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**Wind energy generation systems –
Part 6: Tower and foundation design requirements**

**Systèmes de génération d'énergie éolienne –
Partie 6: Exigences en matière de conception du mât et de la fondation**



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INTERNATIONAL ELECTROTECHNICAL COMMISSION

WIND ENERGY GENERATION SYSTEMS –**Part 6: Tower and foundation design requirements****FOREWORD**

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FDIS	Report on voting
88/751/FDIS	88/754/RVD

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INTRODUCTION

This document has been developed for the design of onshore wind turbine towers and foundations that will build on and complement the IEC 61400-1 relating to design criteria and provide a complete set of technical requirements for the structural and geotechnical design. The requirements are also applicable to wind turbines covered by IEC 61400-2. It is envisaged that the proposed work will be followed by the development of another part, directed towards the design of offshore support structures, thus also complementing IEC 61400-3-1.

Civil engineering practices associated with the scope of the standard have regional variations. It is not the intention of this document to conflict with those practices but to supplement them particularly in ensuring that all important features of typical wind turbine towers and foundations are fully and correctly considered. To this end, the relevant parts in existing standards for design of steel and concrete structures and for geotechnical design have been identified for participating countries and regions.

The principles included in this document apply to the sections of the tower of an offshore fixed structure above the splash zone if the loading has been calculated according to IEC 61400-3-1.

This document will include the evaluation and calibration of partial safety factors for material strengths to be used together with the safety elements in IEC 61400-1 and IEC 61400-2 for loads and for verification of static equilibrium.

WIND ENERGY GENERATION SYSTEMS –

Part 6: Tower and foundation design requirements

1 Scope

This part of IEC 61400 specifies requirements and general principles to be used in assessing the structural integrity of onshore wind turbine support structures (including foundations). The scope includes the geotechnical assessment of the soil for generic or site specific purposes. The strength of any flange and connection system connected to the rotor nacelle assembly (including connection to the yaw bearing) are designed and documented according to this document or according to IEC 61400-1. The scope includes all life cycle issues that may affect the structural integrity such as assembly and maintenance.

The assessment assumes that load data has been derived as defined in IEC 61400-1 or IEC 61400-2 and using the implicit reliability level and partial safety factors for loads.

2 Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes requirements of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

IEC 61400-1:2019, *Wind energy generation systems – Part 1: Design requirements*

IEC 61400-2, *Wind turbines – Part 2: Small wind turbines*

IEC 61400-3-1:2019, *Wind energy generation systems – Part 3-1: Design requirements for fixed offshore wind turbines*

ISO 2394:2015, *General principles on reliability for structures*

ISO 22965-1, *Concrete – Part 1: Methods of specifying and guidance for the specifier*

ISO 22965-2, *Concrete – Part 2: Specification of constituent materials, production of concrete and compliance of concrete*

ISO 22966, *Execution of concrete structures*

ISO 6934 (all parts), *Steel for the prestressing of concrete*

ISO 6935 (all parts), *Steel for the reinforcement of concrete*

ISO 9016:2012, *Destructive tests on welds in metallic materials – Impact tests – Test specimen location, notch orientation and examination*

ISO 12944 (all parts), *Paints and varnishes – Corrosion protection of steel structures by protective paint systems*

EN 1993-1-9:2005, *Eurocode 3: Design of steel structures – Part 1-9: Fatigue*

EN 1993-3-2:2006, *Eurocode 3: Design of steel structures – Part 3-2: Towers, masts and chimneys – Chimneys*

3 Terms and definitions

For the purposes of this document, the terms and definitions given in IEC 61400-1, IEC 61400-2 and the following apply.

ISO and IEC maintain terminological databases for use in standardization at the following addresses:

- IEC Electropedia: available at <http://www.electropedia.org/>
- ISO Online browsing platform: available at <http://www.iso.org/obp>

3.1

assessment

total set of activities performed in order to find out if the reliability of a structure is acceptable or not

3.2

characteristic load

load accounting for required exceedance probability level and without partial safety factor for loads

3.3

characteristic buckling resistance

load associated with buckling in the presence of inelastic material behaviour, the geometrical and structural imperfections that are inevitable in practical construction, and follower load effects

3.4

component class

classification of the wind turbine structural components according to redundancy and safety requirements

Note 1 to entry: Refer to IEC 61400-1.

3.5

component temperature

local temperature which will affect the material properties of a component

Note 1 to entry: The temperature shall be taken to be the ambient temperature unless protective or active means are provided to change the temperature.

3.6

design lifetime

complete period of time for which the wind turbine will be designed to resist the specified loading including maintenance, idling, power production, starting and stopping

3.7

design load

design force

load (force) used in the action vs resistance equation for a limit state accounting for the required exceedance probability level and partial safety factor for loads

3.8

design resistance

resistance used in the action vs resistance equation for a limit state accounting for the required exceedance probability level and partial safety factor for materials

3.9

design situations

sets of physical conditions representing the real conditions occurring during a certain time interval for which the design will demonstrate that relevant limit states are not exceeded

3.10

dynamic stiffness for foundation

tangent to the plot of linear or angular displacement of the foundation against force or moment applied to the foundation at zero or small strain ($<10^{-4}$ soil shear strain)

Note 1 to entry: Normally for concrete foundations, this is calculated for the effect of the soil mechanics alone assuming a rigid foundation. Where foundation flexibility is non-negligible, this should be taken into account in calculating dynamic stiffness.

3.11

effect of actions

effect of actions (or action effect) on structural members or on the whole structure

Note 1 to entry: Internal force, moment, stress and strain are examples of action effect on structural members. Deflection and rotation are examples of action effect on the whole structure.

3.12

fatigue limit state

structural failure due to damage accumulation under effects of repeated loading

3.13

geometrically and materially nonlinear analysis

GMNA

analysis performed on the global structure based on shell bending theory applied to the perfect structure, using the assumption of nonlinear large deflection theory for the displacements, and adopting a nonlinear elasto-plastic material law ignoring the effect of strain hardening

3.14

geometrically and materially nonlinear analysis with imperfection included

GMNIA

analysis performed on the global structure with imperfections explicitly included, based on shell bending theory applied to the imperfect structure, including nonlinear large deflection theory for the displacements, and adopting a nonlinear elasto-plastic material law ignoring the effect of strain hardening

Note 1 to entry: A bifurcation eigenvalue check is included at each load level.

Note 2 to entry: Examples of imperfect structure are unintended deviations from the ideal shape, imperfections in boundary conditions and residual stresses.

3.15

internal loads

three orthogonal forces and three orthogonal moments that are reacted on an arbitrary plane cut through the structure

Note 1 to entry: This relates to the usage of the term "loads" in IEC 61400-1, IEC 61400-2 and IEC 61400-3-1. This differs from the use in other areas of civil engineering. Often, the arbitrary plane is aligned with some physical interface or with two axes of a local axis system. The singular internal load would be one of the three forces or three moments. See also 5.4.6.

3.16

materially nonlinear analysis

MNA

analysis performed on the global structure based on shell bending theory applied to the perfect structure, using the assumption of small deflections, but adopting a nonlinear elasto-plastic material law ignoring the effect of strain hardening

3.17**nominal concrete cover**

layer of concrete between the concrete surface and the closest reinforcement surface including the specified tolerance for placing of reinforcement

Note 1 to entry: The nominal concrete cover shall be calculated as the minimum concrete cover plus the specified tolerance.

3.18**nominal ratio**

ratio of values that are fixed on non-statistical bases, for instance on acquired experience or on physical conditions

3.19**nominal value**

value fixed on non-statistical bases, for instance on acquired experience or on physical conditions

3.20**partial safety factor for load**

factor for increasing a characteristic value of an effect of actions to calculate the design value of an effect of actions

3.21**partial safety factor for material**

factor for decreasing a characteristic value of material property to calculate the design value of material property

3.22**plastic limit load**

limit load, determined assuming idealized conditions of elastic-plastic material behaviour, perfect geometry, perfect load application, perfect support and material isotropy

Note 1 to entry: The plastic limit load is determined assuming the idealized conditions of elastic-plastic material behaviour, perfect geometry, perfect load application, perfect support and material isotropy.

3.23**plastic reference resistance**

plastic limit load, determined assuming the idealised conditions of rigid-plastic material behaviour, perfect load application, perfect support and material isotropy

3.24**post-tensioned concrete**

prestressing of the structure that has been achieved by tensioning the tendons or bolts after casting of the concrete

Note 1 to entry: Tendons can be either bonded to the concrete or un-bonded.

3.25**prestressed concrete**

concrete structure which is prestressed before it is taken into operation

Note 1 to entry: Prestressing is applied by tendons or bolts, and the tendons may be pre-tensioned (before casting) or post-tensioned (after casting).

3.26**pre-tensioned concrete**

concrete which is pre-tensioned by tensioning the tendons prior to casting of the concrete

Note 1 to entry: Tendons are bonded to the concrete.

3.27

primary structure

structural elements which are designed for the purpose of carrying the loads due to the rotor nacelle assembly

3.28

reference standard

document presenting co-ordinated design analysis methods with actions and resistance characteristic values and safety factors, published by a credible expert private, regional, national or international body

3.29

rotor nacelle assembly

RNA

part of a wind turbine carried by the support structure

3.30

secondary structure

structural elements which are designed to carry the loads due to equipment such as access systems, cables, cabinets, internal devices for example dampers

3.31

series of standard

co-ordinated series of documents presenting design analysis methods with action and resistance characteristic values and safety factors published by a credible expert private, regional, national or international body

3.32

signed von-Mises stress

equivalent stress with (theoretical) direction derived from principal stresses

3.33

S1

SLS characteristic load

serviceability limit state load level for design lifetime actions, which relate to continued correct operation of the wind turbine such as clearance of components, and engineering fluid levels, concrete cracking control

3.34

S2

SLS 10⁻⁴ frequent load case

serviceability limit state load level for frequent actions, which are exceeded for 10⁻⁴ of the lifetime, which relate to fracture toughness, corrosion, concrete cracking control, and bonds in prestressed concrete

3.35

S3

SLS 10⁻² frequent load case

serviceability limit state load level for the equivalent to frequent actions, which are exceeded for 10⁻² of the lifetime, which relate to cracked concrete stiffness checks, concrete cracking control, foundation stiffness, inclination and settlement, ground gapping, and pile tension limits

3.36

serviceability

ability of a structure or structural element to perform adequately for normal use under all expected actions

3.37**serviceability limit state**

state which corresponds to conditions beyond which specified service requirements for a structure or structural element are no longer met

3.38**static stiffness for foundation**

secant of the point on the plot of linear or angular deflection of the foundation against force or moment applied to the foundation at a strain compatible with the applied load case

Note 1 to entry: Normally for concrete foundations this is calculated for the effect of the soil deformation alone assuming a rigid foundation. Where foundation flexibility is non-negligible, this should be taken into account in calculating static stiffness.

