Safety Factors – IEC 61400-1 ed. 4

background document

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CONTENT

1 INTRODUCTION

The aim of this document is to describe the basis for the partial safety factors recommended in IEC 61400-1 ed. 4.

The scope is to

- give the basis for selecting materials partial safety factors in 'recognized design codes' – taking into account inspections (e.g. in relation to fatigue)
- give the basis for design assisted by testing – determination of characteristic values for material parameters and load bearing capacities on the basis of test results
- give the basis for modifying load partial safety factors if compared to the 'normal' situation better / additional information (less uncertainty) is available for estimating loads; e.g. modification of safety factors depending on specific site conditions.

Section 2 briefly describes the theoretical basis for calibration of partial safety factors using reliability based methods. The required reliability level is discussed in section 3. In section 4 three basic models for calculating the design value of the load bearing capacity is presented. Next, reliability-based calibration of material partial safety factors is described in the following cases: DLC 1.1 and 6.1 with extreme load; fatigue of welded steel details; DLC 2.1 and 2.2 with extreme load and faults; component / consequence class partial safety factor γ_c. Finally, also the load partial safety factor in typhoon conditions is considered. Section 5 how the uncertainty level influences the partial safety factors.

Finally annexes are presented on Reliability and partial safety factors for tower buckling; Reliability of concrete structures for wind turbines; Safety factors for fatigue of welded details in steel structures for wind turbines; and an overview of the main changes in material partial factors in the CD IEC 61400-1: 2014 ed. 4 compared to IEC 61400-1: 2005 ed. 3.

2 THEORETICAL BASIS FOR RELIABILITY-BASED CALIBRATION OF PARTIAL SAFETY FACTORS

Calibration of partial safety factors for codes and standards was previously performed by judgment based on experience. But during the last 20-30 years reliability-based techniques have been used in the process of calibration as a rational tool to account for the uncertainties related to the strengths, loads and models, see e.g. Madsen et al. [1]. Examples are building and bridge codes in Europe, Canada and USA. Further, reliability analyses have been performed as basis for the partial safety factors in the IEC 61400-1 and -3 standards, see Tarp-Johansen et al. [2], [3] and Tarp-Johansen [4].

In ISO 2394 [5] and the Eurocodes, EN 1990 [6] and Sedlacek et al. [8] the Design Value Format method has been used as basis for the assessment of the recommended partial safety factors. In section 2.1 this approach is described and for illustration applied for some wind components. In section 2.2 a general procedure for calibration of partial safety factors based on the JCSS approach is described.

The wind turbine standard IEC 61400-1 [9] contains detailed requirements related to the calculation of design loads incl. partial safety factors for loads. However, only general requirements with respect to the materials and how to calculate the design load bearing capacities, incl. material partial safety factors. In practice the IEC-standard is used together with a 'recognized' standard for the materials – without consistent requirements to design equations and material partial safety factors. This can result in inconsistent reliability levels – which for some components can lead to a too low reliability level and for others a too high reliability level.

2.1 Design value approach in ISO 2394 and EN 1990

In EN 1990 [6] and ISO 2394 [5] a simplified procedure - 'Design value approach' - for calculation of partial safety factors is described. The basic principle is that design values x_d are calculated from

$$F_X\left(x_d\right)=\Phi\left(\alpha\beta_t\right) \tag{2.1}$$

where F_X is the distribution function for the stochastic variable with coefficient of variation V, β_t is the target reliability level (see section 4), $\Phi(\ )$ is the standard Normal distribution function and

- α = 0.7 for a dominating load variable
- α = 0.28 for a non-dominating load variable
- α = -0.8 for a strength variable

The design value format could be used to estimate how much the partial safety factors in the material Eurocodes could be changed when applied to wind turbines. As described in Sedlacek et al. [8] and EN 1990, annex C [6] it is assumed that the design lifetime for buildings is 50 years and the lifetime target reliability index for ultimate limit states is 3.8 corresponding to an annual reliability index equal to 4.7. If it is further assumed that material strengths are Lognormal distributed and that failure events in different years are statistically independent, see EN 1990 [6] and JCSS [11] then the material partial safety factors can be modified by the factor

$$\eta=\frac{\exp\left(-0.8V\beta_{t,EN1990}\right)}{\exp\left(-0.8V\beta_{t,WT}\right)} \tag{2.2}$$

where

$\beta_{t,EN1990}$ lifetime (50 years) target reliability level in the materials Eurocodes used for calibration of recommended partial safety factors = 3.8 (corresponding annual reliability index = 4.7 assuming independence from year to year)

$\beta_{t,WT}$ lifetime (20 years) target reliability level for wind turbines. If e.g. $\beta_{t,WT}$ = 2.6 then the equivalent annual target reliability index, $\beta_{t,WT,1}$ = 3.5.

Table 1. Partial safety factor reduction factor.

V	0.05	0.10	0.15	0.20
η for $\beta_{t,WT,1}$=3.5	0.95	0.91	0.87	0.83
η for $\beta_{t,WT,1}$=3.3	0.95	0.89	0.85	0.80
η for $\beta_{t,WT,1}$=3.1	0.94	0.88	0.83	0.77

Table 1 shows the reduction factor η for different coefficients of variation and for $\beta_{t,WT,1}$ = 3.1, 3.3 and 3.5. The yield strength of steel and reinforcement strength typically have a coefficient of variation V equal to 0.05 implying a reduction factor equal to 0.94 - 0.95. The coefficient of variation for the compressive strength of concrete is typically 0.15 implying a reduction factor equal to 0.83 - 0.87. Application of these reduction factors to the recommended partial safety factors in Eurocode 3 for steel structures and Eurocode 2 for concrete structures indicates the level of reduction that can be accepted in order to secure the level of reliability implicitly assumed in the IEC 61400-1 standard. It is noted that these reduction factors implicitly accounts for the bias (and hidden safety) in the design equations specified in the considered Eurocodes.

