 m/s</td>
<td>40.5 m/s</td>
<td>43 m/s</td>
</tr>
<tr>
<td>Extreme wave height</td>
<td>11 m</td>
<td>11.7 m</td>
<td>15.4 m</td>
<td>8.1 m</td>
<td>6.41 m</td>
<td>10.1 m</td>
</tr>
<tr>
<td>Design current</td>
<td>0.8 m/s</td>
<td>0.8 m/s</td>
<td>1.2 m/s</td>
<td>1.0 m/s</td>
<td>0.4 m/s</td>
<td>0.4 m/s</td>
</tr>
<tr>
<td>Surge</td>
<td>2.5 m</td>
<td>3 m</td>
<td>2.5 m</td>
<td>3 m</td>
<td>2.85 m</td>
<td>2 m</td>
</tr>
<tr>
<td>Tidal amplitude</td>
<td>2.5 m</td>
<td>1 m</td>
<td>0.75 m</td>
<td>0.75 m</td>
<td>0 m</td>
<td>0 m</td>
</tr>
<tr>
<td>Weibull scale factor for $H_s$ distribution</td>
<td>1.25 m</td>
<td>1.2 m</td>
<td>1.4 m</td>
<td>1.6 m</td>
<td>0.5 m</td>
<td>0.64 m</td>
</tr>
<tr>
<td>Weibull shape factor for $H_s$ distribution</td>
<td>1.21</td>
<td>1.23</td>
<td>1.23</td>
<td>1.35</td>
<td>1.23</td>
<td>1.23</td>
</tr>
<tr>
<td>Design ice thickness</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.6 m</td>
<td>0.4 m</td>
</tr>
<tr>
<td>Soil conditions</td>
<td>gravelly sand</td>
<td>medium dense sand</td>
<td>medium dense sand</td>
<td>medium dense sand, silted</td>
<td>medium to dense sand, gravel, boulders</td>
<td></td>
</tr>
</tbody>
</table>

† Wave data at SE-1 from measurements, data for DE-1 related to SE-1

### Table 10.5: Compatibility of support structure concepts and sites

<table>
<thead>
<tr>
<th>Concept</th>
<th>UK-1</th>
<th>NL-1</th>
<th>NL-5</th>
<th>DK-1</th>
<th>DE-1</th>
<th>SE-1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monopile (soft-stiff or soft-soft †)</td>
<td>+</td>
<td>++</td>
<td>-</td>
<td>++</td>
<td>++</td>
<td>+</td>
</tr>
<tr>
<td>Gravity monotower (soft-stiff)</td>
<td>+</td>
<td>+</td>
<td>O</td>
<td>O</td>
<td>-</td>
<td>+</td>
</tr>
<tr>
<td>Lattice tower (soft-stiff, piled or gravity)</td>
<td>-</td>
<td>-</td>
<td>O</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Tripod (soft-stiff, piled or gravity)</td>
<td>+</td>
<td>+</td>
<td>++</td>
<td>O</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

† soft-soft dynamic characteristics only promising for rotor speed above 19 rpm
Figure 10.3: Capital cost and energy cost of preliminary designs

Figure 10.4: Energy yield and energy cost of preliminary designs

Figure 10.5: Capital cost for tower and pile including OWEC installation (left axis) and hub height (right axis) against water depth
Similar NL-5 with the prime wind conditions suffers from its remote location leading to higher dielectric transmission losses and lower availability in addition to the high investment for the shore connection and support structure. A cost level competitive to the best two sites would require lower investment and higher energy yield, e.g. due to a taller structure. This could be achieved, for instance, by 30% lower investment for the support structure and 12% more energy, corresponding to an increase of the hub height from 60 m to 100 m. It is unlikely that a tripod design can manage this task alone. More mature technology with higher efficiency and/or higher power will be required.

Now we spend a closer look to the support structure properties in Table 10.6. Generally the stiffness criterion and the aerodynamic or combined wind and wave fatigue determine the design. The tower base diameter is chosen equal or smaller than the subsoil pile diameters. Therefore, it is not possible to push the tower design into buckling determined conditions. The design eigenfrequency of all designs is very close to the exclusion range at the rotor resonance. Large weights and pile dimensions make the soft-stiff NL-1 and the soft-stiff NL-5 not feasible. The other concepts are regarded as reasonable preliminary designs.

For all soft-stiff concepts the minimum allowed hub height with respect to wave and blade clearance, \((10.2)\), is optimal. Only at the shallow site DE-1 a soft-soft monopole with a double conical tower and 80 m hub height is realistic and results in approximately the same energy cost as a shorter soft-stiff solution.

Figure 10.5 plots the pile and tower investment including cost for site investigation and OWEC installation of the soft-stiff designs against the water depth. Both pile and tower costs are increasing proportionally to the water depth. Together the increase is approximately 5% per metre water depth leading to a 60% more expensive structure at the British site with 20 m depth compared to the German site with only 8 m.

The joint between pile and tower is located at the platform level. This means that for larger water depth the increase in hub height results in a longer pile and in a constant tower length since always the minimum hub height is chosen. One might be wondering why the tower cost are still rising with water depth. The reasons lie in the higher wind

| Table 10.6: Main data of preliminary monopole support structures |
|------------------|----------------|-----------------|----------------|-----------------|----------------|
|                  | DK-1 | NL-1 soft-stiff | SE-1 | DE-1 (NL-1 soft-stiff) | DE-1 soft-stiff (NL-5) | UK-1 |
| Hub height above MSL [m] | 56   | 58             | 56   | 54             | 107             | 80   | 60   | 59   |
| Eigenfrequency [Hz]       | 0.40 | 0.40           | 0.39 | 0.40           | 0.26            | 0.26 | 0.40 | 0.40 |
| Pile diameter [m]         | 3.75 | 4.29           | 3.8  | 3.5            | 5.25            | 3.5  | 5.1  | 4.5  |
| Pile wall thickness [mm]  | 75   | 75             | 75   | 70             | 105             | 85   | 80   | 80   |
| Tower mass [t]            | 99   | 98             | 107  | 91             | (393)           | 234  | (106) | 101  |
| Pile mass ‡ [t]           | 311  | 432            | 357  | 248            | (744)           | 309  | (614) | 497  |

\(^{‡} \text{Monopile design not viable under this conditions.} \quad ^{‡} \text{including transition piece}\)
loads for more exposed sites and especially the extra material needed to maintain the soft-stiff characteristics for a taller structure.

**Selection of site and concepts for further development**

The conceptual design should consider either the DK-1 or NL-1 site. The 5% difference in energy cost between both sites is not significant with respect to the accuracy of the estimates. Moreover the onshore grid connection cost at the Danish site is considerably higher and would more than compensate the price difference since a 150 kV connection point for a maximum 250 MW is 54 - 70 km away. A 400 kV grid node is even 150 km further lacking [10.18]. In contrast, heavy industry and an oil fired plant are located at IJmuiden, the shore connection point for NL-1 [10.1].

The availability of a met-ocean database is another scientific argument supporting the Dutch NL-1 site.

Both the soft-stiff and the soft-soft monopile offer opportunities. The conceptual design phase should trace a wind turbine - structure configuration that is better compatible with the high aerodynamic fatigue loads. In particular, a higher rotor speed, which would also result in cost reductions of the wind turbine, should be considered.

**10.2 Conceptual design**

During the conceptual design the specific properties of the selected sites and concepts are considered and the principal dimensions of the designs are determined. Based upon criteria related to the entire wind farm, in our case the levelised production cost, the final design and site are chosen.

**10.2.1 Treatment of dynamics and design**

**Environmental description**

Large offshore wind farms should be designed and certified according to the local site conditions which establish the base for a so-called 'S-Class Certification' according to the Germanischer Lloyd or IEC 61400-1 ed. 2 standard.

If no site measurements are available, dedicated computer models or met-ocean databases have to be evaluated by consultants. In the best case a three-dimensional scatter diagram with classes for significant wave height, $H_s$, zero-crossing period, $T_z$, and mean wind speed at hub height, $V$, is available. Otherwise a two-dimensional wave scatter diagram has to be correlated with the Weibull wind speed distribution by comparing the cumulative distributions (Section 8.2).

If no joint-probabilities for the extreme wind, wave and current conditions exist, the individual extremes have to be combined according to the Germanischer Lloyd's guidelines.

Again wind and waves can be considered as collinear and omnidirectional.

**Dynamic characteristics**

A finite element model of the support structure should be used for the determination of the relevant eigenfrequencies. Evaluation of a Campbell diagram [10.19] reveals possible resonances between natural frequencies and rotor excitations.
Depending on the available design tools and data, the soil stiffness of piled foundation can be either modelled by distributed springs with P-y curves or equivalent foundation springs for the translational and rotational degrees of freedom. Variations in water depth and soil conditions should be accounted for.

