Opti-OWECS Final Report Vol. 2:

Methods Assisting the Design of Offshore Wind Energy Conversion Systems

M. Kühn [1] (editor), T.T. Cockerill [2], L.A. Harland [3],
R. Harrison [2], C. Schöntag [1], G.J.W. van Bussel [1], J.H. Vugts [3]

[1] Institute for Wind Energy, Delft University of Technology,
[2] Renewable Energy Centre, University of Sunderland,
[3] Workgroup Offshore Technology, Delft University of Technology

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Institute for Wind Energy
Faculty of Civil Engineering and Geoscience,
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Summary

The integrated treatment of an offshore wind energy conversion system (OWECS) requires assistance of particular methods or tools in order to account for the interactions of the subsystems and to evaluate the system with respect to overall criteria (aspect-systems). The second volume describes four of such 'building-blocks' which have been developed or extended during the project.

Part A: Development of a cost model for offshore wind energy

A comprehensive analysis of the sub-system options for OWECS is carried out as preparation of the design work and the economic analysis. For instance, different generic support structure types, options for grid connection, etc. are investigated.

In close cooperation with the design work a computer based cost model for the economic assessment of certain OWECS concepts was developed next. The model also allows investigation of the effect of changes in important parameters on the cost of energy from an OWECS, including the wind speed, the support structure height, the size of the turbine, the distance from the shore, etc.

The model was validated through re-evaluation of some well known OWECS proposals and some real plants. Its predictions were compared to published data for the chosen validation cases and were found to exhibit an acceptable correspondence.

Although cost modelling studies in wind energy are not new, this model has a number of novel features in relation to its predecessors. One particular point is that many of the calculations are undertaken on what could be termed a first principles basis. This means that the model reaches its cost estimates by actually making a preliminary design of an OWECS meeting parameters specified by the user, albeit in a highly simplified manner, and then costing the result. This is in contrast with some previous work which estimated costs by merely interpolating between pre-specified costs. A further feature of the model is the flexibility in configuration that it incorporates. Options are available to allow both detailed examination of well defined OWECS concepts and more general study of broad trends.

Part C: Optimisation of operation and maintenance

The operation and maintenance (O&M) aspects of offshore wind farms have been analysed in a comprehensive way. Two striking differences with onshore wind farms are that the accessibility of an offshore farm is largely prohibited by bad weather conditions (wave height, wind speed and visibility); furthermore the costs of an offshore operation such as transport or lifting is an order of magnitude larger than onshore.

A Monte-Carlo program simulating the O&M behaviour of an offshore wind farm was developed. With the help of this program it is possible to investigate various possibilities for deployment of maintenance hardware, crews and O&M strategies. As to equipment a distinction can be made between crew transport (e.g. vessel and helicopter) and lifting equipment (e.g. modified jack-up, crane vessel and built-in lift system).

The program simulates the O&M behaviour of an OWECS over a lifetime period by following the state of each component of the wind farm one time step at a time.
At the start of each simulation run, the failure rates of the used wind turbines and the O&M strategy has to be defined. Further, the number of crews and shifts, the kind and quantity of equipment, the site parameters etc. have to be specified. Different maintenance strategies can then be evaluated by changing the input parameters, e.g. for the time interval between preventive maintenance visits.
Stochastic events, such as the occurrence of failures (of the wind turbine components) or the state of the weather, are simulated by a random number generator acting on the assumed probability distributions.
At the end of the simulation run the total O&M costs, the achieved availability and the produced energy of the wind farm are presented as output.

Part C: Reliability methods

In conventional design practice for offshore structures the environmental conditions are determined on the basis of independent estimates of extreme wave, extreme current and extreme wind conditions, each having a return period of e.g. 50 years. These conditions are next assumed to occur at the same time and to act in the same direction. This results in environmental design conditions and a corresponding global load condition with a very long return period and an unnecessarily conservative design. Recent advances in offshore engineering have led to a reliability based design method that takes correlations between the environmental conditions into account which allows matching of structural design with pre-defined risk criteria.

The reliability based design method consists of four steps:

I Definition of the environment in terms of storm events, in which wave, current and wind conditions are correlated instead of in independent environmental conditions. An essential requirement for this is the availability of a large database containing information on the simultaneous occurrence of wind, waves and current at the intended site during a long period (e.g. 25 years).

II Determination of the long term distribution of the extreme response of the structure during an arbitrary storm. A storm consists of a succession of sea states, each with its associated current and wind conditions. The most straightforward manner to determine the response behaviour in a particular sea state is, in principle, to perform a time domain simulation using a FE model. This has to be repeated for many different realisations of the same sea state, for all sea states in a storm and for all storm events in the database. The huge computational effort involved can be drastically reduced by application of the technique of constrained random simulations, but remains substantial. By appropriate combination of the individual response distribution from each simulation the long term distribution of the extreme response during an arbitrary storm can be determined.

III Determination of the long term distribution of the extreme response during the lifetime of the structure. The results of step 2 are now combined with the probability that a storm will actually occur, using the storm arrival rate derived from the database.

IV Determination of the probability of failure of the structure in a given lifetime by combining the result from step 3 with information on the ultimate strength of the structure. When the lifetime is longer than the duration of the database this inevitably requires extrapolation, which should be done with care. The results of this step make it possible to perform an economic risk evaluation or, alternatively, to determine an environmental load level for structural design which meets a pre-defined reliability level.

For an example support structure located at a demanding North Sea site it turned out that applying the structural reliability method reduced the extreme design loading with about 40% relative to the conventional design approach.

Part D: Overall dynamics of offshore wind energy converters

The dynamic properties of an offshore wind energy converter (OWEC) and its complex loading result in interactions between several sub-systems. Certain phenomena, e.g. drive train dynamics, are restricted mainly on some components while others, e.g. support structure fatigue, aerodynamic damping, controller and generator behaviour, require a model of the entire OWEC or even inclusion of certain aspects of the wind farm (e.g. wake effects or electrical interactions between different units).
Such aspects of OWEC dynamics played a pronounced role during the Opti-OWECS project. As a matter of fact this reaped benefits by a more cost-efficient design solution, e.g. soft-soft instead of soft-stiff monopile, and more reliable design calculations, e.g. by consideration of inherent uncertainties of the environmental conditions.

Analytical tools previously developed for OWEC as modal analysis and time domain simulations have been extended by state-of-the-art methods in both wind energy and offshore technology. Furthermore, dynamic considerations and application of an OWEC design tool formed an integral part of the design process (see integrated OWECS design approach) rather than only a check of the final design solution.

Table of contents

1. Introduction

1.1. Overview on the JOULE III project Opti-OWECS

In the scope of the framework of the Non Nuclear Energy Programme JOULE III (Research and Technical Development) the European Commission supported the project 'Structural and Economic Optimisation of Bottom-Mounted Offshore Wind Energy Converters' (Opti-OWECS) under grant JOR3-CT95-0087 from January 1996 to December 1997.

Objectives of the Opti-OWECS project

It was the particular mission of the Opti-OWECS project to extend the state-of-the-art, to determine required methods and to demonstrate practical solutions which will significantly reduce the electricity cost. This will facilitate the exploitation of true offshore sites in a medium time scale of 5 to 10 years from now.

The specific objectives included:

- A cost estimate and comparison of offshore wind energy converters of different sizes and different design concepts.
- An estimate of the cost per kWh of offshore wind energy at sites in different regions of the EU.
- Development of methods for the simultaneous structural and economic optimisation of offshore wind energy converters with due consideration of the site characteristics.
- At least one typical design solution for a bottom-mounted offshore wind energy conversion system.

Partnership and responsibilities

The project was an international co-operation of engineers and researchers from the wind energy field, offshore technology, power distribution and universities.
The group of participants includes:

- Institute for Wind Energy (IvW), Delft University of Technology

Dutch research group active since more than 20 years in various fields of wind energy applications including major offshore wind energy research since 1992.

- Kvaerner Oil & Gas, Ltd. (KOGL)

Major engineering and construction company, settled in the United Kingdom, with an established track record for implementing innovative concepts for offshore oil and gas developments.

- Kvaerner Turbin AB (KT)

Swedish wind turbine manufacturer with expertise in the design of multi-megawatt machines (since the 1970s) and participant in another large study on offshore wind energy (1991).

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- Renewable Energy Centre, University of Sunderland (US)

British research group involved in techno-economic studies of renewable energy sources since 1978 among two major projects on wind energy costs.

- Workgroup Offshore Technology (WOT), Delft University of Technology

Dutch research group with particular expertise in fluid loading of offshore structures and probabilistic methods, maintaining good relations with Shell Research Rijswijk.

- Energie Noord West (ENW)
  Dutch utility supplying 600,000 households in North-Holland and operating wind farms since more than 12 years among which the first Dutch offshore plant (Lely, 1994).

Kvaerner Oil & Gas, Ltd. and Kvaerner Turbin AB both form part of the international Kvaerner group which is organised in seven core business streams - KOGL being part of the Oil & Gas stream and KT being part of the Energy business.

The role of the partners is summarised in Table 1.1-1.

Partner	Role	Major scientific tasks
IvW	coordinator	- general expertise on (offshore) wind energy, - overall dynamics of OWEC, - wind turbine reliability, operation & maintenance, - design of grid connection and farm layout, - assistance in the cost analysis of OWECS, - aerodynamic rotor design,
KOGL	contractor	- general expertise on offshore technology, - design of support structure and installation procedure, - assistance in the cost analysis of OWECS
KT	contractor	- general expertise on wind turbine technology, - adaptation of wind turbine to offshore conditions
US	contractor	- concept and economic analysis of OWECS - development of cost models for OWECS, - estimate of costs of offshore wind energy at European sites
WOT	contractor	- general expertise on offshore technology, - structural reliability consideration, - assistance in the cost analysis of OWECS
ENW	sub-contractor (of IvW)	- general expertise as utility and as operator of (offshore) wind farms, - design of grid connection

Table 1.1-1: Distribution of responsibilities among the partners

1.2. Relation of this report to other work done within Opti-OWECS

The project continued the previous work in the scope of JOUR 0072 and made use of recent developments in wind engineering and offshore technology. The study considered the most feasible and the most probable concepts for the near future i.e. horizontal axis wind turbines rated approx. 1 - 3 MW and erected on bottom-mounted support structures in the Baltic or the North Sea.

The work content of the project comprised three consecutive major tasks:-

- Task 1 Identification
  The main cost drivers of offshore wind energy were identified and the base case concepts and the reference sites were selected.

- Task 2 Development
  The economic and structural optimisation and improved design methods were developed in three parallel tasks. A cost model for manufacturing, installation and operation and maintenance of offshore wind farms was compiled. Design concepts for all main sub-systems, i.e. wind turbine, support structure, grid connection and operation and maintenance, were investigated and the best combination for a certain sites is selected. Also particular design methods for OWECS such as structural reliability considerations and overall dynamics of OWEC were further developed.

- Task 3 Integration
  In the final phase the work of the former tasks was integrated and the relationships between them were fully considered. The achieved progress was demonstrated in a typical design solution for OWECS. Moreover, energy costs at different European sites or regions were estimated.

The project lasting from January 1996 to December 1997 was subdivided into three phases. Figure 1.2-1 presents an overview of the work packages (shaded boxes) and tasks (boxes enclosed in dashed lines) of the Opti-OWECS project.

The final reporting has been organised in a more coherent way with a view to the subjects considered rather than in the sequence the work was carried out. Therefore the report available to the public is subdivided into six volumes:-

· Vol. 0 Executive Summary [1.1]
· Vol. 1 Integrated Design Methodology for OWECS [1.2]
· Vol. 2 Methods Assisting the Design of OWECS
· Vol. 3 Comparison of Cost of Offshore Wind Energy at European Sites [1.3]
· Vol. 4 A Typical Design Solution for an OWEC [1.4]
· Vol. 5 User Guide OWECS Cost Model [1.5]

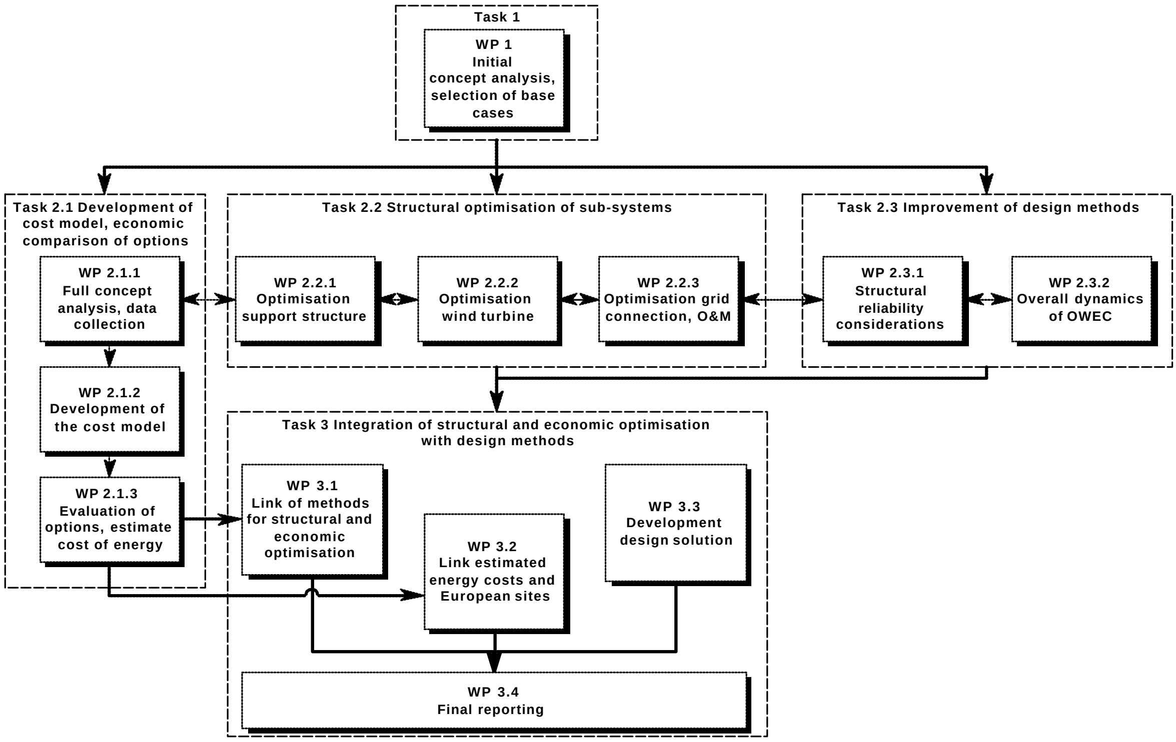

Figure 1.2- 1 Opti-OWECS project organisation

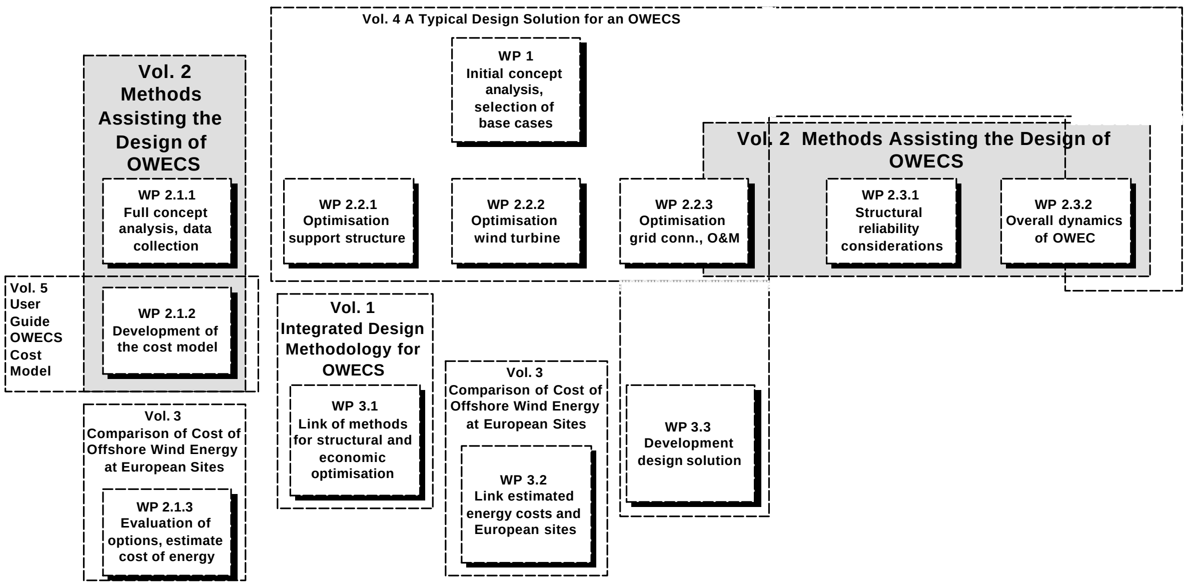

Figure 1.2- 2 Interrelation between Opti-OWECS workpackages and final report

As illustrated by figure 1.2-2 the different reports cover all work packages. Since it should be possible to review and use the volumes separately, it was necessary to address some items in more than one report. However, in such a case the individual documents consider these issue from different points of view, e.g. development of cost model in Vol. 2, economic evaluation in Vol. 3 and user guide in Vol. 5.

This document ‘Methods Assisting the Design of Offshore Wind Energy Conversion Systems’ is Volume 2 of the final report. It consists out of 4 separate parts on design tools and methods.

1.3. Organisation of the report

This report consist out of 4 separate parts on design tools and methods which can be read independently from each other.

Part A concerns the concept analysis and cost modelling of OWECS and is written by Sunderland University.

A tool to optimise operation and maintenance strategies of offshore wind farms has been developed by the Institute for Wind Energy (Delft University of Technology) and is described in part B.

Part C covers an advanced reliability method to access the extreme loading and response of an OWEC; this part is written by the Workgroup Offshore Technology (Delft University of Technology).

Finally, in part D the design tool is described, by the Institute for Wind Energy (Delft University of Technology), for the dynamic analysis of the total OWEC (wind turbine and support structure) under combined wind and wave loading.

Note that each part has a own section on table of contents, conclusions, recommendations and references. For convenience, the four parts are divided by coloured pages.

OWECS terminology

Use is made of a terminology for OWECS which has been developed and successfully applied during the project (see appendix A of Vol.1 [0-2], [0-6]). In order to avoid misunderstandings there are two essential conventions that should be appreciated. Firstly, the acronym “OWECS” (standing for Offshore Wind Energy Conversion System) or 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 any infrastructure for operation and maintenance. 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).

1.4. References

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Final Report Opti-OWECS Vol. 2

Methods Assisting the Design of OWECS
Part A:
Concept Analysis, Cost Modelling and Economic Optimization

T. T. Cockerill

Renewable Energy Centre
University of Sunderland, UK

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Table of Contents

Acknowledgements

Many people have contributed to this document. Major contributors with a clearly definable direct input to this document are listed below. Others have contributed in ways that are less easy to acknowledge, and their assistance is recorded as minor contributors.

Except where noted otherwise, all sections were written by and are the responsibility of the editor. It should be noted that many of the ideas and some of the work described in these sections were produced by the minor authors. While it is difficult to distinguish these contributions, it should be noted that the editor cannot take credit for all of the work described.

Editor and major contributor

T.T. Cockerill (University of Sunderland, UK)

Major contributors

W.A.A.M. Bierbooms (Institute for Wind Energy, Delft University of Technology, NL)
M.C. Ferguson (Kvaerner Oil & Gas, Ltd., UK)
L.A. Harland (Workgroup Offshore Technology, Delft University of Technology, NL)
M. Kühn (Institute for Wind Energy, Delft University of Technology, NL)
C. Schöntag (Institute for Wind Energy, Delft University of Technology, NL)
G.J.W. van Bussel (Institute for Wind Energy, Delft University of Technology, NL)
J.H. Vugts (Workgroup Offshore Technology, Delft University of Technology, NL)

Minor contributors

B. Görranson (Kvaerner Turbin AB, SE)
R. Harrison (University of Sunderland, UK)
S. Köhler (Institute for Wind Energy, Delft University of Technology, NL)
H. Kouwenhoven (Energie Noord West, NL)
T. Pauling (Institute for Wind Energy, Delft University of Technology, NL)
A. Sandström (Kvaerner Turbin AB, SE)

List of symbols

Only the more important symbols used in the test are defined here. Other symbols are defined on their first use.
Where symbols have more than one interpretation, their meaning should be apparent from the context.

Symbol	**Definition**
l	Tip speed ratio, *also* slenderness ratio
h_{array}	Array efficiency
$h_{availability}$	Availability efficiency
h_{gb}	Gearbox efficiency
h_{gen}	Generator efficiency
$h_{transmission}$	Electrical transmission efficiency
r	Density (of air, unless noted)
$Y()$	Mode shape
y_r	Slenderness factor
A	Swept area, *also* area
A_w	Weibull shape parameter
a	Annuity factor
C	Cost
C_{decom}	Decommissioning costs
C_{found}	Foundation flexibility factor
$C_{om,t}$	Annual operation and maintenance costs in year t
C_p	Coefficient of performance
$C_{r,t}$	Annual retro-fit cost in year t
C_{salve}	Salvage value
$C_{s,t}$	Annual social cost in year t
C_t	Total on-going costs in year t
c_d	Coefficient of drag
$c_{d,ax}$	Axial drag co-efficient for rotor.
c_m	Inertia coefficient
D	Diameter
D_R	Rotor diameter
D_T	Diameter of tower segment
E	Young's modulus
E_{farm}	Energy output of a wind farm
$E_{turbine}$	Energy output of one turbine
E_{useful}	Useful energy output
E_y	Total annual energy production (kWh)
e	Exponential constant

F	Force
$F()$	Cumulative probability function
F_{ax}	Axial thrust
F_i	Fatigue force range associated with lifetime n_i
$f()$	Probability density function
f_0	Natural frequency
G	Gust reaction factor
G_s	Modified gust reaction factor
I	Moment of inertia
I_{eq}	'Equivalent' moment of inertia of a stepped beam
I_{tot}	Total investment cost
K_{lat}	Translational stiffness at support structure foundation
K_{rot}	Rotational stiffness at support structure foundation
k_T	Structure roughness
L	Length (of beam), Height of support structure
LPC	Levelised (energy) production cost
l_j	Length (height) of tower segment j
M_{top}	Mass on top of support structure
M	Moment
M	Inverse slope of S-N curve
m_{eq}	Equivalent mass per unit length of a stepped beam
m_j	Mass of tower segment j
N_b	Number of blades
N	Number of turbines
n_d	Number of transmission cables
n_e	Economic lifetime
n_i	Fatigue lifetime (cycles or years)
$P()$	Windspeed distribution, *also* Turbine power curve
P	Power, *also* rotor rotation frequency
$P_{electrical}$	Electrical power
P_{rotor}	Power at a turbine rotor
P	Energy flux
Re_D	Reynolds number based on diameter
R	Real rate of interest
T_t	Time period (of oscillations)
TOM	Total levelised 'downline' costs
T	Time, wall thickness
$t_{divisor}$	$= {D_T}/{t}$, Specified section diameter to wall thickness ratio
V	Windspeed
$\bar{v}_c$	Weibull characteristic velocity
$\bar{v}_M$	Annual mean wind speed a hub height
v_i	Wind speed node

v_r	Rated wind speed
Dv	Width of wind speed band
W	Section modulus
X_j	Height of tower section j above foundation

1. Introduction

This section of volume 2 describes the physical, economic and empirical principles upon which the Opti-OWECS offshore wind energy cost model is founded. Some of the descriptions are necessarily simplified, but as much detail as is practical has been provided. The report aims to furnish the reader with sufficient understanding to comprehend the detailed calculations undertaken by the model, but it does not contain information elating to the manner in which the calculations are implemented within the model. In other words, while the report is comprehensive, it is far from a 'cook-book' for cost model development.

Although the report does discuss the validation of the model, it does not detail any predictions or parameter studies. Volume 3 [1-1] and section 4.7.1 of volume 4 [1-2] discuss ways in which the model can be used for site comparison and selection work, whereas section 10.3 of volume 4 deals with use of the model for parameter studies. A user guide to the model is available in volume 5 [1-3].

Discussion within this report begins with an OWECS concept analysis carried out at the beginning of the Opti-OWECS project. This analysis formed the basis for the cost model's development and as such does not take account of any of the insights provided by the modelling and other activities within the project. An updated version of the concept analysis that takes full account of the authors' recent experience may be found in sections 3 and 4 of volume 4.

2. Concept analysis for offshore wind energy converter systems

2.1 The analysis of OWECS concepts

2.1.1 The fundamental concepts of an OWECS

An offshore wind energy converter is 'defined' by five major features : (i) the choice of location, (ii) the choice of turbine (or turbines), (iii) the turbines' support structure, (iv) the number and physical layout of the turbines, (v) the connection of the plant to the shore and the public electricity grid and (vi) the hardware and strategy employed for operation and maintenance (O&M). In turn, each of these features comprises a number of subsystems that together make a conceptual specification. Reference should be made to [2.1-1] for a precise definition of each of the subsystems.

There are a number of options open for each of the features listed. The more 'technical' subsystems, the turbine, the support structure and the grid connection,

Turbine subsystem	Major Choices
Power control system	• Blade pitch control (passive or active) • Stall control (passive or ‘active’) • Yaw control • Partial pitch control
Blade options	• Flexible or stiff • Material • Number of blades
Rotor speed options	• Fixed speed • Dual speed • Full variable speed • Partial variable speed
Hub options	• Rigid • Teetered • Hinged
Drive train	• Modular gearbox • Integral gearbox • Direct drive
Safety system	• Aerodynamic brakes (stall/pitch) • Mechanical brakes • Yaw controlled braking • Electrodynamic braking

Table 2.1-1: Major turbine subsystem options

have a limited number of technologies available. Tables 2.1-1 - 2.1-4 list the overall conceptual choices for each of these systems. Some systems, and in particular the site variables and the farm layout, are continuous, able to take an infinite number of values in principle. While the scope of the project limits the range practically available, it is not sensible to attempt to specify the possible values explicitly. In any case it will be found that the capabilities of the technical options limits still further the range of these variables.

Support structure 'subsystem'	**Major choices**
Overall concept	• Stiff-stiff • Soft-stiff • Soft-soft
Tower	• Monotower • Braced monotower • Lattice tower
Installation	• Lifted • Floated
Foundation	• Skirted • Gravity • Piled

Table 2.1-2: Bottom mounted Support structure options

Grid connection subsystem	**Major choices**
Power collection within wind farm	• Connection concept • Current type - AC - DC • Physical cable type - undersea - above surface
Transmission to shore	• Current type - AC - DC • Cable type - undersea - above surface

Table 2.1-2: Grid connection major options

Not all of the options are compatible with others, and of those that are, not all produce a desirable result. This concept analysis discusses the options and their interactions, and starts to consider qualitatively how they might be crafted into a practical OWECS.

For the turbine, a large number of combinations are possible. Rather than attempting to consider all the possible combinations, much of the turbine analysis focuses on specifically offshore issues and on identifying development lines - fundamental philosophies which could underlie future designs. Necessarily, not every technical option will find a place in a development line.

Operation and Maintenance (O&M) aspects	**Major Choices**
O&M 'hardware'	• Location of maintenance base - offshore - onshore • Type of offshore base (if any) - vessel - platform - jack-up • Crew transport - vessel - helicopter • Lifting equipment - crane vessel - jack-up barge - self propelled modified jack-up
O&M Strategy	• PM[1] & CM maintenance strategy • Opportunity maintenance strategy • CM-only maintenance strategy • Periodic check maintenance strategy • No-maintenance strategy

Table 2.1-3: Major operation and maintenance options.

For the other OWECS main systems, the main discussion will be more comprehensive.

We will begin by discussing the technical economics of OWECS implementation, and identifying areas for technical investigation, along with the more important ways in which OWECS design differs from that of onshore farms. Next some 'fixed points' within each of the six major OWECS features will be established by looking at previous work in the field, intended to form fundamental features of any offshore design. With this background the concept analysis will move on first to consider in depth the turbine, the support structure and then the other remaining areas.

[1] In this context, PM stands for Preventive Maintenance, and CM for Corrective Maintenance.

2.1.2 Technical economics

While wind itself is free of charge, the machines required to convert it into electricity and their maintenance are not. Thus, wind generated electricity is priced at a level intended to enable an investor in wind power plant to recover their initial costs, operate and maintain the machines and, hopefully, generate a return.

Land based wind turbines have developed to such a degree of sophistication that the electricity they produce can be priced at a level competitive with conventional energy sources [2.1-2]. Siting a wind turbine offshore is intrinsically much more expensive than land based construction, making the cost of electricity from offshore plant considerably higher than that from comparable onshore installations. At the same time, offshore wind energy converters offer a number of attractive advantages over their land based colleagues, in particular (i) stronger, more reliable winds than onshore, (ii) more space than onshore, (iii) the absence of any (human) inhabitants who might be disturbed by the development.

Despite currently adverse economics, there are strong reasons to pursue offshore wind energy. These reasons, though, are not sufficiently strong that it is likely to be widely adopted without an improvement in the overall costs. Substantial development work is necessary in order to stand any chance of making electricity from offshore farms competitive with existing sources

2.1.3 Areas for attention

The major cost of an offshore wind energy converter system is the expense of the initial investment required to establish the project. This cost itself has three approximately equal components: the turbine machinery, the support structure and the electrical equipment/grid connection. Parameters relating to the site itself influence the precise contribution each of these makes to the overall cost. The distance to shore is the most important, with the turbine becoming less dominant in expense the further the farm is from the shore. The annual mean wind speed has a small influence on the investment cost, but a strong impact on the energy cost. In contrast the average wave height at any location has a less significant influence on the costs.

Aside from the initial investment, the next largest expense confronting the owner of an OWECS is the cost of its operation and maintenance. Indeed, earlier studies [2.1-3] have shown that operation and maintenance can account for as much as one-third of the cost of the electricity produced. Furthermore, there appears to be a strong linear relationship between the financial burden of O&M and the resulting electricity price, such that even relatively small changes in O&M costs or farm availability could have a substantial influence on the economic viability of an OWECS project.

There are therefore, four technical areas in which a project to (economically) optimise OWECS design should focus, specifically, the support structure, the turbine design, the electrical equipment/grid connection and the operation and maintenance procedures.

The first three areas have been examined in some depth individually, both within the offshore wind energy field and by workers with a wider interest. There is, as an example, considerable experience of constructing efficient offshore support structures among those involved in the exploitation of North Sea oil and gas. Equally wind turbine manufacturers are accustomed to building machines as cheaply as possible for onshore use, and the installation of high capacity power cables over long distances offshore holds few surprises for utility companies.

Several studies have investigated the applicability of these existing technologies for OWECS. The culmination of this work has been the construction of a number of demonstration projects, which have essentially adapted existing components for OWECS use. In order to improve upon the successes of the demonstration projects, it is sensible to attempt to take a more integrated approach to OWECS design than has been possible to date [2.1-4]. Thus attention should be focused on the turbine, the support structure, the electrical equipment/grid connection and operation and maintenance aspects together, taking full account of the complex interactions between them, and not only optimising them individually. Inevitably, to make such an integrated approach successful, account must be taken of the OWECS farm layout and the likely site parameters of the intended locations.

2.2 Aspects of wind turbine design for offshore use

2.2.1 Wind turbine components

The main components comprising a horizontal axis wind turbine are:

- The rotor, which extracts kinetic energy from the wind and transforms it into mechanical energy;
- The drive train and support system, which transfers loadings from the rotor to the nacelle superstructure and the support structure, and conveys power from the rotor to the generator;
- The electrical system which converts the mechanical energy into electrical energy.
- The power control system, which limits and conditions the extracted power;

In general, the range of components available for offshore use does not differ greatly from those suitable for onshore deployment, and little attention will be given to their relative advantages and disadvantages which are well considered elsewhere [2.2-1]. Most attention will be paid here to the issues that influence the design of turbines specifically for offshore use.

Current OWECS make use of turbines originally designed for onshore use, but with adaptations to render them suitable for the harsh offshore environment. Future

OWECS will be able to employ custom designed machines, but in the short term, any forthcoming schemes will probably have to adapt onshore machines. The issues apply equally to either approach, but with a custom designed scheme a greater range of solutions is possible.

2.2.2 Design issues for wind turbines

Very few definitive statements can be made about the design of turbines for offshore use. One of the ‘clear’ facts is that only large machines, certainly with capacities in excess of 1MW, should be considered. The large fixed costs of installing each individual OWEC mean that smaller machines are unlikely ever to be economic for offshore use. At the same time, although less certainly, it would be unwise to plan to use units much larger than 3 MW, as this effectively represents the limit of current and near-future technology. The largest size it is realistic to consider for the foreseeable future is a ‘stretched’ version of a current 3MW design, perhaps capable of producing approximately 4 MW of power.

Wind turbines sited offshore need to be 'marinised' in relation to their counterparts on dry land. At the simplest level, the differences might only consist of the avoidance of materials unsuitable for the harsh conditions offshore, but it seems likely that there will be advantages in taking more extensive measures. The Vindeby project [2.2-2] for example employed machines with sealed nacelles to prevent corrosion of the internal mechanisms. Custom designs could go further, perhaps designing components to be intrinsically resistant to the offshore environment.

Wind conditions offshore differ from those found on land. For the most part offshore machines experience higher static loads, but lower dynamic loads than their onshore counterparts. This has implications for the absolute and fatigue strengths of many components.

The ‘alternative environment’ provided by offshore use may have certain other influences. Unlike onshore where there may be people to disturb, the noise produced by the turbine need not be a major design consideration. Thus turbine features aimed primarily at reducing noise, for example by reducing the rotor speed at lower wind speeds, are no longer so attractive.

The absence of human neighbours in an offshore environment also means that violent machine failures may be acceptable. Onshore, these are very dangerous to those near the machine, and must be avoided under all circumstances. At a remote offshore site the only loss would be that of the machine and possibly adjacent machines if the failure is exceptionally violent. It may be practical, therefore, to design offshore machines that operate very close to their failure condition, with the objective of reducing capital costs. This though is a previously unexplored possibility that would be subject to the (probably unwilling) approval of offshore certification organisations.

It is already clear from the contrasts between onshore and offshore conditions that the optimal design of an offshore turbine will differ considerably from that of an onshore machine. Throughout this report, it will become apparent that interactions between the turbine and the other major ‘components’ of an OWECS i.e. the support structure, the farm layout and the electrical connection, and the facilities available for operation and maintenance often serve to widen the difference in optima even further.

2.2.3 Desirable features of wind turbines for offshore use

There are a number of fundamental qualities by which an OWECS, or indeed almost any machine, can be evaluated. Some of the most important are mentioned here, but the discussion is by no means comprehensive. It should of course be remembered that all these criteria are secondary to our main objective, that of reducing the electricity price per kWh from offshore wind farms.

Reliability

As with almost any machine, a wind turbine for use offshore must be reliable, in other words the mean time between failures (MTBF) should be as long as is practical. This though must be balanced against the other features, and particularly the capital cost.

Reliability is a particularly important issue for offshore wind turbines, thanks to the considerable expense of repair and the difficulties implicit in reaching offshore locations.

Accessibility and maintainability

The maintainability of an OWECS is a measure of how much effort is required to perform particular maintenance operations. To some extent increasing the maintainability of a turbine can offset reliability problems by reducing the costs of individual operations. An OWECS designed from the outset to simplify maintenance will be easier and cheaper to operate than one that takes no account of such issues. Maintainability may well have an influence on OWECS capital cost, and this must be considered.

Efficiency

In this context, an efficient machine is one that makes best use of the wind, producing the maximum quantity of electrical power. Efficiency must of course be balanced against the initial capital cost and machine reliability. Since the mean wind speed offshore is higher than onshore, high efficiency might be a secondary criterion.

Operating lifetime

The operating lifetime of a turbine can be regarded as the time from its installation until it becomes uneconomic to keep in operation. Turbines with a long operating lifetime are preferable to those without, provided the extra life has no detrimental effect on the capital cost. It may be sensible to specify different operating lifetimes for different OWECS components, for example giving the support structure and grid connection a longer life than the turbine. A new turbine could be affixed to the existing support structure when the first one was life expired.

Capital (investment) cost

The capital cost of a turbine is effectively its purchase price.

Safety

A turbine installation must meet all necessary safety requirements.

2.3 Support structure design options

2.3.1 Basic principle

There are three general approaches to the construction of WECS support structure, the over-riding objective being to avoid resonance of the structure with any likely periodic excitation force. For land based machines these are usually aerodynamic in origin, the lowest frequency driver being the rotation frequency with higher frequency excitation at the blade passing frequency equal to the number of blades times the rotation frequency.

Typical designs from the early days of the industry, are of the so-called stiff-stiff variety whereby the support has an eigenfrequency above the rotation frequency of the rotor and the blade passing frequency. Recent years have seen the use of soft-stiff towers which have eigenfrequencies carefully pitched between the rotation frequency and the blade passing frequency, and have the advantage both of reducing variable aerodynamic loads and providing a lighter solution. Soft-soft structures, with eigenfrequencies below the rotation frequency are also possible, but are only constructed for large wind turbines or when large hub heights are needed.

2.3.2 OWECS support structures

Compared to land based machines, design of support structure for OWECS is complicated by the need to accommodate hydrodynamic as well as aerodynamic forces. Wave loading makes soft-soft designs difficult to implement. Difficulties in the accurate construction of foundations offshore mean that soft-stiff designs must be considered very carefully, since it would be quite possible for inaccuracies to push the natural frequency into either of the forbidden zones. From a structural point of view, a stiff-stiff tower is the best option.

Adopting an integrated design approach means that the support structure cannot be considered independently from the turbine. A soft-soft or soft-stiff structure is preferable from the viewpoint of the turbine designer, as it provides much needed damping for dynamic forces on the whole assembly. With a stiff tower, that damping has to be provided elsewhere.

From the outset, some consideration must be given to the means by which the support structure and turbine will be installed at their offshore site. As an example, lifting an unwieldy nacelle onto a previously located support structure is not a desirable offshore operation, and therefore it might be preferable to install a previously mated support structure and nacelle in a single operation. This would have implications for the choice of foundation, ruling out any possibility of pile driving which would damage the previously installed turbine.

2.3.3 OWEC support structure concepts

Bottom mounted support structure concepts for large OWEC units fall into a number of generic types which can be broadly categorised by the nature of their foundation, their method of installation, their structural configuration and the material from which they are constructed. The options available for each of these are dealt with in the following sections.

Foundations

Options for offshore foundations are basically of three types:-
- Piled
- Gravity Based
- Skirted

Piled foundations make-up the most common form of offshore foundations. The standard method of installation is to drive the pile into the seabed using a steam or hydraulically driven hammer. The handling and driving of the pile generally requires the use of a crane, ideally a crane vessel.

It is thought likely that the vibrations resulting from the piling operations would present too great a risk to component parts of the mechanical and instrument equipment housed in the nacelle and as a result piling would need to take place prior to placement of the nacelle.

The structure can be configured as a monopile or have piles that are driven through sleeve elements and are attached to the main structure by either a grouted or swaged connection. As such, the pile provides the means of transferring both tensile and compressive loads from the structure into the seabed.

Piles themselves are of simple tubular construction which is inexpensive to produce and provides a least cost fabrication option. Given its dependence on the provision of cranage, the piled foundation is more likely to be used in combination with a lift installed support structure.

The gravity foundation, unlike the piled foundation, is designed with the objective of avoiding tensile loads between the support structure and the seabed. This is achieved by providing sufficient deadloads to stabilise the structure under the overturning moments which result from wind and wave action.

Where the gravity loads from the support structure and nacelle are insufficient to maintain overall stability, additional ballast will be necessary. This may take the form of sand, concrete, rock or iron ore and can be either installed in the construction yard or alternatively placed following positioning of the main structure.

Gravity structures come into their own when the environmental loads are modest and the deadloads are significant or when appreciable cost reduction can be achieved by avoiding the dependence on a heavy crane vessel.

Skirted foundations, also known as buckets, are similar in appearance to gravity foundations but are characterised by long skirts around their perimeter. Unlike a gravity foundation, the skirted variety is designed to transfer transient tensile loads and relies on undrained soil behaviour. Its application for wave load dominated structures can be significant owing to the transitory nature of the loading. Its suitability for large OWEC structures is however questionable owing to the sizeable static component of wind loading.

Installation

OWEC units lend themselves to a variety of different methods of installation involving various forms of piecemeal installation through to placement of the complete unit including the nacelle and rotor as a single piece. Installation itself is either achieved by lifting or by floating in the component parts. The following addresses the options available for the OWEC support structures.

Lifting of the OWEC is in principle the most straightforward method of installation and given access to a crane vessel of sufficient capacity and reach it should be possible to install the OWEC units simply and efficiently.

Although relatively light by offshore standards, the height of a support structure designed for multi-megawatt OWEC is beyond the capability of all but the largest offshore crane vessels. Lifting the unit in several pieces offers possibilities yet the height of the final section combined with the awkwardness and delicacy of the rotor assembly again limits the vessels capable of the operation to just a small number of the worlds largest.

One of the major benefits of using a heavy lift vessel is that it has the capability and has all the necessary equipment needed to pile the structure to the seabed. Ideally the nacelle complete with the rotor would be pre-installed on the support structure in the fabrication yard thereby maximising the time for commissioning; however owing to the vibrations associated with pile driving, installation of the unit as a single piece appears only possible using a gravity foundation.

The possibility of lifting the structure using a purpose built (or modified) jack-up could be considered, however the lengthy duration of the installation operation is likely to offset any benefits

Floating the support structure into place offers the possibility of installing the complete support structure and avoiding the necessity of using a major crane. As a floating body the structure would need to be either constructed in a dock or lifted from a quay. It would need either inherent buoyancy or auxiliary buoyancy to float and have sufficient stability both for transportation and during lowering.