3.39**support structure**

part of a wind turbine consisting of the tower, sub-structure and foundation

Note 1 to entry: See Figure 1 of IEC 61400-3-1:2019.

3.40**tower section**

unit of a tower that can be transported from a factory to the installation site, especially of a steel tubular tower made of a number of permanently connected smaller steel tubes, the whole unit being provided with bolted connections for on-site assembly at the upper and lower ends

3.41**ultimate limit state**

limit states which generally correspond to the maximum load bearing capacity

Note 1 to entry: This generally corresponds to the maximum load-carrying resistance of a structure or structural element but in some cases to a strain or deformation limit.

3.42**wind turbine specification**

document and associated materials containing information relating to the full turbine system and the rotor nacelle assembly (RNA) enabling the tower and foundation design, and identifying the wind turbine that can be used with the tower and/or foundation

4 Symbols and abbreviated terms**4.1 Symbols**

A_1	reduction factor parameter
A_s	effective stress cross section
A'	effective foundation area around the line of action of the resultant force for F_{zd}
b	weld neck thickness (normally equal to the thickness of the connected tower shell)
b_{tot}	flange width
B_1	reduction factor parameter
c	flank height of the weld preparation
C_1	reduction factor for the influence of the opening
D_{Ed}	damage sum
e	eccentricity, generally of loading

f_{bd}	design value of bond strength between rock and cement grout
f_{ck}	characteristic compressive concrete cylinder strength
f_{cck}	characteristic grout compression strength
$f_{c,r}$	meridional design buckling stress
f_{pk}	characteristic tensile strength of prestressing reinforcement steel
$f_{p0,1k}$	characteristic 0,1 % proof-stress of prestressing steel
f_{yk}	characteristic reinforcement yield strength
F_d	ultimate design load
F_d	design value of destabilising action including unfavourable partial safety factor for load
$F_{p,C'}$	preload bolt force used in the design calculations
$F_{p,C}$	preload bolt force given in the relevant design code or technical approval
F_{zd}	design value of vertical force acting on the soil formation
G	shear modulus of the soil reduced from G_o to account for non-zero soil strain
G_o	small-strain shear modulus of the soil
$K_{R,dyn}$	dynamic rotational stiffness of the foundation subjected to overturning moments
$K_{R,stat}$	static rotational stiffness of the foundation subjected to overturning moments
l	distance from transition radius to weld preparation
L_{anchor}	full length of an anchor
L_{fixed}	bonded length of an anchor
L_{free}	free length of an anchor
L_f	factor for loss of preload, which is not considered in the applied design code
$M_{d,overturning}$	design value of destabilizing moment from wind load and other loads
$M_{d,stabilizing}$	design value of stabilizing moment from gravity load and backfill
n	number of force transmitting interfaces in the connection
$n(\Delta\sigma_i)$	number of load cycles for a given range $\Delta\sigma_i$
$N(\Delta\sigma_i)$	number of load cycles at failure for the range $\Delta\sigma_i$
P_k	characteristic tensile strength of pre-stressing steel
$P_{lock-off}$	lock off load for rock anchor
$P_o, P_{lock-off}$	load transferred to the foundation by the rock anchor after completion of stressing operation (after locking)
P_p	anchor proof load
$P_{p0,1k}$	anchor yield strength
q_{Ed}	plastic (uniform) ground pressure based on eccentricity calculation
q_{Rd}	design values of bearing capacity of soil in ultimate limit state
r	transition radius
R	foundation radius in contact with the subgrade
R_a	surface roughness parameter

R_d	design value of stabilising action comprising gravity and soil resistance
R_{Rrk}	reduced reference buckling resistance
R_{Rpl}	plastic reference resistance
t_f	flange thickness
t_{tot}	total flange thickness including weld neck
$T_{\text{extreme,min/max}}$	extreme temperature range
$T_{\text{erection,min/max}}$	allowable temperature range to be defined for the erection of the tower
ΔT_k	characteristic temperature action on concrete towers
ΔT_1	uniform temperature difference in concrete towers with respect to the temperature at the time of erection
ΔT_2	cosine shaped temperature difference on concrete towers caused by solar irradiation
ΔT_3	temperature difference for concrete towers between inside and outside wall surfaces
α	ratio of moduli of elasticity for reinforcement steel and concrete
$\gamma_{f,Temp}$	partial safety factor for temperature actions on concrete towers
γ_{inf}	partial safety factor for favourable prestress
γ_M	partial safety factor for material
γ_{M3}	partial safety factor for rock anchor bond strength
γ_{sup}	partial safety factor for unfavourable prestress
δ	opening angle along the girth of the door opening
ε_{tot}	total strain
θ	diameter of bore hole
μ	friction coefficient
ν	poisson's ratio (of the soil)
σ_{hs}	structural hot spot stress for a linear elastic material model
$\sigma_{y,d}$	design yield stress of the material
$\Delta\sigma_{R-N}$	fatigue strength represented by "S-N-curves" (Wöhler curves)
$\sigma_{x,R,d-EC}$	meridional design buckling stress according to EN 1993-1-6
$\sigma_{ }$	one component of the Cauchy stress tensor in one principal direction of Cartesian coordinate system (e.g. σ_x)
$\sigma_{\perp}$	one component of the Cauchy stress tensor in a principal direction of Cartesian coordinate system perpendicular to $\sigma_{ }$ (e.g. σ_y or σ_z)
τ	Shear stresses in component in the plane of the directions of $\sigma_{ }$ and $\sigma_{\perp}$

4.2 Abbreviated terms

DLC	design load case
FLS	fatigue limit state
GIR	ground investigation report
GMNA	geometrically and materially nonlinear analysis

GMNIA	geometrically and materially nonlinear numerical buckling analysis with imperfections included
MNA	materially nonlinear analysis
psfl	partial safety factor for loads
psfm	partial safety factor for materials
RNA	rotor nacelle assembly
SI	geotechnical site investigation
SLS	serviceability limit state
ULS	ultimate limit state

5 Design basis including loading

5.1 General

Clause 5 is intended to cover the engineering design basis required for the major engineering disciplines to establish support structures for onshore wind turbines.

In order to provide a standard that will be useful to the industry, a comprehensive treatment of some topics is provided where there is currently no relevant international reference. For well-known topics for the design of structural members, Clause 5 is intended to be used in conjunction with suitable reference standards for support structure design. It is allowable to use suitable reference standards that provide the required level of safety as specified in IEC 61400-1 or IEC 61400-2. Only other IEC and ISO documents will be referenced directly in the text.

5.2 Basis of design

5.2.1 Basic principles

The structure shall be dimensioned in such a way that it withstands the applied actions during pre-commissioning with and without nacelle and rotor, erection and the design lifetime. If required, this shall also include transport to initial installation or decommissioning. In general, a consideration of all load cases given in IEC 61400-1 or 61400-2 is sufficient for dimensioning. However, the additional loads due to inclination of the tower and the foundation as well as influences from temperature and vortex induced vibrations shall be taken into account. A consideration of additional actions according to local standards may also be necessary.

The design of the structure shall be in accordance with a selected recognized series of standards or a reference standard. The series of standards or reference standards shall appropriately cover the necessary areas of the design of wind turbine support structures and where necessary be supplemented with additional requirements as specified in this document. In particular, the verification methods shall correctly account for the execution standards of this series to ensure a consistent design. The combination of standards from different series shall only be permitted if it has been verified that the combination does not reduce the reliability level.

The series of standards or reference standard shall comply with IEC 61400-1 or IEC 61400-2, wherever necessary, for instance in regards to reliability levels and environmental conditions.

Minimum values of partial safety factors for materials may be taken from the selected series of standards or reference standard unless specified in this document and as long as it is verified that these fulfil the required reliability level stated in IEC 61400-1 or IEC 61400-2 in combination with the given characteristic loads.

In Annex A, some standards are listed that fulfil the requirements above. The application of other standards not listed in Annex A is possible if the required reliability level stated in

IEC 61400-1 or IEC 61400-2 is met. The methods and safety factors shall not be mixed from different reference standards so as to reduce the safety level.

The defined resistances in the selected recognized design codes are only valid if the material requirements according to 6.3 and 7.3 are fulfilled.

5.2.2 Durability

The support structure shall be designed in such a way that time dependent changes of characteristics do not affect the behaviour of the support structure unexpectedly over the design lifetime. This includes also sufficient dimensioning against fatigue and, if applicable, earthquakes.

Based on the design assumptions and environmental conditions, appropriate corrosion protection systems shall be considered for the whole support structure. Where corrosion protection is done by a coating, it shall be chosen and applied according to ISO 12944 (all parts).

Furthermore, the service and maintenance intervals according to Clause 9 shall be observed.

5.2.3 Principles of limit state design

Conditions that concern human safety or the integrity of the structure are considered as ultimate or fatigue limit states. As a simplification, the conditions before component failures may be treated as limit states.

In general, verifications for the following limit states shall be carried out for structures:

- failures due to excessive deformations or due to the transfer of the entire structure or of its components to a kinematic state or rupture or an unstable condition (ultimate limit state, ULS);
- failure of the entire structure or its components due to fatigue or other time-dependent effects (fatigue limit state, FLS);
- limit states that consider the function of the structure or one of its components under normal service conditions or the appearance of the structure (serviceability limit state, SLS).

The ultimate and fatigue limit states shall be verified for the design situations. Verifications for the fatigue limit state shall be referred to the design lifetime of the structure.

5.2.4 Structural analysis

The dimensioning shall be carried out with appropriate models of the structural system in respect of the limit states.

It shall be verified that the ultimate and fatigue limit states are not exceeded, if the appropriate design values are used for

- the loads,
- the material properties,
- the component properties, and
- the geometric dimensions in the calculation models.

The verifications shall be carried out for all decisive design situations and load cases. Where necessary, imperfections and deformations of the structure shall be included.

The verification shall also include the assessment of damping and frequency of the system as specified in the design requirements.

5.2.5 Assessments by tests

The design and calculation of the structure may also be carried out in combination with tests as described in ISO 2394 and IEC 61400-1:2019, Annex K. Laboratories carrying out the tests should ensure that suitable requirements for calibration are followed.

NOTE Guidance about good practices for calibration can be found in ISO/IEC 17025.

In respect of values of resistance in principle, the derivation of both characteristic or design values from tests is possible. The tests shall be carried out in a way that the required safety and reliability level for the design situation according to IEC 61400-1 or IEC 61400-2 is met in the verification (see also 7.6.2.2 of IEC 61400-1:2019).