**Aerodynamic damping**

The variation of the aerodynamic damping for different support structure concepts and for different mean wind speeds should be studied. Especially for soft designs, potentially prone to hydrodynamic fatigue, a detailed description is required for the determination of the hydrodynamic response. Different methods for the determination of the damping are discussed in Section 7.1.3. Attention should be given to the possible reduction of the damping ratio at post-rated wind speeds due to stall or the dynamics of the control system.

**Foundation uncertainty**

Once soil parameters are available one can investigate the sensitivity of the foundation behaviour with respect to variations in the secondary soil parameters, e.g. the stiffness of the entire soil or of certain soil layers. Variation of the primary soil parameters as the internal friction angle or shear strength is most worthwhile when these parameters have become available from soil investigations. Therefore, such a treatment is proposed at least for the detailed design phase.

Because foundations tend to behave stiffer than expected, the positive tolerance in the soil parameters may be taken as greater compared to the negative tolerance. In the sensitivity study of the reference designs in Section 9.5.2 indicative safety factors of $\gamma_m = 0.5$ and $\gamma_m = 3.0$ have been considered uniformly for all soil layers. After such analysis the design ranges for the eigenfrequencies can be established on a more rational basis than in the previous section.

**Fatigue and extreme response**

Critical load cases should be identified for the different design concepts. Apart from extreme load cases mentioned in Section 10.1.1 other conditions, e.g. emergency stop, (near-)breaking waves, breaking ice, might be of relevance. Superposition of aerodynamic and hydrodynamic extreme responses from separate computations is reasonable. The aerodynamic and hydrodynamic fatigue can be calculated with the simplified analysis based upon the superposition of short-term fatigue (Section 8.1) if correlated wind and wave data exist.

**Economic evaluation**

An OWECS cost model can be of great value also during the conceptual design. While the model might be still used for the overall evaluation, the cost of certain subsystems can be estimated directly by input from the designer or from suppliers rather than by the general cost algorithms of the model.

**10.2.2 Conceptual design of the Opti-OWECS wind farm**

During the conceptual design phase of the Opti-OWECS study a simultaneous optimisation of the wind turbine and of the support structure was carried out. The main goal was to reduce cost by lowering the quite considerable aerodynamic fatigue loads. Therefore a parameter study on aerodynamic fatigue loads as function of rotor speed
and support structure stiffness was carried out by time domain simulations of about 20 different configurations. It was not practical to consider combined wind and wave fatigue. This was a major limitation because softer designs were subject to lower aerodynamic but higher hydrodynamic fatigue. The final Opti-OWECS design combined a higher rotor speed of 22 rpm instead of 19 rpm and a softer monopile structure with a natural frequency of 0.29 Hz instead of the soft-stiff gravity monotower with 0.52 Hz considered as conceptual Opti-OWECS design. The solution was found only after a time-consuming iterative design process including several sets of time domain simulations of the combined response. Nonetheless an approximately 50% (!) reduction of the dimensioning loading was achieved.

In the scope of this chapter the parameter study is repeated by the author. Now it is possible to account for combined fatigue by one of the simplified fatigue analysis methods. For this study with many different configurations the long-term superposition of the aerodynamic and hydrodynamic fatigue (Section 8.2) rather than the above-mentioned superposition of short-term fatigue responses is chosen.

Comparison of rotor design options

In the scope of the Opti-OWECS study, van Rooij [10.1] investigated to reduce the extreme parking and fatigue loads and blade weight by maintaining a reasonable energy yield for constant diameter and rated power. The exercise started with the rotor design of the land-based version of the WTS 80 with 19 rpm. Two higher rotor speeds of 22 and 25 rpm were considered for an annual average wind speed of 8.5 m/s at hub height. The higher rotor speeds result in tip speeds of 92 and 105 m/s which are only feasible due to the relaxed noise constraints offshore. In general, a larger rotor speed decreases the required chord length along the blade span.

Enlarging the rotor speed from 19 rpm to 22 rpm the chord reduction was found to be significant even for a constant maximum chord at the blade root and same airfoils. The new blade design for 22 rpm (Figure 10.6) obtains approximately 1% higher energy yield than the original design. A further reduction of the blade area for rotating at a higher speed of 25 rpm results in a slight change of the maximum power coefficient but the effect on the total energy yield is small. Hence no adjustment of the blade geometry was proposed for 25 rpm.

The result should be seen as the first step towards a new rotor design for offshore application. The weight of the blade and thus also the cost will be reduced for the new chord distribution and the more fatigue benign offshore wind condition. A structural re-design of the internal blade structure was not further investigated since it would require a major involvement of a blade manufacturer.

Comparison of support structure configurations

A fictitious monopile support structure is considered for the parameter study. The stiffness of the reference configuration with $f_0 = 0.4$ Hz is varied by artificially adjusting the modulus of elasticity by a factor between 0.4 and 3 corresponding to a first eigenfrequency between 0.25 and 0.69 Hz. This approach works well for the calculation of the aerodynamic tower top loads, however, results in simplifications concerning the contribution of the inertia loading along the support structure. The damage equivalent tower top loads are generated by time domain simulations for various combinations of stiffness and rotor speed whilst spectral analysis is applied for the hydrodynamic fatigue loads at the NL-1 site with 15 m water depth (Table 8.1). Direct quadratic superposition obtains the combined fatigue loads.
Figure 10.6: Original blade layout of WTS 80 for rotor speed 19 rpm and optimised blade for 22 rpm

Figures 10.7 to 10.9 show the equivalent bending moment of the monopile 8 m below the mudline as function of the support structure stiffness and for different loadings.

Aerodynamic fatigue loads for a rotor speed of 22 rpm (solid line in Figure 10.7) increase rapidly in the vicinity of the strong 2P blade excitation. The quite pronounced amplification of the aerodynamic fatigue loads in the vicinity of the rotor frequency (1P) originates from the mass unbalance of the rotor according to the Guidelines of Germanischer Lloyd. In practice the effect might be considerably lower because the eccentricity of recent wind turbine rotors is 5 to 10 times lower.

Softer configurations exhibit higher hydrodynamic fatigue. The increase of the dynamic amplification is, however, partially reduced due to the higher aerodynamic damping ratio. For the stiffer structures the loading lowers decreasingly towards the quasi-static response (dotted line in Figure 10.7). The ratio between the wave fatigue loading of different pile diameters (dashed lines) is proportional to the square of the diameter owing to the dominant hydrodynamic inertia forces.

In general, wind generated fatigue dominate the investigated configurations. Only for very soft structures and large pile diameters the hydrodynamic fatigue is more important than aerodynamic fatigue.

Figure 10.8 combines the fatigue loads for the two pile diameters from the previous figure and shows the pure aerodynamic loads for a rotor speed of 22 rpm as reference. The quadratic superposition of the load components is clearly visible. A soft-soft solution is beneficial only for a pile diameter which is small, at least close to the water surface, while a soft-stiff design is reasonable also for larger piles. Hence, Figure 10.9
Figure 10.7: Separate wind and wave fatigue loading against support structure stiffness for rotor speed 22 rpm

Figure 10.8: Combined wind and wave fatigue loading against structure stiffness for rotor speed 22 rpm

Figure 10.9: Combined wind and wave fatigue loading as function of support structure stiffness and rotor speed
presents the combined fatigue for a 3 m pile in the soft-soft region (solid line) and for a 4 m pile in the soft-stiff range (dashed lines). Comparison of the three rotor speed variants confirms the prospects of both a soft-soft solution with either 22 or 25 rpm and a soft-stiff design with higher total loads for the original rotor speed of 19 rpm.

Support structure concepts

With aid of the MonOpt code two monopile support structures are designed with a hub height of 58 m (MSL). The water depth corresponds to the maximum value of the NL-1 site ranging between 15 and 20 m (MSL). The modelling from the previous section is mainly followed and is not repeated here except a remark on the design ranges for the fundamental eigenfrequency. The ±15% tolerance for structural uncertainty from the feasibility study is maintained in order to anticipate on the variation in water depth to be considered in the detailed design phase.

The designs are similar to the Opti-OWECS solution (Figure 9.4) with a tapered section below the water surface and a grouted joint between pile and transition piece. The reduction of the water piercing cross section is absolutely required to keep the hydrodynamic loads and required amount of material within an acceptable range. Figures 10.10 and 10.11 compare the geometry and the strength utilisation of the soft-stiff and the soft-soft concept. Both designs are driven by fatigue and stiffness requirements. The fatigue strength utilisation along the height is given in the lowest of the subplots. In addition, the buckling utilisation of the soft-stiff design and the contribution of hydrodynamic fatigue to the total utilisation at the soft-soft design are shown.