Obviating the requirement for a crane vessel, a floated support structure is best suited to a gravity base foundation.

Configuration

The support structure configuration can be categorised as three basic types i.e.:-

- monotower - a single column
- braced monotower - a central column stayed by bracing elements
- lattice tower - a fully braced structure

Each has advantages, the monotower and braced monotower provide the benefits of simplicity whilst the lattice tower offers the most structurally efficient solution.

Materials

The candidate materials for the tower elements of the OWEC support structure are primarily steel and pre-stressed concrete. Steel offers the benefits of being some four times stiffer and stronger per unit mass than concrete and as such it offers the potential for appreciably lighter structures. It is this combination of stiff and light construction combined with steels flexibility in the construction of braced structures that make steel the preferred material. Its reduced weight also reaps benefits with the structure being lifted more easily or requiring less buoyancy for floating.

The material to be adopted for the foundation is less clear. In the case of piled foundations, steel presents the obvious solution whilst for gravity foundations, steel or concrete may be appropriate with sand, rock or iron ore used for ballast material.

2.3.4 Concept evaluation

From the preceding sections it can be concluded that potential support structures for the OWEC development can be identified from the following:-

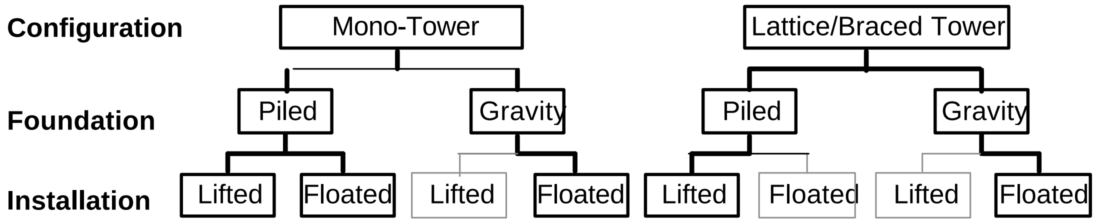


Figure 2.3-1: Support structure options**.**

The skirted foundation option is not included since it is unlikely to be viable owing to appreciable static component on the wind loading.

Thus a total of eight potential support structure types are available. From these, five (highlighted above in figure 2.3-1) have been selected for further consideration. Included are the piled/lifted and gravity/floated structures, along with one variant of the piled/floated concept for the monotower only. It is only practical to erect piled monotowers using a piece-meal approach wherein the pile is installed first and the tower, probably with the turbine already attached, is positioned in a separate operation. This latter part of the installation process could in principle be carried out equally well with lifting of floating techniques.

2.4 Grid connection and farm layout options

2.4.1 Overview

Previous studies have shown that an economic offshore wind farm should be large. The exact size is hard to specify, but somewhere in the region of 60 - 300 MW total power appears reasonable. There are a number of ways to physically arrange the large number of individual machines required for such installations. The layout chosen will influence both the aerodynamic efficiency of the whole farm, and the cost of wiring the individual turbines together.

In order to provide useful electricity, an OWECS must be connected to a land based power grid. This connection comprises two parts: firstly the individual turbines must be wired together to ‘collect’ the power, and secondly one or more cables must run between the array and a public electricity grid onshore (the ‘power transmission’ cable). The choice of power collection scheme, however, is closely linked with to the array layout, which is why the two superficially disparate features are considered together.

2.4.2 Grid connection

The ‘grid connection’ is considered to be the electrical system that collects the power provided at the turbine connection points, collects the power at the central cluster point(s) and transmits it to the onshore connection point with the public grid.

The power collection consists of:

- transformers to collection voltage (usually at every turbine)
- switch gear and circuit breakers
- cables or transmission lines inside cluster

The following components can be distinguished as comprising the power transmission system:

- transformer to transmission voltage
- inverters (if any)
- switch gears and circuit breakers
- transformers to voltage of the public grid (if any)
- cables or transmission lines

With regard to connection to the public grid, a distinction should be made between a wind farm, of say 60-300 MW, and one or several wind turbines. Due to the large amount of power involved the wind farm will be connected at a higher voltage level. This is advantageous because in general there are less restrictions in case of a connection at a higher level and more options are available for possible required adaptations (e.g. for reactive power).

In figure 2.4-1 the basic options for the grid connection are given; in principle these apply for both offshore as onshore wind farms. All kind of variations of these basic options are possible. No real technical restrictions are foreseen because nowadays electronic components are available for a wide variety of applications and they are modular. The main choice which has to be made is between an AC or DC connection to shore. Also for the power collection there is a choice between AC and DC. The first two options, A1 and A2, are the ones commonly used for onshore farms. Also some limited onshore experience exists with lay-outs in the fashion of option B. Option C, AC coupling of all wind turbines together with an DC connection to shore (‘AC island’), may cause technical problems with respect to achieving stable operation.

In the following the options for the components will be discussed. Although the generator is, in this project, regarded as part of the wind turbine it will be discussed here because of its implications for the grid connection.

The generator types commonly employed in wind turbines are the induction generator and the synchronous generator. The advantage of the induction type is its simplicity and corresponding low cost, but it requires reactive power. The synchronous generator, in combination with an AC-DC-AC link, allows for variable speed operation of the wind turbine which results in a higher energy yield (assuming constant lambda operation) and lower fatigue loadings. The voltage level of the common generators for megawatt turbines varies from 460 V to 1.2 kV. A higher voltage level can be advantageous in order to reduce the cost for transformation to higher voltages for transmission.

Transformers are used to change the voltage level. A high voltage level is advantageous for power transmission because the losses (due to Ohmic resistance) depend on the square of the current; by increasing voltage level the current is decreased, for the same electrical power.

Several voltage levels can be used within an offshore wind farm. It is possible to employ a transformer for every wind turbine to bring the generator voltage level to the voltage level I. The power of a cluster consisting of a number of wind turbines can be collected and transformed to another voltage level II. The power of all clusters can then be collected at the connection point of the wind farm and prior to transmission to shore, transformed to voltage level III. The number of voltage levels will depend on the total power of the wind farm and on the cost of the transformers. The chosen voltage levels should be in accordance with European standards.

Switch gear is necessary to deal with any short circuits. The installation of switch gear at each turbine and/or at the collection point(s) must be determined by the requirements, in terms of availability and safety, of the owner of the offshore wind farm.

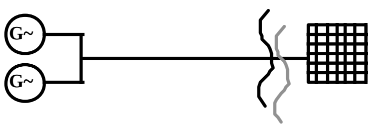
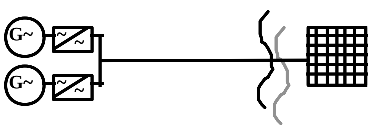
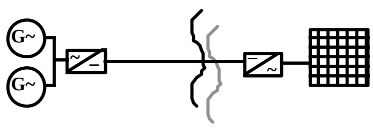
For the transmission lines one has to consider both the collection inside the farm and the transmission to shore. The options for the collection inside the farm are to use (submarine) cables or overhead lines [2.4-1]. The substantial advantage of overhead lines is the relative (very) low costs compared to submarine cables (including laying costs). Perhaps offsetting this economic advantage is that their reliability in a marine climate might not be adequate, and it seems unlikely that overhead lines would be practical for very large turbine spacings. Furthermore it should be checked whether the lines might present an obstacle during installation and maintenance activities.

Transmission can be either be AC or DC. AC transmission involves high dielectric losses (the isolation material acts as a capacitor); these losses are proportional to the cable length and the voltage. Three wires are necessary for AC transmission corresponding to the 3 phases. DC transmission requires expensive converters. For short distances AC transmission is the most cost effective option and for large distances DC transmission is preferable. The cross over point depends on the costs of the components involved and will be further investigated.

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; for the year 2015 1000 MW is expected to be feasible at approximately the same cable cost per km [2.4-2].

Currently used generators operate with AC as well as the public grid. 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 costs, at equal power, and lower energy losses. As alternative IGBT switches can be used which do not need reactive power.

The design criteria for lay-out of the cables or lines are the costs of the involved components and the reliability. A star connection results in the highest reliability compared to a circuit or chain connection. The higher capital costs for the extra cables for a star connection should be balanced with the higher energy yield.

	AC Connection to shore	DC Connection to shore
AC Connection within the wind farm	[A1]  [A2] 	[C] 

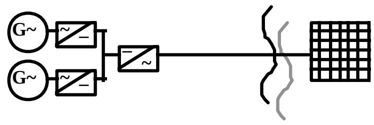
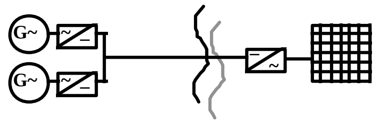
DC Connection within the wind farm	[B] 	[D] 

Figure 2.4-1: Basic grid connection options.

2.4.3 Wind farm lay-out

The wind turbines in a farm can be placed regularly in lines (rectangular) or in several sub-clusters as in the Blekinge study. A wind turbine which is placed inside the farm (and thus standing in the wake of another wind turbine) will experience a lower mean wind speed and a larger turbulence intensity. The larger turbulence results in larger fatigue loads. Models exists to predict these wake effects [2.4-3].

In order to limit the power losses, wind turbines in an onshore farm are placed at a distance of 3 to 5 D (rotor diameters) from each other perpendicular to the prevailing wind direction; in the other direction the spacing is 8 to 10 D. For offshore wind farms it may be necessary to have a larger spacing. The reasons for this are threefold. Firstly, equalisation between the mean wake velocity and the (unchanged) ambient wind speed outside the wake, needs a longer distance behind the turbine (because of the lower absolute turbulence intensity). Therefore offshore wind farms with the same spacing as onshore wind farms have lower aerodynamic cluster efficiency. Secondly, the relative increase of the turbulence intensity in the wake is larger in an offshore situation. According to [2.4-4] the calculated increase of turbulence intensity in case of the Vindeby lay-out is about 100 % (from 7 % above sea to 14 % in the wake); onshore the increase would be about 40 % (from 14 % to 19 %).

Another reason to use a larger spacing is that in general the restrictions on the area available to the farm are less for an offshore situation. The lower losses due to a larger spacing should be balanced with the higher costs for the power cables (including laying costs and the power losses along these cables) inside the farm. The soil properties and variation of water depth at some specific sites may be such that the actual farm lay-out should be different from the 'optimum' lay-out.

2.5 Operation and maintenance options

2.5.1 Introduction

Labour costs and spare parts are the main cost drivers of operation and maintenance (O&M) for onshore wind turbines. In contrast, the O&M costs of an offshore wind farm are dominated by the expenses of transportation to and from the offshore site. Careful consideration of the O&M strategy for an offshore wind farm is thus essential, to minimise the number of trips that are necessary. In this discussion, attention will only be given to the requirements of the wind turbine; it being assumed that the support structure and grid connection are essentially ‘maintenance free’.

An operation and maintenance strategy can be established using the following steps:
- consider turbine design
- consider maintenance approach
- consider O&M 'hardware'
- define O&M strategy

In general these four aspects have to be considered several times in order to reach the best O&M strategy.

The chosen wind turbine design determines the behaviour of the whole system comprising the 'wind farm' in the first place. The frequency of failures and the required preventive maintenance tasks depend on the reliability of the wind turbine. The maintainability of the turbine, e.g. how easy it is to exchange nacelles or other components, will have implications for the choice of lifting equipment. These examples already show the importance of coming to the right decision when considering the wind turbine's design and concept. An optimum operation and maintenance strategy will never lead to a system performing better than the chosen performance in the design of the wind turbine.

The next step requires the selection of the appropriate maintenance approach which takes the requirements of the chosen wind turbine design into account. There are two different types of maintenance action: Preventive Maintenance (PM) aims to reduce the occurrence of failures, and Corrective Maintenance (CM) that involves action only after a failure has occurred. Any approach to maintenance can employ, either or a combination of both of these actions.

With a knowledge of the wind turbine design and the maintenance approach, it is possible to estimate the resulting work load necessary for maintenance, in man hours per year, and to determine the number of personnel required to ‘operate’ the farm. Armed with this information, selecting the 'maintenance hardware' is the next step. Decisions about a possible maintenance base, crew transporting devices, lifting equipment, etc., have to be taken. The size and type of any lifting equipment required depends on the wind turbine's maintainability. The number of crew transporting devices necessary depends on the chosen maintenance strategy and the failure rates of the turbines, which in turn determines the probability of simultaneous occurrence of failures. In choosing the crew transport device the expected weather conditions (wind speed, wave height and visibility) are important.

Finally, the maintenance strategy has to be specified. With respect to the overall objective, i.e. minimising levelised production costs, the O&M and capital costs involved have to be weighed against the produced energy, and thus income generated. Increasing the maintenance efforts will improve the overall availability of the wind farm but will also increase the costs related to O&M.

2.5.2 Possible hardware for operation and maintenance

Various specialised equipment is available to simplify the performance of heavy maintenance tasks on offshore installations. As far as OWECS are concerned the major issues are (1) the position and nature of a maintenance base from which activities can be 'launched', (2) the means by which personnel are transported to and from the OWECS and (3) the choice of heavy lifting equipment. Decisions must be made by balancing the additional hardware costs against any savings in maintenance costs and increase in energy production brought about through improved availabilities.

This section reviews the options available for the main maintenance equipment, and highlights their advantages and disadvantages.

Location of maintenance base

Maintenance operations would be considerably simplified by the adoption of a permanently or regularly manned maintenance base. It may well be possible, however for maintenance to be undertaken in a perfectly satisfactory manner without such a facility. A maintenance base, if necessary, could be located onshore, either at an existing harbour or a purpose built site, or offshore close to the wind farm.

From the outset, the proposal of a purpose built on-shore maintenance base along the coast in order to minimise the distance offshore wind farm to shore, can be ruled out. The sheer numbers of existing, well equipped harbours along Europe's coasts mean that a purpose built solution cannot be justified. The few minutes travelling time saved will not compensate the initial investment costs for erecting such a base, with facilities, including cranage, docking etc., that are readily available at any existing harbour.

Whether the costs of an offshore base are justified, depends to a great extent on the distance from the offshore wind turbine cluster to the nearest suitable harbour. The costs for crew transportation from a mainland base to the wind turbines and the additional costs for transporting every component, requiring major overhaul, to the mainland base have to be weighted against the erection costs of an offshore base.

Possible offshore maintenance bases

A support vessel would provide accommodation for crews, permanently stationed within the wind farm. In case of a sudden weather change while working on the wind farm, it offers a relatively safe retreat for travelling maintenance crews. The support vessel is able to move around the wind farm and so help to reduce the travelling time between the individual wind turbines. At regular intervals it can return to a harbour for relief crews and fresh stocks of spares etc. However, the support vessel does not offer a stable working 'platform', and certain maintenance operations would be dependent on the sea being calm. In addition the available space onboard is limited. Thus, executing major overhauls, e.g. blades, gearboxes, onboard of a support vessel at a regular basis seems very unrealistic.

A maintenance base could be sited on a fixed structure located centrally in the wind turbine cluster. The base could, in contrast to the support vessel, not only offer accommodation facilities but also workshops for overhauling major components, such as blades or gearboxes. Combining the structure with the electricity transformer housing could be a possible way of reducing the initial investment costs.

The advantages of a support vessel can be combined with those of the purpose built support structure in a self propelled jack-up platform. It is able to move around the wind farm and, once jacked-up, it offers a stable working platform unaffected by the state of the sea. Such a self propelled jack-up platform seems to be the preferable choice since it can be equipped with a high capacity crane as well as crew accommodation.

Crew transport

Access by helicopter or vessel seems to be the most reasonable approach for offshore wind farms. A sample cost comparison of helicopter against vessel access shows that the helicopter offers the fastest but most expensive alternative. However, the downtime costs, saved by using the faster helicopter, do not compensate the higher operating cost of the helicopter.

The advantage of using helicopters for wind turbine access, lies in the decreased weather dependency. This advantage also has to be weighed against the initial modification costs in order to adopt the wind turbines for helicopter access.

Lifting equipment

At least two maintainability approaches for the wind turbine design concept can be distinguished:

- Firstly, failures could be repaired through the exchange of individual components
- Alternatively, machines could be designed to allow the modular exchange of assemblies of components.

The use of helicopters and of a lifting system built into the OWECS are both possible solutions if the first approach is chosen. However, it should be kept in mind that helicopter lifting operations are expensive, susceptible to wind gusts and therefore not possible in poor weather, and impractical if components have to be heaved in order to remove bolts, etc. The helicopter is the most expensive lifting device with respect to the ratio of costs and lifting capacity.

Lifting equipment built into the structure of the OWECS is ready to hand, whenever it is needed. Thus, a fast reaction time in case of lifting equipment demand, is ensured. However, providing every wind turbine with a lifting system means high initial investment costs.

For the second approach, the exchange of modules, three alternative lifting devices are practicable:

- crane vessel
- jack-up barge
- self propelled jack-up platform

Crane vessels come in three different types: the flat bottom barge type, the ship shape type, and the semi-submersible vessel type. Lifting operations with these types of crane are dependent on the wave height, which restricts the execution of the operations to certain weather conditions. Jack-up barges and self propelled jack-up platforms are also dependent on the wave height, but only while not being in the jacked up position. Once in working position, they offer a stable working platform where lifting operations can be executed regardless of the wave height. The lifting operation itself is, limited to a certain maximum wind speed, which is more or less the same with all three alternatives.

2.5.3 Possible operation and maintenance strategies

It is possible to conceive of a number of plausible O&M strategies for OWECS installations. Each attempts to balance capital costs, operational costs, and energy production in a different way. In considering the ideas, it is important to remember that our objective is to minimise the levelised cost of the electricity produced by the offshore farm. This is not the same as maximising the energy production, and indeed the most economic scheme may be one which sacrifices a little electricity for a great reduction in maintenance costs.

In practice, all wind turbine/OWECS concepts are likely to have teething troubles at their introduction. For a period immediately after the construction of a wind farm, say 6 months, a special commissioning maintenance regime would have to be in operation until the teething troubles were ironed out. For the subsequent mainstream operation, the following maintenance strategies have been identified.

The no-maintenance strategy

With this strategy neither preventive nor corrective maintenance tasks are executed. The failure, and thus shut-down, of individual wind turbines during operation of the wind farm is taken into account in the original OWECS design. One approach would be to incorporate redundancy in the number of wind turbines, that is, more wind turbines are built than are initially necessary to produce the farm's design power output. During the lifetime of the plant, many machines would fail, but the redundancy should be sufficient for the farm to always meet or exceed its design power output.

Alternatively decreasing turbine availability over time has to be accepted as a design 'feature'. Thus the rated capacity of the plant would decline towards the end of its life.

The only intervention 'permitted' within the farm would be the exchange of failed nacelles with fully functional replacements. Wind turbines could be exchanged either when the availability drops below a predefined minimum overall value, or the wind turbines have been in operation for a specified period. Alternatively one could also think of replacing single wind turbines as soon as they fail. In certain circumstances, and particularly this latter regime, the good components from removed nacelles could be recycled and used to build refurbished units for future replacement operations.

The only-CM-maintenance strategy

With this strategy only corrective maintenance tasks are carried out. Wind turbines are repaired either as soon as they fail, or left until a number have failed and repaired in batches. Under this scheme, no permanent maintenance crews are needed for the actual corrective maintenance tasks. Suitable crew could be hired on a stand-by basis to be mobilised at short notice, or from maintenance companies on demand.

The opportunity-maintenance strategy

This strategy is very similar to the only-CM-maintenance strategy. The main intention is to execute CM tasks, on demand. However, if a wind turbine undergoes corrective maintenance, the opportunity is also used to carry out preventive maintenance tasks on the same turbine. This means that preventive maintenance is executed at very irregular intervals, and only after a failure of the wind turbine. The philosophy behind this strategy is to reduce the number of visits to the wind turbines.

The PM & CM maintenance strategy

Under this scheme, a full range of PM tasks are pre-scheduled and carried out at all turbines to a well planned timetable. Complete CM is also undertaken as and when necessary. This is essentially the maintenance strategy currently employed for land based wind farms.

It has to be born mind that, for onshore wind farms, labour costs and spare parts are the main drivers of the O&M costs. The costs of transport and access to land based wind turbines represent only a minor part of the overall O&M costs. In contrast, for an offshore wind farm, O&M costs are strongly affected by the efforts for transportation and access to the platforms. Therefore, for offshore installations, the number of visits to the wind turbines needs to be carefully controlled.

The light-PM and light-CM maintenance strategy

As with the previous strategy, both corrective and preventative maintenance work would be carried out. The significant difference with the previous scheme, however, is that the scope of the operations would be limited to those below a certain costs/complexity. For example, replacement of small components, which could be performed with a minimum of equipment and a very small crew would certainly be undertaken. Large operations requiring heavy equipment and comparatively many workers, for example blade exchanges, would not be permitted. Machines that required large scale repairs, would either be abandoned or replaced in infrequent batch operations, as with the no-maintenance strategy.

The idea behind this scheme is to achieve a reasonable balance between full scale, strategies, and radical, minimal or zero maintenance approaches. Full maintenance makes best use of the initial investment in the farm, but is operationally expensive. Minimal maintenance is cheap to perform, but wastes a certain amount of the initial investment by leaving failed machines standing idle.

The periodic check maintenance strategy

Wind turbines are accessed at regular, scheduled, intervals. During each visit, the wind turbines are thoroughly inspected, after which any necessary PM and CM tasks are completed. Aside from the regular visits, no other maintenance work is performed, so that, for example, failed turbines are left inoperable until the next scheduled visit.

2.5.4 Evaluation of maintenance strategies

Qualitative comparison of the proposed maintenance strategies cannot be taken very much farther. In order to provide a more solid basis for decision making, a simulation tool has been developed for evaluating the strategies as a function of turbine design, overall OWECS design, and maintenance hardware employed.

3. Cost modelling of offshore wind energy converter systems

3.1 Introduction to cost modelling

3.1.1 Background to cost modelling

An engineering cost model attempts to estimate the costs associated with a technical system. Starting from an outline specification of the system, the model attempts to simulate the first stage of the engineering design process. Using a combination of fundamental principles and empirical relations, a cost model 'sizes' the options available for a system, and estimates their costs.

Such cost models are extremely useful for investigating the sensitivity of the overall cost to changes in one or more of the design parameters. They have been used considerably within process industries and with respect to other renewable energies [3.1-1]. Until recently, their application to wind energy had been restricted to the NASA MOD projects [3.1-2] [3.1-3], but new contributions are mounting rapidly. The model described in this text is a direct descendant of two recent models, one developed by the University of Sunderland in the UK [3.1-4], and one by the Institute for Wind Energy at Delft University of Technology in the Netherlands [3.1-5] [3.1-6].

3.1.2 Particular objectives of this work

The primary objective of this work is to develop a cost model that can assist in the evaluation of the technical options open to the development of offshore wind energy. In this respect the cost model must be able to produce a representative estimate of the cost of the energy (ECU/kWh) that a range of plausible outline schemes might produce. The additional provision of a cost breakdown, detailing the estimated contribution to the overall energy cost made by the various facets of an OWECS would provide further benefit.

The immediate purpose of the model is to enable the evaluation of the design options and sites identified as suitable during the course of the Opti-OWECS project. It is important to note that these options are distinct from the development concepts outlined in the concept analysis, although there is some correspondence between the two. Full details of the studies will be given during their description in volume 4 [3.1-7] of this report. Results from the studies have had a direct influence on the course of the final OWECS design produced by the Opti-OWECS project, as it is those options that were identified as most economic that have been developed further.

There are also some more forward looking objectives for the model, and in particular, it will be used to perform sensitivity studies that will shed additional light on the economics of offshore wind energy.

3.2 Quantitative economics of wind energy

In order to calculate a representative figure for the price of energy produced by any particular wind farm, estimates of four quantities are needed: the total investment cost needed to establish the farm, the annual energy production, the total ongoing or 'downline' costs of the farm, including such expenses as operation and maintenance, and finally economic parameters such as the prevailing interest rate. Conventional economic practice is to calculate the energy cost from these four pieces of information in such a way as to take account of the 'time value' of money. In other words the energy costs are discounted over the economic lifetime of the plant.

The IEA [3.2-1] suggest that the levelised production cost (LPC) is used as the measure of the (minimum) economic price of electricity from a wind farm. The LPC is defined as

$$LPC = \frac{I_{tot}}{a\,E_y} + \frac{TOM}{E_y} \qquad (3.2\text{-}1)$$

where I_{tot} is the total investment cost, E_y is the total annual energy output from the farm and TOM represents the total levelised downline cost. The remaining term in the expression, a, is the annuity factor and is a function of the real rate of interest, r, over the plant economic lifetime n_e such that

$$a = \frac{1-\left(\frac{1}{1+r}\right)^{n_e}}{r}. \qquad (3.2\text{-}2)$$

It is important to note here that the plant economic lifetime represents only the period over which the plant must recoup its investment costs along with any interest/dividends payable on the financing of those costs. The economic lifetime is a purely economic parameter and does not have any direct relationship with the technical lifetime of the plant, although it would of course be extremely inadvisable to adopt an economic lifetime longer than the technical life.

Returning to the total levelised downline costs, TOM, this term is intended to represent all costs incurred after the initial construction and commissioning of the plant. Many of these are lkely to be 'on-going' in nature, and due account must be taken of their time value. Operation and maintenance costs in particular are likely to be distributed over the life of the plant. A reasonable approach to discounting these costs is to make annual estimates of each and in effect discount the annual totals. Thus the contribution to downline costs made by on-going expenses is given by

$$TOM)_{annual} = a^{-1}\sum_{t=1}^{n_e}\left[(C_t)\times(1+r)^{-t}\right] \qquad (3.2\text{-}3)$$

where t is a year index and C_t represents the total ongoing costs for year t of operation.

There is no requirement for C_t to remain constant from year to year, and in practice it is unlikely to do so. Typically, the total annual cost might have three components: the annual operation and maintenance cost $C_{om,t}$, the annual social cost $C_{s,t}$ and the annual retrofit cost $C_{r,t}$ such that:

$$C_t = C_{om,t} + C_{s,t} + C_{r,t}\,. \tag{3.2-4}$$

While the maintenance cost is self-explanatory, the others are not. The social cost represents the costs of the energy production which are borne by third parties external to the project and are not reflected directly in the market price of the energy. The retrofit cost is intended to account for any large scale repairs or replacements that might be required during the lifetime of the plant.

Other parts of the downline cost will be 'one-off' expenses, the major one being decommissioning the plant at the end of its life, C_{decom}. This cost may be offset by the residual value of any of the salvaged plant components C_{salve}, including any maintenance equipment that is no longer required, but it is more than likely that the salvage value will be negative. Other one-off expenses will only apply to specific cases and will not be considered here. These costs may be discounted over the life of the plant

$$TOM\big)_{one\ off} = a^{-1}\left(C_{decom} - C_{salve}\right)(1+r)^{-n_e}\,. \tag{3.2-5}$$

Thus, the total downline costs are given by

$$TOM = TOM\big)_{annual} + TOM\big)_{one\ off}\,. \tag{3.2-6}$$

3.3 The structure of the cost model and its limitations

The economic analysis shows that the cost model must provide estimates of the energy production, the investment cost and the downline cost for each OWECS concept of interest. Consequently the model consists broadly of three separate sections responsible for each of the three values. None of the calculations is straightforward, and the model is further subdivided into linked sections. The structure of these sections, as summarised in figure 3.3-1 will be outlined here, along with the assumptions that are essential in order to make progress, and the limitations they impose.

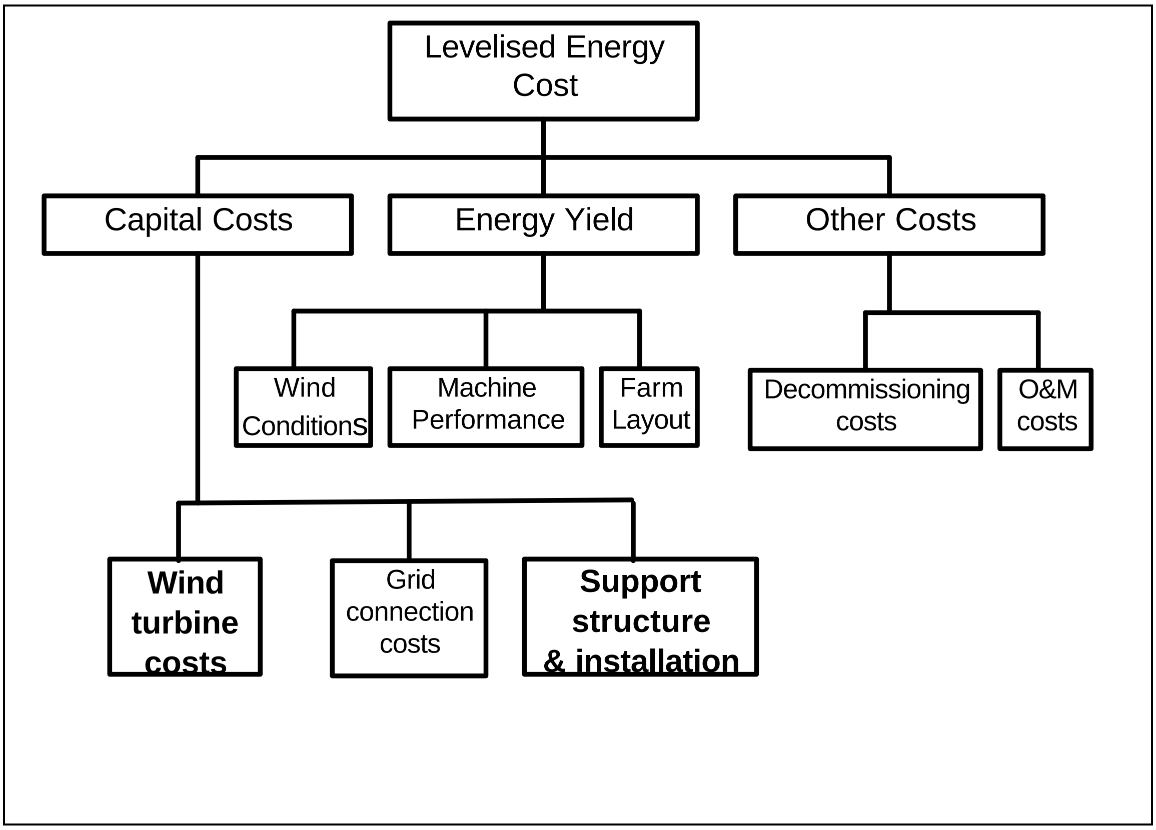


Figure 3.3-1: An OWECS as seen by the cost model.

The investment (i.e. capital) cost calculation in particular consists of a number of stages reflecting the physical construction of an OWECS. Separate sub-models deal with the cost contributions of the wind turbine, the support structure and installation, and the grid connection. In most cases similar approaches are taken to calculating the costs. Firstly the basic design drivers, that is the physical parameters of the site, the overall energy requirements, and any constraints on the OWECS, are used to calculate a design objective for the subsystem under consideration. Next the options available to meet that design objective are automatically sized and costed. Where a choice is available the cheapest is selected. Having repeated the calculation for all OWECS subsystems, the individual costs are totalled to provide an estimate of the overall investment required for the proposed OWECS.

Calculation of the investment cost is necessarily very approximate. Where more accurate information is available a priori, facilities are incorporated to bypass the model and directly specify costs and other data. This provides more reliable results, but at the expense of generality.

Estimation of the energy production is a much more robust process than that for the investment costs. Given sufficiently accurate initial data, the calculation is capable of producing good results for individual wind turbines. Approximate means are used to account for array losses and downtime.

Downline costs are very difficult to estimate. The cost model however only attempts to estimate two of the values, one of which is the annual operation and maintenance cost, assumed to be constant from year to year. A very approximate estimate of the decommissioning cost is also made. Facilities exist for the other downline costs to be input manually, should appropriate information be available.

4. Energy production calculation

4.1 Basic principles

To calculate the energy produced by a wind turbine over a period of time three pieces of information are necessary (a) the electrical power produced as a function of the wind speed, P(v) (b) the time the turbine has been in operation $t_{operation}$ and (c) the proportions of that time that the wind speed has taken any particular value, that is the wind speed distribution f(v). The total energy production is given by

$$E_{turbine} = t_{operation} \times \int P(v) f(v) . \qquad (4.1\text{-}1)$$

When multiple turbines are combined to form a wind farm, the model assumes that all the turbines perform identically and are subject to the same wind regime. In practice this is unlikely to be the case, as the turbines at the upwind areas of the farm 'spoil' the wind for those further downwind. The downwind turbines perform less well than those upstream, resulting in an overall farm performance below that which would be naively expected. This reduced performance is accounted for by the use of an array efficiency, such that the energy produced by a wind farm of n identical turbines is

$$E_{farm} = n \times h_{array} \times E_{turbine} . \qquad (4.1\text{-}2)$$

Not all of the energy generated by an OWECS is available to perform useful tasks. A significant quantity of electricity is dissipated as heat in the collection and transmission cables. Electrical losses are accommodated in the model using a transmission efficiency such the energy available at the onshore public power grid is

$$E_{useful} = h_{transmission} \times E_{farm} . \qquad (4.1\text{-}3)$$

It is the power delivered after accounting for such losses, in other words the power delivered to the shore, that is of importance in evaluating the economic performance of an OWECS.

4.2 Generated electrical power as a function of wind speed

The relationship between the wind speed and the electrical energy can be dealt with in two ways by the cost model. If available the relationship between the two can be specified directly as a look up table of corresponding of values for the wind speed and the useful electrical power available from the turbine

Alternatively the model can take a more fundamental approach as follows. The kinetic energy flux of wind blowing is given by

$$p = \tfrac{1}{2}\rho v^3 \qquad (4.2\text{-}1)$$

where v is the wind velocity (assumed steady for simplicity) and ρ is the air density. The power extracted from such a flux by a wind turbine rotor that sweeps an area A is

$$P_{rotor} = \tfrac{1}{2} C_p A \rho v^3 . \qquad (4.2\text{-}2)$$

C_p in this equation is known as the coefficient of performance of the rotor and represents the effectiveness of the rotor in extracting energy from the wind. Values of C_p are widely available for various aerofoil/turbine blade types, presented as a function of the wind turbine tip-speed ratio, defined as

$$\lambda = \frac{\text{velocity of blade tip}}{\text{wind speed}} . \qquad (4.2\text{-}3)$$

A knowledge of the rotational speed of the wind turbine is then sufficient to enable estimation of the power captured by the rotor at any wind speed at which it operates. Most wind turbines also have a cut-in wind speed below which the rotor is not allowed to turn because it would not produce a worthwhile power, and a cut-out wind speed above which the rotor is prevented from turning to avoid overstressing components. When the wind speed is outside of the range between the cut-in and cut-out speed then the power produced should be assumed to be zero, irrespective of the predictions of the C_p-λ based calculation.

Clearly the calculation is more difficult for wind turbines that do not operate at a constant pitch angle. For such designs a C_p curve is required for each operating regime, as well as a set of wind speed values over which each curve applies. Variable speed wind turbines also present problems - for such calculations it is assumed that wind turbines always operate at their optimum C_p.

The power captured by the rotor is not equal to the useful electrical power produced by the turbine as two lossy processes are involved before the power is converted to electricity. Firstly the shaft power is transmitted through the drive shafts and gearbox (if any) with efficiency η_{gb}. Next the generator produces electricity with a conversion efficiency η_{gen}. Thus the useful electrical power available at the wind turbine output is

$$P_{electrical} = \eta_{gb}\eta_{gen}P_{rotor} . \qquad (4.2\text{-}4)$$

The advantage of this more complex approach is that it enables investigations into the effect of variations in rotor size, or indeed the changes that might be produced by using a different aerofoil (i.e. changing the C_p-λ curve). Such studies are not possible if the power output is given directly as a function of the wind speed.

4.3 Operating time

In calculating the annual energy production of a wind farm using equation (4.2-4), it is assumed that all the plant has the potential, in principle, to operate continuously throughout the year. Of course this will not be the case. Individual turbines will experience downtime due to component failures and occasionally the whole farm may be taken off-line by large scale problems, a fault in the grid connection being an example. The effect of downtime on the energy production is accounted for by the availability, which represents the proportion of a year for which an average turbine is available to produce energy. Thus the total annual operation time is

$$t_{operation} = h_{availability} \times 24 \times 365 \quad h\,/\,year\ . \qquad (4.3\text{-}1)$$

Note that this is purely an 'average' type calculation, and within this model no attempt is made to correlate downtime with wind conditions. In other words no account is taken of whether downtime occurs during particularly windy or calm conditions which would affect the energy production.

The availability can either be set explicitly or estimated from the maintenance model discussed in section 6.

4.4 Wind speed distribution

The proportion of time that the wind has any particular velocity at the farm location is assumed to be given by the Weibull distribution. Thus, the probability of the wind adopting any speed v at an instant is given by

$$f(v) = \frac{A_w}{\bar{v}_c}\left(\frac{v}{\bar{v}_c}\right)^{A_w - 1} e^{-\left(v/\bar{v}_c \right)^{A_w}} \qquad (4.4\text{-}1)$$

where A_w is the logarithmic base, known as the Weibull shape parameter, $\bar{v}_c$ is the characteristic (average) wind speed, termed the Weibull scale parameter.

Using this expression the wind speed distribution can be estimated from the two Weibull parameters and these values may be input explicitly into the model. Alternatively the model makes the reasonable [4.4-1] assumption that A_W=2 and estimates the scale parameter from the annual mean wind speed at the turbine hub height $\bar{v}_M$ with the following relation [4.4-2]

$$\bar{v} = \frac{\bar{v}_M}{\left[0.568 + \left(\frac{0.434}{A_W} \right)^{\frac{1}{A_W}} \right]}\ . \qquad (4.4\text{-}2)$$

4.5 Numerical evaluation

The numerical evaluation of equation (4.1-1) must be done on a discrete basis within the model. The wind speed range is therefore divided into a series of bands each with median wind speed v_i and width Δv_i giving limits of $v_i \pm \frac{\Delta v_i}{2}$, as shown in figure 4.5-1. The proportion of total time that the wind spends within each band is calculated from the cumulative Weibull distribution function obtained by integrating (4.4-1),

$$F(v) = 1 - e^{-\left(\frac{v}{\bar{v}}\right)^{A_W}} \qquad (4.5\text{-}1)$$

such that

$$\begin{array}{l}\textit{Time in windspeed band} \\ \textit{with median wind speed } v_i\end{array} = f(v_i) \times Dv_i = F\left(v_i + \frac{Dv}{2}\right) - F\left(v_i - \frac{Dv}{2}\right). \qquad (4.5\text{-}2)$$

A value for average power produced in each band $P(v_i)$ is calculated using the equations outlined above with the median wind speed v_i. Since the power varies with the square of the wind speed, this procedure is inconsistent. The inaccuracies it introduces are likely to be negligible compared to other uncertainties in the model, and it does at least have the virtue of simplicity.

Equation (4.1-1) can be evaluated numerically as

$$E_{turbine} = t_{operation} \times \sum_i P(v_i) f(v_i). \qquad (4.5\text{-}3)$$

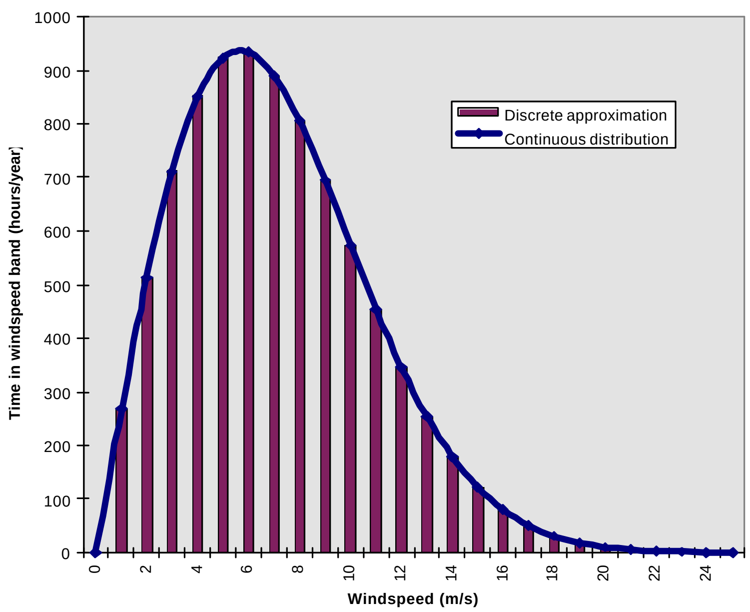


Figure 4.5-1: Comparison of the discrete and continuous wind speed distributions.

4.6 Operation in a wind farm: array effects

The array efficiency of a wind farm is difficult to predict. It is a complex function of the aerodynamics of the wind turbines, the turbulence levels, the spacing and physical arrangement of the turbines and the wind direction.

An estimation method based on data generated with a state-of-the-art farm layout simulation utility [4.6-1] by the Institute for Wind Energy is employed. Array efficiencies for wind farms with a variety of turbine diameters, spacings and layouts have been calculated. Where possible, these values are used directly. For intermediate values of the parameters, array efficiencies are calculated using a cubic spline that interpolates the data points.

4.7 Transmission losses

Transmission of the electricity produced by an offshore wind farm to shore can be subject to significant losses. This is accounted for as part of a grid connection model developed by the Institute for Wind Energy, that forms an integral part of the cost model. For further details are provided in section 5.3 and [4.7-1].

5. Investment cost calculation

The investment cost model is composed of three main parts; the wind turbine cost, the support structure and installation cost and the grid connection cost.

5.1 Wind turbine cost

Two means of estimating the wind turbine cost are incorporated in the model, one very simple, and the other more generally capable.

