RECOMMENDATION FOR

TECHNICAL APPROVAL OF OFFSHORE WIND TURBINES

DECEMBER 2001

The Danish Energy Agency's Approval Scheme for Wind Turbines

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1. INTRODUCTION

This recommendation (hereafter referred to as the Recommendation) is an annex to "Technical Criteria for Type Approval and Certification of Wind Turbines in Denmark" and contains instructions and supplementary information about technical requirements for approval of offshore wind turbines.

The Recommendation has been prepared by a working group, set up by the "Advisory Committee for Approval of Wind Turbines in Denmark" in December 1999, under the auspices of the Danish Energy Agency. The working group assessed the need for detailed instructions in relation to the Danish Approval Scheme and has subsequently prepared the present Recommendation, which constitutes an update of the previous edition of june 2001 (only in Danish).
The present English version is a translation of the original Danish edition of December 2001. The latter is the legally valid recommendation – in case of any differences.

Text with small font shall be read as guidelines. Annexes serve as guidelines only.

The Recommendation is largely based on results from the research project: "Design grundlag for vindmølleparker på havet" ("Design Basis for Offshore Wind Turbines"), EFP-1363/99-0007, which the project management has kindly put at the Committee's disposal.

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1.1 Application

In the Recommendation efforts have been made to give an account of technical criteria for offshore wind turbines which are sufficient for:

- conceptual and detailed design of foundations
- design/adjustment of wind turbines

To some extent the following subjects have also been dealt with:

- access and working conditions during erection and operation
- materials and corrosion

1.2 Preconditions and references to codes of practice

When designing wind turbines for offshore siting and operation, the level of safety shall correspond to the level which has so far been maintained by Danish wind turbine manufacturers in terms of erection and operation of onshore wind turbines in Denmark. This level is attainable by complying with the codes of practice specified in the following:

Erection and grid connection of wind turbines in Denmark, both onshore and in Danish waters, require that wind turbines have a Danish type approval. This type approval is based on:

- *Technical Criteria for Type Approval and Certification of Wind Turbines in Denmark (TC), supplemented by the present and other valid recommendations under TC*
- *DS 472, Last og sikkerhed for vindmøller (load and safety for wind turbines)*
- *Additional Danish and foreign standards and codes as listed in TC*

For wave loads general reference is made to:

- DS 449 “Pælefunderede offshore konstruktioner” (“Pile-supported offshore steel structures”)

DS 449 has not been updated together with the other Danish codes of practice, however, guidance in DS 449 is still applicable. Application of partial safety factors in DS 449 in conjunction with the new construction codes is not allowed.

For ice loads general reference is made to:

- API Recommended practice 2N, 2nd ed. (1995) “Recommended practice for planning, designing and constructing structures and pipelines for arctic conditions”.

Partial safety factors, etc. in DS 472 have not been revised together with the Danish construction codes (DS 409, 2nd ed.: 1998, DS 410, 4th ed.: 1998, DS 411, 4th ed.: 1999, DS 412, 3rd ed.: 1998, DS 413, 5th ed.: 1998, DS 414, 5th ed.: 1998, DS 415, 4th ed.: 1998), and an addendum to DS 472 is therefore prepared.

If there is a wish for applying other codes or methods an account of how the same level of safety has been obtained, i.e. specified in the above mentioned Danish codes, is required.

In connection with the choice of a particular site or the environmental impact assessment (EIA), it is normally assumed that a risk assessment has been made containing, inter alia, a quantification of the risk of collision with third party vessels with a differentiation of the types of vessels and expected corresponding ship impact energies.

1.3 Definitions

Cf. DS 472.
However, v_b and $v_{b,0}$, cf. DS 410, 4th ed., 1998.
V_{eN}, (only in Annex D), cf. IEC 61400-1, 2nd edition.

2. CLIMATIC PARAMETERS AND SAFETY IN RELATION TO DS 472

2.1 Addendum to DS 472

2.2 Information about the addendum to DS 472, can be found at www.vindmoellegodkendelse.dk. Corrections to DS 472 with addendum

This section contains modifications applicable to offshore conditions.

2.2.1 Annual mean wind speeds

Wind conditions	**Parameter**
Annual mean wind speed The stated annual mean wind speeds are applicable to structural calculations only.	50 m height. To be extrapolated according to DS 472 where $z_0 = 0.001$ m. The North Sea: 10.0 m/s The interior Danish waters: 8.5 m/s *Or calculation according to relevant documentation*

2.2.2 Safety level and integrated safety

Structural safety: As the turbines shall be designed in accordance with current Danish codes, the design shall aim at ensuring that the same level of safety is obtained as is otherwise applicable to onshore wind turbines in Denmark.

2.2.3 Partial safety factors

Load conditions are defined, and load combinations and preconditions are examined in section 6.2, DS 472.

In accordance with previous practice and DS 415, the partial safety factor $\gamma = 1.0$ is used for the weight of parts of the structure and for the weight of soil and groundwater, respectively, as these are conservatively estimated (or are documented on the basis of measurements). This applies to all load cases. When filling materials are used in closed spaces, the weight should be estimated (extra) conservatively. Similarly, when filling materials are used in open spaces, possibly subjected to scour protection, the weight estimation should be particularly cautious. Upon accept of damage on a particular scour protection, the weight of the scour protection should be severely reduced.

External load conditions are examined in conjunction with wind loads. In case of other external load conditions than wind loads, the partial safety factor can be determined by the relevant coefficients of variation on the annual extreme, cf. section 3.4.3.

2.2.4 Simplified formula for turbulence intensity in farms

If the distance to the closest situated neighbouring turbine is at least 5 rotor diameters, the following simplified formula for turbulence intensity inside the farm can be applied:

$$I_T = \sqrt{0.15^2 + I_0^2} \ ,$$

where I_0 is the turbulence in the ambient flow.

3. LOADS AND LOAD CASES

3.1 Calculation method

3.1.1 Scope of the dynamic structure

General observations:
"The wind turbine system" comprises the following components: rotor, nacelle, tower, mechanical and electric transmission, operating and safety systems as well as foundation plus underlying/surrounding soil. Depending on the particular stiffness of the system the following methods are applicable to structural calculations.

Method 1
Unless it can be demonstrated that the foundation structure plus underlying/surrounding soil is "sufficiently" stiff[1], the wind turbine system (as defined above) shall be considered as a unity. Structural calculations are, consequently, made for the system as a whole.

Method 2
If the foundation structure plus underlying/surrounding soil is "sufficiently" stiff[1] and a well-defined horizontal cut between tower and foundation of the turbine has been established, structural calculations can be divided into 1) a calculation of the structure from the horizontal cut and upwards, and 2) a calculation of the structure from the horizontal cut and downwards.
If there is a need for separate approvals (and as a consequence hereof separate calculations) of the wind turbine and foundation, a definition of the horizontal cut between tower and foundation is required. The horizontal cut can be defined at a level where part of the tower is calculated as forming part of the foundation structure. It is required that the horizontal cut is defined at a level which is above the level of the highest waterline. The level of the highest waterline shall for this purpose be calculated as a 50-year storm surge water level plus maximum wave crest in a corresponding 3-hour sea condition plus 1 meter in order to take various uncertainties into consideration.

In case of separate approvals of wind turbine and foundation, the wind turbine manufacturer shall document the resulting characteristic cutting forces, which are transferred from the wind turbine to the foundation in the horizontal cut, in a separate document.

Similarly, in a separate document the supplier of the foundation shall document equivalent foundation stiffness and damping conditions of all relevant load combinations for utilisation in the horizontal cut when undertaking load calculations for the wind turbine.

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[1] The expression "sufficiently stiff" signifies that the stiffness of the foundation is of such a nature as to allow its dynamics during loading to have no or only insignificant bearing on the dynamics of the turbine. If method 2 is applied, this shall be documented.

3.1.2 *Scope of simulations*

When calculating loads by means of simulation it is a well-known fact that the result is dependent on the ”seeds” on the basis of which the calculation is initiated. Consequently, the simulation must be repeated with varying seeds.

If time simulation is applied for determination of extreme and/or fatigue loads, assuming that the simulations consist of 10 minutes series, the number of simulations with varying seeds should at least amount to five per load case. In case longer or shorter time series are used, the number of simulations shall be adjusted accordingly.

It should be noted, however, that additional simulations may be required if it is not sufficient simply to apply the mean value of the extreme events. This is for instance the case when extreme events are to be established for the normal operation of the turbine, where the result of e.g. 10 minutes simulations shall be extrapolated to longer periods.

When undertaking fatigue calculations 1 seed per load interval can be applied, i.e. provided that seeds are changed during load intervals, and provided that approximately five load intervals have an equal impact on the result. Here, reference should also be made to Annex C5.

When time simulation is applied for determination of extreme loads, the characteristic response is defined as the mean value of the extreme events in the different time series.

3.2 Loads

Characteristic values are defined as the 98% quantile of the distribution of the annual extreme value for the load. This corresponds to the load with a 50-year recurrence period.

In certain design calculations loads with other recurrence periods shall be applied. If the loads, which correspond to these recurrence periods, have not been defined, the values in the below table can be applied, assuming that the distribution of the extreme load corresponds to a Gumbel distribution. The T-year load is highly dependent on the coefficient of variation (COV) of the load, which must therefore be estimated. In DS 410 an assumption of a COV=0.23 on extreme wind load for T < 50 years and a COV = 0.40 for T > 50 years is made.

COV T *[year]*	0.05	0.10	0.15	0.20	**0.23**	0.25	0.30	0.35	0.40	0.45	0.50	0.60	0.70
1	0.865	0.758	0.671	0.599	**0.561**	0.538	0.486	0.441	0.402	0.368	0.337	0.285	0.243
5	0.921	0.858	0.806	0.764	**0.742**	0.728	0.697	0.671	0.648	0.628	0.610	0.579	0.554
10	0.945	0.900	0.865	0.835	**0.819**	0.810	0.789	0.770	0.754	0.740	0.727	0.706	0.689
20	0.968	0.943	0.923	0.906	**0.897**	0.892	0.880	0.869	0.860	0.852	0.845	0.833	0.823
25	0.976	0.957	0.942	0.929	**0.922**	0.918	0.909	0.901	0.894	0.888	0.883	0.873	0.866
50	1.000	1.000	1.000	1.000	**1.000**	1.000	1.000	1.000	**1.000**	1.000	1.000	1.000	1.000
100	1.024	1.043	1.058	1.071	1.078	1.082	1.091	1.099	**1.106**	1.112	1.118	1.127	1.134
200	1.048	1.086	1.117	1.142	1.156	1.164	1.182	1.198	**1.212**	1.224	1.235	1.253	1.268
500	1.079	1.143	1.194	1.236	1.258	1.272	1.303	1.329	**1.352**	1.372	1.390	1.421	1.446
1000	1.103	1.185	1.252	1.307	1.336	1.354	1.394	1.428	**1.458**	1.484	1.508	1.547	1.580
10000	1.183	1.328	1.446	1.544	1.595	1.626	1.696	1.757	**1.810**	1.857	1.898	1.968	2.025

Table 1: The relation between the T-year load and 50-year load for different coefficients of variation of the annual extreme load distribution (for p=exp[-1/T]).

The relation between the *T*-year load and the 50-year load is shown graphically in Figure 1.

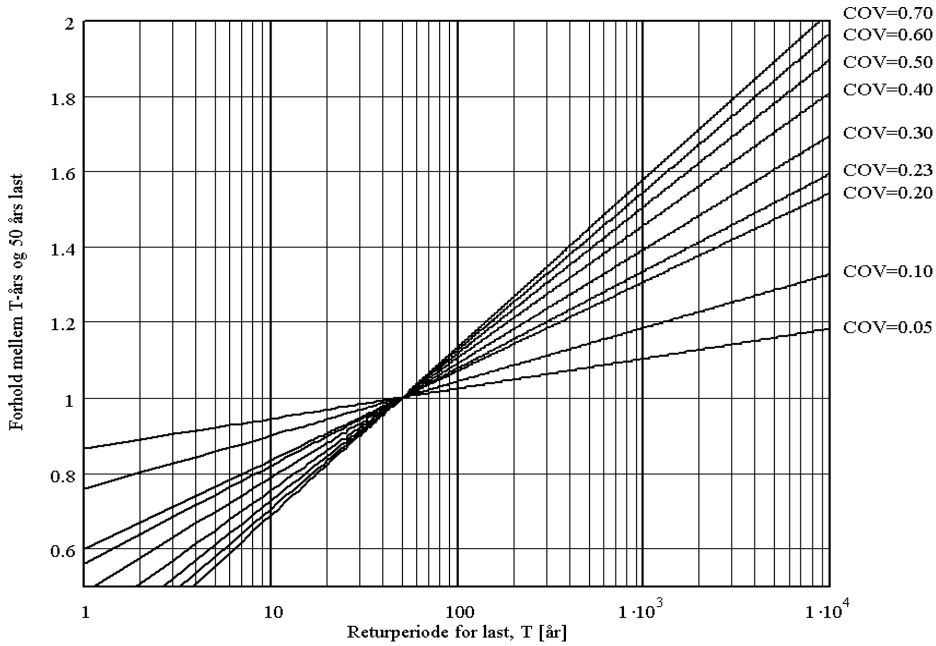


Figure 1: The relation between the T-year load and 50-year load (for p=exp[-1/T]).

3.3 Load cases

3.3.1 Wind

Annex A contains load cases corresponding to the cases which, as a minimum, shall be assessed under the Danish Approval Scheme. Apart from these load cases, an analysis shall be made of whether additional, more severe cases can be established for the reference wind turbine. In the affirmative, such cases shall also be defined and calculated.

Annex B contains load cases, which refer to IEC 61400-1. These cases are not of current interest and need not be calculated for a Danish approval. In connection with certification in other countries it may be required, however, that the load cases in Annex B are calculated.

3.3.2 Waves

Loads are determined in accordance with principles described in DS 449, which are applicable to deeper waters. In shallow waters, where most offshore turbines are sited, the following conditions become of paramount importance:

- finite wave heights
- wave crests are considerably higher than troughs (up till approx. 3 times the height of the trough rather than having the same magnitude as troughs)
- the crest only appears down to 1/3 of the wavelength (rather than approximately half the wavelength)
- velocities in breaking waves become considerably higher, especially at the crest (in the range of $u_{max} = \sqrt{gh}$, where h = the water depth)
- the wave profile becomes asymmetric lengthwise due to the fact that the steepness of the wave profile is greater towards the crest than after the crest
- the wave height distribution is changed (from the normally assumed Rayleigh distribution)

Illustrative figures are shown in Annex E.