Application of the design value format is an approximate technique to estimate partial safety factors. A more accurate technique is to use the methods described in the following section.

2.2 JCSS approach for code calibration

The general procedure for calibration of partial safety factors described in the following is based on the procedure recommended by the Joint Committee on Structural Safety (JCSS), see Faber & Sørensen [12]. Code calibration can be performed by judgment, fitting, optimization or a combination of these, see e.g. Madsen et al. [1]. Calibration by judgment has been the main method until 20-30 years ago. Fitting of partial safety factors in codes is used when a new code format is introduced and the parameters in this code are determined e.g. such that the same level of safety is obtained as in the old code or calibrated to a target reliability level. In practical code optimization the following steps are generally performed:

1. *Definition of the scope of the code*
2. *Definition of the code objective*
3. *Definition of code format*
4. *Identification of typical failure modes and of stochastic model*
5. *Definition of a measure of closeness*
6. *Determination of the optimal partial safety factors for the chosen code format*
7. *Verification*

Ad 1. The class of structures and the type of relevant failure modes to be considered are defined.

Ad 2. The code objective may be defined using target reliability indices or target probability of failures depending on the use and characteristics of the considered class of structure. These can be determined by referencing to the reliability indices implicitly or explicitly assumed in existing codes or based on other criteria, e.g. based on the LQI (Life Quality Index) concept if life safety is important, see JCSS [13]. Recommendations on target reliabilities can also be found in e.g. EN 1990

[6] and ISO 2394 [5]. It is important to note that the target reliabilities are linked closely to the stochastic models used for the uncertain variables and the applied limit states.

Ad 3. The code format includes: how many partial safety factors should be used, should load partial safety factors be material independent, should material partial safety factors be load type independent how to use the partial safety factors in the design equations rules for load combinations. In general for practical use the partial safety factors should be as few and general as possible. On the other hand a large number of partial safety factors are needed to obtain a consistent reliability level and also to obtain economical and safe structures for a wide range of different types of structures.

Ad 4. Within the class of structures considered typical failure modes are identified. Limit state equations and design equations are formulated and stochastic models for the parameters in the limit state equations are selected. Also the frequency at which each type of safety check is performed is determined. The probabilistic model for the basic random variables should be selected very carefully. Guidelines for the selection can be found in JCSS Probabilistic Model Code [14]. In general the following main recommendations can be made:

Strength or resistance variables are often modeled by Lognormal distributions. This avoids the possibility of negative realizations. In some cases it can be relevant also to consider Weibull distributions for material properties e.g composite material. This is especially the case if the strength is governed by brittleness, size effects and material defects. The coefficient of variation varies with the material type considered. Typical values are 5% for strength of steel and reinforcement, 15% for the concrete compression strength and (~ 5-20%) for strength of composite materials. The characteristic value is generally chosen as the 5% quantile.

Variable loads (wind and wave loads) can be modeled in different ways. The simplest model is to use a stochastic variable modeling the largest load within the reference period (often one year). This variable is typically modeled by an extreme distribution such as the Gumbel distribution or the Weibull distribution. The coefficient of variation is typically in the range 15-30% and the characteristic value is chosen as the 98% quantile in the distribution function for the annual maximum load.

Permanent loads are typically modeled by a Normal distribution since it can be considered as obtained from many different contributions. The coefficient of variation is typically 5-10% and the characteristic value is chosen as the 50% quantile. However, for wind turbines often more accurate estimates can be made.

Model uncertainties are in many cases modeled by a Lognormal distributions if they are introduced as multiplicative stochastic variables and by Normal distributions if they are modeled by additive stochastic variables. Typical values for the coefficient of variation for the model uncertainty are 3-25% but should be chosen very carefully. The characteristic value is generally chosen as the 50% quantile.

Ad 5. The partial safety factors $\boldsymbol{\gamma} = (\gamma_1, \gamma_2, ..., \gamma_n)$ representing all load and material safety factors are calibrated such that the reliability indices corresponding to L different representative failure modes defined by vectors $\mathbf{p}_j$ are as close as possible to a target reliability level represented by a target reliability index β_t with a given reference time, typically one year. Equivalently a target probability of failure could be used.

The optimal partial safety factors are obtained from:

$$\min_{\boldsymbol{\gamma}} \; W(\boldsymbol{\gamma}) = \sum_{j=1}^{L} w_j \left(\beta_j(\boldsymbol{\gamma}) - \beta_t\right)^2 \tag{2.3}$$

where $w_j, j = 1,2,...,L$ are weighting factors indicating the relative frequency / importance of the different design situations. Instead of using the reliability indices in (2.3) to measure the deviation from the target, for example the corresponding probabilities of failure can be used. Also, a nonlinear objective function giving relatively more weight to reliability indices smaller than the target compared to those larger than the target can be used. The above formulations can easily be extended to include a lower bound on the reliability or probability of failure for each failure mode.

Ad 6. The optimal partial safety factors are obtained by numerical solution of the optimization problem in step 5. The reliability index β_j for combination *j* is obtained from a limit state equation written as:

$$g(\mathbf{x}, \mathbf{p}_j, \mathbf{z}) = 0 \tag{2.4}$$

where $\mathbf{z}$ represents the design parameters, e.g. cross-sectional dimensions.

For given partial safety factors $\boldsymbol{\gamma}$ the reliability index is obtained as follows. First, the optimal design $\mathbf{z}$ is determined using the design equations and relevant design constraints. The design equation is written:

$$G(\mathbf{x}_d, \mathbf{p}_j, \mathbf{z}) = 0 \tag{2.5}$$

where $\mathbf{x}_d$ are design values obtained from characteristic values of the stochastic variables $\mathbf{x}_c$ and the partial safety factors $\boldsymbol{\gamma}$.

Ad 7. A first guess of the partial safety factors is obtained by solving the optimization problem (2.3). Next, the final partial safety factors are determined taking into account current engineering judgment and tradition.