Both designs are feasible but show also potential problems. Series installation of the soft-stiff monopile with the diameter of 4.7 m and enormous weight of over 350 t is challenging and beyond the capabilities of most contractors. The soft-soft pile can be easier and faster handled but the large wall thickness at the grouted joint is difficult to manufacture. So, the performed cost estimates, mainly based upon the steel weight, are only indicative.

Evaluation on global criteria

The decision on the final design, the WTS 80M turbine with 22 rpm and a soft-soft monopile is made after a trade-off between the wind turbine and support structure options (Table 10.7). The higher rotor speed yields lower fatigue loads and enables a weight reduction of the rotor blade and the support structure. Extreme operating loads are likely to be higher for this options but in total, fatigue is governing. A saving in the wind turbine investment of 5% is conservatively estimated. In addition, the new blade gains slightly more energy. The soft-soft monopile offers further significant cost reduction. In total, the energy cost is estimated to be approximately 5% lower than for the base case.

The achieved saving in this section is lower than during the Opti-OWECS project where a soft-stiff gravity monowrap with the WTS 80 was compared to the soft-soft monopile with the WTS 80M (Figure 10.9). Thanks to the integrated methodology the relatively stiff gravity monowrap option has already been discarded during the feasibility study in Section 10.1.2 and only an evaluation of two relatively similar monopile designs has been made.
**Figure 10.10:** Soft-stiff monopile concept
(Pile 361 t, transition piece 90 t, tower 101 t)

**Figure 10.11:** Soft-soft monopile concept
(Pile 300 t, transition piece 94 t, tower 98 t)
Table 10.7: Comparison of conceptual designs for NL-1 at 20 m (MSL)

<table>
<thead>
<tr>
<th></th>
<th>WTS 80 (19 rpm)</th>
<th>WTS 80M (22 rpm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>soft-stiff monopile</td>
<td>soft-soft monopile</td>
</tr>
<tr>
<td>Turbine investment</td>
<td>0%</td>
<td>-5%</td>
</tr>
<tr>
<td>Support structure investment &amp; OWEC installation</td>
<td>0%</td>
<td>-8%</td>
</tr>
<tr>
<td>Energy yield</td>
<td>0%</td>
<td>+1.3%</td>
</tr>
<tr>
<td>Levelised production cost</td>
<td>0%</td>
<td>-5%</td>
</tr>
</tbody>
</table>

10.3 Detailed design

During the detailed design the final concept is worked out and drawings, specification and the bill of material are produced. In our case the support structure design is specified for the entire wind farm including variation of the local parameters.

10.3.1 Treatment of dynamics and design

Environmental description

The environmental description should be based upon correlated wind and wave data obtained for the actual site from measurements or hindcasted data (Section 4.2). Consideration of directional data, as shown in Figure 4.17, is desirable because of the large differences in fetch between the landwards and the seawards directions. Integrated non-linear time domain simulation should be used for the final load calculations. So, lumped sea states have to be established for the fatigue calculations, as described in Section 8.3, in order to limit the computational effort to a reasonable extent.

Dynamic characteristics

The sensitivity of the eigenfrequencies and the damping with respect to a systematic variation of all environmental, structural and operational properties should be studied with a finite element code and/or an integrated OWEC design tool. Tables 5.5 and 5.6 list conditions resulting in either minimum or maximum eigenfrequencies and damping. Beforehand it is not always clear whether the highest aerodynamic and hydrodynamic loads will occur for the minimum or the maximum of the expected eigenfrequencies. For fatigue loads which accumulate over long time it might be appropriate to assume average conditions, e.g. mean sea level, moderate marine growth, low lateral pile loading, etc. So different structural models of the OWEC should be established for fatigue and extreme load calculations assuming the most onerous conditions.

Aerodynamic damping

The application of integrated, non-linear time domain simulations ensures the most accurate description of the aerodynamic damping. In particular the effects of stall and of the dynamics of the control system are fully taken into account.
Foundation uncertainty

The most rational approach for estimating the foundation uncertainty is to investigate the sensitivity in the primary soil parameters directly. Taking sand as example, these are the internal friction angle, the modulus of subgrade reaction and undrained density. The behaviour of the entire OWEC is studied also with respect to other uncertainties as scour and settlement. The location and the magnitude of the extreme stresses in the foundation vary for different soil conditions. Analysis of the foundation uncertainty is one part of the above-mentioned systematic investigation of the variation of the dynamic characteristics.

Fatigue and extreme response

If combined aerodynamic and hydrodynamic fatigue is important, it should be analysed by non-linear, time domain simulations of lumped load cases (Section 8.3). Otherwise, one of the simplified analysis approaches might be sufficient for a check of the design. In case extreme loads govern the design, other dedicated analysis methods such as structural reliability methods [10.20, 10.21] or push-over analysis with a non-linear finite element code might be adequate.

Economic evaluation

Investment and energy costs could be calculated on the basis of the estimated component costs. Uncertainties in the gross energy yield, OWECs availability, operation and maintenance cost, investment cost and economic parameters should be evaluated to establish the most likely energy cost and its uncertainty.

10.3.2 Detailed design for the Opti-OWECS wind farm

Variation of soil conditions

In absence of actual soil investigations for different locations of the NL-1 site we assume medium dense sand within the entire wind farm. Referring to Table 4.6 the uncertainty of the soil parameters is established for the subsequent dynamic simulations (Table 10.8). The initial subgrade reaction is varied much larger compared to the values recommended by the API Guidelines [10.22] for a friction angle of 29° and 36°, respectively. For the design calculations the minimum soil stiffness is assumed. In an additional parameter study the sensitivity of the first two fore-aft eigenfrequencies at 20 m water depth (MSL) is investigated for a wider range of soil properties than expected at the site (Figure 10.12). Now the relation between friction angle and

<table>
<thead>
<tr>
<th>Design condition</th>
<th>Submerged unit weight $\gamma_{sub}$ [kN/m$^3$]</th>
<th>Internal friction angle $\phi$ [deg]</th>
<th>Initial subgrade reaction $K$ [MPa/m]</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum soil stiffness</td>
<td>8</td>
<td>30</td>
<td>7</td>
<td>cyclic loading, pre-loading 300 kN</td>
</tr>
<tr>
<td>Maximum soil stiffness</td>
<td>10</td>
<td>36</td>
<td>40</td>
<td>static loading, no pre-loading</td>
</tr>
</tbody>
</table>
Figure 10.12: Sensitivity of the fore-aft natural frequencies of the NL1-20m monopile to simultaneous variation of the internal friction angle and the initial subgrade reaction at a submerged unit weight of 10 kN/m³

subgrade reaction according to API is applied (see lower axis). Acceptable behaviour is observed for medium dense to very dense sand but the frequencies drops dramatically if only loose sand or medium dense sand with portions of silt and clay occur. Under such conditions the structure becomes too soft and is no longer viable as consequence of excessive wave excitation. Further reinforcement of the wall thickness is not practicable and an increase of the outer diameter is more than absorbed by the higher (inertia dominated) wave loading proportional to the square of the structural dimension. In such a situation one has to choose a tripod or gravity monotower as alternative.

The influence of scour is not considered since some form of scour protection is generally required in the Southern North Sea. For this structure with pronounced dynamic behaviour the avoidance of seabed erosion is essential.

Analysis of combined wind and wave fatigue

The soft-soft monopile design is driven by fatigue. Therefore, time domain simulations of the simultaneous wind and wave response with lumped load cases from Table 8.3 are performed. The response during idle or failure state, when no aerodynamic damping is present, is more onerous than the combined aerodynamic and hydrodynamic fatigue during production. As illustration Figure 10.13 shows the fatigue loading along the structure for both 85% and 100% availability. Over the entire length the former conditions results in considerably higher loads.

At the grouted joint and below the seabed the utilisation for omnidirectional loading even exceeds slightly 100%. A proper consideration of the different loading for different directions would require several times higher computational efforts. In a simplified
Figure 10.13: Fatigue loading of the monopile at 20 m water depth for different availabilities (Omnidirectional and collinear wind and wave loading)

approach the effect of the directionality of the loading is estimated. The omnidirectional response is distributed over eight wave direction sectors with the frequency from Figure 4.17. A priori it cannot be judged whether this treatment is conservative, probably it is not, therefore it is used only qualitatively. The damage equivalent stresses are reduced by about 20% and the estimated life is increased by 120 to 140%. The minimum calculated fatigue life occurring at the grouted joint is estimated to be between 17 and 39 years; most likely it is closer to the upper bound of this range and therefore larger than the design life of 20 years.