These conditions necessitate that particular methods, i.e. comprising effects of shallow waters (incl. refraction and breaking) and diffraction, are required in order to determine both wave conditions and loads resulting from the waves.

Dimensioning of the structure for wave forces requires partly an analysis of extreme events, and partly a fatigue test of the structure. In case of plunging breaking waves local stability shall be examined separately.

3.3.2.1 The design wave condition

The extreme event, which is to be dimensioned, is characterised by the design wave height $H_{sd}= H_{s,XX}$, i.e. the significant wave height which has a recurrence period of XX years.

The corresponding maximum wave and probable wave period interval are determined.

Wave breaking
Dimensioning must be undertaken out of consideration to broken or breaking waves. In case of shallow water waves the corresponding maximum water level shall be dimensioned.

Top breaking
Solitary waves can be simulated according to the stream function wave theory or Stokes fifth-order theory, cf. /4/. Time series for shallow waters waves (but without plunging breaking) in the form of wave elevations and velocities can be simulated, cf. /5/. The following expression applies to the maximum particle velocity in a top breaking wave $u_{\max} \leq 1{,}0 \cdot \sqrt{gh}$

Plunging breaking
Special conditions for structures during their exposure to plunging breaking waves shall be examined. The limits for plunging breaking are determined, cf. /2/, via the relation ξ between the bottom slope tanβ and the square root of the wave steepness. The steepness of the wave is calculated on the basis of the deep water wave height H_0 or the breaking wave height H_b:

$$\xi_0 = \frac{\tan\beta}{\sqrt{H_0/L_0}}$$

$$\xi_b = \frac{\tan\beta}{\sqrt{H_b/L_0}}$$

Plunging breaking shall be calculated if either ξ_0 or ξ_b are represented in the following intervals

$0.5 < \xi_0 < 3.3$
$0.4 < \xi_b < 2.0$

Consideration must be given to the fact that the crest of breaking waves is considerably higher than the trough. The maximum particle velocity in the breaking wave is given by the expression $u_{max} = 1{,}25 \cdot \sqrt{gh}$, cf. /3/, and shall be applied as the velocity in a monotonous velocity profile for the entire wave above the still water level. Below the still water level a velocity profile is applied, cf. conventional wave theory. If the structure is considerably larger below the water surface than above the surface, this may cause plunging breaking, and a quantification of the effects hereof must be given.

Simulation

With regard to simulation of irregular waves please be referred to the references in 3.3.2.4 and annex to "Designgrundlag for vindmølleparker på havet" ("Design basis for offshore wind turbines"), EFP-1363/99-0007.

3.3.2.2 Wave forces

Dimensioning of wave forces shall be undertaken as described by:

a) Inertia forces F_i Function of the accelerations du/dt of the mass of water around the foundation of the wind turbine

b) Current forces F_d Function of the current velocity u (combined wave and current velocity)

c) Pressure forces F_t Function of the water surface elevation

Pressure forces (integrated over the area) are identical with inertia forces (acceleration integrated over the volume). If the effective volume of the structure in the water is large, i.e. in relation to the length over which there is a fairly constant acceleration in waves, this must be taken into account by calculating the ultimate pressure differences. In case of structures dominated by pressure /inertia forces, effects from the finite wave heights must be calculated. In case of calculations with combined waves and current, the stationary current is added to the orbital wave velocity by means of vector summation.

In shallow waters the correlation between water level and wave conditions shall be carefully assessed. Furthermore, when calculating local stability in shallow waters, shock forces from plunging breaking waves shall be added, if this load case is relevant.

The load determination shall be undertaken on the basis of methods, which result in the required level of certainty. The more impact the wave and current loads have on the wind loads, the more precise and reliable the applied methods must be. Simplified methods for load determination are given in /6/. The design and size of the structure in relation to the wavelength constitute crucial elements for determining whether the pressure gradients or velocities in the wave profile and wave and current forces can be calculated on the basis of a detailed wave and current simulation.

For structures where wave and current loads are crucial, load determination must, until the numeric methods are fully reliable, be based on model testing. Alternatively, conservative estimates for the wave and current loads can be applied.

Regular wave forces

Pursuant to DS 449 loads on foundations with a diameter of more than 0.2 wavelength *L* shall be calculated by means of diffraction theory. Due to the changed steepness conditions in shallow waters, diffraction effects for foundations with corresponding less diameters shall be taken into account (down to 0.13 wavelength in low waters instead of down to 0.2 wavelength in deeper waters). Cf. /4/. For foundations with a diameter of less than 0.13 wavelength, Morrison’s formula can be applied for determination of loads. However, effects from finite wave heights shall be calculated, unless it can be demonstrated that such effects are insignificant.

If wavelengths are long relative to the characteristic dimensions of the foundation, vortex shedding may occur which shall be included in the load basis.

3.3.2.3 Wave shock force

Wave shock pressure may occur even without plunging breaking. Dimensioning must be undertaken in accordance with DS 449 as the wave shock force is calculated as a triangular impulse growing from 0 to the maximum value in 0.01 sec. and thereafter decreasing from the maximum value to 0 in the course of 0.1 sec.

3.3.2.4 References

/1/ DS 449 Pælefunderede Offshore Stålkonstruktioner (DS 449 Pile-supported Offshore Steel Structures)

/2/ Fredsøe & Deigaard, 1992 ”Mechanics of Coastal Sediment Transport”, World Scientific

/3/ Svendsen, I.A., 1979 Bølgebrydning (Wave breaking), ISVA, DTU

/4/ Svendsen, I.A. og Justesen, P.,1984 ”Forces on slender cylinders from very high waves and spilling breakers”, Symp. Description and Modelling of Directional Seas, DHI, DTU

/5/ Madsen, P., Bingham, H. and Liu, H., 2000 ”The ultimate Boussinesq formulation for highly dispersive and highly nonlinear water waves”, ICCE 2000, Sydney, Australia

/6/ Lundgren. H., 1972 ”Bølgeproblemer i Oceanteknikken” (”Wave problems in Ocean Engineering”). ISVA, DTU

3.3.3 Current

3.3.3.1 Flow velocity components

The following flow contributions shall be taken into account:

- Tide generated current
- Barometrically generated current
- Current caused by wind surge, locally or in connection with large water regions
- Surface current generated by the wind shear force

If wind turbines are sited within a wave breaking zone on a coast, consideration shall also be given to the longshore current generated by the shear force of the breaking waves along the coast.

As a general parameter for describing the current, the surface current velocity *U(0)* shall be applied for all components.

3.3.3.2 Current profile

Flow contributions are established, cf. DS 449. Contributions from tide generated current, barometrically generated current, and current caused by storm surge are gathered in a flow velocity component. The distribution of this flow velocity component over the depth is determined on the basis of a power profile where the current velocity $Us(z)$, as a function of the height z above the water surface, is:

$$Us(z)=Us(0)\ (1+z/h)^{1/7}$$

h denotes the water depth.

The wind driven flow component U_V is calculated according to DS 449, decreasing linearly down to 20 m below the mean water surface:

$$U_V(z)= \quad U_V(0)\ (1+z/20)$$

At depths of less than 20 m the current profile is cut off at the seabed. For determination of possible scouring at the seabed, the wind induced surface current shall be included in the power-current profile with the surface velocity $U_V(0)$ for calculation of current velocities at the seabed.

3.3.3.3 Calculation of current forces

Current loads shall be calculated, cf. DS 449

When combining waves and current, the stationary current shall be added by means of vector summation to the wave generated current velocities.

Vortex shedding is examined in accordance with DS 449 B 2.2.

3.3.3.4 References

/1/ DS 449 Pælefunderede Offshore Stålkonstruktioner (Pile-supported Offshore Steel Structures)

3.3.4 Water level

3.3.4.1 Water level

Determining water levels shall be established. The determination shall describe both tide conditions and storm surge.

Likewise, for determination of ice loads the relevant determining water level shall be established. This is of particular importance when the structure is designed with ice force reducing sloping surfaces.

The load impact of the water level shall be taken into consideration in case of buoyancy on the structure and for determination of wave and current loads.

Splash zone

A splash zone shall be determined, i.e. usually defined between the normally occurring high water and corresponding significant wave crest height, and the normally occurring low water with corresponding significant wave trough height.

Normally occurring water level can e.g. be defined as high/low water with a recurrence period of at least 3 hours/year. Due to reflection the significant wave crest/trough can be estimated in the following way: A distance above or below water level on significant wave height. The height of the splash zone can possibly be limited to the top of a possible platform, thereby allowing parts of the structure, which are withdrawn considerably from the edge of the platform to be exempted from the splash zone.

3.3.5 Scour

3.3.5.1 Scour

The foundation of the wind turbine shall be dimensioned with particular consideration to the maximum possible scour of the seabed around the foundation. This includes an analysis of the climatic, seasonal and interannual changes in the level of the seabed.

The maximum water particle velocities, incl. current velocities on the seabed, are used as the basis for the computation. The reinforcement of the resulting bed shear force caused by the foundation is determined on the basis of the KC-figure (with and without current) and the relation between the characteristic dimensions of the foundation and the wavelength. Cf. DS 449.
It may be necessary to carry out tests for determination of reinforcement on bed shear stress and stability conditions for the chosen scour protection. Allowable damage is determined dependent on the estimated consequences. The scour protection can, for instance, function as a stabilising element.

The risk of scour outside the scour protected area shall be taken into account.

3.3.5.2 References

/1/ Sumer.B.M. and Fredsøe, J., 2000 “Wave scour around structures”. Advances in Coastal and Ocean Engng., Vol. 4.
/2/ Sumer.B.M. and Fredsøe, J. 1997 “Scour around a large vertical circular cylinder in waves”. OMAE 1997, Vol. 1A, ASME

3.3.6 Ice

The load determination shall be undertaken on the basis of methods, which result in the demanded certainty. Reference is generally made to /5/. The more impact the ice load has in relation to the wind loads, the more precise and reliable the applied methods must be.
Until more experience has been gained within this particular field, it is recommended that load determination is based on model testing with artificial ice. If the structure is flexible in relation to the definition for method 1in chapter 3.1.1, the tests should also encompass a model where elastic conditions are included.

Existing methods are primarily based on ice loads from floating floes in interior Danish waters dominated by current. When dimensioning foundations in more narrow waters, the basis for the dimensioning and methods must be reassessed.

3.3.6.1 Ice parameters

The characteristic ice load is determined on the basis of freezing degree-days (K_{max}) by the following site dependent parameters.

- Compressive strength r_u,
- Bending strength r_f,
- Thickness *e*
- Floe size
- Operational velocity for floes

For interior Danish waters the following values are normally applied:

Annual risk of deviation	0.2	0.1	0.02	0.01	8 x 10^{-4}	10^{-4}
Recurrence period	5-year	10-year	50-year	100-year	1250-year	10.000-year
K_{max} (-°C 24 h.)	170	245	410	480	744	960
r_u (Mpa)	1.0	1.5	1.9	2.0	2.4	2.6
r_f (Mpa)	0.25	0.39	0.50	0.53	0.64	0.69
e (m)	0.33	0.42	0.57	0.63	0.80	0.91

Upon proper documentation of insignificant or none characteristic values for ice thickness in the North Sea, the ice load can be ignored as load case.

In addition, the following parameters and general values are given:

Density, ice, ρ_i	900 kg/m^3
Gravity, ice, γ_i	8.84 kN/m^3
Modulus of elasticity, E	2 GPa
Poisson's condition, ν	0.33
Ice-ice coefficient of friction, μ	0.1
Ice-concrete dynamic coefficient of friction, μ	0.2
Is-steel dynamic coefficient of friction, μ	0.1

The attack height of the ice load is dependent on the particular water level variations, which are established on the basis of water level statistics for months with ice and possible sloping surfaces on the foundation.

3.3.6.2 Static ice load

Dimensioning shall be undertaken for horizontal and vertical static ice loads. Loads from ice on horizontal structures are calculated according to DS 410, /1/, i.e. by using the stated parameters in section 6.3.
For structures with sloping parts the ice load is calculated on the basis of Ralstons's formulas, /2/, if the ice attacks the sloping parts, and if from the top side or under side of the ice there is at least 0.5 m to the transition from the sloping parts to the vertical parts. Structures with ice force reducing cone are typical examples of structures with sloping parts.

Dimensioning of local ice pressure, r_{local}, shall be denoted by the expression, /3/:

$$r_{local} = \left[5\left(\frac{e^2}{A_{local}}\right) + 1\right]^{0.5} r_u$$

where r_u is the characteristic compression strength of the ice, e is the thickness of the ice, and A_{local} the area above which the local ice pressure appears. The local ice pressure cannot exceed 20 MPa.

An upper limit may exist for the ice load due to the possible size of the ice floes, current and wind in the area as well as the kinetic energy of the ice floes.

Load from possible pile-up in front of the foundations shall be assessed.

3.3.6.3 Dynamic ice load

The dynamic behaviour of the ice shall be taken into account. As regards foundations in areas dominated by current, it is normally the dynamic ice load, which is dominating when wind and ice loads are combined. The method from /4/ can be applied for estimation of the ice loads.

3.3.6.4 References

/1/ DS 410 Norm for last på konstruktioner, Dansk Standard, 4. udgave, 1998 (DS 410 Norm for loads on structures, Danish Standard, 4th edition, 1998)
/2/ Progress Report 66, ISVA, DTU, 1988
/3/ The Øresund Link: ”Ice Loads”, 1995
/4/ “Granskningsnote til design basis for iskræfter, Middelgrunden”, dateret 1999-11-30 (“Assessment note reg. design basis for ice forces, the Middelground“, dated 1999-11-30)
/5/ “API Recommended practice 2N, 2nd ed., 1995”. Recommended practice for planning, designing and constructing structures and pipeline for arctic conditions

3.3.7 Icing

Icing of the turbine structure is caused by e.g. spray or atmospheric icing. Most often, spray causes icing of the lower sections of the turbine structure, whereas atmospheric icing influences surfaces on the entire structure.

In case of atmospheric icing the turbine shall be examined for extreme loads during normal operation, where icing must be expected up to the cut out wind speed. A simple icing model is applied as indicated in the DIBT-richtlinien /1/. The turbine shall be examined in situations, where:

a) All rotor blades are iced
b) All rotor blades, except one, have been iced

Furthermore, icing shall be included in the fatigue analysis on the basis of the guidelines in the German DIBT-richtlinien /1/. The duration of the icing event shall be set at 7 days per year at a minimum.

