Deutsches Institut für Bautechnik [German Institute for Civil Engineering]

DIBt

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Guideline for Wind Turbines[1]

Effect and proof of stability for tower and foundation

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[2] NB: The revised edition of March 2015 does not contain any key changes to the October 2012 edition in terms of content but rather editorial corrections. Therefore, the designation of the 2012 edition has been retained with the additional wording "Revised version of March 2015"

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1 Scope

This Guideline applies to stability verifications for the tower and foundation of wind energy plants. Based on the determinations of DIN EN 61400-1, it contains provisions on actions on the entire wind energy plant, including the associated safety factors, which are to be used as the basis for determining the force variables from the machine acting on the tower and foundation (see Section 9.2.4) for the assessment thereof. The assessment of the machine itself is not the subject of this Guideline. Reference is made to the ninth regulation pertaining to the Product Safety Act (Machinery Regulation) (9th regulation of the Product Safety Act) transposing Directive RL 2006/42/EC of the European Parliament and of the Council of 17 May 2006 on machinery and amending Directive 95/16/EC (OJ L 157 of 09/06/2006, p. 24, L 76 of 16/03/2007, p. 35).

Furthermore, it may be necessary to observe the plant-related water protection requirements pursuant to § 62 of the Water Resources Management Act (Wasserhaushaltsgesetz – WHG).

DIN EN 61400-1 applies to the safety requirements on the machine.

Furthermore, the safety system must include two or more braking systems (mechanical, electrical or aerodynamic) which are suitable for bringing the rotor to a halt or idle from any operating condition. At least one braking system must be capable of keeping the system in an intrinsically safe state, even in the event of a mains power failure.

The application of the issues of DIN EN 61400-1 specified in the following is approved for determining the actions. In each case, the standard with all associated corrections and annexes shall be deemed valid, although the text of this Guideline shall only refer to the respective base document by specifying its date of issue (printed in boldface in the following).

DIN EN 61400-1:2011-08	**Alternatively: DIN EN 61400-1:2004-08**
	DIN EN 61400-1 Corrigendum 1:2005-12

The respective issue to be applied shall be applied in its entirety with respect to determining the actions. Mixing various issues is not permissible. This concerns such things as details regarding the load case definitions and evaluation methods. Any potential requirements present on the design, dimensioning and execution of the tower and foundation do not apply in connection with this Guideline.

Whenever this Guideline makes reference to DIN EN 61400-1 without specifying the date of issue, then the provisions of the issue in question, which was selected to be applied in its entirety, shall apply.

One exception concerns the determination of the effective turbulence within a single wind farm, which must be conducted in accordance with DIN EN 61400-1:2011-08.

Wind turbines with a rotor sweep of less than 200 m² and alternating voltage of under 1 000 V or DC voltage of 1 500 V may be verified in accordance with DIN EN 61400-2. In particular, the safety system must include two or more braking systems (mechanical, electrical or aerodynamic) which are suitable for bringing the rotor to a halt or idle from any operating condition for small wind turbines as well. At least one braking system must be capable of keeping the system in an intrinsically safe state, even in the event of a mains power failure.

The design, dimensioning and execution of the tower and foundation of wind turbines are based on the relevant Technical Building Regulations (Technische Baubestimmungen) for comparable structures such as antenna supports, chimneys and masts, etc. unless this Guideline specifies otherwise.

Furthermore, requirements concerning the inspection and maintenance of the turbine are made so that the stability of the tower and foundation is ensured over the intended design service life.

The Guideline does not take the particulars of wind turbines which are built in the open water of the North Sea and Baltic Sea (offshore turbines) into account. This Guideline also does not take the particularities of vertical axis turbines and braced systems into account. However, the provisions specified here can be applied accordingly to such turbines.

2 Terms and definitions

2.1 Definitions

The definitions of the following terms are to be observed in connection with the rules of this Guideline. They may differ from the definitions used in energy yield calculations and other bodies of rules.

- Wind turbine:

Plant which converts the kinetic energy of the wind into electrical energy

- Foundation and ground-structure interaction (ground torsion spring):

Steel, reinforced concrete or pre-stressed concrete component, including concrete base, which is in contact with the ground over the natural or filled-in terrain line and ground

- Tower:

The part of a wind turbine above the foundation which supports the machine, including any guy wires which may be present

– Steel tower: Tower consisting of one or more steel pipe segments
– Pre-stressed concrete tower: Tower made of pre-stressed in-situ concrete or prefabricated parts
– Hybrid tower: Reinforced concrete or pre-stressed concrete tower with fitted steel pipe tower

- Machine:

The mechanical part of the wind turbine on the tower, including the rotor blades, as well as the hub, shaft, gearbox, control and electrical components, generator, bearings and brakes

- Design service life:

The calculated duration used as the basis for the wind turbine design

- Rated output:

The maximum long-term output which results from the output curve.

- Rated rotation speed n_R:

Rotation speed of the rotor at rated wind speed

- Idle state:

A wind turbine's ready-to-operate state without power output, during which the rotor turns slowly

- Mean wind speed:

(10-minute average) of wind speed at a height z above ground

- Mean 50-year wind speed $v_{m50}(z)$:

mean wind speed at height z above ground, depending on the topography of the location, that is statistically achieved or exceeded on mean once every 50 years (corresponds to an annual exceedance probability of 0.02). Corresponds to $v_m(z)$ under DIN EN 1991-1-4, taking into account DIN EN 1991-1-4/NA

- Mean one-year wind speed $v_{m1}(z)$:

mean wind speed at height z above ground, depending on the topography of the location, that is statistically achieved or exceeded on mean once yearly

- 50-year gust speed $v_{p50}(z)$:

Extreme wind speed value (3-second average) at height z above ground, depending on the topography of the location, that is achieved or exceeded once every 50 years, as per DIN EN 1991-1-4/NA

- one-year gust speed $v_{p1}(z)$:

Extreme wind speed value (3-second average) at height z above ground, depending on the topography of the location, is achieved or exceeded once yearly.

- Basic wind speed v_b:

50-year wind at a height of 10 m over level, open terrain, determined over a time period of 10 minutes ($v_b = v_{b,0}$ da c_{season} and $c_{dir} = 1$, see DIN EN 1991-1-4 in conjunction with DIN EN 1991-1-4/NA)

- Annual mean wind speed v_{ave}:

Mean wind speed at hub height determined over several years

- Rated wind speed v_r:

The lowest mean wind speed at which the rated output is reached

- Cut-in wind speed v_{in}:

The lowest mean wind speed at which the wind turbine will run

- Cut-out wind speed v_{out}:

The greatest mean wind speed at which the wind turbine will run

2.2 Definitions

A	Area
a	Horizontal distance between the tower axes of two neighbouring wind turbines
cf	Aerodynamic force coefficient
$c_s c_d$	Structural coefficient
D	Rotor diameter
F	Force, load
f_0	Natural frequency
f_R	Excitation frequency of the running rotor
h	Height of the rotor centre point (hub height) above the terrain, tower height
I_T	Turbulence intensity
M	Momentum
m	Number of rotor blades, exponent of the S/N curve
m_E	Ice mass
N	Load cycle
n_R	Rotor's rated rotation speed
q	Dynamic pressure (back pressure)
R	Rotor radius
s	Dimensionless horizontal distance between the tower axes of two neighbouring turbines in relation to the rotor diameter
T_{ed}	Reference temperature
T_0	Duration of action
t_S	Depth of the rotor blade at the point under linear extrapolation of the front and rear edge
t_W	The greatest depth of the rotor blade in the vicinity of the root
v_b	Basic wind speed
v_{m50}	Mean 50-year wind speed
v_{m1}	Mean one-year wind speed
v_{p50}	50-year gust speed
v_{p1}	one-year gust speed
v_{hub}	Wind speed at hub height
v_{ave}	Annual mean wind speed at hub height
v_{out}	Cut-out wind speed at hub height
v_{out}	Cut-in wind speed at hub height

v_{hub} Rate wind speed at hub height

x, y, z Coordinates (see Figure 4)

α Terrain roughness exponent

β Angle of incidence of incoming wind

γ_F Partial safety coefficient for the action

γ_M Partial safety coefficient for the resistance

δ Logarithmic damping decrement

ϑ Ratio regarding the rotor blade depth, $\vartheta = t_s/t_w$.

ξ Dimensionless longitudinal ordinate on the rotor blade

ρ Air density

ρ_E Thickness of the ice

σ Stress

$\Delta\sigma$ Stress variation range

Indices

d Design values

k Characteristic values

1 one-year value

50 50-year value

2.3 Comparison of terms and designations

Table 1

DIN EN 61400-1:2004	DIN EN 61400-1:2011	DIBt Guideline
Designation/meaning of formula symbols	**Designation/meaning of formula symbols**	**Designation/meaning of formula symbols**
v_{ref} Reference wind speed Base parameters to define the type classes. Other relevant design parameters are derived from this. 10-minute average value of extreme wind speed **at hub height** with a recurrence period of 50 years.	v_{ref} Reference wind speed Base parameters to define the type classes. Other relevant design parameters are derived from this. 10-minute average value of extreme wind speed **at hub height** with a recurrence period of 50 years.	$v_{m50}(h)$ Mean 50-year wind speed Mean wind speed **at hub height** calculated in accordance with DIN EN 1991-1-4 + NA or Chapter 7.3.2.1, taking wind zone and terrain category into account, with a recurrence period of 50 years.
v_r Design wind speed	v_r Design wind speed	v_r Rated wind speed
Design output	Design output	Rated output
A (Abnormal [German: Anormal]) Nature of the design condition	A (Abnormal [German: Anormal]) Nature of the design condition	A (Abnormal [German: Außergewöhnlich]) Action combination group

Annex A provides a detailed comparison of wind speed designations (for information).

3 Construction documentation

The construction documents include the following:

A The wind turbine's technical data with the following information in particular:
1 Model name
2 Manufacturer
3 Configuration (type sheet)
4 Control and braking system
5 Rotor blade type
6 Operating data required for determining the actions and dimensioning the tower

B Overview of the turbine and layout, if applicable

C Construction description of the tower and foundation with the following specifications:
1 Wind speed zone (design and site, if applicable)
2 Design service life
3 Ground conditions

D Force variables for verifying the tower and foundation and other bases for dimensioning (see Section 9)

E Stability evidence for the tower and foundation (verifications in the limit state of bearing capacity and performance capability), including vibration examinations

F Design drawings for the tower and foundation with all of the necessary information and technical requirements for the execution of steel constructions (see Eurocode 3 series of standards: Design of Steel Structures) and reinforced and pre-stressed concrete constructions (see series of standards Eurocode 2: Design of concrete structures).

G Installation instructions (such as stress instructions, manufacturer's instructions for the foundation in accordance with DIN EN 13670)

H Expert opinion on the foundation (expert foundation report)

The following documents must also be present for wind turbines:

I Expert opinions in which special requirements on the construction and operation of the wind turbine must be formulated (if applicable)
1 Expert opinion confirming the force variable for the verification of the tower and foundation, rotor blades and machine construction (expert load report)
2 Expert opinion on the verifications of the safety equipment (expert safety report)
3 Expert opinion on the verifications of the rotor blades
4 Expert opinion on the verifications of the mechanical components and covering of the machine house, hub (expert machine report)
5 Expert opinion on the verifications for the electrical components and lightning protection

Other documents which must be evaluated by the expert who compiled the expert machine report:

J Operating instructions

K Commissioning protocol (pre-printed form)

L Maintenance requirement book (see Section 15)

4 Technical Building Regulations

Unless this Guideline determines otherwise, the Technical Building Regulations (Technische Baubestimmungen) shall apply, in particular with respect to the actions DIN EN 1991-1-1, -1-3 and -1-4; the basic standards of the DIN EN 1993 series of standards shall apply to steel structures, the series of standards DIN EN 1992 shall apply to reinforced concrete and pre-stressed concrete structures, and DIN EN 1997 shall apply to the foundation. All standards from the Eurocode series must always be applied in connection with their national annexes.

Furthermore, the provisions of DIN EN 1993-3-2, Chapter 5.2 may be applied to steel solid-walled towers.

5 Materials and execution

Only materials which comply with the Technical Building Regulations may be used. In accordance with building inspection regulations, the use of other materials requires special usability verification, i.e. by way of a general building inspection approval or consent on a case-by-case basis.

Reinforced and pre-stressed concrete components shall be executed in accordance with DIN EN 13670.

Tensioning procedures which are used for pre-stressing wind turbines must be approved for their intended purpose in the wind turbines or for this corresponding field of application.

6 Execution classes

In accordance with DIN EN 1090-2 Annex B, steel towers of wind turbines or parts of wind turbines made of steel are to be allocated to stress category SC2 and consequence class CC2. According to this, execution class EXC3 is considered to be the minimum requirement for wind turbines. Corresponding requirements for execution can be found in DIN EN 1090-2 Annex A.3.

7 Actions

7.1 General

Actions on wind turbines shall be assumed in accordance with DIN EN 61400-1. See also Section 1. Further actions under this section and action combinations under Section 8 must also be taken into account.

7.2 Inertia and gravity loads

7.2.1 Constant gravity loads (dead loads)

The characteristic values of the dead loads shall be determined using the calculated values pursuant to DIN EN 1991-1-1. If materials which are not specified in these standards are used, then their actual unit weights should be taken as the basis for the load determination.

7.2.2 Inertial forces from mass eccentricities

The rotor imbalances described in DIN EN 61400-1 shall be applied. Furthermore, the additional inertial forces from mass eccentricities resulting from ice loads should be determined for the case that one rotor blade is not covered with ice (see Section 7.4.6) if operation under ice loads cannot be ruled out with certainty.

7.2.3 Earthquake

Actions from earthquakes shall be observed in accordance with DIN EN 1998-1 including DIN EN 1998-1/NA; significance category 1 may be assumed in the process.

The overlap with the wind turbine loads based on load cases D.5 and D.6 is a simplified observation in this case. A more precise observation pursuant to DIN EN 61400-1:2011 may be conducted as an alternative.

7.3 Aerodynamic loads

7.3.1 General

The aerodynamic loads shall be determined in accordance with DIN EN 1991-1-4 in consideration of the special determinations of DIN EN 61400-1 and this Guideline.

The wind conditions pursuant to DIN EN 61400-1 shall generally apply. Deviations from DIN EN 61400-1 shall be specified below.