Local site conditions

So far we have considered only the maximum water depth in order to check the viability of the design concept. During the detailed design the variation of the mean sea level between 15 and 20 m and different soil conditions need attention. We assume that 100 offshore wind energy converters are uniformly distributed over the range of local conditions and that the same met-ocean data are valid all over the site. Adaptation of the support structure to such local conditions should balance logistic and procurement efforts against the possible cost saving owing to tailored design. After a parameter study it is decided to propose two monopile designs suited for water depths of either below or above 17.5 m. The same tower and transition piece can be used, however, the pile length and stiffness varies. With respect to the pile driving equipment it is advantageous to maintain the pile diameter and to restrict the modifications to the pile length and wall thickness. Furthermore a lower wall thickness eases the manufacturing. Both piles have constant length above MSL and below the mudline, as the height of the cylindrical part between the seabed and the tapered section is adjusted to the local water depth.

Table 10.9 compares the main properties and the indicative costs of both designs, denoted as NL1-17.5 and NL1-20. The different water depth and soil conditions result in an approximately 8% to 10% spreading of the first natural frequency. Under all circumstances at least 10% distance to the 1P rotor excitations remains.
A further weight reduction of the monopile might be possible if a reduced wall thickness of, say, 40 mm at the lower half of the penetration depth where pile loads are low and the soil stiffness is high. Such measures require, however, a check of both possible buckling during installation and increased hydrodynamic excitation. The structural modifications affect the energy cost\(^7\) only by a fraction. Nonetheless they are required in order to avoid a too stiff characteristics at low water depths.

<table>
<thead>
<tr>
<th>Property</th>
<th>Design</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>NL1-17.5</td>
</tr>
<tr>
<td>Water depth (MSL)</td>
<td>15 - 17.5 m</td>
</tr>
<tr>
<td>Hub height (MSL)</td>
<td>58 m</td>
</tr>
<tr>
<td>1(^{st}) fore-aft natural frequency†</td>
<td>0.292 - 0.33 Hz</td>
</tr>
<tr>
<td></td>
<td>(0.80 - 0.90 P)</td>
</tr>
<tr>
<td>Pile dimensions</td>
<td>Ø 3.65 m x 60 mm</td>
</tr>
<tr>
<td></td>
<td>x 40 - 42.5 m</td>
</tr>
<tr>
<td>Tower mass</td>
<td>98 t (identical design)</td>
</tr>
<tr>
<td>Transition piece mass</td>
<td>92 t (identical design)</td>
</tr>
<tr>
<td>Pile mass</td>
<td>225 - 239 t</td>
</tr>
<tr>
<td>Investment for support structure and OWEC installation</td>
<td>≈ 1.12 M€</td>
</tr>
<tr>
<td>Levelised production cost</td>
<td>≈ 4.4 €/kWh</td>
</tr>
</tbody>
</table>

\(^†\) including variation of water depth and uncertainty of soil conditions

**General remarks**

The example within this chapter might be driven by the specific wind turbine design. Nonetheless, the demonstrated methodologies for the optimisation of the dynamics of both wind turbine and support structure and the adaptation to the site parameters is promising for different types of bottom-mounted offshore wind energy converters. Most wind turbine manufacturers design standard machines and major modifications of these so-called 'platforms' to tailor them to certain wind turbine classes are limited (Section 3.2). Therefore a project specific design of wind turbine platforms is questionable even for larger offshore projects. Nonetheless, support structures, grid connection and procedures for installation, operation and maintenance will differ from project to project and an integrated design approach is certainly beneficial. Major parameters to be adjusted will include: foundation type, height of the access platform, hub height, dynamic characteristics, tower design, wind turbine control features and possibly small variations of rotor speed, diameter or generator power.

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\(^7\) The average energy cost of 4.5 €/kWh at an annual average wind speed of 9 m/s at hub height corresponds to the estimate of 5.1 €/kWh for 8.4 m/s annual average wind speed of the Opti-OWECs project. After the European project it turned out that the reference height in the NESS database was accidentally taken as too high, which caused a too low estimate of the energy yield.
CHAPTER 11

CONCLUSIONS

The excellent prospects of offshore wind energy depend, from a technical perspective firstly on further reduction of energy cost through innovative design and economies of scale, and secondly upon the integration of large quantities of wind energy generated electricity into the international energy system. The work described in this thesis contributed to the former aspect. More specific conclusions are related to the three major fields of the research. In addition, some recommendations for further work are given.

Design approach for offshore wind farms

The economic exploitation of the offshore wind energy potential requires a symbiosis of wind energy, offshore technology and power management. Although this seems obvious, progress is by no means evident and requires an open attitude in each community.

Design methodologies recently developed for complex civil engineering systems are also adequate for offshore wind farms. Within the novel, integrated design approach for offshore wind energy conversion systems the goals of the design process are controlled effectively on the system level by overall criteria such as levelised energy costs, adaptation to site conditions, installation and commissioning efforts, availability and overall dynamics.

With respect to the economies of scale and the required development time a project specific design of wind turbine platforms is questionable even for larger offshore projects. Nonetheless, support structure, grid connection and procedures for installation, operation and maintenance will differ from project to project and an integrated design approach is beneficial.

An integrated approach is also required when designing wind turbines for the required high reliability and availability at exposed offshore sites.

The application of a common offshore wind energy terminology is valuable and more than pure convention.

A separate handling of tower and foundation might be more convenient in the short-term, but probably only a sub-optimum design solution will be achieved compared to the treatment of the support structure as one part. Likewise, the consideration of the wind turbine and support structure as one entity, the offshore wind energy converter, results in a potentially more appropriate and more economic designs. Design of an offshore wind energy conversion system includes not only the wind turbines and their support structures but likewise the operations and maintenance aspects and the grid connection.
Dynamics of offshore wind energy converters

An integrated, non-linear dynamic model of an offshore wind energy converter (OWEC) in the time domain accounts for the interactions between the subsystems and the significant non-linearities in the dynamics and the aerodynamics of the wind turbine. It is required for certification calculations and analysis of the non-linear response under extreme conditions. The high computational effort for the analysis of the simultaneous aerodynamic and hydrodynamic response under fatigue conditions can be reduced using a new method which constructs a low number of lumped load cases.

For the early design stage, two simplified analysis methods were developed. They are based upon the superposition of the damage equivalent stress ranges from separate analyses of the aerodynamic and hydrodynamic response with standard design tools in the time domain and frequency domain, respectively. The weighted quadratic superposition is founded on a theoretical base but showed larger scatter than the direct quadratic superposition of damage equivalent loads which was established empirically. The approaches require the estimation of the aerodynamic damping of the support structure. A generally available method is the evaluation of transient vibrations in steady wind for a turbine with constant rotor speed or in turbulent wind for variable speed turbines. As a conservative measure only 80% of the estimated damping should be applied as additional structural damping of the fundamental fore-aft mode when analysing the hydrodynamic response alone.

One particular reference design of a pitch regulated, constant-speed offshore wind energy converter with a soft-soft support structure has been investigated with the novel methods. The results show good agreement with integrated, non-linear time domain simulation and suggest that an accuracy of ±10% or better in terms of long-term equivalent loads compared to simulations could be achievable by proper application. Further studies at different configurations are required to judge a general accuracy.

The third, new method of constructing a low number of characteristic load cases by lumping of sea states has been demonstrated as an accurate and effective method to reduce the computational efforts for time domain simulations to a reasonable extent.

In the investigations of the structural dynamics of offshore wind energy converters the support structures was emphasised because it is most specific and most affected by the offshore siting. Its dynamics are largely determined by the combination of:

- first fore-aft and lateral natural frequency,
- associated mode shape and global distribution of support structure stiffness,
- response of the second and higher eigenmodes, which can be important for extreme loading associated with breaking waves and dynamic ice loading,
- magnitude of aerodynamic damping ratio, depending on wind turbine and support structure design and operational conditions,
- sensitivity to loading, e.g. water-piercing cross section of the structure, ice cone
- site specific environmental parameters. (High quality site data are absolutely required for proper support structure design, while the wind turbine could be designed with a generic classification of conditions compatible with the actual site.)
- wind turbine characteristics including control.

Understanding is far from easy but the variety of influences offers opportunities to tailor the dynamics and to optimise the design.
Conclusions

A relatively stiff foundation together with a soft tower lowers the sensitivity to the soil conditions, reduces the dynamic wave loading and can improve the economics. Such an integrally designed structure contradicts the rule-of-thumb from the offshore engineering which says that structures with a natural frequency below 0.4 Hz should be avoided with respect to dynamic wave excitation. Although lower support structure stiffness can be beneficial with respect to the wind turbine induced fatigue loading, it attracts hydrodynamic fatigue and can cause high extreme loads during emergency shut-down or extreme wind conditions. High support structure stiffness may conflict with wind turbine induced fatigue and generates high structural and installation costs.