In case of a parked turbine calculations shall be based on a 30 mm thick icing on all turbine components. The density of ice can be calculated as 900 kg/m^3. In the North Sea, the thickness of the icing shall be increased to 150 mm on components at levels up to +20.0 as a result of spray. For wind farms in the interior Danish waters, icing at levels up to +20.0 can be set at100 mm.

Alternative methods for icing analysis can be applied, e.g. the WECO-project /2/.

3.3.7.1 References

/1/ Richtlinie: *Windkraftanlagen Einwirkungen und Standsicherheitsnachweise für Turm und Gründung* (Fassung Juni 1993), Deusche Institut für Bautechnik (DIBT-richtlinie)

/2/ *Wind Energy Production in Cold Climate (WECO)*, EU-project (see http://www.fmi.fi/TUT/MET/energia)

3.3.8 Ship impact

When dimensioning offshore wind turbines, the following situations shall be taken into account in connection with ship impact:

-Ultimate limit state:	Calling of characteristic service vessel with stem or stern at direct call (longitudinally) against appropriate fendering of the structure.
-Accidental limit state:	Unintended collision with floating vessel – large working vessel (crane barge or similar), alternatively, collision with an unauthorised vessel.

3.3.8.1 Design criteria

When dimensioning offshore wind turbines in connection with ship impact, the following design criteria shall be observed:

The ultimate limit state: Damage, which reduces the load carrying capacity of the turbine, must not be inferred on the main structure.

The accidental limit state: Normally, it is not possible to protect the main structure against damage. In connection with the siting of a particular offshore wind turbine farm, cf. section 1, it is assumed that a risk assessment of ship impact has been undertaken, incl. differentiation of types of vessels and corresponding assumed ship impact energies. The robustness of the structure is assessed in relation hereto. If it is feasible, within reasonable, practical and economic limits, to reinforce the structure, thereby reducing the risk of damage caused by ship impact significantly, such measures shall be taken.

3.3.9 Loads during erection

Criteria shall be defined for allowable external conditions during transport, erection and replacement. With point of departure in the applied working procedures and vessels, the following marginal values shall be stated:
- Wind
- Waves
- Water level
- Current
- Ice

Lifting fittings and procedures shall, in accordance with the stated external conditions, be of such a nature as to prevent damage on the structure. This shall be documented.

The strength of transport fittings, lifting fittings and additionally mounted equipment is not encompassed by the type approval, but will normally require a certification.

Reference is also made to existing codes and guidelines for sea transport and hoisting.

3.3.9.1 References

/1/ DS/R 461 Transport og installation af offshore konstruktioner (transport and installation of offshore structures)
2/ DNV (2000) Rules for planning and execution of marine operations

3.4 Simultaneous loads

3.4.1 Background

For determination of the response of the reference turbine structure to the time-dependent loads, dynamic calculation methods shall be applied. In case of non-linear behaviour particular conditions may exist which necessitate the use of other partial safety factors than the ones listed in DS 409 and DS 472 when undertaking calculations with more than one time-dependent load.

Dimensioning shall include an analysis of extreme response and fatigue. This is illustrated in the figure below. The external load *F(t)* is composed of a number of individual loads: Wind loads (DS 472 and the present document), loads from waves, current, tide and ice, $F_i(t)$. On the basis of the calculated series of load response, the largest response and load spectrum shall be calculated, respectively.

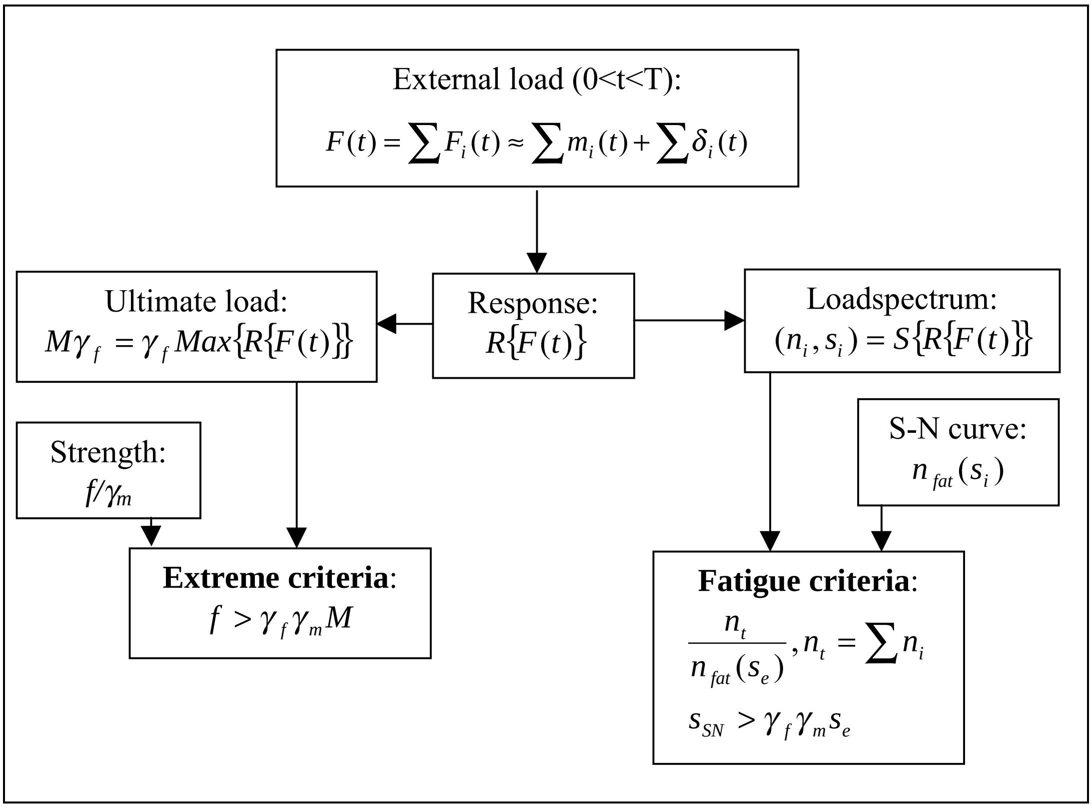


Illustration of dimensioning of specific section of the structure

When carrying out the extreme analysis, the given extreme response is compared with the capacity in such a way that certainty in the measurement is added by using partial safety factors on the response as well as on the characteristic strength of the material.

As an integral part of the fatigue calculations, the life time consumption of the load spectrum and the SN-curve of the material are calculated. On the same basis, a calculation of the equivalent voltage amplitude of the total number of cycles can be measured, if deemed to be necessary. To ensure certainty in the measurement, there is a partial safety factor on the strength of the material.

On both extreme- and fatigue-loads the partial safety factor is set at 1.2. cf. addendum to DS 472. In Danish approval the increase of the loads can be avoided however, as measurements verifying the size of the loads are required.

In practice, it is not allowed to calculate the response of the structure over its entire life time, time step after time step. Thus, a number of load cases are selected which in conjunction are assumed to result in the same level of certainty as the actual load event. For this purpose, the load is usually thought to be decomposed into a (current) mean value[2] with a pertubation around this. This decomposition is applicable to both time simulation and quasi-static observations.

[2] In the case of wind, for instance, this corresponds to the mean load of V_{10min}

3.4.2 General observations

The following shall be taken into account:

- Partial safety factors shall, as illustrated, be added *afte*r the response calculation. Partial safety factors for loads are found the addendum to DS 472.
- Time simulation shall be applied for the dimensioning.
- Normally, 10 minutes time series are applied, i.e. to the extent specified in section 3.1.2.
- Load cases with combined loads shall be chosen to ensure that the same level of certainty is obtained as would otherwise apply to a separate load.

It shall be examined whether the below mentioned loads are sufficient in each individual case. If necessary, additional load cases shall be added.

Here, attention is drawn to conditions which can change the dynamic properties of the wind turbine in the course of the estimated life time, such as corrosion, scour, altered geotechnical properties, etc.

The partial safety factors on the extreme wind cf. addendum to DS 472 provide sufficient certainty for the annual extreme loads with a coefficient of variation of 40%. If separate loads (as e.g. ice) have a higher coefficient of variation, the partial safety factor shall be increased. If it has been documented that separate loads, as e.g. inertia loads from shallow water waves, have a lower coefficient of variation, the partial safety factor can be reduced.

It shall be ensured that the risk of deviation during the life time of the structure is the same for all separate and combined loads.

When partial safety factors are added after response calculations, one partial safety factor can be applied only. If all external loads do not have the same coefficient of variation, and thereby the same partial safety factor, a choice must be made as to the specific partial safety factor. A co-weighing of partial safety factors from different external loads can with due consideration to the impact of the loads on a particular section of the structure and in a particular load case be undertaken in accordance with the following principle: The characteristic external loads are added individually and the response of the chosen section of the structure is calculated for each load. Hereafter, the individual responses are combined, thus constituting a characteristic response, and in case partial safety factors of external loads are used, a combined design response. The relation between the design response and the characteristic response constitutes the weighted partial safety factor. For determination of the weighted partial safety factor, the response calculations and combination can be undertaken on the basis of appropriate, simple models. See Annex C for guidance reg. a number of extreme load cases for mainly stiff foundations. If it is intended that the number of weighted partial safety factors shall be reduced, it is sufficient simply to choose one weighted partial safety factor for all sections of the structure and for all load cases, if it can be documented that this partial safety factor provides a conservative design.

In terms of fatigue calculations, the dependency on the direction for the response from wind turbine loads in the tower base should only be applied with cautiousness, and only where the response transverse to the wind direction has been demonstrated on the basis of load measurements. This is due to the fact that damping is normally very low parallel to the rotor disc and because the loads/response are very badly defined in this direction.

3.4.3 Correlated climatic conditions

Conditional distribution functions shall be established for the different climatic conditions, thereby allowing for determination of correlated values at a chosen probability level for wind speed, wind direction, wave height, water level, current conditions and ice.

Normally, one decisive external factor is chosen (e.g. wind speed for a particular direction or ice condition and corresponding wind load). Hereafter, conditional distributions and corresponding loads for the additional external conditions are determined.

As long as an overall description of statistics is not available, several probable determining combined load cases for the same external factor may appear, dependent on the chosen probability level. In these cases, additional scenarios must be assessed and the decisive scenario selected. Below is given an overview of such scenarios.

3.4.3.1 Wind and hydraulic loads

Extreme and fatigue loads

- Loads from wind, waves, current and tide (= hydraulic loads) are combined into one load which shall be calculated for simultaneous loading.
- Hydraulic loads shall be calculated for all extreme load cases as well as for all fatigue load cases, as indicated in Annex A.
- In connection with the simulation the significant wave height is applied, which corresponds to the particular wind speed, as a basic parameter, thereby assuming that the conditions are stationary.

Correlation between the wind directions where the wind load is largest, and the directions where the wave load is largest, does not necessarily exist. The wave load is normally dominated by the direction with the largest fetch.

3.4.3.2 Wind, ice and current loads

- Loads from wind, current and ice shall be calculated for simultaneous loading.
- The ice load can be dominated by the direction with the largest current velocity.

Extreme load

- Extreme dynamic ice load simultaneously with wind load corresponding to wind speed with 1-year recurrence period.

- Dynamic ice load simultaneously with maximum operational load from wind[3], cf. Annex C.

In waters dominated by current, it can normally be assumed that the extreme static total-ice load combined with wind load less than the extreme dynamic total-ice load combined with wind load, due to the static total-ice load only appears for a limited period of time when the ice is breaking.

Fatigue

It can be assumed that static ice load does not have any impact on the fatigue of materials. Furthermore, it is known that (dynamic) ice load only occurs a limited number of times in each 50-year period.

- The total period of time over the life time of the structure, where the ice gives cause to breaking and thereby dynamic load, shall be estimated.
- The life time consumption for this period shall be calculated with a load case with extreme dynamic ice load and usual loads for the normal operation of the turbine at a mean wind speed of 15 m/s.

3.4.4 ”Static check”

Due to the often non-linear nature of the turbine structure and the loads, standard quasi-static calculations will be less reliable. In case there is a need for verifying such calculations, the usual codes of practice are applied.

[3] E.g. wind load during operation at wind speeds of 20-25 m/s corresponding to a scenario where the ice load appears after the breaking of the ice following a hard ice winter in a rough wind situation in the months of February/March. Dependent on the control system of the reference turbine, other mean wind speeds may be relevant.

4. FOUNDATIONS

4.1 General observations

Existing codes of practice form the basis for the dimensioning of wind turbine foundations.

The primary basis shall be “Foundations (Recommendation to Comply with the Requirements in the Technical Criteria for the Danish Approval Scheme for Wind Turbines)”, the Danish Energy Agency 1998, where reference is made to e.g. DS 415 “Fundering” (“Foundation”) and DS 449 “Pælefunderede off-shore stålkonstruktioner” (“Pile-supported offshore steel structures”).

It should be noted that in accordance with the Danish Approval Scheme, foundation and turbine shall, even though they are calculated and designed separately, ultimately be calculated and approved as one unified system.

Regarding particular conditions for specific foundation concepts, cf. Annex F.

4.2 Geotechnical category and safety class

Determination of the geotechnical category of the structure follows the guidelines in DS 415 “Fundering” (“Foundation”).

For foundations/soil conditions where deformation properties of the soil exert a decisive influence on the eigenfrequencies of the structure, the foundation shall be referred to a geotechnical category 3. If deformation properties of the soil only have a limited impact, a normal geotechnical category is applied.

Determination of the safety class of the whole structure, or parts hereof, follows the guidelines in DS 409, “Sikkerhedsbestemmelse for konstruktioner” (“Safety provisions for structures”). Foundation and tower can usually be referred to a normal safety class.

4.3 Geotechnical investigation

The scope of required geotechnical investigations for the two types of geotechnical categories appears from DS 415 “Fundering” (“Foundation”) and DS 449 “Pælefunderede off-shore stålkonstruktioner” (“Pile-supported offshore steel structures”)

<u>Geotechnical/geophysical investigation programme</u>

The geotechnical /geophysical investigation programme shall be planned and implemented in such a way that the specific foundation concepts are taken into consideration.

Preliminary examinations can e.g. comprise of bathymetry, side-scan sonar and seismology. These analyses cannot stand alone, but shall be followed up by actual geotechnical investigations.

As a minimum, the final test programme for each specific site shall include a point measurement. The number of point measurements are determined on the basis of the actual geology, foundation concept, number of foundations (isolated wind turbines or turbine farm) and the result of possible preliminary geophysical investigations combined with a possible general knowledge of the geology of the specific site.