5.1.1 Simple model

The former simply uses two real turbines, along with their real costs and loadings as calculated by their manufacturers. Of course this is an extremely constraining approach, it limits the number of wind turbine variations that can be considered to two!

Both of the wind turbines incorporated within the model are very modern being specifically the NEG Micon M2300 1000kW wind turbine and the Kvaerner-Turbin WTS-80 3MW wind turbine. The latter indeed exists only as a paper design although it will be realised in the near future.

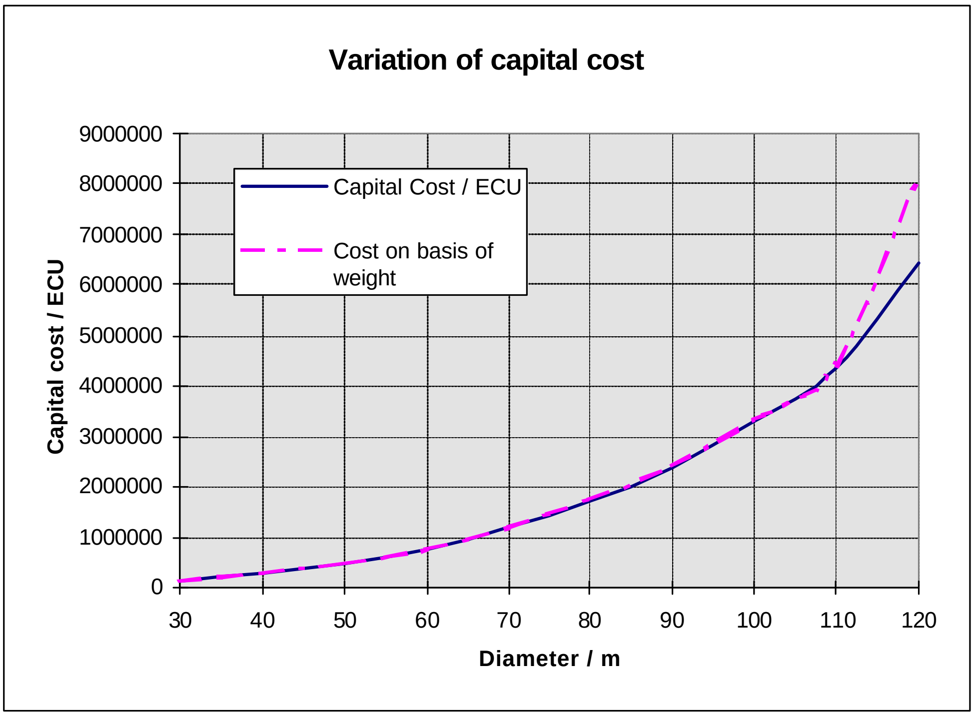


Figure 5.1-1 : Assumed dependence of turbine capital cost on rotor diameter. The solid line shows the relationship used, and the dashed line shows for comparison, the relationship predicted on the basis of turbine weight alone.

5.1.2 General model

The more general implementation adopts a simplified version of the weight based turbine cost modelling approach of Harrison and Jenkins [5.1-1]. There is insufficient space here to give a detailed description of the operation of this model and reference should be made to the original report on the technique.

The Harrison-Jenkins cost model was setup to simulate a turbine comparable to the Kvaerner-Turbin WTS-80. The blade diameter was varied between 30 m and 120 m and the predicted influence on the rated power, cost and weight of the turbine noted. The results from the investigation are summarised in figures 5.1-1, 5.1-2 and 5.1-3.

Modelling rated power, diameter, weight and cost variations

The results from the Harrison-Jenkins model have been used to compile a look up table describing the rotor diameter, turbine cost and turbine weight as a function of the rated turbine power output. The table is incorporated within the Opti-OWECS cost model. The user is free to specify the required rated turbine power as an input parameter, with the model interpolating the other quantities from the tables.

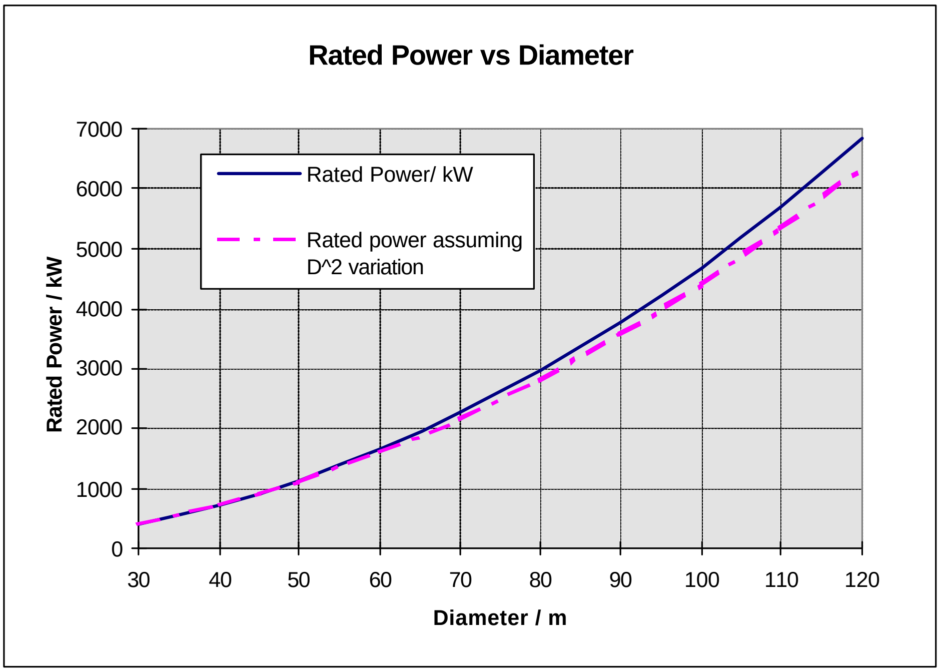


Figure 5.1-2 : Relationship between turbine rotor diameter and rated power. The solid line shows the relationship predicted by the Harrison-Jenkins model, as used in the calculations, while, for comparison the broken line gives a simple prediction.

While the general form of the results from the Harrison-Jenkins cost model have been shown to be representative [5.1-1], the absolute values are rather less reliable. A

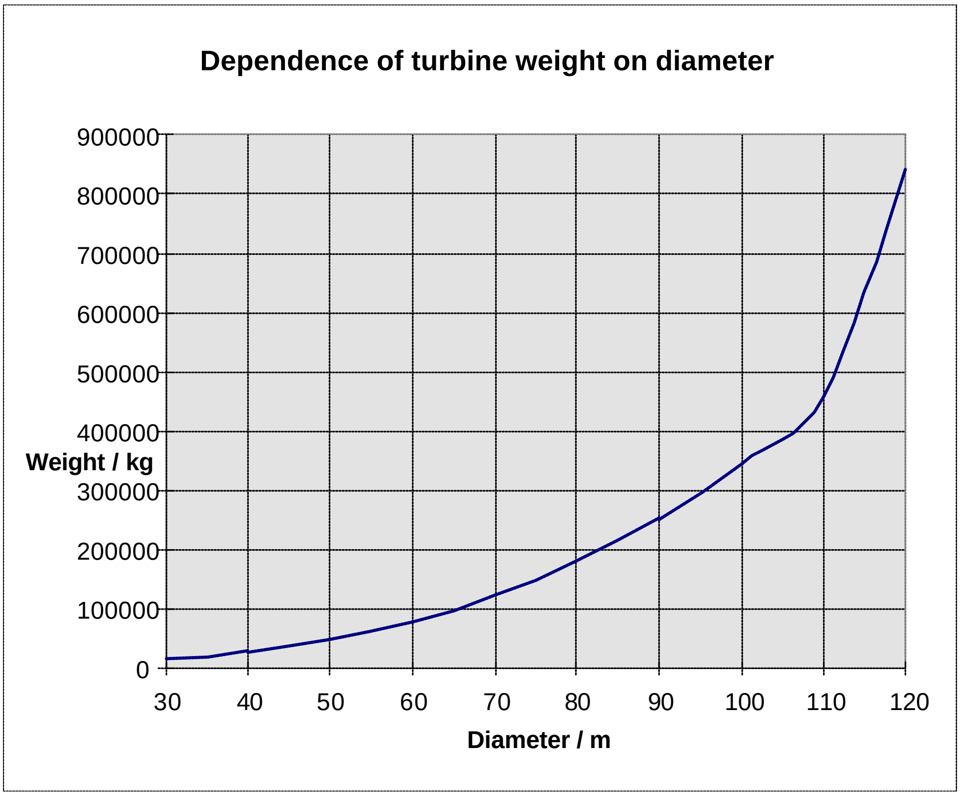


Figure 5.1-3 : Relationship between turbine rotor diameter and weight as predicted by the Harrison-Jenkins cost model.

calibration mechanism has therefore been incorporated in the Opti-OWECS model. One fixed point data set (power rating, cost, weight, diameter) is user specifyable. All turbine specific data estimates are scaled such that the relationship between the variables passes through the specified point. In other words, the look up table will specifies the form of the relationship between the major turbine parameters, whereas the absolute values are set by the calibration point.

Modelling the tower-top fatigue loadings

The results from the Harrison-Jenkins model do not address one important effect of changes in turbine size that must be accounted for by the Opti-OWECS cost model, specifically how the fatigue loads might vary with turbine size. In the absence of better information suitable for use within the cost model, the procedure adopted by Pauling [5.1-2] has been employed here. It is assumed that the fatigue loads are related to the axial thrust on the rotor at rated conditions, given by

$$F_{ax} = c_{D,ax} \frac{r_{air} V_r^2}{2} \cdot \frac{p D_r^2}{4} \qquad (5.1\text{-}1)$$

where r_{air} is the density of air, v_r is the rated wind speed, and D_r is the rotor diameter. Thus the fatigue loading is assumed to scale in magnitude with the square of the rotor diameter. The fatigue loadings calculated by Kühn [5.1-3] for the WTS-80 machine and described in the next section are used as a basis from which to estimate fatigue loadings for turbines of differing size.

5.2 Support structure cost

Overview

The support structure cost model has been developed specifically for the calculations of the Opti-OWECS project with substantial assistance from KOGL and WOT. The calculations are quite involved and it is only possible to give an overview here.

The objective of the model is to provide a representative cost figure for the most economical support structure that will serve in the situation under examination. Any costing is clearly very dependant on the design of the support structure, so in order to make its calculation, the cost model must incorporate a rudimentary means of support structure design. There is interest in assessing the effects of changing the turbine loadings exerted on the support structure, changing the height of the support structure above MSL, and varying the environmental conditions including the water depth, the wave loadings, and wind loadings directly on the support structure. The model needs to be capable of accommodating such changes.

An extremely rudimentary model of a lattice tower is implemented within the model. Only a fixed design is possible, suitable for use with the WTS-80 turbine in waters of 25 m depths, with a unit cost and weight hard coded in the models internal routine.

Effort has focused mainly on producing a model capable of dealing with monotower designs, with either a gravity base or a piled foundation. An outline description of a monotower can be provided by defining the section thickness and diameter as a function of height above sea level. In most cases there are a large number of thickness and diameter profiles that could provide a technically acceptable support structure. A search routine has been developed, therefore, that automatically investigates the range of acceptable towers, subject to certain specified criteria such as the overall height, that are kept constant, and identifies the most economical.

Design and costing approach

A great simplification in the analysis is brought about by treating the monotower as if it was made of a number of discrete sections. Figure 5.2-1 illustrates the concept for a tapered monotower. Each of the tower sections is of equal height, and there is no change in properties within each section. Details of connection to the foundation are ignored completely. It should be understood that the stepped towers modelled by the routines are an approximation to a smooth tower. There is no intention of really designing a 'discontinuous' tower which might pose considerable problems in practice.

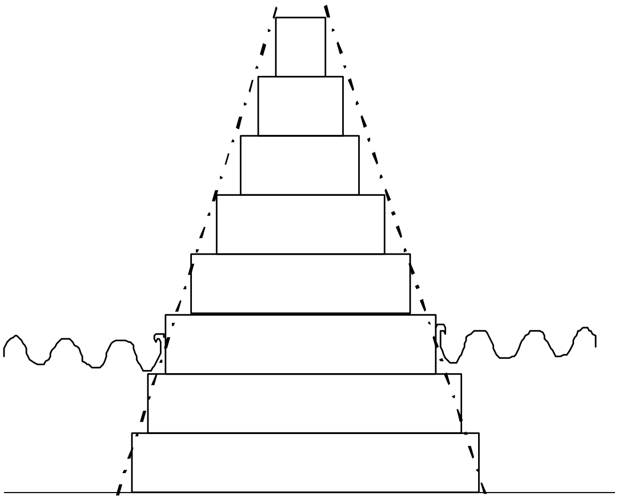


Figure 5.2-1: The stepped monotower approximation.

An iterative design procedure is employed, as illustrated by figure 5.2-2, with the discrete support structure sections being initially sized on the basis of fatigue caused by fluctuating loads produced by the turbine alone. The fluctuating loads themselves are a function of the natural frequency of the support structure (see later for a description of how they are obtained), and once an initial sizing of the sections has been obtained, the natural frequency of the support structure is estimated. If this differs significantly from the initially assumed value, then the sections are resized and the frequency re-estimated, the process continuing until convergence. The fatigue based calculation assumes that ratio of the thickness of the section wall to the overall diameter remains constant throughout the whole height of the support structure, although the particular ratio used can be specified.

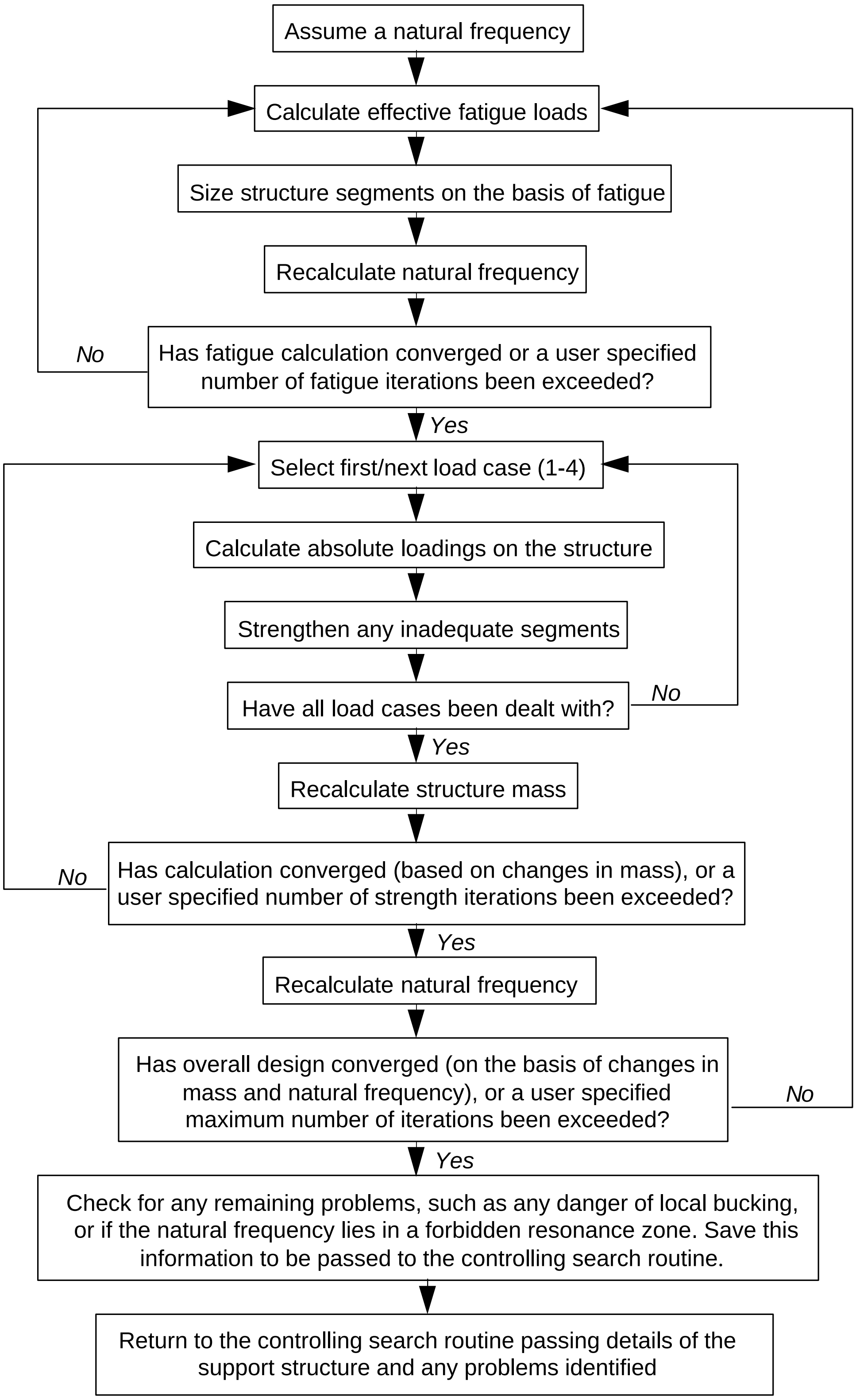


Figure 5.2-2 : Flow chart illustrating the design procedure for a single support structure.

Once a tower that satisfies fatigue considerations has been obtained, the ultimate strength of each section has to be checked against the maximum loads on the entire assembly. Unlike the fatigue calculation the ultimate strength check includes not only forces on the turbine, but also wave and wind forces on the support structure. If any of the sections are found to be incapable of supporting the maximum absolute loads, then they are suitably resized.

Any such changes, of course, influence the natural frequency of the support structure, and hence the fatigue loading. Thus a further estimation of the natural frequency is calculated. If this is significantly different from that used for the fatigue design, then the process returns to the beginning repeating the fatigue calculation. In this “second pass”, the existing design is used as a basis and where the previously computed ultimate loadings demand a larger section modulus than the new fatigue calculation, then the larger section is retained. The remainder of the design process is then completed, and iteration of the outer loop continued until the support structure design has converged.

With the dimensions of a completely satisfactory support structure design available, a cost estimate can be made. This includes four contributions: materials, the cost of constructing the support structure, installation at the wind farm sites and other costs such as project management.

A numerical search routine evaluates a range of support structures, designed according to this algorithm, produced by varying the thickness/diameter ratio employed in the fatigue sizing, and attempts to identify the most economical design.

The following sections describe the workings of the various aspects of the support structure design procedure in greater detail.

5.2.2 Fatigue dimensioning

If the fundamental eigenfrequency of the support structure is beyond the wave excitation range or a location in sheltered waters has been chosen, it can be assumed that support structure fatigue of an OWEC is driven almost exclusively by periodic and stochastic loadings from the turbine. Calculation of such turbine fatigue loadings is an involved process beyond the scope of a cost model, and thus a simplified method is employed. The fatigue loads exerted by the Kvaerner-Turbin WTS-80 machine have been calculated as a function of the natural frequency of the support structure [5.2-1], and are given in table 5.2-1. It should be noted that the negative loadings within the table result from the calculation method employed, and only the magnitude of the values is of physical significance. For the first iteration of the fatigue design routine, an assumed initial natural frequency is used to estimate the fatigue loadings by interpolation from the calculated values.

Fundamental support structure frequency / Hz	0.3	0.35	0.39	0.44	0.55	0.67
Frequency ratio f_o/f_r	0.82	0.95	1.06	1.2	1.5	1.83
Effective tower top moment range / MNm	4.58	-2.40	-0.98	2.86	1.87	-1.62
Effective tower top force range / MN	0.176	0.724	0.683	0.254	0.387	0.773

Table 5.2-1: *Effective tower top loading ranges for $2x10^6$ cycles and 30 year design lifetime calculated for the WTS-80 machine at 22 rpm with a Raleigh distributed wind speed assuming a mean of 8 m/s, and a turbulence intensity of 14%.*

The fatigue loads estimated by this method are valid, of course only for the WTS-80 machine mounted on a structure with a design lifetime of 30 years and for the assumed site conditions. It is of interest however, to investigate the effects that different machines and changes in the design lifetime might have on the design and cost of the support structure. Fatigue loads for turbines other than the WTS-80 are initially estimated from the table and then scaled according to their rotor diameter, as described in section 5.1.2.

To accommodate changes in the design lifetime, the fatigue loadings from table 5.2-1 are scaled according to relations of the form:

$$F_2 = F_1 \cdot \sqrt[m]{\frac{n_2}{n_1}} \tag{5.2-1}$$

where

- F_i Force or moment range associated with lifetime n_i
- n_i Lifetime in load cycles or years
- m Inverse slope of the S-N curve employed for the fatigue design calculation

This expression is effectively the well known relation between loadings for differing fatigue lives given by [5.2-2].

Using the calculated turbine fatigue loadings, the maximum moments at the mid-plane of each support structure segment are calculated. The routine then consults a standard S-N [5.2-3] curve to establish the minimum section modulus necessary to withstand the loading over the working life of the support structure. The section modulus can be translated into the required tower diameter and wall thickness.

With the monotower, the relationship between the section modulus W, the wall thickness, t, and the diameter, D_T, is given by

$$W = \frac{pD_T{}^2 t}{4}. \qquad (5.2\text{-}2)$$

It is assumed here that throughout the support structure, the wall thickness and section diameter are related by

$$t = \frac{D_T}{t_{divisor}}. \qquad (5.2\text{-}3)$$

Hence

$$D_T = \sqrt[3]{\frac{4\, t_{divisor}\, W}{p}} \qquad (5.2\text{-}4)$$

which yields the section diameter. The value of $t_{divisor}$ is set externally, by the optimisation routine, and is the same at every section of any particular tower.

The original determination of W for this calculation involves not only a simple 'reading off ' of the required value from the S-N curve , but also the application of a partial safety factor and a thickness correction. While the safety factor may be obtained from standard data [5.2-4], the thickness correction for the reference fatigue strength is given by

$$\left(\frac{t}{25}\right)^{0.25} \qquad (5.2\text{-}5)$$

where the thickness t in millimetres is related to the section diameter by equation (5.2-3).

In the initial determination of W therefore, a reasonable thickness value is assumed, and used to calculate the correction. In general the thickness of the adjacent tower section is used, although other options are possible and for the first section a default value is employed. When the diameter becomes available from equation (5.2-4), the thickness can be calculated using equation (5.2-3). This new t value can be used to re-estimate the thickness correction, thence obtain an improved section modulus and so forth, continuing to iterate until a reasonable convergence is reached. To keep the calculation reasonably swift, the convergence need not be very good for cost modelling purposes.

Once all the segments have been sized, the natural frequency of the structure is recalculated. If the result differs substantially from the initial assumption, then the fatigue calculation is repeated using the most recent frequency evaluation. Iteration of the fatigue design process is continued until consecutive values of the structure frequency no longer change significantly.

5.2.3 Dynamics calculation and design

The natural frequency of the support structure must not fall within any of the forbidden ranges within which it would be excessively excited by periodic forces generated by the turbine. For a ‘stepped’ support structure of general geometry, a working approximation to the fundamental period is given by [5.2-5]

$$T_t^2 = \frac{4p^2\left(M_{top} + m_{eq}L\right)L^3}{3EI_{eq}}\left[\frac{48}{p^4} + C_{found}\right] \qquad (5.2\text{-}6)$$

where

$$I_{eq} = \frac{\sum_{j=1}^{n} I_j l_j \cos^2\left(\frac{px_j}{2L}\right)}{L} \qquad (5.2\text{-}7)$$

$$m_{eq} = \frac{\sum_{j=1}^{n} m_j l_j \left(1 - \cos\left(\frac{px_j}{2L}\right)\right)^2}{L} \qquad (5.2\text{-}8)$$

$$C_{found} = \frac{3EI_{eq}}{K_{eq}L} \qquad (5.2\text{-}9)$$

$$K_{eq} = \frac{K_{rot}K_{lat}L^2}{K_{rot} + K_{lat}L^2}. \qquad (5.2\text{-}10)$$

If the resulting natural frequency f_0 lies within one of the forbidden bands specified by

$$\begin{aligned} 0.8P < f_0 < 1.2P \\ 0.8N_bP < f_o < 1.2N_bP \end{aligned} \qquad (5.2\text{-}11)$$

where P is the rotational frequency and N_b is the number of blades, then the support structure geometry must be changed. The nature of the change depends on how the stiffness must be affected to bring the structure into a permitted frequency band. Decisions regarding the dynamics of the support structure design and the resolution of any difficulties are taken by the optimisation routine rather than by the detailed calculation section. Details of the procedure will be given in section 5.2.5

The routine can in principle design structures of the soft-soft, soft-stiff or stiff-stiff variety. Predictions for soft-soft configurations are best considered unreliable, thanks to their sensitivity to hydrodynamic fatigue which is not taken into account by the cost model. No cost-model produced results for soft-soft structures will be presented in this report.

Determination of the foundation stiffness

The stiffness of the support structure foundation plays a crucial role in determining the overall stiffness of the structure/turbine combination. For the gravity foundation, the stiffness is very difficult to determine in a simple manner, and has to be input manually. This simple approach is not disadvantageous, since the model assumes a fixed design for the gravity foundation.

In the case of the monopile the stiffness can either be input manually, or calculated using a 'fixity length' approach. In this latter case, the stiffness pile is modelled by assuming that it provides a fixed clamping point some distance below the sea bed. The length is determined as a function of the pile diameter, the soil type and the length of the pile.

5.2.4 Absolute strength calculation

The final calculation needed for the support structure design is to ensure that it is sufficiently strong to resist the absolute forces exerted on the wind turbine and structure without buckling failure.

Firstly the maximum loads on the support structure must be calculated, including not only the loads from the turbine but also wind, wave and any ice forces on the support structure itself. Four load cases are considered using conditions taken from the Germanische-Lloyd regulations [5.2-6], cases E.2.1 (extreme 50 year gust combined with a reduced wave), E.2.2 (extreme 50 year wave combined with a reduced gust), S.2.1 (fault with pitch system combined with annual gust and reduced wave) and E.2.4 (extreme 50 year sea-ice loading). Next, the maximum loading at each segment of the structure under each of the load cases must be calculated. The largest of the loadings from the four load cases at any segment is used to asses whether the segment is sufficiently robust, and if necessary the segment is redesigned. The effects of wind, wave and ice forces on the support structure must be accommodated along with wind loadings for the turbines under each of the load cases.

Computationally, the approach taken is to calculate all of the appropriate loads for each tower segment, storing the results in a series of array, with each element of the array representing a particular support structure segment. The maximum absolute loading experienced by each segment is deduced by considering the cumulative effect of all the forces that influence it. For the calculations described here, it is conservatively assumed that all the loads act simultaneously in the same direction.

Support structure wind loadings

The monotower support structure is subjected to wind loads up its entire height, given by

$$F = \frac{r_{air}}{2} v^2 c_d A \qquad (5.2\text{-}12)$$

where ρ_{air} is the air density, v is the wind speed, and A is the cross-sectional area. The basic drag coefficients of each section can be estimated from [5.2-7]

$$c_{d0} = 1.2 + \frac{0.8 \log(10 k_T / D_T)}{1 + 0.4 \log(\mathrm{Re}_D / 10^6)} \qquad (5.2\text{-}13)$$

where k_t is the structure roughness, D_T is the diameter of the tower segment and Re_D is the Reynolds number based on the tower segment diameter. A value for the drag force felt by each segment is calculated by numerically integrating equation (4.2-12) over the height of the segment, taking account of the variation of wind speeds with height using the standard power law relationship. Calculations are performed over the full height of the structure above the mean sea level, neglecting any shielding from the wind that heavy seas might offer.

The drag coefficients calculated by expression (5.2-13) are for infinite cylinders of the specified diameter. Being finite in extent and having a free end, the drag experienced by the support structure will be less than that predicted.

To compensate for this over estimation, the drag coefficients for the support structure segments are reduced by a 'slenderness' factor y_r that accounts for their finite extent. The slenderness factor is calculated according to Eurocode 1 [5.2-8]. It must be born in mind however that the support structure does not stand alone, as it has a nacelle and turbine are mounted on its top, and this will reduce the effect caused by the free end. Before the loads are multiplied by the slenderness factor, therefore, the drag reduction it represents is halved by modifying the slenderness factor. The relationship between the basic drag co-efficient predicted by equation (5.2-12), Cd_0, and the drag co-efficient used to calculate the support structure loadings, Cd is

$$Cd = \frac{y_r + 1}{2} Cd_0 \qquad (5.2\text{-}14)$$

Eurocode 1 provides slenderness factor data in the form of a graph. For ease of use in the cost model, this has been converted into an analytic form. For the case closest to a wind turbine and a support structure, the slenderness reduction factor is a function of the slenderness ratio l such that

for $l \leq 10$ $$y_r = \frac{1}{90}(l + 53) \qquad (5.2\text{-}15a)$$

for $l \geq 10$ $$y_r = \frac{1}{360}(l + 242). \qquad (5.2\text{-}15b)$$

The slenderness factor is equal to the ratio of the height of the support structure (from mean sea level to the base of the nacelle), divided by the diameter; using the convention of figure 5.2-3, $l = l/b$. Since the diameter of the support structure varies over its length, an average diameter is employed here.

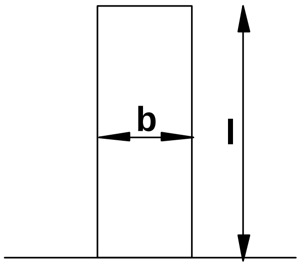


Figure 5.2-3: Definition of the slenderness ratio.

Turbine wind loadings

Static wind loadings on the turbine blades and nacelle are transmitted to the support structure and must be accounted for. Equation (5.2-12) may be employed given suitable areas and drag factors for the blades and nacelle.

The model contains values for the area of the blades of the WTS-80 machine in both fully feathered and operating positions, as well as the frontal area of the nacelle. For turbines other than the WTS-80, the values are scaled with the swept area of the blades. Representative drag factors for the WTS-80 are used to calculate the wind loadings in all cases, although the user can specify alternative values.

Gust response factor

Use of the gust response factor is an attempt to accommodate dynamic effects within the pseudo-static calculation of wind loading. The total wind loads on the support structure segments and the turbine / nacelle are multiplied by a gust response factor G, before being used to assess the absolute loadings. Thus, if all the simple wind drag forces are represented by F_{drag}, the wind forces used in design calculations are given by

$$F_{wind} = GF_{drag} \ . \tag{5.2-16}$$

The pure gust reaction factor G accounts for both the increase in wind speed from the extreme 50 years, 10 minutes value $\overline{v_E}$ to the extreme 50 years 5s gust v_E and the dynamic amplification of the structure's response. In contrast, it is more convenient here to use the modified gust reaction factor G_s which only embraces the latter effect and thus can also be applied to wind gusts with averaging periods other than 10 minutes. The modified and pure factors are simply related with

$$G_s = \left(\frac{\overline{v_E}}{v_E}\right)^2 G \ . \tag{5.2-17}$$

The gust reaction factor used in the cost model is calculated according to DIBt guidelines [5.2-9]

$$G = 1 + r\sqrt{B + \frac{sF}{d_B}} \tag{5.2-18}$$

where the terrain factor, r, is

$$r = 2 \times g \times 2.45 \times \sqrt{0.005} \times \left(\frac{h}{10} \right)^{-0.16} \qquad (5.2\text{-}19)$$

the base gust factor *B* is

$$B = \frac{8}{9} \int_0^{\frac{900}{h}} \frac{1}{1 + \frac{xh}{450}} \cdot \frac{1}{1 + \frac{xb}{120}} \cdot \frac{x}{\left(1 + x^2\right)^{4/3}} dx\ , \qquad (5.2\text{-}20)$$

the size factor *S*,

$$S = \frac{p}{3} \cdot \frac{1}{1 + \frac{8}{3} \frac{f_o h}{v_m}} \cdot \frac{1}{1 + 10 \frac{f_o h}{v_m}} \qquad (5.2\text{-}21)$$

and the gust energy factor *F*

$$F = \frac{4}{3} p \frac{x_o^2}{\left(1 + x_o^2\right)^{4/3}} \qquad (5.2\text{-}22)$$

with

$$x_0 = 1200 \frac{f_o}{v_m} \qquad (5.2\text{-}23)$$

and

g	=	3.5
h	=	hub height
b	=	tower diameter
f_0	=	structure eigenfrequency
v_m	=	10 minute, 50 yr windspeed at hub height
d_B	=	logarithmic damping decrement, usually 0.1

The DIBt guidelines are formulated assuming that the support structure has a constant diameter, which is not the case here. The calculation of the gust reaction factor is performed with the height averaged diameter of the segments above sea level.

Support structure wave loadings

The quasi-static wave loadings on each segment of the modelled support structure, at or below sea level, are calculated from the Morison equation [5.2-10], analytically integrated over the height of the segment.

The Morison equation assumes that wave forces per unit length on a structure are comprised of an inertial component f_i and a drag component f_d, the total force at any instant being given by the sum of the two components. The inertial component is

$$f_i = \frac{C_m r_w p D^2}{4} a_w(z,t) \qquad (5.2\text{-}24a)$$

and the drag component

$$f_d = \frac{C_d\, r_w D}{2} \left| v_w(z,t) \right| v_w(z,t) \qquad (5.2\text{-}24b)$$

Using linear (Airy) wave theory, the expressions for the velocity and acceleration of the water particles are respectively,

$$v_w(z,t) = \frac{w_w H_w}{2} G_W(z) \cos(-w_w t) \qquad (5.2\text{-}25)$$

and

$$a_w(z,t) = \frac{w_w^2 H_w}{2} G_W(z) \sin(-w_w t) \qquad (5.2\text{-}26)$$

using the deep decay function

$$G_W(z) = \frac{\cosh(kz)}{\sinh(kd)} \qquad (5.2\text{-}27)$$

where z is the upwards positive height co-ordinate having an origin at the still water line.

The values of the drag and inertia coefficients are taken to be the same as those used by KOGL for the detailed support structure design, i.e. $C_d=0.74$ and $C_m=2.0$, although the cost model allows the user to vary these values.

The resulting expressions are a function of time and the wave height. A numerical search routine examines the behaviour of the loadings over a wave cycle, identifying the maximum loading for each segment, using the wave height appropriate for the load case under consideration.

As with the other loading calculations, the forces and moments are calculated on a segment by segment basis and stored within an array along with their corresponding points of action. This information can be used to estimate the wave loading at any point on the structure.

Support structure ice loading

Ice loads experienced by structures in the Baltic can be significant. The model estimates ice loads on unprotected cylindrical support structures using a method from [5.2-11], which gives the maximum force exerted by crushing ice as

$$F = k_1 k_2 k_3 b h s_{ip} \qquad (5.2\text{-}28a)$$

where

k_1	=	Structure shape factor
		0.9 (round shape), 1.0 (rectangular shape)
k_2	=	Ice to structure contact factor
		1.0 for an adfrozen floe
		1.5 for a thick ice collar frozen to the structure
		0.5 for continuously cracking ice
h	=	Ice thickness
b	=	Structure width (at a depth of h/3 from ice upper surface)
s_{ip}	=	Compressive strength of ice
		3.0 MPa for current or wind driven intact ice at the coldest time
		2.5 MPa for slow moving intact ice at the coldest times
		1.5 MPa for moving intact ice in the spring
		1.0 MPa for partially weakened melting ice

and the shape ratio factor is

$$k_3 = \left(1 + 5\frac{h}{b}\right)^{0.5}. \qquad (5.2\text{-}28b)$$

The model user is free to specify the ice thickness and compressive strength.

The model incorporates a means to estimate the loads and extras cost that would be experienced by a support structure fitted with an ice-cone. It appears though that, in tidal waters or areas with considerable surge, an ice cone offers no advantages over an unprotected structure, and the calculation procedure will not be detailed here.

Mass eccentricity

The eccentricity of the nacelle mass causes further quasi-static loading of the support structure. The resulting moment is assumed to be given by

$$M_{ecc} = 0.8 m_{top} g l_{ecc} \qquad (5.2\text{-}29a)$$

with

$$l_{ecc} = \frac{D_R}{25} \qquad (5.2\text{-}29b)$$

where

M_{ecc}	=	Moment due to nacelle mass eccentricity
m_{top}	=	Mass of the nacelle

l_{ecc} = Moment arm between nacelle centre of gravity and support structure centre line.

Limit state criteria

For each of the load cases considered, every segment of the support structure, under the combined action of the loadings described above, is assessed for local buckling and for ultimate strength. If either of these criteria are not satisfied, then the structure is ‘marked’ as unsatisfactory and rejected as a design solution.

Buckling is assumed to occur if the section diameter/wall thickness ratio of any segment exceeds a either a user specifyable value or a default value of 175.

Ultimate strength is assessed by calculating the overall loading on the segment, and thence the maximum wall stress. The result is compared to a user specified maximum permissible stress.

5.2.5 Structure optimisation

The structural optimiser ‘drives‘ the structural design routines outlined in the preceding sections, designing a series of support structures within the constraints input by the user, and identifying the most economical. Identification of the optimal support is an involved process, and it will not be practical to give more than an overview of its operation here.

Figure 5.2-4 summaries the operation of the optimisation algorithm. Essentially the routine varies the thickness to diameter (*t/D*) ratio used to design the structure, and identifies that which results in the use of the least amount of material. The optimisation is constrained, however, and must avoid selecting structures which have unacceptable natural frequencies or may be prone to buckling.

The search begins by designing three structures, one with the lowest practical *t/D* ratio, one with the largest ratio, and one with *t/D* of 55. The natural frequencies are examined. In the event that all three support structures are found to have resonance problems, then the design process is adjusted to produce stiffer structures. This is achieved by gradually scaling up the fatigue loads until at least one of the *t/D* ratios produces a dynamically acceptable structure. This is a rather arbitrary solution to the problem of resonance, as in practice there are likely to be better solutions to the problem of resonance. For cost modelling purposes, however it has the effect of making support structure specifications that are prone to resonance economically disadvantageous and therefore unlikely to be chosen in any parameter study.

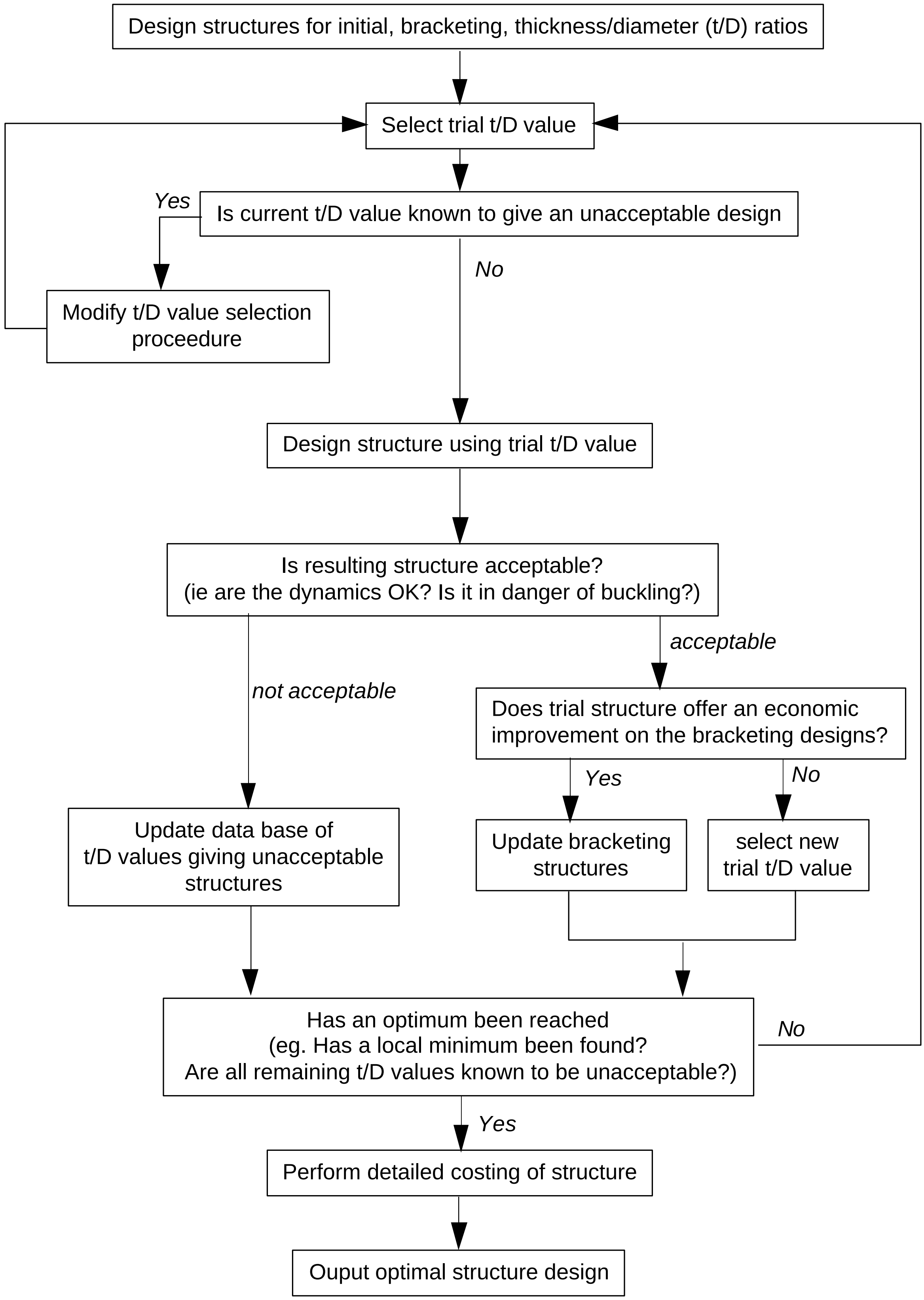


Figure 5.2-4; Flow chart for the structural optimisation procedure.