ρ = 1.225 kg/m^3 may be assumed as a calculated value for the air density, contrary to DIN EN 1991-1-4.

The wind speed should be assumed to act independently of the geographic direction (c_{dir} = 1).

7.3.2 Wind conditions

7.3.2.1 Extreme wind conditions

The mean 50-year wind speed, $v_{m50}(h)$, to be assumed in accordance with the wind zone of the wind turbine, must be determined in accordance with DIN EN 1991-1-4, including NA.

In general the elevation profile for the mean 50-year wind speed $v_{m50}(z)$ and turbulence $I_v(z)$ for rough terrain up to max. terrain category III are calculated using the formulas in accordance with Tables NA-B.2 and/or NA-B.4 in DIN EN 1991-1-4/NA.

Roughness greater than in terrain category III may not be applied.

For locations in terrain category I and terrain category II, the equations (GL 1) and (GL 2) may be used for simplification purposes.

The turbulence intensity may be simplified as assumed in the following:

$$I_V(z) = 0{,}128 \cdot \left(\frac{z}{10}\right)^{-0{,}05} \qquad \text{(GL 1)}$$

The associated elevation profile for the mean 50-year wind speed (including the mean wind profile for the turbulent wind field) may be assumed as follows:

$$v(z) = 1{,}15 \cdot v_{b,0} \cdot \left(\frac{z}{10}\right)^{0{,}121} \qquad \text{(GL 2)}$$

The values for the mean one-year wind, $v_{m1}(z)$ is determined from the 50-year wind, $v_{m50}(z)$ by multiplying it by the factor 0.8.

These determinations apply to the extreme wind model, meaning that the wind conditions must be adjusted accordingly for the corresponding load cases from DIN EN 61400-1.

7.3.2.2 Operating wind conditions

The annual mean wind speed at hub height, v_{ave}, shall be assumed according to GL (GL 3) and/or GL (GL 4), unless lower values can be verified specifically to the site.

$$v_{ave} = 0{,}18 \cdot v_{m50}(h) \qquad \text{(GL 3)}$$

$$v_{ave} = 0{,}20 \cdot v_{m50}(h) \qquad \text{(GL 4)}$$

Equation (GL 4) applies to North Sea islands.

The annual mean wind speed v_{ave} in WZ1 and WZ 2 is to be set at the value from WZ 3.

It is recommended to verify the turbine for the turbulence intensity of turbulence category A in accordance with DIN EN 61400-1 in order to ensure that all German locations are covered when the individual turbines are constructed. If this is deviated from, it will be necessary to assess the turbulence based on the site; see Chapter 16.

It is permissible to specify operating conditions and extreme wind conditions, including the turbulence category, independently of wind zones in accordance with DIN EN 1991-1-4 and to verify them in the scope of a type test.

The type test and expert load report must contain these specifications. The wind zones and terrain categories covered must be explicitly specified in the process.

7.3.3 Influences of neighbouring physical structures, terrain roughness and topography on the suitability of the site

Site-specific examinations must be conducted to determine whether local increases in turbulence due to influences of neighbouring wind turbines or the site's wind conditions will jeopardise the suitability of the site.

The suitability of the site must be inspected in accordance with Chapter 16, “Site suitability of wind turbines.”

The loads acting on the wind turbine must at least be determined using the site-specific wind parameter values.

If the turbine is designed for turbulence category A, the impact of local increased turbulence on the suitability of the site need not be investigated if the following conditions are met:

$$a \geq 8D \quad \text{für} \quad v_{m50}(h) \leq 40 m/s \qquad \text{(GL 5)}$$

$$a \geq 5D \quad \text{für} \quad v_{m50}(h) \geq 45 m/s \qquad \text{(GL 6)}$$

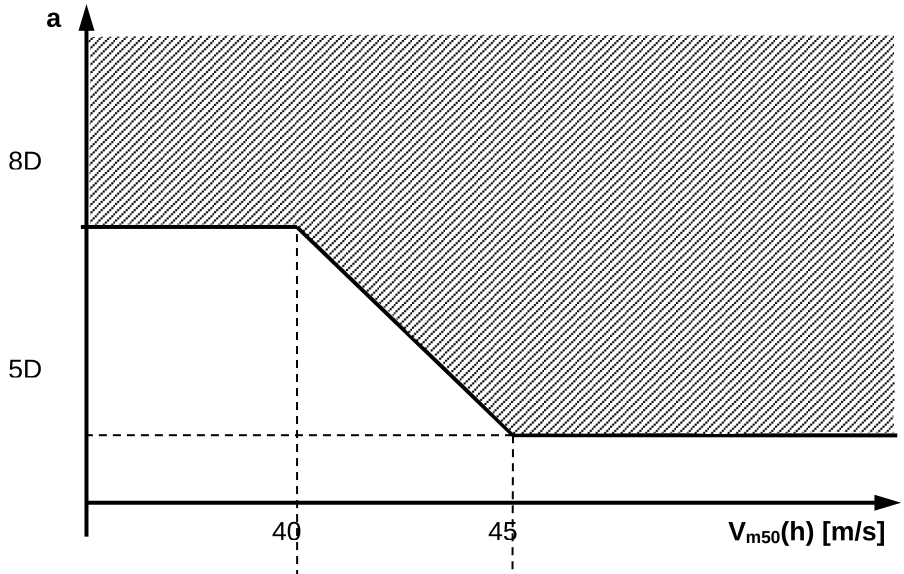


Figure 1: Diagram of requisite distances (shaded area)

Where:

a is the distance between the tower axes of two neighbouring wind turbines

D is the respective larger rotor diameter

$v_{m50}(h)$ is the mean 50-year wind at hub height

a should be interpolated linearly for interim values of $v_{m50}(h)$. The combinations of wind speed and terrain category which appear in Germany have already been taken into account in the process.

7.3.4 Wind loads for the state during installation or maintenance

For the examination of the states during installation, the wind speed $v_{b,0}$ and/or the dynamic pressure resulting from the wind speed may be reduced depending on the duration of said state as well as the protective measures selected in accordance with DIN EN 1991-1-4, taking DIN EN 1991-1-4/NA into account.

The maximum permissible mean wind speed must be specified by the manufacturer for the examination of the maintenance conditions. It must be ensured that the maintenance work is only conducted up to the maximum mean wind speed specified by the manufacturer.

The wind speed must be suitably increased for determining the loads in order to achieve a sufficient safety distance for the permissible mean wind speed. The following values should be assumed for this purpose:

- When applying a deterministic wind field, it is necessary to take an EOG (= extreme operating gust) in accordance with DIN EN 61400-1:2006 into account, based on a mean wind speed of 10 m/s above the permissible mean wind speed.
- When applying a turbulent wind field, it is necessary to increase the mean wind speed by 5 m/s in comparison with the permissible mean wind speed.

7.3.5 Wind load in the event of ice accretion

In the event of ice accretion, the wind load must be determined by the reference area of the bearing structure which has been enlarged by the ice accretion on all sides (see Section 7.4.6). In the case of timber frames, it will be necessary to apply the aerodynamic force coefficients accordingly to the block coefficient which has been altered by the ice covering.

7.3.6 Actions from vortex shedding

Actions from vortex shedding may lead to vibrations at a right angle to the wind direction (transverse vibrations), especially with towers with circular or nearly circular cross-sections, see Section 9.4.

7.4 Other actions

7.4.1 Imperfections, actions from uneven settling

In addition to elastic deformations of the supporting structure and building ground under the action of exterior loads, the following undesired deviations of the tower axis from the vertical position must be taken into account as constant actions:

- Tilting of the tower axis with 5 mm/m to cover imprecision in manufacturing and installation and effects from solar radiation from a single side
- Tilting resulting from uneven settling of the building ground or change to the support conditions[3]

Actions from imperfections and uneven settling must be added to the actions acting in an unfavourable direction which result from the total dynamic calculation.

7.4.2 Pre-stressing force

The pre-stressing of concrete structures is taken into account in accordance with DIN EN 1992-1-1.

7.4.3 Earth pressure

Earth pressures with unfavourable effects (such as with locations on slopes) must be taken into account.

7.4.4 Foundation water pressure

Foundation water pressure with unfavourable actions must be taken into account. A design water level at the height of ground level shall be applied if no other values are documented. Correspondingly higher water levels must be taken into account in flood areas.

The design water level taken as the basis for type calculations shall be specified in the plan documents.

NB: The foundation water pressure is to be applied as a constant load. If the water level up to ground level is applied, then it can be calculated with $\gamma_F = 1.1$.

7.4.5 Thermal action

For towers made of pre-stressed concrete, the following temperature proportions must be taken into account in order to record effects from temperature in comparison with the installation temperature of 15 °C and from effects of solar radiation (Figure 2):

- a proportion of $\Delta T_{N,1} = \pm 35$ K constantly acting over the circumference and cross-section thickness
- a constant proportion, over the cross-section thickness, of $\Delta T_{N,2} = \pm 15$ K over the circumference continuing in a conical path along a circumferential sector of 180°
- a linearly varying temperature difference of $\Delta T_M = \pm 15$ K over the wall thickness in a longitudinal and circular direction

[3] For type calculations, a settling difference between the outer edges of the foundation of 40 mm or a tilting of the tower of 3 mm/m may be assumed as a reasonable value for this action. The correctness of this assumption in the specific case should be confirmed by means of an expert soil report.

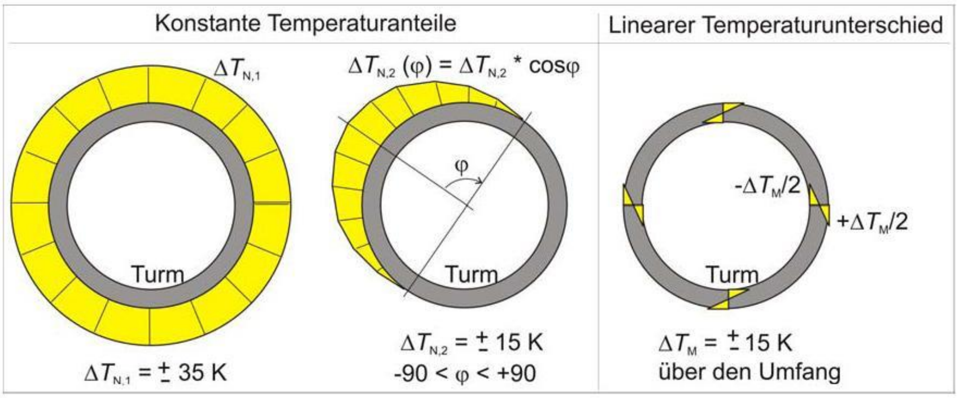


Figure 2: Representation of temperature proportions $\Delta T_{N,1}$, $\Delta T_{N,2}$ and ΔT_M

Konstante Temperaturanteile	Constant temperature proportions
Linearer Temperaturunterschied	Linear temperature difference
Turm	Tower
über den Umfang	Over the circumference

The turbine's operation may cause a greater linearly variable temperature difference to set in, which must be taken into account in place of $\Delta T_M = \pm 15$ K.

The temperature load cases (Equation (GL 7)) should be superimposed with the characteristic value of Group N (Table 2: LF D.1) in the limit state of bearing capacity. The combination coefficient $\psi_{temp} = 0.6$ should be applied in the process.

In the limit state of bearing capacity, the temperature load case should be superimposed with the associated load case pursuant to Table 2 depending on the type of pre-stressing:

<u>Pre-stressed concrete without composite material</u>:	quasi-constant combination:	D.3
<u>Pre-stressed concrete with composite material</u>:	common combination:	D.2

$$\text{Max.} \begin{cases} \Delta T_{N,1} + \Delta T_{N,2} \\ \Delta T_M \\ (\Delta T_{N,1} + \Delta T_{N,2}) + 0.75\ \Delta T_M \\ 0.35\ (\Delta T_{N,1} + \Delta T_{N,2}) + \Delta T_M \end{cases} \qquad \text{(GL 7)}$$

The combination coefficient $\psi_{temp} = 0.6$ should be applied in both cases.

During the superimposition, the temperature shares should each be applied individually or in combination according to Equation (GL 7).

7.4.6 Ice loads

In the case of idle turbines, the ice loads for all structural parts exposed to weathering shall be determined in accordance with DIN 1055-5.

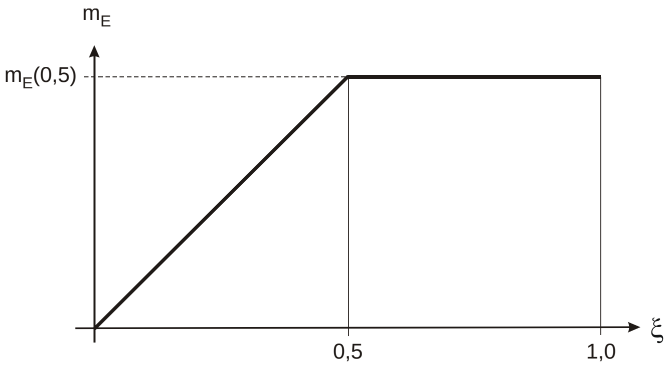
If operation under ice loads cannot be ruled out with certainty, then the ice accretion on the rotor blades must be taken into account using an assumed mass distributed over the length of the rotor blade mE(□) in accordance with

Figure 3 and Equation (GL 8). The ice mass should be assumed to be acting on the front profile edge of the rotor blade.

$$m_E(0{,}5) = c_E(R) \cdot \vartheta(1+\vartheta) \cdot \rho_E \cdot t_w^2 \qquad \text{(GL 8)}$$

Where:

$$c_E(R) = 0{,}3 \cdot e^{-0{,}32R} + 0{,}00675 \qquad \text{(GL 9)}$$



ξ: is the dimensionless longitudinal ordinate on the rotor blade

Figure 3: Ice accretion on rotor blades for turbines in operation

8 Action combinations

The exterior conditions and actions specified in DIN EN 61400-1 in observance of the additional specifications from Table 2 and

Table 3 should be combined to determine the stresses; see paragraph 1 of DIN EN 61400-1.

The respective partial safety coefficients to be applied pursuant to Table 5 or Table 6 are defined with the correspondingly allocated action combination groups.

For the action combination groups designated with F (fatigue), it is only necessary to record the fatigue safety verification. The actions of the individual operating states are accumulated to accomplish this.

The combinations of actions of Groups N (normal and extreme), A (extraordinary) and T (transport and construction) must be examined separately.