Point measurements can be undertaken as pure geotechnical drillings or as a precisely determined number of geotechnical drillings supplemented by CPT-tests. As a supplement to the geotechnical determination, “vibro cores” may be undertaken. In connection with the drilling operation, soil samples are taken as well as a specific number of A-tube samples of all main soil layers and particular soil layers. It cannot be assumed that “vibro cores” can be used as intact samples.

The test programme shall be planned and implemented in such a way that subsequent geotechnical inspections (and laboratory work) and calculations can be referred to a geotechnical class 3.

As guidance, the following scope of investigations should be expected:

Pile foundation: As a minimum, a point measurement should be made for each foundation. Upon erection of a large wind farm, point measurements can be combined with a number of short “vibro-core” drillings and, dependent on the results of the CPT-tests, a number of genuine geotechnical drillings, where SPT-tests and/or vane tests are undertaken together with samples for classification purposes and, possibly, laboratory tests. In case of a piled tripod foundation, bottom conditions which vary significantly may necessitate that 2 or 3 probings combined with a drilling for each foundation are carried out.

Direct foundation (and suction buckets): As a minimum a geotechnical drilling is undertaken for each foundation. The same scope of tests and samples is applicable as in the case of pile-supported foundations. In situations with large foundation diameters and/or varying bottom conditions, the drilling can be combined with a number of CPT-tests spread over the surface of the foundation

Laboratory tests and computation models

Static conditions: When correlating vane tests and laboratory tests (triaxial pressure and tensile tests and DSS-tests) it shall be established how the triaxial pressure and tensile strength, the vane strength and the strength from a DSS-test can be related.

Dynamic conditions: Dynamic effects in the areas/levels where the foundation forms part of soil calculations shall be examined, and resistance must be documented. The investigation shall be undertaken after the rupture zone (elastic cases). When dimensioning for cyclical/dynamic effects, soil parameters and computational method shall be selected in accordance with the chosen load history, the resulting stresses and deformations.

Deformations: The magnitude of soil deformations shall be calculated and analysed in both the plastic (due to permanent deformations) and elastic areas.

Particular attention should be paid to the application of high soil strengths. If high tensile stresses are calculated in soil, it shall be documented that large deformations do not occur which will cause deviation from the specified project requirements. Even though a mobilisation of parameters of deformation can normally be calculated during increased loading, it shall always be checked that ruptures do not occur in the soil with subsequent accelerating strain increase.

4.4 Check-up and supervision

Investigations shall be undertaken in accordance with DS 415, “Fundering” (“Foundation”) and DS449, “Pælefunderede offshore stålkonstruktioner” (“’Pile-suported offshore steel structures”) with the below mentioned modifications and additions.

4.4.1 Detailed inspection of bed topography

For foundation types which are sited on the seabed or on a crushed rock layer, and whose mode of operation is dependent on the bed topography, a detailed inspection shall be undertaken just before the installation of the foundation. The scope of the inspection is determined with due consideration to the completed preliminary investigations and the sensitivity of the structure to seabed conditions which deviate from the presupposed

A diving inspection or underwater video inspection of the seabed undertaken just before the installation will normally be sufficient.

4.4.2 Pile driving

The full course of the piling work shall be registered in a logbook according to the specifications in DS 449, “Pælefunderede stålkonstruktioner på havet” (“Pile-supported offshore steel structures”).

For single piles (vertical piles), the angle of the pile with verticality below the pile driving shall be registered regularly and entered into the logbook.

In situations where the vertical pile carrying capacity is not critical, registration of possible formation of soil plug can be omitted.

4.4.3 Scour

If an upper limit for scour on the seabed or a possible layer of scour protection around the foundation has been established during the projecting, it shall be examined regularly after the installation that this limit is not exceeded.

5. MATERIALS AND CORROSION

This section only applies to concrete and steel structures as well as to corresponding protection systems for foundations and towers until the lower edge of the nacelle, excluding blades, gear/generator and installations. Furthermore, in this section emphasis is given to conditions, which are of importance to the durability. The structures shall be corrosion protected in such a way that damage does not occur which in the expected life time of the structure may cause a lower level of safety than otherwise prescribed in relevant construction code(s). Foundation piles shall be included in the corrosion protection. When choosing materials for parts of the structure, welded joints, bolts, reinforcement and adhesion for the structure, it shall be ensured that alloys are not applied which will function as cathodes for the additional structure. The risk of corrosion will be present if metals with different standard electric potential (Volt) (relative to a standard copper electrode) come into contact.

The estimated design life time for the individual project is defined. In terms of the foundation and anchoring of the tower it is advantageous to choose a longer period than the design life time for the tower, which is accessible and easier to repair or replace than the foundation.

For a definition of the so-called "splash zone" concept mentioned in this section, please refer to 3.3.4.1.

Lightening protection shall be carried out in the system, cf. section 6.

5.1 Concrete structures and protection systems

The steel reinforcement in reinforced concrete structures shall be protected against corrosion. The best way to obtain this is by ensuring that there is: a sufficient concrete cover, a dense concrete structure, a limitation of crack formation and crack widths, and compliance with current rules for minimum reinforcement and reinforcement distribution.

It is recommended that concrete structures are divided into two environmental classes dependent on the geometric siting in relation to the splash zone:

In the splash zone:

- Minimum concrete cover is 50 mm
- Maximum calculated crack width is 0.1-0.2 mm

Outside the splash zone:

- Minimum concrete cover is 40 mm
- Maximum calculated crack width is 0.2-0.3 mm

Calculation of crack widths shall be based on the method given in /4/, section 6.3., and be undertaken for the most frequently occurring loads during normal operation. In connection with the assessment of the size of the crack width, contributions from contraction, creeping and temperature differences shall be added to the observed reinforcement stresses emanating from external loading.

At the same time, it is recommended that the composition of the concrete corresponds to environmental class E, high class supervision, cf. references /5/ and /6/, and that the following minimum requirements are fulfilled:

- The characteristic compression strength of the concrete, $f_{ck} > 40$ MPa
- Water/cement ratio for the concrete, $v/c < 0.40$
- Maximum aggregate size, $d_{max} < 32$mm or minimum distance between reinforcement bars
- Maximum distance between non-prestressed reinforcement bars is 150-200 mm.
- Application of reinforcement with relatively insignificant reinforcement diameters (D=12-20 mm) to the extent possible.
- Application of requirements for minimum reinforcement and distribution of reinforcement as recommended in /4/.

Cable ducts for prestressed reinforcement are injected with grout after mounting.

In general, the following precautions shall be taken during installation:

- Requirements for the composition of concrete shall be adjusted in such a way that it is possible to obtain concrete with a reasonable degree of workability, while at the same time ensuring that a sufficient degree of durability is obtained. Reinforcement arrangements, geometry, etc. shall be carried out in an appropriate manner. Application of special features such as curing membrane shall be included.
- Measures are taken which ensure that defects and damage do not occur on the concrete structures. In particular, it shall be ensured that effects from temperature and moisture do not damage the concrete structure. A thorough preparation and control of the casting process shall be ensured.
- Systems are established which ensure the durability of the structure despite defects and damage on the concrete structures.
- Assembly details at the transition between tower and foundation shall be designed with a gradient so as to minimise a pile-up of chlorides and moisture.

The intentions shall be incorporated, and the quality of the design shall be ensured, inter alia, by means of imposing stricter requirements with regard to the composition of the concrete in order to obtain a suitable workability, and with regard to the handling and protection of the concrete during the hardening process.

The risk of crack formation when casting parts of the structure shall be minimised. Normally greater differences in temperature than ΔT=12-15 °C measured over the cross section are not allowed

To ensure a good execution, a pretest should be undertaken of the concrete work. This shall be carried out, cf. /6/, section 9.4. Test concreting on a big scale is rather expensive and time-consuming, and the scope of these should therefore be proportional with the overall production and the calculated effect of the test concreting.

With regard to ensuring the desired life time, special measures shall be taken to remedy (possible) damage occurred in the design phase, cf. /6/, sections 9.10 and 11.

It is recommended to use cathodic protection in accordance with /7/ as additional corrosion protection of the reinforcement. Likewise, application of rustproof reinforcement on exposed parts of the structure shall be considered.

Furthermore, in the splash zone it should be considered to apply glass fibre reinforced epoxy based paint as surface protection of the concrete. Alternatively, a corrosion protecting steel hood can be used around the foundation.

5.2 Steel structures and protection systems

Normally, steel structures for wind turbines shall be designed in accordance with DS 412 in hot-rolled soft steel with the designations S275, S235 or S355, which fulfill the requirements in DS/EN 10025 or similar standard, e.g. DIN 17100.

Welded nails joints shall be designed in compliance with DS 412 and DS/ENV 1090. Bolts and screws, etc. are designed in accordance with DS/ENV 20988.

It shall be assessed whether it will be beneficial to take advantage of the enhanced strength from choosing a high class supervision.

Generally, surface protection shall be executed in correspondence with environmental classes C5-M and Im2 (maritime environment) in accordance with e.g. DS 1090 and DS/EN ISO 12944 Malinger og lakker – korrosionsbeskyttelse af stålkonstruktioner med malingssystemer, (Paints and Enamels – corrosion protection of steel structures with paint systems).

The following corrosion protection is recommended dependent on the siting in relation to the splash zone.

Above the splash zone:

Steel surfaces above the splash zone are normally protected with paint.

In the splash zone:

Steel structure components in the splash zone shall be protected by corrosion protection systems, which are suitable for resisting the aggressive environment in this zone. Recognised design practice involves the application of corrosion allowance as main system for corrosion protection in the splash zone, i.e. the wall thickness is increased due to corrosion. The particular corrosion allowance for a given location shall be assessed in each particular case. However, as guidance for calculation of corrosion allowance it can generally be assumed that the rate of corrosion in the splash zone is in the range of 0.3 – 0.5 mm/per year (ref. /1/). It should be noted that, in general, the rate of corrosion will increase proportionally with the age of the structure.
It is recommended to combine the protection system based on corrosion allowance with surface treatment, e.g. with glass fibre reinforced epoxy paint. It is normal prac-

tice not to take into consideration that the surface treatment reduces the rate of corrosion.

Below the splash zone:

Submerged and inner steel surfaces which are exposed to loads from seawater, e.g. the inside of a pile, ought to be protected cathodically with sacrificial anodes and/or with impressed current supplemented by surface treatment. As regards recommendations concerning design of cathodic protection systems, limits for required steel corrosion potential, etc., please refer to references /2/, /3/ and /7/.

In a zone around the seabed it is recommended to combine the cathodic protection with a corrosion allowance of 3 mm on e.g. piles, and to calculate a reduced fatigue life time, which takes into account that an optimal cathodic protection is not obtainable in this area.

References:

/1/ : DNV “Rules for Classification of Fixed Offshore Installations”, January 1998
/2/ : DNV Recommended Practice RP B401 “Cathodic Protection Design”, 1993
/3/ : DS Rekommendation DS/R 464 “Korrosionsbekyttelse af Stålkonstruktioner i marine omgivelser”, 1988 (DS Recommendation DS/R 464 ”Corrosion Protection of Steel Structures in Marine Surroundings”, 1988)
/4/ : DS 411, “Norm for betonkonstruktioner”, 4. udgave, 1999 (”Norm for Concrete Structures”, 4th edition, 1999)
/5/ : DS 481, “Beton Materialer”, 1. udgave, 1999 (DS 481, ”Concrete Materials”, 1st edition, 1999)
/6/ : DS 482, ”Udførelse af betonkonstruktioner”, 1. udgave, 1999 (DS 482, ”Design of Concrete Structures”, 1st edition, 1999)
/7/ : prEN 12473: Generelle principper for katodisk beskyttelse i havvand (General Principles for Cathodic Protection in Sea Water). DS. 1996

6. ADDITIONAL CONDITIONS

6.1 Occupational safety

Work inside the wind turbine
The rules governing work on offshore wind turbines are identical with the rules governing occupational safety in relation to work on similar onshore wind turbines. Reference is made to section 3.6 in the Technical Criteria.

Manning
Manning requirements corresponding to manning rules for unmanned platforms on the Danish continental shelf should be observed. Reference is made to “Guidelines for design of unmanned production platforms (UP)".

This entails e.g. that procedures shall be drawn up for:
- Manning of the wind turbines.
- How the environmental conditions are monitored when the wind turbines are manned, and when the staff will be evacuated.

Ship transport and landing arrangement
Ship transport to/from the wind turbines and transfer of staff are covered by the instruction of the Danish Maritime Authority: "Teknisk forskift A nr. 2 om arbejdets udførsel om bord på skibe" ("Technical instruction A no. 2 regarding the execution of work onboard vessels").

- Among other things, the following subjects are dealt with: Minimizing of risks.
- Assessment of risks which cannot be prevented.
- Elimination of risks at the source.
- Adjustment of the work to human beings.

The Danish Maritime Authority shall accept solutions and procedures.

Helicopter transport
The Danish Civil Aviation Administration (CAA) is the approving authority in connection with helicopter transport and hoist operations.

It is expected that hoist operations will be subject to implementation in accordance with JAR/OPS 3.005(Z), Helihoist Operation.

It is expected that an actual platform for helicopters shall comply with relevant requirements in BL 3-5. BL 3-5 have been drawn up with particular reference to offshore platforms for oil and gas production. It must therefore be expected that certain requirements in BL 3-5 can be abandoned in relation to offshore wind turbines.

6.2 Lightening recommendation.

Reference is made to DEFU lynrekommandation 25 (DEFU lightening recommendation 25). Dimensioning of lightening protection shall be combined with dimensioning of cathodic protection. See also DEFU Rep. 394 Lyn beskyttelse af vindmøller (- 9: Forhold vedr. korrosion af offshorefundamenter, - 10: Beregning af inducerede strømme og spændinger), (DEFU Rep. 394 Lightening protection of wind turbines (- 9: Conditions reg. corrosion of offshore foundations, - 10: Calculation of induced currents and stresses)).

6.3 Marking

Marking of obstacles in air space
Generally, the marking of wind turbines with respect to aviation shall follow the rules in BL 3-10 ”Bestemmelser om luftfartshindringer” (Regulations on obstacles in air space). In BL 3-10 it is stipulated that the marking shall be carried out in accordance with the following rules:

- 0-100 m: No marking is necessary.
- 100-150 m: The necessity for marking is decided by CAA.
- Over 150 m: Marking is a requirement.