The decision as to what type of support structure to design is made on the basis of outcome of the three initial designs (after having been stiffened if necessary to ensure at least one provides an acceptable support). If any of the initial cases lie within the soft-stiff range, in other words if a soft-stiff tower is at all possible, then the routine aims to design a soft-stiff tower. If none of the designs lie within the soft-stiff range, then a stiff-stiff tower becomes the objective. Soft-soft towers are permitted by the routine but for reasons noted earlier, do not provide reliable results.

Assuming that at least some of the starting point support structures are dynamically satisfactory, then the search process can begin. The optimisation technique employed is a greatly modified version of the well known bisection method [5.2-12], in which the optimum is continuously bracketed by three points, which will be denoted here as low, mid and up (figure 5.2-5). A trial *t/D* value is selected, lying between the 'low' and 'up' values, and a new structure designed using it. The qualities of the new structure are examined. If it is found to be both structurally satisfactory and less massive than the structures associated with the adjacent up or low *t/D* values, then the bracketing values are updated. The routine maintains a database of those *t/D* values which give unsatisfactory support structures along with the reasons for their being unsatisfactory, and this is updated as further unsatisfactory supports are discovered. *t/D* values in the database are excluded from reconsideration.

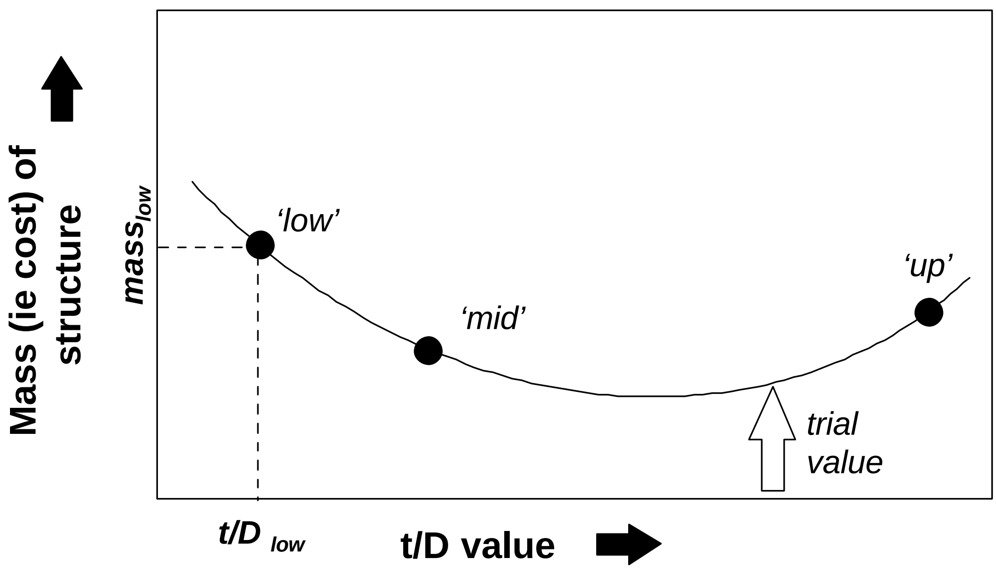


Figure 5.2-5 : Illustration of the bisection search routine.

The trial process continues iteratively, and by alternately testing values between the mid and lower bracketing points, and between the middle and upper bracketing points gradually converged on the least massive support structure, at the same time eliminating structurally unacceptable designs. When all three bracketing values lie within a user specified tolerance of each other, the search finishes returning the middle bracketing point as the optimal tower design.

A number of pragmatic assumptions, based on intuition rather than analysis, are made by the search routine. While these have been found to be valid in nearly all the cases tested so far, some unusual circumstances may cause the routine to fail to converge, or behave erratically. The most important limitations of the optimisation are:

- It assumes that a single constrained minimum exists within the range of *t/D* values initially bracketed Thus the routine is liable to become unstable if no minimum is found. In addition no attempt is made to avoid convergence on false minima, although none of the cases investigated have exhibited false minima.

- It assumes that there is a 'simple' and continuous relationship between the *t/D* ratio of the tower and its natural frequency, such that, in broad terms, increasing the *t/D* ratio always changes the natural frequency in the same direction (be that an increase or a decrease) and vice versa. The relationship does not necessarily have to be univalued, but problems may arise if the behaviour is too 'erratic'. All the cases examined so far have satisfied this assumption.

- It assumes that forbidden zones are contiguous with respect to *t/D*, in other words that all the support structures that are unsatisfactory due to 1P resonance can be characterised by *t/D* values lying within a simple range, and that no acceptable towers lie within that range. Thus, were supports with 1P resonances to have *t/D* values lying in two distinct groups, say in the range 40-50, and 70-80, with acceptable supports lying between 50-70, then the search routine would behave erratically. A similar constraint applies to other forbidden zones (higher resonances). In the cases investigated so far, this assumption has been satisfied.

- To keep the speed of processing acceptably rapid, a few shortcuts have been taken in the implementation of the search routines. As a result of this, the routine can 'miss' optima, and become stuck in an infinite loop, in cases where only a very narrow range of *t/D* values result in acceptable structures. A few cases have exhibited this limitation.

5.2.6 Cost calculation

The costs associated with the monopile and monotower (gravity base) structures are calculated in slightly different ways. Both employ the same computational approach, parameterising the cost calculation as a set of equations that relate certain unit costs and features of the structure to the overall cost of a structure and OWECS. The unit costs can be specified by the user of the cost model, and those employed here have been arrived at in consultation with Kvaerner Oil & Gas.

The discussion here takes no account of decommissioning costs which will be important in any real OWECS. Decommissioning costs are accounted for in an extremely approximate manner as a user specified fixed percentage of the overall costs. The decommissioning costs are added to the overall cost of the structures for an OWECS after being discounted over the life of the OWECS.

Monotower with gravity base

The cost of the monotower support structure comprises four parts which will be dealt with separately such that

$$\mathrm{C}_{\text{overall}} = \sum_i \mathrm{C}_{\substack{\text{material i}\\ \text{structure}}} + \mathrm{C}_{\substack{\text{construction}\\ \text{structure}}} + \mathrm{C}_{\text{installation}} + \mathrm{C}_{\text{management}}\,. \qquad (5.2\text{-}30)$$

Given knowledge of the quantity of any material i needed for a single structure, the cost for the whole farm is given by

$$C_{\substack{material\,i\\ structure}} = \begin{pmatrix} material\ i \\ weight\ per\ unit \end{pmatrix} \times \begin{pmatrix} no.\,of \\ units \end{pmatrix} \times \begin{pmatrix} material\ i \\ \cos t\ per\ kg \end{pmatrix}. \qquad (5.2\text{-}31)$$

By default, two materials are considered: steel for the superstructure, the weight of which is calculated using the method outlined above, and ballast for the foundation with a weight estimated as a fixed fraction of the steel weight.

The cost of construction of an entire OWECS system within a dry-dock (it is assumed that the turbine is supplied pre-assembled, ready for mounting onto the support) arises from the expense of the dry-dock, the labour required for actually assembling the structures, and the cost of constructing equipment needed for installation of the units at their site

$$C_{\substack{construction\\ structure}} = C_{drydock} + C_{labour} + C_{installation}\,. \qquad (5.2\text{-}32)$$

Dry dock costs are partly fixed, the cost of establishing the facility, and partly a function of the build time

$$C_{drydock} = \begin{pmatrix} establishment \\ \cos t \end{pmatrix} + \left[\begin{pmatrix} annual \\ operating\ \cos t \end{pmatrix} \times \left(\frac{no\ of\ units}{production\ rate} \right) \right]. \qquad (5.2\text{-}33)$$

Labour costs can be evaluated on the basis of the time required to assemble the three major parts of the support structure, namely the tower, the foundation and the ballast. For each material, denoted *i*, the total labour cost is:

$$C_{labour\ i} = \left(\frac{rate\ per}{man\ hour} \times \frac{man\ hours\ per}{tonne\ material\ i} \times \frac{tonnes\ material\ i}{per\ machine} \right) \times \frac{no.\ of}{machines}. \quad (5.2\text{-}34)$$

The total cost of the specialised equipment required to install the gravity based OWEC units depends on the amount of equipment required. The structures will be installed in batches, as explained in the next section, rather than all at once and thus only enough installation equipment to deal with a single batch is required. Hence the total cost is

$$C_{instalation} = \begin{pmatrix} cost\ of\ each \\ equipment \end{pmatrix} \times \begin{pmatrix} no\ of\ machines \\ installed\ per\ batch \end{pmatrix}. \quad (5.2\text{-}35)$$

Installation of the assembled support structure and turbine units is best carried out in batches. The costing calculation must also be done on a batch basis, such that the total installation cost may be estimated as

$$C_{installation} = \left(\frac{total\ no\ of\ units}{units\ per\ batch} \times C_{batch} \right) + C_{barges} + C_{equipment} \quad (5.2\text{-}36)$$

where C_{barges} is the one-off construction cost of barges needed for the installation, and $C_{equipment}$ is the one-off purchase cost of ancillary equipment.

The cost of installing each batch C_{batch} is comprised two components: the general costs of offshore operation, $C_{offshore}$ and the hire of tugs for motive power C_{tugs}. Both of these can be costed in a similar way, with a mobilisation cost, the actual costs during installation and a cost for any anticipated downtime due to weather problems or similar. The tug costs also include the expenses of towing the assemblies to their site. Thus the expressions used are

$$C_{batch} = C_{offshore} + C_{tugs} \quad (4.2\text{-}37)$$

with

$$C_{offshore} = \left(\frac{mobilisation}{cost\ per\ day} \times no\ of\ days \right) + \left(\frac{installation}{cost\ per\ day} \times \frac{installation}{days\ per\ unit} \times \frac{units}{per\ batch} \right) + \left(\frac{weather\ downtime}{cost\ per\ day} \times \frac{downtime}{days\ per\ unit} \times \frac{units}{per\ batch} \right) \quad (5.2\text{-}38)$$

$$C_{tugs} = \begin{pmatrix} mobilisation \\ cost\ per\ day \end{pmatrix} \times \begin{pmatrix} no\ of \\ days \end{pmatrix} + \begin{pmatrix} towtime \\ cost\ per\ day \end{pmatrix} \times \begin{pmatrix} towtime \\ days\ per\ unit \end{pmatrix} \times \begin{pmatrix} units\ per \\ batch \end{pmatrix} + \begin{pmatrix} weather\ downtime \\ costs\ per\ day \end{pmatrix} \times \begin{pmatrix} days \\ per\ unit \end{pmatrix} \times \begin{pmatrix} units\ per \\ batch \end{pmatrix} . \quad (5.2\text{-}39)$$

Barge costs and equipment costs must be estimated externally from the model.

Other costs are estimated simply as a percentage of the overall support structure and installation costs. Small percentages are assigned separately to the tasks of Project Management, Design, Supervision of the construction work, Supervision of the installation work, and Insurance and certification procedures.

Monopile support structure costs

For the monopile the costs are divided into two main groups, reflecting the very different construction procedure necessary in comparison to the gravity based support. Installation of the monotower would occur in two separate steps, firstly dealing with the foundation (i.e. the pile itself) and next installing the tower, thus:

$$C_{overall} = C_{foundation} + C_{tower} \quad (5.2\text{-}40)$$

Costs here will be developed for an entire OWECS rather than a single support structure.

The foundation costs are distributed among procurement, physical installation, engineering and certification and investigation of the site, such that

$$C_{foundation} = C_{found\ procurement} + C_{found\ installation} + C_{EngCert} + C_{investigation} \quad (5.2\text{-}41)$$

The procurement cost is comprised of the material costs for the piles themselves, for the j-tubes, the cathodic protection and for floatation equipment. All these costs are evaluated as a unit cost, multiplies by the weight of material per pile and the number of piles. Installation costs are made up of a fixed mobilisation costs, and a variable cost dependent on the number of piles to be installed and their weight. Engineering and certification costs are essentially fixed. Costs for the site investigation have three components: the geophysical and environmental work involved in selecting a site is regarded as a fixed cost, whereas the geotechnical investigation needed to identify the precise location for each pile is costed as fixed rate multiplied by the number of piles.

Costs for the tower, that part of the structure above the water surface, arise from procurement, assembly and installation

$$C_{tower} = C_{tower\ procurement} + C_{assy} + C_{tower\ installation} \qquad (5.2\text{-}42)$$

Material procurement is simply costed at a rate depending on the amount of material required to construct the tower:

$$C_{tower\ procurement} = \begin{pmatrix} tower \\ weight \end{pmatrix} \times \begin{pmatrix} material\ cost \\ per\ tonne \end{pmatrix} \times \begin{pmatrix} number\ of \\ towers \end{pmatrix} \qquad (5.2\text{-}43)$$

Both assembly and installation costs are partly fixed, and partly dependent on the number of towers required for the OWECS.

5.3 Grid connection cost

Calculation of the grid connection cost is undertaken by a specialised model developed at the Institute for Wind Energy [5.3-1] in co-operation with Energie Noord West. The grid connection model is integrated closely with the main cost model, and requires no direct user interaction. As the grid model is detailed in a separate report, only a brief overview will be provided here.

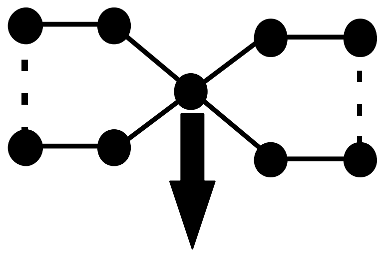


Figure 5.3-1: Circuit type connection of turbines.

There are many ways of connecting together individual wind turbines to 'collect' electricity prior to its transmission to shore, but most schemes can be classified as of one of three type, circuit connection (figure 5.3-1), chain connection (figure 5.3-2) or star connection (not shown, but with an obvious interpretation). Each of the connection schemes has certain advantages. The circuit scheme, for example, is more reliable than the chain type connection, as all the turbines would remain connected were any single cable to fail, but it is also more expensive requiring a larger quantity of cable. The star connection scheme requires the longest cable lengths and is thus the most expensive, but provides the greatest reliability in the event of multiple cable failures.

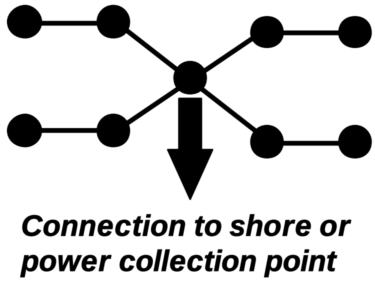


Figure 5.3-2: Chain type connection of turbines.

A physical limit exists on the number of turbines it is possible to connect in one circuit as a result of both the capacity of the cables and the voltage drop along the cable length. The maximum number of turbines per circuit is therefore a function of the turbine rated capacity and the spacing of the turbines. Turbines in a large wind farm are therefore connected in a clustered arrangement, with the clusters connected to a central power collection point, as illustrated diagramatically in figure 5.3-3.

The IvW model allows the user to investigate a wide range of connection schemes, estimating both the capital cost and the electrical efficiency of the arrangement. In addition to the geometric layout, both electrical features including the use of AC or DC equipment and the nature of any transformers and mechanical features, such as whether overhead or undersea cables are employed, can be varied. Full account of the predicted electrical transmission losses is taken in calculating the energy delivered by the wind farm to the shore.

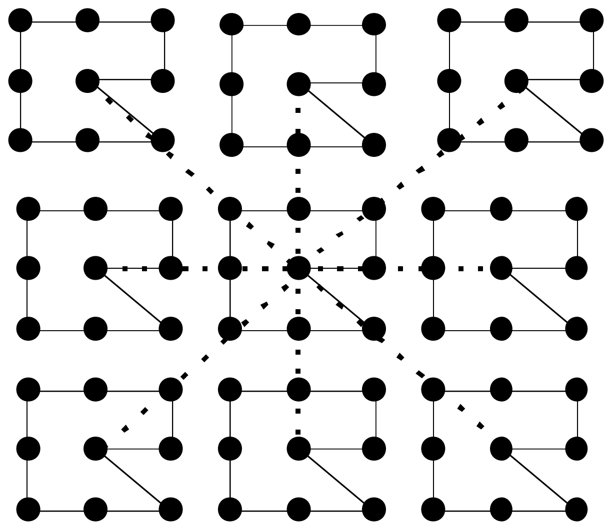


Figure 5.3-3: Layout of turbines and connections as assumed by the cost model. Circles represent individual turbines, solid lines represent 'local' circuit connections, and dotted lines represent trunk cables connecting all the circuits to a central point.

6. Downline costs estimation

6.1 Operation and maintenance costs

Two methods to assess the operation and maintenance costs of an OWECS are included in the model. One is an overly simple and inflexible calculation, the other uses the results of a sophisticated Monte-Carlo simulation of OWECS failures and repair strategies.

6.1.1 Simple model

The simple model estimates operation and maintenance costs by taking a percentage of the total investment cost. Percentages can be specified by the model user, with suitable values being available in the literature.

This simple approach to maintenance costs is highly inaccurate, especially in view of the wide variation of the published values. It does however have the advantage of good generality.

6.1.2 Monte-Carlo simulation based model

The Institute for Wind Energy have developed a computer program that estimates OWECS maintenance costs from Monte-Carlo simulations of wind turbine failures [6.1-1]. The program simulates the operation and maintenance behaviour of an offshore wind farm over a period of time by following the state of each ‘component’ involved e.g. turbine, crew, equipment etc. one time step at a time. Stochastic events, such as the occurrence of failures or the state of the weather are simulated by using a random number generator, acting on assumed probability distributions. Wind turbine failure characteristics can be specified, so that differing machine concepts such as expensive high reliability wind turbines, or low capital cost, disposable wind turbines can be compared. Differing outline maintenance strategies, for example whether repairs are carried out immediately when a wind turbine fails or repairs are only performed when a number of wind turbines have failed, can also be stipulated.

Using the program, maintenance costs for a range of turbines of differing overall reliability have been investigated. Sensitivities of these costs to variations in some important parameters, such as distance to shore, number of turbines in a farm, and the overall maintenance strategy have also been evaluated. The Opti-OWECS cost model incorporates the results of this work in the form of a multi-dimensional look up table, which is used to evaluate the maintenance costs on the basis of the user input.

There are many conceivable OWECS which would fall outside the limited range in which maintenance cost estimates from the calculated cases remain valid. In such situations, it is advisable to use the simple approach to maintenance cost estimation, and discretion must be exercised by the user in deciding which approach to employ for any study.

6.2 Other downline costs

In most cases, the only significant on-going cost associated with an OWECS will be that of operation and maintenance. It may, however, be interesting for the OWECS designer to investigate whether refurbishment or replacement of certain features of the farm, part way through its operating life is economically attractive. As an example, would replacement of the wind turbines after a number of years of operation, whilst leaving the original support structures in place, be economically feasible? For this reason, the model incorporates a facility to specify a schedule of ‘other’ costs to be incurred during each year of wind farm operation. The costs may be specified on an absolute basis, or as a percentage of the initial construction cost of the OWECS.

7. Cost model implementation

The vast majority of the model is implemented as a set of routines written in the Visual Basic for Applications language of Microsoft Excel 5.0 for Windows [7-1]. The spreadsheet and data graphing facilities of Excel have been exploited to ease the input of data and interpretation of the results, but efforts have been made to keep as much of the hard calculation as possible confined to Visual Basic routines. Figure 7-1 illustrates the general philosophy.

This approach has a number of advantages. From the developers viewpoint using a 'proper' programming language rather than spreadsheet macros and functions eases the work considerably. Visual Basic code is quicker to write and debug than a conventional spreadsheet. It is also far more flexible, having a full range of control structures that greatly simplify iterative calculations. From the end users viewpoint however, there is essentially no difference between the finished model and a regular spreadsheet. Data is input into a conventional looking spreadsheet grid, calculations are performed, and the results presented both in a grid and graphical formats.

As far as possible, the Visual Basic routines have been written to be robust. Input data is checked to ensure that it is within ranges that do not invalidate any modelling assumptions and limitations. Intermediate results are, where practicable, checked for consistency. Unacceptable data causes the model to halt, usually with an explicit error message.

A guide to the use of the cost model is provided in volume 5 of this report [7-2].

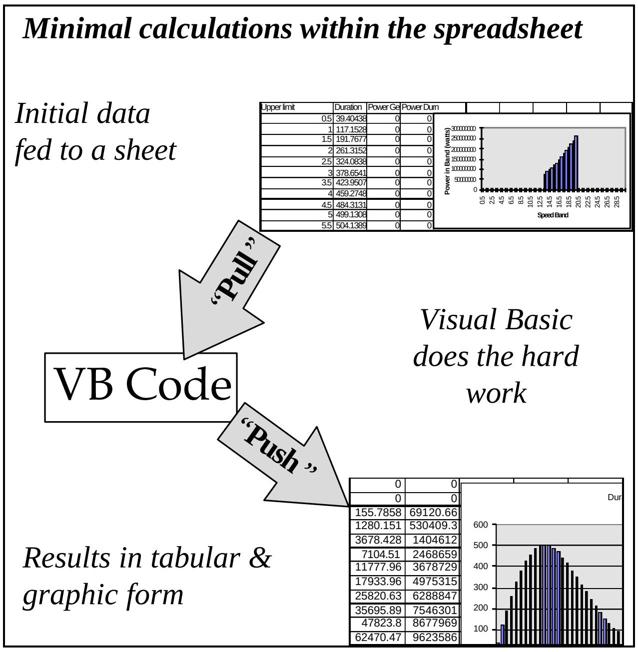


Figure 7-1: The 'philosophy' underlying the cost model.

8. Validation of the model

8.1 Introduction

Before using the cost model ‘in anger’ to compare the base cases of the concept analysis, the cost model must be verified by applying it to some known cases. This was undertaken by using the model to repeat two paper studies of OWECS economics. Further discussion on the accuracy of the model may be found in volume 4, where the model’s predictions are compared to the final design solution produced by the Opti-OWECS project.

A particular difficulty in validating the model is presented by the fact that it has been designed to investigate the economics of large OWECS using at least 25 turbines each of 3MW rated capacity. All existing offshore wind farms are considerably smaller than this size, with the largest, at Tuno-Knøb, being comprised of 10 machines of 500kW capacity. Indeed, until very recently, not even serious OWECS proposals approached the size that can be dealt with by the model.

As a result, model validation is only possible through comparing predictions to the cost estimates presented in a number of ‘paper-only’ studies. Even within this limitation, the validation process is something of a ‘black-art’ as very few publications present sufficient detail to allow the model to examine reproduce exactly the conditions they assume.

The main interest underlying the development of the model is for *comparison* of OWECS concepts and sites. It is clearly unrealistic to expect any general cost model to produce quantitatively accurate cost estimates. Thus, for the validation, it is sufficient that the model predicts the main features of the cases considered, and that its results rank them correctly with regard to economic attractiveness.

8.2 Comparison

Two paper studies of OWECS were recalculated using the model. The studies were selected on the basis that they employ OWECS technology similar to the range of options supported by the model.

8.2.1 Skegness study

A cost model developed at the Institute for Wind Energy [8.2-1] was used to produce a cost estimate for a 60MW offshore wind farm sited near to Skegness in the UK. Table 8.2-1 shows the major parameters used for the original study, which gave a levelised electricity cost estimate of 0.137 ECU/kWh.

Parameter	Value
Turbine power	3MW
No of turbines	20
Support struct. height above sea level	68m
Turbine spacing ratio	5 Diameters
Mean annual wind speed	6.45 m/s at 10 m
Distance to shore	5 km
Distance to grid	0 km
O&M Cost	2% of installation costs
Decommissioning costs	0
Annual interest rate	5%
Economic lifetime	20 years
~~Economic lifetime~~Profit assumed on investment cost	10%~~20 years~~

Table 8.2-1: Parameters used for Skegness calculation.

The study was repeated using the new cost model. Identical parameters were adopted where possible, and in particular the 3MW turbine and the gravity based monotower support structure (designed for 15 m water depth) were employed. A new cost estimate of ~~0.117~~0.114 ECU/kWh was obtained, which compares fairly well with the original result. Figures 8.2-1 and 8.2-2 show the breakdowns of the costs provided by the original and the new model respectively. While there are discrepancies, the results are comparable.

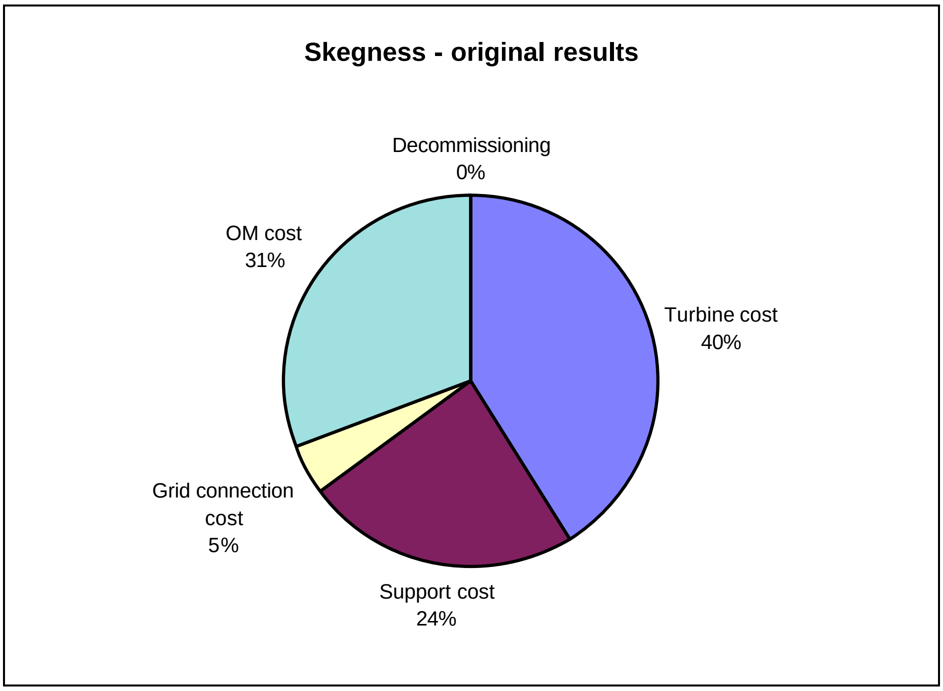


Figure 8.2-1: Original cost breakdown for the Skegness site.

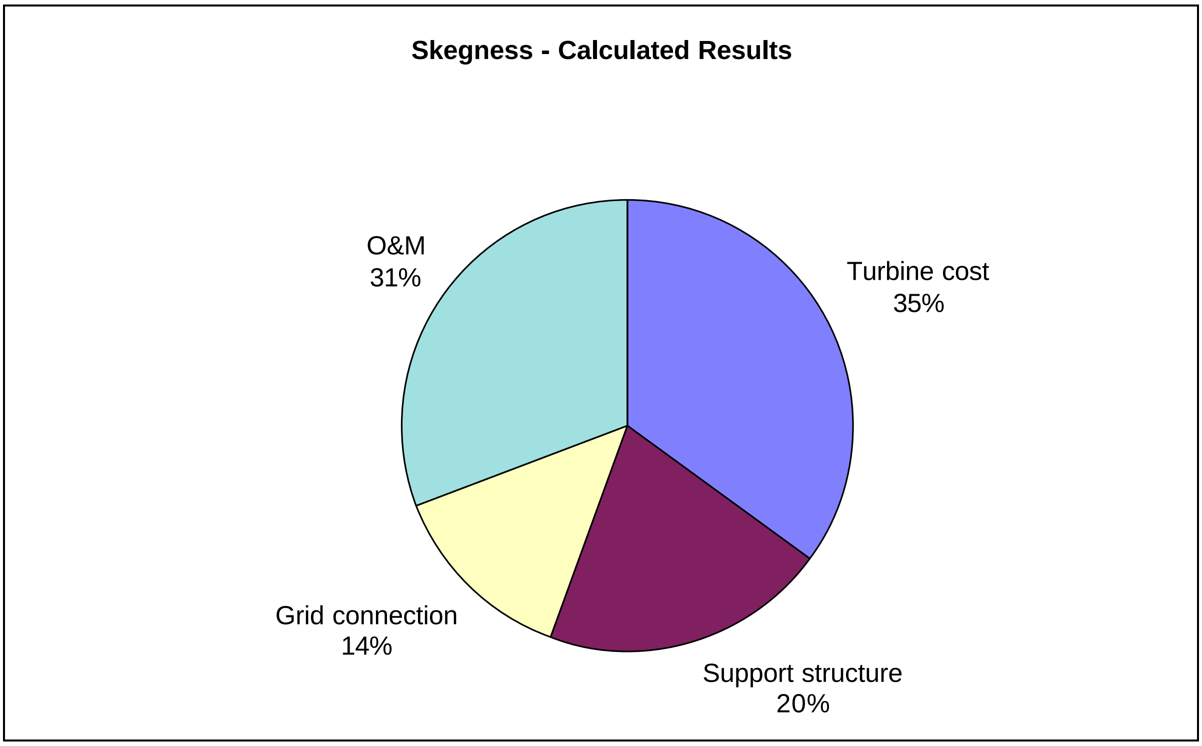


Figure 8.2-2: Calculated cost breakdown for the Skegness site.

8.2.2 SK Power study

The SK Power study [8.2-2] produced a cost estimate for an OWECS sited near to Gedser. With major parameters as in table 8.2-2, a levelised electricity production cost of 0.0650 ECU/kWh was obtained.

Parameter	**Value**
Turbine power	1MW
No of turbines	180
Support struct. height above sea level (hub height)	48m
Turbine spacing ratio	9 Diameters
Mean annual wind speed	8.2s at 47m
Distance to shore	17km
Onshore distance to grid	20 km
O&M Cost	1.4% of installation costs
Decommissioning costs*	0
Annual interest rate	5%
Economic lifetime	20 years

*Table 8.2-2: Major parameters for the SK Power Study calculation. (Note that values marked * are estimated for use in the cost model, and do not appear in the original study).*

Once again the study was repeated using the cost model. In this case the original work employed 180 x 1MW turbines, so to give the same overall production capacity, 120 x Micon 1.5 MW machine were used for calculations. This size of turbine is a long way from the base case design of 3MW, making the support structure design routine somewhat unreliable. For this verification therefore, the support structure calculation was disabled completely and the support structure cost from the original study employed directly. This approach produced a levelised energy production cost of 0.065 ECU/kWh, while the overall cost breakdowns for the original and the new calculations are compared in figures 8.2-3 and 8.2-4.

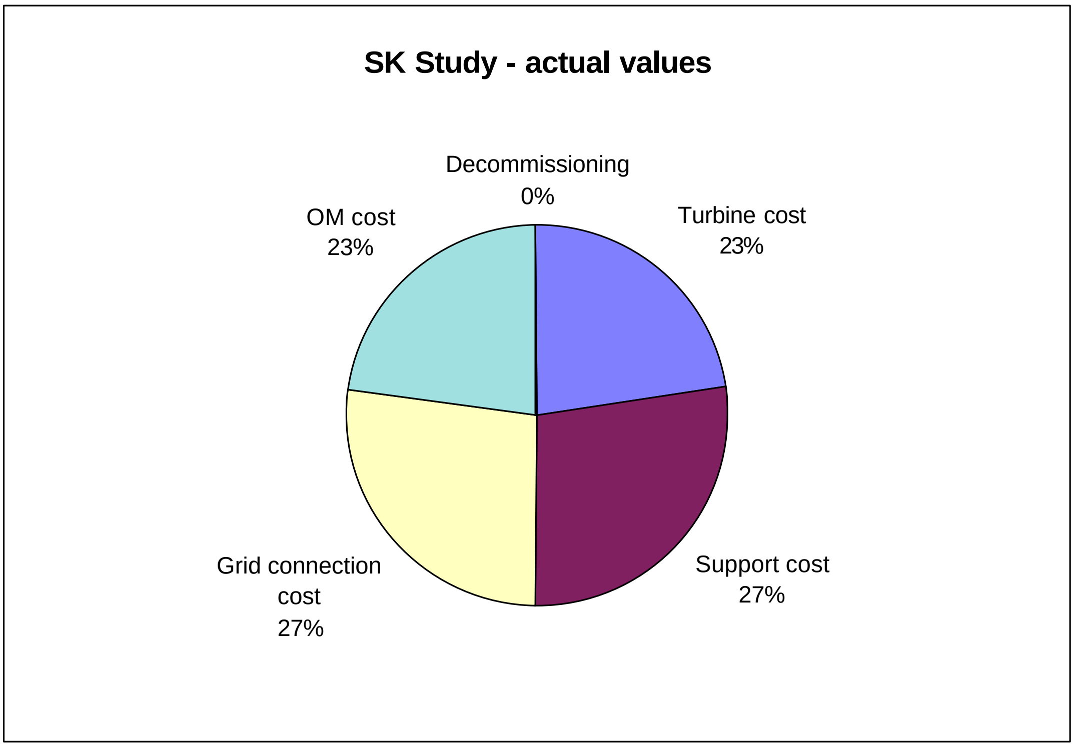


Figure 8.2-3: Original cost breakdown for the SK study.

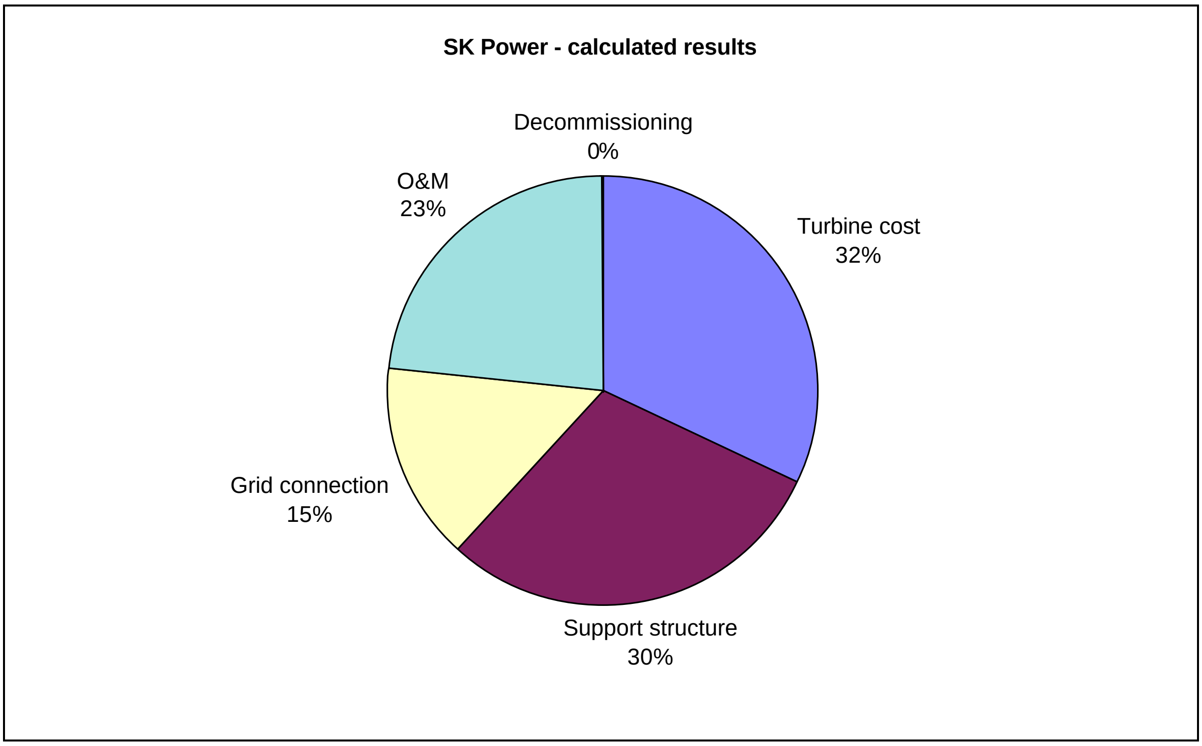


Figure 8.2-4: Calculated cost breakdown for the SK study.

8.3 Conclusion

The verifications presented here are very limited and not entirely satisfactory from a methodological viewpoint. The lack of suitable cases against which to verify the model, however, makes it difficult to see how the validation procedure could be improved. At the time of writing, a number of large scale OWECS proposals have been announced, which if realised, would provide much better test cases for the model. A further difficulty encountered is that much of the published literature describes OWECS in insufficient detail to be useful for validation of the model.

It is possible to draw some positive points from this validation exercise however. Firstly, the overall costs and cost breakdowns predicted by the model are 'broadly correct'. Secondly, the model does distinguish the relative economic appeal of the Skegness and SK-Power studies very well.

9. Conclusions and recommendations

The conclusions for this part of volume 2 relate only to matters that have become apparent as a direct result of the development work needed to produce the cost model. It has been possible to draw further conclusions from calculations undertaken with the model, but these are discussed in volumes 3 and 4, along with the numerical results on which they are based.

Similarly, the recommendations herein are concerned with ways in which the overall functionality of the model could be extended and improved. Shorter term issues, such as the future correction of ‘bugs’ that have been discovered within the model’s code, are discussed in volume 5. Equally, recommendations based on numerical results are described on volumes 3 and 4.

9.1 Conclusions

1. A ‘whole system’ OWECS cost model, based on sound physical and economic principles as far as possible, has been developed. Considerable efforts have been made to address in a rigorous way a number of issues that previous models have avoided including grid connection, installation and operation and maintenance.

2. The cost model has been written in a widely available format (Microsoft Excel 5) which should ensure that the wind energy community can make use of it. Care has been taken to ensure that the model is highly user configurable and modular. These latter points should ensure that the model can be easily updated to take account of future developments in offshore wind energy.

9.2 Recommendations

1. The lattice tower model should be advanced to a state comparable with that of the monotower.

2. The monotower model should be improved to allow a variable thickness/diameter ratio over the tower height. When combined with further guidance from offshore engineers, this would greatly improve the model’s ability to design realistic OWEC support structures. With such enhancements the model could form a first level design tool.

3. The structure optimisation routines would also benefit from improvement. A simulated annealing approach, when combined with the enhancements of point 2 above would provide an exceedingly powerful optimisation and design tool.

4. Means of automatically optimising OWECS designs should be investigated, as suitable routine coupled with the model. Although work has been undertaken here,

a genetic approach would appear to be well suited to the task of overall OWECS optimisation.

5. The foundation models for the gravity base and piled concepts should be improved further.

6. The support structure costing model should be improved to account for the effect of relatively small changes in support structure design (e.g. hub height) on the installation cost.

7. A relation between turbine capital cost and reliability should be developed and implemented in the model.

8. The estimation of tower top effective fatigue loads should be improved, especially with regards to changes in the turbine specification.

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Opti-OWECS Final Report Vol. 2:

Methods Assisting the Design of OWECS
Part B:
Optimisation of Operation and Maintenance

C. Schöntag, G.J.W. van Bussel

Institute for Wind Energy, Delft University of Technology

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Table of contents

1. General notes on operation and maintenance

In this chapter a general introduction of Operation and Maintenance (O&M) will be given as well as an overview of typical O&M tasks for an onshore wind turbine. Further some introducing remarks are given concerning offshore O&M.
A more extensive description of O&M of (offshore) wind farms can be found in [1-1].

1.1 Introduction to maintenance

Every operating system will, at some point of its lifetime, experience malfunctioning and failures. The objective of maintenance is to bring the system back to a state of failure free operation.
Two different types of maintenance policies exist: Preventive Maintenance (PM) that aims at a prevention of failures and Corrective Maintenance (CM) that implies a maintenance action after a failure has occurred.

Preventive maintenance:

PM actions have the objective to reduce the occurrence of failures. PM breaks into two sub types:

- Periodic Preventive Maintenance in which equipment or components are replaced or renewed at regular time intervals before they become worn. This is usually done with components whose replacement costs are low.
- Condition-based Maintenance in which the decision to schedule maintenance is based on periodic inspections. If components are showing signs of degradation appropriate maintenance actions will be carried out.
  - Condition Monitoring: components are periodically or continuously monitored in order to find indications of degradation so that 'as-needed' maintenance can be performed before component failure.
  - Condition Judging: Inspections of components are routinely accomplished, e.g. oil filter inspection. Dependent on the outcome further actions are taken.

Corrective maintenance

CM is performed to restore failed or malfunctioning equipment and components. The occurrence of the breakdown is stochastic, thus CM can not be scheduled.
The failed components can either be replaced or repaired.

Before choosing a maintenance policy for a system, the basic intentions of carrying out maintenance have to be defined. At least two basic approaches can be distinguished, the reliability- and the cost-based approach.
Using a reliability-based approach, maintenance is planned and executed to prevent the reliability index of the system from dropping below a certain specified value at any means. The system must not fail (e.g. nuclear power plants). *Reliability* is considered to be more important than costs.
With a cost-based approach, the costs associated with failure and repair, as well as the results from previous inspections are taken into consideration before planning the next maintenance steps. Here, the dominant factors are *costs* against *reliability.*

Offshore wind energy converters are low-risk structures since major failure will mainly result in loss of investment and loss of energy yield rather than in damage to human beings or the environment. Therefore a reliability-based approach is not required for offshore wind farms and a cost based maintenance approach is adopted.