If the series of standards or reference standard used for the verification of the structure provides rules for assessments by tests, these shall also be considered.

5.3 Materials

Specific requirements are given for each type of material in 6.3 and 7.3.

All material shall be treated in accordance with relevant standards in regards to quality requirements and test conditions.

Characteristic values of the material properties shall be taken from corresponding standards unless sufficient test data is available to reach the required reliability level implied in IEC 61400-1.

5.4 Loads

5.4.1 Use of IEC 61400-1 or IEC 61400-2 load cases and partial safety factors for loads

The loads used to assess the ultimate limit state and fatigue limit state of towers and foundations according to this document shall be derived according to IEC 61400-1 or IEC 61400-2 and shall use the safety factors specified in IEC 61400-1 or IEC 61400-2 unless otherwise specified in this document. The tower or foundation may be designed for alternative environmental conditions, i.e. conditions that differ from those used in the rotor nacelle assembly verification. Additionally, the site specific loads may be distributed around the tower to take account of the wind direction, for example fatigue loads on any one point on the tower or foundation may be reduced by accounting for wind direction where for example position of openings can be arranged in the less fatigue loaded zones. This shall only be applied in the case of towers or foundations for a specific site.

The dynamic analysis methods for internal load calculation shall use the soil/foundation dynamic stiffness for all load levels.

In regions, where seismic activity may possibly affect the loading, the seismic load cases shall also be calculated as specified in accordance with IEC 61400-1 or IEC 61400-2 and the appropriate regional seismic standards. A guideline on the seismic load estimation on tower and foundation is described in Annex O.

5.4.2 Superseding of IEC 61400-1 or IEC 61400-2 partial safety factors for materials

The partial safety factors for materials given in this document supersede requirements in IEC 61400-1 or IEC 61400-2 for onshore wind turbine tower, foundation and geotechnical design.

5.4.3 Serviceability load levels

5.4.3.1 General

Serviceability load levels shall be used for the serviceability limit states, that is, according to ISO 2394:2015, 2.2.10, a "limit state concerning the criteria governing the functionalities related to normal use"¹.

Wind turbine towers and foundation differ from most civil engineering structures in a way that they have a very small ratio of static loads to dynamic loads. Therefore, for design of wind turbine towers and foundation, SLS loads according to ISO 2394 are modified such that the SLS load levels S1, S2 and S3 are applied, corresponding to the characteristic load level and to two frequent load levels, of different levels of frequency. The more frequent load level (see S3 below) is used for situations generally analysed with a quasi-permanent load level.

In case of serviceability limit state analyses, the same load simulations shall be used as above but the partial safety factor for load specified in IEC 61400-1 or IEC 61400-2 shall be replaced by $\gamma_{\text{serviceability}}$, where:

$$\gamma_{\text{serviceability}} = 1,0 \quad (1)$$

NOTE There are multiple load levels used for SLS analyses. Some of these load levels are directly specified in IEC 61400-1 or IEC 61400-2. However, others are derived from the same load simulations using the specifications below.

5.4.3.2 S1 – Characteristic value of normal and transport cases

The values are equal to extreme values from the complete set of cases denoted by use of normal psfl. This SLS load level shall normally be represented by time history data in an orthogonal system at relevant points in the system, which may then be used to generate time histories of the stresses through linear and non-linear models. It is allowed to represent this SLS load level by a matrix of orthogonal loads at critical points in the structure if the combination of loads will give a conservative stress.

The psfl for SLS as required by this document shall be applied.

5.4.3.3 S2 – $p = 10^{-4}$ from fatigue spectrum load levels

The values are equal to 0,01 percentile values from the complete set of design load cases (DLC) 1,1 as required by IEC 61400-1:2019, 7.6.2. For each structural component, a probability distribution shall be created for the load in the direction that produces the highest stresses. Alternatively, the following simplifications may be applied:

- for the foundation, the loads in the cross wind direction may be ignored;
- for the tower, the required percentile value may be derived from a probability distribution of the resultant load at each tower location.

NOTE $p = 10^{-4}$ implies a period of exposure to higher loads of 0,87 hours per year.

5.4.3.4 S3 – $p = 10^{-2}$ from fatigue spectrum load levels

This is derived as for S2 above, but the 1 percentile values are used.

NOTE $p = 10^{-2}$ implies a period of exposure to higher loads of 87 hours per year.

¹ No serviceability limit states are defined in IEC 61400-1.

5.4.4 Load combinations in ULS

5.4.4.1 Prestressing in concrete

Prestressing shall be classified as a permanent action. Prestress in most situations is intended to be favourable and, for the ultimate limit state verification, the partial safety factor for loads $\gamma_{f,prestress}$ shall be used.

$$\gamma_{f,prestress,favourable} = 0,9 \quad (2)$$

If the prestress is unfavourable, then the minimum partial factor for global and local analysis shall be as follows.

$$\gamma_{f,prestress,unfavourable} = 1,1 \quad (3)$$

When measures are taken to reduce the uncertainty on the prestress load, the factors can be taken as $\gamma_{f,prestress,favourable} > 0,9$ and $\gamma_{f,prestress,unfavourable} < 1,1$. While a high accuracy of the tensioning devices may improve the accuracy of the initial level of prestress, the remaining uncertainty with respect to the losses of prestress shall also be considered if $\gamma_{f,prestress,favourable}$ and $\gamma_{f,prestress,unfavourable}$ are to be modified.

For local effects (e.g. load introduction areas) and stability analysis (e.g. buckling), $\gamma_{f,prestress,unfavourable}$ shall in no case be taken as less than 1,1.

5.4.4.2 Seismic loading

Seismic loading shall be superposed with operational loading (see IEC 61400-1 or IEC 61400-1:2019, 11.6). The partial safety factor for seismic loading is given in IEC 61400-1.

5.4.4.3 Other climatic loads such as snow loads

Snow and climatic loads are normally considered insignificant but shall be considered if relevant in site specific cases serviceability limit state load, S1, with a partial safety factor for load as follows.

$$\gamma_{f, climatic action} = 1,0 \quad (4)$$

5.4.4.4 Temperature effects

Thermally induced stresses shall be checked in the presence of serviceability limit state load, S1, in combination with the temperature effect with partial safety factor for load as follows, when temperature effects are the dominating load.

$$\gamma_{f,temperature action} = 1,35 \quad (5)$$

Load combinations with other loads shall be found in 7.2.3.1 for concrete structures.

5.4.4.5 Buoyancy effects

Buoyancy effects shall be accounted as described in IEC 61400-1:2019, 7.6.2.3.

5.4.5 Structural damping values to be used in load calculations

The tower and foundation modal damping values used in the derivation of loads described in 5.4.1 shall be justified by relevant measurement or analysis. Annex P gives some guidance.

The damping shall be calculated conservatively accounting for geometry, materials and mode shapes in the case of towers consisting of a combination of concrete, steel tubular, lattice or other structural sections.

Structural damping does not include the following sources:

- aerodynamics of the structure;
- control actions, either aerodynamic or electro-mechanical;
- mechanical dampers;
- soil damping.

However, all of the above may be accounted for in the appropriate part of analysis, for example aerodynamics and control actions damping are modelled in the aero-elastic modelling. Mechanical dampers and soil effects may be modelled through explicit addition to the structural damping.

5.4.6 Definitions and methods for use of internal loads

The loads provided by the methods in IEC 61400-1 or IEC 61400-2 are external loads that can be used to calculate the loads between internal sections and interfaces such as flanges. The methods of structural analysis shall take account of this. This is a particular issue for use of loads with finite element modelling where it is necessary to ensure that internal loads are represented correctly at the relevant interface but cannot be applied directly as the interface cannot be a free edge or shall be constrained correctly. See also Annex N.

5.4.7 Definition of required load data for fatigue analysis

The fatigue loads shall be available as time history data in an orthogonal system at relevant points in the system, which may be used to generate the stresses at critical locations through linear and non-linear models. The load shall be appropriate to the structural component under analysis.

Fatigue analysis methods shall use rain flow cycle counting (refer to IEC 61400-1 or IEC 61400-2).

The data shall be presented as Markov matrices which give the number of cycles of load for each bin of load mean combined with each bin of load cycles. Analyses shall either use the most severe value of the bin range to ensure conservatism or a value weighted according to the used SN-slope. There is no limit to how large or small the bin ranges may be.

Other forms of fatigue load representation are allowed to be used for appropriate purposes. The use of range cycles without means is allowed for certain structural details, for example welding, or where a conservative mean load assumption is made. Damage equivalent loads are only allowed to be used in situations with selection of S-N curve slope and reference number of cycles, where the single linear slope is chosen to conservatively represent the actual S-N curve.

5.4.8 Definition of required load data for extreme load level

The extreme load level shall be represented by a table of design loads (three forces and three moments for a defined orthogonal axis system, plus the resultant horizontal force and overturning moment) at critical points in the structure selected in such a way that the

combination of loads will give the most conservative stress. This table provides the extreme value in each load component and the contemporaneous values in the other load components².

For clarity in some strength calculation methods, the table of orthogonal characteristic loads shall also be provided, that is, without the application of ψ_{fl} .

The use of contemporaneous loads in the strength assessment shall follow IEC 61400-1 or IEC 61400-2.

5.4.9 Vortex induced vibration

Lateral oscillations forced by flow instability due to the local tower aerodynamics, especially at the tower modal frequencies, commonly known as vortex induced vibrations, shall be considered in the calculation of fatigue life and extreme loading³. The damage caused by the vortex induced cross wind vibrations (ViV) shall be assessed for all states of the tower and turbine assembly process, for periods of maintenance and standby.

The damage caused by ViV may be neglected up to a value of damage sum of $D = 0,10$.

Otherwise, the damage due to the ViV in excess of 0,1 and the damage in cross wind direction for fatigue loads due to the turbine lifetime shall be added.

In the calculation of the actions due to the ViV, the aerodynamic damping shall not be applied.

5.4.10 Loads due to geometric tolerances and elastic deflections in tower verticality

The loading effect of displacement of the tower and rotor-nacelle assembly away from the designed undeflected vertical position shall be included in the loads as defined in IEC 61400-1 or IEC 61400-2 or shall be separately calculated as a requirement of this document.

The following influences shall be considered:

- horizontal displacement due to elastic deflection;
- difference between static and dynamic rotational stiffness of the foundation;
- imperfections;
- loadings due to differential settlement;
- temperature, including differential heating affects;
- non-linear deflections due to cracked cross-sections of concrete towers and foundations.

The load calculation methods required by IEC 61400-1 or IEC 61400-2 include the loading effect of gravitational loads due to elastic deflections of the structures from initial positions, including the apparent stiffness reduction.