At demanding offshore sites, the hydrodynamic fatigue loading during parking, when no aerodynamic damping is present, can exceed the combined aerodynamic and hydrodynamic fatigue during power production. Consequently, fatigue analyses have to consider different values of availability. Aerodynamic damping is the most significant source of damping while hydrodynamic damping is negligible for bottom-mounted structures and soil damping of the fundamental mode of piled or gravity systems is low. Dynamic analyses have to account for the uncertainties and variability in the soil properties and met-ocean parameters. Suitable treatments for different stages of the design process have been proposed. Large safety factors should be applied on estimated soil parameters even after in-situ tests and both lower and upper bounds require attention.

Most analyses were carried out for a 3 MW, 80 m two-bladed wind turbine with constant speed - variable pitch control. This uncommon machine concept generates high fatigue loads and does not fully represent the multi-megawatt machines currently under development. The use of three blades, larger rotor diameters, variable rotor speed and advanced control affect lower specific loads and may result in distinct dynamic behaviour. Nonetheless, there are indications that fatigue becomes also the design driver for more advanced multi-megawatt turbines with a lower specific rating, i.e. larger rotor diameters. In such a case the presented results are valid in a qualitative way. It is expected that the monopile concept is technologically and economic viable only up to a water depth of 15 - 20 m (MSL). Up to such water depths, for moderate pile diameters close to the water surface and not too soft structures, the wind or ice loads are higher than wave and current loads. Beyond such conditions the required diameter and/or the wall thickness become too large with respect to installation and construction, respectively.

Tripod support structures are promising for greater water depths and larger turbines. The high stress concentrations at the tubular joints are an inherent disadvantage which is conflicting with the significant magnitude of the fatigue loads for multi-megawatt turbines.

Design optimisation of offshore wind farms

A site specific design and certification is economical attractive for offshore wind farms. The higher mean and lower dynamic wind loads and the very specific hydrodynamic and geotechnical conditions are not compatible with standardised onshore designs. In comparison to the land-based situation, the relaxed noise restrictions, importance of reliability and operation and maintenance aspects and higher installation cost favour eventually distinct wind turbine design for offshore siting.
Dedicated offshore design can be more reliable and safer. The example of the wind farm Lely was used to show how the sensitivity to variations in the soil properties and the fatigue due to wind and wave response can be reduced leading to a more durable design solution.

Design optimisation by means of tailored dynamics can also achieve lower capital and energy costs as illustrated during the Opti-OWECS study, by the development of a 300 MW wind farm in the Dutch North Sea through three successive stages of the design process.

**Recommendations for further research and development**

First experience with the integrated OWECS design approach was gained during the Opti-OWECS project. Obviously such a methodology needs further development by usage in desk-top studies as well as implementation projects.

Under the conditions and constraints of large 'real' projects, it has to be picked out in which form the approach is suitable and compatible with the different organisations involved.

The proposed Design for RAMS (Reliability, Availability, Maintainability and Serviceability) design approach has not been investigated in great depth so far and its procedural as well as scientific development is a worthwhile project of its own.

Establishment and harmonisation of specific standards for offshore wind farms have not progressed very far, yet. The existing guidelines of Germanischer Lloyd and the Design Bases for Offshore Wind Farms in Denmark should be further developed and validated through measurements. Such work has started in the scope of two EU projects and national projects, recently and should be incorporated in the compilation of a separate offshore extension of the international IEC 61400 series.

In the same scope the time domain analysis approach of the overall dynamics should be further refined and verified by measurements. The DUWECs simulation code needs further development. Examples include the aerodynamic and structural modelling of the wind turbine and the tower-nacelle interface.

The risk involved in large offshore wind farms could be reduced by improving the understanding of the offshore wind conditions and associated other met-ocean parameters in order to facilitate more reliable predictions of the energy yield, optimum design and optimum strategy for installation as well as operation and maintenance. The use of high quality met-ocean database, recently developed in the offshore oil and gas industry, can be of great value.

Considerable research is required to develop reliable means of computing the extreme response of offshore wind energy converters. Structural reliability methods recently developed in the field of advanced offshore engineering should be extended to the extreme loading of offshore wind energy converters, especially during production. Loading by (near-)breaking waves in shallow waters and breaking sea ice loads should be investigated to achieve safe but not overly conservative structures.

The use of the wind turbine control system of pitch-regulated designs for lowering of the aerodynamic and hydrodynamic fatigue loads should be studied. Also the investigation of active or passive mass dampers could be worthwhile for reducing the lateral support structure response [11.1, 11.2].
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APPENDIX A: OFFSHORE WIND ENERGY TERMINOLOGY

Offshore wind energy is a fairly young and multidisciplinary field. Unfortunately, no uniform terminology exists and misunderstandings can occur quite easily. Therefore, within the Opti-OWECS project one has agreed upon a particular terminology, conventions and reference systems in order to make the internal and external communication more effective. Here a shortened version of reference [A.1] is presented.

Preface
In principle, the common practice concerning notation and convention within the considered disciplines i.e. wind engineering, offshore technology and engineering economics should be used. However, harmonization is required in the description of the entire system and its components, the interfaces between sub-systems and the structural design. The two former aspects are treated in this appendix.

1 Offshore wind energy conversion system (OWECS)

offshore wind energy conversion system (OWECS)
Entire system, comprising (usually) several wind energy converter units, for conversion of wind energy into electric power including the wind turbines, the support structures, the grid connection to the power delivery point and operation and maintenance aspects. Note that the environment i.e. air, water and soil as well as the utility grid is not considered as a part of the OWECS.

offshore wind farm
Synonym for OWECS.

2 Subsystems of the OWECS
Subsystems of an OWECS (Figure A-1) comprise either physical part of the system, e.g. wind turbine, support structure, or important aspects such as operation and maintenance aspects.

OWECS (Offshore Wind Energy Conversion System, offshore wind farm)
  ┌─────────────────────────────────────────────────────────────┐
  │ offshore wind energy converter (OWEC)                      │
  │    ┌─────────────────────────────────────────────────────┐  │
  │    │ wind turbine                                       │
  │    │    ┌──────────────────────────────────────────────────┘
  │    │    │ support structure
  │    │    │    ┌───────────────────────────────────────────────┐
  │    │    │    │ grid connection                                 │
  │    │    │    │ operation and maintenance aspects            │
  └─────────────────────────────────────────────────────────────┘

Figure A.1: Subsystems of the (bottom-mounted) OWECS
offshore wind energy converter (OWEC)
Single unit of the OWECS comprising wind turbine and support structure. ¹

wind turbine (WT)
Component of an offshore wind energy converter that transforms wind energy into electric power on generator voltage or AC-rectifier voltage, comprising rotor, nacelle with entire interior, control and safety system and electrical turbine system.

support structure (bottom-mounted)
Structure that supports the wind turbine and transfers the loading into the soil. Hence, the support structure comprises both the tower and the foundation.

grid connection and wind farm layout
This comprises two main parts that are considered for convenience as one subsystem. Firstly, the electrical system that takes the power provided at the turbine connection points and collects it at the wind farm collection point(s) and successively transmits it to the onshore connection point with the public grid. Secondly, the physical arrangement of the OWEC units.

operation and maintenance aspects
Auxiliary facilities, equipment and strategy required for operation, maintenance, control and administration of an OWECS

3 Boundaries of OWECS components

wind turbine and support structure
The fixed end of the yaw mechanism of the nacelle is defined as the boundary between the (horizontal axis) wind turbine and the support structure. All geometric and dynamic conditions are expressed with respect to the reference frame of the support structure.

wind turbine and grid connection
The turbine switch gear or circuit breaker is defined as boundary between electrical system of the turbine and the grid connection. The voltage at the connection point corresponds to the generator or the inverter (if any). Although a transformer might be installed at the wind energy converter unit, it is regarded as part of the grid connection.

grid connection and utility grid
The power of the OWECS is provided as three-phase AC at the voltage level of the utility grid to which the wind farm is connected. In absence of other explicit conventions the connection point is situated at the first dry location onshore regardless the actual grid infrastructure on land.

¹ No plural of the abbreviation ‘OWEC’ should be used; instead one may use ‘OWEC units’, full spelling or the singular form ‘OWEC’, if possible.
The main components of the OWECS sub-systems are defined by Figure A.2.