1.2 PM Schedule for onshore wind turbines

To establish the operation & maintenance strategy it is necessary to first estimate the preventive maintenance requirements for the wind turbines. As experience with offshore wind turbines is still very limited it is necessary to rely on documentation on preventive maintenance concerning land based machines.
Preventive maintenance will normally involve the following tasks:

- Visual inspection of the unit, both in service (e.g. to detect noise) and also out of service (e.g. blade inspection).
- Lubrication of mechanical components, such as blade bearings and links.
- Replacement of any worn items, such as brake pads, oil filters etc.
- Inspection of hydraulic system. Gas pressure test of accumulators
- Re-torquening of bolts on critical components
- General inspection of electrical components

These tasks are normally due at a 6 month interval, for example for the VestasV39, and WEG MS3 wind turbines.
Once a year a detailed inspection is required, this includes a visual inspection of the blades, generator and gearbox. Every fifth year the blades, generator and gearbox have to undergo a major overhaul. Every tenth year certain components have to be exchanged. For the VestasV39 this includes the exchange of (from 1.2-1]):

- Generator Bearings
- Yaw Pads
- Pitch Linkage Bearings
- Pitch Cylinder
- Friction Clutch
- Transmission Shaft Gear

Man hours required for maintenance of Vestas V39		
Service period	First year	Subsequent years
3 month 2 per year	25 x 2 = 50	None
6 month	15	15
12 month	25	25
unscheduled	50	50
Total	140	90

Table 1.2-1: Man hours for preventive maintenance

Table 1.2-1 gives a summary of the man hours required for preventive maintenance tasks on the Vestas V39 [1.2-1]. Reference [1.2-2] gives the annual maintenance requirement of the MS-3 as 11 hours for the six month service and 19 hours for the 12 month service (every 5 years the 12 month service takes 23 hours).
The man hours required for preventive maintenance tasks were estimated at 64 hours for the annual service, and 16 hours for the 6 monthly service. The average annual cost for parts and materials used in PM actions is predicted as 4.000 ECU per offshore wind turbine [1.2-1]. For comparison, [1.2-2] assumea 750 ECU for the MS-3 and 1.500 ECU for the LS-2, based onshore.
It should be kept in mind that the given man hours and parts & materials costs represent experience with rather small land based wind turbines. How multimegawatt wind turbines of the near future will compare to these figures has to be determined yet.

1.3 Introduction to offshore maintenance

Weather limited access of OWECS

Access of the wind turbines is dominated by the state of the weather. The current wave height, wind speed and visibility will determine whether access to the wind turbines will be possible. These limiting factors depend, to a great extent, on the chosen access device. The ability to land by boat, for example, is dominated by the wind speed, wave height, wave type and visibility, whereas access by helicopter is not limited by the sea state, but by wind speed and visibility.
Offshore weather can be divided into two seasons, a long winter term with higher wind speeds and increased storm frequency and a shorter summer term with a more 'friendly' weather environment. Trinity House Lighthouse Services, for example, which are responsible for the maintenance of navigation aids off the coast of England, avoids boat landings for routine maintenance during the winter term at all [1.3-1].
Accessing wind turbines by helicopter has been examined in several projects. The advantage of helicopters over vessels is, that helicopter flights are not limited by the sea state and can operate in winds up to 15-20 m/s. So far, however, no existing offshore wind farm considers this access possibility as a regular way of approach. Yet, the North Sea Oil and Gas Industry relies completely on helicopters for transport to offshore structures, [1.3-2].

Remote control

Offshore wind turbines should be equipped with remote control systems in order to decrease the number of required maintenance trips to the wind turbines. All protective equipment that could trip, such as an overcurrent relay on the supply to the yaw motors, must have a remote reset ability. Important parameters of the wind turbines, such as rotor speed, pitch angle, power, voltage etc., have to be monitored by transducers and recorded by the SCADA system (Supervisory Control and Data Acquisition). Closed-circuit TV (CCTV) cameras could be useful for remote fault diagnosing and especially for estimating the sea state when considering a maintenance visit.

2. Contribution of O&M to cost of energy

In this chapter the O&M cost factors are dealt with. Furthermore the availibility is discussed, which partially determines the energy yield. Also the reliability data of wind turbines is discussed, which determine the need for CM.

2.1 Investigation of O&M cost factors

A break down identifies the factors contributing to the O&M costs. In the case of the maintenance costs these are:

- Facility
- Stock
- Transportation
- Lifting equipment
- Operations personnel
- Maintenance equipment
- Maintenance downtime

Factors contributing to operations costs are:

- Taxes
- Insurance
- Land rent
- Administration

These cost factors have been analysed. As an example a short overview for the lifting equipment will be given.
Over the assumed lifetime of the OWECS it is inevitably that major components of the wind turbines, e.g. blades, gearboxes, generators, will fail and need to be overhauled or replaced. Furthermore a replacement of major parts must be anticipated when their condition cannot be determined unambiguously when installed on the wind turbine. The weight of these components results in the need of heavy lifting equipment, (table 2.1-1).

	Vestas V63	**Näsudden II**	**NedWind NW53**	**WEG MS4**
Rated power (MW)	1.5	3	1	0.6
Blade (each)	3.5	9.6	5.2	1.68
Rotor (incl. hub)	21	41	21	5.05
Nacelle	55	120	44	-
Towerhead	76	161	65	21.6
Tower	80	1,500	44.5	21

Table 2.1-1: Component masses [t] of large wind turbines

Either the major component must be manufactured in such a way that they can be broken down into lighter parts to minimise the required size of the heavy lifting equipment, or complete wind turbine modules or even the complete nacelle are replaced to minimise the duration of the exchange operation. This requires a maintainability study already in the design phase of the wind turbine, in order to take the offshore situation into account. It may even result in the need of special designed wind turbines used offshore to reduce the maintenance costs and/or offshore maintenance equipment.

In general five alternative lifting concepts are available:

- jack-up barge
- crane vessel
- liftboat (purpose adapted self propelled jack-up platform)
- helicopter
- built-in lifting system

Several factors have to be kept in mind when deciding upon the right choice of lifting equipment. The actual lifting operation should be as insensitive to wind speed and wave height as possible. The lifting equipment, when rented from local suppliers should have a high availability and offer short lifting operations, as offshore work is very expensive. Minimum modifications to the existing OWEC design should be required to adapt the wind turbine to the chosen lifting method. When the lifting equipment is bought as part of the OWECS infrastructure, attention has to be paid to low initial investment and running costs.
Furthermore, the low water depth most offshore wind farms will be erected in, has to be taken into account, which might limit the use of certain lifting devices.

The analysis of the lifting operation and of the equipment shows that two general approaches towards the lifting operation can be distinguished: either the use of heavy lifting equipment and the design of the OWEC as modular as possible to reduce the time spent exchanging the failed component, and thus the expensive hiring time of the lifting equipment, or the possibility to break down the major components into sub-components to reduce the weight and thus the required cranage capacity.

	heavy duty crane	wind dependency	wave dependency	investment costs	running costs	availability	lifting durations	rate
buy:								
liftboat	purpose built	0	+	--	-	+	++	++
crane vessel	++	0	0	--	-	+	0	+
jack up barge	++	0	+	--	-	+	+	0
helicopter	--	+	++	0	0	+	++	0
hire:								
liftboat	not posible to hire for that purpose				/	/	/	/
crane vessel	++	0	0	++	--	-	0	+
jack up barge	++	0	+	++	--	–	+	0
helicopter	--	+	++	++	-	0	++	0

(-: negative; +: positive)

Table 2.1-2: Evaluation of different lifting concepts

2.2 Availability

Availability, in simple terms, is the proportion of time a system is capable of performing its tasks.

Key factors that affect the mean time between failure of a wind turbine and, thus, the frequency of shutdowns are:

- design of the OWEC and reliability of the used components
- number of components
- number of potential causes of shutdown
- spurious component failure resulting in spurious trips
- shutdowns necessary for maintenance
- human error
- environmental impact, e.g. saline environment

Key factors that affect the mean time to repair are:

- rapid and efficient fault finding, this includes remote identification of the fault and predetermination of necessary actions
- modularity of the wind turbine's design
- access to the OWEC (weather condition !)
- access to the individual components of the OWEC
- amount on in-situ maintenance requirements

- mobilisation time of maintenance crews
- availability of required spare parts and equipment

The downtime costs

Downtime costs are no direct costs in the sense of costs that arise when the system is down, but rather costs in the sense of loss of revenue due to the system being down. Thus, the downtime costs are penalty costs for taking an operating system out of action, i.e. for preventive maintenance reasons, or for not repairing a failed system. For an accurate assessment of the downtime costs the state of the system before switching into the unavailable state has to be taken into account.

2.3 Reliability data

Disposal of reliability data of applied wind turbines is a crucial factor for an assessment of the availability that an offshore wind farm may be expected to achieve. However, reliability data of modern 1-3 MW wind turbines are not readily available from manufacturers as these machines are either still very new and their installed number limited so that no reliable feedback from operators has been attained yet, or the wind turbines are still in the stage of prototypes and their expected reliability is confidential.
Therefore alternative methods of gathering reliability data had to be chosen. In general there are 5 different approach methods:

- *similar equipment method:* proposed design is compared with similar design for which reliability data is known
- *extrapolation of data from trials:* reliability data from development tests and trials can be extrapolated
- *generic part method:* combination of individual part reliability to form a complete system reliability; failure of parts is independent of each other
- *engineering or expert judgement*
- *reliability data banks*

It was chosen to rely on the last method. Several data bases with reliability data for wind turbines exist:

- EUROWIN
- EPRI
- Wind Stats
- ISET
- Wind Energy VIII, Landwirtschaftskammer Schleswig Holstein, [2.3-1]
- others

Yet, these data bases lack information about the very modern, large wind turbines due to the above mentioned reasons. Furthermore reliability figures are summarised over many different types and rated power of wind turbines so that it is not possible to estimate type specific reliability data. An exception offers Wind Energy VIII. Here, available reliability data of wind turbines installed in Schleswig Holstein, Germany is listed according to type and rated power.

Failure Class	event/year	MTBF (hours)
Blades	0,44	19923
Gearbox/Generator/Yaw	0,14	64933
Electronics/ Control System	0,29	30757
Hydraulic	0,22	40303
Electric	0,37	23730
Others	0,33	26246
Total	1.79	-

Table 2.3-1: Assumed failure classes and their MTBF

Based on this data six failure classes and their MTBF (Main Time Between Failure) were defined to be considered in an availability assessment. A reduction of the total failure rate with about 25% is assumed to be achievable for offshore applictaion withhout major modification of the design. Thus a base case offshore design with respect to O&M is defined, see table 2.3-1. For that purpose several failure classes were combined to create a 'new' failure class. However, it should be kept in mind that these failure classes are still relative crude 'guestimates'.
The names given to the failure classes should not be taken too literally; the class "Blades" include all heavy components which requires the need of an external crane in case of replacement.

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3. Evaluation of O&M

3.1 Introduction

The operation and maintenance strategy can be established along the following steps:

- consider turbine design
- consider maintenance approach
- consider O&M 'hardware'
- define O&M strategy

In general these four aspects have to be considered more than once in order to come to the best O&M strategy. The chosen wind turbine design determines the behaviour of the system 'wind farm' in the first place. The frequency of failures and the required preventive maintenance tasks depend on the reliability of the wind turbine's design. The choice of the lifting equipment, for example, is influenced by the maintainability of the wind turbine's design, e.g. exchange of nacelle or breakdown into smaller components. These few examples already show the importance of coming to the right decision when considering the wind turbine's design and concept. The keyword is 'design for RAMS (Reliability Availability Maintainability and Serviceability)'. An optimum operation and maintenance strategy will never lead to a system performing better than the chosen performance in the design of the wind turbine.
The next step requires the selection of the appropriate maintenance approach which takes the requirements of the chosen wind turbine design into account, e.g. PM required/not required. With the wind turbine design and the maintenance strategy at hand, it is possible to estimate the resulting work load in man hours per year, and to determine the number of required maintenance personnel.
Considering the 'hardware' is next. Decisions about a possible maintenance base, crew transporting devices, lifting equipment, etc., have to be taken. The required lifting equipment depends on the wind turbine's maintainability design, the number of required crew transporting devices on the chosen strategy and the failure rates of the turbines that determine the probability of several failure occurring at the same time.
Finally, using the Maintenance Strategy Decision Tree, the 'fine tuning' of the operation and maintenance strategy is done.

In the following the O&M evaluation will be treated in more detail. First some tools and methods of evaluation are described. Next, in section 3.3 the hardware will be dealt with and in section 3.4 the maintenance strategy will be discussed. Finally, in section 3.5 some combinations of O&M strategy and turbine design are examined.

3.2 Tools and methods of evaluation

Levelised production costs (LPC)

The LPC formula (see part A of this Volume) takes the capital costs plus the costs of resources, devoted to O&M, as well as the produced energy of the wind farm into account. Therefore, it is possible to determine the optimum trade-off between these

three factors by decreasing the LPC to a minimum. Thus, the final objective of the optimisation task are minimum levelised production costs.
As the LPC formula also requires the input of the initial investment costs and the annuity factor, an example offshore wind farm is defined.

Simple cost comparison

The applicability of several factors with respect to offshore wind farming can be separately evaluated without having to examine them in a global context. The different possible choices of the lifting equipment, for example, can be compared by simply calculating the costs of a specific lifting operation using different lifting devices.
Some more advanced methods are based on Markov chain modelling. Exact calculations are however only possible for simplified systems

Monte-Carlo simulation

Other aspects require the consideration of the behaviour of the entire OWECS. Therefore a simple Monte-Carlo program simulating the operation & maintenance activities of an offshore wind farm was written. With the help of this program it is possible to investigate various possibilities of deployment of O&M ‘hardware’ and O&M strategies.

The program simulates the operation and maintenance behaviour of an offshore wind farm over a period of time by following the state of each component of the wind farm, e.g. turbine, crew, equipment etc., one time step at a time. Stochastic events, such as the occurrence of failures or the state of the weather, are simulated by using a random number generator, acting on the assumed probability distributions.

At the beginning of each simulation run, the failure rate of the used wind turbines and the operation and maintenance strategy has to be defined. The number of crews, number of shifts worked per day, and the days worked per week have to be specified, also the kind and quantity of equipment the crews can fall back on, e.g. the number of vessels.
Different maintenance strategies can be evaluated by changing, for example, the input parameters for the time intervals of the year where PM and/or CM is carried out.
At the end of the simulation interval the total O&M costs, the achieved availability and the produced energy of the wind farm are given as an output.

Example offshore wind farm

For an evaluation of different operation & maintenance strategies a 'test' offshore wind farm is necessary, especially when working with the help of the Monte Carlo simulation program, where certain specific wind farm inputs, such as the annual potential energy output of the turbines, are required. Also, certain cost figures, such as the initial investment costs, are necessary to estimate the levelised production costs. Therefore an example offshore wind farm has to be defined.
The assumed wind farm consists of 100 wind turbines with a rated power of 1,2 MW, see (table 3.2-1). Six failure classes and their reliability parameters, i.e. MTBF and MTTR (Mean Time To Repair), are considered to apply to these wind turbines. The failure rates have been taken according to table 2.3-1. Futhermore it is assumed that

for blade failures only, lifting equipment is required. Also assumptions have been made about the repair time for each failure class.
The mean wind speed is 8.5 m/s during the winter and 6.5 m/s during the summer months. A lower availability in the winter will therefore result in higher energy losses compared with the summer period. The annual potential energy output of one turbine amounts to 3,62 GWh, assuming a 6 monthly winter and summer period, respectively.
The initial investment costs account to 191 million ECU. The lifetime of the wind farm is assumed to be 20 years. With a discount rate of 5% the annuity factor amounts thus to 12,46.
The mean wind speed and storm distribution assumed in this example offshore wind farm are valid for the North Sea.

Rated power		100*1.2 MW
Mean wind speed	m/s	7.5
Annual energy output	10^6 kWh	280
Turbine costs	M ECU	71.0
Total initial investment costs	M ECU	191.1

Table 3.2-1: Reference offshore wind farm

3.3 The hardware

Location of maintenance base

The question whether a maintenance base is necessary in the first place, depends to a great extend on the number of installed wind turbines and the kind of repairs and overhauls that will be carried out. For an evaluation of the different locations and kind of possible maintenance bases it is assumed here that a maintenance base is certainly justified.

Onshore
The proposal of a purpose built maintenance base along the coast in order to minimise the distance offshore wind farm to shore, can be ruled out. The sheer numbers of existing, well equipped harbours along Europe's coasts do not justify such a solution. The few minutes travelling time saved will not compensate the initial investment costs for erecting such a base, with facilities, including cranage, docking etc., that are readily available at any existing harbour.

Offshore
Whether the costs of an offshore base are justified, depends to a great extent on the distance offshore wind turbine farm to the next harbour. With the wind turbines erected just a few kilometres offshore, for examples three kilometres in the case of the wind farm at Tunø Knob, an offshore maintenance base is certainly not required. If a wind farm is considered, e.g. 175 km offshore, the case is totally different. Here, the costs for crew transportation from a mainland base to the wind turbines and the

additional costs for transporting every component, requiring major overhaul, to the mainland base have to be weighted against the erection costs of an offshore base.

Type of offshore maintenance base

Three variants for an offshore base are introduced: the support vessel, a central, purpose built support structure, and the liftboat.

Support vessel
The support vessel provides accommodation for crews, permanently stationed within the wind farm. In case of a sudden weather change it offers a relatively safe retreat for travelling maintenance crews. The support vessel is able to move around the wind farm and so help to reduce the travelling time between the individual wind turbines. At regular intervals it can return to a harbour for relief crews and fresh stock of spares, etc.
However, the support vessel offers no stable working 'platform' and the available space onboard is limited. Thus, executing major overhauls, e.g. blades, gearboxes, onboard of a support vessel at a regular basis seems very unrealistic.

Support structure
A maintenance base could be based on fixed structure located centrally in the wind farm. The base could, on the contrary to the support vessel, not only offer accommodation facilities but also work shops for overhauling major components, such as blades or gearboxes. A combination with the voltage transformation housing could be a possible way of reducing the initial investment costs.

Liftboat
The liftboat combines the advantages of a support vessel with those of the purpose built support structure. It is able to move around the wind farm and, once jacked-up, it offers a stable working platform unaffected by the state of the sea.
However, the main reason the liftboat being the preferable choice is that it provides a high capacity crane as well as crew accommodation.

Crew transport

The access either by helicopter or by vessel turned out to be the two most reasonable approaches for offshore wind farm. A cost comparison of helicopter against vessel access shows that the helicopter offers the fastest but most expensive alternative. However, the downtime costs, saved by using the faster helicopter, do not compensate the higher operating cost of the helicopter (see also the comparison given in Volume 4).
Thus, the only advantage of using helicopter for wind turbine access, lies in the decreased weather dependency. This advantage has to be weighted against the initial modification costs in order to adopt the wind turbines for helicopter access.

Lifting equipment

A detailed analysis of the lifting equipment has been carried out. The choice of the lifting equipment is dependent on the chosen maintainability approach in the design phase of the wind turbine. At least two approaches can be distinguished:

- breakdown of the components
- modular exchange of components

The use of helicopters and of the built-in lifting system are possible solutions if the first approach is chosen.
However, it should be kept in mind that helicopter lifting operations are expensive, susceptible to wind gusts, and not possible if components have to be heaved in order to remove bolts, etc. The helicopter is the most expensive lifting device with respect to the ratio costs and offered lifting capacity.
A possible built-in lifting equipment is ready at hand, whenever it is needed. Thus, a fast reaction time in case of lifting equipment demand, is ensured. However, providing every wind turbine with a lifting system means high initial investment costs.

For the second approach, the exchange of modules, three alternative lifting devices exist:

- crane vessel
- jack-up barge
- liftboat

Crane vessel come in three different types: the flat bottom barge, the ship shape type, and the semisubmersible vessel type. Lifting operations with these types of cranes are dependent on the wave height, which restricts the execution of the operations to certain weather conditions. Jack-up barge and liftboat are also dependent on the wave height, but only while not being in the jacked-up position. Once in working position, they offer a stable working platform where lifting operations can be executed regardless of the wave height. The lifting operation itself is, of course, limited to a certain wind speed, which is more or less the same with all three alternatives.

An comparison liftboat contra crane vessel was carried and the outcome is summarised in (table 3.3-1).

number of nacelles to be exchanged in a batch	Semi-Sub	Ship Shape	Flat-Bottom	Liftboat
20	234	227	178	156
40	227	220	171	78

Table 3.3-1: Exchange costs in kECU per nacelle

It can be seen that the liftboat offers the most cost effective way of executing lifting operations if more than 20 lifting tasks per year are required.

3.4 The maintenance strategy

The next step in establishing the O&M strategy requires the selection of the maintenance strategy. In the following five strategies will shortly be discussed with respect to applicability in the field of offshore wind farming. Next it will be dealt with how to make a choice between these maintenance strategies.

Typical maintenance strategies

No-Maintenance Strategy
With this strategy neither preventive nor corrective maintenance tasks are executed. The failure, and thus the shut-down, of wind turbines is intentionally taken into account. Either a redundancy of the wind turbines has to be considered, in order to achieve a certain minimum availability of the wind farm, or a decreasing availability over time has to be accepted.
The wind turbines are exchanged when either the availability drops below the predefined minimum availability of the wind farm, or the wind turbines reach the end of their nominal lifetime. Alternatively one could also think of replacing single wind turbines as soon as they fail.
The application of a strict No-Maintenance strategy is not economical under present offshore regulations. Also the assumed MTBF of more than 5 years is far too optimistic looking at reliability data of present wind turbines. Thus, it can be said that the No-Maintenance Strategy might be a choice of the future, when wind turbine design has improved dramatically, but, at present, it is no sensible approach for 'maintaining' an offshore wind farm.

Only-CM-Maintenance Strategy
With this strategy only corrective maintenance tasks are executed. The wind turbines are repaired either as soon as they fail, or a certain number of failed wind turbines has been reached. Here again, no permanent maintenance crews - for carrying out the actual corrective maintenance tasks - are needed. These crews could be hired on a stand-by basis to be mobilised at short notice, or from maintenance companies on demand. Whether other, permanent, crews at the base are required, i.e. for overhauling blades, generators, etc., depends on the size of the wind farm and the amount of work one intends to carry out under own supervision.
With present wind turbines being designed for onshore use, the Only-CM-Maintenance strategy seems not appropriate. The number of required preventive maintenance tasks to be undertaken are too high to justify such a strategy. However, with a purpose designed wind turbine for offshore application, where special emphasis is put on the ability of self-maintaining, such as self-lubrication bearings, this strategy appears reasonable.

Opportunity Maintenance Strategy
This strategy is very similar to the Only-CM-Maintenance strategy. The main intention is to execute CM tasks. However, if a wind turbine undergoes corrective maintenance, the chance is used and preventive maintenance tasks are also carried out at the same turbine. This means that preventive maintenance is executed at very irregular intervals, and only after the failure of the wind turbine. The philosophy behind this strategy is to reduce the number of visits to the wind turbines.

Looking at reliability data of present wind turbines and assuming a state of the art required preventive maintenance visit every 6 months, the Opportunity-Maintenance strategy seems to be the most suited strategy for maintaining existing wind turbines offshore. Taking the failure rates of the wind turbines it can be seen that the average failure rate is in the range of two events per year. These failures will, of course, not occur in 6 monthly intervals, but a wind turbine is not expected to break down just because of the preventive maintenance task being overdue for a few months. In case of no occurrence of failures, a preventive maintenance operation can still be launched, if the required PM schedule of the wind turbine is overdue for too long.

PM & CM Maintenance Strategy

With this strategy scheduled PM tasks, as well as CM tasks, are executed. This strategy is currently used for onshore wind farms. It has to be kept in mind that, for onshore wind farms, labour costs and spare parts are the main cost drivers of the O&M costs. Costs of transport and access to the land based wind turbines contribute only a minor part to the overall O&M costs. O&M costs of an offshore wind farm are immensely affected by the efforts for transportation and access to the platforms. Therefore the number of required visits to the wind turbines has to be kept as limited as possible.

Of all the strategies, the PM & CM Maintenance strategy, as it is applied for onshore wind farms, requires the most visits to the wind turbines with scheduled PM visits and CM visits on demand. This might be an appropriate approach for onshore wind farms, where the costs of crew transportation are negligible compared to other cost factors. However, in the case of offshore wind farming, where crew transportation is one of the main cost drivers, everything has to be done to reduce the required maintenance visits to the wind turbines.

Periodic Check Maintenance Strategy

Here, the wind turbines are accessed at regular, scheduled, intervals. PM and CM tasks are executed on demand, after a thorough inspection of the wind turbines. Again, no permanent maintenance crews are required, as these visits are scheduled.

As the wind turbines will be equipped with some kind of SCADA system for remote operation, fault detection and fault diagnose, the periodic check maintenance is obsolete.

Moreover it does not fit in the concept of reducing the number of visits significantly.

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Choice of the Operation & Maintenance strategy

Having chosen the turbine concept, the maintenance approach, and the 'hardware', the Maintenance Decision Tree, see (fig. 3.4-1), is finally used to define the details of the operation and maintenance of the offshore wind farm. For example the working season, where maintenance tasks are executed, the crew hiring strategy, or the weather approach have to be defined.
However, the determination of the 'final' optimum approach is not possible with, for example, simple cost comparisons. Many stochastic events, such as the state of the weather, have to be taken into account, when examining the costs of the defined operation and maintenance strategy and the achieved availability. Therefore, the use of simulation programs, such as the described Monte-Carlo simulation, is suggested.

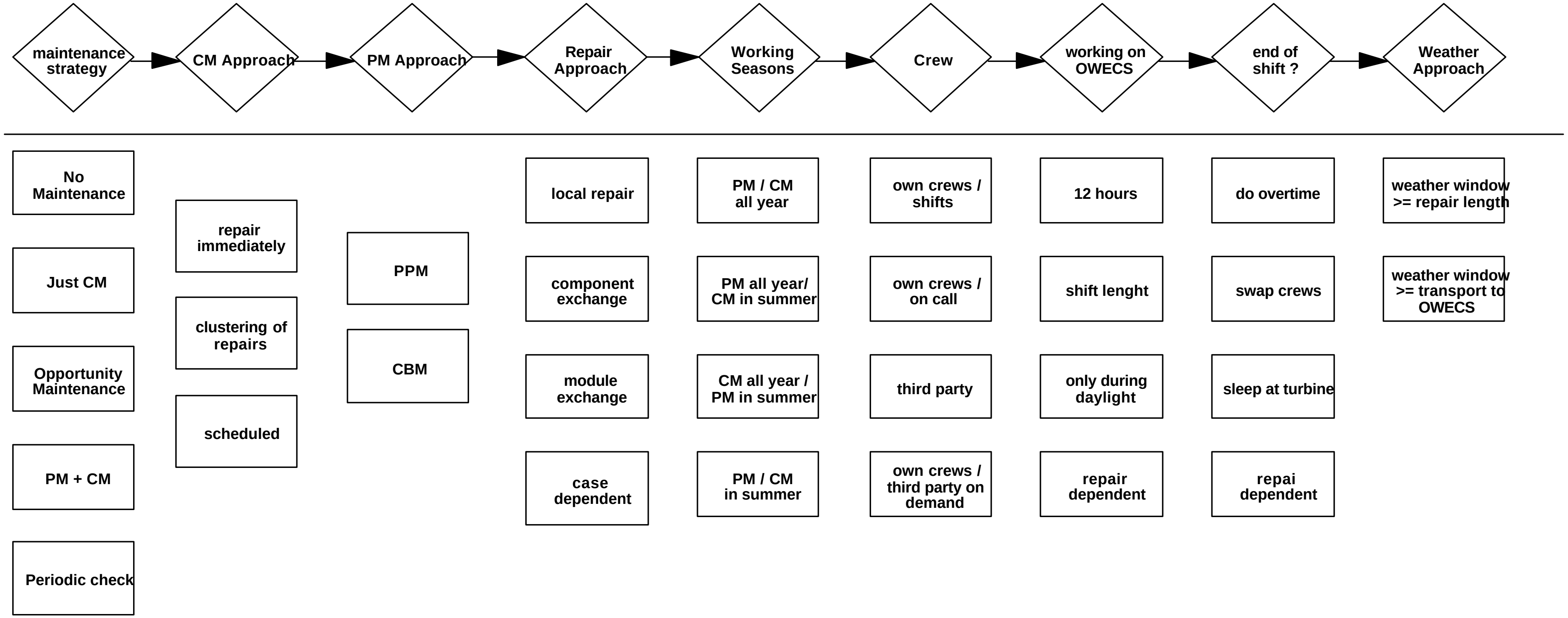

Figure 3.4-1: Maintenance strategy decision tree for an offshore wind farm

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3.5 Examination of some turbine design concepts

Choosing the operation & maintenance strategy starts already in the developing phase of the wind turbine. The design will determine the later behaviour of the wind turbine. The future energy yield and maintenance costs are defined in that phase.
An evaluation of three turbine concepts, highly theoretical, as none of these purpose designed wind turbines for offshore application exist yet. The three turbine concepts are, see also appendix B of part A of this volume:

- Disposable Turbine Design
  A lowest capital cost machine consistent with a predictable lifespan.
- Robust Offshore Turbine Design
  A more reliable turbine by improving the engineering.
- Advanced Lightweight Turbine Design
  A turbine with a high energy yield and flexible in order to reduce the loads.

Detailed specifications about their reliability and maintainability are, based on broad assumptions. Nevertheless, it is possible to evaluate the general advantages, respectively disadvantages of each concept. In the case of the disposable turbine concept it can even be demonstrated that this kind of concept is not suitable for offshore use, assuming that present offshore regulations are also valid for offshore wind farming.

Examination of a No-Maintenace strategy in connection with a Disposable Turbine design

For an investigation of the proposed disposal turbine concept together with a No-Maintenance strategy the defined offshore wind farm with 100 wind turbines is chosen as a test case.
A maximum turbine lifetime of 5 years is assumed for the Disposable Turbine concept. 5 years is the maximum time interval after that thorough PM is required for fixed offshore structures. As in this case no maintenance whatsoever is carried out, it is assumed that the turbines will be shut down and dismantled after 5 years to comply with the regulations. The very optimistic assumptions is made that no turbine will fail within the 5 year cycle.

The levelised production costs LPC are calculated for the No-Maintenance concept in connection with the Disposable Turbine Design. Two different economic wind farm lifetimes are considered, 5 years and 20 years respectively. The interest rate is assumed at 5 %.
In the latter case, with an economic wind farm lifetime of 20 years the wind turbines will be dismantled and exchanged after every 5 years using a flat-bottom crane vessel. The typical duration for installing a turbine with that kind of vessel is 4.4 days. As each turbine has to be dismantled and a new turbine installed the complete operation length for the exchange of one turbine is estimated to be 6 days. Thus 600 vessel rent days are necessary for the exchange of the complete wind farm. The day rate is assumed to be 37.500 ECU and for mob-and demobilisation an additional 500.000 ECU is added.
Thus, for one complete exchange operation of all 100 turbines the vessel costs amount to 23 million ECU. A new set of 100 turbines is estimated at 71 million ECU.

For a first approximation with respect to the final dismantling operation of the wind farm at the end of its lifetime, only the vessel costs for the removal of the turbines are taken into consideration. Thus the dismantling costs are assumed to be in the range of one exchange operation, namely 23 million ECU.

With a lifetime of the wind farm of 20 years all turbines have to be exchanged after 5, 10, and 15 years respectively, and finally dismantled. For these four operations the total vessel costs amount to 92 million ECU and three new turbine sets a` 100 turbines to 213 million ECU. All other costs, e.g. operation costs, are not included in this calculation. Table 3.5-1 shows the levelised production costs for this test case.

total initial investment costs	M ECU	191,1
interest rate	%	5
economic lifetime	years	20
annual energy output	10^6 kWh	280
total levelised other costs	M ECU/a	14,9
LPC	**ECU/kWh**	**0,11**

Table 3.5-1: LPC for No-Maintenance Strategy

(Economic wind farm lifetime of 20 years)

In the second test case, with an economic lifetime of only 5 years the entire wind farm will be shut down after that period. No new turbines will be installed but the wind farm will be dismantled. It is assumed that the only costs that arise within these 5 year are the total initial investment costs and the dismantling costs. For the dismantling costs the same assumptions as in the previous case are taken. In this case the levelised production costs amount to 0,17 ECU, see (table 3.5-2).

total initial investment costs	M ECU	191,1
interest rate	%	5
economic lifetime	years	5
annual energy output	10^6 kWh	280
total levelised other costs	M ECU/a	4,2
LPC	**ECU/kWh**	**0,17**

Table 3.5-2: LPC for No-Maintenance Strategy

(Economic Wind Farm Lifetime of 5 years)

The tables 3.5-1 and 3.5-2 show that even with these very optimistic assumptions it is not possible to achieve a competitive energy price in the near future with that kind of maintenance strategy and turbine concept.

- *Examination of Robust Offshore Turbine Design versus Advanced Lightweight Turbine Design*

At present, an evaluation and especially a comparison of these two concepts is very theoretical as no explicit data concerning reliability and maintainability is available.

The robust offshore turbine design aims to reduce the required maintenance visits and thus costs, by decreasing the number of possible failure causes using less and/or over-specified components. The overall objective is to increase the reliability of the system 'wind turbine', and thus reduce the number of required visits to the wind turbine.

The advanced lightweight turbine design aims the other direction. Here, the reduction of the O&M costs per produced kWh is achieved by increasing the energy production of the wind turbine. By being able to operate in a wide range of wind conditions the energy yield is maximised.
Detailed examinations of those two concepts with respect to feasibility, reliability, maintainability, availability, and development costs are required, in order to determine the most promising concept.

Using the Monte-Carlo simulation program, the effect of the reliability of the wind turbine upon the overall performance of the wind farm can be demonstrated.
The operation and maintenance of the defined example wind farm is simulated over a period of two years. A PM & CM maintenance strategy is chosen, where preventive maintenance is carried out approximately every 3500 hours, and corrective maintenance tasks as soon as possible, e.g. if crew available, vessel available, etc. The crews are assumed to work in 12-hour shifts, one shift per day, 7 days per week, each crew consisting of 2 workers.
The liftboat option is chosen for carrying out the lift operations and the vessel option for access to the wind turbines. The liftboat is also used as a permanent maintenance base within the offshore farm.

In the next step, the failure rate of the reference case is increased by a third, as well as decreased by a third. The results of each simulation are shown in (table 3.5-3). As can be seen from the table the relation between energy output and change in the failure rate is (strongly) non-linear.

	Increased failure rate	Reference case	Decreased failure rate
Availability [%]:	82.1	96.0	98.2
Deviation [%]:	-13.8	0	+2.6
Energy output [kWh] :	$5.36\ 10^8$	$6.22\ 10^8$	$6.38\ 10^8$

Table 3.5-3: Effect of failure rate on energy output

4. Conclusions

Conclusions concerning the OWECS operation and maitenance are given as follows:-

- The weather is a decisive factor for O&M actions.
- An optimisation of the O&M strategy has to be carried out with respect to the levelised production costs rather than the pure O&M costs.
- The lifting equipment required for exchanging major components, such as blades, gearboxes, etc., together with the devices for crew transportation are identified as the main cost drivers of the O&M costs.
- An examination of the lifting operation showed that the use of a self propelled modified jack-up platform is very promising with respect to the costs per lifting operation, assuming an offshore wind farm where at least 20 lift operations per year are required.
- The use of vessels seems to be a more cost effective method of crew transportation to and from the wind turbines compared to a helicopter.
- Remote control and monitoring are mandatory to reduce the number of visits.
- An Opportunity-Maintenance-Strategy is proposed to reduce the maintenance visits.

5. Recommendations

The following recommendations are made concerning operation and maintenance:-

- For accurate O&M investigations more detailed reliability data of large 1-3 MW wind turbines is necessary. Especially the examinations of purpose designed wind turbine concepts for offshore application is required to determine the cost effectiveness of these concepts compared to existing 'onshore' wind turbine designs placed offshore.

- Design concepts with reduced maintenance requirements should be developed.

- A concept of a purpose adapted self propelled liftboat should be developed.

- Adaptation to site of weather simulation (summer and winter season)

- Application of an O&M simulation model is required for the detailed evaluation of the O&M strategy.

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Appendix A: Abbreviations

CM	Corrective Maintenance
LPC	Levelised Production Costs
MTBF	Mean Time Between Failure
MTTR	Mean Time To Repair
O&M	Operation and Maintenance
PM	Preventive Maintenance
RAMS	Reliability Availability Maintainability and Serviceability
SCADA	Supervisory Control and Data Acquisition (system)

Opti-OWECS Final Report Vol. 2:

Methods Assisting the Design of OWECS
Part C:
Reliability Methods

L.A. Harland, J.H. Vugts

Workgroup Offshore Technology, Delft University of Technology

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Delft University of Technology
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TABLE OF CONTENTS

1. General considerations on the design of offshore structures

The design of an offshore structure comprises a number of separate but related steps, among which is the actual structural design which has the objective of satisfying the functional requirements and safely withstanding the loading to which the structure will be subjected. This task includes many and diverse activities such as dimensioning, material selection, structural analysis, fatigue assessments, etc. [1-1]. As part of the overall design process it is one of the most time-consuming steps and the very basis for achieving a safe and economic design.

In the structural design of offshore structures the environmental conditions play a very important role because these subject the structure to large loading. For fixed offshore structures the environmental loading is normally dominated by wave action, while wind loading is only a small part of the total loading experienced. The design of conventional offshore structures is therefore mainly aimed at resisting extreme hydrodynamic forces.

Appropriate design values for the met-ocean variables wind, waves and current are difficult to determine. The empirical methods used to predict extreme environmental conditions are mostly very conservative. The problem of specifying met-ocean design conditions is one of estimating environmental variables corresponding to some return period, typically 50 or 100 years, on the basis of data from measured or hindcast time series extending over a relatively short period, say 5 to 25 years. To obtain design wave heights for the North Sea and some other areas, a widely used approach has involved fitting cumulative distributions to the significant wave heights of successive three hour sea states. It is common practice to neglect both the correlation between sea states and the uncertainty of the extreme wave within a sea state. The cumulative distributions thus determined are subsequently used to estimate e.g. a 50 year return period value for the wave height. Similar processes are used for each met-ocean variable separately, thus completely ignoring any correlation between met-ocean parameters mutually as well. The resulting separate 50 year wind, 50 year wave and 50 year current are then conservatively assumed to occur simultaneously and to act in the same direction. For an offshore structure, this will lead to a “design load” that is clearly much more severe than the “true” 50 year load [1-2].

A particular application of the conventional design approach more suited for OWEC has been proposed by Germanischer Lloyd and Garrad Hassan [1-3]. It considers the correlation of wind and waves within an extreme event and also the possibly different contribution of wind and wave loads to the OWEC response in comparison to more common offshore installations. The OWECS design guidelines of Germanische Lloyd (GL) [1-4] also follow this approach.

However, over the years significant advances have been made in Offshore Engineering Technology for the petroleum industry as a result of continued research and development efforts and the accumulated experience of actual field developments, notably those in very severe environments. In recent years reliability based methods have successfully been developed for the design and/or assessment of offshore structures of certain types in respect of environmental loading, notably loading due to wave action. An overview of these developments is presented in [1-5]. These methods essentially aim to determine the long term distribution over the lifetime of the structure of those extreme responses that are of prime interest for its structural reliability.

The response distribution then serves as the “load distribution” in the classical reliability analysis sense. Once this distribution is known and a similar distribution can be determined for the criterion against which the particular response under consideration should be assessed, i.e. the “resistance distribution”, then the “probability of failure” of this response meeting the specified criterion can be determined. This is illustrated graphically in figure 1-1.

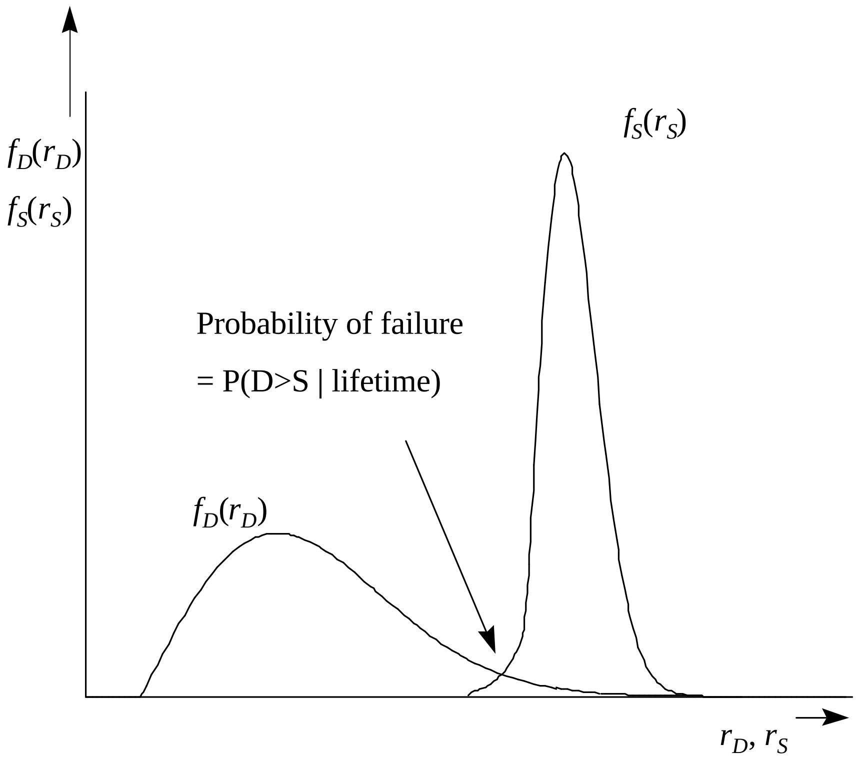

Figure 1-1 Probability of failure

$f_D(r_D)$ = probability density function of the Demand (“load distribution”)

$f_S(r_S)$ = probability density function of the Supply (“resistance distribution”)

r = response variable under consideration

Combining the results for all responses of interest the true probability of structural collapse can be calculated. This procedure has been worked out and has been successfully applied to the case of the “demand” being hydrodynamic (drag) loading on fixed steel structures of space frame configuration experiencing quasi-static structural response [1-6]; the “supply” is then naturally the strength of the structure in the form of the load the structure can withstand before collapse. In this way a considerable reduction (20%-30%) in design environmental force for a desired minimum reliability of the structure has been obtained. Note that this reduction is based on the return period of the loading, or more precisely the return period of the structural response, rather than on the return period of separate environmental conditions. More general applications are under active consideration and development.