The allowable tolerances in tower verticality⁴ shall be stated in the design documentation and shall include initial effects during assembly and installation, short term effects due to thermal expansions and long term effects due to permanent soil subsidence. The effect of tower verticality on gravitational loads shall be taken into account separately during the structural analysis of tower and foundation.

² The highest resulting load will not necessarily be governed by an extreme value in one direction.

³ Procedure 2 in EN 1991-1-4:2005, Annex E, can be used to estimate the amplitude of vortex induced vibrations and the corresponding inertia force per unit length.

⁴ The wind industry has specified typical limits of 5 mm lateral displacement per metre of height for the installation, manufacturing and thermal effects, and 3 mm/m for uneven subsidence.

Additional loads due to second order effects shall be considered in the tower and foundation calculation for ultimate limit state analysis. As a simplification, they may be assumed to have negligible influence on the stiffness of the structure. Accordingly, second order effects may be included as additional quasi-static loads (predominantly bending loads due to vertical forces and horizontal displacements of the tower).

Many state of the art aero-elastic load simulation software include the above-mentioned second order effects so additional quasi-static analysis above may not be required. This shall be made clear in the mechanical and structural design loading specifications.

5.5 Load data and interface reporting requirements

5.5.1 Purpose

A tower or foundation shall only be shown to be compliant with this document if the design has been based on certain data which is specific to the wind turbine rotor nacelle assembly to be used with that tower and foundation. The interface document may relate to a tower and foundation combination, or a foundation only.

5.5.2 Wind turbine specification

The load data and interface reporting shall present certain cardinal information for the wind turbine to ensure unambiguous association with a specific wind turbine rotor nacelle assembly configuration. This data shall include at least the following:

- model designation;
- RNA design wind class or Class S environmental conditions (see below in case of alternative environmental conditions);

It is allowable for any of the environmental conditions for the tower design to differ from those for the RNA. In case of such alternative environmental conditions, these conditions shall be individually specified. These alternative environmental conditions would not imply a change in the site suitability of the RNA.

- top flange height above ground level;
- assumed foundation inertial parameters, (may be stated to be negligible);
- allowable tolerances, including verticality due to build and settlement;
- foundation and geotechnical allowable dynamic and static (if appropriate) stiffness level, rotational and lateral.

The following information is only required if the data is intended for a tower design:

- assumed system fundamental bending mode frequency range in both principal axes;
- damping values using in load derivation, including justification if not compliant with 5.4.4.5;
- assumed tower external geometry;
- tower top flange configuration;
- other tower constraints, for example cables, dampers.

The following information is only required if the data is intended for a foundation design:

- tower – foundation interface details;
- foundation interfaces for electrical and other services, including limits on settlement;
- also note requirements relating to piles in 8.6.

The following parameters are not mandatory but may assist in the association of the data to the turbine type:

- rotor blade designation;

- rated power;
- undeflected rotor diameter;
- control software and parameter identification.

If the loads analysis shows that the inclusion of tower modes higher than the fundamental increases any critical load by more than 3 %, the assumed range for the other modes shall be provided.

In the case that the tower top flange design is provided separately, sufficient information shall be provided to reproduce this design and to ensure a correct geometric, functional and structural interface.

5.5.3 Time history data

All relevant loading data shall be made available in time history form for tower design.

5.5.4 Load origins

It shall be ensured that data is available at all critical points. It is suggested that at least the following critical points, known as origins, are considered; tower top, tower bottom, and at relevant levels (e.g. connection points for tower sections secured with fasteners) in the tower.

5.5.5 Load components

Load data shall be presented in six component form as internal loads at the required origins on a specified cross section; forces in three orthogonal right-handed axes and moments about those axes, where one axis is normal to the cross section.

5.6 General structural design requirements

5.6.1 Secondary structural influence

The effect of secondary structural elements on the strength or fatigue life of the primary structure shall be included in the analyses.

5.6.2 Fatigue analysis

The calculated fatigue damage due to different range levels shall be summed in the fatigue verification according to the Palmgren-Miner's rule. The damage sum D_{Ed} that results from the critical fatigue loading shall satisfy the following condition:

$$D_{Ed} = \sum [n(\Delta\sigma_i) / N(\Delta\sigma_i)] \leq 1,0 \quad (6)$$

where

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

$N(\Delta\sigma_i)$ is the number of load cycles at failure for the range $\Delta\sigma_i$ accounting for mean stress when appropriate and for the partial safety for materials and loads.

The damage sum limits shall be lower than 1,0 if specified in the reference standard.

5.7 Delivery documentation

The documentation shall be consistent with IEC 61400-1:2019, 12.6.

6 Steel towers

6.1 General

Steel design principles are mostly well defined and documented in the literature, including reference standards. Clause 6 provides additional or more detailed rules and guidance that are specific to onshore wind turbine towers and complement the more general requirements of existing standards.

Clause 6 provides requirements in the form of general statements and analytical methods. Guidance on acceptable calculation methods which are considered to comply with the requirements are presented in Annex A to Annex G.

6.2 Basis of design

The basis of design requirements are given in 5.1.

6.3 Materials

6.3.1 General

In 6.3, the requirements for structural steels and bolt material are specified. Other materials may be used if their material properties (e.g. ductility) comply with the assumptions of the verification methods.

6.3.2 Structural steels

6.3.2.1 Structural steels requirements

Only structural steel that meets the following requirements regarding the ductility, fracture toughness and weldability shall be used.

To ensure a better weldability, better resistance of aging and a lower brittle fracture risk, structural steels shall be semi-killed under normal climate conditions and fully-killed under cold climate conditions (deoxidation according to recognized methods).

NOTE A list of structural steels that fulfil the requirements regarding ductility and fracture toughness are given in Annex B.

6.3.2.2 Ductility

Nominal ratio of tensile strength to yield strength shall be equal to or greater than 1,10.

Elongation at failure shall be greater than or equal to 15 %.

6.3.2.3 Fracture toughness

The fracture toughness is defined as "minimum impact energy to be absorbed at test temperature" for standard size V-shape probes according to ISO 148-1. The probe cutting direction of welded specimens shall consider ISO 9016:2012.

The required fracture toughness shall be determined considering:

- the lowest component temperature (radiation losses included);
- the part thickness;
- the critical stress level in regard to the yield strength (depending on the plate thickness);
- the hot dip galvanization, if applicable.

The determination of the required fracture toughness of the material shall be done according to the reference standard or by fracture mechanic methods.

The critical stress level may be determined as nominal stresses in the critical cross-sections and structural parts at the location of the expected crack propagation (e.g. at circumferential collar stiffeners or flanges)⁵.

The lowest component temperature shall correspond with the critical stresses that shall be determined at load level S2 (see 5.4.3.3).

For normal climate conditions, a lowest component temperature of –30 °C shall be applied for the steel grade selection. For cold climate conditions, the temperature for steel grade selection shall be 10 °C lower than the lowest ambient temperature during operation. Effects of cold forming and increased strain rates may then be neglected.

As an alternative to the above, the lowest component temperature may also be determined by using the actual lowest temperature in the normal temperature range. In that case, effects of cold forming and increased strain rate shall be taken into account⁶.

The fracture toughness of the material shall be verified by destructive testing. The results of the toughness tests shall be listed in material certificates. Here, pass-fail criteria of the Charpy tests may be taken from the reference standard.

Regarding the fracture toughness of ring flange connections, the decisive material thickness may be chosen under the following conditions.

- a) For ring flanges with a weld neck, the decisive material thickness correlates to the connecting tower shell thickness (b according to Figure 1) under the following conditions⁷.

The distance between the circumferential weld seam toe and the finished product (upper side of the flange surface) shall satisfy Formula (7) (parameters r and l according to Figure 1). Otherwise, the flange shall be evaluated as "without weld neck".

$$r + l \geq \max. (r, 10 \text{ mm}) \quad (7)$$

If no weld preparation is specified, a double V butt weld preparation with a bevel angle of 45° (to both sides) may be assumed for calculating the effective distance, c .

$$c = b/2 \quad (8)$$

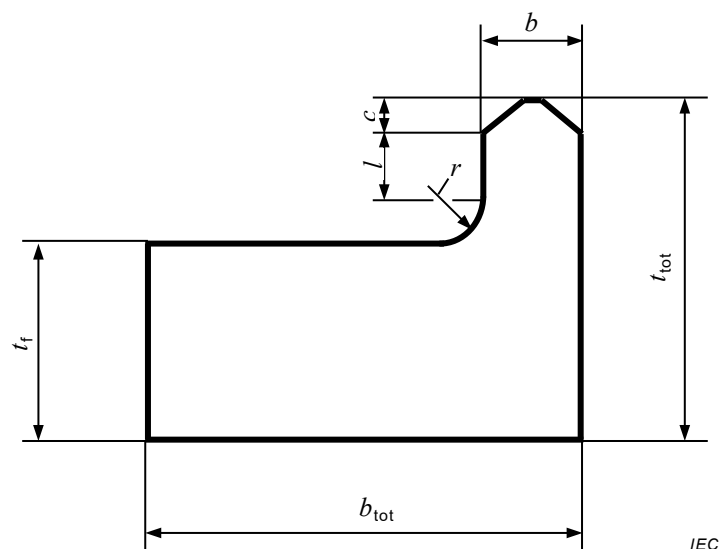
Conservatively, the whole flange thickness (t_{tot} according to Figure 1) can also be taken into account for the verification of sufficient material fracture toughness.

- b) For ring flanges without a weld neck, the decisive material thickness correlates to flange thickness (t_f according to Figure 1) of the ring flange.

⁵ Within the acknowledged state of the art, stress concentrations at openings, the tower top or other structural details do not need to be considered.

⁶ For definition of the climate conditions, please refer to IEC 61400-1.

⁷ The location of the weld toe may be coincident with the end of the weld preparation

**Key**

- t_f flange thickness
- t_{tot} total flange thickness including weld neck
- b_{tot} flange width
- r transition radius
- c flank height of the weld preparation
- b weld neck thickness (normally equal to the thickness of the connected tower shell)
- l distance from transition radius to weld toe ($= t_{tot} - t_f - r - c$)

NOTE In Figure 1, the length l is defined as the distance from the end of the radius to the beginning of the bevel edge. In regards to the formula, it might be necessary to take a shorter distance l into account as the position of the weld toe may not be identical to the end of the bevel edge but nearer to the radius due to an overlapping of the weld seam.

Figure 1 – Flange notations as an example of an L-flange

6.3.2.4 Through-thickness properties

The requirement of a necessary through-thickness property for the material arises from welding, when shrinkage of welds is restrained locally or globally in through-thickness direction, and needs compensation by local plastic through thickness strains. Therefore, for plates that are loaded perpendicular to the rolling direction, required properties according to a selected recognized standard shall be verified.