The extreme wind speed model (EWM) is a turbulent extreme wind model based on the mean wind speed (50-year wind $v_{m50}(z)$ or one-year wind $v_{m1}(z)$).

Alternatively, a stationary extreme wind speed model based on gust speed , (50-year gust speed $v_{p50}(z)$ or one-year gust speed $v_{p1}(z)$) may be applied. The values for the one-year gust speed, $v_{p1}(z)$ is determined from the 50-year gust speed, $v_{p50}(z)$ by multiplying it by the factor 0.8.

The temperature load cases pursuant to Chapter 7.4.5 should be superimposed with the characteristic value of Group N (Table 2: LF D.1) in the limit state of bearing capacity.

The temperature loads must be taken into account when verifying the decompression and/or when verifying the crack width control in accordance with Section 11.2.5.

Table 2: Additional load cases

Operating conditions (Reference to DIN EN 61400-1)	DLC	Wind conditions	Other conditions	Action combination groups and/or load case groups to be evaluated
Various operating conditions according to the load cases to be evaluated	D.1	NB: The specified load cases are used for verifications at the limit state of	Characteristic value: Design value of all evaluated load cases (evaluation in accordance with DIN EN 61400-1)	N and T (not including earthquakes)

	D.2	performance capability. $V_{in} \leq V_{hub} \leq V_{out}$	<u>Common actions:</u> Stresses with a probability of being exceeded of p = 10^{-4} (equivalent to 17.5 hours in 20 years)	The evaluation comprises all load cases of Table 3
	D.3		<u>Quasi-constant actions:</u> Stresses with a probability of being exceeded of p = 10^{-2} (equivalent to 1 750 hours in 20 years)	
1. Production mode	D.4	NWP $v_{hub} = v_r$	Ice loads (see 7.4.6)	F
	D.5	NWP $v_{hub} = v_r$	Earthquake	A
5. Emergency cut-out	D.6	NWP $v_{hub} = v_r$	Earthquake	A
6. Parked (still or idling)	D.7	EWM return period: 50 years $v_{hub} = v_{m50}(h)$	Yaw error β=0°	N(γ_F=1.5)
	D.8	EWM return period: 50 years $v_{hub} = v_{m50}(h)$	Analogous to DLC 6.1 pursuant to DIN EN 61400-1, except with wind conditions pursuant to Section 7.3.2	N
	D.9	EWM return period: 50 years $v_{hub} = v_{m50}(h)$	Analogous to DLC 6.2 pursuant to DIN EN 61400-1, except with wind conditions pursuant to Section 7.3.2	A
	D.10	EWM return period: 1 year $v_{hub} = v_{m1}(h)$	Analogous to DLC 6.3 pursuant to DIN EN 61400-1, except with wind conditions pursuant to Section 7.3.2	N
7. Parked (idling with malfunction)	D.11	EWM return period: 1 year $v_{hub} = v_{m1}(h)$	Analogous to DLC 7.1 pursuant to DIN EN 61400-1, except with wind conditions pursuant to Section 7.3.2	A
8. Transport, maintenance, repair	D.12	EWM return period: 1 year $v_{hub} = v_{m1}(h)$	Analogous to DLC 8.2 pursuant to DIN EN 61400-1, except with wind conditions pursuant to Section 7.3.2	A

Table 3: Load cases for verifying fatigue safety

Operating conditions (Reference to DIN EN 61400-1)	**DLC (DIN EN 61400-1: 2006)**	**DLC (DIN EN 61400-1: 2004)**	**Wind conditions**	**Other conditions**	**Frequency to be applied**
1. Production mode	D.4	D.4	NWP $v_{hub} = v_r$	Ice loads	7 days per year
	1.2	1.2	NTM $v_{in} \leq v_{hub} \leq v_{out}$		According to wind speed distribution
2. Production mode with appearance of a malfunction	2.4	2.3	NTM $v_{in} \leq v_{hub} \leq v_{out}$	1. Overspeed @ v_r 1. Overspeed @ v_{out}	7 times a year 3 times a year
			NTM $v_{in} \leq v_{hub} \leq v_{out}$	Operation with extreme yaw error	24 hours a year
			NTM	Mains power failure	20 times a year, application of various wind speeds in accordance with the wind speed distribution

3. Start	3.1	3.1	NWP	Start @ v_{in} Start @ v_r Start @ v_{out}	1 000 times a year 50 times a year 50 times a year
4. Stop	4.1	4.1	NWP	Stop @ v_{in} Stop @ v_r Stop @ v_{out}	1 000 times a year 50 times a year 50 times a year
6. Parked (still or idling)	6.4	6.2	NTM $v_{hub} \leq 0.7$ $v_{m50}(h)_{,0}$		According to wind speed distribution

Note:

- It may be necessary to take additional load cases or other frequencies into consideration for the fatigue evaluation according to turbine concept (regulation, operation management, maintenance, etc.).
- Load case D.4 will have to be taken into account if operation under ice loads cannot be ruled out with certainty.

9 Determining the design force variables

9.1 General

The force variables for designing the tower and foundation shall be determined using a total dynamic calculation in observance of the rules pursuant to Section 9.2.

Contrary to this, a simplified calculation of the tower structure may be conducted according to Section 9.3 for horizontal axis turbines if it is ensured that there is a sufficient distance between the tower's natural frequencies $f_{0,n}$ and the excitation frequencies f_R and/or $f_{R,m}$ during continuous operation in accordance with Equation(GL 10) and(GL 11)Equation . The simplified procedure may also be used for turbines which are in “out of order” state.

In continuous operation, it is necessary to verify a sufficient distance between the tower's natural frequencies $f_{0,n}$ and the excitation frequencies f_R and/or $f_{R,m}$ in accordance with Equations (GL 10) and (GL 11).

$$\frac{f_R}{f_{0,1}} \leq 0{,}95 \qquad \text{(GL 10)}$$

$$\frac{f_{R,m}}{f_{0,n}} \leq 0{,}95 \quad \text{or} \quad \frac{f_{R,m}}{f_{0,n}} \geq 1{,}05 \qquad \text{(GL 11)}$$

Where:

f_R is the maximum rotation speed of the rotor in normal operating range

$f_{0,1}$ is the first natural frequency of the tower

$f_{R,m}$ is the throughput rate of the m rotor blades

$f_{0,n}$ is the n^{th} natural frequency of the tower

The number n of the natural frequencies to be determined must be selected to be at least so large that the greatest calculated natural frequency is at least 20 % greater than the blade throughput rate.

The natural frequencies of the tower should be determined and specified for the vibration system to be examined, assuming elastic material behaviour. The influence of the foundation must also be taken into consideration in the process.

The calculated values should be varied by ± 5 % in order to take uncertainties in the calculation of the natural frequencies into account.

Operational vibration monitoring must be conducted for turbines for which Equation (GL 10) and Equation (GL 11) are not fulfilled during continuous operation, meaning that they are being operated near the resonance range.

9.2 Total dynamic calculation

9.2.1 General

Stresses on the entire system should be determined using a total dynamic calculation according to the elasticity theory. It must be taken into consideration that action components for certain verifications may also have favourable effects. The individual component of the force variables generally do not have an in-phase progression, meaning that the least favourable points in time must be picked out here.

The partial safety coefficient procedure may not be used for a total dynamic calculation in the period. In this case, it will be necessary to proceed in accordance with Section 10.2.

9.2.2 Requirements

The following influence parameters with respect to wind model, aerodynamic, structure dynamics and function must be taken into account for a total dynamic calculation of the wind turbine.

- **Wind model**

The wind model must meet the conditions pursuant to DIN EN 61400-1. In addition, the following should be observed:

The influences from tower shadows may be estimated according to the potential theory.

NOTE: Satisfactory results can generally be achieved at a wind speed sampling rate of 4 per second and a load sampling rate of 20 per second.

A number of at least 10 · 10 points (depending on diameter) in relation to the rotor is recommended.

A simulation time of 600 seconds per wind speed class at a class breadth of approximate 2.0 m/s is recommended for fatigue safety verifications.

- **Aerodynamics**

The following influences must additionally be taken into account for calculating aerodynamic loads:

- Hub and tip vortex
- Blade displacement, oscillation, etc.
- Dynamic stall
- Dynamic wake

NOTE: Satisfactory results can generally be achieved by applying the blade element theory at 15 elements per rotor blade.

- **Structural dynamics**

The following influences must additionally be taken into account when examining the structural dynamics:

- Effect of centrifugal force on the rotor blade's stiffness
- Torsional stiffness of the power train
- Elastic bearing of the machine
- Generator stiffness and damping (the mesh can be seen as infinitely stiff)
- Foundation with soil properties

NOTE: It will generally suffice to factor in only the natural frequencies which are under 5 Hz.

- **Function**

The controller properties should be represented to approximate reality. The time progressions, such as those during yawing and braking, should be taken into account in the process.

A yaw error based on the operational management, albeit a yaw error angle in a range between at least $\beta = 0°$ and $\beta = \pm 8°$ must be taken into account for all action combinations, except DLC 6.1, 6.2 and 6.3 pursuant to DIN EN 61400-1.
The yaw error angle pursuant to DIN EN 61400-1 must be assumed for the action combinations DLC 6.1, 6.2 and 6.3.

9.2.3 Accounting for the foundation

The rigidity of the building ground is of particular importance for the operating behaviour of wind turbines. The interaction between the ground and structure must be taken into account at all times. In the case of a total dynamic calculation, this can be determined with reasonable approximation using springs for the rotation and horizontal displacement, regardless of the frequency, with dynamic soil characteristics (see Section 12.2.1).

The specification of the spring stiffness and/or soil characteristics for the total dynamic calculation can be determined in observance of the stiffness of the foundation body and the shear lags occurring in the ground. In the process, it will be necessary to determine the shear lags for the load level which results from load case D.3 (quasi-constant actions).

The stiffness module can be approximately estimated for very small expansions $E_{s,max}$, depending on the rigidity module for static loads, whereby, without more detailed verification, the lower value of the specified range should be applied.

The recommendations of the “Building Ground Dynamics” Work Group (Arbeitskreis Baugrunddynamik)[4] contain information on the value of dynamic soil characteristics.

9.2.4 Force variables

The total dynamic calculation results in the time progressions of all force variables for the action combinations examined in the cross-sections relevant to the design of the tower and foundation. These force variables must be determined for the verifications in the limit state of bearing capacity and performance capability.

Table 4: Depiction of the force variables for verifications in the limit state of bearing capacity and/or performance capability (see Figure 4 for description of the coordinate axes)

Interface:											
Limit state:	Bearing capacity / Performance capability										
	DLC *)	$v(h)$ [m/s]	β °	F_x [kN]	F_y [kN]	F_z [kN]	M_x [kNm]	M_y [kNm]	M_z [kNm]	F_{res} [kNm]	M_{res} [kNm]
max F_x	....										
min F_x											
max F_y											
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max M_y											
min M_y											
max M_z											
min M_z											
max F_{res}											
max M_{res}											

*) Action combination, see Section 8

For simplification purposes, only the extreme values of the force variables together with the remaining simultaneously occurring force variables for the cross-sections under consideration may be specified for the verification against strength and stability failure as well as for the verification in the limit state of performance capability (see Table 4).

[4] Recommendations of the Arbeitskreis "Baugrunddynamik" (‘Building Ground Dynamics’ Work Group), Deutsche Gesellschaft für Geotechnik e.V. (DGGT), Berlin 2002

The force variables for the fatigue safety verification may generally[5] be simplified in the form of stress collectives, but the associated mean values may also be specified, if necessary (see Section 9.6.2).

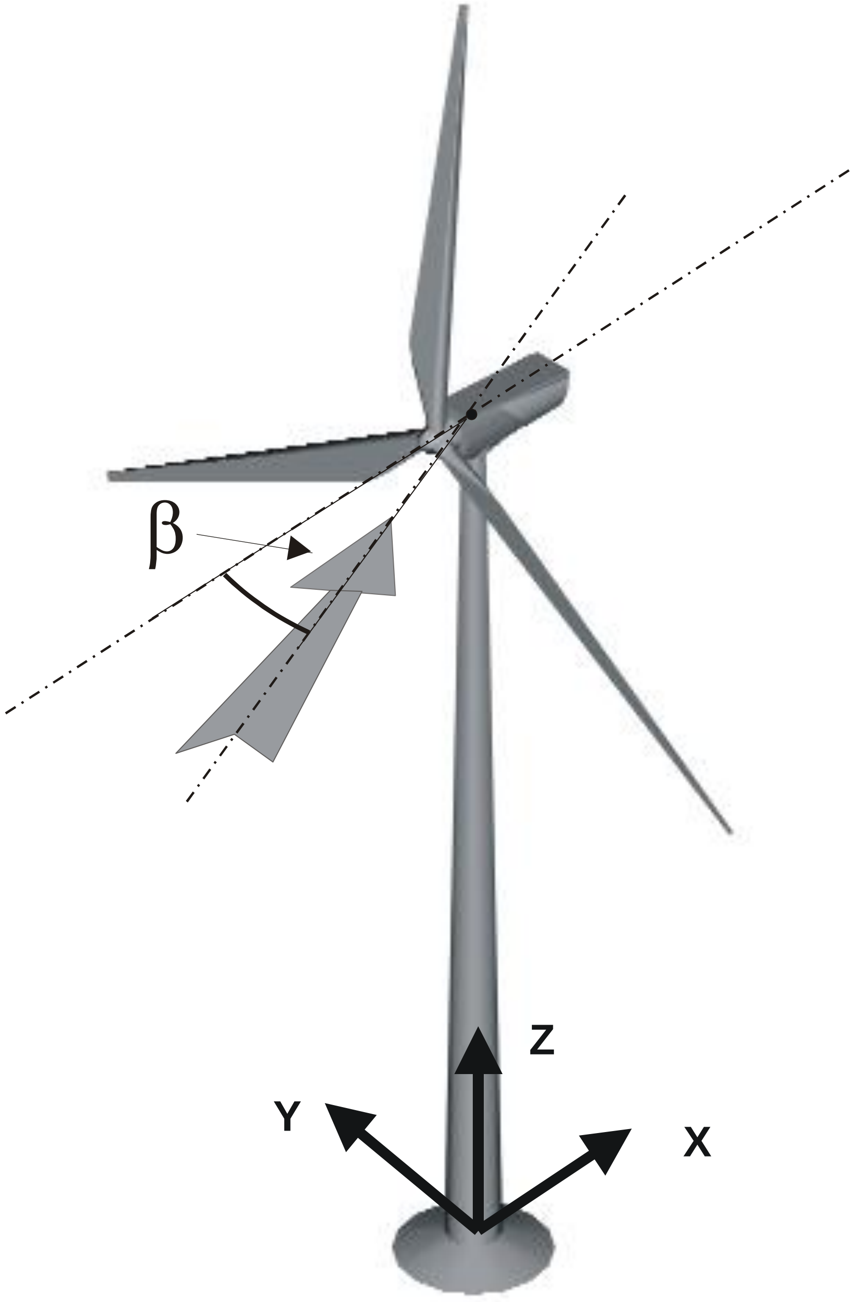

Figure 4: Coordinate system for the tower

9.3 Simplified calculation

9.3.1 General

The simplified calculation may only be used for tower structure verifications within the scope of the provisions of Section 9.1. In the process, the force variables determined from the total dynamic calculation and specified according to Table 4 must be used as actions on the tower at the machine/tower interface. The force variables at all other points on the tower will then be derived from these actions. In the process, the wind load on the tower of the respective action combination must be taken into account based on magnitude and direction (see Section 9.6.2).