OWECS
- wind turbine
  - rotor
  - (mechanical) drive train
  - nacelle enclosure
  - control and safety system
  - electrical turbine system
- support structure
  - tower
  - foundation
- grid connection and wind farm layout
  - power collection
  - power transmission
    - offshore
    - onshore
  - wind farm layout
- operation and maintenance aspects
  - maintenance facilities and equipment
  - control, safety and administration facilities
  - operation and maintenance strategy

**Figure A.2:** OWECS sub-components
## APPENDIX B: DATA OF THE REFERENCE SITES AND DESIGNS

<table>
<thead>
<tr>
<th>Site</th>
<th>Wind conditions</th>
<th>MSL</th>
<th>Hydrodynamic conditions</th>
<th>Fetch</th>
<th>Soil type</th>
</tr>
</thead>
</table>
| Lely A3          | $V_{ave} = 7.7$ m/s  
$k = 2$  
$\alpha = 0.14$  
$h_{hub} = 41.5$ m | 5 m | $H_{max} = 1.77$ m  
$T_{seg} = 6.2$ s  
$U = 0.5$ m/s  
$surge = 0.2$ m | 0.8 - 32 km | sand and clay                                       |
| Lely A2, A2-mod  |                 | 10 m|                         |       | sand and clay, slip at top layer                 |
| Utgrunden (Turbine No. 4) | $V_{ave} = 8.5$ m/s  
$k = 2$  
$\alpha = 0.11$  
$h_{hub} = 65$ m  
$V_{ref} = 42$ m/s | 10 m| $H_{250} = 3.44$ m  
$T_{seg} = 7.6$ s  
$H_{max} = 6.4$ m  
$surge = 1.2$ m  
$t_{ice} = 0.4$ m | 8 - 300 km | medium dense  
to dense sand, gravel, boulders, some clay |
| NL-5             | $V_{ave} = 10.1$ m/s  
$k = 2.2$  
$\alpha = 0.11$  
$h_{hub} = 59$ m | 25 m| $H_{max} = 15.4$ m  
$T_{seg} = 12.5$ s  
$U = 1.2$ m/s  
$surge = 2.75$ m  
$tide = 0.75$ m | 50 - 2000 km | weak to firm clay | |
| Blekinge         | $V_{ave} = 8.3$ m/s  
$k = 2$  
$\alpha = 0.11$  
$h_{hub} = 51.5$ m | 15 m| $H_{max} = 9.7$ m  
$T_{seg} = 11$ s  
$U = 0.4$ m/s  
$surge = 0.8$ m  
$t_{ice} = 0.6$ m | 7 - 500 km | weak to firm clay | |
| NL-1             | $V_{ave} = 9$ m/s  
$k = 2.2$  
$\alpha = 0.11$  
$h_{hub} = 58.6$ m | 15 - 20 m| $H_{250} = 6.9$ m  
$T_{seg} = 7.7$ s  
$H_{max} = 12.8$ m  
$T_{seg} = 9.5$ s  
$U = 0.8$ m/s  
$surge = 3$ m  
$tide = 1$ m | 11.5 - 2000 km | medium dense sand                   |

$q = \text{wind shear exponent}$  
$k = \text{Weibull shape factor}$  
$H_{max} = \text{extreme 50 years wave height}$  
$T_{250} = \text{50 year zero crossing period}$  
$H_{250} = \text{50 year significant wave height}$  
$surge = \text{storm surge}$  
$tide = \text{extreme tidal amplitude}$  
$t_{ice} = \text{thickness of extreme 50 years sea-ice}$  
$T_{seg} = \text{wave period associated with } H_{250}$  
$U = \text{50 year design current}$  
$V_{ave} = \text{annual average wind speed at } h_{hub}$  
$V_{ref} = \text{50 year wind speed at } h_{hub} (10 \text{ min mean})$
### Table B.2: Lumped load cases for six directional sectors at the NL-1 site

<table>
<thead>
<tr>
<th>No.</th>
<th>$H_s$ [m]</th>
<th>$T_s$ [s]</th>
<th>$V_{in}$ [m/s]</th>
<th>Prob. [$%$]</th>
<th>$H_s$ [m]</th>
<th>$T_s$ [s]</th>
<th>$V_{in}$ [m/s]</th>
<th>Prob. [$%$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.57</td>
<td>3.92</td>
<td>&lt;5.0</td>
<td>200</td>
<td>0.44</td>
<td>3.54</td>
<td>&lt;5.0</td>
<td>139</td>
</tr>
<tr>
<td>2</td>
<td>0.86</td>
<td>4.83</td>
<td>&lt;5.0</td>
<td>91</td>
<td>0.70</td>
<td>4.40</td>
<td>&lt;5.0</td>
<td>46</td>
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<td>3</td>
<td>0.74</td>
<td>4.00</td>
<td>6.0</td>
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<td>3.74</td>
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<td>4</td>
<td>0.82</td>
<td>3.92</td>
<td>7.5</td>
<td>139</td>
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<tr>
<td>5</td>
<td>0.99</td>
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### Table B.2 (cont.): Lumped load cases for six directional sectors at the NL-1 site

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$V_{tn}$ = hourly mean wind speed at 60 m height
**Table B.3: Soil conditions**

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**Utgrunden** $^\dagger$  
$y_{sub} = 10 \text{ kN/m}^3$, $\varphi = 35^\circ$, $K = 28 \text{ MPa/m}$

**NL-5**  
$G_{dyn} = 8.3 \text{ MPa, } \nu = 0.5$

**Blekinge**  
$G_{dyn} = 8.3 \text{ MPa, } \nu = 0.5$

**NL-1** $^\ddagger$  
$\varphi = 30^\circ$ (P-y curves generated by Kvaerner Oil & Gas Ltd.)

$^\dagger$ average soil conditions as used for sensitivity analysis, softer soil assumed for limit state analysis  
$^\ddagger$ assumed in Chapter 9

- $Y_{sat}$ = saturated unit weight  
- $Y_{sub}$ = submerged unit weight  
- $C_u$ = undrained shear strength of clay  
- $\varphi$ = internal friction angle of sand  
- $K$ = modulus of initial subgrade reaction  
- $G_{dyn}$ = dynamic shear modulus  
- $\nu$ = Poisson ratio
Table B.4: Main support structure data

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<td>tower: $\bar{b}<em>{\text{base}}$ 3.19 m x 12 mm, $\bar{t}</em>{\text{top}}$ 1.9 m x 12 mm, 37.9 m length</td>
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<td>Lely A2</td>
<td>monopile: Ø 3.2 m x 35 mm x 26 m, 13.5 m penetration</td>
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<td>tower: see Lely A3</td>
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<td>monopile: Ø 3.2 m x 35 mm x 34 m, 20 m penetration,</td>
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<td>double conical contraction between -4 m and 3 m (MSL) including</td>
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<td>a cylindrical section Ø 2.5 m x 50 mm x 4 m</td>
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<td>tower: see Lely A3, but stretched to 45.4 m length</td>
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<td>Utgrunden (Turbine No. 4)</td>
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<td>transition piece: Ø 3.15 m x 9.8 m</td>
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<td>tower: $\bar{b}<em>{\text{base}}$ 3.15 m, $\bar{t}</em>{\text{top}}$ 2.6 m, double conical section Ø 2.25 m, 54.55 m length</td>
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<td>Gravity lattice tower</td>
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<td>height triangle edge length</td>
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<td>- 23 m (MSL)</td>
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<td>- 7 m (MSL)</td>
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<td>including transition between -10 m and -7 m (MSL) and upper section Ø 2.5 m x 100 mm x 6 m</td>
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<td>transition piece: Ø 3.8 m x 60 mm x 15 m with 5 m grouted joint</td>
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</table>