In the above procedure partial safety factors for all loads and strengths / resistances are obtained simultaneously. For wind turbines the following comments can be made to each of the above steps:

a) The scope could be design of structural components of wind turbines incl. blades, tower, main frame, main shaft and foundation. It is important to define a set of standards / codes to be used for calculation of design loads and resistances / load bearing capacities. In some standards the models used to calculate the design resistances contain 'hidden' safety in the parameters / formulas to be used. These hidden safety elements are very important to be accounted for in a code calibration.
b) The target reliability level should be defined considering the design process used for wind turbines using wind turbine classes. Based on the information used for assessment of partial safety factors in IEC 61400-1, see Tarp-Johansen et al. [2], [3] and Tarp-Johansen [4] and observed failure rates, a target annual reliability index β_t equal to 3.1 – 3.8 corresponding to an annual probability of failure equal to 10^{-4} – 10^{-3}, see section 4. A higher target reliability level could be expected for offshore wind turbines compared to onshore wind turbines due to larger consequences of failure, see chapter 4.
c) The partial safety factors could be common partial safety factors for loads and for the materials:
    - One partial safety factor for each strength parameter (e.g. used for concrete structures in Eurocodes)

- One partial safety factor for the load bearing capacity for a structural component (e.g. used for steel structures in Eurocodes)
- or a combination (e.g. used for geotechnical design in Eurocodes).

d) Failure modes to be considered are among others: yielding, excessive deformation, collapse, local and global buckling (stability) and fatigue. For each of the failure modes a design equation has to be formulated according to the model in the standard used (e.g. Eurocode 3 for design of steel towers). Further, for each failure mode a limit state equation has to be formulated corresponding to the design equation, but including stochastic variables for loads, resistances and model uncertainties.

e) Next, stochastic models have to be formulated for the stochastic variables, incl. statistical dependencies / correlations.

f) The objective in (2.3) can be used, maybe supplemented by a lower limit: $\beta_j(\boldsymbol{\gamma}) \geq \beta_{\min}$.

g) The optimization problem is solved. A stepwise procedure can be applied where the load partial safety factors are chosen (based on a preliminary optimization), and the material partial safety factors are calibrated individually for each material (steel, concrete, composites, etc.).

h) The verification and adjustment to experience and practical aspects is very important, and is typically done by standardization committees.

2.3 Reliability-based calibration of partial safety factors

In section 2.2 a general approach for calibration of partial safety factors is described. However, in practice a stepwise, iterative procedure can be used. First, steps 1 - 3 in the JCSS approach are carried through and the load partial safety factors are chosen based on experience, practical aspects and preliminary reliability evaluations. Next, partial safety factors for resistances and material strengths are calibrated individually for each application area. These partial safety factors are calibrated to the required reliability level using limit state equations corresponding to the design equations to be used in practice and stated in the design standard. Step 7 in the JCSS approach is finally carried through for the complete set of partial safety factors.

3 REQUIRED RELIABILITY LEVEL

The probability of failure of a failure mode modeled by a limit state equation $g(\mathbf{X})$ is

$$P_f = P(g(\mathbf{X}) \leq 0) \cong \Phi(-\beta) \tag{3.1}$$

where β is the reliability index and $\Phi(\ )$ is the standard Normal distribution function. The relationship between the reliability index and the probability of failure is shown in the table 3.1.

P_F	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}
β	2.3	3.1	3.7	4.3	4.7	5.2

Table 3.1. Relationship between reliability index, β and probability of failure, P_F.

The target reliability level can be given in terms of a maximum annual probability of failures (i.e. reference time equal to 1 year) or a maximum lifetime probability of failure (i.e. for wind turbines a reference time equal to 20 – 25 years).

For civil and structural engineering standards / codes of practice where failure can imply risk of loss of human lives target reliabilities are generally given based on annual probabilities. The optimal reliability level can be found by considering representative cost-benefit based optimization problems where the life-cycle expected cost of energy is minimized with appropriate constraints related to acceptable risks of loss of human lives, e.g. based on LQI (Life Quality Index) principles.

Examples of reliability levels required (implicitly) in some relevant standards / codes (for normal consequence / reliability class) are:

- Building codes (in Europe): Eurocode EN1990:2002:
  - The Eurocodes implicitly recommends a target reliability based on lifetime probabilities corresponding to a lifetime reliability index equal to 3.8, see e.g. Sedlacek et al.[8] where the basis for the material partial safety factors are described (assessed using the design value approach, see section 2.1).
  - Extreme load: annual: $P_F = 10^{-6}$ ($\beta = 4.7$) or lifetime (50 years): $P_F = 10^{-4}$ ($\beta = 3.8$)
    - Assumptions on stochastic models: Strengths: Lognormal; Permanent loads: Normal; Variable loads: Gumbel
  - Fatigue: design life (50 years): $P_F = 0.06 – 10^{-4}$ ($\beta = 1.5 - 3.8$) depending on possibility for inspections and criticality

- Fixed steel offshore structures, see e.g. ISO 19902:2007:
  - manned: annual $P_F \sim 3\ 10^{-5}$ or $\beta = 4.0$
  - For structures that are unmanned or evacuated in severe storms and where other consequences of failure are not very significant: annual $P_F \sim 5\ 10^{-4}$ or $\beta = 3.3$

- ISO 2394:1998 [5]
  - Extreme load: see table 3.2
  - Fatigue: design life: $P_F = 10^{-2} – 10^{-3}$ ($\beta = 2.3$-3.1) depending on possibility for inspections

- JCSS recommends reliability requirements based on annual failure probabilities for structural systems for ultimate limit states, see Table 3.3. These are based on optimization procedures and on the assumption that for almost all engineering facilities the only reasonable reconstruction policy is systematic rebuilding or repair.