For flanges with weld necks produced from plates (plate thickness is the same as flange height t_{tot} according to Figure 1), the neck height may only be added to favourably improve the results if the distance from the weld transition to the end of the fillet radius ($l + r$ according to Figure 1) corresponds to at least the half of the neck thickness (b according to Figure 1):

$$l + r \geq b/2$$

For flanges produced from plates that are loaded longitudinally to the rolling direction (plate thickness is the same as the flange width b_{tot} according to Figure 1), and for flanges that are produced from a seamlessly rolled ring or from a bar rolled on all sides, through-thickness properties need not be verified.

In Annex D, a verification method of the through-thickness property is provided.

6.3.2.5 Weldability

The chemical composition of the steel shall be suitable for welding according to the execution standards on which the design is based.

6.3.2.6 Freedom from lamination

Sufficient freedom from lamination for plates loaded perpendicular to the rolling direction shall be verified (e.g. by ultrasonic testing). Acceptance criteria should be derived from referenced design and manufacturing standards.

6.3.3 Bolts and anchors

The material for bolts and anchors shall comply with the requirements stated in ISO 898-1 or equivalent. Bolts bigger than those covered by ISO 898 (M39) shall be derived rationally from those covered in the reference standard. For the bolts in the ring flange connections, only classes 8.8 and 10.9 are allowed.

Bolts of grade 12,9 may be used if compliant with the recognized design code. Such bolts shall not be hot dip galvanized. An alternative coating system as well as the mechanical properties in combination with the procedure for applying the pretension to the bolts shall be evaluated in this case.

The applicable temperature range of the bolt material shall correspond to the climate conditions of the wind turbine specification.

A comparison of local standards and usage of best practice can be found in Annex A.

6.4 Ultimate strength analysis for towers and openings

6.4.1 General

The verifications shall be carried out with the requirements for ultimate strength analysis according to 7.6.2 of IEC 61400-1:2019.

6.4.2 Partial safety factors

Partial safety factors shall be chosen based on the applied standard. Minimum safety factors for ULS are defined in IEC 61400-1 or IEC 61400-2.

Towers for wind turbines consist of generally non-damage tolerant structural components resulting in component class 2 according to IEC 61400-1.

6.4.3 Verification of ultimate strength

The verifications shall be carried out with the most unfavourable of all design load case combinations of gravitational loads, resultant bending moment, torsion moment and resultant shear force.

6.4.4 Tower assessment

In cylindrical and conical tubular steel towers, the stresses required for the ULS verification may be determined according to shell membrane theory. That means, for example, for the transfer of wind loads, the elementary pipe bending theory may be applied. Shell bending moments due to irregular wind pressure around the perimeter or restraint stresses due to edge disturbances at flanges or stiffeners may be neglected. At transitions with varying conicity, which are not supported by ring stiffeners or flanges, the local circumferential membrane stresses and shell bending moments due to the force deviation shall be considered.

For tower areas weakened by openings and other specific detail assessments, see 6.4.5.

6.4.5 Detail assessments

6.4.5.1 Openings

For openings in tubular steel towers, the stress concentration at the opening edge shall be considered.

6.4.5.2 Tower top flange

In the stress analysis of the tower top flange, stress concentrations shall be considered for the circumferential weld seam and for the transition from a weld neck to the flange, which may be a radius (see Figure 1).

Non-linear effects shall be considered in the load transfer from the yaw bearing, including any uneven load path from the nacelle to the top flange and the effect of the bearing contact angles and changes in contact points, which requires detailed information from the wind turbine specification. A flange design may be presented in the wind turbine specification for use on any tower design given that the flange design load envelope equals or exceeds the design load envelope for the tower.

6.4.5.3 Dealing with local plastifications

Without detailed material nonlinear analysis, local plastifications shall be limited to small regions for ULS.

For ULS, the total strain can be estimated with the Neuber's rule using von Mises stress hot spot stresses derived with a linear elastic material model. Residual stresses from welding and imperfections may be neglected. In case the Neuper's rule is used to estimate total strain in case of plastifications, the total strain shall be limited to 1 %.

$$\varepsilon_{\text{tot}} = (\sigma_{\text{hs}})^2 / (\sigma_{\text{y,d}} E) \leq 0,01 \quad (9)$$

where

ε_{tot} is the total strain;

σ_{hs} is the structural hot spot stress for a linear elastic material model;

$\sigma_{\text{y,d}}$ is the design yield stress of the material.

If fatigue assessment for welded details is performed with local approaches, then no local plastification (evaluated based on the extrapolated structural stress) shall occur for the maximum fatigue load level from the fatigue load cases determined in Table 2 of IEC 61400-1:2019, unless more detailed assessment of low-cycle-fatigue is performed. Here, residual stresses from welding and imperfections may be neglected.

6.5 Stability

6.5.1 General

Stability analysis of steel towers shall be performed considering the relevant failure modes for the specific type of structure, i.e. shell buckling for tubular towers or beam buckling of lattice towers.

Stability analysis for steel tube towers may be carried out based on the length of the section between flanges if the flanges are either L- or T-flanges. For other concepts, it shall be verified that the section connections will provide the necessary boundary conditions to only calculate section wise, otherwise the full tower height shall be used.

The verifications shall be carried out with the most unfavourable of all load case combinations of gravitational loads, resultant bending moment, torsion moment and resultant shear force.

6.5.2 Partial safety factor

Partial safety factors shall be chosen based on the applied verification method. Minimum safety factors for buckling verifications are defined in 5.4.⁸

6.5.3 Assessment

The buckling safety verification for the wall of a tubular steel tower shall be carried out with either of the following techniques:

- analytical verification according to recognized procedures;
- numerically assisted buckling safety verification.

Different degrees of consideration of non-linearities of material, geometry (MNA, GMNA) as well as imperfections (GMNIA) are described in literature, for details see for example references in Annex A.

If numerically assisted buckling safety verification is used, the main input parameters are the plastic reference resistance, elastic reference resistance and imperfection sensitivity. The plastic reference resistance may be calculated numerically, or (where available) by algebraic formulae. Reduction of resistance due to presence of imperfections shall consider the worst case scenario, either by direct inclusion of imperfections in the numerical model, or alternatively by slenderness dependent knock-down curves.

The imperfections considered in the design shall cover the potential geometrical, structural and material imperfections that are expected to occur during fabrication and installation.

6.5.4 Door frames/stiffeners

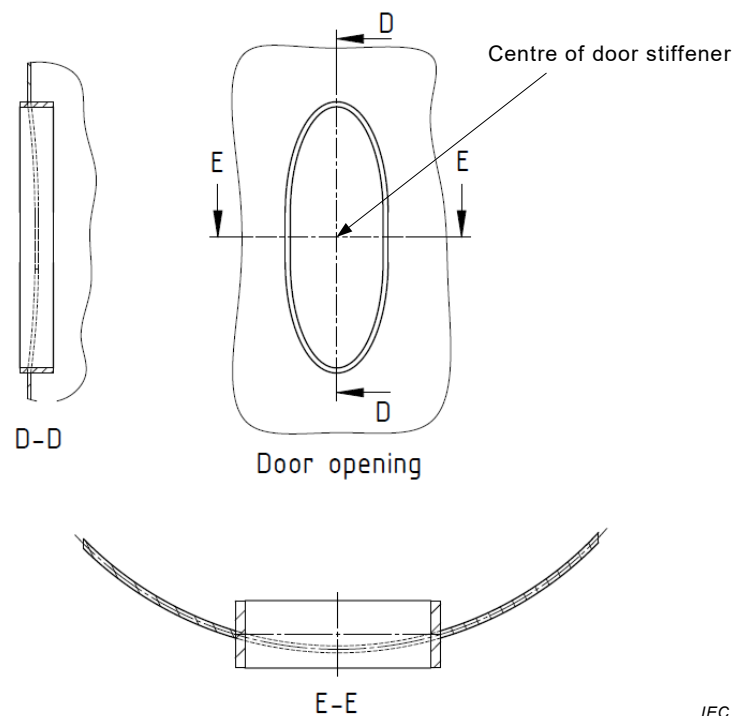
The buckling safety verification for openings with or without stiffeners in a tubular steel tower shall be carried out with one of the following techniques:

- a geometrically and materially nonlinear numerical buckling analysis with imperfections included (GMNIA);
- an analytical verification according to the methods suggested by Velickov (see Annex E);

For openings with stiffeners, the cross section of the stiffener should be centred to the tower shell in the 3 and 9 o'clock position of the stiffener (see Figure 2 and Figure E.2). For cylindrical shells, a misalignment from the centre is allowed in the range up to the centralization of the stiffener to the tower shell in the 12 o'clock position of the stiffener. For conical shells, additional considerations may be necessary.

- alternative methods from literature including modifications, simplifications or extensions of above-mentioned approaches may be considered if they lead to a comparable safety level.

⁸ The limitation of the diameter to thickness ratio D/t related to the method according to EN 1993-1-6:2007 which is stated in IEC 61400-1:2019, 7.6.2.4, can be omitted according to the background document [5].



IEC

Figure 2 – Door opening geometry

6.6 Fatigue limit state

6.6.1 General

The verifications shall be carried out with the fatigue load combinations according to 5.4. Every fatigue load bearing component shall be verified.

For tubular towers, the usage of the dominant (fore-aft or side-side) bending moment can be considered as sufficient for the fatigue verification but only at regions without discontinuities, for example not in the area of the door.

In case of guyed structures, additional fatigue load requirements (e.g. galloping) shall be considered.

6.6.2 Partial safety factor for materials

Towers for wind turbines contain generally non-damage-tolerant structural components resulting in component class 2 according to IEC 61400-1.

The use of component class 1 instead of component class 2 for specific details of a component needs to be investigated by additional verifications, for example fracture mechanics, to show that local failure does not lead to a failure chain with severe consequences.

Methodologies for determining the partial safety factors shall be chosen under consideration of Annex K of IEC 61400-1:2019 to reach the intended reliability level.

The frequency of inspection shall be evaluated and determined against the selecting details, materials and stress levels and the provisions made according to the damage tolerant method according to the Annex K of IEC 61400-1:2019.

6.6.3 Assessment

Verifications may be carried out by different approaches. Reference to details of applicable methodologies is made in Annex F. The method selected shall include effects of geometrical and structural imperfections from material production and execution (e.g. the effect of tolerances and residual stresses from welding).

A cut-off limit of fatigue strength in the S/N curves for any number of load cycles shall not be applied.

In general, applicable methods include, but are not limited to, $\Delta\sigma_R$ - N , notch strain and fracture mechanics methods, taking into account post fabrication treatment. A consideration of post fabrication treatment is only valid if it is included in the reference standard or verified by tests under consideration of Annex K of IEC 61400-1:2019.