The procedure can in principle also be applied in reverse order, i.e. when the long term distributions of the critical responses in the desired lifetime have been determined and a desired minimum reliability against "failure", i.e. the response not meeting the specified criterion, has been set, design environmental conditions can be specified that ensure that this minimum reliability level is in the desired lifetime achieved.

The fundamental difference of such a reliability based approach with the conventional approach is that the correlation between different met-ocean variables acting at the same time and the correlation between consecutive values of the environmental parameters are now taken into account. The method consists in essence of four related steps. Making the generalisation from hydrodynamic loading to structural response and from quasi-statically to dynamically behaving structures these steps may be described as follows.

The first step is to acquire a large dataset of the environment at an offshore structure's location. The database should contain information of all relevant environmental parameters over a period of, say, 25 years. The environment is next defined in terms of storm events rather than individual sea states. From the database all storms are identified while determining the joint values of all met-ocean variables.

The second step is the determination of the response of the structure in a storm. For each storm the relevant extreme responses are determined using a finite element program. In this manner time series of joint met-ocean variables are transformed into corresponding time series of extreme responses in storms, from which the long term statistics of an extreme response occurring during an arbitrary storm of arbitrary severity can be determined. As the statistical distribution thus derived will clearly only relate to environmental events that are present in the database an extrapolation to survival conditions is necessary; i.e. extrapolation to include storm severities far beyond that of storms included in the database. The extrapolation enables one to determine an extreme response level that occurs on average no more frequently than once in 50, 100, 1000 or even more years.

In the third step a model for the occurrence of storms at the intended location is derived from the data in the database. Combining this model of the local climate for stormy weather with the long term distribution from step 2 of the extreme response given that an arbitrary storm occurs, the long term distribution of the extreme response during the lifetime of the structure at the specific site can be determined. Note that both these elements are site dependent.

Finally, if a failure criterion for the ultimate level of response that can be accommodated is next defined and its associated uncertainty determined, the combination with the long term distribution of the extreme response in the desired lifetime allows a probability of "failure" of the structure during a given time period to be determined.

It should expressly be noted that when applied to structural design all procedures which are based on the extreme response due to environmental loading imply of necessity that the design of a structure is governed by considerations of ultimate strength. If other failure mechanisms predominate, e.g. fatigue, these procedures lose their validity. In the application of the reliability method in the Opti-OWECS project it will be assumed throughout that the structural design is based on strength.

Obviously, the procedure described above may hold promise with a view to optimisation of the support structure of an OWEC unit. Therefore it has been investigated whether the (suitably amended and extended) methodology that has been developed for structures for the petroleum industry can also be applied to the support structure of an OWEC. The desired minimum reliability level for an OWEC can be set lower than for most offshore structures for the petroleum industry. An OWEC is unmanned, does not represent a single source of supply, is not a one-off structure, has small environmental impact in case of failure and involves far less investment. The desired reliability level for an OWEC is thus essentially governed by more direct economic considerations. However, the existing procedures need to be extended to include aspects which are of minor influence only for offshore structures for oil and gas developments. The most important of these aspects are:

- significant windloading on the rotor;
- inertia as well as drag wave loading on the support structure;
- dynamic response of the support structure.

These extensions have a significant effect on the manner in which the long term distribution of the extreme response during a given lifetime can be determined, as will be discussed in section 2.2. In section 2.1 the methodology as adapted to the current application will first be discussed in more detail. In section 3 a possible design for a support structure of an OWEC at a location in the North Sea will be discussed to which the reliability based design method will be applied. Furthermore, section 3 will discuss the contents of the met-ocean database and the environmental loading on the structure. Section 4 discusses the necessary preparations before applying the reliability based design method. Finally, in section 5, the results of the procedure as applied are presented together with the results as obtained from conventional design conditions.

2. Support structure design basis using structural reliability considerations

2.1 The steps in a reliability based design method

2.1.1 Definition of the environment in terms of storms

The hindcast database

The first step of a reliability analysis of an OWEC support structure is the analysis of the data in the database. The database used for the present application relates to a gridpoint on the Dutch continental shelf in the southern part of the North Sea and has been abstracted from the North European Storm Study (NESS) database [2-1]. This database contains information on all relevant met-ocean parameters over a period of some 25 years sampled at three hourly intervals; this interval is the traditional period during which sea states are assumed to be stationary. The term “sea state” will thus henceforth be used to refer to environmental conditions which are assumed to be stationary, i.e. constant in a statistical sense, over a period of 3 hours.

The oceanographic parameters of interest for the OWECS project include wave and current parameters: the significant wave height H_s; the peak spectral period T_p; the mean wave direction q_m; a measure of the directional spreading of the waves around the mean; the current velocity U_c; the current direction q_c. The current speed given in the NESS database is the depth averaged value. The meteorological parameters which will be used in the analysis consist of the mean wind velocity V_w and its direction q_w.

The usual practice in offshore engineering of relating all met-ocean parameters to H_s as the leading parameter will be followed, although the calculations during the course of the project will have to demonstrate that this assumption is also correct for the present application.

Directional sectors

In determining the distribution of the extreme response for the reliability analysis the effect of directional sectors should be taken into account. Clearly, due to persistent or recurrent weather patterns, tides, fetch limitations, etc. the met-ocean variables will not be homogeneously distributed over the directions. Therefore the database will be divided into a number of directional sectors (e.g. with a width of 30º-45º) to obtain better homogenised sub-databases. Furthermore, any non-rotationally symmetric structural configuration will have different properties in different directions. The response distribution of the structure will hence be sensitive to the direction of the met-ocean conditions, while the ultimate strength distribution will in general also be different for different directions. Therefore, all subsequent analyses are if possible performed per directional sector on the sub-databases.

In section 3.1 the content of the database of a possible location for an offshore wind farm is discussed further.

Storm

A storm event is defined as a period of build-up, a peak and a subsequent decrease of the environmental conditions, notably in regard of the governing significant wave height parameter H_s. A storm event is thus defined as a succession of sea states during which H_s is greater than a particular threshold. The determination of the threshold will be further discussed in section 3.1. The severity of a storm is identified by the maximum value of the significant wave height during the storm and is named $H_{smax}(s)$. The H_s in a storm is shown schematically in figure 2-1 together with the history of some other met-ocean variables during the storm. The sequence of the sea states in storm s is indicated by the counter i, where i = 0 refers to the peak of the storm and $-I \leq i \leq +J$. From the 25 years of data all storms, with a total number of S, are selected.

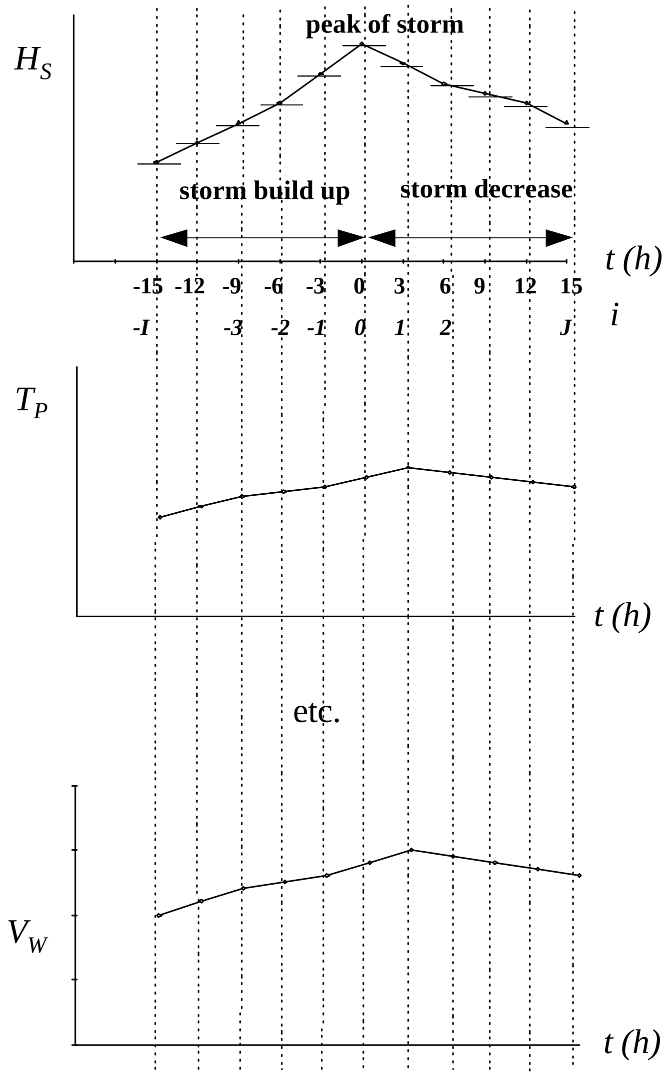

Figure 2-1. Example history of a few met-ocean parameters during a storm.

2.1.2 Long term distribution of the extreme response in an arbitrary storm of arbitrary severity

Basic assumption

When the storm events for the location of interest have been determined the response of the structure can be evaluated, provided that the behaviour of the structure can be modelled with reasonable confidence. Eventually the interest is in its extreme response with only a (very) small probability of being exceeded during the expected lifetime of the structure; this implies extrapolation. Extrapolation is by necessity always based on assumptions and the challenge is to decrease the influences of these assumptions as much as possible.

The basic assumption in the adopted approach is that the distribution of the extreme response in a given storm is of a standard shape. Naturally, for more severe storms the extreme response will be larger than for less severe storms. However, according to the assumption made it should be possible to scale these distributions back to the same standard shape, the so-called **generic distribution of normalised extreme response**. Extrapolation is then solely based on storm severity.

Distribution of the extreme response given a sea state

As discussed in the previous section, a storm consists of a succession of sea states. The responses given a sea state are therefore to be determined first. The responses given a storm are then obtained by combining the results for the sea states.

The method that will be chosen to predict the responses of the OWEC subject to the time varying wave conditions during a sea state will be discussed in section 2.2. Using this method the probability distribution, F_e, of the extreme response, r_e, given a sea state (identified by H_s, T_p, U_c, V_w, etc.) can be determined (equation 2-1):

$$F_e\Big(r_e|H_s(i),T_p(i),U_c(i),V_w(i),......\Big) \qquad (2\text{-}1)$$

The responses of interest may e.g. be the total lateral force and overturning moment transmitted to the foundation, the stress at any point of the support structure or the displacement of the top of the structure. The response will be indicated here by the general notation 'r'. Note that the directions of waves, current and wind with respect to the structure, which play an important role in the response calculations, are duly taken into account as the parameters of sea state i include this directional information.

Distribution of the extreme response given a storm

The determination of the distribution function (equation 2-1) is repeated for all sea states in the storm for $-I \leq i \leq +J$. The distribution of the extreme response for an entire storm is then easily obtained, as shown by equation 2-2:

$$F_e(r_e|storm\ \mathrm{s}) = \prod_{\mathrm{-I}}^{\mathrm{J}} F_e\Big(r_e|H_s(i),T_p(i),U_c(i),V_w(i),......\Big) \qquad (2\text{-}2)$$

As the sequence of sea states in a storm is empirical and differs from storm to storm the distribution function 2-2 cannot be determined theoretically. Therefore the distribution $F_e(r_e|storm\ s)$ will always have to be determined in a purely numerical way even though the distribution of the extreme response in a particular sea state may in special cases be derived analytically. However, for an OWEC which is subject to both non-linear wave and non-linear wind loading such a theoretical derivation is not possible (see further section 2.2).

The distribution function 2-2 is derived for all storms selected from the database.

Generic distribution

The next step is to normalise the distribution of the extreme response given that a storm occurs by a suitable reference value, for which one of the three measures of central tendency is chosen. These are the mean value: $r_{e,mean}$, the median value: $r_{e,med}$ (which is easiest to determine) and the most probable maximum value: $r_{e,mpm}$ (which is the most frequently used in offshore engineering practice). It remains to be evaluated which measure of central tendency can best be used to normalise $F_e(r_e|storm\ s)$. In the first instance the median value $r_{e,med}(s)$ will be adopted as the normalising factor as it can easily be determined from $F_e(r_e|storm\ s) = 0.5$.

After normalisation of $\underline{r_e}$ to $r_e/r_{e,med}$ for all storms per directional sector in the (sub)database we have S distributions of normalised extreme response. These are subsequently averaged to determine the *generic* distribution of normalised extreme response for an arbitrary storm for this direction (equation 2-3).

$$F_{e,gen}\left(\underline{r_e}\middle|arbitrary\ storm\ with\ severity\ r_{e,med}\right) = \frac{1}{S}\sum_{s=1}^{S} F_e\left(\frac{r_e}{r_{e,med}(s)}\middle|storm\ s\right) \quad \text{(2-3)}$$

How to use a generic distribution?

The advantage of normalising the extreme response in a storm is that the information regarding the response distribution and storm severity are uncoupled, where it should be noted that $r_{e,med}(s)$ is used as a measure of storm severity. Irrespective of the severity of the storm the distribution of the normalised extreme response has a standard shape. Rather than the whole distribution only its measure of central tendency for a storm of a particular severity for the particular directional sector has to be estimated.

The median extreme response, $r_{e,med}(s)$, values in the S storms are collected and used to determine an equally empirical distribution function of response severity $F_e(r_{e,med})$, which reflects storm severity in this direction for the location studied. However, the probability distribution of $r_{e,med}$ does not directly provide information about the response in the rarer, more severe storms with return periods greater than the length of the (sub)database. Therefore extrapolation is necessary to predict the upper tail of the distribution. Experience with application of the method to offshore structures for the petroleum industry suggests that either the Weibull or the Generalised Pareto distribution are suitable for this extrapolation, but other extreme value distributions can also be used.

Subsequently, the probability density function of the median extreme response $f_{e,fitted}(r_{e,med})$ can be determined by differentiation of the above empirically fitted distribution function. The probability of occurrence of a particular storm causing a response severity $r_{e,med}$ is now by definition:

$$p_o(r_{e,med}) = f_{e,fitted}(r_{e,med}) \cdot dr_{e,med} \qquad \text{(2-4)}$$

The product of the generic extreme response distribution (2-3), in absolute rather than normalised form, and the probability density function (2-4) is the contribution to the overall probability distribution of the extreme response, given that a storm causing that particular level of median response occurs. By integrating this product over all possible levels of median response, i.e. effectively integrating over all possible storm severities occurring from a particular direction at the particular location of interest, the total probability distribution for a storm occurring at random for the direction considered is determined:

$$F_{e,gen}\left(r_e \middle| storm\ occurring\ at\ random\right) =$$
$$\int_{(r_{e,med})\min}^{(r_{e,med})\max} F_{e,gen}\left(r_e \middle| arbitrary\ storm\ with\ severity\ r_{e,med}\right) \cdot f_{e,fitted}\left(r_{e,med}\right) \cdot dr_{e,med} \qquad \text{(2-5)}$$

In principle the response or storm severity ranges from zero to infinity, but as the number of storms in the (sub)database is limited it extends in practice over the range between a lower limit $(r_{e,med})_{\min}$ and an upper limit $(r_{e,med})_{\max}$.

2.1.3 Long term distribution of the extreme response during the lifetime of an OWEC

The result obtained in equation 2-5 is conditional on a storm passing within a specified directional sector. However, the OWEC does not experience a storm constantly during its lifetime. Therefore the probability **that** a storm will pass an OWEC from this direction needs to be taken into account. Storm arrivals may be treated as a Poisson process [1-6]. The mean arrival rate of the storms can be estimated from the (sub)database as $n=(S+1)/t$, where t is the total duration of the met-ocean database. The probability distribution of the extreme response in an interval T (with T the desired lifetime) is then, finally:

$$F_e\left(r_e \middle| nT\right) = \left(F_e\left(r_e \middle| storm\ occurring\ at\ random\right)\right)^{nT} \qquad \text{(2-6)}$$

The distribution 2-6 is effectively the "load distribution" of figure 1-1 for the specified direction in the classical sense of a reliability analysis. The whole procedure as outlined in the sections 2.1.1 - 2.1.3 is shown schematically in figure 2-2.

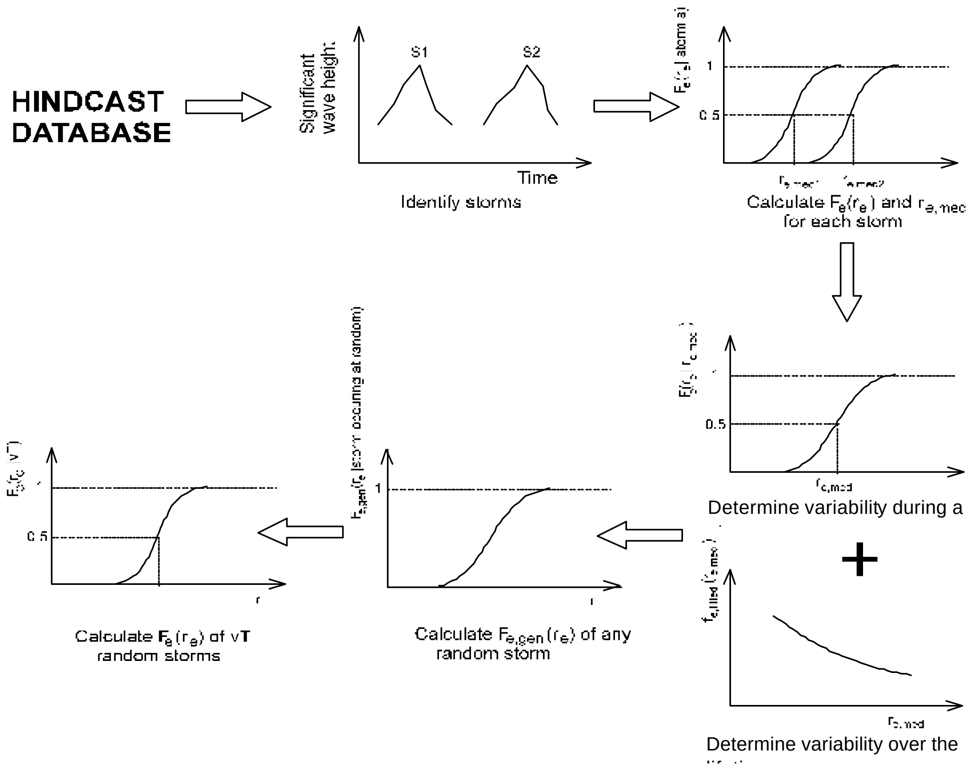

Figure 2-2 The determination of the extreme response distribution with a desired return period [1-6].

2.1.4 Probability of failure of an OWEC during its lifetime

The long term distribution of the extreme response as determined in section 2.1.3 is the “load” or more generally in terms of any response parameter the demand distribution of figure 1-1. Combining this function with the “resistance” or supply distribution for the response of the structure of interest, yet to be determined, the probability of failure can be calculated by:

$$prob_F = \int_{-\infty}^{+\infty} H_D(r_e) \cdot f_S(r_e) \cdot dr_e \qquad (2\text{-}7)$$

where $H_D(r_e)$ is the complementary distribution function of the demand (i.e. the probability of the demand on r_e exceeding a particular value, $H_D(r_e)=1-F_e(r_e|nT)$ for a chosen lifetime T) and $f_S(r_e)$ is the probability density function of the supply in r_e. “Failure” is to be understood as the response being considered not meeting the specified criterion. The concept can be applied to structural strength but also to other phenomena of interest to an OWEC.

The integration extends in principle over all values of r_e, i.e. from minus to plus infinity. However, experience for offshore structures consistently indicates that the demand distribution is invariably (very) wide, while the supply distribution is (very) narrow. This is also clearly illustrated in figure 1-1. Therefore a limited range of integration from $r_{e,min}$ to $r_{e,max}$ which covers the relevant part of the tail of the demand distribution and includes the entire supply distribution suffices in practice. This is shown in figure 2-3. Consequently equation 2-7 may be replaced by equation 2-8:

$$prob_F \approx \int_{r_{e,\min}}^{r_{e,\max}} H_D(r_e) \cdot f_S(r_e) \cdot dr_e \tag{2-8}$$

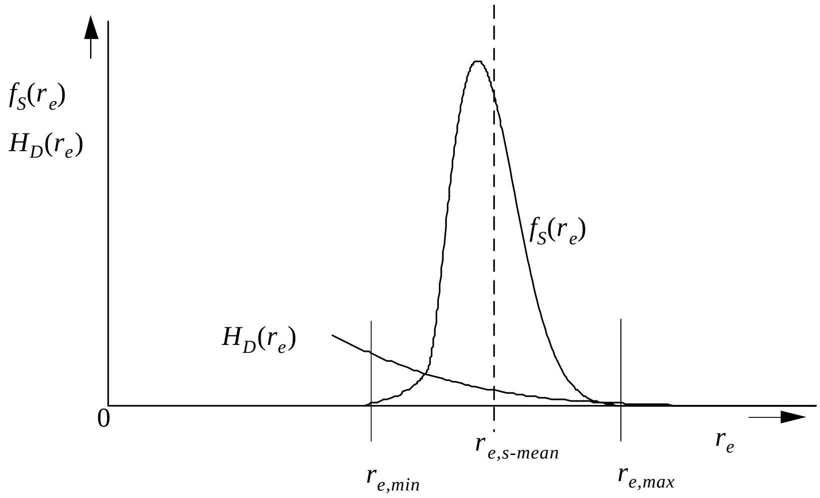

Figure 2-3 Probability of failure during lifetime.

In what follows we will generally focus on an application of the method to global environmental loading on the support structure and its structural strength. In view of the narrowness of the probability density function for the structure's strength or resistance, the exact shape is of no great importance for the resulting probability of failure. Hence a distribution shape may be assumed, for which usually a normal distribution is taken. A reference value of the structure's strength can be determined with the aid of advanced non-linear structural analysis software packages which can predict the collapse of a structure. This reference strength may be interpreted as the mean or a similar but different characteristic value of the normal probability density function.

As probabilities of failure of practical interest are always very small it is their order of magnitude rather than their exact value which is meaningful. An inconsequential further approximation may therefore be made to equation 2-8:

$$prob_F \approx \int_{r_{e,\min}}^{r_{e,\max}} H_D(r_e) \cdot f_S(r_e) \cdot dr_e \approx H_D(r_{e,s-mean}) \tag{2-9}$$

The implication is that an adequate approximation of the "probability of failure" can simply be read from the long term distribution of any extreme response in a desired lifetime (e.g. 1 yr, 50 yrs or 1000 yrs) at the mean "supply" value of that response, $r_{e,s\text{-}mean}$, where "failure" is defined in very general terms as the supply of the response under consideration not satisfying the demand on that response (see figure 2-4). In the case of demand being global loading on the structure and supply being structural strength, "failure" relates to collapse of the structure. This is the case for which the procedure has been worked out for structures for the petroleum industry (cf. section 1).

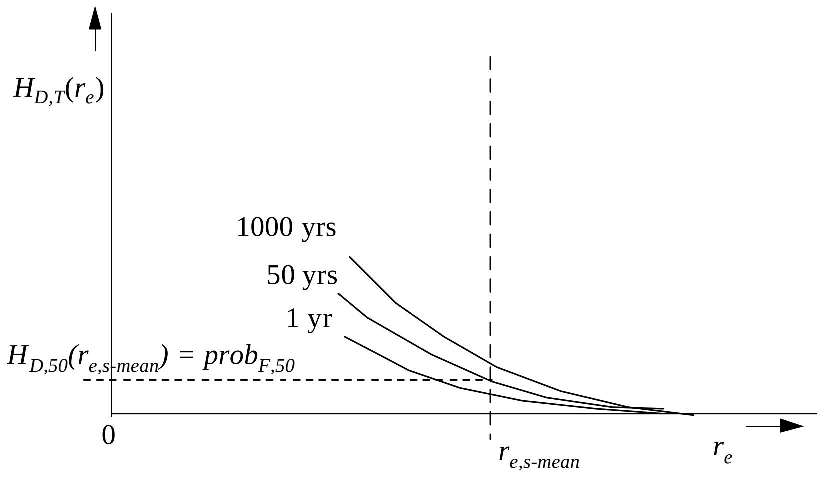

Figure 2-4 Approximation to the probability of failure during a chosen lifetime.

Considering a structure with a constant strength in time, one would expect the probability of collapse to increase with an increase of the desired lifetime. This is also illustrated in figure 2-4 where the probability of failure during a desired lifetime of 50 yrs, $prob_{F,50}$, is larger than the probability of failure during a desired lifetime of 1 yr.

As we are generally interested in small probabilities of failure the representation as used in figure 2-4 is not very practical. Therefore the same information is plotted differently. Existing experience with long term distributions $H_D(r_e)$ for several phenomena in offshore engineering indicates that for large values of r_e the upper tail of the distribution decreases approximately in an exponential manner. Plotting the upper tail of $H_D(r_e)$ on a semi logarithmic scale versus r_e thus produces a downward sloping approximately straight line. Switching the axes with $H_D(r_e)$ (and hence $prob_F$ according to equation 2-9) plotted horizontally and r_e vertically the picture is transformed into an upward sloping straight line (figure 2-5).

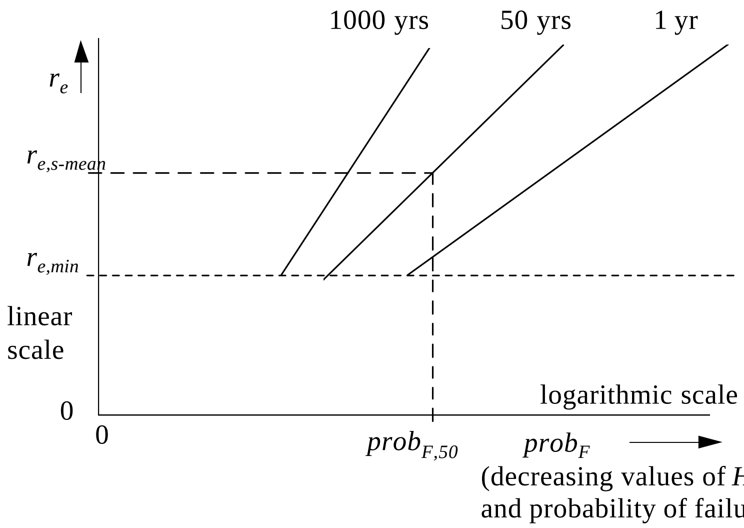

Figure 2-5 Approximation to the probability of failure during a chosen lifetime plotted on semi-logarithmic scales

Now the “probability of failure” in association with a chosen desired lifetime can be determined much more easily.

Having determined figure 2-5 a reverse procedure is now also possible: based on a selected probability of failure in a desired lifetime the global loading to collapse can be determined. Provided that a suitable generic load model is available it is now relatively simple to determine a set of environmental conditions which will generate that level of global loading on the structure. A possible load model will be discussed in section 2.2.2. This set of environmental conditions then simply serves to provide a suitable load condition for detailed structural design; it is not unique and no particular meaning may be attached to its selection. In principle a similar procedure is likely to be possible for any other response than environmental loading by replacing a generic load model by a generic response model for the particular response being considered. However, such possibilities have so far not been investigated.

2.2 How to estimate extreme responses in a given sea state?

The procedure outlined in section 2.1 is based on the assumption that it is possible to determine the distribution of the extreme response within one sea state of 3 hours length with high accuracy. Although the present state of knowledge of describing waves, wind, current, the interactions between environment and structure, the behaviour of the structure under loading, etc. is limited this is indeed possible.

There are several ways to determine the extreme response within a sea state:

- Spectral analysis techniques
- Generic Load Model
- Time domain simulation techniques

Depending on the type of problem each technique has its pros and its cons. All are based on transformation of the environmental parameters into the required structural parameters. This transformation is based on the characteristics of the system. These characteristics can be diverse and will depend strongly on the nature of the system involved. The form of these transformations may or may not be known.

2.2.1 Spectral analysis techniques

A relatively simple and straightforward class of problems is that of a linear system for which the influences of wind, current and waves may be separated and the dominant influence is that due to waves. An example of this class, with good approximation, is the heave motion of a semi-submersible in storm events. The heave motion in each sea state is a simple input - output problem with the waves as input and the heave motion as output. The input is fully characterised by the wave spectrum for the sea state $S_i(w,q)$ while the transformation into heave is provided by a linear operator, the frequency response or transfer function. The transformation is carried out by spectral analysis which converts $s_{waves}(i)$ into $s_{heave}(i)$. Using $s_{heave}(i)$ and taking r to represent peak have motions the procedure as described in 2.1 remains entirely valid.

In this particular example wind and current do not affect heave motions at all and the response to wind and current is therefore zero. In general, however, responses due to wind and/or current should also be determined in some manner. Finally the total response can be obtained by combining the responses to the three environmental actions for each sea state. Consequently, the response during the storm is determined as a function of time using the empirical storm history as illustrated in figure 2-1, after which equation 2-2 can be applied.

2.2.2 Generic Load Model

For non-linear problems a transformation is less simple to find. An example of such a case is the non-linear drag forces due to waves and currents. This particular type of problem can be solved by developing a Generic Load Model (GLM) which furnishes an expression to transform environmental parameters into global loading on an offshore structure of a given type [1-6]. Generic load models for quasi-statically responding, drag dominated offshore structures have been developed. These models provide expressions for base shear and overturning moment in terms of linear crest elevation (x), zero crossing period (T), directional spreading factor (F), depth integrated current (u), one minute sustained wind speed (W), angle between mean wave and current directions (u_c), and angle between mean wind and wind directions (u_w.) For the duration of one sea state, all met-ocean variables are treated as constants with the exception of crest elevation. The variability of the extreme response is thus based on the variability of the extreme crest elevation within a sea state. An example of a GLM for base shear [1-6] is given in equation 2-10:

$$F = A_1 u^2 + A_2 u x T \Phi \cos u_c + A_3 \Phi^2 x^2 + \frac{A_4 u \Phi x^2 \cos u_c}{T} + \frac{A_5 \Phi^2 x^3}{T^2} + A_6 \Phi^2 x^2 T^2 + A_7 W^2 \cos u_w \quad \text{(2-10)}$$

where A_1 to A_7 depend on the configuration of the structure and the attack direction. The dominant term in the expression is $A_3\ F^2 x^2$, the component of drag arising from wave kinematics below mean waterline. A similar expression can be obtained for overturning moment, *M*, with A_1 to A_7 replaced by A_{11} to A_{17}. The constants A_1 to A_7 and A_{11} to A_{17} can be obtained by fitting expression (2-10) to the results of time domain (or other) simulations.

Generic models of this general form can be used to represent the extreme global responses of many space frame structures [1-6]. For structures where the extreme loading is exclusively governed by the inertia part of Morison's equation, with or without diffraction effects, the problem is similar to the heave motion in section 2.2.1.

Adaptation for the OWEC-structure

A generic Load Model as discussed in this section can possibly be adapted for the OWEC-structure. Necessary extensions or adjustments should concentrate on the inclusion of the significant windloading on the rotor and inertia wave loading on as well as dynamic response of the support structure. It may very well be that through fitting of an expression such as 2-10 to the results of time domain simulations, a Generic Load Model for the OWEC-structure which will comply with all situations may be found.

If this would indeed be possible then the process described in section 2.1 can be performed very quickly; it may even be possible to automate the process. However, before a Generic Load Model approach can be used it has to be established that it is possible to construct a GLM for dynamically responding structures.

2.2.3 Time domain simulation techniques

More in general the extreme response of a system to an environment, i.e. the response to the simultaneous actions of wind, waves and current, may be obtained by computer simulation. Provided that the behaviour of the model can be modelled with reasonable confidence, the response of the system for each interval i in a storm (see figure 2-1) can be determined.

Random time domain simulation

The most straightforward and potentially most accurate manner to determine the distribution of the extreme response in a sea state is to simulate the response behaviour for many different realisations of the same sea state. From each simulation only one extreme response can be obtained, so using multiple simulations an empirical distribution can gradually be built-up. This approach is very time consuming and hence expensive. This method is therefore impractical to analyse the extreme response given all sea states in the storms selected from the database.

Constrained random time domain simulation

Recent work [2-2] has demonstrated that constrained random time domain simulations can be used effectively to determine the distribution of the extreme structural response in a robust, faster and cheaper way than with the “brute force” full random simulations referred to in the preceding paragraph. The technique of constraining random time domain simulations ensures that in each random wave simulation that drives the structure’s response, a wave crest of height A_{crest} is present at a prescribed location in time and place. The thus constrained but otherwise random simulations are then used to determine statistical relations between crest elevation and the associated peak (dynamic or quasi-static) response. These statistical relations are subsequently convoluted with the distribution of A_{crest} to finally determine the distribution of the extreme response in the given sea state.

In figure 2-6 time series of surface elevations are shown: one for a purely random simulation and one for a constrained simulation with a crest height of 5 m at 60 s. It has been demonstrated that the extreme response distribution can be determined with a considerable reduction (50 to 100 times) in simulation time compared to the approach based on full random simulations.

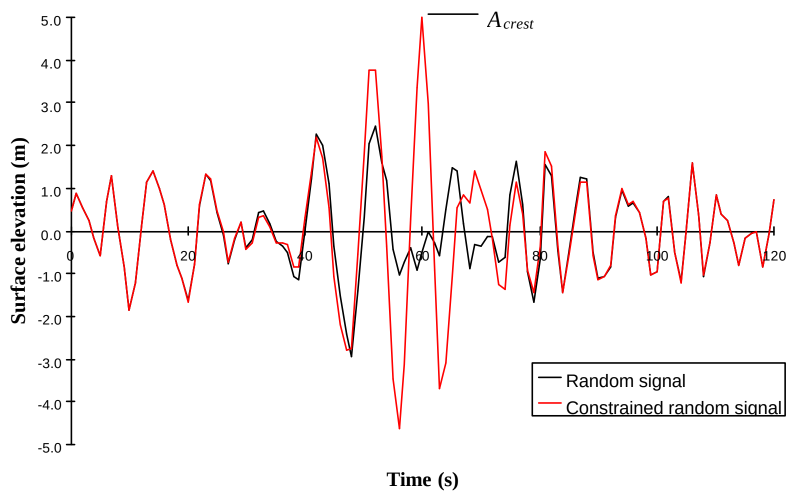

Figure 2-6 Time series of a random wave signal constrained at a crest height of 5m at 60 s.

2.2.4 Approach adopted in this work

Using constrained simulations the reliability based method outlined in section 2.1 becomes within reach of practical application. This method has therefore been adopted for an evaluation of a preliminary design of an OWEC at the North Sea location. The approach summarised below will be followed:

- Determine the distribution of the extreme response for a number of typical sea states using full random simulations.
- Estimate the same distributions by means of constrained simulations. Determine the optimal number of constrained simulations which produce the distribution of the extreme response with the same accuracy as full random simulations. This number will then be used throughout the analysis of the entire database.
- Apply the reliability design based method to the OWEC at the North Sea location. This still involves a very considerable amount of simulation work (about 2 months of computer time for the case considered).
- Evaluate whether the use of a (suitably modified) Generic Load Model can produce the same results. The use of a GLM will then reduce the required analysis time considerably. Please note that this part of the work is not essential as the procedure has already been executed in the previous step.

3. Description of OWEC support structure and environment

3.1 The selected North Sea location

Possible locations for an offshore wind farm

A short study has been performed on the selection of a suitable location for an offshore wind farm in the Dutch North Sea. The wind farm is of medium size and has a power rating of approximately 200 to 300 MW.

In the search of a location for an offshore wind farm near the Dutch coast the following areas were excluded:

- Environmentally protected areas
- Marine traffic lanes
- Military zones
- Oil and gas pipelines
- Oil and gas platforms

These restrictions are shown on the map of a part of the North Sea near the Dutch coast which is given in figure 3-1. From the figure possible locations can easily be deduced.

Several possible locations are marked on figure 3-1:

I 17½ km WNW of IJmuiden
II 25½ km SW of IJmuiden
III 47 km WNW of IJmuiden
IV 40 km NW of The Hague
V 75 km NW of IJmuiden
VI 55 km WNW of The Hague
VII 90 km NW of The Hague

The selection of the 'most attractive' location is based on several considerations:

- Available area to locate a 200-300 MW wind farm
- Wind speed regime
- Distance from the wind farm to a 150 kV connection point of the onshore electric net
- Water depth
- Distance to a harbour as an operations and maintenance base

For phase 2 of the Opti-OWECS project it was decided to choose location V for the application of the reliability based design method. This location can accommodate a large offshore wind farm, has the best wind regime, is suitably located for a connection to the onshore net and is close to harbour facilities. Furthermore, the water depth of approximately 25 m was appreciably different from a location in the Baltic Sea which was also considered in phase 2 of the Opti-OWECS study. For the development of a design solution in phase 3 of the Opti-OWECS project location I has been chosen. As it had in the meantime also become clear that for the particular circumstances in the current project fatigue (mainly wind induced fatigue) was the governing factor for structural design of the support structure the reliability based design method was not pursued further for phase 3. This report on methodology therefore continues to discuss the phase 2 selection of location V.

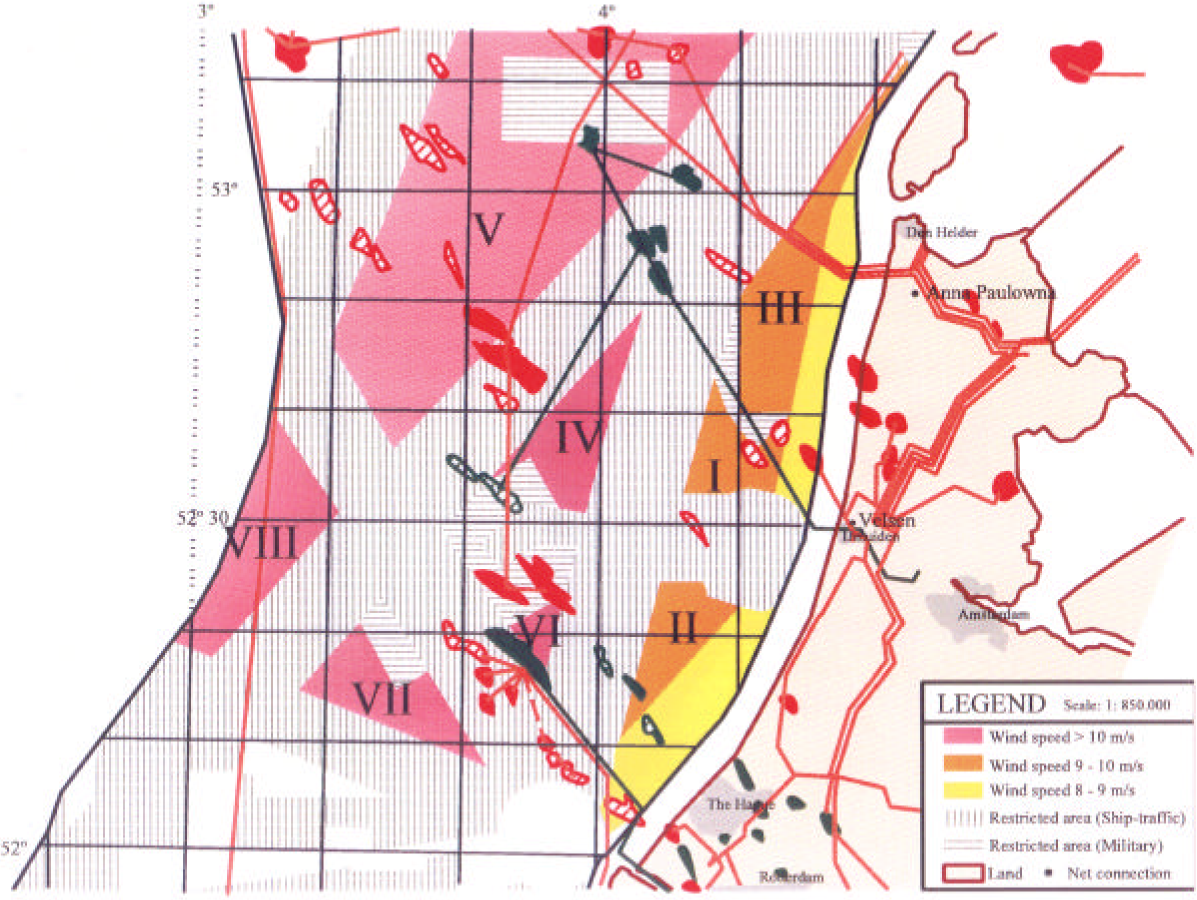

Figure 3-1 Possible locations for offshore wind farms along the west coast of The Netherlands.

Available database for the location

To characterise the environmental conditions in the selected area a specific point was chosen which was considered representative for location V as a whole. This is “gridpoint 568” with co-ordinates *53º 01’ 56” N* and *4º 04’ 21” E* for which a large amount of data was abstracted from the NESS database.

The North European Storm Study (NESS) [2-1] was initiated in order to produce a high quality hindcast database of winds, waves, storm surges and depth-integrated currents for the North European continental shelf. The hindcast provides in essence 25 years worth of met-ocean data as derived from a numerical simulation of atmospheric pressure and wind patterns that has been coupled with models describing how the energy of the wind is converted into waves and currents. The project was funded by eleven participants (nine oil companies and two government departments) and the work was carried out by meteorological and oceanographic institutes in five European countries. Quoting from reference [2-1]:

> *The initial drive behind the NESS project was the belief that by using a model, it would be possible to extract far better value from the measurements which were available if they could be used to produce, through numerical models, a more homogeneous and longer-term data set over the complete North Sea area. The project took more than 5 years to complete at a cost of over £ 2.1 million and has resulted in a major asset to the industry from which both significant cost-savings and improvements to the safety of operations can be expected for many years.*

For “gridpoint 568” the following information is available:

- simulated wind and wave conditions at 3-hourly intervals for 25 winter periods from 1964 to 1989, each period being of 6 months duration (October to March);
- wind and wave conditions, also at 3-hourly intervals, during three summer periods (April to September) from 1977 to 1979; this supplementary information makes it possible to derive operational statistics from three years of continuous data;
- wind and wave conditions during 50 summer storms (i.e. storms which occurred during the 6 months from April to September) in the periods 1964-1976 and 1980-1989;
- surge height and depth integrated surge and tidal currents for 250 storms which occurred during the 25 winter periods, each storm period having a duration of approximately four days.