The marking shall be agreed with CAA, including Tactical Air Command Denmark (The Ministry of Defence). This is due to the fact that this authority may place heavier demands on the marking due to the use of rescue helicopters, which fly at a low altitude. The Danish Navigation and Hydrography Administration shall be involved in the determination of the specific aviation marking, as this marking may possibly have an impact on navigation.

Buoyage
The scope of the buoyage is decided on a case-to-case basis. The builder shall come up with a proposal for buoys, possibly with input from the Danish Navigation and Hydrography Administration.
The Danish Navigation and Hydrography Administration will decide if the proposed buoyage system is acceptable.

6.4 Noise emission

The same rules apply as for onshore installations.

6.5 Environmental impact assessment

The EIA assessment (environmental impact assessment) is a supplement to the technical approval of wind turbine installations. However, it should be noted that it is the builder who, in connection with the permission for erection of offshore wind turbines, shall prepare an EIA assessment. The requirements for the contents of the EIA assessment are in alignment with the EC environmental impact assessment directive of 27th June 1985 with modifications of 3rd March 1997 and executive order no. 815 of

28th August 2000 regarding assessment of environmental impact (EIA) in relation to offshore power plant.
In a note of February 2000, the Danish Environment and Energy Ministry set up guidelines for the preparation of the EIA assessment for offshore wind turbines.

7. ANNEXES

Annexes serve as guidance, supplemented by the load cases stated in section 3, Loads and load cases.

It should be noted that specific requirements may apply to electric systems of wind turbines due to desired grid regulation properties. These requirements constitute a tightening of the rules in relation to the existing onshore practice, and possible new load cases in connection herewith, i.e. which have not been covered by Annex A, shall thus be taken into account.

ANNEX A: LOAD CASES ACCORDING TO DS 472 AND THE DANISH APPROVAL SCHEME:

Load situation	**DLC**	**Wind conditions**	**Other conditions**	**Calcula-tion type**	**Partial safety factors**
Normal load cases					
Normal operation	6.2.1.1	$V_{start} < V_{nav} < V_{stop}$ Turbulence from Annex A1 log. wind profile, Kaimal spectrum, exponential coherent function.	Yaw error (distribution) is calculated Surplus for farm turbulence Many time series of wind data necessary to obtain extreme value	U	DS 472 (Table 5.4) with addendum
	6.2.1.1	$V_{start} < V_{nav} < V_{stop}$ Turbulence from Annex A1 log. wind profile, Kaimal spectrum, exponential coherent function. Weibull distribution	Yaw error (distribution) is calculated Surplus for farm turbulence Assessment of necessity for additional time series of wind data	F	DS 472 (Table 5.4) with addendum
Start and transient load during switch between generators	6.2.1.2	$V_{nav} = V_{min}$, V_{nom} og V_{max} Turbulence from Annex A1 log. wind profile, Kaimal spectrum, exponential coherent function.	Surplus for farm turbulence	U	
From free wheeling (or still stand) to normal operation and switch between generators (at particular wind speed)	6.2.1.2	$V_{nav} = V_{min}$, V_{nom} og V_{max} Turbulence from Annex A1 log. wind profile, Kaimal spectrum, exponential coherent function.	Surplus for farm turbulence Unless otherwise documented, the following can be applied in DK (per year): 2000 low wind starter 700 generator switches ⇑ 700 generator switches ⇓ 50 high wind starter	F	
Stop or transition to controlled free wheeling	6.2.1.3	$V_{nav} = V_{min}$, V_{nom} og V_{max} Turbulence from Annex A1 log. wind profile, Kaimal spectrum, exponential coherent function.	Surplus for farm turbulence	U	
Normal stop-sequence	6.2.1.3	$V_{nav} = V_{min}$, V_{nom} og V_{max} Turbulence from Annex A1 log. wind profile, Kaimal spectrum, exponential coherent function.	Surplus for farm turbulence Unless otherwise documented, the following can be applied in DK (per year) 2000 low wind stop 50 high wind stop	F	
Stand still or controlled free wheeling	6.2.1.4	$V_{nav} < V_{min}$ $V_{nav} > V_{max}$ (in case of free wheeling) Weibull distribution		F	

Extraordinary load cases					
Extreme Wind conditions 50-year recurrence period	6.2.2.1	$V_{nav} = V_{10min}$ Turbulence from Annex A1 log. wind profile, Kaimal spectrum, exponential coherent function.	Combined with most unfavourable blade, rotor and yawing positions (the structure of the turbine can possibly exclude certain combinations of rotor position and wind direction) Electric grid cannot be calculated as being present	U	
	6.2.2.1	$V_{nav} = V_{2s}$	Combined with most unfavourable blade, rotor and yawing position (the structure of the turbine can possibly exclude certain combinations of rotor position and wind direction) Electric grid cannot be calculated as being present	U	
	6.2.2.1	$V_{nav} = 10 \rightarrow 25$m/s Simultaneously with wind direction $0° \rightarrow 90°$ in 30 seconds		U	
Transport, assembly and erection of wind turbine	6.2.2.2	Wind speed given by manufacturer		U	
Functional test	6.2.2.3	Wind speed given by manufacturer	Manual operation	U	
Emergency stop	6.2.2.4	$V_{nav} = 1.3 \cdot V_{max}$		U	
Activation of air brakes	6.2.2.4	$V_{nav} = 1.3 \cdot V_{max}$	Most unfavourable yaw error Yaw error	U	
Free wheeling with activated air brakes	6.2.2.4	$V_{nav} = 0.5 \cdot V_{10min}$ Turbulence from Annex A1 log. wind profile, Kaimal spectrum, exponential coherent function.	Surplus for farm turbulence	U	
	6.2.2.4	$V_{nav} = 0.5 \cdot V_{10min}$ Turbulence from Annex A1 log. wind profile, Kaimal spectrum, exponential coherent function.	Surplus for farm turbulence Most unfavourable yaw error 50 hours	F	

Failure in yaw system	6.2.2.5	$V_{nav} < V_{max}$ Turbulence from Annex A1 log. wind profile, Kaimal spectrum, exponential coherent function.	Surplus for farm turbulence Most unfavourable yaw error (incl. rear wind)	U	
	6.2.2.5	$V_{nav} < V_{max}$ Turbulence from Annex A1 log. wind profile, Kaimal spectrum, exponential coherent function.	Surplus for farm turbulence Most unfavourable yaw error 50 hours Possibly exploitation of extra supervision reduces yaw error/duration	F	
Failure in one of the safety systems	p. 32	$0.75 \cdot V_{2s}$	Rotation frequency must not exceed $n_{r,max}$	U	
Working conditions	Technical Criteria p. 31	$V_{nav} < V_{max}$	Blocking of rotor, pitch and yaw system	U	
Failure in blade angle adjustment One blade in most unfavourable position	6.2.2.5	$V_{nav} < V_{max}$ Turbulence from Annex A1 log. wind profile, Kaimal spectrum, exponential coherent function.	Yaw error Surplus for farm turbulence	U	
	6.2.2.5	$V_{nav} < V_{max}$ Turbulence from Annex A1 log. wind profile, Kaimal spectrum, exponential coherent function.	Yaw error Surplus for farm turbulence 200 hours	F	
Failure in air brake system "tip" brakes not in normal position	6.2.2.5	$V_{nav} < V_{max}$ Turbulence from Annex A1 log. wind profile, Kaimal spectrum, exponential coherent function.	Yaw error Surplus for farm turbulence	U	
	6.2.2.5	$V_{nav} < V_{max}$ Turbulence from Annex A1 log. wind profile, Kaimal spectrum, exponential coherent function.	Yaw error Surplus for farm turbulence 200 hours	F	
Accidental state					
Free wheeling with a malfunctioning aerodynamic brake	6.2.3	$V_{nav} = V_{max}$ Turbulence from Annex A1 log. wind profile, Kaimal spectrum, exponential coherent function.	Yaw error Surplus for farm turbulence	U	
	6.2.3	$V_{nav} = V_{max}$ Turbulence from Annex A1 log. wind profile, Kaimal spectrum, exponential coherent function.	Yaw error Surplus for farm turbulence 100 hours	F	

Annex B: LOAD CASES, with reference to the sections (DLC) in IEC 61400-1:

Load situation	DLC	Wind conditions*	Wave conditions	Ice conditions	Other conditions	Calculation type	Partial safety factors
1) Energy production	1.1	NTM $V_{hub}=V_r$ or V_{out}				U	N
	1.2	NTM $V_{in}<V_{hub}<V_{out}$				F	*
	1.3	ECD $V_{hub}=V_r$				U	N
	1.4	NWP $V_{hub}=V_r$ or V_{out}			External electric failure	U	N
	1.5	EOG_1 $V_{hub}=V_r$ or V_{out}			Loss of grid	U	N
	1.6	EOG_{50} $V_{hub}=V_r$ or V_{out}				U	N
	1.7	EWS $V_{hub}=V_r$ or V_{out}				U	N
	1.8	EDC_{50} $V_{hub}=V_r$ or V_{out}				U	N
	1.9	ECG $V_{hub}=V_r$				U	N
2) Production where failure occurs	2.1	NWP $V_{hub}=V_r$ or V_{out}			Control system failure	U	N
	2.2	NWP $V_{hub}=V_r$ or V_{out}			Protection system or subsequent internal electric failure	U	A
	2.3	NTM $V_{in}<V_{hub}<V_{out}$			Control or protection system failure	F	*
3) Upstart	3.1	NWP $V_{in}<V_{hub}<V_{out}$				F	*
	3.2	EOG_1 $V_{hub}=V_{in}$, V_r or V_{out}				U	N
	3.3	EDC_1 $V_{hub}=V_{in}$, V_r or V_{out}				U	N
4) Normal Stop	4.1	NWP $V_{in}<V_{hub}<V_{out}$				F	*
	4.2	EOG_1 $V_{hub}=V_r$ or V_{out}				U	N
5) Emergency stop	5.1	NWP $V_{hub}=V_r$ or V_{out}				U	N
6) Parked (standing still or running)	6.1	EWM $V_{hub}=V_{e50}$			Possible loss of grid	U	N
	6.2	NTM $V_{hub}<0.7V_{ref}$				F	*
7) Parked and failure	7.1	EWM $V_{hub}=V_{e1}$				U	A
8) Transport, assembly, maintenance and repair	8.1	To be inserted by the manufacturer				U	T

Abbreviations, please refer to the next page

* If none (normal) cut out wind speed V_{out} is defined, the value V_{ref} should be applied.

Explanation to table with load cases:

DLC	Design load case
ECD	Extreme coherent gust with direction change
ECG	Extreme coherent gust
EDC	Extreme direction change
EOG	Extreme operating gust
EWM	Extreme wind speed model
EWS	Extreme wind shear
Subscript	Recurrence period in years
NTM	Normal turbulence model
NWP	Normal wind profile model
F	Fatigue
U	Ultimate
N	Normal and extreme
A	Abnormal
T	Transport and erection
•	Partial safety factor for fatigue

Annex C: WEIGHTED PARTIAL SAFETY FACTOR AND EFFECTS OF A MULTI-REPLICATED EVENT

C1 Introduction

Below is given a brief description of a method for determination of a combined determining load on stiff foundations for offshore wind turbines by means of a weighted partial safety factor. The method is applicable notwithstanding whether or not the loads are the result of a combination of extreme events, or whether they are the result of a situation with an operational load which occurs several times together with a corresponding wave load or an extreme ice load. The centre of attention is solely on cylindrical structures equipped with a curved cone (which bends the ice downwards) to minimise ice loading, previously done at e.g. the “Middelgrunden”. The method has been developed with a view to determining the design loads, which are applicable to the foundation, on the basis of maximum values in time series for wind, wave and ice loads obtained by means of a mixture of simulations and tests.

A precise dimensioning presupposes a number of simulations, execution of model testing (with ice and waves) and subsequent combined simulations. Nonetheless, the Annex comes up with a number of proposals for approximated methods, which can be used for rough calculations. As a minimum, the following should be examined:

a) how big a difference is there between the mean value and the mean-max event
b) that the approximation of the quadratic model of composition is satisfactory
c) that the 10 minutes extreme event is close to a normal distribution, and
d) that the coefficient of variation of the combined extreme response can be weighted linearly in relation to the maximum values

C2 Determination of load combination in relation to a chosen level of probability for extreme loading

Below is given a preliminary and simplified model for determination of the weighted partial safety factor. It is critical for the result that a careful selection of the combined event for wind load and wave/ice load is undertaken.

As the partial safety factor of the wind load on 1.5 corresponds to a situation where the wind load is given by the probability $p = 7.6 \times 10^{-4}$/year (T = 1320 years), partial safety factors for combined loads (f_R) can be determined on the basis of a comparison of results from 2 simulations as the relation between the largest combined load corresponding to the 1320/year load and 50/year load ($p = 7.6 \times 10^{-4}$/year and $p = 2 \times 10^{-2}$/year), respectively. Given that a situation with ice floes of such an insignificant size that waves can occur is not assumed to be determining, the following two load combinations are observed:

1. 1320/year event for both wind and wave load, and
2. 1/year operational event for wind load combined with the 1320/year event for ice load.

The corresponding partial safety factors are determined by:

Combined wind and wave load:
$f_R = R_{max}$ (wind + waves for p = 7.6 x 10^{-4}/year) / R_{max} (wind + waves for p = 2 x 10^{-2}/year)

Combined wind and ice load:
$f_R = R_{max}$ (wind for p = 1/year + ice for p = 7.6 x 10^{-4}/year / R_{max} (wind for p = 1/year + ice for p = 2 x 10^{-2}/year)

As regards wind load it is assumed that the partial safety factor more or less pays equal attention to the coefficient of variation and to the uncertainty attached to the model. It is emphasized that the chosen method implies that the relative significance of the model uncertainty, which is assumed in case of wind load calculations, is also assumed to be applicable to wave and ice loads, i.e. the larger the coefficient of variation of the load is, the greater is also the uncertainty attached to the model. As the distributions for wave and ice loads for the given type of foundation are often determined by means of model testing, analysis of external conditions and interpolation, it is crucial that an assessment is made of whether the test results and field measurements are attached with model uncertainties, which relative significance is comparable to the model uncertainty in relation to wind loads. To be on the safe side, it should be demonstrated that the test results and the analysis of field conditions overrate the loads. Note should also be taken of the fact that the selected load combinations are believed to be sufficient for the chosen type of foundation, but that they are not necessarily valid in general.