When using the $\Delta\sigma_R$ - N approach and stress components, every relevant stress component ($\sigma_{\parallel}, \sigma_{\perp}, \tau$) shall be considered in the fatigue calculation. If using principal stresses, the applied S/N curve shall be consistent with the direction of principal stress. Additional guidance is provided in Annex F and related literature.

Signed equivalent (von Mises) stress shall not be used for the fatigue assessment.

Independent of the method or reference standard followed, the reliability level shall not be less than those intended by IEC 61400-1 or IEC 61400-2.

6.6.4 Details

Welded attachments or drilled holes shall be assessed for the effect of local stress concentrations. Further information can be found in Annex F.

The required structural stress for the fatigue calculation of the weld seam of the door stiffener shall be assessed by one of the following approaches:

- 1) modelling the door stiffener with weld seam;
- 2) modelling the door stiffener without weld seam and extrapolation to the hot-spot stress according to the International institute of welding (IIW) – see IIW-2259-15;
- 3) other comparable methods.

6.7 Ring flange connections

6.7.1 General

The regulations stated in 6.7 are valid for both L- and T-flange connections.

6.7.2 Design assumptions and requirements, execution of ring flanges

Ring flange connections shall be tightened in a controlled manner in several steps according to a reference standard and/or approved procedures.

The design pretension force of the bolt shall be limited to 70 % of the yield strength of one bolt. Otherwise, it shall be proven that the tensioning system is able to prestress the bolts in a way that the pretension force is reached but the bolt is not overstressed (i.e. prestressing above yield limit may be allowed under consideration of plastic deformation capability of the bolt).

All flange gaps k in the area of the tower wall result in increased fatigue loading of the bolts, particularly when they extend only over a part of the circumference. The damage influence grows with decreasing spanning length l_k over the circumference of the flange.

The tolerances shall be stated in the drawings or working documents.

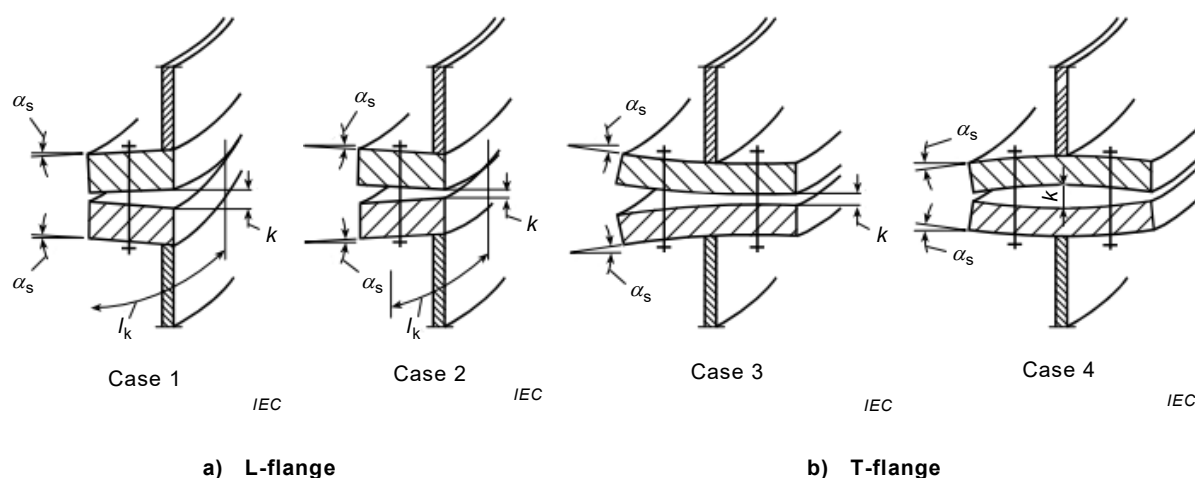


Figure 3 – Flange gaps k in the area of the tower wall

Following completion of the production of the individual tower sections, the flatness deviation per flange to the inside of the connecting surface (taper) of each flange (see Figure 3, cases 2 and 3) shall be checked and should not exceed the values in Table 1 noting that the region near to the tower wall is decisive. Outside taper is not allowed in general without further consideration.

Table 1 – Flange tolerances

Characteristic	Limiting value
Flatness deviation per flange ($k/2$ according to Figure 3) around the entire circumference	2,0 mm
Flatness deviation per flange ($k/2$ according to Figure 3) over a segment of 30°	max. 1,0 mm
Taper α_s to the inside of the connecting surface of each flange	$0,0^\circ$ to $0,7^\circ$
Outer flange surfaces limit α_s before using taper washers	2°

If larger flatness deviations are assumed in the design, the fatigue limit state analysis shall include the effect of these larger flatness deviations (e.g. possible opening of the flange for the calculation of axial and bending bolt forces). Additionally, the design pretension level assumed in the bolt calculation shall be reduced by the value of preloading required to close the flange gaps in the area of the tower wall.

It shall be guaranteed that sufficient local compressive preload of the flange contact areas is achieved from the preloading force of each individual bolt through accurate production of the flanges and their welded connections, including accurate preloading.

In case the contact surfaces of the flanges do not have a sufficient compressive stress at the tower outside after preloading the bolts, suitable measures shall be taken. Suitable measures may include reworking, shimming or filling out the damage-relevant gaps in the absence of applied preloading.

The shims or filler material shall have sufficient modulus of elasticity and compressive strength (yield point under compression) to replicate the effect of the parent flange material. The gap-filling should be executed such that contact is produced preferably prior to the preloading process, but at the latest after applying 10 % of the preload, either in the direct vicinity of each bolt or in the area between each individual bolt and the tower wall (including the area directly under the tower wall itself).

If, after the preloading, the remaining inclination α_s of the outer flange surfaces (see Figure 3) exceeds the limiting value of 2° , suitable taper washers with sufficient hardness shall be used instead of the normal washers.

The welded connection of the flange to the tower wall may be assumed to be unaffected by edge distortion of the tower shell, if

- radius between flange surface and welding neck is at least 10 mm,
- the distance between the circumferential weld seam toe and the finished product (upper side of the flange surface) shall satisfy requirements in 6.3.2.3,
- the weld toe to flange surface distance requirement apply to production and repair welding, and
- inside taper per flange is limited to $0,7^\circ$.

In case these requirements above are met, for L-flanges only the flange bolts and the circumferential weld seams shall be verified for fatigue limit state.

In case these requirements above are not met, detailed analysis for ultimate and fatigue limit state need to be performed for the flange, the welding between flange and tower shell and the tower shell in the proximity of the welding. In addition, the detail category for the weld shall be chosen as for a flange connection without a welding neck.

6.7.3 Ultimate limit state analysis of flange and bolted connection

In the ultimate limit state analysis of the flange connections, the preloading force of the bolts need not be considered, i.e. the ultimate limit state analysis may be performed as for a non-preloaded bolted connection.

A simplified calculation method according to Petersen/Seidel or Tobinaga (see Annex G) may be used if it covers flange gaps of the magnitude tolerated in execution of the work.

The calculation method needs to consider at least the following three failure modes:

- 1) failure of bolt due to rupture;
- 2) failure of bolt due to rupture combined with yield hinge in tower shell and/or flange material;
- 3) yield hinges in tower shell and/or in flange material.

The influence of the axial load in the tower shell shall be considered when calculating the yield bending strength of the tower shell and/or the flange material.

Favourable loads (reducing the load on the bolted connection, e.g. weight), if included, shall use the partial safety factor for loads for favourable loads.

6.7.4 Fatigue limit state analysis of bolted connection

In the fatigue safety analysis of the flange connection, the fatigue loading of the bolts may be determined with consideration of the compressive preloading of the flanges, provided the following conditions are met.

For the fatigue calculation, the pretension force of the bolts may be applied with a maximum of 90 % of the design pretension force $F_{p,C}$, providing that the bolts are retightened after initial relaxation of the bolted connection. The retightening shall be done after minimum 240 power production hours but, in any case, not later than six months after commissioning. Otherwise, 70 % of the design pretension force $F_{p,C}$ shall be used for the fatigue calculation.

The fatigue safety verification shall be based on the non-linear bolt force function $F_S = f(Z)$ from which the fatigue range of the bolt force F_S can be read off for a given range of the tower shell force Z (see Figure 4).

The non-linear bolt force function may be derived from a simplified calculation method according to Schmidt/Neuper (see Annex G) if it covers flange gaps of the magnitude tolerated in execution of the work.

In the determination of the bolt force function with the aid of a more sophisticated calculation method (for example FEM using contact or spring elements), the flange gaps tolerated in the production shall be considered as imperfections. If the bolt force function is determined without imperfections with the aid of such an ideal calculation method, the flange gaps tolerated in the execution of the work may be taken into account by a suitable increase in the initial gradient of the bolt force function based on studies into the effect of imperfections which is qualitatively shown in Figure 4.

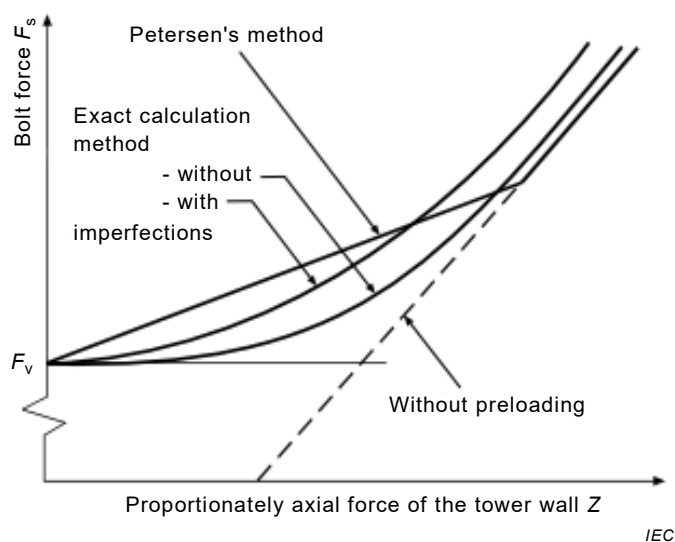


Figure 4 – Bolt force as a function of wall force

When using calculation methods which do not consider the influence of the bending moment on the bolt (e.g. simplified calculation method from Schmidt/Neuper, see Annex G), the fatigue safety of the bolt shall be determined using detail category 36*, as presented in Figure 5.

For bolts larger than M30, a reduction of the S/N curve by the factor k_s with $k_s = (30 \text{ mm}/d)^{0.25}$ shall be taken into account, where d is the nominal diameter of the bolt.