[5] See Section 8 (for example) for exceptions

For simplification purposes, all action components may be assumed to be acting simultaneously at their maximum value, or, if their action is favourable, to be acting at their minimum value.

The actions at the machine/tower interface may also be used for other tower variants, as long as said tower variants possess at least the same bending and flexural stiffness and also meet the condition according to Equation (GL 10) in continuous operation.

If the simplified calculation is used, then the machine's masses and mass moments of inertia as well as the tower's natural frequencies which were used as the basis of the calculation must also be specified, in addition to the force variables at the machine/tower interface pursuant to Section 9.2.4.

9.3.2 Wind-caused vibrations of the tower in the wind's direction

For verifications pursuant to Section 9.3.1 for turbines in “out of order” condition, the vibrating action of the tower in the wind's direction caused by the gustiness of the wind must be determined by applying a static substitute load. When using the turbulent extreme wind model EWM in a quasi-static calculation, the wind load acting directly on the tower in the wind's direction resulting from gust speed (3 s mean value of 50-year gust speed $v_{p50}(z)$ or one-year gust speed $v_{p1}(z)$) must be multiplied with the structural coefficient $c_s c_d$. An equivalent procedure to determine $c_s c_d$ is specified in DIN EN 1991-1-4, taking DIN EN 1991-1-4/NA into account.

Where tower structures not susceptible to vibration under DIN EN 1991-1-4 are used with the stationary extreme wind model EWM, based on gust speed (3 s mean value of 50-year gust speed $v_{p50}(z)$ or one-year gust speed $v_{p1}(z)$), the structural coefficient $c_s c_d = 1$ may be assumed.

For verifications pursuant to Section 9.6.1 for turbines in “operational” state, the vibrating action of the tower in the wind's direction caused by the gustiness of the wind may be disregarded, that is, the structural coefficient $c_s c_d = 1$ may be assumed.

9.4 Transverse vibrations caused by vortices

The stresses on towers with circular or roughly circular cross-sections resulting from vibrations caused by vortices at a right angle to the wind's direction (transverse vibrations) should be determined according to the procedure specified in DIN EN 1991-1-4.

The damage resulting from transverse vibrations caused by vortices may be disregarded up to a value of D=0.10. Otherwise, the damage resulting from transverse vibrations caused by vortices and the damage in the transverse direction resulting from the load cases defined in

Table 3 must be added for the fatigue safety verification.

The aerodynamic damping (see 9.5) may not be applied when calculating the stresses resulting from transverse vibrations caused by vortices.

9.5 Logarithmic damping decrement

The total damping is made up of the two proportions structural damping and aerodynamic damping (however, see 9.4). In the process, the logarithmic decrement δ for the total damping results in:

$$\delta = \delta_s + \delta_a \qquad \text{(GL 12)}$$

Where:
δ_s is the logarithmic decrement of the structural damping
δ_a is the logarithmic decrement of the aerodynamic damping

Where no more precise values are recorded, the following may be assumed as the logarithmic decrement for structural damping:
in the case of steel towers $\delta_s = 0.015$
in the case of pre-stressed concrete towers $\delta_s = 0.04$
angenommen werden.

Hybrid towers will require more precise considerations in observance of the geometry, material and intrinsic shapes.

The logarithmic decrement of the aerodynamic damping δ_a may be assumed to be $\delta_a = 0.05$ for all types of towers for determining actions from earthquakes according to the modal method, unless a more precise calculation is conducted.

9.6 Force variables for the fatigue safety verification

9.6.1 Requirements

The actions pursuant to DIN EN 61400-1 and Section 7, the influence parameters pursuant to Section 9.2.2 and the following specifications must be taken into account in determining the force variables for the fatigue safety verification:

- The start-up and normal shutdown procedures must be assumed in the arrangement with frequencies pursuant to
- Table 3 in observance of the dynamic increases when passing through the tower resonance.
- If operation under ice loads cannot be excluded with certainty, then ice loads pursuant to Section 7.4.6 should be assumed for 7 days a year at rated output, in which 1 rotor blade is not covered with ice and the other ones are covered with an ice mass of 50 % according to
- Figure 3.
- Unless determined otherwise, the duration of action of the force variables from vortex shedding may be assumed with the following values:
  - 0.5 years for the installation state without the machine
  - 1 year for the idle and maintenance state with the machine
- The turbine's design service life should be assumed to be at least 20 years.

9.6.2 Stress collectives

If the fatigue safety verification is conducted on the basis of stress collectives, then these stress collectives must be determined via calculation for the cross-sections in question by simulating the decisive requirements for the fatigue in accordance with Section 9.6.1 and, if necessary, supported by measurements in accordance with IEC TS 61400-13. In the process, the vibration widths of the force variables should be superimposed in an unfavourable manner.

For simplification purposes, the collectives may be represented as an envelope (i.e. in trapezoidal form) of the stress collectives contained in the simulation. Uniform load cycle numbers should be defined for all action components in the process. The associated average values must be specified.

NOTE: In general (if $\Delta My > \Delta Mx$) it will suffice to factor in the action components rotor thrust F_x, pitch moment M_y and tower torsional moment M_z. Here pitch and tower torsional moment may be assumed to have a 90° phase shift.

10 Safety concept

10.1 General

The verifications shall be conducted for different limit states by proceeding with the use of partial safety coefficients. The structure will no longer meet the design requirements if these limit states are exceeded. These limit states are:

- Limit states of bearing capacity
- Limit states of performance capability.

10.2 Limit states of bearing capacity

For a total dynamic calculation (in accordance with Section 9.2) the actions must be assumed at $\gamma_F = 1.0$. If it is not possible to differentiate individual action proportions in the force variables, then the structural integrity verifications must be conducted with force variables multiplied by γ_F, in which the largest partial safety coefficient of the action combination group in question must be applied in accordance with Table 5 or Table 6.

Table 5: Partial safety coefficients γ_F of the actions for verifications in the limit states of bearing capacity for verifications in accordance with DIN EN 61400-1:2004

Action	Action combination group		
	N normal and extreme	A Abnormal	T Transport/construction
Inertia and gravity loads Unfavourable	1.35*)	1.1	1.25
Favourable	1.0	1.0	1.0
Pre-stressing **)	1.0	1.0	1.0
Wind loads	1.35***)	1.1	1.5
Functional forces	1.35	1.1	1.5
Thermal action	1.35	–	–
Earthquake	–	1.0	–

*) If methods such as weighing the mechanical parts of the turbine verify that the actual unit weights do not differ from the assumed unit weights by more than 5 %, then γ_F = 1.1 may be used for the calculation.

**) Possible pre-stressing dispersions must be taken into account at the limit state of performance capability pursuant to DIN EN 1992-1-1, 5.10.9. The value r_{inf}, r_{sup} must be applied at the limit state of fatigue pursuant to DIN EN 1992-1-1, 5.10.9.

***) The shearing forces for the tower and foundation in action combination DLC 6.1 pursuant to DIN EN 61400-1 must be determined with γ_F = 1.35 as well as γ_F = 1.5, in which it is not necessary to take cross-flow (angle of incidence of incoming wind β = 0, see DLC D.7 pursuant to Table 2) into account in the case of γ_F = 1.5. The least favourable force variable combination of the two variants shall be the decisive one.

Table 6: Partial safety coefficients γ_F of the actions for verifications in the limit states of bearing capacity for verifications in accordance with DIN EN 61400-1:2011

	Unfavourable loads Type of design state (see Table 3)			**Favourable[1)] loads**
Action	**Normal (N)**	**Abnormal (A)**	**Transport and construction (T)**	**All design states**
Inertial and gravity loads, wind loads, functional forces	1.35*), ***)	1.1	1.5	0.9
Pre-stressing **)	1.0	1.0	1.0	0.9
Thermal action	1.35	--	--	0.9
	--	1.0	--	1.0

*) For design load case DLC 1.1, it will be necessary to assume a partial safety coefficient of f =1.25 for the loads for load calculation via statistical extrapolation for wind speeds between Vin and Vout.If the characteristic value for the force of gravity Fgravity for normal design states can be calculated for the relevant design state and gravitation is an unfavourable load, then the partial safety coefficient for the combined stress of gravity and other influences may be assumed as follows: If the characteristic value for the force of gravity $F_{gravity}$ for normal design states can be calculated for the relevant design state and gravitation is an unfavourable load, then the partial safety coefficient for the combined stress of gravity and other influences may be assumed as follows:

$$\gamma_t = 1{,}1 + \varphi\,\zeta^2$$

$$\varphi = \begin{cases} 0{,}15 & for\ DLC1.1 \\ 0{,}25 & other\ wise \end{cases}$$

$$\zeta = \begin{cases} 1 - \left|\frac{F_{gravity}}{F_k}\right| ; & |F_{gravity}| \le |F_k| \\ 0 ; & |F_{gravity}| \ge |F_k| \end{cases}$$

1) Pre-stressing and gravity loads which considerably reduce the overall stress are favourable loads.

**)	Possible pre-stressing dispersions must be taken into account at the limit state of performance capability pursuant to DIN EN 1992-1-1, 5.10.9. The value r_{inf}, r_{sup} must be applied at the limit state of fatigue pursuant to DIN EN 1992-1-1, 5.10.9. The value $\gamma_F = \gamma_P = 0.9$ relates to determining actions pursuant to DIN EN 61400-1:2011. DIN EN 1992-1-1, 2.4.2.2 and 5.10.8 shall apply for the design of concrete structures at the limit state of bearing capacity. Accordingly $\gamma_{,\square} = 1.0$ generally applies instead of Table 6.
***)	The shearing forces for the tower and foundation in action combination DLC 6.1 pursuant to DIN EN 61400-1 must be determined with $\gamma_F = 1.35$ as well as $\gamma_F = 1.5$, in which it is not necessary to take cross-flow (angle of incidence of incoming wind $\beta = 0$, see DLC D.7 pursuant to Table 2) into account in the case of $\gamma_F = 1.5$. The least favourable force variable combination of the two variants shall be the decisive one.

For verifications against strength and stability failure, it will be necessary to factor in the increase of the force variables resulting from non-linear influences (such as second order theory, condition II). In the case of a total dynamic calculation with dynamic soil characteristics, additional effects from second order theory will result during these verifications. These additional effects must be determined using static soil characteristics which result for a load level under characteristic actions (load case D.1).

$\gamma_F = 1.0$ must be used for the verification against fatigue.

The following verifications must be conducted at the limit states of bearing capacity:

Verification against:

- **strength failure** in accordance with Section 11.1.2
- **stability failure** in accordance with Section 11.1.3
- **fatigue** in accordance with Section 11.1.4

The recommendations of the "Building Ground Dynamics" Work Group (Arbeitskreis Baugrunddynamik) contain information on the value of dynamic soil characteristics[6].

10.3 Limit states of performance capability

The design values of the actions for the verifications must be determined at the limit states of performance capability with the characteristic values ($\gamma_F = 1.0$).

The following verifications must be conducted at the limit states of performance capability:

Verification of

- **deformation limitation** in accordance with Section 11.2.3
- **stress limitation** in accordance with Section 11.2.4
- **crack width control** in accordance with Section 11.2.5

11 Verifications for the tower

11.1 Verifications at the limit states of bearing capacity

11.1.1 Partial safety coefficients

The resistances must be determined in observance of the partial safety coefficients γ_M in accordance with the relevant bodies of rules (see Section 4). See Section 11.1.4 regarding the partial safety coefficients γ_M for the verification against fatigue.

11.1.2 Strength failure

The verifications must be conducted with the least favourable of all action combinations of Groups N, A and T.

DIN EN 1992-1-1 shall be applied for the verification for reinforced concrete and pre-stressed concrete. In the process, the force variables of the tower shaft may be determined according to the pipe bending theory

6 Recommendations of the Arbeitskreis "Baugrunddynamik" ('Building Ground Dynamics' Work Group), Deutsche Gesellschaft für Geotechnik e.V. (DGGT), Berlin 2002

as long as the wall thickness amounts to at least 1/20 of the radius. This does not apply to local verifications in the area of apertures in the tower, nor does it apply to stresses from thermal actions pursuant to Section 7.4.5.

The DIN EN 1993-1 series of standards shall be applied to the verification for steel towers.

In cylindrical and conical steel pipe towers, the stresses required for the structural integrity verification may be calculated according to the shell membrane theory. This means that the elementary pipe bending theory may be applied to the degradation of the wind loads (for example). It is not necessary to factor in shell bending moments from wind pressure distributed unevenly over the circumference of the tower or constraint stresses from edge disturbances at flanges or struts. It is necessary to factor in local circumferential membrane forces and shell bending moments which result from the transfer of force at transitions with differing conicity. Section 13.2 must be observed for areas of the tower which are weakened by apertures.

NOTE: The verification procedure described here corresponds to the terminology of DIN EN 1993-1-1 of an elastic structural calculation with plastic cross-sectional load-bearing capacities for the local tower wall force variables, but elastic cross-sectional load-bearing capacities for the global tower force variables.

11.1.3 Stability failure

The verifications must be conducted with the least favourable of all action combinations of Groups N, A and T.