†WTS 80, ‡WTS 80 M (marine)
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Module : General</strong> (general data)</td>
<td></td>
</tr>
<tr>
<td>cut-in wind velocity</td>
<td>5. m/s</td>
</tr>
<tr>
<td>cut-out wind velocity</td>
<td>25. m/s</td>
</tr>
<tr>
<td>rated wind velocity</td>
<td>13.7 m/s</td>
</tr>
<tr>
<td>rated aerodynamic rotor power</td>
<td>3.785 kW</td>
</tr>
<tr>
<td>generator angular velocity</td>
<td>161.79 rad/s</td>
</tr>
<tr>
<td>rated tip-speed ratio</td>
<td>6.73</td>
</tr>
<tr>
<td>rotor angular velocity</td>
<td>2.2038 rad/s</td>
</tr>
<tr>
<td>communication time step</td>
<td>0.075 s</td>
</tr>
<tr>
<td>number of integration steps per interval</td>
<td>3</td>
</tr>
<tr>
<td>4th order Runge-Kutta integration scheme</td>
<td>.true.</td>
</tr>
<tr>
<td><strong>Module : ROTOR2</strong> (rotor &amp; nacelle)</td>
<td></td>
</tr>
<tr>
<td>number of blade sections (blade elements)</td>
<td>10</td>
</tr>
<tr>
<td>airfoil used between 6 m ≤ z &lt; 26.4 m</td>
<td>FFA W3 22.1%</td>
</tr>
<tr>
<td>airfoil used between 26.4 m ≤ z &lt; 32.2 m</td>
<td>transition from W3 22.1% to LS-1</td>
</tr>
<tr>
<td>airfoil used between 32.2 m ≤ z &lt; 40 m</td>
<td>LS-1 - 15 % by FFA</td>
</tr>
<tr>
<td>dimensionless power loss factor</td>
<td>1</td>
</tr>
<tr>
<td>blade contour</td>
<td></td>
</tr>
<tr>
<td>radius [m]</td>
<td>chord [m]</td>
</tr>
<tr>
<td>5.000</td>
<td>3.524</td>
</tr>
<tr>
<td>5.500</td>
<td>3.672</td>
</tr>
<tr>
<td>6.752</td>
<td>4.038</td>
</tr>
<tr>
<td>10.252</td>
<td>4.400</td>
</tr>
<tr>
<td>13.752</td>
<td>4.312</td>
</tr>
<tr>
<td>17.252</td>
<td>3.400</td>
</tr>
<tr>
<td>20.752</td>
<td>2.735</td>
</tr>
<tr>
<td>24.252</td>
<td>2.300</td>
</tr>
<tr>
<td>27.752</td>
<td>1.950</td>
</tr>
<tr>
<td>31.252</td>
<td>1.600</td>
</tr>
<tr>
<td>34.752</td>
<td>1.300</td>
</tr>
<tr>
<td>38.252</td>
<td>1.000</td>
</tr>
<tr>
<td>39.500</td>
<td>0.830</td>
</tr>
<tr>
<td>40.000</td>
<td>0.700</td>
</tr>
<tr>
<td>twist with respect to rotor plane [°]</td>
<td></td>
</tr>
<tr>
<td>13.40</td>
<td></td>
</tr>
<tr>
<td>13.40</td>
<td></td>
</tr>
<tr>
<td>13.40</td>
<td></td>
</tr>
<tr>
<td>10.00</td>
<td></td>
</tr>
<tr>
<td>6.96</td>
<td></td>
</tr>
<tr>
<td>4.86</td>
<td></td>
</tr>
<tr>
<td>3.48</td>
<td></td>
</tr>
<tr>
<td>2.55</td>
<td></td>
</tr>
<tr>
<td>1.66</td>
<td></td>
</tr>
<tr>
<td>0.65</td>
<td></td>
</tr>
<tr>
<td>-0.20</td>
<td></td>
</tr>
<tr>
<td>-0.86</td>
<td></td>
</tr>
<tr>
<td>-0.95</td>
<td></td>
</tr>
<tr>
<td>-1.00</td>
<td></td>
</tr>
<tr>
<td>blade mass between hinge and tip</td>
<td>6382. kg</td>
</tr>
<tr>
<td>flapwise blade inertia with respect to hinge</td>
<td>4.48 · 10^5 kgm^2</td>
</tr>
<tr>
<td>lead-lagwise blade inertia with respect to hinge</td>
<td>4.48 · 10^5 kgm^2</td>
</tr>
<tr>
<td>blade centre of gravity with respect to hinge</td>
<td>12.488 m</td>
</tr>
<tr>
<td>dimensionless blade hinge offset in terms of radius R</td>
<td>0.15</td>
</tr>
<tr>
<td>rotational stiffness of blade flap spring</td>
<td>1.039 · 10^6 Nm/rad</td>
</tr>
<tr>
<td>rotational stiffness of blade lead-lag spring</td>
<td>2.513 · 10^6 Nm/rad</td>
</tr>
<tr>
<td>flap damping ratio as fraction of critical damping</td>
<td>1. %</td>
</tr>
<tr>
<td>lead-lag damping ratio as fraction of critical damping</td>
<td>2. %</td>
</tr>
<tr>
<td>inertia of rotor shaft</td>
<td>2.5 · 10^6 kgm^2</td>
</tr>
<tr>
<td>rotational stiffness of rotor shaft</td>
<td>7.5 · 10^9 Nm/rad</td>
</tr>
<tr>
<td>rotational damping of rotor shaft</td>
<td>5. · 10^7 Nms/rad</td>
</tr>
<tr>
<td>time constant of wake retardation</td>
<td>4. · R / V</td>
</tr>
</tbody>
</table>
**Table B.6 (cont.): Input data for typical DUWECS model of the WTS 80 M turbine**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Module : Rotor2 (rotor &amp; nacelle)</strong></td>
<td></td>
</tr>
<tr>
<td>air density</td>
<td>1.25 kg/m³</td>
</tr>
<tr>
<td>gravity acceleration</td>
<td>9.81 m/s²</td>
</tr>
<tr>
<td>wind shear exponent for power law</td>
<td>0.11</td>
</tr>
<tr>
<td>yaw angle of undisturbed wind velocity</td>
<td>30°</td>
</tr>
<tr>
<td>variation of yaw angle by sine law during simulation</td>
<td>.true.</td>
</tr>
<tr>
<td>nacelle tilt angle (hub up positive)</td>
<td>3°</td>
</tr>
<tr>
<td>(inclined) distance between hub centre and tower axis</td>
<td>4.908 m</td>
</tr>
<tr>
<td>outer radius of tower for tower shadow model</td>
<td>1.6 m</td>
</tr>
<tr>
<td>dimensionless mass eccentricity of rotor</td>
<td>0.001</td>
</tr>
<tr>
<td>total nacelle mass (used only for steady tower top loads)</td>
<td>141,000. kg</td>
</tr>
<tr>
<td>x-coordinate of nacelle centre of gravity</td>
<td>-2.45 m</td>
</tr>
<tr>
<td><strong>Module : Tower32 (support structure)</strong></td>
<td></td>
</tr>
<tr>
<td>number of used mode</td>
<td>10.</td>
</tr>
<tr>
<td>number of modes used as dynamic degrees of freedom</td>
<td>6.</td>
</tr>
<tr>
<td>structural damping ratio as fraction of critical damping</td>
<td>0.5 %</td>
</tr>
<tr>
<td>modified Pierson-Moskowitz spectrum</td>
<td>.true.</td>
</tr>
<tr>
<td>significant wave height</td>
<td>3.81 m</td>
</tr>
<tr>
<td>zero up-crossing period</td>
<td>5.92 s</td>
</tr>
<tr>
<td>number of wave components in spectral discretisation</td>
<td>100.</td>
</tr>
<tr>
<td>equidistant frequency spacing</td>
<td>.false.</td>
</tr>
<tr>
<td>minimum ratio of PSD at peak and at cut-in frequency</td>
<td>0.002</td>
</tr>
<tr>
<td>cut-off frequency for spectral discretisation</td>
<td>0.637 Hz = 4 rad/s</td>
</tr>
<tr>
<td>refined spacing near fundamental eigenfrequency</td>
<td>.true.</td>
</tr>
<tr>
<td>odd number of refined frequencies around eigenfrequency</td>
<td>13.</td>
</tr>
<tr>
<td>minimum of average frequency increment in spectral tail</td>
<td>0.025</td>
</tr>
<tr>
<td>ratio between max. and min. frequency increment at tail</td>
<td>3.1416</td>
</tr>
<tr>
<td>linear wave theory with Wheeler stretching</td>
<td>.true.</td>
</tr>
<tr>
<td>wave diffraction according to MacCamy and Fuchs</td>
<td></td>
</tr>
<tr>
<td>gravity acceleration</td>
<td>9.807 m/s²</td>
</tr>
<tr>
<td>water density</td>
<td>1025. kg/m³</td>
</tr>
<tr>
<td>water depth</td>
<td>20. m</td>
</tr>
<tr>
<td><strong>Module : Transs (gear box)</strong></td>
<td></td>
</tr>
<tr>
<td>transmission ratio</td>
<td>70.227</td>
</tr>
<tr>
<td>equivalent drive train inertia at rotor side</td>
<td>6.9 · 10⁶ kgm²</td>
</tr>
<tr>
<td><strong>Module : Asm1 (asynchronous generator)</strong></td>
<td></td>
</tr>
<tr>
<td>slope of the torque curve in operating point</td>
<td>3618.5 Nms/rad</td>
</tr>
<tr>
<td>grid voltage</td>
<td>690. V</td>
</tr>
<tr>
<td><strong>Module : Ancon (analog controller)</strong></td>
<td></td>
</tr>
<tr>
<td>maximum pitch rate</td>
<td>5. %/s</td>
</tr>
</tbody>
</table>
Table B.7: Pitch controller and actuator model of the WTS 80 M turbine

pitch angle $\Theta$ \hspace{2cm} rotor speed $\Omega$

Wind turbine

Controller

PI-$T_1$ controller with 1st order delay for pitch actuator characteristics

$$H(s) = \frac{K_p}{t_i} \frac{t_i s + K_p}{t_a s^2 + t_i s}$$

where: \hspace{1cm} $H(s)$ transfer function
\hspace{1cm} $s$ complex frequency $s$
\hspace{1cm} $K_p$ gain, linearly interpolated between
\hspace{1cm} $K_p = 1.6 \text{ s at } V = 15 \text{ m/s and } K_p = 2.6 \text{ s at } V = 24 \text{ m/s}$
\hspace{1cm} $t_i$ integration time constant, linearly interpolated between
\hspace{1cm} $t_i = 0.65 \text{ s at } V = 15 \text{ m/s and } t_i = 1.28 \text{ s at } V = 24 \text{ m/s}$
\hspace{1cm} $t_a$ actuator time constant, $t_a = 0.5 \text{ s}$
SAMENVATTING

Windenergie buitengaats zal een belangrijke duurzame energiebron worden voor verschillende Noord-Europese landen. Uitgaande van kleine prototypeprojecten in de jaren negentig, die de technische en principiële economische haalbaarheid aantoonden, vindt nu een snelle ontwikkeling plaats. In het jaar 2000 zijn voor het eerst windenergieconverters in de megawatt klasse buitengaats geplaatst; vanaf 2002 zullen die standaard gebruik gaan worden voor grote windparken van 100 tot 160 MW. Op middel tot lange termijn bestaan er plannen voor multi-megawatt turbines en windparken tot één gigawatt vermogen.