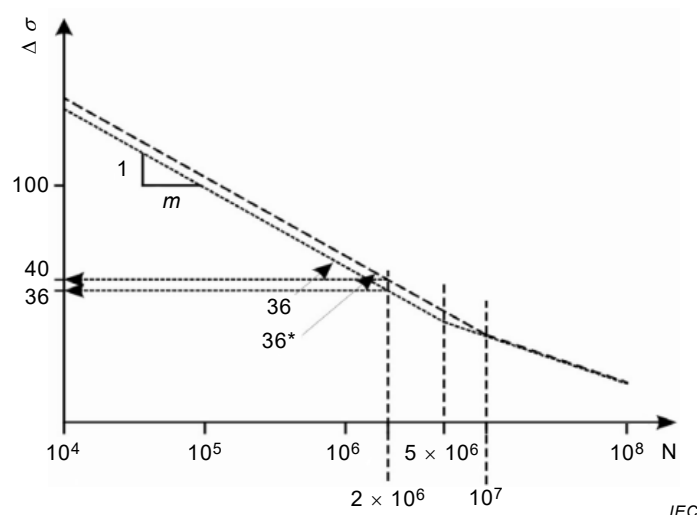


Figure 5 – S-N curve for detail category 36

6.8 Bolted connections resisting shear through friction

6.8.1 General requirements

Bolted connections in connections and splices of building elements of the main structure shall be designed for ultimate limit state (ultimate slip resistance) and fatigue limit state (slip resistance for long term fatigue load conditions).

For the ultimate limit state, it shall be verified that for each bolt in the connection the maximum force acting at the shear joint does not exceed the limiting slip force. In addition to calculating the ultimate limit state (ultimate slip resistance), the following verifications shall be performed:

- the maximum design force acting on a bolt in a shear joint shall not exceed the limiting hole bearing design resistance;
- the maximum design force acting on a bolt in a shear joint shall not exceed the limiting elastic design resistance of the net cross-section at bolt holes for the ultimate limit state;
- the maximum design force acting on a bolt in a shear joint shall not exceed the shear design resistance per shear plane;
- the maximum force acting on a bolt in a combined tension and shear joint does not exceed the limiting slipping design resistance for the ultimate limit state;
- the plate materials that are connected by the friction connection shall have sufficient design resistance for fatigue loads, especially considering local stress increase due to holes (including considerations due to manufacturing of holes) and any eccentricities for ultimate and fatigue limit state.

Fatigue verifications of the plates and bolts, both shearing and bearing, shall use S-N curves from a reference standard or relevant fatigue tests of the connections, ensuring that maximum and minimum distances between bolts are not exceeded.

The actual bolt preload shall exceed the preload assumptions in design throughout the design lifetime. The connection may be designed on the assumption of prescribed maintenance of the bolts or without, and the preload assumption shall account for this. The maintenance shall be clearly indicated in the delivery documentation. Minimum requirements for maintenance for design without test are given in 6.8.3.

In connections with risk of contact corrosion between bolt and hole, special corrosion protection measures shall be made in particular when using hot-dipped galvanized building elements.

6.8.2 Test-assisted design

A maintenance free connection can be achieved by a test assisted design as follows:

- for the fatigue limit state, it shall be verified that the friction utilization for maximum load in the fatigue load cases does not exceed the friction utilization for maximum applied loads in testing;
- the design shear resistance force of the friction connection shall be calculated according to the following formula:

$$F_{s,Rd} = \frac{n \cdot \mu}{\gamma_M} F_{p,C'} \quad (10)$$

where

$$F_{p,C'} = L_f F_{p,C} \quad (11)$$

where

$F_{p,C'}$ is the preload bolt force used in the design calculations;

$F_{p,C}$ is the preload bolt force given in the relevant design code or technical approval;

L_f is the factor for loss of preload, which is not considered in the applied design code;

μ is the friction coefficient;

n is the number of force transmitting interfaces in the connection;

γ_M is the partial safety factor for materials.

The preloading force $F_{p,C'}$ shall account for the loss of preload in the bolt connection by the means of the factor L_f or by introducing a measurement campaign and a maintenance program based on the results of the measurement campaign.

For determining L_f , two approaches are possible.

- 1) Testing of representative samples of the friction connection to determine a maintenance free connection.

The test specimens shall resemble the joint used in the actual tower design. This means at least the following parameters shall be considered:

- clamp length;
- surface preparation, condition and layer thickness;
- bolt set type, size, length, grade, quality, contact surface of bolt, nut and washers;
- pre-tension force;
- quality of the base material;
- temperature ranges;
- geometry of the connection (e.g. eccentricities, number of bolts in load direction, distance of bolts, thickness of clamped parts);
- size and manufacturing of holes;
- expected maximum load levels for ULS and FLS;
- installation method.

- 2) Assuming a value for L_f in the design phase and validating by an appropriate measurement campaign on the friction connection that the assumption for L_f is conservative.

The measurement campaign shall start directly after the pretension process. The measurement equipment shall not change the relevant parameters compared to the used bolted connections in the tower.

If necessary, a maintenance program shall be defined on the basis on the results of the measurement campaign.

Tests, measurements and interpretation of results shall be conducted according to a recognized standard and be evaluated according to IEC 61400-1:2019, Annex K.

For a maintenance free test based design, the friction coefficient μ shall be tested in combination with the determination of the L_f factor.

For a design with a measurement campaign and a maintenance program, the friction coefficient shall be tested according to a recognized standard while taking into account the geometry of the connection, the detailed surface conditions at the real tower and the level of pretension force.

This shall be in compliance with recognized design standards and IEC 61400-1. The following value of γ_M is recommended.

$$\gamma_M = 1,25 \quad (12)$$

6.8.3 Design without test

For a non-test assisted design, the following applies:

- the ultimate slip resistance of the connection shall be designed according to the reference standard;
- the pre-stressing force shall be checked by inspection and, if necessary, ensured by retightening, within the first six months of installation, albeit not immediately after commissioning.

7 Concrete towers and foundations

7.1 General

Concrete design principles are mostly well defined and documented in the literature, including reference standards. Clause 7 provides additional or more detailed rules and guidance that are specific to onshore wind turbine towers and foundations and complement the more general requirements of existing standards.

Clause 7 provides requirements in the form of general statements and analytical methods. Guidance on acceptable calculation methods which are considered to comply with the requirements are presented in Annex H to Annex K.

7.2 Basis of design

7.2.1 Reference standard for concrete design

The reference standard shall comply with the basic principles of IEC 61400-1 or IEC 61400-2 and should be in compliance with ISO 19338. The reference standard shall give the principles required for the design against fatigue for all possible failure modes.

7.2.2 Partial safety factors

The partial safety factors given in IEC 61400-1, IEC 61400-2 and in 5.4 as well as the following factors shall be applied for design of concrete structures as the minimum requirements. The safety factors from the selected reference standard should be considered, and they shall fulfil at least the target reliability of IEC standards.

For the general verifications at the SLS, the partial safety factor for the materials can in general be taken as $\gamma_M = 1,0$.

In the case of prestressed concrete in ULS, the prestress losses and possible variations in the prestress shall be considered. The variations shall be considered by applying the least favourable of the factors $\gamma_{f,prestress,favourable}$ and $\gamma_{f,prestress,unfavourable}$.

In the case of prestressed concrete in FLS and the SLS the variations shall be considered by applying the least favourable of the factors r_{sup} and r_{inf} . For FLS and SLS, the recommended factors to be used for post-tensioning with bonded tendons are $r_{sup} = 1,10$ and $r_{inf} = 0,90$, but the recommended factors for post-tensioning with unbonded tendons and for pre-tensioning are $r_{sup} = 1,05$ and $r_{inf} = 0,95$.

When measures are taken to reduce the uncertainty on the prestress load, the factors r_{sup} and r_{inf} can be taken as $r_{sup} = r_{inf} = 1,0$. While a high accuracy of the tensioning devices may improve the accuracy of the initial level of prestress, the remaining uncertainty with respect to the losses of prestress shall also be considered if r_{sup} and r_{inf} shall be modified.

Where high resistance of a member is unfavourable, an upper value of the characteristic resistance shall be used in order to give a low probability of failure of the adjoining structure.

The upper value shall be chosen with the same level of probability of exceedance as the probability of lower values being underscored. In such cases, the partial safety factor for material shall be 1,0 for calculating the resistance that is used when actions are applied on adjoining members.

7.2.3 Basic variables

7.2.3.1 Thermal actions

7.2.3.1.1 General requirements

Concrete towers are subject to the induced stress effect of temperature variations with respect to the temperature at the time of erection as well as to temperature gradients within the tower cross-section as shown in Figure 6.

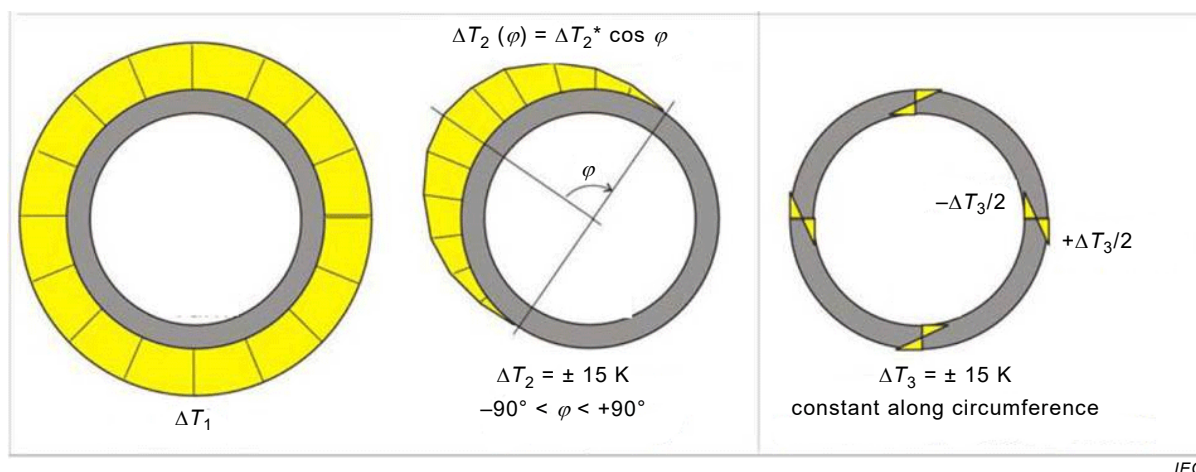


Figure 6 – Thermal effects around tower cross-section

These characteristic temperature actions shall be considered in the design as explained in the following.