The bulge safety verification for the wall of a steel pipe tower or other saucer-shaped steel components may also be conducted as a numerically assisted bulge safety verification in accordance with paragraphs 8.6 and 8.7 of DIN EN 1993-1-6.

11.1.4 Fatigue failure of steel structures

The verifications must be conducted with the action combinations of Group F pursuant to

Table 3.

Verification for steel tower structures is based on DIN EN 1993-1-9. Regular maintenance and recurrent inspection pursuant to Section 14 is assumed. The partial safety coefficient to be applied can be found in Table 7.

Contrary to the provisions in DIN EN 1993-1-9, a fatigue resistance threshold value for load cycles $N > 10^8$ may not be applied (see Figure 5).

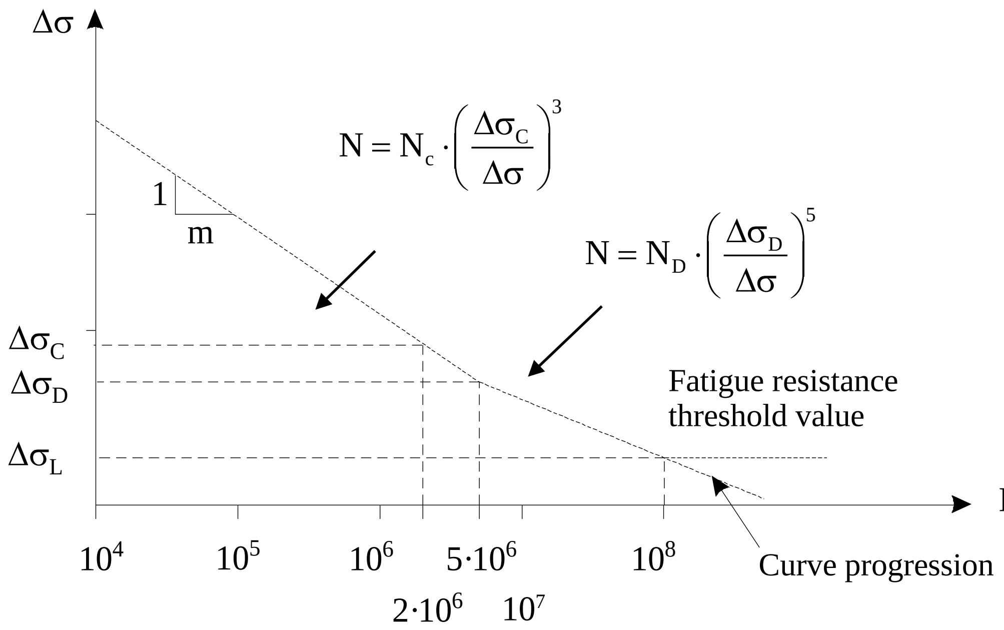

Figure 5: Fatigue resistance for steel (S-N curve)

Table 7: Partial safety coefficient γ_M for verifications against fatigue for towers made of steel

Able to be inspected	γ_M	
	Components which can tolerate damage	Components which cannot tolerate damage
Yes	1.0	1.15
No	1.15	1.25

NOTE: Wind turbines generally contain components which cannot tolerate damage.

A partial safety coefficient of 1.15 must generally be applied for components that are able to be inspected.

All components which are accessible should be deemed ‘able to be inspected.’ These include components such as all circumferential and longitudinal weld seams of steel pipe towers as well as the screws of ring flange connections. These components must be examined during the recurrent inspections (see Chapter 15).

In departure from this, a coefficient of 1.25 should be taken into account if monitoring measures during the inspection are not possible, such as in the case of components which are encased in concrete.

The fatigue resistance reference value $\Delta\sigma_c$ can be found in the notch class catalogues of DIN EN 1993-1-9, Tables 8.1 to 8.10 and DIN EN 1993-3-2 Annex C, depending on the notch class in question.

NOTE on DIN EN 1993-3-2 C2(1): Increasing notch classes merely by changing the quality level of the weld seam is not permissible. An increase in the notch class must be justified by means of experiments such as in accordance with the rules of DIN EN 1990.

The structural stress concept pursuant to DIN EN 1993-1-9 Annex B may be used as an alternative to the nominal stress concept. A sheet metal thickness degradation $k_s = (25/t)^{0.2}$ is used where sheet thickness t> 25 mm, with t applied in [mm].

To supplement the notch class catalogues, the notch detail T-flange/sheet metal casing is provided for as follows:

- The notch class pursuant to DIN EN 1993-1-9 Table 8.5 Detail 1 shall be used for the sheet metal casing (notch point 1 in Figure 6).

- The conservative approach of a notch class of 90 and a sheet metal thickness degradation $k_s = (25/t_F)^{0.2}$ shall be used for sheet thickness t_F>25 mm (notch point 2 in Figure 6). The decisive bending stresses shall be determined by applying an even stress distribution from the concrete compressions, unless more precise examinations justify more favourable approaches.

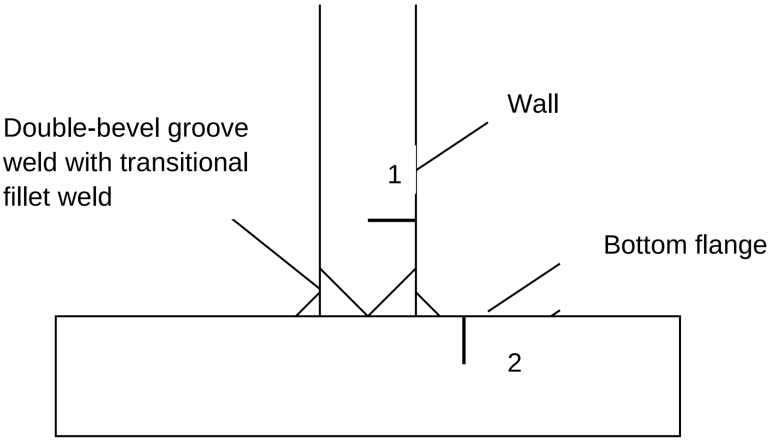

Figure 6: Verification points for the detail 'T-flange/sheet metal casing'

NOTE: Examinations conducted indicated that different Eurocode regulations for the parameter range of wind turbines may result in highly conservative designs.

11.1.5 Fatigue failure of steel and pre-stressed concrete structures

With towers and foundations made of pre-stressed concrete and/or reinforced concrete, it will be necessary to conduct fatigue safety verifications for the concrete, concrete reinforcing steel and pre-stressing steel. The calculated damages of varying load ranges may be added in the verification against fatigue according to the Palmgren-Miner rule. In the process, the damage sum D_{Ed} resulting from the decisive fatigue stress must meet the following condition:

$$D_{Ed} = \Sigma[n(\Delta\sigma_i)/ N(\Delta\sigma_i)] < 1.0 \qquad \text{(GL 13)}$$

Where

$n(\Delta\sigma_i)$ is the number of load cycles applied for a load range of $\Delta\sigma_i$

$N(\Delta\sigma_i)$ is the number of bearable load cycles applied for a load range of $\Delta\sigma_i$

The S-N curves pursuant to DIN EN 1992-1-1, paragraph 6.8.4 must be applied for verifying the concrete reinforcing steel and pre-stressing steel.

The fatigue analyses for pre-stressed concrete structures must be conducted for both the pre-stressing force immediately after the press is depressed as well as for the pre-stressing force after the creep, contraction and relaxation, unless a more precise calculation is conducted over time. In the process, it is necessary to factor in time-dependent losses due to creep, contraction and relaxation pursuant to DIN EN 1992-1-1, paragraph 3.1.4.

The following S-N curves for the concrete must be applied for verifying the concrete under pressure or shear force stress:

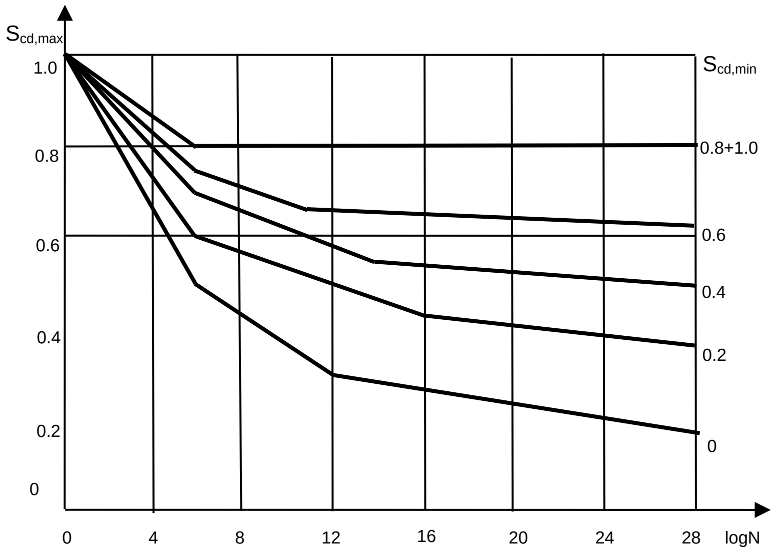

Figure 7: The concrete's S-N curve under compressive stress

The S-N curve pursuant to Figure 7 is based on the following equations[7]:

For

$$0 < S_{cd,\min} < 0{,}8$$
$$\log N_1 = \left(12 + 16 \cdot S_{cd,\min} + 8 \cdot S_{cd,\min}{}^2\right) \cdot \left(1 - S_{cd,\max}\right)$$
$$\log N_2 = 0{,}2 \cdot \log N_1 \cdot \left(\log N_1 - 1\right)$$
$$\log N_3 = \log N_2 \cdot \left(0{,}3 - (3S_{cd,\min}/8)\right) / \Delta S_{cd}$$

If $\log N_1 \leq 6$ then $\log N = \log N_1$

If $\log N_1 > 6$ and $\Delta S_{cd} \geq 0{,}3 - (3S_{cd,\min}/8)$

then $\log N = \log N_2$

if $\log N_1 > 6$ and $\Delta S_{cd} < 0{,}3 - (3S_{cd,\min}/8)$

then $\log N = \log N_3$

where

$$S_{cd,\min} = \gamma_{Sd} \cdot \sigma_{c,\min} \cdot \eta_c / f_{cd,fat}$$
$$S_{cd.\max} = \gamma_{Sd} \cdot \sigma_{c,\max} \cdot \eta_c / f_{cd,fat}$$
$$\Delta S_{cd} = S_{cd,\max} - S_{cd,\min}$$

A more detailed verification for the concrete is not required for wind turbines with a nominal load cycle of $N_{nom} = m \cdot n_R \cdot T_0 \leq 2 \cdot 10^9$ if the condition according to (GL 14) is complied with:

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[7] From: CEB-FIP Model Code 1990, Bulletin d'Information No. 213/214

$S_{cd,max} \leq 0.40 + 0.46\, S_{cd,min}$ (GL 14)

Where: $S_{cd,min} = \gamma_{Sd} \cdot \sigma_{c,min} \cdot \eta_c / f_{cd,fat}$
$S_{cd,max} = \gamma_{Sd} \cdot \sigma_{c,max} \cdot \eta_c / f_{cd,fat}$

$\gamma_{Sd} = 1.1$ is the partial safety coefficient for recording the imprecisions of the stress calculation model

$\sigma_{c,max}$ is the magnitude of the maximum concrete compressive stress under the action combinations of Group F pursuant to Table 2

$\sigma_{c,min}$ is the magnitude of the minimum concrete compressive stress in the pressure zone at the same point at which $\sigma_{c,max}$ occurs, determined for the lowermost value of the action ($\sigma_{c,min} = 0$ shall be applied for tensile stresses)

η_c is the factor to take into account the distribution of concrete compressive stress in accordance with Volume 439[8], Eq. (8); for simplification, $\eta_c = 1.0$ may be used.

$f_{cd,fat}$ is the calculation value for the concrete's fatigue resistance under compressive stress:
$f_{cd,fat} = 0.85 \cdot \beta_{cc}(t) \cdot fck \cdot (1 - f_{ck}/250) / \gamma_c$

f_{ck} is the characteristic compressive cylinder strength in N/mm²

γ_c is the partial safety coefficient for concrete

$\beta_{cc}(t)$ is the coefficient for factoring in the time-dependent increase in the concrete's strength. $\beta_{cc}(t)$ may not be set higher than 1.0 if the simplified equation (GL 14) is applied, corresponding to a cyclical initial loading at a concrete age of ≥ 28 days. In the case of a cyclical initial loading at a young concrete age, it will be necessary to determine $\beta_{cc}(t) < 1.0$ and take it into account in the verification; $\beta_{cc}(t)$ must be determined in accordance with DIN EN 1992-1-1, paragraph 3.1.2(6).

In general, the following must be examined when using the simplified verification procedure:

- maximum load range,
- load range with the greatest concrete compressive stress $\sigma_{c,max}$,
- load range with the lowest concrete compressive stress $\sigma_{c,min}$,
- load range with the greatest mean concrete compressive stress value.

11.2 Verifications at the limit states of performance capability
11.2.1 Action combinations

The actions for the verifications at the limit states of performance capability are Table 2 defined in :

- DLC D.1: Characteristic (rare) actions
- DLC D.2: Common actions
- DLC D.3: Quasi-constant actions

These actions shall be applied to the verifications defined in the corresponding technical standards in combination with actions from temperature.

[8] Deutsche Ausschuss für Stahlbeton (German Committee for Reinforced Concrete – DAfStb) (ed.), Volume 439 "Fatigue resistance value of reinforced and pre-stressed concrete components with explanatory notes on verifications pursuant to CEB-FIP Model Code 1990", 1994 edition, Beuth Verlag Berlin

11.2.2 Partial safety coefficient

The partial safety coefficient for the resistance variables shall amount to γ_M = 1.0 for verifications at the limit states of performance capability.

11.2.3 Deformation limitation

If the operation of the turbine does not lead to any special requirements, then it will not be necessary to limit the deformation.

11.2.4 Stress limitation

With towers and foundations made of pre-stressed concrete and/or reinforced concrete, it will be necessary to limit the concrete compressive stresses for the rare action combination D.1. in accordance with Table 2 to 0.6 f_{ck}. Otherwise it will be necessary to take substitute measures pursuant to DIN EN 1992-1-1, paragraph 7.2(2).

With towers and foundations made of pre-stressed concrete, it is additionally necessary to limit the concrete compressive stresses under the constant action of dead loads and to limit the pre-stressing to 0.45 f_{ck} in accordance with DIN EN 1992-1-1, paragraph 7.2(3).