Alternatively, it would be possible to determine that the 3 acting external loads contain a relatively small, mean or large coefficient of variation and corresponding model uncertainty with the following range of partial safety factors, and to apply this range in connection with the load combination:

Load	Coefficient of variation and model uncertainty for external loads	Partial safety factor
Wind	Mean	1.5
Waves (top breaking, inertia forces dominating)	Small	1.2
Ice (ice cone on foundation)	Large	1.8

Table C1 Range of partial safety factors on external loads

The following section describes an additional method for the handling of combined loads in a given mode of operation, where many events occur several times with the same wind conditions. This method is also applicable to extreme ice loading, where it is also assumed that a certain amount of repeated events occur together with extreme ice conditions.

C3 General observations regarding the combination of stationary stochastic time series

If two stochastic independent stationary wave time series are overlaid (i.e. with same direction), the power spectrum of the combined time series will equal the sum of the two power spectra S_η.

From wave series 1 : $m_{01} = \int_o^\infty S_{\eta 1}(f)df$ = variance of $\eta_1^2(t) = \sigma_1^2$, $H_{s1} \cong 4\ \sigma_1$, $H_{1(1\%)} = 1.5\ H_{s1}$
From wave series 2 : $m_{02} = \int_o^\infty S_{\eta 2}(f)df$ = variance of $\eta_2^2(t) = \sigma_2^2$, $H_{s2} \cong 4\ \sigma_2$, $H_{2(1\%)} = 1.5\ H_{s2}$
where σ denotes the deviation, H_s denotes the significant wave height, and $H_{(1\%)}$ denotes the wave height, which is exceeded by 1% of the waves.

The aggregate time series is given by the expression: $m_{0t} = m_{01} + m_{02}$, $\sigma_t = (\sigma_1^2 + \sigma_2^2)^{0.5}$, $H_{st} \cong 4\ \sigma_t$, $H_{t(1\%)} = 1.5\ H_{st}$

The expressions for H_s og $H_{1\%}$ are approximated, even though the wave periods deviate considerably. Thus, the variance of the two overlaid signals is combined linearly, while the standard deviations are combined quadratically. All other parameters are approx. proportional with the deviation. Even in situations with a relatively big difference in the periods of the time series, where the combined spectrum becomes double-peaked, the combined parameters of the wave train can be related to the total deviation.

Simultaneously, in terms of non-correlated stochastic force/bending moment time series, the variance of two combined times series is likewise denoted as the sum of the two original variances, i.e. linearly, while the standard deviations are combined quadratically. When the content of the period in question is somewhat different and the physical character results in a different function of distribution, a different factor (K) may appear between the maximum event (mean-max) minus the mean value and the deviation. It cannot, however, be different than the combination of wave train with different wave spectra.

The most simple way to weigh this is to assume that the maximum (mean-max) in the combined time series can be calculated as a quadratic sum of the deviations from the mean values plus the linear sum of the mean values. This way, an automatic weighing of the factor k_t on the deviation is obtained with which the maximum events deviate from the mean value:

$F_t = F_{mean\ 1} + F_{mean\ 2} + ((F_{max,1} - F_{mean\ 1})^2 + (F_{max,2} - F_{mean\ 1})^2)^{1/2}$
$F_{max,1} = F_{mean\ 1} + k_1\sigma_1$
$F_{max,2} = F_{mean\ 2} + k_2\sigma_2$
$F_t = F_{mean\ 1} + F_{mean\ 2} + k_t\sigma_t$
$\sigma_t = (\sigma_1^2 + \sigma_2^2)^{1/2}$
$k_t\sigma_t = ((k_1\sigma_1)^2 + (k_2\sigma_2)^2)^{1/2}$

On the basis of simulations carried out by Risø, among others, it has been demonstrated that external loads on offshore wind turbine foundations in a number of instances have approximately followed this simple model of composition, despite the fact that the main content of the wind load is dominated by a tower period around 2.5. s., while the wave load typically operates with a period twice that long. Examples of application and establishment of a procedure for determination of a consistent set of determining conditions on the basis of parameters given by means of simulations are presented below. The more different the two force time series are with regard to the content of the period and the factor between the maximum event minus the mean value and deviation, the less accurate a simple model of composition will be. An ice load can e.g. contain a high frequent component, which must be assessed individually. Likewise, the ice load may change its type of rupture and mean value, which again must be assessed individually.

For fully correlated events, i.e. where the maximum in one of the time series appears simultaneously with the maximum in the other, the following applies:

$$F_t = F_{max,1} + F_{max,2} = F_{mean\ 1} + F_{mean\ 2} + ((F_{max,1} - F_{mean\ 1})^1 + (F_{max,2} - F_{mean\ 1})^1)^{1/1}$$

If necessary a partly correlated empirical combination can be defined on the basis of:

$$F_t = F_{mean\ 1} + F_{mean\ 2} + ((F_{max,1} - F_{mean\ 1})^n + (F_{max,2} - F_{mean\ 1})^n)^{1/n},$$

where $1 < n < 2$

C4 Example of determination of weighted partial safety factors for extreme wind and wave loads

Example:

				Wind load (max.)*		Wave load (max.)**		Wind + wave load***		f_{Fx}	f_{My}
Example	Water depth	Combi-nation	Frequency	F_x	M_y	F_x	M_y	F_x	M_y		
	M			MN	MNm	MN	MNm	MN	MNm	-	-
1	5.8	All	$2x10^{-2}$	0.56	38.8	1.60	10.6	1.90	41.5	1.00	1.00
	5.8	Wind	7.6 $x10^{-4}$	0.84	58.2					1.50	1.50
		Waves				1,92	13,1			1,20	1,20
		Wind + waves		0.84	58.2	1.92	13.1	2.39	61.4	1.26	1.46
2	10	All	$2x10^{-2}$	0.56	41.1	2.20	24.2	2.50	52.3	1.00	1.00
	10	Wind	7.6 $x10^{-4}$	0.84	61.7					1.50	1.50
		Waves				2.64	29.0			1.20	1.20
		Wind + waves		0.84	61.7	2.64	29.0	3.09	73.2	1.24	1.40

* DS472

** Determined on the basis of model testing combined with collection of statistics (preliminary typical estimate)

*** Determined by means of simulations of combined time series. Preliminary estimate: It is assumed that the mean wind loads represent half of the maximum wind loads, and that the combined loads can be calculated on the basis of
$F_x = 0.5\ F_{x,wind} + ((0.5\ F_{x,wind})^2 + (F_{x,wave})^2)^{0.5}$ and $M_y = 0.5\ M_{y,wind} + ((0.5\ M_{y,wind})^2 + (M_{y,,wave})^2)^{0.5}$

Table C2 Example of determination of partial safety factors when using method C2

It should be noted that in this example test results weighted with the probability distribution for relevant wave and water level conditions have found that the partial safety factor for wave load on the safe side can be set at 1.20, and that the example is based on the preconditions a) and b) from section C1. Thus, in so far as concerns precondition a), it is assumed that the mean wind force constitutes half of the mean-max event. It appears from the above that the partial safety factor on the combined load for horizontal force in the above example is in the range of 1.25, while the partial safety factor on the bending moment is in the range of 1.45.

C5 Composition of operational loads with corresponding wave load

First, the number of repetitions *n* of the given mode of operation over the life time for operational wind load and operational wind load combined with wave load, respectively, are determined. A philosophy of certainty is defined based on the assumption that the averaged weather condition will deteriorate in the entire life time, i.e. corresponding to a situation where the recurrence period of the maximum event in the observed mode of operation occurs twice as often as normally. The number of events are therefore multiplied by a factor 2

Hereafter, a number of simulations are carried out for wind load and simulations/model testing of wave load for determination of the maximum response for each of the external loads. On the basis of these the distribution function of the maximum event is determined by means of the method described in section C3. As the tail of this distribution is particularly important to the extrapolation from 1 to *n* repetitions of the observed mode of operation, the number of selected simulations shall, in order to be able to determine the distribution of the tail with certainty, be considerably higher than the five mentioned in section 3.1.2. for determination of the mean value of the maximum event. Otherwise, it should be assumed that the extreme events have a certain distribution (e.g. Gumbel) and then simply estimate the parameters in this distribution on the basis of an appropriate number of simulations, which must usually be higher than five. It is, among other things, important to note that the number of simulations set forth in section 3.1.2 only apply to determination of mean values. Determination of other distribution properties usually demands more simulations. It is also important to note that a conclusion, which rests on a higher number of simulations is not necessarily better than a result, which rests on a distribution assumption combined with a limited number of simulations.

Preliminary analyses have shown that it can be assumed that at least the extreme event for the shear force below the foundation during normal operation is based on a normal distribution. In terms of the bending moment, analyses show that the maximum event rests on a Gumbel distribution. On the basis of the different simulations, the best estimate of the deviation is determined. If there are deviations from the assumption of a normal distribution, emphasis is given to the most rare events on the basis of which a conservative estimate for deviations in the approximated normal distribution is made. Based on the number of events (*n*), the factor K is hereafter determined by which the deviation in relation to the mean-max value shall be multiplied in order to allow the probability of deviation to become 1/*n*. The factor K for the normal distribution is shown in Fig. C1.

Fig. C1 k-factor in normal distribution

In Fig. C2 and C3 the relative distributions in relation to the mean-max events for horizontal force (F_x) and bending moment (M_y) are illustrated.

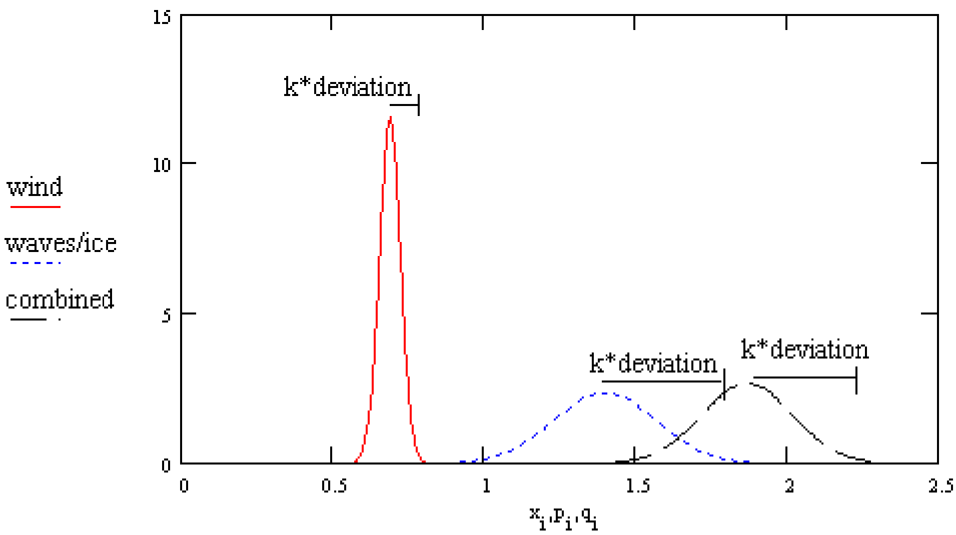


Fig. C2 Illustration of functions of distribution for maximum horizontal force from wind, waves/ice and combined wind with waves/ice for a given simulation period and in a given mode of operation.

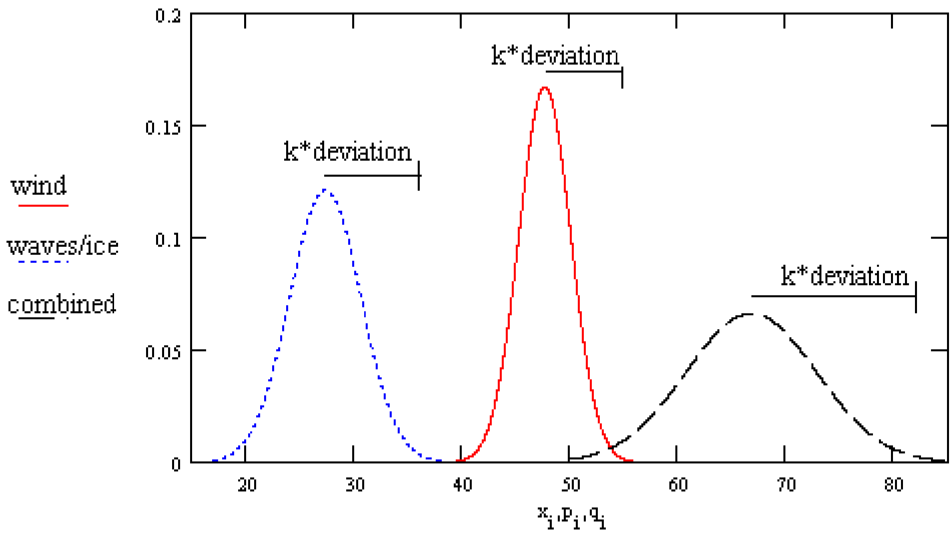


Fig. C3 Illustration of functions of distribution for maximum bending moment from wind, waves/ice and combined wind with waves/ice for a given simulation period and in a given mode of operation.

Example

Ex.	Water depth	No. of 10 min. events in life time	Wind load (max.)			Wave load (max.)			Wind + wave load**					
	m		F_{xv} (mean - max)	M_{yv} (mean - max)	V_v	F_{xb} (mean - max)	M_{yb} (mean - max)	V_b **	F_x (mean - max) *	M_y (mean - max) *	1+ kV_{Fx}	1+ kV_{My}	F_x (max) ***	M_y (max) ***
			MN	MNm		MN	MNm		MN	MNm			MN	MNm
1	5,8	3.000	0.69	44.8	0.05	1.40	9.3	0.12	1.87	47.4	1.35	1.22	2.52	57.9
2	10	3.000	0.69	47.7	0.05	1.93	21.2	0.12	2.39	58.0	1.37	1.26	3.27	72.9

* Preliminary estimate of mean-max. values (based on the assumption that the mean wind load constitutes 65 % of maximum wind load):
$F_x = 0.65\, F_{x\ wind} + ((0.35\, F_{x,wind})^2 + (F_{x,wave})^2)^{0.5}$ and $M_y = 0.65\, M_{y,wind} + ((0.35\, M_{y,wind})^2 + (M_{y,,wave})^2)^{0.5}$

** Only applicable to heavy shallow wave loads dominated by inertia forces

*** F_x (max) = F_x (mean-max) x $(1 + kV_{Fx})$

*** M_y(max) = (M_y (mean-max) x $(1 + kV_{My})$
where k is determined on the basis of the number of events for the normal distribution (k = 3.1 for n = 1000, k = 3.4 for n = 3.000, k = 3.75 for n = 10.000, k = 4.05 for n = 30.000)

Table C3 Example of determination of maximum combined operational wind force and corresponding wave force

Preliminary estimate: $V_{Fx} = (V_v \times F_{xv} + V_b \times F_{xb})/(F_{xv} + F_{xb})$, $V_{My} = (V_v \times M_{yv} + V_b \times M_{yb})/(M_{yv} + M_{yb})$

It should be noted that the example is based on the preconditions c) and d) in section C1. As regards this particular load case, there is no partial safety factor but a factor = 1 + kV of approx. 1.35 for horizontal force and approx. 1.25 for bending moment, which allows for the number of repetitions and for the coefficient of variation. Thus, contribution from the model uncertainty is not included in the determination of the partial safety factor. Furthermore, it should be noted that V_v denotes the coefficient of variation of F_{xv} and M_{yv} and simultaneously of V_b, V_{Fx} og V_M. The coefficient of variation is defined by:

$$V_V = \frac{\text{deviation of max } F_{XV}}{\text{mean - max of } F_{XV}}$$

C6 Composition of extreme ice load with corresponding (operational) wind load

First, the number of repetitions for the given mode of operation in the life time for extreme ice load combined with operational wind load are determined. A number of simulations/model testing of ice load and wind (see section C5 for a discussion on the required number of simulations) are carried out for determination of the distribution of the maximum event. In the case of ice load conversion to an event corresponding to a frequency of 2×10^{-2} (characteristic load) and 7.6×10^{-4} (design load), respectively, is used. In the example below, this corresponds to a situation where the characteristic load is multiplied by a factor of approx. 2.0 (for the kind of bending rupture which occurs on cone structures) in order to find the ice load corresponding to a frequency of 7.6×10^{-4}. Hereafter, equivalent simulations of the combined events are carried out.