Not all the data contained in the database is of interest for determining the extreme response of the structure. The environmental parameters used in the present work are:

- H_s significant wave height (m)
- T_p peak spectral period (s)
- q_m mean wave direction (degree)
- U_c depth integrated current speed (m/s)
- q_c current direction (degree)
- V_w one hour mean wind speed at 10 m height (m/s)
- q_w mean wind direction (degree)

As different parameters and tools often use different definitions, careful attention should be given to the meaning and application of each parameter; e.g. the directions of waves and wind in the database are given clockwise from North and “coming from” while the available software packages use anti-clockwise and “pointing toward” directions for waves, wind and current.

Definition of a “storm”

As referred to in section 2.1.1 a threshold value is needed to identify a succession of sea states which is subsequently defined as a storm. The minimum number of the successive sea states is taken to be 3 in order to incorporate the build up and decrease of a storm (as illustrated in figure 2-1). The selection should be made such that there is enough information within each storm to perform a proper statistical analysis while the total number of storms and individual sea states remains small enough to be practically manageable. In applications for conventional offshore structures it has been suggested to use a threshold value equal to 30-40% of the maximum H_s in the database. For “gridpoint 568” the maximum H_s is 7 m giving a threshold value of 2.5 m. In table 3-1 the number of storms and individual sea states are summarised as a function of the threshold value. It should be kept in mind that by simulation the distribution of the extreme response must be determined for each individual sea state. Although the use of constrained simulations gives accurate estimates for the distributions relatively quickly (say 1 hour per sea state), the total simulation effort is still very considerable. Based on a threshold value of 2.5 m this would imply 9319 hours of computer simulation. This roughly corresponds to 12 months of non-stop simulation. It has therefore been decided to decrease the number of sea states by increasing the storm threshold value. By selecting a threshold value of 3.5 m it is believed that the number of storms is sufficiently large for the application of the reliability based design method while the number of sea states is sufficiently small for the method to become technically practicable.

$H_{threshold}$	N_{storms}	$N_{sea\ states}$
2.5 m	1054	9319
3.0 m	736	5590
3.5 m	479	3184
4.0 m	250	1527
4.5 m	121	708

Table 3-1 The relation between threshold value, the number of storms and the number of individual sea states in the database.

Below a brief summary of the met-ocean conditions is given to get a better insight in the weather conditions at the intended location.

Environment at “gridpoint 568”

The database for the intended location of the OWEC’s provides a unique opportunity to consider the interrelationships between the main environmental parameters: wave height, wind speed, current speed and their associated directions. The figures given below are all based on the values of the environmental parameters at the peak of a storm (see section 2.1.1 for the definition of the peak of a storm).

In figure 3-2 the significant wave height is plotted against the mean wave direction with the wave direction taken as 'coming from'. In the figure the influence of the continental land mass on the height of the waves can clearly be seen.

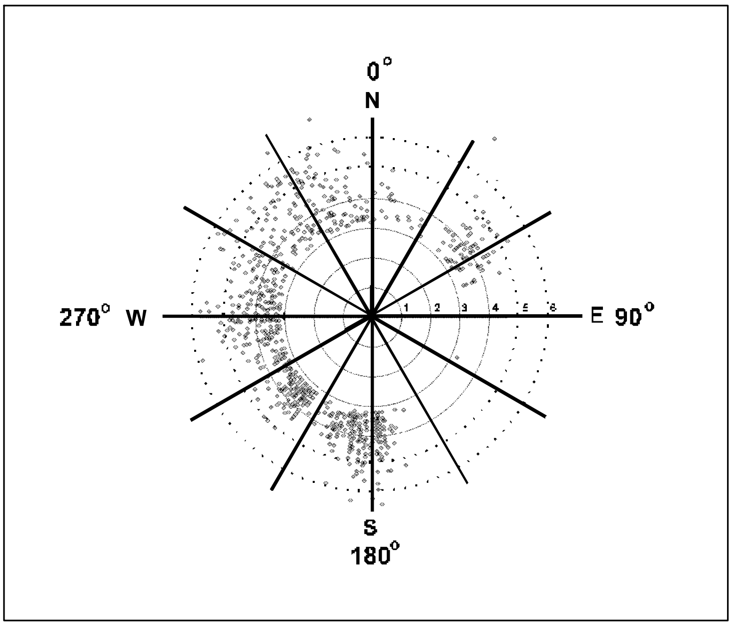

Figure 3-2 Polar plot of significant wave height and mean wave direction ('coming from').

Next the directions of the waves and wind are compared in figure 3-3. It appears that the mean wind and mean wave direction are strongly correlated, which is of no real surprise as the waves are generated by the wind.

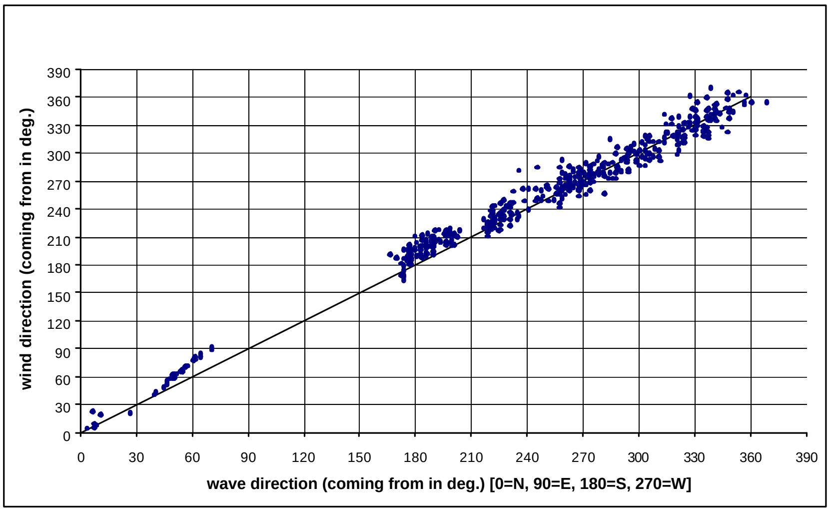

Figure 3-3 Mean wave direction vs. mean wind direction (both defined as 'coming from').

In contrast to their directions, the hourly mean wind speed and the significant wave height at the peak of the storms are appreciably less well correlated (figure 3-4). Part of the scatter can be easily explained by differences in water depth and fetch length for different wind directions, resulting in different significant wave heights for the same wind speed.

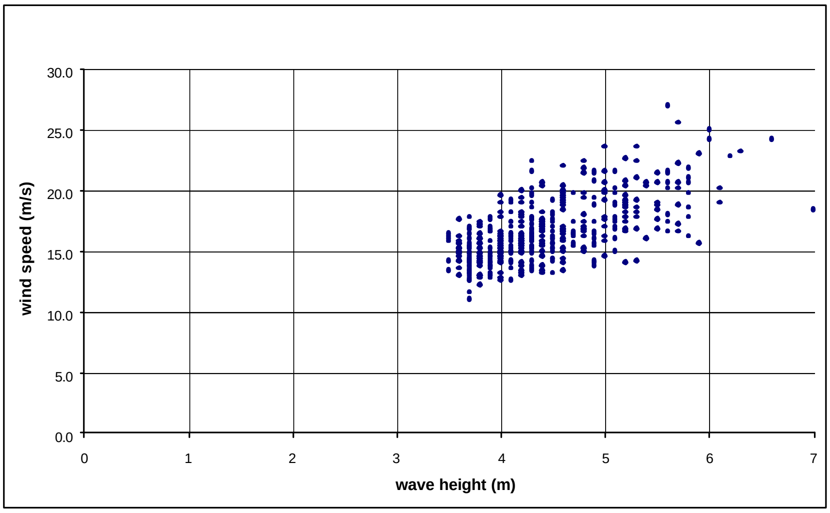

Figure 3-4 Significant wave height vs. hourly mean wind speed at 10 m height.

It has been found that the depth integrated current speed is not correlated to either the significant wave height or the mean hourly wind speed. As an illustration, the depth integrated current speed is plotted against the hourly mean wind speed in figure 3-5. The current speed plotted against the significant wave height would have given a similar amount of scatter and is therefore not shown.

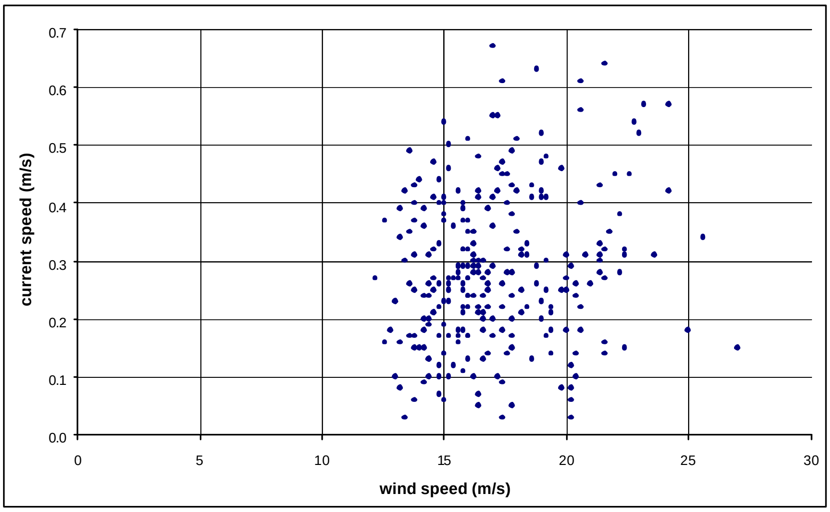

Figure 3-5 Hourly mean wind speed at 10 m height vs. depth integrated current speed.

The observation that the current does not depend on wave height or wind speed strongly supports the approach to determine the design environmental conditions as outlined in section 2.1. Applying the conventional approach, the chance is very small that the individual 50 years wave height, wind speed and current speed occur at the same time. The observation that the current is hardly correlated to either wind or waves is generally valid for the whole North Sea and is the very reason why North Sea structures for the petroleum industry are 'overdesigned' [3-1].

Finally, the depth integrated current speed and its direction are given in figure 3-6. The figure shows two peaks which are separated by 180 degrees which reflects the influence of the tide on the current.

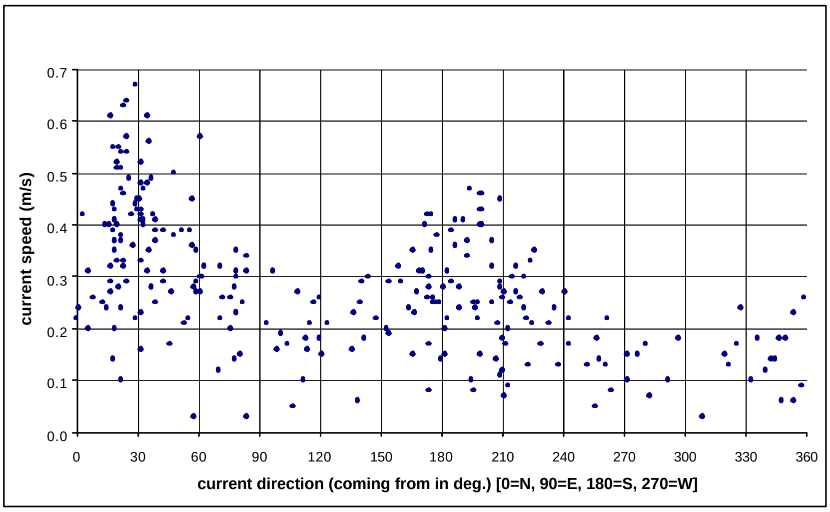

Figure 3-6 Current direction (defined as 'coming from') vs. depth integrated current speed.

3.2 Model of the support structure

The OWEC support structure (as designed by partner Kværner Earl & Wright, KEW [3-2]) is a lattice tower with a height of 84 m. It has three legs which are supported on the ocean floor by three foundation pods. The distance between the pods is 50 m (figure 3-7).

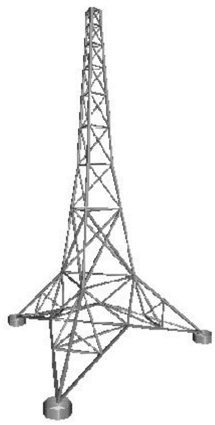

Figure 3-7 The OWEC support structure

The behaviour of the support structure is determined using the non-linear response analysis program NIRWANA [3-3]. The structure is idealised according to the finite element method applying straight beam elements. The structure/soil interaction may be idealised by linear or non-linear springs. In this work, however, linear foundation behaviour has been assumed. A free vibration analysis has been performed to determine the natural periods of the structure. In table 3-2 the values for the first three modes are given. Figure 3-8 shows the corresponding mode shapes.

No.	*Natural Period*	*Description*
1	1.35 s	Bending mode
2	1.33 s	Bending mode
3	1.24 s	Torsion mode

Table 3-2 The natural periods of the support structure as determined by a free vibration analysis using NIRWANA.

The three mode shapes and associated natural periods are in good agreement with the results obtained by KEW.

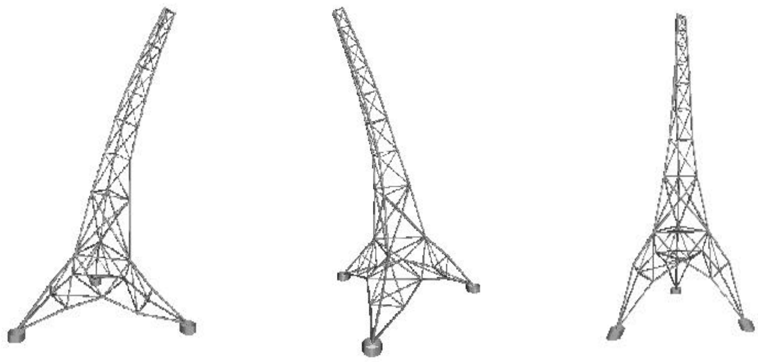

Figure 3-8 The mode shapes of the support structure as determined by a free vibration analysis using NIRWANA.

In table 3-3 the Dynamic Amplification Factors for the extreme horizontal force at the seabed are given for a range of sea states. Here the DAF is defined as the ratio of the mean extreme dynamic horizontal force in a 3 hour period and the mean extreme quasi-static horizontal force in that period. Due to the statistical uncertainty in both values the DAF's all have a value of about 1.0; in table 3-3 a DAF of e.g. 0.95 cannot 'statistically' be distinguished from a DAF of 1.05. The influence of dynamics is expected to be greatest, in a relative sense, in the sea states with smaller values of H_s. These sea states have their peak in the energy distribution at periods which are closer to the dominant natural periods of the lattice tower. Consequently, it is concluded that dynamics do effectively not play a role and can be left out of consideration in the further analysis of this **specific** structure at this **specific** location.

Sea state	*DAF*
H_s = 2.5 m, T_p = 7.0 s	1.02
H_s = 3.5 m, T_p = 9.0 s	1.00
H_s = 4.5 m, T_p = 10.0 s	0.98

Table 3-3 Dynamic Amplification Factor of the extreme horizontal force at the seabed due to time varying wave loading.

3.3 Hydrodynamic loading

The hydrodynamic forces due to waves and current on small diameter structural elements are traditionally calculated using the Morison equation [see e.g. 3-4] with the current assumed to be constant during a sea state. According to this force model the load per unit length on a member of a (vibrating) system may be written as:

$$dF_{h,t} = \frac{p}{4}\left(C_M - 1\right)D^2\, r\left(\dot{u}_t - \ddot{x}_t\right) + \frac{p}{4} D^2\, r\dot{u}_t + \frac{1}{2} C_D D r\left(u_t + U_c - \dot{x}_t\right)\left|u_t + U_c - \dot{x}_t\right| \quad \text{(3-1)}$$

where

$dF_{h,t}$	-	hydrodynamic force per unit length of the member (N/m)
C_M	-	inertia coefficient (-)
C_D	-	drag coefficient (-)
r	-	mass density of sea water (kg/m^3)
D	-	diameter of the member (m)
u_t	-	wave induced water particle velocity (m/s)
$\dot{u}_t$	-	wave induced water particle acceleration (m/s^2)
U_c	-	current speed (m/s)
$\dot{x}_t$	-	structural velocity of the member (m/s)
$\ddot{x}_t$	-	structural acceleration of the member (m/s^2)

As discussed in section 3.2, dynamic effects do not have to be taken into account in the response analysis of the lattice tower. Therefore, the terms in equation 3-1 reflecting the structural movements, $\dot{x}_t$ and $\ddot{x}_t$, are set to zero. The term dependent on the velocity of the fluid is well-known as the drag force, the terms dependent on the acceleration of the fluid are combined and generally referred to as the inertia force. The drag and inertia coefficients were taken to be the same as those used by partner Kværner Earl & Wright for the design of the structure [3-2], i.e. C_D=0.74 and C_M = 2.0. This choice can be debated as the selected values of the coefficients have to be compatible with the method in which they are used. The KEW values are common values in conjunction with particle kinematics based on design wave methods. For random time domain simulations the water particle kinematics approximate the real kinematics in a sea state and, consequently, the hydrodynamic coefficients should also be the best estimate of the real value of the coefficients.

The time-varying water particle velocities and accelerations are derived from the water surface elevation where the energy content of the ocean surface is taken to be fully determined by the parameters H_s and T_p. The calculation of the wave kinematics is based on the linear wave theory which presupposes that the (random) ocean surface is built up from numerous independent wavelets each having a particular amplitude, frequency, direction and a random phase. Assuming that all wavelets are travelling in the same direction this can be written analytically as:

$$h_i(t) = \sum_{i=1}^{N} C_i \cdot \cos\,\left(k_i x - w_i t + j_i\right) \quad \text{(3-2)}$$

where:

h_i	-	surface elevation (m)

C_i - amplitude of an individual wavelet (m)
w_i - angular frequency of an individual wavelet (rad/s)
j_i - random phase angle
t - time (s)
x - the horizontal co-ordinate of the point of interest (m)
k_i - wave number of an individual wavelet, given by the dispersion relation:

$$\frac{w_i^{\,2}}{g} = k_i \cdot \tanh(k_i d) \tag{3-3}$$

with:

g - acceleration due to gravity (m/s^2)
d - water depth (m)

The wavelet amplitude, C_i, and frequency, w_i, are correlated by the wave spectrum which describes the energy content of the ocean surface as a function of frequency. Wave spectra can be determined from large measurement programmes at the site to incorporate the characteristics of the environment at that location. However, such programmes are expensive and time-consuming and it is therefore common practice in offshore engineering to use a standard form of wave spectrum. The wave spectrum as used in this work to identify the ocean surface in a 3 hour period is a JONSWAP spectrum [see e.g. 3-4]. This spectrum is determined by the significant wave height, H_s and the associated peak spectral period, T_p of the ocean surface.

The horizontal water particle velocities necessary for the drag force due to wave action are determined using equation (3-4).

$$u(t,z) = \sum_{i=1}^{N} w_i \cdot G_s(z,d,k_i) \cdot C_i \cdot \cos(k_i x - w_i t + j_i) \tag{3-4}$$

with depth decay function:

$$G_s = \frac{\cosh(k_i \cdot (z+d))}{\sinh(k_i \cdot d)} \tag{3-5}$$

where:

z - Cartesian co-ordinate, positive upward and zero at still water level (m)

Equation 3-5 gives the relation of the water particle velocity as a function of depth: the velocities decrease with depth.

The horizontal acceleration used to determine the inertia wave forces are determined using:

$$\dot{u}(t,z) = \sum_{i=1}^{N} w_i^{\,2} \cdot G_s(z,d,k_i) \cdot C_i \cdot \sin(k_i x - w_i t + j_i) \tag{3-6}$$

A limitation of the linear wave theory is that the kinematics of the water particles within a (large) wave crest cannot be determined; linear wave theory is only valid up to Mean

Sea Level. In this way the total hydrodynamic load is estimated unconservatively. Therefore a modification has been applied to the depth decay function (equation 3-5) by using a stretching technique as originally proposed by Wheeler [3-5].

3.4 Aerodynamic loading

Next to hydrodynamic loading, the support structure of the OWEC is also subjected to aerodynamic loading. The aerodynamic loading can be separated into loading on the turbine and loading on the support structure. The extreme combined wind load experienced in a 3 hour period will be taken as a constant value during the sea state.

Wind loading on turbine

The loading on an installed wind turbine can be separated into two stages which are dependent on the wind speed:

- loading on turbine during operation
- loading on turbine during stand-by

Since we are interested in the design values of structural response with a large return period (e.g. 50 or 100 years), it is assumed throughout in the calculations that the turbine is in stand-by mode during the passage of a storm.

The extreme aerodynamic loads from the parked rotor and nacelle which act on the tower top for this machine can be determined with equation 3-7:

$$F_{a,turbine} = g_s \cdot 78 \cdot \left(\frac{V_{1\min}}{60} \right)^2 \qquad (3\text{-}7)$$

where:

$F_{a,turbine}$ - aerodynamic thrust load on the turbine at stand-by (perpendicular to the rotor plane) (kN)
V_{1min} - 1 minute mean wind speed at hub height (m/s)
g_s - modified gust response factor (taken as 1.1)

This formula is valid for the 3 MW wind turbine of Kværner Turbin [3-6].

Wind loading on the support structure

The aerodynamic drag thrust load on the support structure of the OWEC can be determined using equation 3-8, see [3-7].

$$F_{a,structure} = g_S \frac{r_{air}}{2} V_{1,\min}^2 \left(h_i\right) \sum_i c_{di}\left(h_i\right) A_i\left(h_i\right) \qquad (3\text{-}8)$$

where:

$F_{a,structure}$ - the aerodynamic drag force due to (extreme) mean wind (N)
r_{air} - air density (kg/m^3)
h_i - height (above still water level) of component i (m)
c_{di} - drag coefficient of component i
A_i - effective area of component i (m^2)

i - component (tower, nacelle)

Realistic values for the different coefficients have been implemented in a spreadsheet which has been developed by partner Institute for Wind Energy (IvW). This spreadsheet has been used to determine the wind force on the support structure for different wind speeds.

4. Testing of constrained simulation methodology

4.1 Preparation of computations

The application of the reliability based design method to the OWEC support structure is very time-consuming. Therefore some effort has been spent at the beginning of the process to optimise the procedure of the extreme response analysis which must be performed for all sea states found in the storm database. It is, therefore, investigated first how the prime building block of the reliability based method, i.e. the distribution of the extreme response in a sea state $F_e(r_e| H_s(i), T_p(i), U_c(i), V_w(i), ...)$ in equation 2-1, can be estimated accurately in the most efficient way. Second, the distribution of the extreme response during a storm $F_e(r_e \,|\, storm\ s)$ in equation 2-2 is considered which may also give some opportunities to optimise the analysis of the database. All results presented in this section concern again the total horizontal force at the sea bed or base shear. However, any other response variable could have been selected equally well.

As discussed in section 2.2.3, the constrained simulation methodology is based on determining the distribution of the maximum response associated with one particular wave crest height, A_{crest} embedded in an otherwise random simulation. These so-called conditional maximum response distributions are determined for a range of wave crest heights. Finally, the conditional maximum response distributions are combined with the probability of exceedence of each wave crest height in the sea state to determine the distribution of the extreme response in the sea state.

To determine a conditional maximum response distribution for a specific wave crest height using constrained simulations of short time length is relatively simple. The accuracy of the conditional maximum response distribution, and thus the accuracy of the final extreme response distribution, strongly depends on the number of constrained simulations per crest height that is used. This process must be repeated for a number of wave crest heights. In this work only 4 constrained wave crest heights will be used. Now the accuracy of the final answer, the extreme response distribution, only depends on the total number of constrained simulations which are distributed over the 4 wave crest heights. Obviously, a balance has to be found between the total number of constrained simulations and the accuracy of the extreme response distribution.

4.2 Results from random simulations

To demonstrate that constrained random simulations can indeed be used efficiently to determine the distribution of the extreme response in a sea state, fully random simulations were performed first. The distribution of the extreme response obtained using multiple random simulations of 3 hours length can then be used as a reference distribution for the results obtained from the constrained simulations. Two sea states of different severity (see table 4-1) have been considered to bracket the range of sea states which need to be analysed and thus extend the validity of the comparisons that were made.

	H_s	T_p	U_c	V_w

sea state 1	2.5 m	7 s	0 m/s	0 m/s
sea state 2	4.5 m	10 s	0 m/s	0 m/s

Table 4-1 Environmental parameters of 2 sea states which are analysed using fully random time domain simulations and constrained random simulations.

For each sea state 11 simulations of 3 hours length have been performed with the structure modelled both dynamically and quasi-statically; these 2x2x11=44 simulations took in total 90 hours of computer time. For each sea state, the largest response in each 3 hours simulation was determined. Using the 11 extreme values an empirical cumulative probability distribution of the extreme response in a sea state could be constructed. In figure 4-1 the empirical distributions are given for the extreme dynamic and quasi-static base shear for both sea states.

The figure clearly illustrates that dynamics are not important for the structure and location considered and can therefore be neglected in further analyses. The main statistical properties of the distributions are given in table 4-2.

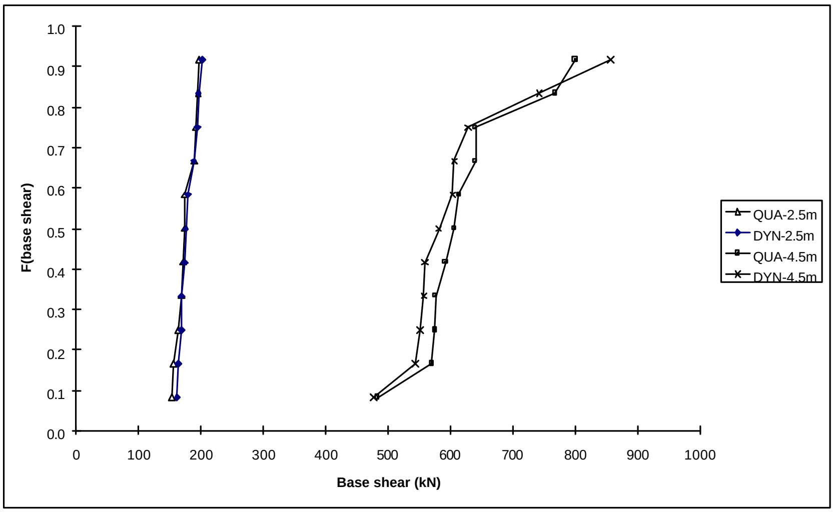

Figure 4-1 The distribution of the extreme dynamic and quasi-static base shear for sea state 1 and sea state 2 determined from 11 random simulations of 3 hours length.

The standard error, which is defined as the standard deviation of the extreme response divided by the square root of the number of estimates, is a measure of the accuracy of the mean of the extreme response. The ratio of the standard error to the corresponding mean extreme response can be used as a yardstick for the accuracy of the results for the two sea states. It can then easily be seen that the distribution of the extreme (quasi-static) response for sea state 1 is of higher accuracy than for sea state 2.

	sea state 1		sea state 2	
	Quasi-static	*Dynamic*	*Quasi-static*	*Dynamic*
Mean	175 kN	179 kN	624 kN	610 kN
Standard error	5 kN	4 kN	27 kN	32 kN
Standard deviation	15 kN	14 kN	90 kN	105 kN

Table 4-2 Statistical properties of the extreme dynamic and quasi-static base shear for sea states 1 and 2 - determined from 11 random simulations of 3 hours length.

4.3 Resulst from constrained random simulations

In order to find the most efficient simulation procedure, different numbers of constrained simulations have been used to determine the distribution of the extreme response in a sea state. As stated before, constrained simulations will be performed for 4 different crest heights ($A_{crest,i}$) in a sea state. These crest heights are set in relation to the significant wave height of the sea state:

$$\begin{aligned} A_{crest,1} &= 0.5 \cdot H_s \\ A_{crest,2} &= 0.75 \cdot H_s \\ A_{crest,3} &= 1.0 \cdot H_s \\ A_{crest,4} &= 1.5 \cdot H_s \end{aligned}$$

The conditional maximum response distributions given each wave crest height were determined using 20, 50 and 100 constrained simulations per wave crest height. Having 4 wave crest heights, the total numbers of simulations are then 80, 200 and 400 per sea state. The distribution of the extreme response in a sea state is next obtained by convolution of the conditional maximum response distributions with the (Rayleigh) distribution of the wave crest height.

This procedure produces one extreme response distribution with one value each of the mean, m_{re}, and standard deviation, s_{re}, of the extreme response. The accuracy of this distribution is unfortunately unknown. However, by repeating the analysis for the same environmental conditions but with different random seeds more combinations of m_{re} and s_{re} are produced. Now a simple statistical analysis can be performed to get more information about the accuracy of the extreme response distribution. This information is contained in the mean and standard deviation of both m_{re} and s_{re} e.g.:

$$mean\left(m_{r_e}\right) = \frac{1}{N}\sum_{i=1}^{N} m_{r_e} \quad \text{and} \quad stdev\left(m_{r_e}\right) = \sqrt{\frac{1}{N}\sum_{i=1}^{N}\left(m_{r_e} - mean\left(m_{r_e}\right)\right)^2} \qquad (4\text{-}1)$$

Similarly the $mean(s_{re})$ and $stdev(s_{re})$ can be determined. The $stdev(m_{re})$ will be used as a measure of accuracy of the extreme response distribution as it can easily be compared with the standard error of the mean obtained from fully random simulations. In tables 4-3 and 4-4 the results for both sea states as obtained using constrained simulations are compared to the results obtained using fully random simulations which are used as a reference.

H_s = 2.5 *m*	Constrained random simulations			Fully random simulations
	*80*64 s*	*200*64 s*	*400*64 s*	*11*3 hrs*
$mean(m_{re})$	194 kN	197 kN	184 kN	175 kN
$stdev(m_{re})$	22 kN	16 kN	16 kN	5 kN
$mean(s_{re})$	13 kN	14 kN	14 kN	15 kN

Table 4-3 Accuracy of the extreme response statistics in sea state 1 obtained using 80, 200 and 400 constrained random simulations of 64 s length and 11 fully random simulations of 3 hours length.

H_s = 4.5 *m*	Constrained random simulations			Fully random simulations
	*80*64 s*	*200*64 s*	*400*64 s*	*11*3 hrs*
$mean(m_{re})$	637 kN	609 kN	611 kN	610 kN
$stdev(m_{re})$	59 kN	16 kN	13 kN	32 kN
$mean(s_{re})$	80 kN	79 kN	79 kN	105 kN

Table 4-4 Accuracy of the extreme response statistics in sea state 2 obtained using 80, 200 and 400 constrained random simulations of 64 s length and 11 fully random simulations of 3 hours length.

Focusing first on the higher sea state the following conclusions can be drawn from table 4-4:

- the extreme response statistics can be determined efficiently using constrained simulations
- the accuracy of the extreme response statistics determined through constrained simulations increases with an increase in the (constrained) simulation effort
- the accuracy obtained by 200 and 400 constrained simulations is better than the accuracy of the 11 fully random simulations.

Considering now the lower sea state it is clear from table 4-3 that the first two of these conclusions probably remain valid but that a considerably larger number of constrained simulations would be necessary to obtain similar accuracy to the 11 fully random simulations. Fortunately, for the extreme response distribution the higher sea states are of much greater interest.

An important further conclusion can be drawn by comparing the results of the constrained simulations for the two sea states. Taking constant constrained simulation effort and using a minimum number of constrained simulations (in this case e.g. 200 simulations or more) the accuracy of the extreme response statistics will increase for an increased severity of the sea state. This is in contrast with fully random simulations: keeping the random simulation effort constant the accuracy of the extreme response statistics decreases for an increased severity of the sea state.

This apparent contradiction can be explained by the hydrodynamic loading in severe sea states. The loading in such sea states will be drag dominant and hence strongly non-linear. The simulations are as a result non-gaussian. Using fully random simulations more simulation effort is then obviously needed to obtain accurate results. The constrained simulations, however, are pre-conditioned on wave crest height and thus on the associated drag forces (drag force is in phase with crest elevation). Hence they only need to capture the variability of the hydrodynamic forces around the large value of the drag force already included. Therefore, keeping the constrained simulation effort constant, the distribution of the maximum responses associated with a wave crest will be determined with higher accuracy in higher sea states. Consequently, the distribution of the extreme response will also be determined with higher accuracy.

It has been decided that the distribution of the extreme response in each sea state will be determined using 200 constrained simulations divided over 4 wave crest heights. The results so obtained are considered to be sufficiently accurate for a wide range of sea states and can be determined in roughly 1 hour of computer simulation per sea state.

5. Application of the reliability based design method to an OWEC

5.1 Conventional design conditions

Before the method described in chapter 2 is applied to the support structure at the location of interest, first the conventional approach is used to determine the response of the support structure in design environmental conditions. The response considered here for this purpose is the total horizontal force at the sea bed usually referred to as 'base shear'. For each of the individual environmental parameters, the design values associated with a particular return period have been determined. These values are given in table 5-1.

Return period	*years*	*10*	*50*	*100*
Maximum wave height, H_{max}	m	12.6	13.6	14.0
Associated wave period, T_{ass}	s	13.4	14.1	14.4
depth integrated current speed, U_c	m/s	0.69	0.73	0.74
1 hour mean wind speed, V_w	m/s	25.8	27.4	28.0

Table 5-1 Design values of the individual environmental parameters for maximum wave height, hourly mean wind speed and current speed.

Note that the values corresponding to a return period of 50 years have been used by partner KOGL for their strength analysis in the design of the support structure [3-2].

The design base shear can now easily be determined. For the design wave a Stokes 5th order wave model has been applied. In table 5-2 the design base shears associated with various return periods are given.

Return period	*years*	*10*	*50*	*100*
Design 'hydrodynamic' base shear	kN	2470	3040	3210
Design 'aerodynamic' base shear	kN	390	450	470
Total design base shear	kN	2860	3490	3680

Table 5-2 Design base shear for a desired return period determined using a conventional approach.

From table 5-2 it can be concluded that the design of this support structure for extreme storm loading is entirely dominated by the loads due to wave and current action. The contribution of the extreme wind load to the total load is of the order of 5% for the given configuration and location. Note that this contribution would have been higher if the global overturning moment was considered instead of the global base shear. It is recognised that treatment of aerodynamic loading in this work is very crude. However, based on this result it is clear that for this particular support structure and wind turbine at this location and assuming stand-by mode of the turbine and considering base shear, an elaborate (time-varying) wind load model is not required when performing an ultimate strength analysis. The simple approach of incorporating wind loads during a sea state as adopted in this work is good enough for the given structure at the given location.

When the results from table 5-2 are compared with the analyses performed by partner KOGL [3-2] similar values for the design hydrodynamic and aerodynamic horizontal loads are found. This confirms that the structural model and the software tools used in this part of the project provide realistic results.

5.2 Results

As discussed in section 3.1, it will take a lot of time to consider the responses for all the sea states in the storm database. At present the first 361 storms in the first 19.3 years of the database have been analysed. These storms were selected using a threshold value of 3.5 m and were used in the application of the reliability based design method outlined in section 2. The results thus obtained will be compared to the results obtained using a conventional approach which were given in section 5.1. It is believed that the number of analysed storms is large enough for the application of the reliability based design method for the determination of the extreme response distribution in a design life time of e.g 50 or 100 yrs as more than 75 % of the storms found in the database have been examined. The total horizontal force at the sea bed (‘base shear’) has again been taken as the response variable of interest.

5.2.1 The distribution of extreme response in a storm

It will now be illustrated how the distribution of the extreme response in a storm is determined in practice. The storm considered here was recorded on 17th January 1965 and has a total duration of 30 hours (10 successive sea states). In table 5-3 the environmental conditions for each of the 10 sea states are given.

sea state number	H_s (m)	T_p (s)	q_m (deg)	U_c (m/s)	q_c (deg)	V_w (m/s)	q_w (deg)
1	3.7	9.5	231	0.12	261	14.8	235
2	3.7	9.5	232	0.55	22	15.6	236
3	3.9	9.5	238	0.30	67	16.8	248
4	4.2	9.5	243	0.26	180	17.6	253
5	4.4	9.5	251	0.15	267	18.4	266
6	4.3	9.5	253	0.56	20	17.2	255
7	4.4	9.5	249	0.36	49	17.8	249
8	4.5	9.5	251	0.27	168	16.0	263
9	4.4	9.5	259	0.18	229	15.0	278
10	3.9	8.1	271	0.49	19	12.4	284

Table 5-3 The environmental conditions of the sea states present in storm 17th January 1965 with q_m and q_w defined as ‘coming from’ and q_c defined as ‘pointing towards’.

Next, the distributions of the extreme response given each sea state have been determined using 200 constrained simulations. The distribution of the extreme response given the storm can now easily be determined by the product of the extreme response distributions given the sea states (see equation 2-2). Figure 5-1 gives the results for this storm.

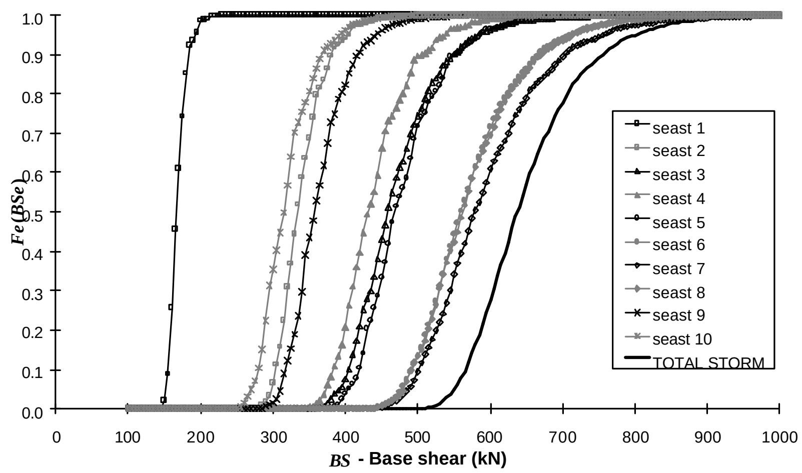

Figure 5-1 The distribution of the extreme base shear in the 17/Jan/1965 storm and the distribution of the extreme base shear in each of the 10 sea states in that storm, as determined using 200 constrained simulations per sea state.

Figure 5-1 illustrates that the distribution of the extreme response in the storm is not solely determined by the peak sea state of the storm (here sea state 8). On the other hand, not all sea states in the storm influence the extreme response distribution for the whole storm to the same degree. Therefore, a further reduction of the simulation effort per storm might be achieved by determining the extreme response distribution for the largest sea states per storm only. It has yet to be investigated at which cut-off sea states in a storm can be 'neglected'. The results presented in this report have been determined by incorporating the extreme response distribution for every sea state in each storm.

5.2.2 Generic distribution of normalised extreme response

The 361 storms analysed so far differ considerably in severity. The largest storm peak significant wave height was found to equal 6.6 m, while the smallest storm peak significant wave height equalled 3.6 m. In figure 5-2 10 examples of the distribution of the extreme response during a storm (equation 2-2) are given.

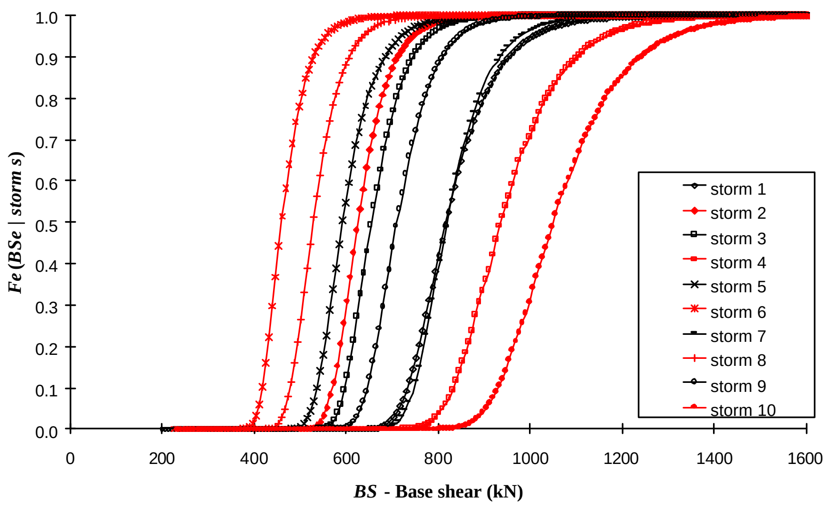

Figure 5-2 10 examples of the distribution of the extreme response during a storm.

The next step is to check whether the distribution of the extreme response in a storm has a standard shape which is independent of the severity of the storm. Therefore each extreme response distribution for a storm has been normalised by its measure of central tendency for which the median value has been taken. Figure 5-3 shows the normalised extreme response distributions for the 10 example storms which have also been used in figure 5-2.