For towers made of pre-stressed concrete with composite, it will be necessary to verify the decompression for the quasi-constant action combination D.3 pursuant to Table 2.

11.2.5 Crack width control

The crack width control must be verified for a calculated crack width of 0.2 mm. In the process, the quasi-constant action combinations D.3 pursuant to Table 2 must be used for components made of reinforced concrete and pre-stressed concrete without composite; the common action combinations D.2 pursuant to Table 2 must be used for components made of pre-stressed concrete with composite. The thermal actions must be applied in accordance with Section 7.4.5.

12 Verifications for the foundation

12.1 Foundation structures

12.1.1 Safety concept

The safety concept described in Sections 10, 11 and 12 shall be applied for verifications of components made of reinforced concrete and pre-stressed concrete as well as components made of steel.

12.1.2 Built-in steel parts

Built-in steel parts shall be verified in accordance with Section 11.1.4.

12.1.3 Reinforced concrete components

Reinforced concrete components shall be verified in accordance with Sections 11.1 and 11.2.5. The verifications against fatigue for the concrete, concrete reinforcing steel and pre-stressing steel and the fasteners must be conducted in accordance with Section 11.1.5 of this Guideline.

Components of the foundation body which extend not more than half a metre into the ground shall be verified with a crack width of 0.2 mm; all other components shall be verified with a crack width of 0.3 mm.

If bases are designed on foundations, then the verifications for the stress and crack width control must be conducted the same way as with the tower (see also 11.2.4 and 11.2.5).

12.1.4 Pile design

The interior bearing capacity of foundation piles must be determined in accordance with Sections 12.1.2 and 12.1.3. The verification of the exterior pile bearing capacity must be conducted in accordance with Section 12.2.4.

12.2 Subsoil

12.2.1 Condition of subsoil

It must be ensured that the properties of the building ground at the site meet the requirements in the static and dynamic calculation.

The foundations of wind turbines shall be allocated to geotechnical category 3 pursuant to DIN EN 1997-1, paragraph 2.1 and/or DIN 1054, paragraph A2.1.2 with respect to the minimum requirements for the extent and quality of geotechnical examinations.

The elasticity and shear moduli for relief and reloading procedures are decisive for dynamic calculations; but the initial loading modulus is generally decisive for static calculations (as long as the ground is not pre-stressed). The ground firmness generally depends on the size of the shear lags induced by the load. The firmness is maximal for very small shear lags; beyond it, the moduli for relief and reloading procedures and initial loading are identical, since the soil in this area reacts nearly elastically. They are also referred to as "dynamic" shear and/or elasticity moduli, since correspondingly small shear lags generally occur under highly frequent loads. In general, the shear modulus G_{max} for very small expansions tabulated for many soil types only applies to highly frequent loads. It may be necessary to reduce this value for wind turbine foundations – especially for shallow foundations on relative compressible soils.

The dynamic ground firmness values for very small shear lags for the total dynamic calculation as well as the static ground firmness values for the settling calculation must be specified in the expert foundation report.

The recommendations of the "Building Ground Dynamics" Work Group (Arbeitskreis Baugrunddynamik) contain information on the value of dynamic soil characteristics[4].

12.2.2 Safety concept

The safety verifications for the building ground must be taken into account in accordance with DIN EN 1997-1 and DIN 1054:2010-12 in observance of this Guideline's special determinations for the limit states of bearing capacity and performance capability. The stresses must be determined from the characteristic values of the actions; non-linear influences pursuant to Section 9.2.3 must be taken into consideration in the process.

The stresses shall be classified as ordinary dynamic stresses in the sense of DIN 1054, paragraph A 2.4.2.1 A(8a) and may thus be regarded as variably static actions.

During the verifications, the action combinations pursuant to Section 8 of this Guideline must be allocated to the design situations BS-P, BS-T or BS-A pursuant to DIN 1054, 2.2 A(4) in accordance with Table 8.
The verifications must be conducted with the least favourable of all action combinations.

Table 8: Allocation of the action combinations pursuant to DIN EN 61400-1 to the design situations pursuant to DIN 1054

Action combination DLC pursuant to DIN EN 61400-1	Design situation pursuant to DIN EN 1997-1 and/or DIN 1054
Load case Groups N and T	BS-P
DLC 8.2	BS-T
Load case Group A (without 8.2)	BS-A

12.2.3 Shallow foundations

12.2.3.1 Limit states of bearing capacity

The verification of the shear failure safety (limit state GEO-2 in accordance with DIN 1054) as well as the verification of the position stability and stability against overturning (limit state EQU) must be rendered. In exceptional cases, such as if the foundation is on an embankment or next to a supporting structure, the overall bearing capacity (sliding failure, limit state GEO-3 in accordance with DIN 1054) shall also be verified.

The verification of the shear failure safety must be conducted in accordance with DIN EN 1997-1 for the actions of the design situations BS-P, BS-T and BS-A pursuant to Table 8. The GEO-2 verification method is applied here in accordance with DIN 1054 and the characteristic soil bearing capacity determined in accordance with DIN 4017. The design value results from dividing the characteristic soil bearing capacity by the partial safety coefficients pursuant to DIN 1054 A 2.4.7.6.3, Table A 2.3. The design value of the stresses which are vertical to the base surface is the result of the sum of the constant and variable stresses multiplied by the associated partial safety coefficients, in which, contrary to DIN 1054 the partial safety coefficients pursuant to Table 5a or 5b of this Guideline shall be applied, meaning that a partial safety coefficient $\gamma_F = 1.35$ may be applied for action combinations of design situations BS-P for wind loads with unfavourable actions, in observance of the footnote in Table 5a or 5b.

The verification of the position stability and stability against overturning (limit state EQU) must be conducted pursuant to DIN EN 1997-1, 2.4.7.2, as well as DIN 1054, 6.5.4 A (3) with the application of the partial safety coefficients in accordance with DIN 1054, A 2.4.7.6.1, Table A 2.1. In deviation from DIN 1054, a partial safety coefficient of $\gamma_F = 1.35$ may also be applied for action combinations of design situation BS-P for the wind loads with unfavourable actions, in observance of the footnote in Table 5 or Table 6.

12.2.3.2 Limit states of performance capability

A separation of the base joint may appear up to the base surface's centre of gravity at most due to the characteristic stress in the base surface resulting from the actions of design situations BS-P and BS-T pursuant to Table 8.

A separating joint may not appear in the base surface due to characteristic stress resulting from the action combination D.3 pursuant to Table 2.

12.2.4 Pile foundations (exterior bearing capacity)

The pile bearing capacity verification must be conducted in accordance with DIN EN 1997-1 and DIN 1054 for the actions of the design situations BS-P, BS-T and BS-A pursuant to Table 8 of this Guideline. The verification procedure GEO-2 shall be applied in accordance with DIN 1054.

The pile bearing capacity design value results from dividing the characteristic pile bearing capacity by the partial safety coefficients pursuant to DIN 1054 A 2.4.7.6.3, Table A 2.3. The design value of the stresses results from the sum of the constant and variable stresses multiplied by the associated partial safety coefficients, in which, contrary to DIN 1054 the partial safety coefficients pursuant to Table 5 of this Guideline shall be applied.

The verification that no tensile stresses in the piles occur under the characteristic values of action combination D.3 pursuant to Table 2 may be conducted as a substitute in place of the verification against fatigue with respect to the exterior bearing capacity.

<u>NOTE</u>**:** The piles should be arranged in an inclined manner so as to absorb the horizontal forces.

13 Construction details

13.1 Ring flange connections for steel towers

Ring flange connections must be checked pre-stressed in accordance with DIN EN 1993-1-8. The pre-stressing force must be limited to the standard pre-stressing force in accordance with F_{p,C^*} DIN EN 1993-1-8/NA.

The note in paragraph 11.1.2 applies accordingly to the structural integrity verification of the flange connections. The pre-stressing force of the screws may be disregarded, meaning that the structural integrity verification may be conducted the way it would be for a screw connection which is not pre-stressed.

Local plastifications (plastic hinges in the flange and/or in the tower casing) may be taken into account in the process. The component resistances for ring flange connections may, for simplification purposes, be determined using plastic failure states (VZ - Versagenszustand) according to Petersen[9] (VZ A and B) as well as according to Seidel[10] (VZ D and E).

During the fatigue safety verification of the flange connection, the fatigue stress of the screws may be determined in observance of the compressive pre-stressing of the flange by way of the trilinear model (see

[9] Petersen, C.: Stahlbau, 3rd edition. Braunschweig, Wiesbaden: Vieweg, 1997

[10] Seidel, M.: "Zur Bemessung geschraubter Ringflanschverbindungen von Windenergieanlagen" ["On the Design of Screwed Ring Flange Connections for Wind Turbines"], Publication Series of the Institute for Steel Construction/Leibniz University Hannover (Volume 20). Aachen: Shaker Verlag, 2001

Figure 8) according to Schmidt/Neuper[11], as long as the tolerances described in the following are maintained:

- Once the production of the individual tower segments has been completed, the flatness deviation per flange may not exceed a value of 2.0 mm over the entire circumference and max. 1.0 mm over a segment of 30°, in which the area at the tower wall is decisive.
- Careful production of the flange and its welding connections and careful pre-stressing must be used to ensure that the pre-stressing force of each individual screw is sufficiently converted into local compressive pre-stressing of the flange contact surfaces in their proportional area.
- The inclinations α_S of the exterior flange surfaces (see **¡Error! No se encuentra el origen de la referencia.**) may not exceed the limit value of 2° after pre-stressing.

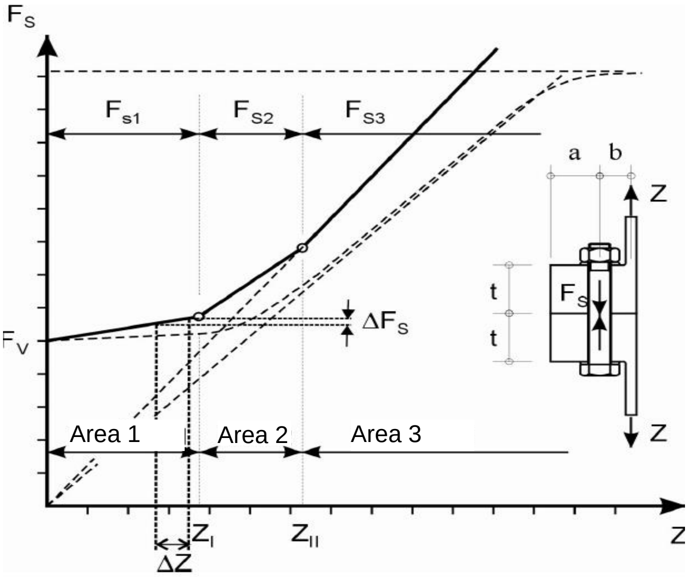


Figure 8: Screw force functions of pre-stressed ring flange connections after pre-stressing according to Schmidt/Neuper[11]

NOTE 1: Inclinations greater than α_S before pre-stressing have no effect on fatigue damage as long as they are reduced to a value below the limit value during the pre-stressing process. A maximum of 90 % of the planned pre-stressing force may be applied when verifying the fatigue safety. In principle, the pre-stressing of screws should be checked and, if necessary, restressed within the 1st six months after installation, but no earlier than the number of operating hours stipulated in the maintenance requirement book. The non-linear screw force function $F_S = f(Z)$ from which the fatigue-relevant load range for the specified load ranges ΔZ of the tower casing force ΔF_S is read off is used as the basis for the fatigue safety verification (see Figure 8).

NOTE 2: The complete Markow or Rainflow matrix may be needed here in place of the stress collective.

The flange separations tolerated during execution should be regarded as imperfections when determining the screw force function with FEM.

If a simplified calculation model is used which only provides normal screw forces, then the verification against notch class 36* must be conducted; see Figure 9.

[11] Schmidt, Herbert; Neuper, Meike: "Zum elastostatischen Tragverhalten exzentrisch gezogener L-Stöße mit vorgespannten Schrauben". Verlag Ernst & Sohn, Stahlbau 66 (1997), Volume 3, p. 163-168

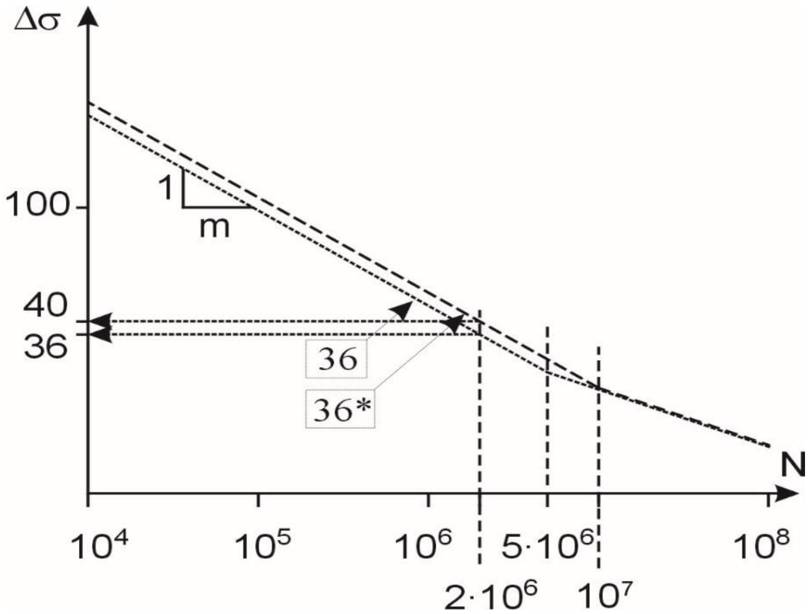
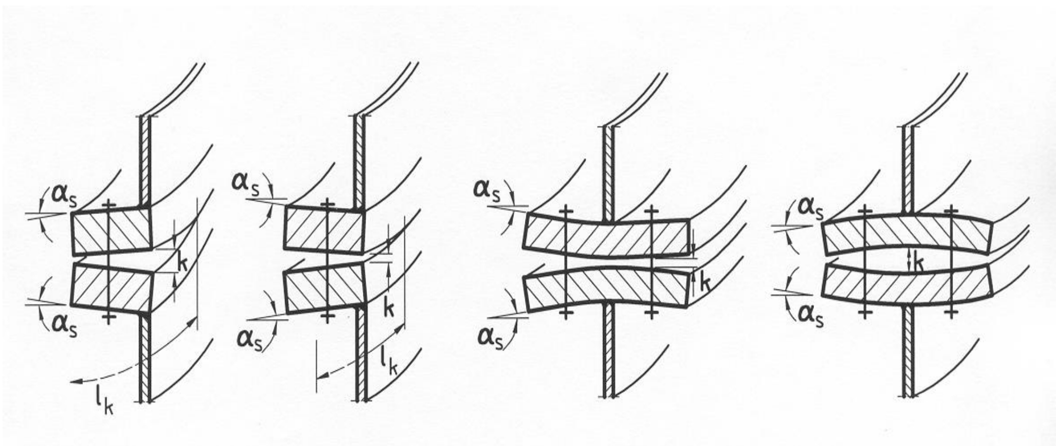


Figure 9: Notch classes for screws

If the limit values for the flange separations specified in the execution documents are not complied with, then it will be necessary to take suitable measures, such as lining the hollow separation spaces which are relevant to the damage before pre-stressing. If the limit value for the inclination α_S is exceeded after pre-stressing, then it will be necessary to install suitable wedge plates of sufficient hardness in place of the flat washers.