Preliminary analyses have shown that it can be assumed that at least the maximum event for the shear force below the foundation is based on a normal distribution. On the basis of the different simulations, the best estimate of the deviation is made. If there are deviations from the assumption of a normal distribution, emphasis is given to the most rare events on the basis of which a conservative estimate of the deviation in the approximated normal distribution is made.

Example:

Frequency 2 x 10^{-2}:

Ex.	Water depth	No. of 10 min. events	Wind load (max.)			Iice load (max.)			Wind + ice load**					
	M		F_{xv} (mean -max)	M_{yv} (mean -max)	V_v	F_{xi} (mean -max)	M_{yi} (mean -max)	V_i **	F_x (me-an-max) *	M_y (mean -max) *	1+ kV_{Fx}	1+ kV_{My}	F_x (max) ***	M_y (max) ***
			MN	MNm		MN	MNm		MN	MNm			MN	MNm
1	5.8	100	0.69	44.8	0.05	1.00	10.3	0.12	1.51	51.1	1.21	1.15	1.82	58.6
2	10	100	0.69	47.7	0.05	1.00	14.8	0.12	1.51	57.1	1.21	1.15	1.82	65.9

Frequency 7.6 x 10^{-4}:

Ex.	Water depth	No. of 10 min. events	Wind load (max.)			Ice load (max.)			Wind + ice load**					
	M		F_{xv} (mean -max)	M_{yv} (mean -max)	V_v	F_{xi} (mean -max)	M_{yi} (mean -max)	V_i **	F_x (me-anl-max) *	M_y (meanl -max) *	1+ kV_{Fx}	1+ kV_{My}	F_x (max) ***	M_y (max) ***
			MN	MNm		MN	MNm		MN	MNm			MN	MNm
1	5,8	100	0.69	44.8	0.05	2.00	20.6	0.12	2.48	58.7	1.23	1.17	3.05	68.4
2	10	100	0.69	47.7	0.05	2.00	29.6	0.12	2.48	68.6	1.23	1.18	3.05	80.7

* Preliminary estimate for mean-max values for foundation with ice cone:
$F_x = 0.65\ F_{x,wind} + 0.55\ F_{x,ice} + ((0.35\ F_{x,wind})^2 + (0.45\ F_{x,ice})^2)^{0.5}$
and $M_y = 0.65\ M_{y,wind} + 0.55\ M_{y,ice} + ((0.35\ M_{y,wind})^2 + (0.45\ M_{y,ice})^2)^{0.5}$

** Only applicable to ice load on cone and with mean wind load = 65 % of maximum wind load

*** F_x (max) = F_x (mean-max) x $(1 + kV_{Fx})$, M_y(max) = M_y (mean-max) x $(1 + kV_{My})$
where k is determined by the number of events for the normal distribution (k = 2.3 for n = 100, k = 3.1 for n = 1000, k = 3.4 for n = 3.000,
k = 3.75 for n = 10.000, k = 4.05 for n = 30.000)

Table C4 Example of determination of maximum combined extreme ice force with operational wind force. Reference is made to the table in section C5 for a description of the coefficients of variation V_v, V_i and so forth.

In this scenario there is a partial safety factor containing the difference between the mean-max values of approx. 1.65 for horizontal force (2.48/1.51 and approx. 1.20 for bending moment (58.7/51.1 and 68.6/57.1), respectively. These factors allow for, inter alia, model uncertainty based on the preconditions given in section C2. In addition a factor = 1 +kV of approx. 1.20 is found which takes due consideration to the number of repetitions and to the coefficient of variation.

At this point an assessment of a stiff foundation with ice load on 55° cone without significant dynamic reinforcement is made only. On the basis of tests with ice load, the following (mean-max) parameters for ice load are determined:

$$F_i = F_{i0} + F_{ivar} + F_{ihigh}$$

where

F_{i0} = quasi stationary component
F_{ivar} = variable component in period interval approx. 1-10 s
F_{ihigh} = high frequent component

The following rough model is applied:

Horizontal: $F_{i0} + F_{ihigh} = 0.55\ F_i$, $F_{ivar} = 0.45\ F_i$,
Vertical: $F_{iz} = 0.5\ F_i$ operating in high water level
Preliminary estimate: $V_{Fx} = (V_v \text{ x } F_{xv} + V_i \text{ x } F_{xi})/(F_{xv} + F_{xi})$
$V_{My} = (V_v \text{ x } M_{yv} + V_b \text{ x } M_{yi})/(M_{yv} + M_{yi})$

On the basis of an assessment note (see Ref. /4/ to section 3.3.3), which describes Ralston's theory and gives an estimate of the time variation, the expressions $F_{i0} = 0.55\ F_i$, $F_{ivar} = 0.45\ F_i$ (i.e. exclusive of a high frequent component) are applied. On the contrary, F_{i0} is calculated on the basis of Ralston's formula to be higher than otherwise experienced from tests.

Annex D. IEC Class S Description

IEC(ENV) 1400-1, 1st edition:1994 (Wind Turbine Generator Systems – Safety Requirements) and IEC 61400-1, 2nd edition:1999 are not valid, neither in Denmark, nor in Europe. To facilitate persons seeking an approval in countries which have implemented the IEC norm, we have made a 'translation' of the Danish reference turbine for an IEC-class S turbine described herein, i.e. as specified in IEC 61400-1, 2nd edition:1999.
In extraordinary situations, it may be necessary to supplement the Danish codes with e.g. DIN standards.

The tabulation in the below table corresponds to the requirements specified in IEC 61400-1, 2nd edition, Annex A. A specification of the structural safety has, however, been added, which shall be taken into consideration.

D.1 Machine parameters

To be negotiated between buyer/seller and be filled out

Machine parameters:	**Parameter**	**Dim.**
Maximum effect		kw
Hub height wind speed - operating range	$V_{in} - V_{out}$	m/s
Technical life time		year

D.2 Wind conditions

Wind conditions:	**Parameter**	Dim
Characteristic turbulence intensity as a function of mean wind speed, **fatigue**	$I = 1/\ln(h/z_0)$ $z_0 = 0.001$ m ([4])	-
Characteristic turbulence intensity as a function of mean wind speed, **extreme wind**	$I = 1/\ln(h/z_0)$ $z_0 = 0.004$ m	-
Annual mean wind speed The stated annual mean wind speeds are applicable to structural calculations only.	50 m height. To be extrapolated according to DS 472 with $z_0 = 0.001$ m The interior Danish waters: 8.5 *Calculation according to* Wasp or similar	m/s
Mean inclination of flow	0	Deg.
Wind speed distribution (Weibull, Rayleigh, measured, other)	Weibull, parameters from European Wind Atlas	
Reference wind speed	*See e.g. addendum to DS 472*	m/s m/s
(NWP) Normal wind profile model and parameters	Logarithmic profile. $v_{10\,\min}(v) = v_b k_t \ln(h/z_0)$ $z_0 = 0.001$m, $k_t = 0.16$	M/s
Turbulence model and parameters	Kaimal, $\chi(L,n) = \exp(-12(nL/V_{10\,\min}))$	m/s
Model for farm-generated turbulence	*See e.g. addendum to DS 472*	
(EWM) Extreme wind speed at hub height	$V_{e50} = v_b k_t (\ln(h_{nav}/z_0) + 3)$, $z_0 = 0.004$ m, k_t see NWP above. $V_{e1} = 0.75 V_{e50}$, V_{eN} *2s mean*	m/s m/s
(EOG) Model for extreme wind gusts and parameters, for 1-year and 50-year recurrence period	Is not applied in DK. For export IEC 61400-1 - model is applied	
(EDC) Extreme wind direction change: model and	Is not applied in DK. For export IEC	

[4] Is applicable to offshore installations. If the turbine is also applied onshore, a higher z_o value shall be selected and be indicated here

parameters for 1-year and 50-year recurrence period	61400-1 - model is applied	
(ECG) Model for extreme coherent wind gusts and parameters	Is not applied in DK. For export IEC 61400-1 - model is applied	
(EDC) Model for extreme coherent wind gusts with change of direction and parameters	In 30 s: both Wind speed 10$\rightarrow$25, Direction 0$\rightarrow$90	M/s, deg
(EWS) Model for extreme vertical wind speed change of parameters	Is not applied in DK. For export IEC 61400-1 - model is applied	
Wind conditions during erection and operation	Separate report	

D.3 Structural safety

The definition of structural safety is not included in IEC 61400-1, Annex A. However, a wind turbine which is designed for another safety level than 61400-1 is also designated as a class S wind turbine (IEC61400-1, section 5.3, para 3). Furthermore, the objective of the definition in Annex A is that there shall be no doubt as to which wind turbine design reference is made to. Consequently, this section D.3 must be included in a class S specification.

The structural safety of the turbines is determined according to Danish codes of practice.

For use in IEC 61400-1 situations, the level of safety can be illustrated by the following crucial partial safety factors[6] for normal safety class

Partial safety factors[5]:

Parameter	**Danish standards, γ**	Quantile p%, COV δ%	IEC 61400-1, γ	IEC 61400-1 with selection of Danish codes for materials
Wind load	**1.5**		1.35	1.35
Gravity load	**1.0**		1.1	1.1
Inertia loads	**1.0**		1.25	1.25
Operational loads	**1.3**		1.35	1.35
Steel, yield stress	**1.30**	5%,5%	>1.04	1.30
Reinforcement	**1.25**	5%,5%	>1.04	1.25
Concrete	**1.48**	5%,15%	>1.16	1.48

The first column depicts a selection of partial safety factors in situations where Danish codes of practice shall be applied exclusively. Comparison with IEC 61400-1 cannot be precise as this code does not have a well defined safety level. This is due to the fact that the choice of national codes of practice as regards materials is optional. In Denmark, for instance, Danish codes on the choice of materials will be applied. Therefore, the safety level of IEC 61400-1 will be dependent on the applied codes of practice. The last column gives an indication of the level, if Danish codes of practice for materials are used in conjunction with IEC 61400-1. The level will be changed, if similar codes from other countries are applied.

Foundation, tower, nacelle and rotor are constructed for a normal safety class[7]. Elements of the structure in the safety system, which have a bearing on the safety, are constructed for a high safety class.

[5] This table serves as an illustration of partial sqfety factors in connection with the IEC, class S description only. As regards requirements to partial safety factors in this offshore wind turbine design basis, reference is made to the previous chapters.

[6] The term 'safety level' is applied as a designation for the combination of chosen partial coefficients and quantiles selected as characteristic values for loads and material strengths.

[7] Safety classes defined in DS

D.4 Electric conditions

The following table should be filled out after negotiations between buyer/seller

Electric grid conditions	**Parameter**	**Dim.**
Normal voltage and interval of variation		V
Normal grid frequency and interval of variation		Hz
Voltage instability		V
Maximum duration of grid failure		sec, h, days
Number of electric failures		$Year^{-1}$
”Auto-reclosing cycles” (description)		
Behavior during symmetric and asymmetric failure (description)		

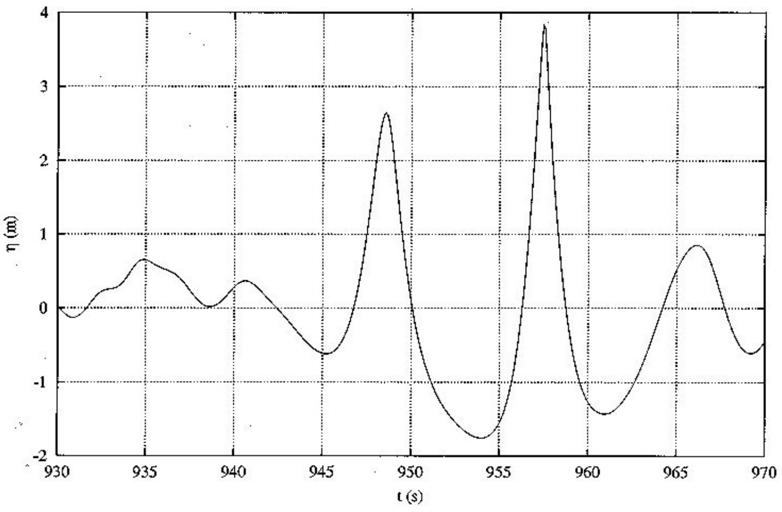
D.5 Other external conditions

Other external conditions	**Parameter**	**Dim.**
Soil strengths, statistics	Part 3	
Soil strengths, dynamics	Part 3	
Normal sea level and extremes	Part 2	
Model for waves and wave direction, extreme heights corresponding to 1-year and 50-year ”recurrence” intervals	Part 2	
Model for current, extreme velocities corresponding to 1-year and 50-year ”recurrence” intervals	*Part 2*	
Model for ice forces: extreme ice forces as a function of cross section, corresponding to 1- year and 50-year ”recurrence” intervals	*Part 2*	
Ship impact	Part 2	
Begroning (description)	Part 2	
Materials (description)	Part 4	
Normal and extreme temperatures	DS 472	°C
Humidity	Detailed examination	%
Air density	DS 472	kg/m^3
Solar rays	1000	W/m^2
Rain, hail, snow and icing (icing: see Part 3)		
Active chemical substances		
Active mechanical particles		
Description of lightening protection system	Part 5	
Earthquake model and parameters		
Salinity	Detailed examination	g/m^3

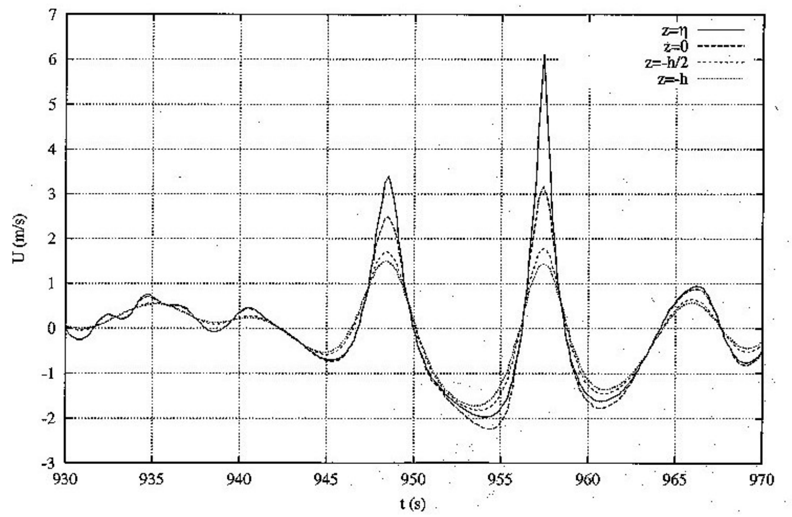
Annex E: Illustrations of waves in low waters

In this Annex examples are given of results from calculation of kinematics in high waves in low waters (Per Madsen and Harry Bingham). The results illustrate the capability of the model to handle steep waves.