In table 3.2 and 3.3 target failure probabilities are given for ultimate limit states based on recommendations of JCSS (Joint Committee Structural Safety – Probabilistic Model Code) [14] and from ISO 2394:1998 [5] with a life-time reference period. The values in [14] are associated with the stochastic models recommended in JCSS PMC [14]. The values in ISO 2394:1998 [5] have been derived with the assumption of Lognormal or Weibull distributions for resistance, Normal distribution for permanent loads and Gumbel extreme value distribution for time-varying loads and using the design value method, see section 2. It is important that the same assumptions (or assumptions close to them) are used if the values given in table 4 are applied for probabilistic calculations.

Table 3.2. Target lifetime reliability index and probability of failure according to ISO 2394 [5].

Relative costs of safety measures	Consequences of failure			
	Small	Minor / Some	Moderate	Large
High cost of safety measures	$\beta^t = 0$, $P_F = 0.5$	$\beta^t = 1.5$, $P_F = 7 \cdot 10^{-2}$	$\beta^t = 2.3$, $P_F = 10^{-2}$	$\beta^t = 3.1$, $P_F = 10^{-3}$
Moderate cost of safety measures	$\beta^t = 1.3$, $P_F = 10^{-1}$	$\beta^t = 2.$ $P_F = 10^{-2}$	$\beta^t = 3.1$, $P_F = 10^{-3}$	$\beta^t = 3.8$, $P_F = 10^{-4}$
Low cost of safety measures	$\beta^t = 2.3$, $P_F = 10^{-2}$	$\beta^t = 3.$ $P_F = 10^{-3}$	$\beta^t = 3.8$, $P_F = 10^{-4}$	$\beta^t = 4.3$, $P_F = 10^{-5}$

Table 3.3. Target annual reliability index and probability of failure according to JCSS [14].

Relative costs of safety measures	Consequences of failure		
	Minor / Some	Moderate	Large
High cost of safety measures	$\beta^t = 3.1$, $P_F = 10^{-3}$	$\beta^t = 3.3$, $P_F = 5 \cdot 10^{-4}$	$\beta^t = 3.7$, $P_F = 10^{-4}$
Moderate cost of safety measures	$\beta^t = 3.7$, $P_F = 10^{-4}$	$\beta^t = 4.2$, $P_F = 10^{-5}$	$\beta^t = 4.4$, $P_F = 5 \cdot 10^{-6}$
Low cost of safety measures	$\beta^t = 4.2$, $P_F = 10^{-5}$	$\beta^t = 4.4$, $P_F = 5 \cdot 10^{-6}$	$\beta^t = 4.7$, $P_F = 10^{-6}$

It should be noted that the β-values (and the corresponding failure probabilities) are formal / notional numbers, intended primarily as a tool for developing consistent design rules, rather than giving a description of the structural failure frequency. E.g. the effect of human errors is not included.

For wind turbines the risk of loss of human lives in case of failure of a structural element is generally very small. Further, it can be assumed that wind turbines are systematically reconstructed in case of collapse or end of lifetime. In that case also target reliabilities based on annual probabilities should be used, see JCSS (2002). The optimal reliability level can be found by considering representative cost-benefit based optimization problems where the life-cycle expected cost of energy is minimized.

It is assumed that for wind turbines:

- A systematic reconstruction policy is used (a new wind turbine is erected in case of failure or expiry of lifetime).
- Consequences of a failure are only economic (no fatalities and no pollution).
- Cost of energy is important which implies that the relative cost of safety measures can be considered large (material cost savings are important).
- Wind turbines are designed to a certain wind turbine class, i.e. not all wind turbines are 'designed to the limit'.

Based on these considerations the target reliability level corresponding to a minimum annual probability of failure is recommended to be

$$P_f = 5 \cdot 10^{-4} \tag{3.2}$$

corresponding to an annual reliability index equal to 3.3. This reliability level corresponds to minor / moderate consequences of failure and moderate / high cost of safety measure. It is noted that this reliability level corresponds to the reliability level for offshore structures that are unmanned or evacuated in severe storms and where other consequences of failure are not very significant.

4 CALIBRATION OF PARTIAL SAFETY FACTORS

4.1 Basic models for calculating the design load bearing capacity

The load bearing capacity, R is assumed to be estimated by the following model:

$$R = b\,\delta\,R(\mathbf{X}, a) \tag{4.1}$$

where

R	the measurable load bearing capacity
$\mathbf{X}$	vector of random variables (e.g. strength and stiffness parameters). Each of the strength parameters is modelled as a LogNormal stochastic variable with coefficient of variation V_X
a	set of deterministic variables, e.g. geometrical parameters
$R(\;)$	model for the load bearing capacity / resistance
δ	model uncertainty assumed to be modelled as a LogNormal stochastic variable with mean value 1 and coefficient of variation V_δ
b	bias in the resistance model, $R(\;)$

Equation (4.1) can be used for reliability analysis if the uncertain parameters are modeled by stochastic variables. It is noted that model uncertainty / bias / additional safety is taken into account though the model uncertainty δ and the bias b. Some calculation models for the load bearing capacity are conservative in the way that some parameters are chosen on the safe side. It is important when deriving the material partial safety factors to account for such additional safety / bias.

For deterministic design three models are considered

- Model 1: first, partial safety factors accounting for uncertainties of the strength and stiffness parameters are used to obtain design values of strength and stiffness parameters and the design value of the resistance model is determined. Next, this value is divided by a partial safety factor accounting for model uncertainty to obtain the design value of the load bearing capacity.
- Model 2: first, the value of the resistance model is calculated using characteristic values of the strength and stiffness parameters. Next, this value is divided by a partial safety factor accounting for the total uncertainty of the resistance model (model uncertainty and uncertainty of strength and stiffness parameters) to obtain the design value of the load bearing capacity.
- Model 3: the characteristic value of the load bearing capacity is obtained e.g. based on tests and this value is divided by a partial safety factor accounting for the uncertainty of the load bearing capacity to obtain the design value of the load bearing capacity.