NOTE 3: All flange separations k in the area of the tower wall (see Figure 10) are relevant to damage for the screws' fatigue stress, especially if they only extend over part of the circumference. In the process, the damaging effect increases as the length of extension l_k over the circumference decreases, meaning that the ratio of k/l_k is decisive.



a) L-flange, b) T-flange

Figure 10: Ring flange connections in steel towers

13.2 Apertures in the wall of steel pipe towers

In general, it is necessary to verify the tower wall's safety against bulging in the aperture area using finite element analyses. A "numerically-assisted bulge safety verification via global MNA and LBA calculation" pursuant to DIN EN 1993-1-6, 8.6 must be conducted. In doing so, the ideal bulge resistance R_{cr} must be determined from a geometrically non-linear elastic calculation (GNA). The narrower area surrounding the aperture may be disregarded when determining the decisive point for determining the plastic reference resistance F_{Rpl}; this narrower area may not be set to be wider than $2\,(r \cdot t)^{0.5}$.

In the area of apertures which are circumferentially stiffened at the edges without pre-bound longitudinal braces (‘collar braces,’ see Figure 11), the bulge safety verification may be simplified as with an unweakened tower wall if the reduced design bulging stress according to Equation (GL 15) is used in place of the design bulging stresses pursuant to DIN EN 1993-1-6:

$$\sigma_{xS,R,d} = C_1 \cdot \sigma_{x,Rd_{EC}} \qquad \text{(GL 15)}$$

Where:

$\sigma_{x,Rd_{EC}}$ Design bulging stress according to DIN EN 1993-1-6

C_1 Reduction factor according to Equation (GL 16) for recording the aperture effect.

$$C_1 = A_1 – B_1 \cdot (r/t) \qquad \text{(GL 16)}$$

where A_1 and B_1 pursuant to Table 9

Table 9: Coefficients for Equation (GL 16)

Strength class	S 235		S 355	
Aperture angle	***A*₁**	***B*₁**	***A*₁**	***B*₁**
δ = 20°	1.00	0.0019	0.95	0.0021
δ = 30°	0.90	0.0019	0.85	0.0021
δ = 60°	0.75	0.0022	0.70	0.0024

δ is the aperture angle in the circumferential direction

<u>The preceding rules apply to:</u>

– tower walls where $(r/t) \leq 160$,

– aperture angles $\delta \leq 60°$,

– aperture dimensions $h_1 / b_1 \leq 3$,

as well as aperture edge braces

– which run around the entire aperture with a constant cross-section,

– whose cross-section area is at least one third of the aperture aberration area, whose cross-section is arranged on the longitudinal edges of the aperture centred on the central wall area (see Figure 11b) and

– whose cross-section part is under the maximum c/t-ratios pursuant to DIN EN 1993-1-1, Table 5.2 for cross-section category 2.

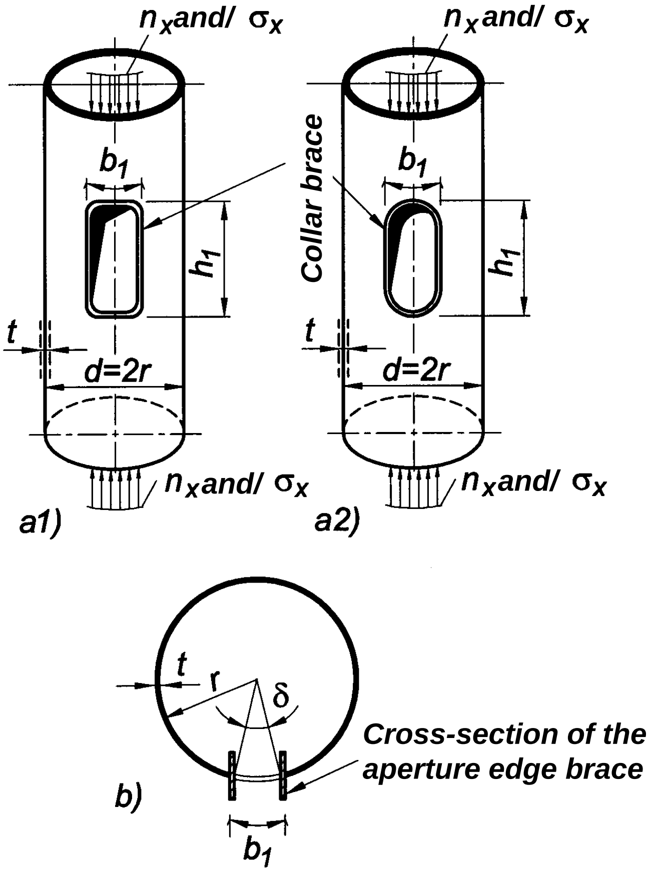


Figure 11: Apertures in the wall of steel pipe towers

13.3 Screw connections under shear stress

Screw connections at connections and joints of components of the main bearing structure must be executed as close-tolerance shear/bearing connections (SLP, SLVP – Scher-Lochleibungs-Passverbindungen) or as friction-grip connections (GV, GVP – gleitfest planmäßig vorgespannte Verbindungen).

With SLP and SLVP connections, it will be necessary to conduct the verifications pursuant to Sections 11.1.2 and 11.1.4 and for the perforated components and screws.

With GV and GVP connections, it must be verified that the maximum force allotted to a single screw in a shear plane does not exceed the limit sliding force according to Equation (GL 16) in the limit state of structural integrity:

$$F_{s,Rd} = 0{,}9 \cdot \frac{\mu}{\gamma_{M,3}} \cdot F_{p,C^*} \qquad \text{(GL 17)}$$

This does not generally apply to screwed ring flange connections.

Where:

F_{p,C^*} is the standard pre-stressing force pursuant to DIN EN 1993-1-8/NA Tables A.1 and A.2 This pre-stressing force must be ensured by inspection and, if necessary, re-stressing, within the first six months as of installation, albeit not immediately after commissioning.

$\mu \leq 0.5$ is the friction coefficient to be determined for executing the contact surfaces in accordance with DIN EN 1993-1-8 Table 3.7 or by means of trials in accordance with Reference Standard

Group 7 in DIN EN 1993-1-8, paragraph 1.2.7 (Reference to DIN EN 1090-2) for the friction surface in question

$\gamma_{M,3} = 1.25$ is the safety factor for the action combinations of Group N and the operating conditions 1 through 4 pursuant to DIN EN 61400-1

$\gamma_{M,3} = 1.1$ is the safety factor for all other action combinations

NOTE 1: Special corrosion protection measures must be taken for close-fit connections with hot-dip galvanised components.

Furthermore, the structural integrity verifications for the perforated components and screws must be conducted in terms of shearing and hole bearing stress.

NOTE 2: These verifications cover the fatigue safety verification.

13.4 Material toughness

13.4.1 Actions for steel grade selection

The occurrence of the lowest component temperature corresponds to an 'extraordinary' design situation, i.e. the force variables for determining the decisive stresses must be determined with load case D.2 (common loads), see also DIN EN 1993-1-10, paragraph 2.2(4).

Wind turbines shall be classified as bridges for determining the reference temperature T_{Ed}. The following may thus be applied in accordance with DIN EN 1993-1-10/NA:

$T_{Ed} = -30°C$

Temperature shifts due to cold forming and increased expansion rate can generally be disregarded.

The partial safety coefficient of the action is $\gamma_F = 1.0$ in each case.

The decisive stressing level σ_{Ed} shall be determined in the decisive cross-section and bearing structure part at the expected crack formation point (in circumferential collar braces, for instance). The acknowledged state of the art shall be put into effect in the application. According to it, excess stresses at the aperture and tower head (and any other relevant structural details) need not be taken into account separately.

13.4.2 Decisive product thicknesses for ring flange connections

The decisive product thickness for ring flanges shall be selected as follows:

- For ring flanges with a neck (weld-neck flange), it is the sheet metal thickness of the connecting tower wall (b pursuant to Figure 12); in the process, the distance from the weld seam transition including the transition radius (l + r in accordance with Figure 12) must correspond to at least half of the connecting sheet metal thickness, otherwise the flange must be evaluated as a flange "without a neck."

In addition, the product thickness of the flange (t_{ges} pursuant to Figure 12) must be evaluated at stressing level $\sigma_{Ed} = 0.25 \cdot f_y(t)$ for welded ring flanges. This additional verification may be omitted for ring flanges produced without seams or ring flanges welded using the flash butt method.

- For ring flanges without a neck, it is the thickness ($t_f = t_{ges}$ pursuant to Figure 12) of the ring flange.

13.4.3 Steel grade selection with respect to properties in the thickness direction

Z-quality pursuant to DIN EN 1993-1-10, Chapter 3, must be verified for sheet metal which is stressed perpendicularly to the production direction.

With flanges made of sheet metal (sheet metal thickness = flange height t_f or t_{ges} in accordance with Figure 12), a weld-neck flange can only be applied to have a favourable action if the distance from the welding seam transition to the run-out of the transition radius (l in accordance with Figure 12) corresponds to at least half of the connecting sheet metal thickness (b in accordance with Figure 12).

It is not necessary to verify Z-quality pursuant to DIN EN 1993-1-10 in the case of flanges made from sheet metal which are placed under stress in a direction parallel to the direction of production (sheet metal thickness = flange width b_{ges} pursuant to Figure 12), as well as with flanges which are made from a seamlessly rolled ring or rod rolled on all sides.

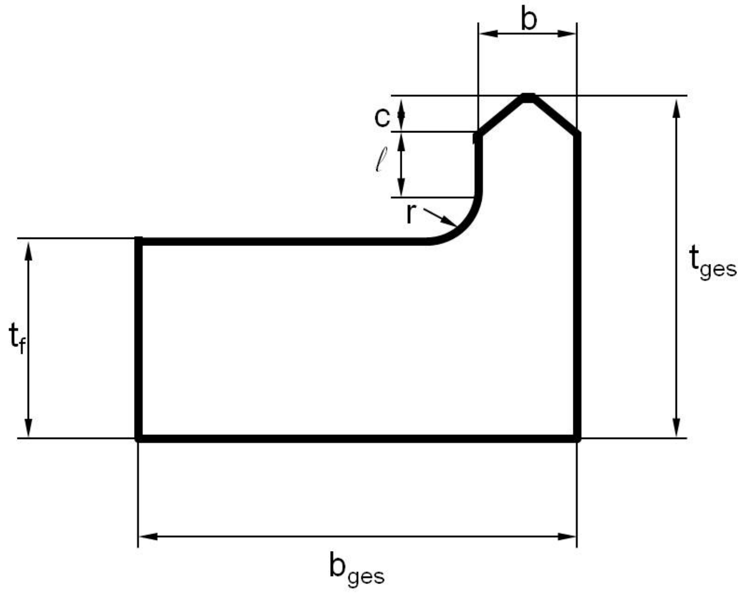


Figure 12: Flange designations based on the example of an L-flange

t_f	Flange thickness
t_{ges}	Overall flange thickness including weld-neck flange
b_{ges}	Flange width
r	Transition radius
c	Flank height of the welding seam preparation
b	Neck thickness (generally equal to the sheet metal thickness of the connecting sheet metal)
l	Distance from the transition radius to the welding seam preparation (= t_{ges} - t_f - r - c)

14 Acceptance and commissioning

Prior to commissioning, it will be necessary to certify that the wind turbine has been constructed in accordance with the civil engineering documents inspected during the construction monitoring and/or construction status inspection on the part of the building inspection body or the inspection engineer. The extent of the measures for the examination and monitoring can be found in the "Recommendations for the construction monitoring of wind turbines" of the Building Construction Monitoring Association (Bauüberwachungsverein - BÜV). The acceptance of the machine is not the subject of these recommendations and is conducted on the basis of the expert opinions on the machine (see Section 3, Number I).

It may be necessary to observe construction monitoring and inspections by official expert organisations in accordance with the VAwS (Ordinance on Installations for the Handling of Substances Hazardous to Water) on the basis of the expert opinion on plant-related water protection (see Section 3, Number I).

15 Recurrent inspections

15.1 General

Recurrent inspections must be conducted by official experts at regular intervals on the machine and rotor blades as well as on the bearing structure (tower and accessible areas of the foundations). The inspection intervals for this are the result of the expert opinions on the machine (see Section 3, Number I). They shall amount to not more than two years, but may be extended to four years if an expert authorised by the manufacturer conducts continuous monitoring and maintenance of the wind turbine (at least on an annual basis).

15.2 Extent of the recurrent inspection

The machine, including the electrical devices of the operation management and safety system and rotor blades, must be examined to ensure that it is in faultless condition. In the process, it will be necessary to conduct the inspections in accordance with the requirements in the maintenance requirement book and potentially additional special requirements in the other expert reports (see Section 3, Number I).

It must be ensured that the safety-related limit values are complied with in accordance with the execution documents appraised.

At least one visual inspection must be conducted for the tower and foundation (foundation cellar and base) in which the individual components are examined from the immediate vicinity.

It is necessary to check whether the tower structure exhibits impermissible damages in terms of structural integrity (i.e. corrosion, cracks, chipping in the load-bearing steel and/or concrete structures) or impermissible changes in comparison with the approved execution (i.e. in terms of the pre-stressing of the screws, the permissible inclination, the required earth surcharge on the foundation).