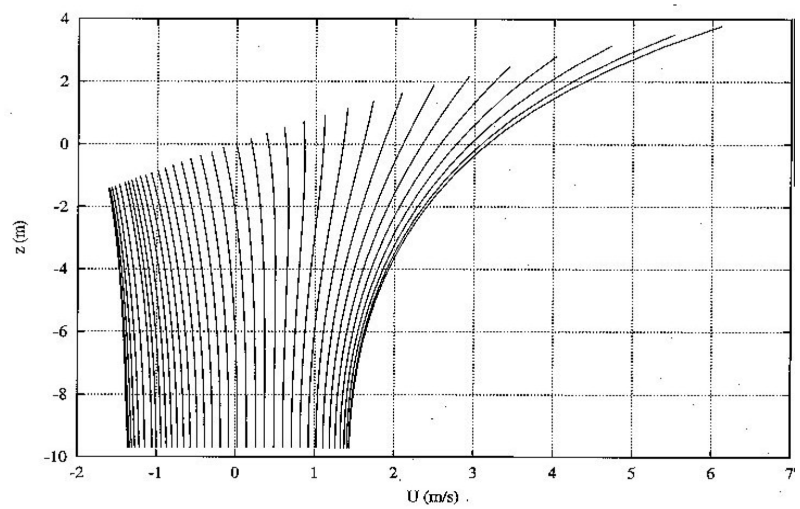
The figure shows the wave elevation (η) as a function of the time (t) for a section of a time series comprising the highest calculated crest near the wind turbine:



The figure below shows simultaneous horizontal velocities u (in m/s) as a function of t near wind turbine in different levels (wave crest ($z = \eta$), mean water surface ($z = 0$), half water depth ($z = -h/2$) and seabed ($z = -h$)) for the same time series as the one indicated above:



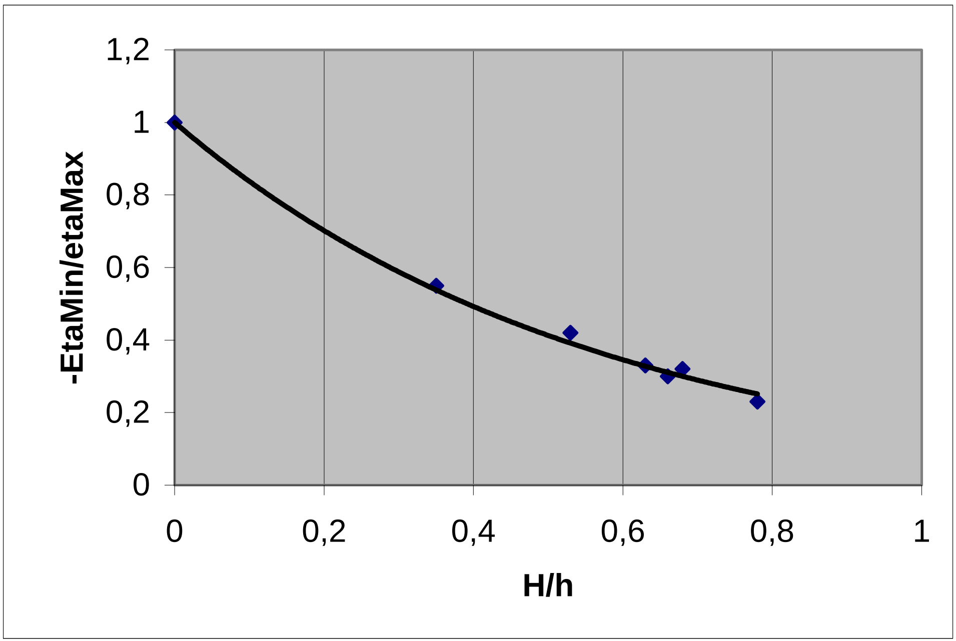
The figure below shows horizontal velocities closest to the highest crest: Time steps for wave troughs before wave crest are shown (approx. t = 954 s) to wave crest (approx. t = 957.5 s).



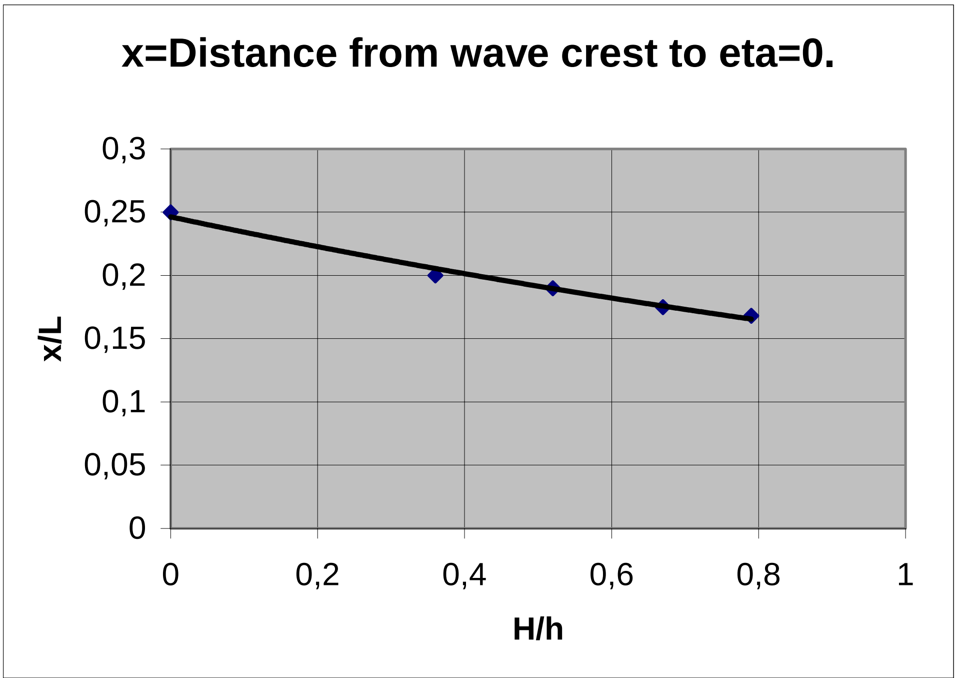
E.1 Typical wave parameters for high waves in low waters (isolated waves calculated on the basis of the stream function wave theory).

The below figure can be used graphically to estimate the wave profile for isolated waves on the basis of 3 points, which can be read when wave height, wave length and water depth have been established, and it is estimated that the wave profile is symmetric around wave crest.

The figure shows the relationship wave trough (EtaMin) / wave crest (EtaMax) for different wave heights (H = wave height, h = water depth):



Distance from mean water surface to crest (L=wavelength)



Annex F – Particular conditions for specific foundation concepts

At any given time it shall be documented that soil conditions with higher strengths and stiffnesses than the ones stated, do not give rise to ruptures in the soil carrying capacity, unacceptable stress concentrations and deformations in both structure and soil with corresponding damage. This also applies to allowable differential settlements of the structure.

For all foundation types the effect of the cyclical load on the soil stiffness shall be assessed, and it shall be demonstrated that no critical response will occur within the elements of uncertainty which are attached to the applied method of analysis. The effect shall possibly be substantiated by means of laboratory tests, where the sample(s) is exposed to a load history corresponding to the most severe dynamic load case.

Direct foundation

The following relevant limit conditions shall, inter alia, be taken into account:

- Total stability failure
- Rupture in soil carrying capacity
- Sliding ruptures
- Combined ruptures in soil and structure
- Ruptures due to foundation movements
- Unacceptable movements and oscillations
- Eigenfrequency analysis

If direct foundation is used the effect of cyclical load on the soil stiffness shall be assessed, and it shall be verified that no critical response will occur within the elements of uncertainty, which are attached to the applied method of analysis. The effect shall possibly be substantiated by means of laboratory tests, where the sample(s) is exposed to a load history corresponding to the most severe load case, which is deemed to emanate from the wind load. As the working curves for structure and soil are difficult to determine, the structure should therefore be treated in a geotechnical class 3.

Sliding
If passive earth pressure is calculated, documentation of the expected damage percent must be provided (e.g. scour), also if filling around the foundation is accounted. The maximum allowable damage percent shall at any given time be adjusted to the specific project.

The sliding analysis shall include both horizontal forces and torsion moments around the vertical axle of the structure.

Sliding shall be examined in 2 cases:

- According to DS 415
- In case the structure is founded on layers of clay, the possibility for softening the layer of clay shall be examined. $c_u = k \times \sigma'$, where the parameter k (typical value:

0.4<k<0.55) is determined on the basis of tests or experienced values for corresponding soil, and with due consideration to the relevant rate of deformation.

Eigenfrequencies
In connection with calculation of eigenfrequencies springs can be attached to the foundation, which demonstrate the stiffness of the soil, see e.g. DNV (1992) Classification Notes N0. 30.4, Foundations.

Furthermore, drainage conditions must usually by assumed in such a way that they are unfavourable to the structure.

In relation to normal Danish geological formations, the following is emphasised:
- Unhardened lime (H1): Friction conditions shall be analysed.
- Cracked hardened lime: If intact samples cannot be found, the geotechnical properties shall be elucidated by means of relevant in-situ tests, e.g. pressiometer tests.

Direct foundation – skirt

If the stability of the foundation is based on a full/partly exploitation of differential water pressure for bearing aspects of brief tensile forces, documentation shall furthermore contain an assessment of safety precautions against hydraulic instability.

If skirt foundations are applied for horizontal bearing aspects, documentation for stability of both the structure and the surrounding soil shall be provided.

If the skirt has been exposed to an obstruction with corresponding damage, and the skirt forms part of the total structure, the contribution of the skirt to the stability of the behaviour of the structure shall not be taken into consideration.

Pile foundation

Pile foundations with large pile dimensions (incl. connection between piles and structure) shall be dimensioned in accordance with the principles in the offshore code DS 449 and conventional offshore practice (see e.g. DNV Class Note 30.4).

The piles shall be dimensioned for possible scour of the seabed around the structure (scour).

The foundation shall be examined in the following situations:

- Elastic ultimate limit state
- Plastic ultimate limit state
- Fatigue, which shall contain the effects of the actual fatigue load on the structures and possible partial damage caused by the effects from pile driving
- Pile driving analysis
- Eigenfrequency analysis

In the analysis of the elastic ultimate limit state, stresses in piles and structure are examined. Only one pile is allowed to reach the yield point as a maximum.

In the analysis of the plastic ultimate limit state, the total stability of the entire structure is analysed. In this analysis, the piles are allowed to yield, as long as the piles can absorb the design loads.

As a first estimate, the pile length of a transverse loaded pile is determined on the basis of the criteria that there must not be any characteristic deflection at the point where the deflection line passes the neutral line for the second time during extreme loading (zero toe-kick).

Usually, the above results in the determination of a somewhat conservative pile length. A more realistic requirement is attached to the permanent deformation (the inclination of the pile in the vertical plane) following a substantial number of load variations together with an aesthetic demand for inclination of the wind turbine tower, and partly a structural demand regarding additional loads on turbine structure and foundation.

The structure shall be dimensioned for the situation where it is intermediately placed on the seabed on carrying plates/pile pattern before the pile driving.

Possible loads on the surrounding structure from pile driving shall be carefully assessed.

In relation to normal Danish geological formations, the following is emphasised:
When transferring experienced values from clay tills from other locations, due emphasis must be given to whether the clay tills do in fact have the same lime content as this may otherwise give rise to a modification of parameters.

Suction buckets

The foundation shall be analysed with respect to the following situations:

- Installation of the suction buckets.
- Plastic ultimate limit condition
- Operational limit condition
- Eigenfrequency analysis
- Shake-up

The buckets shall be dimensioned in such a way that they can be pressed down by their own weight or be sucked down by means of negative pressure inside the bucket. If the buckets are sucked down, it shall be demonstrated that the penetration resistance is lower than the driving force, and that the soil inside the bucket is not elevated apart from the contribution from displaced materials during installation.

As seabed scour along the circumference of the foundation is particularly critical towards the carrying capacity of this type of foundation, particular vigilance in relation hereto shall be exerted.

Geotechnical parameters

A table shall be prepared for the characteristics of the individual soil layers, which clearly states the relevant position(s) and which parameters of strength and deformation are used in the individual soil layers and cases.

Normally, the following geotechnical parameters are established, as defined in DS 415:
Classification parameters (γ', γ_s, I_p, particle distribution curve)
Strength parameters (φ', c', c_u, ”k”, α)
Deformation parameters (E', E_u, K, Q)
Dynamic parameters ($d\varepsilon/dt$, G_{dyn})

The Danish Energy Agency’s Approval Scheme for Wind Turbines / Risø December 2001