Model 1 where design values are determined for the material strength parameters:

$$R_d = \frac{R(X_d, a_d)}{\gamma_\Delta} \tag{4.2}$$

where

a_d	design value for geometrical data
X_d	design values for strength parameters

γ_Δ partial safety factor related to the model uncertainty for the resistance model including bias in resistance model

If more than one strength parameter is used in the resistance model, then design values are applied for each strength parameter in (4.2).

The design value of a strength parameter(s) X_d is determined by:

$$X_d = \eta \frac{X_k}{\gamma_m} \tag{4.3}$$

where

η conversion factor taking into account load duration effects, moisture, temperature, scale effects,…

X_k characteristic value of strength parameter generally defined by the 5% quantile

γ_m partial safety factor for strength parameter depending on the coefficient of variation V_X

If the resistance model is linear in the strength parameters then $R_d = R(X_d, a_d)$ and X_d for each of the strength parameters is obtained using a partial safety factor $\gamma_M = \gamma_m \gamma_\Delta$.

The partial safety factor γ_Δ depends on the uncertainty of the resistance model, incl. bias:

$$\gamma_\Delta = \frac{\gamma_\delta}{b} \tag{4.4}$$

where

γ_δ partial safety factor depending on the model uncertainty with coefficient of variation V_δ without taking into account bias in the resistance model.

The design values can generally be obtained by:

$$R_d = \frac{R\left(\eta_1 \frac{X_{1,k}}{\gamma_{m,1}}, \ldots, \eta_n \frac{X_{n,k}}{\gamma_{m,n}}, a_d\right)}{\gamma_\Delta} \tag{4.5}$$

where

$\frac{X_{i,k}}{\gamma_{m,i}}$ design value of material parameter X_i with characteristic value $X_{i,k}$

a_d design value for geometrical data

$\gamma_{m,i}$ partial safety factor for material parameter X_i

η_i conversion factor for material parameter X_i, accounting for additional effects, e.g. scale effects and time duration effects and, failure type, etc.

γ_Δ partial safety factor for load bearing capacity

If the resistance model is linear then the partial safety factors can be applied directly to the strength parameters:

$$Y_d = R\left(\eta_1 \frac{X_{1,k}}{\gamma_{M,1}}, ..., \eta_n \frac{X_{n,k}}{\gamma_{M,n}}, a_d\right) \quad (4.6)$$

where

$$\gamma_{M,i} = \gamma_\Delta \gamma_{m,i} \quad (4.7)$$

Model 2 where a characteristic resistance is obtained using characteristic values of the material strength parameters and the design value of the resistance is obtained from:

$$R_d = \frac{R(\eta X_k, a_k)}{\gamma_M} \quad (4.8)$$

where

γ_M partial safety factor related to the total uncertainty of the resistance incl. bias.

The total uncertainty of the resistance depends on the model uncertainty δ, the bias of the resistance model and the uncertainty related to the strength parameters X though the resistance function $R(X,a)$. The material partial safety factors are correspondingly obtained from

$$\gamma_M = \frac{\gamma_\delta \gamma_R}{b} \quad (4.9)$$

where

γ_R partial safety factor depending on the resistance uncertainty with coefficient of variation V_R from the strength parameters though the resistance function $R(X,a)$

γ_δ partial safety factor depending on the model uncertainty with coefficient of variation V_δ.

Further, in some cases one partial safety factor is used and applied to the characteristic value of load bearing capacity, $R_k = R(\eta_1 X_{1,k}, ..., \eta_n X_{n,k}, a_d)$:

$$Y_d = \frac{R(\eta_1 X_{1,k}, ..., \eta_n X_{n,k}, a_d)}{\gamma_M} \quad (4.10)$$

Model 3 where a characteristic resistance is estimated based on tests:

$$R_d = \frac{R_k}{\gamma_M} \quad (4.11)$$

where

R_k characteristic resistance estimated based on tests. R_k is generally defined by the 5% quantile

γ_M partial safety factor related to uncertainty of the resistance obtained based on tests, V_R.

The material partial safety factors should be calibrated such that failure probabilities for the relevant failure modes are close to the target reliability level in (4.2). Where relevant statistical uncertainty and uncertainty related to transformation from laboratory to real structure should be included.

It is noted that the three models above are used in e.g. the Eurocodes as follows:

- Eurocode 2 (concrete) uses Model 1
- Eurocode 3 (steel) uses Model 2
- Eurocode 7 (foundation) uses all three models

4.2 DLC 1.1 and 6.1 with extreme load

This section describes calibration of material partial safety factors for DLC 1.1 and 6.1 with extreme loads.

The calibrations are performed assuming that there is

- no bias (hidden safety) in calculation of load effects
- no bias (hidden safety) in calculation of load bearing capacities
- no scale effects, time duration effects,…

i.e. η = 1 and b = 1 corresponding to no conservatism (hidden safety) in the models to calculate load effects and load bearing capacities. If $\eta \neq 1$ or $b \neq 1$ this is accounted for afterwards.

The following generic limit state equation the extreme load effect in operation (DLC 1.1) or standstill (DLC 6.1) is used (without permanent loads)

$$g = z\,\delta\,R - X_{dyn}X_{\exp}X_{aero}X_{str}L \tag{4.12}$$

where

z	design parameter, e.g. cross-sectional area
δ	model uncertainty load bearing model
R	uncertainty in dominating strength parameter
X_{dyn}	uncertainty related to modeling of the dynamic response, including uncertainty in damping ratios and eigenfrequencies
X_{exp}	uncertainty related to the modeling of the exposure (site assessment) - such as the terrain roughness and the landscape topography
X_{aero}	uncertainty in assessment of lift and drag coefficients and additionally utilization of BEM, dynamic stall models, etc
X_{str}	uncertainty related to the computation of the load-effects given external load
L	uncertainty related to the extreme load-effect due to wind loads

The stochastic model in Table 4.1 is used as 'representative', see [16].

Table 4.1: Stochastic models for physical, model and statistical uncertainties.