Voor grootschalige exploitatie van windenergie buitengaats is de ontwikkeling van een volwassen offshore windenergiertechnologie van cruciaal belang. Basis voor een dergelijke ontwikkeling zijn de ervaringen met de windenergiertechnologie op het land, offshore technologie en de bedrijfsvoering van grote elektriciteitscentrales.

Hoe kunnen de economie en de betrouwbaarheid van offshore windenergieconvertersystemen door analyse van de dynamica en een toegespitste ontwerpmethode verbeterd worden?

Deze vraagstelling is het onderwerp van het proefschrift en wordt in vier hoofdbijdragen beantwoord:
- Ontwikkeling en demonstratie van een geïntegreerde ontwerpmethode voor offshore windparken
- Synthese van berekeningsmethodes met een complexiteit toegepast op de verschillende stappen in het ontwerpproces
- Onderzoek van de specifieke dynamica van offshore windenergieconverters en de interactie met hun ontwerp
- Demonstratie van een optimalisatie door afstemming van het dynamisch gedrag. Hieronder worden de belangrijkste aspecten van deze gebieden geschetst.


Voor de analyse van de dynamica van offshore windenergieconverters (die vast verbonden zijn met de zeebodem) wordt een algemene methode opgesteld door samenvoeging van de expertise op gebied van windenergie op land en de offshore
technologie. Daardoor is de analyse van de gelijktijdige responsie van het gehele systeem op wind, golven, stroming en ijs mogelijk.

Deze niet-lineaire simulaties in het tijddomein zijn wel nauwkeurig maar te ingewikkeld voor vermoelingsanalyses in het vroege stadium van het ontwerpproces. Daarom zijn er ook meer eenvoudige methodes ontwikkeld. De equivalentie vermoelingsbelastingen uit tijddomeinsimulaties van de windresponsie van de offshore windenergie-converter in rustig water worden opgeteld bij de resultaten van de lineaire spectrale analyse van de golfresponsie van de ondersteuningsconstructie. Deze splitsing is toelaatbaar als de aërodynamische demping van de paal in rekening gebracht wordt. Voor iedere taak op zich zijn de standaard ontwerpfiguraties uit de windenergie- en offshore technologie toereikend.

Voor certificatieberekeningen wordt nog steeds de voorkeur gegeven aan geïntegreerde, niet-lineaire simulaties van de wind- en golfresponsie in het tijddomein. De spreiding van de wind- en golpparameters resulteert in een, uit praktisch oogpunt, te hoog aantal belastingsgevallen. Door middel van een verdere methode kan dat teruggebracht worden tot een klein aantal karakteristieke belastingsgevallen.

Bij het onderzoek van de dynamica ligt de nadruk op de door wind en golfslag belaste ondersteuningsconstructie, omdat de windturbine weinig door de hydrodynamische excitatie beïnvloed wordt. De interactie tussen het ontwerp en het dynamisch gedrag wordt aan de hand van zeven voorbeelden van offshore windenergie-converters, waarvan er drie daadwerkelijk gebouwd zijn, onderzocht. Het nominale vermogen varieert van 500 kW tot 3 MW en de omgevingscondities betreffen zowel een beschermd binnenwater als ook locaties blootgesteld aan het ruwe klimaat van de Noordzee. De meest belangrijke aspecten zijn: beschrijving van de omgevingscondities, trillingsmodes en frequenties, aërodynamische demping, onzekerheden van het funderingsgedrag en responsie onder vermoelings- en extreme condities.

Aanbevelingen worden gegeven voor de dynamische analyse en het ontwerpproces.

De ontwerpmethodeologie, de analysemethoden en de inzichten in de dynamica worden gedemonstreerd aan de hand van het stapsgewijs ontwerpen en optimaliseren van een 300 MW offshore windpark.

Toepassing tijdens het Europese onderzoeksproject Opti-OWECS en de realisatie van het Uitgronden offshore windpark in de Oostzee toonden reeds de voordelen aan van een geïntegreerde benadering van de dynamica en het ontwerpproces.
ZUSAMMENFASSUNG


Wie lässt sich die Wirtschaftlichkeit und Zuverlässigkeit von Offshore-Windenergiesystemen durch Analyse des strukturdynamischen Verhaltens und eine gezielte Entwurfsmethodik verbessern?

Diese Fragestellung ist Gegenstand der vorliegenden Dissertation und wird in vier Hauptbeiträgen beantwortet:
- Entwicklung und Demonstration einer integrierten Entwurfsmethodik für Offshore-Windparks
- Synthese von Berechnungsverfahren für das strukturdynamische Verhalten von Offshore-Windenergieanlagen, deren Komplexität an die verschiedenen Stadien des Konstruktionsprozesses angepaßt sind
- Untersuchung des besonderen strukturdynamischen Verhaltens von Offshore-Windenergieanlagen und der Wechselbeziehung zu ihrer konstruktiven Ausführung
- Demonstration einer Entwurfsoptimierung durch Abstimmung des strukturdynamischen Verhaltens

Im folgenden werden die wesentlichen Ergebnisse in diesen Bereichen skizziert.


Die Entwurfsmethodik, die Analysemethoden sowie die von den Beispielanlagen abgeleiteten Erkenntnisse werden durch die schrittweise Konstruktion und Optimierung eines 300 MW Offshore-Windparks illustriert.

**CURRICULUM VITAE**

Martin Kühn was born in Linne, now Bissendorf, Germany on 20th May 1963. After passing high school in 1983 and working with mentally disabled people in a mechanical workshop he was educated at Kromschröder AG, Osnabrück. He won the annual award granted by the Chamber of Industry and Trade for the best-trainee-toolmaker in Lower Saxony in 1987.

In 1987 he started to study Mechanical Engineering at the Hannover University and graduated with honours in Physical Engineering at the Technical University of Berlin in 1993. Main subjects included Applied Mechanics, Structural Dynamics and Wind Energy.

A practical training of five months on design, assembly and erection of wind turbines was received at Südwind Energiesysteme, Berlin. The 'Evangelische Studienwerk Villigst e.V.' granted a scholarship from 1989 to 1993 which enabled the conduction of the master thesis 'Dynamics of an Offshore Wind Turbine - Digital Simulation' at the Institute for Wind Energy of Delft University of Technology in 1992/93.

From 1993 to 1999 he was employed as research assistant, lecturer, research scientist and project manager at the Institute for Wind Energy and contributed to various national and European research and consultancy projects on dynamics, design and economics of offshore wind farms. In 1996/97 he coordinated the EU Joule III project Opti-OWECs 'Structural and Economic Optimisation of Bottom-Mounted Offshore Wind Energy Converters' (JOR3-CT95-0087) and compiled the first proposal for the 'Concerted Action on Offshore Wind Energy in Europe' which finally started in the year 2000. More than 35 scientific and technical publications in the field of offshore wind energy were written and a poster award for the 'best overall contribution of scientific contents and presentation' was received at the EEUWEC 1996 Conference.

Since 1999 he is employed at Enron Wind GmbH, Salzbergen, Germany as project manager for offshore wind energy engineering and coordinated the design of the company's first offshore wind farm installed on the Utgrunden reef in 2000.
Offshore wind energy is an emerging renewable energy source in Northern Europe. For the large and increasing amount of offshore wind energy, the development of a mature offshore wind energy technology is crucially important. It should be based on the experiences and the success of the onshore wind energy technology, offshore technology of the petroleum industry and operation of large power plants.

How can analysis of the structural dynamics and a specific design approach, improve the cost-efficiency and reliability of offshore wind energy conversion systems?