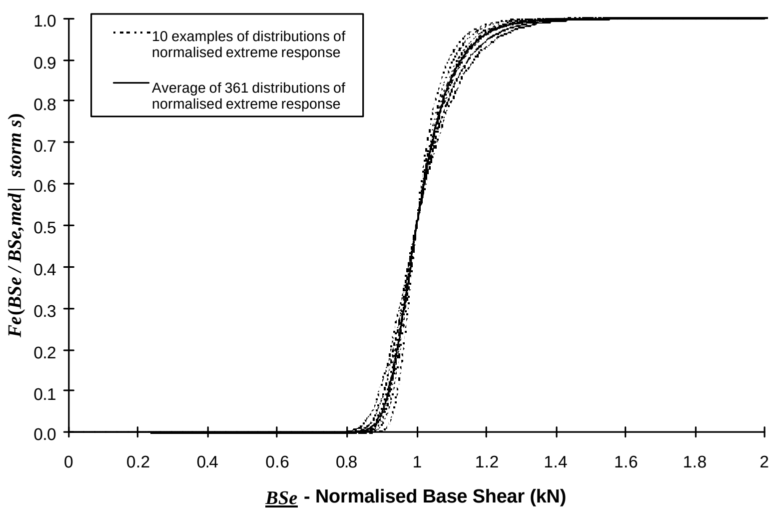

Figure 5-3 10 examples of the distribution of the normalised extreme response and the average distribution of the normalised extreme response based on 361 storms.

In figure 5-3 the generic distribution of normalised extreme response has also been plotted. This is the average of the normalised distributions for all 361 storms (see equation 2-3).

Using the generic normalised distribution it is now possible to determine the distribution of the extreme response for a storm with a given severity, where the median extreme response value is used as an indicator of the severity of a storm.

5.2.3 Distribution of storm severity

In order to estimate the storm severity during a desired lifetime larger than the analysed 19.3 years, the 361 storm severity values are fitted to an extreme value distribution and extrapolated. In section 2.1.2 it was suggested to fit a Weibull or a Generalised Pareto distribution to the median extreme response values. Next, the results are shown when a 3-parameter Weibull distribution (equation 5-1) is adopted.

$$F_{e,fitted}\left(r_{e,med}\right) = 1 - e^{-\left(\left(\frac{r_{e,med}-m}{a}\right)^{k}\right)} \tag{5-1}$$

where:

$F_{e,fitted}$	-	fitted cumulative probability distribution	
m	-	shift parameter	fitted value: m = 449.7 kN
a	-	scale parameter	fitted value: a = 370.8 kN
k	-	shape parameter	fitted value: k = 1.49

In figure 5-4 the empirical cumulative probability distribution and the fitted Weibull distribution for the 'storm severities' are given.

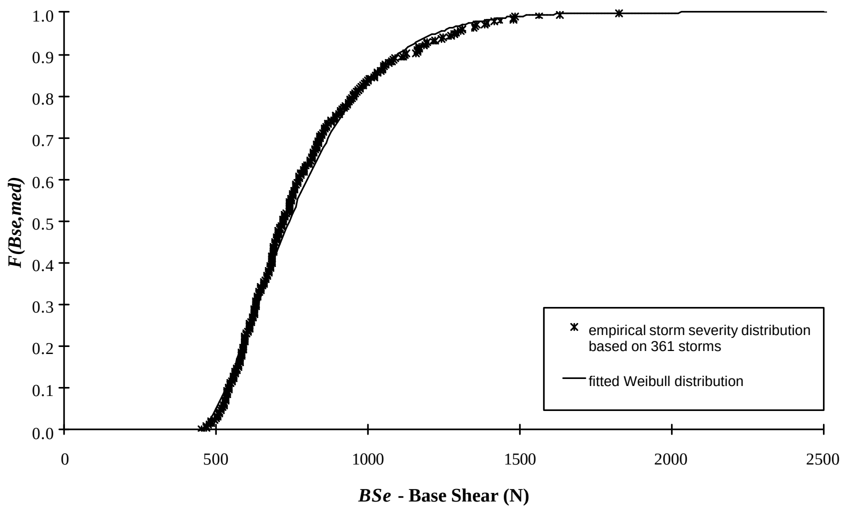

Figure 5-4 The empirical and fitted cumulative distribution of median extreme base shear values in a storm.

Based on figure 5-4 it can be concluded that the Weibull distribution fits reasonably well to the 19.3 year range of storm severities. However, if the complementary distribution is taken for both the empirical and fitted distribution and these are plotted on a semi-(natural) logarithmic scale then the sensitivity of the fit in the high end of the tail can be seen (figure 5-5).

The fitted distribution is slightly unconservative in the upper tail of the probability distribution as a smaller probability of occurence is attached to storm severities with large return periods in comparison to the empirical distribution. Consequently, it is to be expected that the long term extreme response distribution in a design period is estimated unconservatively.

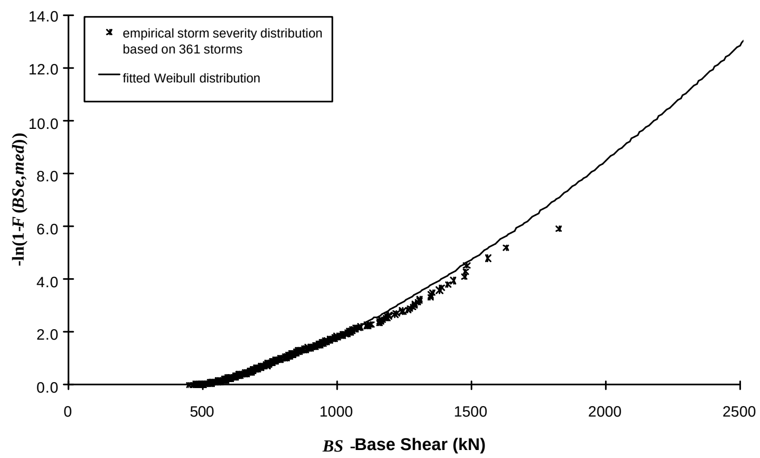

Figure 5-5 The empirical and fitted complementary distribution of median extreme base shear values in a storm plotted on semi-natural logarithmic scales.

5.2.4 Distribution of the extreme response in a chosen lifetime

Distribution of the extreme response given a storm occurring at random

Using the results of the previous sections the distribution of the extreme response for a storm occurring at random has been determined with equation 2-5; the result is shown in figure 5-6. In this distribution the severity of the random storm has been taken into account. However, the probability **that** a storm will pass the support structure is not yet incorporated. This will be done by estimation of the storm arrivals.

Distribution of the extreme response in a chosen lifetime

Treating the storm arrivals as a Poisson process the long term distribution of the extreme response in a chosen lifetime can easily be determined (equation 2-6); the mean arrival rate for the given set of storms is n = (361+1)/19.3 = 18.8 storms per year. In figure 5-6 the long term distributions of the extreme response given various desired lifetimes are shown.

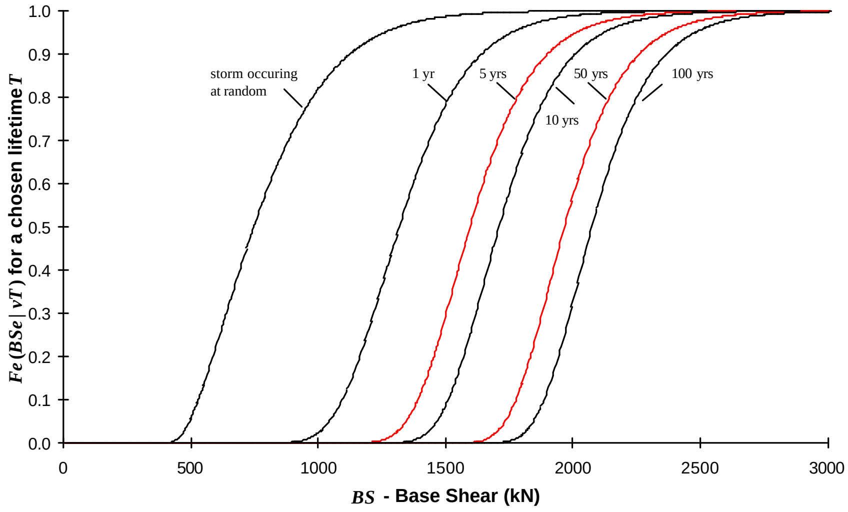

Figure 5-6 Long term distribution of the extreme base shear during a chosen lifetime.

In figure 5-7 the upper part of the complementary long term distribution function $H_D(r_e)$ has been plotted on a semi-logarithmic scale for values of $H_D(r_e) < 0.50$. This is a figure in the form of figure 2-5 with the actual results of the calculations performed inserted. The horizontal axis is the probability that the base shear value along the vertical axis will be exceeded, which may also be considered as a probability of failure when the base shear value represents the mean ultimate strength of the support structure; see equation 2-9. Each curve in figure 5-7 refers to one particular lifetime of the structure.

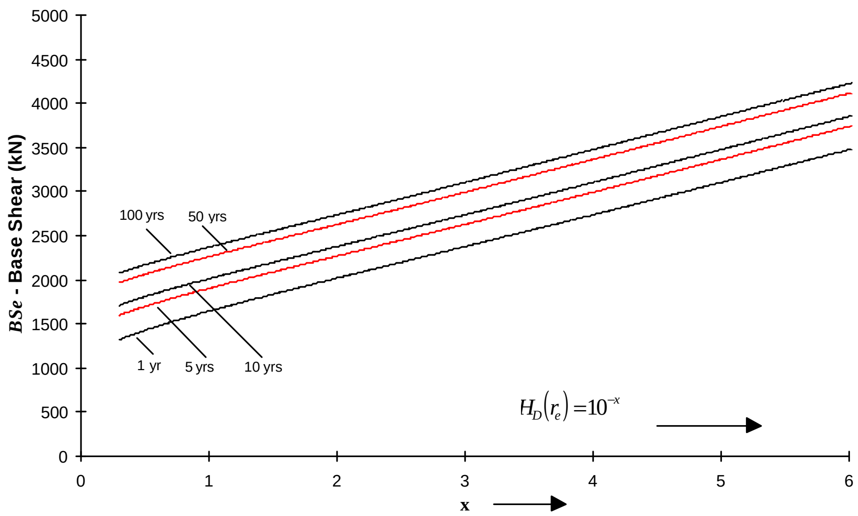

Figure 5-7 The probability of exceedence of a particular extreme base shear value BS_e in a given lifetime of the support structure.

Which is equivalent with: The probability of failure in a given lifetime for a design ultimate strength of the support structure.

5.2.5 Discussion of the results

With these results it is demonstrated that a reliability based design method will be of benefit for this specific structure at its specific location, always provided that the design of the support structure is governed by considerations of extreme strength.

To demonstrate this the total loading as obtained using a reliability based approach and using a conventional approach (see section 5.1) are compared. As the reliability based design method provides the whole distribution of the extreme response rather than one value, a single representative value will first have to be chosen to make the comparison. Without discussing the merits of one particular choice over another the median value of the long term distribution of the extreme response in a chosen lifetime has been selected for the comparison. These values can be read from figure 5-6 or figure 5-7. The results are presented in table 5-4. Note that all values presented here are unfactored. From the table it can be concluded that the design horizontal loading for a conventional design approach is conservative for large design lifetimes.

A good illustration of the power of the reliability based design method is given by determining the probability of failure in a given lifetime for a number of assumed mean ultimate strength values. These probabilities can be read from figure 5-7. The results are summarised in table 5-5.

Return period	*years*	*10*	*50*	*100*
'Conventional' design base shear	kN	2850	3490	3680
Median value of the 'Reliability based' design base shear	kN	1710	1970	2080
reduction	%	40 %	44 %	44 %

Table 5-4 Comparison of the design base shear as obtained using a conventional design approach and a reliability based design approach.

Probability of failure for mean ultimate strength of:	*Lifetime*		
	10 years	*50 years*	*100 years*
2850 kN	$4.9 \cdot 10^{-4}$	$2.4 \cdot 10^{-3}$	$4.9 \cdot 10^{-3}$
3000 kN	$1.9 \cdot 10^{-4}$	$9.5 \cdot 10^{-4}$	$1.9 \cdot 10^{-3}$
3490 kN	$9.2 \cdot 10^{-6}$	$4.6 \cdot 10^{-5}$	$9.2 \cdot 10^{-5}$
3500 kN	$8.6 \cdot 10^{-6}$	$4.3 \cdot 10^{-5}$	$8.6 \cdot 10^{-5}$
3680 kN	$2.9 \cdot 10^{-6}$	$1.4 \cdot 10^{-5}$	$2.9 \cdot 10^{-5}$
4000 kN	$4.0 \cdot 10^{-7}$	$2.0 \cdot 10^{-6}$	$4.0 \cdot 10^{-6}$
4500 kN	$1.7 \cdot 10^{-8}$	$8.7 \cdot 10^{-8}$	$1.7 \cdot 10^{-7}$
5000 kN	$7.0 \cdot 10^{-10}$	$3.5 \cdot 10^{-9}$	$7.0 \cdot 10^{-9}$

Table 5-5 Probability of failure for three different lifetimes and a number of assumed mean ultimate strength values

For design purposes the information in figure 5-7 (or table 5-5 which is entirely the same) can be used in the following manner. First, determine the desired lifetime and a minimum required probability of failure. This corresponds with a minimum value of the ultimate strength of the support structure along the vertical axis that is necessary to achieve this. Next divide this value by a resistance factor to transform the ultimate strength into the nominal or design strength of the structure. This latter value then serves as the design base shear force (including a load factor) due to environmental loading in an otherwise conventional design process. It is now assured that, with a given probability, the design strength equals or exceeds the design base shear. It should be recognised that the process involved is an iterative one: a configuration and preliminary structural dimensions must be assumed to be able to construct figure 5-7 while only after these results are known the design load can be established. However, this is a common feature of all design processes. While the treatment in this work is entirely focused on base shear as an example of a global response parameter, the same procedure can be applied to any global or local response variable of interest.

5.3 Sensitivity analysis of results

In section 5.2.3 it was noted that, for this specific case, the fitting to the storm severities was unconservative for storm severities with large return periods. This inherently implies that the long term distribution of extreme response is also estimated unconservatively. Therefore, it was decided to evaluate the sensitivity of the results from the reliability based design method to various influences. On the basis of 19.3 years of storm data the following influences will be investigated in this section:

- the number of storms that are analysed;
- the type of extreme value distribution that is fitted to the storm severities;
- the way the extreme value distribution is fitted to the storm severities.

In the following sections the results from the analysis of each influence will be compared to the results presented in section 5.2. The comparisons will be mainly focused on the probability of failure in a given lifetime and hence the results are given in the form of figure 5-7.

5.3.1 Number of storms analysed

The application of the reliability based design method is based on analysing the response behaviour of the structure of interest to all storms found in the database. As the database contains, by definition, a limited amount of information about the environment it is hoped that the results from the reliability analysis are not strongly influenced by the length of the database. If instead the results appear to be strongly influenced this would be a dangerous observation as one cannot tell a priori if the obtained estimates of e.g. the probability of failure in a lifetime are reliable. Consequently, the reliability based design method has been applied to the same model at its intendent location for 4 different lengths of the storm database:

a) 79 storms in the first 5.0 years of the database (n = 16.0 storms per year);
b) 153 storms in the first 10 yrs of the database (n = 15.4 storms per year);

c) 237 storms in the first 15 yrs of the database (n = 15.8 storms per year);
d) 361 storms in the first 19.3 yrs of the database (n = 18.8 storms per year).

Note that the definition of a storm, as given in section 3.1, and the application of the reliability based design method to the problem have not been changed. The mean storm arrival rates, n, indicate that there is a strong increase in the number of storms in the last 4.3 yrs of the analysed part of the database.

Distribution of storm severity

In figure 5-8 the fitted distributions of storm severity are plotted on semi logarithmic scale as obtained after the extreme response analysis of the first 5, 10, 15 and 19.3 years of the storm database. The empirical estimates of storm severities have again been fitted to a Weibull distribution (Eqn. 5-1). The values for the parameters of the Weibull distribution are given in the table 5-6:

Parameters of Weibull distribution	*79 storms 5 yrs*	*153 storms 10 yrs*	*237 storms 15 yrs*	*361 storms 19.3 yrs*
Shift parameter m	475.2 kN	449.7 kN	449.7 kN	449.7 kN
Scale parameter a	332.8 kN	383.3 kN	371.1 kN	370.8 kN
Shape parameter k	1.40	1.61	1.56	1.49

Table 5-6 Comparison of the parameters of the Weibull distribution fitted to the median extreme response values given a storm as obtained after analysis of the first 5, 10, 15 and 19.3 years of the database.

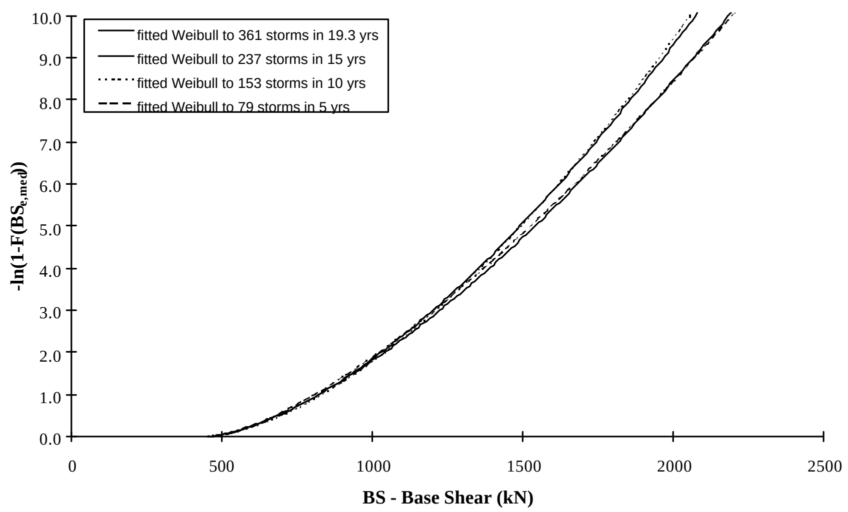

Figure 5-8 The fitted complementary distributions of median extreme base shear values in a storm plotted on semi-natural logarithmic scale based on the analysis of the first 5, 10, 15 and 19.3 years of the database.

It can be seen from figure 5-8 that the fitted distributions based on either 5 or 19.3 years of data are most conservative. Consequently, it is to be expected that the long term extreme response distributions in a chosen life time are also estimated conservatively in comparison with the analyses based on the first 10 and 15 years of the storm database.

Generic distribution of normalised extreme response

Next to the distribution of storm severities the generic distribution of normalised extreme response (Eqn. 2-3) also plays an important role in the reliability based design method. In figure 5-9, this distribution is plotted after analysis of the first 5, 10, 15 and 19.3 years of the storm database. As can be seen from the figure, there is no difference between the four distributions. This means that expected differences in the distribution of the long term extreme response are predominantly due to the difference in the extrapolation of the storm severities.

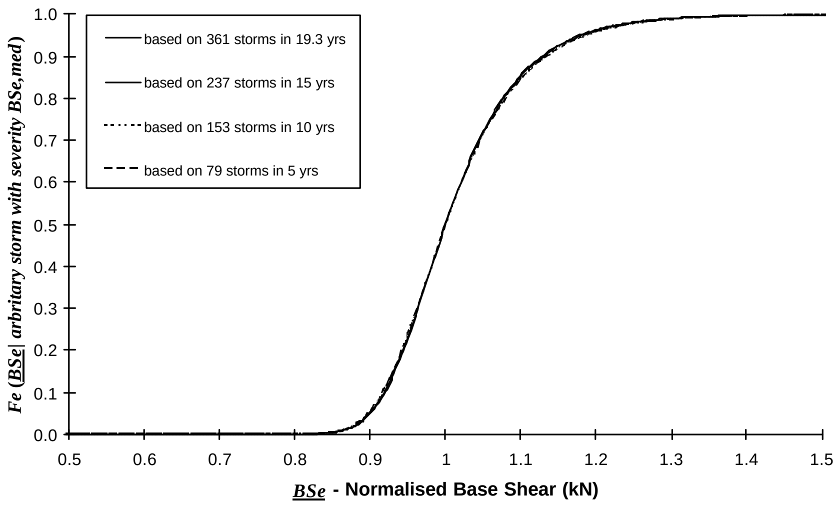

Figure 5-9 The generic distribution of the normalised extreme response as determined by analysing the first 5, 10, 15 and 19.3 years of the storm database.

Distribution of the extreme response in a chosen lifetime

In figure 5-10 the upper part of the complementary distributions of long term extreme response for a lifetime of 50 years are plotted on a semi logarithmic scale as obtained after analysing a varying number of storms. Furthermore, in table 5-7 the median extreme response value in a design lifetime is given for different storm database lengths. In the same table the design values obtained from the conventional design approach are given as well.

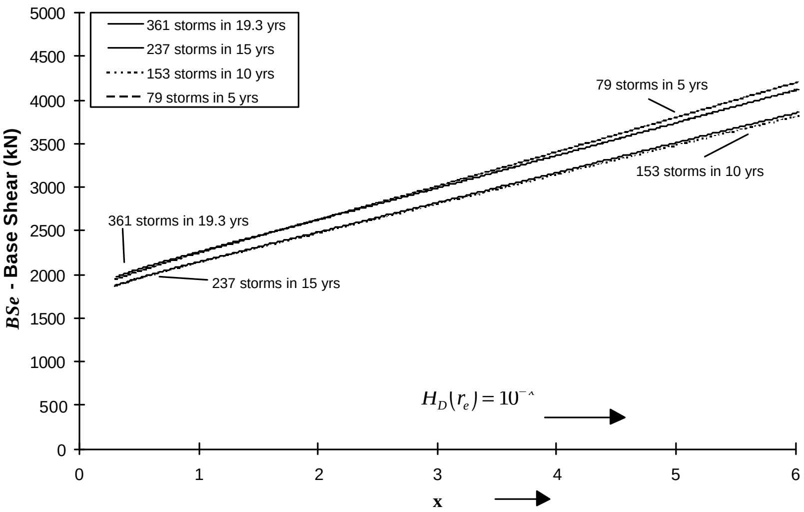

Figure 5-10 The probability of exceedence of a particular extreme base shear value BS_e in 50 yrs lifetime of the support structure as determined from the analysis of the first 5, 10, 15 and 19.3 years of the storm database.

Return period	*years*	*10*	*50*	*100*
'Conventional' design base shear	kN	2850	3490	3680
Median value of the 'reliability based' design base shear using:				
361 storms	kN	1710	1970	2080
237 storms	kN	1635	1870	1975
153 storms	kN	1635	1870	1970
79 storms	kN	1675	1940	2055

Table 5-7 Comparison of the design base shear as obtained using a conventional design approach and a reliability based design approach based on the analysis of the first 5, 10, 15 or 19.3 years of the storm database.

The results in table 5-7 illustrate that there is only a limited sensitivity to different lengths of analysed storm data. All values are significantly smaller than the results from a conventional approach. To further evaluate the sensitivity of the results, the probability of failure given a particular reference strength of the structure can be investigated. However, one should be careful in using the results from figure 5-10 as estimates of the probability of failure since the horizontal axis is a semi logarithmic scale. Taking a reference strength of e.g. 3500 kN, depending on the length of the analysed database can give differences up to a factor of 4 in the probability of failure. However, for reliability considerations this is not too troublesome as one is more concerned about order of magnitude differences.

From these results it can however be concluded that the long term extreme response in a chosen lifetime is mostly influenced by the quality of fit to the storm severity values. Therefore in the following sections more attention is given to the fitting of the storm severities.

5.3.2 Type of extreme value distribution

A fundamental choice when doing extreme value analysis is the question of what type(s) of distribution to choose - for example: Weibull (2- or 3-parameter), Gumbel, exponential and so on. The tendency is to use a 3-parameter Weibull for both the environmental parameters (wave height, current speed, wind speed) and for loads. The 3-parameter Weibull form has enough flexibility to capture a wide range of relationships in the data.

In the computations presented in this report the 3-parameter Weibull distribution has been used to fit the median extreme response values given a storm. Next the results of the reliability based design method are re-determined when a different type of extreme value distribution is fitted to the storm severities. In this section results will be presented when either a Gumbel distribution (Eqn. 5-2) or a GEV distribution (Eqn. 5-3) is used to fit the median extreme response values obtained from the analysis of 361 storms in 19.3 years.

Gumbel: $$F_{e,fitted}\left(r_{e,med}\right) = e^{-e^{\left(-\frac{r_{e,med}-m_{Gumbel}}{a_{Gumbel}}\right)}} \qquad (5\text{-}2)$$

where:

$F_{e,fitted}$ - fitted cumulative probability distribution
m_{Gumbel} - shift parameter fitted value: m_{Gumbel} = 690.5 kN
a_{Gumbel} - scale parameter fitted value: a_{Gumbel} = 160.4 kN

GEV: $$F_{e,fitted}\left(r_{e,med}\right) = e^{-\left(1+k_{GEV}\frac{r_{e,med}-m_{GEV}}{a_{GEV}}\right)^{-\frac{1}{k_{GEV}}}} \qquad (5\text{-}3)$$

where:

$F_{e,fitted}$ - fitted cumulative probability distribution
m_{GEV} - shift parameter fitted value: m_{GEV} = 664.4 kN
a_{GEV} - scale parameter fitted value: a_{GEV} = 149.2 kN
k_{GEV} - shape parameter fitted value: k_{GEV} = 0.196

Storm severity distribution

Figure 5-11 shows the empirical complementary storm severity distribution on the basis of 361 values in the same form as figure 5-5 together with the fitted Weibull, Gumbel and GEV distributions.

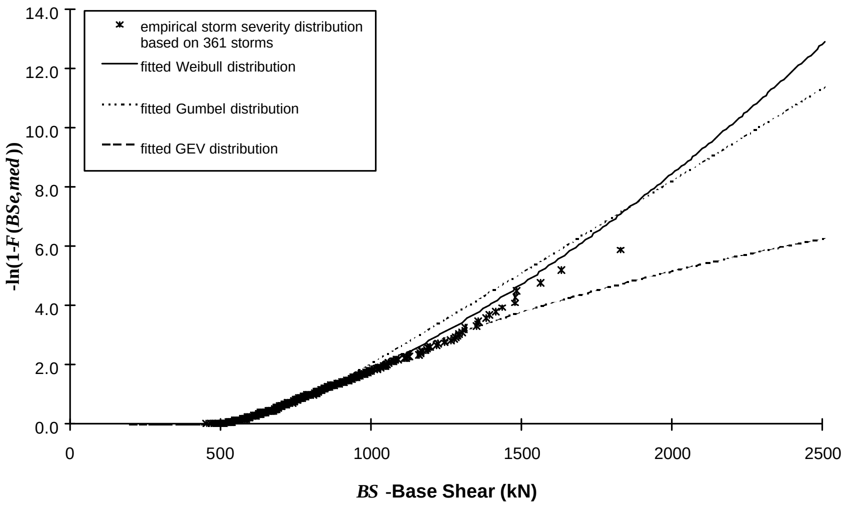

Figure 5-11 The empirical and fitted Weibull, Gumbel and GEV complementary distributions of median extreme base shear values in a storm plotted on semi-natural logarithmic scales based on 361 storms.

As can be seen in the figure the fitted distributions are distinctly different in their upper tails. Where the fitted Gumbel and Weibull distributions show a similar trend, the GEV distribution returns larger probabilities of exceedence for large values of base shear. Since the generic distribution of normalised extreme response remains the same it is to be expected that the long term extreme response distributions will also be different.

In comparison with the results obtained using the Weibull distribution as presented in section 5.2, it is expected that the results obtained using especially the GEV distribution will be more conservative. Application of the Gumbel distribution will only give slightly higher estimates of extreme base shear. This is illustrated in figure 5-12 where the upper part of the complementary long term extreme response distribution for a design life of 50 years is plotted on a semi logarithmic scale for the three types of storm severity distributions.

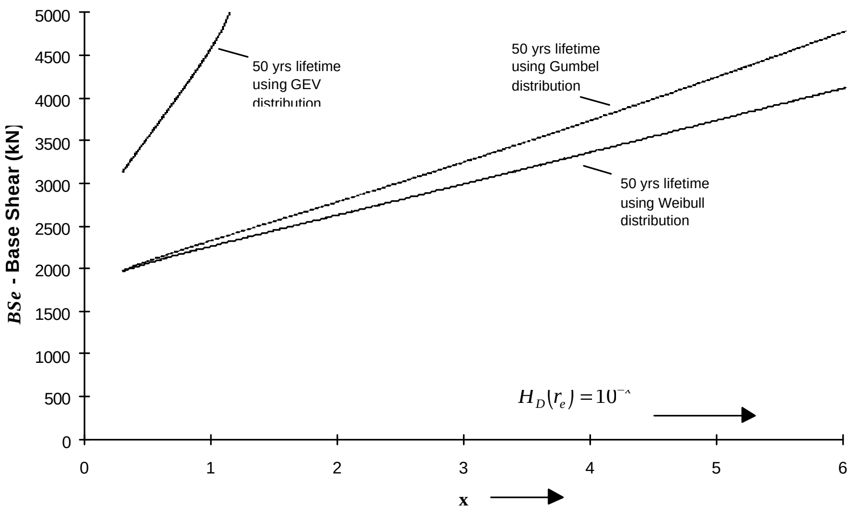

Figure 5-12 The probability of exceedence of a particular extreme base shear value BS_e in 50 yrs lifetime of the support structure as determined using a

Weibull, Gumbel and a GEV distribution for the fitting of 361 storm severity values.

Figures 5-11 and 5-12 further confirm our previous observations that the quality of fit to the upper tail of the storm severity distribution is a critical step in the application of the reliability method. Therefore, the next section will look at the results when aiming at a better fit to the upper tail.

5.3.3 Fitting the upper tail of the storm severity distribution

In this section results are presented when in particular the upper part of the storm severity distribution is used for the fitting of an extreme value distribution. The results in this section have again been obtained using a 3-parameter Weibull. Obviously, the fitting should be done with great care as one does not know a priori which upper part needs to be considered to get a satisfactory and stable fit. Therefore, the percentage of top values from the empirical storm severities that is used in the fitting is varied from 10 % to 100% (all the values) with steps of 10 %. As a result 10 Weibull fittings are obtained which are all applied in the reliability based design method. Figure 5-13 shows these fitted distributions together with the empirical distribution of median extreme response values based on 361 storms. Note that for reasons of clarity in the figure only results for fits to the top 30%, 60% and 100% of all the values are plotted.

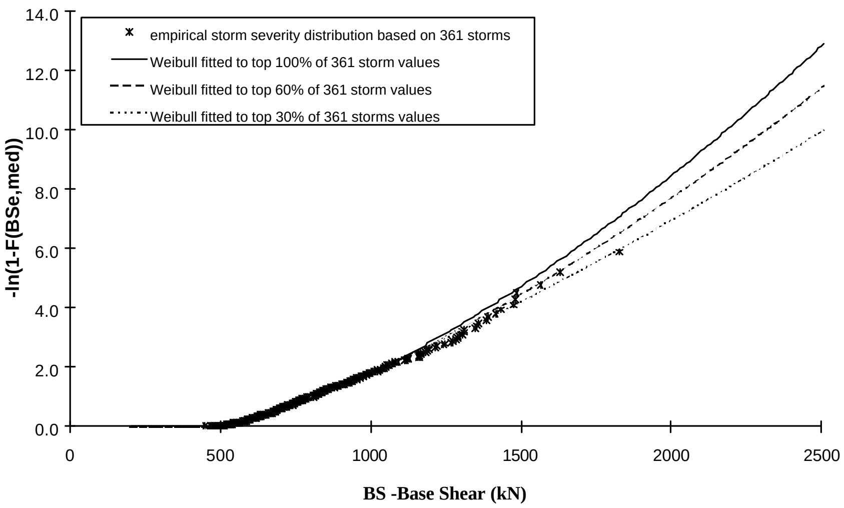

Figure 5-13 The empirical and the fitted Weibull complementary distributions of median extreme base shear values in a storm plotted on semi-natural logarithmic scales based on 361 storms and using the top 30%, 60% and 100% of all the values.

Looking at the figure one could argue that the upper tail of the empirical distribution is better fitted when only the top 30% of the data is used in the fitting procedure. Here this fit is also the most conservative one. It is thus expected that when applied in the reliability based design method the long term extreme base shear in a lifetime is also estimated conservatively using this distribution.

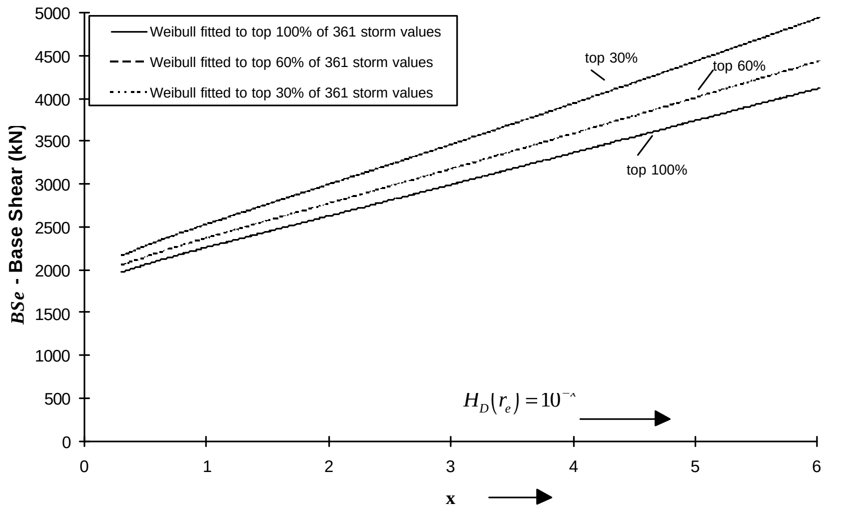

Figure 5-14 The probability of exceedence of a particular extreme base shear value BS_e in 50 yrs lifetime of the support structure as determined using a Weibull fitted to the top 30%, 60% and 100% of the 361 storm severity values.

Figure 5-14 shows the results for the probability of exceedence in a lifetime of 50 yrs as a function of the fitting procedure. It is again seen that the probabilities are influenced by the way the empirical storm severity distribution is fitted. Given a particular value of mean ultimate strength of the support structure (e.g. 3000 kN) the probability of failure can differ by one order of magnitude. The question thus remains which upper part of the storm severity distribution should be used in the fitting procedure. To increase the capabilities for the engineer to make this judgement the median extreme response value in a chosen lifetime can be plotted as a function of the percentage of top storm severity data used. This is illustrated in figure 5-15.

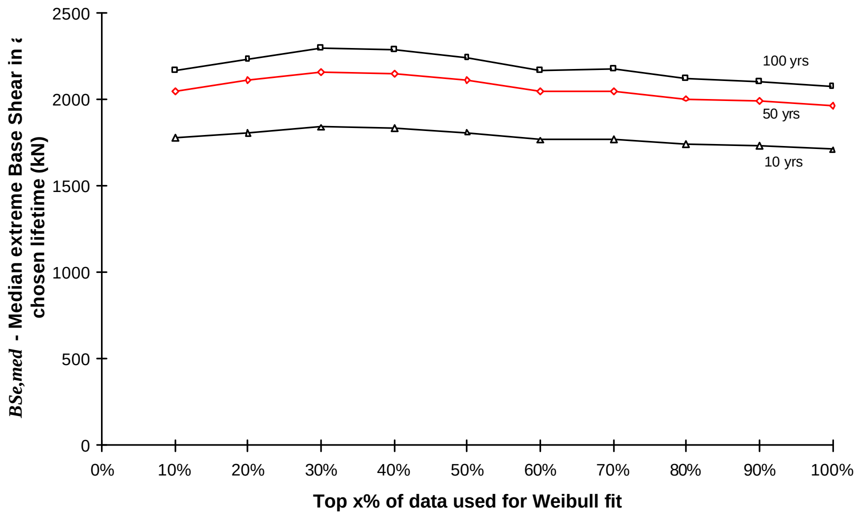

Figure 5-15 The median extreme base shear in a chosen lifetime of 10, 50 and 100 years as a function of the percentage of the top storm severity values used in the fitting procedure.

From the figure it can be seen that reasonably stable estimates for the extreme base shear in a specific lifetime are obtained. For a safe application of the reliability based design method for this specific structure at its specific site location it would hence seem prudent to fit on the basis of the top 30% - 40% of storm severity values.

In figure 5-16 the influence of the fitting procedure on the probability of failure of the structure for a series of assumed structural strengths are presented. For large values of ultimate strength of the structure, e.g. 5000 kN, the difference in estimated probability of failure using different fitting procedures can increase up to a factor of 200 (O^2) which is quite large. For smaller reference strength values the differences are also smaller, e.g. a factor of 20 (O^1) at a mean ultimate strength of 3500 kN.

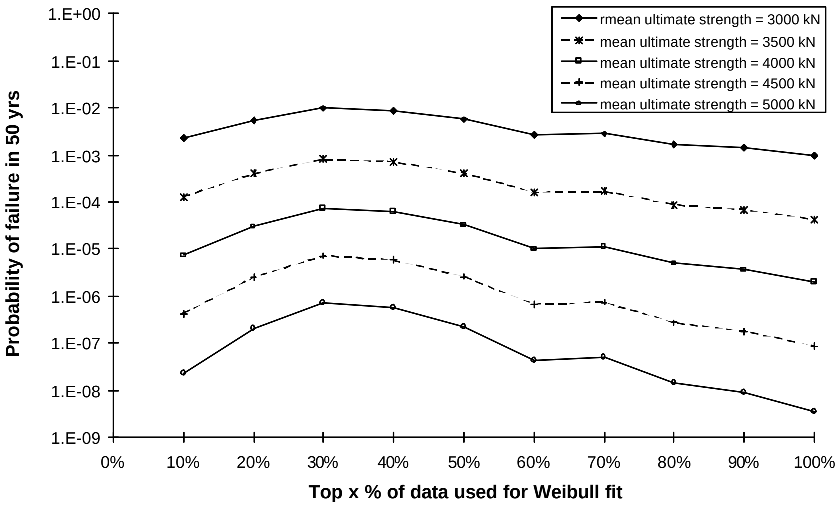

Figure 5-16 The probability of failure for a series of assumed structural strengths of the support structure in 50 years lifetime using a Weibull distribution for the fitting of the upper tail of the storm severity distribution.

It should be reminded that the results presented here are all obtained when using a Weibull distribution to fit the upper tail of the storm severity distribution. We could have repeated the analysis with a different extreme value distribution, e.g. Gumbel or GEV. It is however, expected that when increasingly focusing on the upper tail of the storm severity distribution, the hence obtained fitted distribution will be in better resemblance with the fitted Weibull distribution as shown in figure 5-13. Consequently, the estimated probability of failure in a chosen lifetime will be less influenced by the selected type of the extreme value distribution.

5.3.4 Discussion

The results of the application of the reliability based design method to the support structure of an OWEC at a specific location demonstrate that a conventional approach gives considerably more conservative estimates for the design loads than the reliability based design method. This conservatism is due to the neglect of the correlation of the environmental parameters at any instant in a conventional approach. The exact degree of conservatism is, however, difficult to determine - if possible at all. The reason being that the results of the reliability based design method are sensitive to the fitting of a distribution to the median extreme response values in a storm, which is necessary for the purpose of extrapolation. Superficially, it seems impossible to estimate storm severities with very long return periods (e.g. 100, 1000 years) on the basis of a limited amount of data (e.g 19.3 years). One cynical view of this would be that it involves extrapolation into the realms of fantasy - an impossible problem. Or quoting from

Tucker [6-1]: “The author considers extreme value theory to be probably the most difficult and esoteric subject in the applied oceanographer’s field”.

However, despite these observations, design conditions must be determined for all offshore developments. Consequently, extrapolation into the unknown is unavoidable for both the conventional approach and for the reliability based design method. From the sensitivity analyses presented in the previous sections it is clear that by investigating several influences a good feel can be obtained for how the final results should be interpreted. It is therefore recommended that in the application of the reliability based design method, but also in conventional practice, the following influences are investigated:

- the type of extreme value distribution that is fitted to the storm severities;
- the way the extreme value distribution is fitted to the storm severities.

On the basis of these investigations it is believed that the design engineer obtains relevant information which is of great importance in making intelligent and safe design decisions.

6. Conclusions and recommendations

6.1 Conclusions

- It has been demonstrated that conventional design practice overestimates the design environmental load in a chosen lifetime by a large margin.
- The overestimate is mainly caused by the neglect of correlation between the environmental parameters at any one time.
- The analysis of the database indicates that the current is not correlated with either wind or waves. This in particular is the main reason of the overestimate in design load using a conventional approach.
- The degree of overestimation is difficult to determine as both the conventional design practice and the reliability based design method involve extrapolation to periods much longer than the available databases.
- The results of the reliability based design method are sensitive to the selection and the fitting of a theoretical distribution to the empirical median extreme response values; such a distribution is necessary for the extrapolation to durations beyond the duration of the environmental database. In the application of both the conventional design approach and the reliability based design method careful attention should always be given to the extrapolation of extreme values.
- For the structural configuration at the exposed and relatively deep water North Sea location considered in this study hydrodynamic loading dominates structural design. This justifies the use of a simplistic treatment of wind loads in the form of spatially distributed wind shear, but time independent aerodynamic loading in this work.
- Constrained random time domain simulations can be efficiently used to determine the short term distribution of the extreme response in a sea state with sufficient accuracy. The reduction in simulation time compared to fully random simulations depends on the severity of the sea state but is of the order of 98% for the given problem.

The short term distribution of the extreme response in a sea state is the basic building block for the determination of the long term distribution of the extreme response in a given lifetime. This distribution makes it possible to apply a reliability based design method. Using such a procedure significant savings in the support structure are possible, provided that the design of the support structure is governed by considerations of extreme strength.

6.2 Recommendations for further work

- Apply the reliability based design method also to overturning moment as a global response parameter, as well as to a number of local response variables, e.g. stress in a member.
- Try to formulate a Generic Response Model in analogy with a Generic Load Model to transform the environmental conditions directly into the distribution of the extreme response given a particular sea state.

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M. Kühn

Institute for Wind Energy, Delft University of Technology

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