Variable	Distribution	Mean	COV	Quantile	Comment
R	Lognormal	-	V_R	5%	Strength
δ	Lognormal	-	V_δ	Mean	Model uncertainty
L – DLC 1.1	Weibull	-	0.15	0.98	Annual maximum load effect obtained by load extrapolation
L – DLC 6.1	Gumbel	-	0.2	0.98	Annual maximum wind pressure – European wind conditions
X_{dyn}	Lognormal	1.00	0.05	Mean	
X_{exp}	Lognormal	1.00	0.15	Mean	
X_{aero}	Gumbel	1.00	0.10	Mean	
X_{str}	Lognormal	1.00	0.03	Mean	

The corresponding design equation is written:

$$\frac{z\,R_k}{\gamma_R} - \gamma_f L_k \geq 0 \qquad (4.13)$$

where

R_k characteristic value of load bearing capacity

L_k characteristic value of variable load

γ_M partial safety factor for load bearing capacity

γ_f partial safety factor for load effect = 1.35

It is noted that it is assumed that the characteristic value for the load bearing capacity is determined as the 5% quantile of the strength parameter and the mean value of the model uncertainty. It could be considered instead to use the 5% quantile of the product $\delta \cdot R$. It is also noted that γ_f = 1.35 is used both for DLC 1.1 and DLC 6.1.

Table 4.2. Partial safety factor for load bearing capacity, γ_R. DLC 1.1 with Weibull distribution COV=0.15. Target (annual) reliability index = 3.3. γ_f = 1.35.

	V_δ=0,00	0,05	0,10	0,15	0,20
V_R =0,05	1.16	1.18	1.24	1.35	1.49
0,10	1.12	1.14	1.20	1.29	1.43
0,15	1.11	1.13	1.19	1.28	1.40
0,20	1.13	1.15	1.20	1.28	1.40
0,25	1.17	1.18	1.23	1.31	1.42

Table 4.3. Partial safety factor for load bearing capacity, γ_M. DLC 6.1 with Gumbel distribution COV=0.20. Target (annual) reliability index = 3.3. γ_f = 1.35.

	V_δ=0,00	0,05	0,10	0,15	0,20
V_R =0,05	1.14	1.16	1.20	1.28	1.40
0,10	1.09	1.11	1.15	1.22	1.33
0,15	1.07	1.08	1.12	1.19	1.29
0,20	1.06	1.08	1.11	1.18	1.27
0,25	1.07	1.09	1.12	1.19	1.28

Comments:

- It is seen that the required partial safety factor decreases for increasing coefficient of variation (COV) for the resistance. The reason is that the characteristic value (5% quantiles) for *R* decreases more for increasing COV than the resulting design value of the load bearing capacity (obtained by the reliability analyses) decreases. The same effect is seen when the coefficient of variation for the load-effect is increased.
- The partial safety factors in Table 4.2 are conservative compared to Table 4.3.

The calibrated partial safety factors in Table 4.2 is as an example used to obtain partial safety factors for some structural materials and for illustration choices of b are made:

- Structural steel component with yielding failure criteria:
  - o V_R=0.05, V_δ=0.05, b=1.1 (ductile failure with extra load bearing capacity), $\gamma_R \approx 1.18$ →

    Resulting partial safety factor γ_M = 1.18 / 1.1 = 1.06 ~ 1.1

- Structural steel component with buckling failure criteria:
  - V_R=0.05, V_δ=0.13, b= 1 / 0,85 (approx average bias in buckling model in EC 2, see annex B in 'Example of stochastic modeling ….), $\gamma_R \approx 1.28$ →

    Resulting partial safety factor γ_M = 1.28 * 0.85 = 1.09 ~ 1.1
- Structural concrete component with failure criteria dominated by concrete compression strength:
  - V_R=0.10, V_δ=0.05, b=1.0, $\gamma_R \approx 1.14$ →

    In Eurocode 2 for concrete it is taken into account that test specimens are not taken from the structure and therefore a conversion factor 1.15 is introduced (see Sedlacek et al. [8]) implying γ_M = 1.14 * 1.15 ~ 1.3
- Structural concrete component with failure criteria dominated by reinforcement strength:
  - V_R=0.05, V_δ=0.05, b=1.0, $\gamma_R \approx 1.18$ →

    Resulting partial safety factor γ_M = 1.18 ~ 1.2

If a load partial safety factor equal to 1.35 is used, then partial safety factors were calibrated in section 4.2 such that the reliability level becomes equal to the target reliability level P_F^t specified by (3.2). Based on the calibrated partial safety factors in table 4.2 the partial safety factors γ_m and γ_M in Table 4.4 and γ_δ in Table 4.5 are derived such that approximately the same resulting partial safety factors are obtained as using Table 4.2 for given combinations of V_X / V_R and V_δ. It is seen that γ_m and γ_R both are equal to one for all coefficients of variations. Reasons for that are that part of safety is provided though use of the 5% quantile as characteristic value and that the reliability level is relatively low (compared to the reliability level required for e.g. buildings).

Table 4.4. γ_m, γ_M - partial safety factor for strength parameter or resistance.

Coefficient of variation for strength parameter in model 1, V_X or resistance in model 2 and 3, V_R	≤5 %	10 %	15 %	20 %	25 %
γ_m in model 1 or γ_M in model 2 and 3	1.0	1.0	1.0	1.0	1.0

Table 4.5. γ_δ - partial safety factor for model uncertainty.

Coefficient of variation for model uncertainty for resistance model in model 1, V_δ	0 %	5 %	10 %	15 %	20 %
γ_δ	1.15	1.20	1.25	1.35	1.45

It is noted that the partial safety factors in Tables 4.4 and 4.5 are calibrated without taking into account the bias b and with the characteristic value for the model uncertainty equal to 1.

Alternatively the values in Table 4.4a and 4.5a can be used, but the small variations for γ_m and γ_M are considered inconvenient.