- a) ΔT_1 is a uniform temperature difference with respect to the temperature at the time of erection (constant temperature along circumference and across wall thickness). ΔT_1 shall be taken as the least favourable of

- $T_{\text{extreme,max.}} - T_{\text{erection,min.}}$
- $T_{\text{extreme,min.}} - T_{\text{erection,max.}}$

where

$T_{\text{extreme,min.}}$ and $T_{\text{extreme,max.}}$ are the extreme temperature range as defined in IEC 61400-1:2019, 6.4.2.1.

$T_{\text{erection,min.}}$ and $T_{\text{erection,max.}}$ are the allowable temperature range to be defined for the erection of the tower.

This temperature action may generally be neglected provided the tower does not incorporate or is not restrained by materials of differing thermal expansion coefficients. On the basis of ΔT_1 , a reasonable assumption shall be made for the temperature difference between the tower and foundation which may cause tension due to restraint.

- b) $\Delta T_2 = \pm 15 \text{ K}$ is a cosine shaped temperature difference caused by solar irradiation on one side of the tower (constant temperature across wall thickness and cosine shaped along the circumference).

Additional second order loads ($P - \Delta$ effect) resulting from the deflection due to ΔT_2 may be neglected if a tower misalignment of at least 5 mm/m lateral deflection per metre height is applied as the geometric tolerance in the structural design of the tower (differential settlement of the foundation shall be considered separately as per 8.5.3.3).

Furthermore, it may be assumed that restraint caused by ΔT_2 is covered by ΔT_3 if the bending moment in the wall caused by ΔT_3 is calculated based on linear analysis (uncracked cross section, linear stress-strain relationship and mean value of the modulus of elasticity).

- c) $\Delta T_3 = \pm 15 \text{ K}$ is a temperature difference between inside and outside wall surfaces (constant temperature along the circumference, varying linearly across the wall thickness). The value of $\pm 15 \text{ K}$ is intended to cover environmental temperature changes; ΔT_3 may need to be increased if significant other heat sources are to be placed in or near the wind turbine tower.

Effects caused by the release of hydration heat in in-situ concrete towers are not included in $\Delta T_1 - \Delta T_3$ and shall be taken into account separately.

The above mentioned temperature actions generally cover the normal environmental conditions defined in IEC 61400-1 or IEC 61400-2. Special regional climatic conditions outside the normal environmental conditions shall be taken in to account when applying temperature actions.

The different temperature components shall be superimposed so that the characteristic temperature action ΔT_k shall be taken as the least favourable of:

$$\begin{aligned} &\Delta T_1 + \Delta T_2; \\ &\Delta T_3; \\ &(\Delta T_1 + \Delta T_2) + 0,75\Delta T_3; \\ &0,35(\Delta T_1 + \Delta T_2) + \Delta T_3. \end{aligned}$$

7.2.3.1.2 Ultimate limit state requirements

In the ULS, temperature actions shall be superimposed with the ultimate design loads F_d , of the load cases N according to Table 2 of IEC 61400-1:2019. The partial safety factor for temperature actions shall be taken as $\gamma_{f,Temp} = 1,35$ so that $\Delta T_d = \gamma_{f,Temp} \Delta T_k$. The least favourable of the following combinations shall be considered in the design.

$$\begin{aligned} &F_d + 0,6\Delta T_d \\ &0,6F_d + \Delta T_d \end{aligned}$$

The psfm, $\gamma_{f,Temp}$ may be reduced to 1,0 if the effects are determined based on linear elastic analysis.

7.2.3.1.3 Serviceability limit state requirements

In the SLS, temperature actions shall be superimposed with F_{SLS} , the load value of the applicable SLS load cases as defined in Annex H.

For load cases S2 and S3 (see 5.4.3), it suffices to consider the combination of $F_{SLS} + 0,6\Delta T_k$ in the design, while for load case S1 also the combination of $0,6F_{SLS} + \Delta T_k$ shall be considered.

7.2.3.2 Prestress

The prestress of concrete structures considered in 7.2.3.2 is applied by tendons made of high-strength steel (wires, strands or bars) or high-strength bolts.

Tendons and bolts may be embedded in the concrete. They may be pre-tensioned and bonded or post-tensioned and bonded or unbonded.

Tendons may also be external to the structure with points of contact occurring at deviators and anchorages.

7.3 Materials

The concrete structure materials shall be in accordance with ISO 22965-1 and ISO 22965-2 for concrete and ISO 6934 (all parts) and ISO 6935 (all parts) for reinforcement unless requirements in the selected reference standards are more critical. In either case, the requirements of this document will supersede.

7.4 Durability

7.4.1 Durability requirements

In order to achieve the required design lifetime of the structure, adequate measures shall be taken to protect each structural element against the relevant environmental actions. The requirements defined in the reference standard concerning durability shall be fulfilled.

The requirements for durability shall be included when considering the following:

- structural conception;
- material selection;
- construction details;
- execution;
- quality control;
- inspection;
- verifications;
- special measures (e.g. use of stainless steel, coatings, and cathodic protection).

7.4.2 Exposure classes

If not specified in the reference standard, the exposure classes as defined in ISO 22965-1 may be used.

7.4.3 Concrete cover

The concrete cover requirements shall include the exposure class, the quality of the concrete and the design lifetime of the structure. Other parameters may be of importance in specific cases.

The nominal concrete cover shall be specified on the construction drawings or specifications.

7.5 Structural analysis

7.5.1 Finite element analysis

A finite element model shall be suitable for the purposes of the investigation. The following items shall be considered if they are expected to affect the relevant results of the analysis to a significant extent.

- The structure shall be in equilibrium with the applied loading and boundary conditions.
- The order and type of each element shall be sufficient to capture the expected patterns of deformations.
- A mesh sensitivity study is usually required to assess if the model is adequately predicting the deformation of the structure, and thereby the load-distribution within the structure.
- Second order deformations and three-dimensional effect shall be considered.
- The compatibility of deformations shall be considered, including strain offsets between materials types, and the decomposition of strain. Load-path dependency may also need to be considered.
- The inelastic response of materials, including cracking of concrete and yield of reinforcements, shall be considered when necessary. Since the principle of superposition does not apply when materials exhibit an inelastic response, a separate analysis is required for each factored load combination.

- The degradation of resistance mechanisms, such as bond, slip resistance along cracks, spalling, and other factors that affect the stiffness and strength of structural components shall be considered.
- Time-dependent effects, such as creep of concrete and relaxation of prestressing, shall be considered when the effects are significant.

More detailed guidance on finite element analysis can be found in Annex I.

7.5.2 Foundation slabs

Solid foundation slabs subjected to two-dimensional bending shall be analysed with one or more of the following methods.

- Analysis based on linear elasticity
This method is based on the theory of elasticity adopting a linear moment-curvature relationship. It may be used for serviceability, fatigue, and ultimate limit states.
- Analysis according to linear elasticity with limited redistribution of bending moments
Linear analysis with limited redistribution of the moments may be applied to the analysis of foundation slabs for the verification at the ultimate limit state only. The reduction of the moment shall be performed assuming the average value for an appropriate width, providing that the average moments for the same width at the corresponding section are adjusted to satisfy equilibrium. The rotation capacity needs to be checked.
- Plastic analysis
Statically admissible plastic moment fields which satisfy the equilibrium condition may be found directly (e.g. by applying the strip method) or by starting from a linear analysis. This method may be used for the ultimate limit state only.
- Non-linear analysis
The nonlinear analysis covers ULS, FLS, and SLS, provided that equilibrium and compatibility are satisfied and adequate nonlinear behaviour for materials is assumed.

7.5.3 Regions with discontinuity in geometry or loads

Discontinuity regions shall be assessed with force models which have been validated with test results and/or theoretical considerations. These models may be strut-and-tie model systems, stress fields, or similar, that satisfy equilibrium conditions.

If there is no recognized calculation model for the member in question, the geometry of the model may be determined from the stress condition for a homogeneous un-cracked structure in accordance with the theory of elasticity.

Methods for determining the strength of a load-resisting truss (consisting of struts, ties, and joints (or nodes)) shall follow the reference standards including maximum concrete stress capacities for struts and nodal regions, and rules for determining the dimensions of struts, ties, and nodal regions.

The shape of the load resisting truss is not prescribed by this document.

For the design of discontinuity regions in which a complex and likely indeterminate strut-and-tie model is used, extra caution shall be taken to ensure that the members can obtain and maintain their capacity until the formation of a plastic truss mechanism.

More guidance regarding strut-and-tie modelling can be found in Annex K. See also the normative Annex J regarding connections between concrete foundations and steel towers.

7.5.4 Cast in anchor bolt arrangements

Bolts and anchor flanges cast into the concrete for anchorage of a steel tower or similar shall be designed according to Clause 6 where appropriate. The impact on the concrete shall be considered similar to impact from prestressing tendons.

7.6 Concrete to concrete joints

Concrete to concrete joints are considered either interfaces between reinforced concrete cast in situ with significant delay or joints between prefabricated pieces (typically secured by means of post-tensioned reinforcement) with or without grout.

Concrete to concrete joints shall be considered at the ULS, FLS and SLS verifications, for all concrete support structures.

Shear strength shall be verified in the joint, which is largely affected by the concrete coefficient of friction across the joint, reinforcing across a cast in situ joint and level of post-tensioning force across precast element joints. The coefficient of friction is a function of the surface roughness, which is commonly described by the surface roughness parameter, R_a . The coefficient of friction shall be verified according to the reference standard.

Due to the variable cyclic loading of the wind turbine, the risk of a reduction of the joint capacity in shear shall be considered.

The compressive load capacity of the grout and the adjacent members shall be verified considering the stiffness of the grout and the geometry of the joint. Where imperfections such as mismatch in segmental construction can occur, these shall be considered in design. The induced transverse stresses shall be taken by reinforcement where necessary.

Non-linear analysis (load dependent stiffness) is generally required with reduced stiffness consideration of the joint (at ULS, it shall be assumed that opening or cracking can occur, unless the design is verified to avoid it).

Special attention shall be applied in those zones subjected to stress concentration effects, such as pre-stressing anchorages, non-uniform contact areas and geometrical and/or material discontinuities.

Tension capacity through the joint can be provided by post-tensioned bars, post-tensioned strand or passive reinforcement.

Tolerances at joints shall be defined in accordance with the execution requirements as stated in 7.10.

Tolerances at joints shall be such as to allow parts to fit together as intended.

7.7 Ultimate limit state

7.7.1 General

In general, the selected reference standard shall be followed for ULS design.

7.7.2 Shear and punching shear

The minimum amount of shear reinforcement required in the reference standard should be used even when it is not required to satisfy ULS requirements. This reinforcement will mitigate size effects in large members, and also help address the detrimental effects of cyclic shear actions. The spacing of this reinforcement should not exceed the limits of the reference standard.