At least one visual inspection and looseness inspection must be conducted for planned pre-stressed screws.

15.3 Documents for the wind turbine to be inspected

At least the following documents must be examined for the recurrent inspection:

- Maintenance requirement book
- Test reports on the civil engineering documents for the tower and foundation
- Expert report on the machine

- Special requirements in the expert load report
- Special requirements in the expert soil report
- Construction permit documents

- Operating instructions
- Commissioning protocol
- Reports of previous recurring inspections and monitoring and maintenance
- Documentation of changes and any potential repairs to the turbine and approvals, if applicable

15.4 Measures

15.4.1 Repairs

A time frame for proper repair must be established for faults which the official expert has detected.

The repair must be conducted by the wind turbine manufacturer, a company authorised by the manufacturer or a company specialising in this field which has all of the necessary knowledge, documents and tools.

15.4.2 Shutdown and recommissioning

The turbine must be decommissioned immediately in the event of faults which jeopardise the structural integrity of the wind turbine, either entirely or partially or which could cause immediate hazards resulting from the machine and rotor blades.

Recommissioning once the faults have been rectified will require approval from an official expert.

15.5 Documentation

The result of the recurrent inspection must be recorded in a report which must contain at least the following information:

- The official expert conducting the inspection
- Manufacturer, type and serial number of the wind turbine as well as the main components (rotor blades, gearbox, generator, tower)
- Site and operator of the wind turbine

- Total operating hours
- Wind speed and temperature on the day of the inspection
- Persons present during the inspection
- Description of the extent of the inspection
- The result of the inspection and any potential special requirements

A report must be completed on any repairs conducted on the basis of structural-integrity-related special requirements.

The operator must keep this documentation for the entire duration of the wind turbine's use.

16 Site suitability of wind turbine

A type inspection and/or individual inspection for the turbine must be on hand in order for the site suitability inspection to be conducted.

16.1 Existing turbines in the event of a wind farm expansion or alteration

In the event of a wind farm alteration, the verification of the site suitability may continue to be conducted according to the DIBt 2004 procedure for existing turbines which were constructed based on DIBt 1995 or DIBt 2004.

16.2 New turbines

A site inspection must be used as the basis for determining the site conditions. The relevant guidelines should be applied in the process. The following procedure is recommended as an alternative to DIN EN 61400-1:2011-08 for verifying the site suitability of wind turbines in wind farms. .

a. The following specifications at hub height will be required for verifying the site suitability:
   i. Mean wind speed (v_{ave})
   ii. Shape parameter of the Weibull function (k)
   iii. Mean wind shear exponent (α)
   iv. Mean air density (p)
   v. Mean turbulence intensity as well as the turbulence intensity's standard deviation at 15 m/s (for recording the characteristic or representative turbulence intensity).
   vi. 50-year wind speed ($v_{b,0}$) in accordance with wind zone maps or determined according to the Gumbel method[12] (for instance), if necessary (this is the case if the turbine is to be built in a higher wind zone than that which is covered in the type inspection and/or individual examination, for example).
   vii. Wind direction distribution of the turbines in question

b. A simplified comparison is conducted using the turbine referred to in 16.2.a. Where the following conditions are met, the site suitability of the wind turbine can be confirmed.
   i. Comparison of the mean wind speed
      (1) The mean wind speed at the site is at least 5 % lower than that according to the type or individual inspection, or
      (2) The mean wind speed at the site is lower than that according to the type/individual inspection and the shape parameter k of the Weibull function is greater than or equal to 2.
   ii. Comparison of the effective turbulence intensity pursuant to DIN EN 61400-1:2011-08 between 0.2-v_{m50}(h) and 0.4 v_{m50}(h) of the design turbulence NTM
   iii. Comparison of the 50-year wind speed:
      (1) The wind zone according to the type/individual inspection covers the wind zone of the site in question according to the wind zone map (it may be necessary to observe the detailed regulations pursuant to DIN EN 1991-1-4, Section 4.3.3 including NA for uneven terrain positions), or
      (2) The 50-year wind speed (v_{m50}(h))according to the type/individual inspection covers the 50-year wind speed at the site (see 16.2.a.vi)

[12] European Wind Turbines Standards II or Harris I, "Gumbel revisited: A new look at extreme value statistics applied to wind speeds", Journal of Wind Engineering and Industrial Aerodynamics 59, 1996

c. If one of the conditions is not met, it will be possible to proceed as follows in observance of all specifications in 16.2.a:
   i. If conditions 16.2.b.i or 16.2.b.ii are not met, the site suitability may be verified on the basis of a load comparison of the operational strength loads (comparison of the site-specific loads for the load assumptions of the type/individual inspection). The effective turbulences from at least v_{in} to 0.4 $v_{m50}(h)$ must be present in order to conduct this verification. The turbulence intensity for wind speeds which are not covered in the expert report must be assumed to be constant with the value for the greatest wind speed detected in order to determine the operational strength classes.
   ii. If condition 16.2.b.iii is not met, the site suitability may be verified on the basis of a load comparison of the extreme loads (comparison of the site-specific loads for the load assumptions of the type/individual inspection).

Effects caused by terrain roughness and topography must be taken into account in the evaluation accordingly. The procedure may be used for all sites which are not deemed to be topographically complex. The topographic complexity can be determined using the procedure pursuant to DIN EN 61400-1:2011-08.

17 Continued operation of wind turbines

17.1 Application of the "Guideline for the Continued Operation of Wind Turbines"[13] - Evaluation of Tower and Foundation

The "Guideline for the Continued Operation of Wind Turbines" presents the possibility of evaluating wind turbines with respect to their continued operation once the design service life has elapsed, which this Guideline generally assumes to be 20 years.

The test methods specified in the “Guideline for the Continued Operation of Wind Turbines” allow the assessment for continued operation of wind turbines according to the state of the art. It should be noted that safety regarding the statement depends on the stability of the scope and execution of test methods and on the experts appointed to perform sampling, execution and assessment.

In principle, there are two different procedural verifications: the analytical and the practical method.

The analytical method is conducted by recalculating the wind turbine in observance of the site-specific turbine and its local boundary conditions.

The practical method is a test conducted by an inspection of the wind turbine which includes both the visual inspection and non-destructive testing methods and a sample from the bearing structure as well, if necessary.

In derogation of the “Guideline for the Continued Operation of Wind Turbines” the following apply:

- The practical method must be documented by additional static calculations incorporating the body of rules which is currently in effect (the analytical method is not meant here).
- The analytical method must be supported by additional representative material testing on the tower and an expert report of the foundation.

17.2 Experts

All inspections of the wind turbine and evaluations of loads and/or components of the wind turbine due in the scope of the evaluation for continued operation in accordance with this Guideline must be conducted by suitable independent official experts for wind turbines.

The official experts charged with the evaluation for the continued operation must have relevant training and must meet the technical requirements for the evaluation of the overall turbine. Accreditation in accordance with DIN EN ISO/IEC 17020 or DIN EN 17065 or equivalent is required.

The evaluation of the requirements of plant-specific water protection and the implementation thereof shall be conducted by an official expert organisation in accordance with the VAwS (Ordinance on Installations for the Handling of Substances Hazardous to Water).

18 Standards cited

[13] Germanischer Lloyd Industrial Services GmbH: Guideline for the Continued Operation of Wind Turbines, in: IV Vorschriften und Richtlinien Industriedienste, 1 Windenergie, 2009 edition, Germanischer Lloyd, Hamburg;

DIN 1054:2010-12	Subsoil - Verification of the safety of earthworks and foundations – Supplementary rules to DIN EN 1997-1
DIN 1055-5:2005-07	Impacts on supporting structures Part 5: Snow loads and ice loads
DIN 4017:2006-03	Subsoil - Calculation of design bearing capacity of soil beneath shallow foundations
DIN EN 1090-2:2011-10	Execution of steel structures and aluminium structures – Part 2: Technical requirements for steel structures; German version EN 1090-2:2008+A1:2011
DIN EN 1991-1-1:2010-12	Eurocode 1: Actions on supporting structures; Part 1-1: General actions - Densities, self-weight, imposed loads for buildings; German version EN 1991-1-1:2002 + AC:2009
DIN EN 1991-1-3:2010-12	Eurocode 1: Actions on supporting structures; Part 1-3: General actions, ice loads; German version EN 1991-1-3:2003 + AC:2009
DIN EN 1991-1-4:2010-12	Eurocode 1: Actions on supporting structures; Part 1-4: General actions - Wind actions; German version EN 1991-1-4:2005 + A1:2010 + AC:2010
DIN EN 1991-1-4/NA:2010-12	National Annex - Nationally determined parameters - Eurocode 1: Actions on supporting structures; Part 1-4: General actions - Wind loads
DIN EN 01/01/1992:2011-01	Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings; German version EN 1992-1-1:2004 + AC:2010
DIN EN 01/01/1993:2010-12	Eurocode 3: Design of steel structures – Part 1-1: General rules and rules for buildings; German version EN 1993-1-1:2005 + AC:2009
DIN EN 1993-1-6:2010-12	Eurocode 3: Design of steel structures – Part 1-6: Strength and stability of shell structures; German version EN 1993-1-6:2007 + AC:2009
DIN EN 1993-1-8:2010-12	Eurocode 3: Design of steel structures – Part 1-8: Design of joints; German version EN 1993-1-8:2005 + AC:2009
DIN EN 1993-1-8/NA:2010-12	National Annex - Nationally determined parameters - Eurocode 3: Design of steel structures – Part 1-8: Design of joints
DIN EN 1993-1-9:2010-12	Eurocode 3: Design of steel structures – Part 1-9: Fatigue; German version EN 1993-1-9:2005 + AC:2009
DIN EN 10/01/1993:2010-12	Eurocode 3: Design of steel structures – Part 1-10: Material toughness and through-thickness properties; German version EN 1993-1-10:2005 + AC:2009
DIN EN 1993-1-10/NA:2010-12	National Annex - Nationally determined parameters - Eurocode 3: Design of steel structures – Part 1-10: Material toughness and through-thickness properties
DIN EN 1993-3-2:2010-12	Eurocode 3: Design of steel structures – Part 3-2: Towers, masts and chimneys - Chimneys; German version EN 1993-3-2:2006
DIN EN 1997-1:2009-09	Eurocode 7: Geotechnical design – Part 1: General rules; German version EN 1997-1:2004 + AC:2009
DIN EN 1998-1:2010-12	Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings; German version EN 1998-1:2004 + AC:2009

DIN EN 1998-1/NA:2011-01	National Annex - Nationally determined parameters - Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings
DIN EN 13670:2011-03	Execution of concrete structures; German version EN 13670:2009
DIN EN 45011-1998-03	General requirements for bodies operating product certification systems (ISO/IEC Guide 65:1996); Trilingual version EN 45011:1998
DIN EN 61400-1:2004-08	Wind turbines – Part 1: Safety requirements (IEC 61400-1:1999); German Version EN 61400- -1:2004)
DIN EN 61400-1	Wind turbines – Part 1: Safety requirements Corrigendum 1:2005-12 (IEC 61400-1:1999); German Version EN 61400-1:2004
DIN EN 61400-1:2006-07	Wind turbines – Part 1: Design requirements (IEC 61400-1:2005); German Version EN 61400- -1:2005)
DIN EN 61400-1:2011-08	Wind turbines – Part 1: Design requirements (IEC 61400-1:2005 + A1:2010); German Version EN 61400-1:2005 + A1:2010
DIN EN 61400-2:2007-02	Wind turbines – Part 2: Safety of small wind turbines (IEC 61400-2:2006); German Version EN 61400-2:2006)
IEC / TS 61400-13:2001-06	Wind turbines – Part 13: Measurement of mechanical loads
DIN EN ISO/IEC 17020:2012-07	Conformity assessment - Requirements for the operation of various types of bodies performing inspection (ISO/IEC 17020:2012); German and English version EN ISO/IEC 17020:2012
DIN EN ISO/IEC 17065:2013-01	Conformity assessment - Requirements for bodies certifying products, processes and services (ISO/IEC 17065:2012); German and English version EN ISO/IEC 17065:2012

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DIBt

Annex A
(informative)

Comparison of wind speed designations of different standards and guidelines (informative)

Designation	Description	Averaging interval	Reference height	Recurrence period N	DIN EN 61400-1	DIN EN 1991-1-4/NA	DIBt 2015
					Formula symbol	**Formula symbol**	**Formula symbol**
Reference wind speed	Base parameters to define the type classes. Other relevant design parameters are derived from this. 10-min average value of extreme wind speed at hub height	10 minutes	Hub height h	50 years	v_{ref}	- / -	- / -
Basic wind speed value	Base parameters to determine wind speed depending on wind turbine wind zone	10 minutes	10m	50 years	- / -	$v_{b,0}$	- / -
Basic wind speed	Base parameters to determine wind speed depending on wind zone taking into account direction and season	10 minutes	10 m	50 years	- / -	$v_b = v_{b,0} \times c_{dir} \times c_{season} = v_{b,0}$ (see DIN EN 1991-1-4/NA)	v_b
50-year wind	Extreme mean wind speed at hub height	10 minutes	Hub height h	50 years	$v_{50}(h) = v_{rev}$	$v_m(h)$	$v_{m50}(h)$
one-year wind	Extreme mean wind speed at hub height	10 minutes	Hub height h	1 year	$v_1(h) = 0.8\ v_{50}(h)$	$v_m(h) \times c_{prob}$ (1 year)	$v_{m1}(h) = 0.8\ v_{m50}(h)$
50-year gust speed	Extreme wind speed value determined over 3 seconds	3 seconds	Hub height h	50 years	v_{e50}	$v_p(h)$	$v_{p50}(h)$
one-year gust speed	Peak value or extreme wind speed value determined over 3 seconds	3 seconds	Hub height h	1 year	v_{e1}	$v_p(h) \times c_{prob}$ (1 year)	$V_{p1}(h) = 0.8\ v_{m50}(h)$
Annual mean wind speed	Mean value over at least 1 year	>1a	Hub height h	- / -	v_{ave} according to type class	- / -	$v_{ave} = 0.18 \times v_{m50}(h)$ or $0.20 \times v_{m50}(h)$
Design wind speed / rated wind speed	Wind speed at which the design output / rated output of the wind turbine is reached	10 minutes	Hub height h	- / -	v_r	- / -	v_r