Table 4.4a. γ_m, γ_M - partial safety factor for strength parameter or resistance.

Coefficient of variation for strength parameter in model 1, V_X or resistance in model 2 and 3, V_R	≤5 %	10 %	15 %	20 %	25 %
γ_m in model 1 or γ_R in model 2 and 3	1.05	1.025	1.0	1.025	1.05

Table 4.5a. γ_δ - partial safety factor for model uncertainty.

Coefficient of variation for model uncertainty for resistance model in model 1, V_δ	0 %	5 %	10 %	15 %	20 %
γ_δ	1.1	1.15	1.2	1.3	1.4

The same examples as above are considered but now using the partial safety factors in table 4.4 and 4.5:

- Structural steel component with yielding failure criteria:
  - V_R =0.05, V_δ=0.05, b=1.1 (ductile failure with extra load bearing capacity), $\gamma_R \approx 1.20$ → Resulting partial safety factor γ_M = 1.20 / 1.1 ~ 1.1

- Structural steel component with buckling failure criteria:
  - V_R =0.05, V_δ=0.13, b= 1 / 0,85 (approx average bias in buckling model in EC 2, see annex B in 'Example of stochastic modeling ….), $\gamma_R \approx 1.31$ → Resulting partial safety factor γ_M = 1.31 * 0.85 ~ 1.1

- Structural concrete component with failure criteria dominated by concrete compression strength:
  - V_R =0.10, V_δ=0.05, b=1.0, $\gamma_R \approx$ 1.20 → In Eurocode 2 for concrete it is taken into account that test specimens are not taken from the structure and therefore a conversion factor 1.15 is introduced (see Sedlacek et al. [8]) implying γ_M = 1.20 * 1.15 ~ 1.4

- Structural concrete component with failure criteria dominated by reinforcement strength:
  - V_R =0.05, V_δ=0.05, b=1.0, $\gamma_R \approx$ 1.20 → Resulting partial safety factor γ_M = 1.2

4.3 Fatigue of welded details in steel structures

This section considers calibration of partial safety factors for welded details in steel structures. The details are described in Annex C.

Basically a linear SN-curves is considered with the SN relation written

$$N = K(\Delta\sigma)^{-m} \tag{4.14}$$

where N is the number of stress cycles to failure with constant stress ranges $\Delta\sigma$. K and m are dependent on the fatigue critical detail. Appendix C describes the model for a bi-linear SN-curve.

The fatigue strength $\Delta\sigma_F$ is defined as the value of S for $N_D = 2 \cdot 10^6$.

If one fatigue critical detail is considered then the annual probability of failure is obtained from:

$$\Delta P_{F,t} = P_{\mathrm{COL|FAT}} P(\text{Fatigue failure in year } t) \tag{4.15}$$

where $P(\text{Fatigue failure in year } t)$ is the probability of failure in year *t* and $P_{\text{COL|FAT}}$ is the probability of collapse of the structure given fatigue failure - modelling the importance of the detail / consequence of failure.

The probability of failure in year *t* given survival up to year *t* is estimated by

$$\Delta P_{F,t} = P_{\text{COL|FAT}}\left(P(g(t) \le 0) - P(g(t-1) \le 0)\right) / P(g(t) \le 0) \tag{4.16}$$

where the limit state equation is based on SN-curves, Miner's rule for linear accumulation of fatigue damage and by introducing stochastic variables accounting for uncertainties in fatigue loading and strength, see Annex C. The stochastic model shown in Table 4.6 is considered as representative for a fatigue sensitive welded steel detail where the fatigue strength is represented by a bi-linear SN-curve. It is assumed that the design lifetime is T_L = 25 year.

Table 4.6. Stochastic model.

Variable	Distribution	Expected value	Standard deviation / Coefficient Of variation	Comment
Δ	N	1	COV_Δ = 0.30	Model uncertainty Miner's rule
X_{Wind}	LN	1	COV_{Wind}	Model uncertainty wind load
X_{SCF}	LN	1	COV_{SCF}	Model uncertainty stress concentration factor
m_1	D	3		Slope SN curve
$\log K_1$	N	determined from $\Delta\sigma_D$	$\sigma_{\log K_1}$ = 0.2	Parameter SN curve
m_2	D	5		Slope SN curve
$\log K_2$	N	determined from $\Delta\sigma_D$	$\sigma_{\log K_2}$ = 0.2	Parameter SN curve
$\Delta\sigma_F$	D	71 MPa		Fatigue strength
$\log K_1$ and $\log K_2$ are fully correlated				

For deterministic design partial safety factors are introduced:

- γ_f : a fatigue load partial safety factor multiplied to the fatigue stress ranges obtained by e.g. Rainflow counting.
- γ_m : a fatigue strength partial safety factor. The design value of the fatigue strength is obtained by dividing the characteristic fatigue strength by γ_m.

The characteristic fatigue strength can be defined in various ways, namely based on

- the mean minus two standard deviations of $\log K$.
- the 5% quantile of $\log K$, i.e. the mean minus 1.65 times the standard deviation of $\log K$.
- the mean of $\log K$.

If the SN-curves are obtained by a limited number of tests then statistical uncertainty has to be accounted for. This can also be done in various ways, namely based on

- Bayesian statistics, see e.g. ISO 2394 [5] and EN 1990, annex D [6]. See also the informative annex K in IEC 61400-1 ed 4 'Calibration of structural material safety factors and structural design assisted by testing', Table K.4 (for characteristic values defined by 5% quantiles).
- Classical statistics. The characteristic SN-curve can be obtained using a 75% confidence level of 95% probability of survival for log *N* (as recommended in e.g. EN 1993-1-9 [7]). Note that a 75% confidence level using classical statistics results in approximately the same characteristic SN-curves as by application of Bayesian statistics.

The required product of the partial safety factors $\gamma_f \gamma_m$ is obtained using the stochastic model in Table 4.6, the limit state equation in Annex C, the characteristic SN-curve defined as the mean minus two standard deviations of $\log K$.

Table 4.7 shows the required $\gamma_f \gamma_m$ for $\Delta P_{F,\max}$ = 5 10^{-4} (normal/high consequence of failure) and for the characteristic fatigue strength defined as the mean minus two standard deviations of $\log K$. The corresponding reliability indices are 3.3 and 2.6. The results are shown as function of the total coefficient of variation of the fatigue load: $COV_{load} = \sqrt{COV_{Wind}^2 + COV_{SCF}^2}$.

Table 4.7. Required partial safety factors $\gamma_f \gamma_m$ given $\Delta\beta_{\min,FAT}$ as function of *COV* for fatigue load.

$\Delta\beta_{\min,FAT}$ \ COV_{load}	0,00	0,05	0,10	0,15	0,20	0,25	0,30
3,3 (5 10^{-4})	1.04	1.06	1.12	1.21	1.32	1.43	1.56

Assuming that a coefficient of variation for the fatigue load ranges is typically within the interval 15-20% the partial safety factor γ_f in table 4.8 and

γ_m = 1.25

are recommended.

Table 4.8. Recommended partial safety factor for fatigue stress ranges, γ_f.

Coefficient of variation, COV_{load}	0-5 %	5-10 %	10-15 %	15-20 %	20-25 %	25-30 %
γ_f	0.85	0.90	0.95	1.00	1.10	1.20

Typically the coefficient of variation, COV_{load} will be at least equal to 15%.

If the characteristic fatigue strength is based on the 5% quantile of $\log K$, or the mean of $\log K$ then the partial safety factor γ_m has to be increased with the factors:

- 5% quantile of $\log K$: factor 1.04
- mean of $\log K$: factor 1.23

Next, the consequence of failure can be included by the concepts 'damage tolerant' and 'safe life' reliability assessment methods based on the following descriptions (from EN 1993-1-9:2005 [7]).

a) damage tolerant method
- selecting details, materials and stress levels so that in the event of the formation of cracks a low rate of crack propagation and a long critical crack length would result,
- provision of multiple load path

- provision of crack-arresting details,
- provision of readily inspectable details during regular inspections.

b) safe-life method
- selecting details and stress levels resulting in a fatigue life sufficient to achieve the target β – value at the end of the design service life. No inspections are required.

Generally, for the 'Damage tolerant' approach either the structure is redundant or inspections are performed (or a combination of these).

The fatigue strength partial safety factor is then generalised according to Table 4.8. The partial safety factors are assumed to correspond to normal consequences of failure, i.e. component class 2 in IEC 61400-1.

Table 4.8. Recommended values for partial safety factor for fatigue strength, γ_m.

Assessment method	γ_m
Damage tolerant	1.10
Safe life	1.25

4.4 Design Load Cases with faults

In Design Load Case 2 a wide range of load cases related to faults are considered. These DLCs are very dependent on assumed type and frequency of faults and the consequences of the faults. Further, they depend on the site and wind turbine type considered. In this section a special, generic case related to DLC 2.1 and 2.2 is considered.

The annual failure rate is assumed to be estimated from

$$\lambda_{F,E} = \lambda_E P\big(g(X) \leq 0 \,|\, E\big) \qquad (4.17)$$

where
λ_E annual rate of faults
$P\big(g(X) \leq 0 \,|\, E\big)$ probability of failure given fault

For DLC 2 the acceptable annual failure rate is assumed to correspond to the acceptable annual failure probability for DLC 1 and 6, i.e.

$$\lambda_{F,E} \leq \lambda^{\max} \qquad (4.18)$$

where
$\lambda^{\max}$ maximum acceptable annual failure rate = $5 \cdot 10^{-4}$ per year

Table 4.9. Stochastic models for physical, model and statistical uncertainties.

Variable	Distribution	Mean	COV	Quantile	Comment
R	Lognormal	-	V_R	5%	Strength
δ	Lognormal	-	V_δ	Mean	Model uncertainty
Q – DLC 2.x	Weibull	-	V_Q	Mean of upper half of density function ≈ 77% quantile	Load effect given faults, see IEC 61400-1, section 7.6.2

X_{dyn}	Lognormal	1.00	0.05	Mean	
X_{exp}	Lognormal	1.00	0.15	Mean	
X_{aero}	Gumbel	1.00	0.10	Mean	
X_{str}	Lognormal	1.00	0.03	Mean	

$P(g(X) \leq 0 | E)$ is estimated using the following representative limit state equation, which is basically the same as the limit state equations considered for DLC 1 and 6:

$$g(X) = z\,\delta\,R - X_{dyn} X_{\exp} X_{aero} X_{str} Q \quad (4.19)$$

where

z design parameter, e.g. cross-sectional area

δ model uncertainty for resistance model

R uncertainty in dominating strength parameter in resistance model

X_{dyn} uncertainty related to modeling of the dynamic response, including uncertainty in damping ratios and eigenfrequencies

X_{exp} uncertainty related to the modeling of the exposure (site assessment) - such as the terrain roughness and the landscape topography

X_{aero} uncertainty in assessment of lift and drag coefficients and additionally utilization of BEM, dynamic stall models, etc.

X_{str} uncertainty related to the computation of the load-effects given external load

Q uncertainty related to the load-effect given faults

The stochastic model in Table 4.9 is used as 'representative' uncertainties.

The corresponding design equation is written:

$$\frac{z\,R_k}{\gamma_R} - \gamma_F Q_k \geq 0 \quad (4.20)$$

where

R_k characteristic value of load bearing capacity

Q_k characteristic value of variable load

γ_R partial safety factor for load bearing capacity

γ_F partial safety factor for load effect

It is noted that it is assumed that the characteristic value for the load bearing capacity is determined as the 5% quantile of the strength parameter and the mean value of the model uncertainty. The partial safety factor γ_R is obtained from Table 4.5.

γ_F is determined as a function of λ_E such that (4.18) is fulfilled.

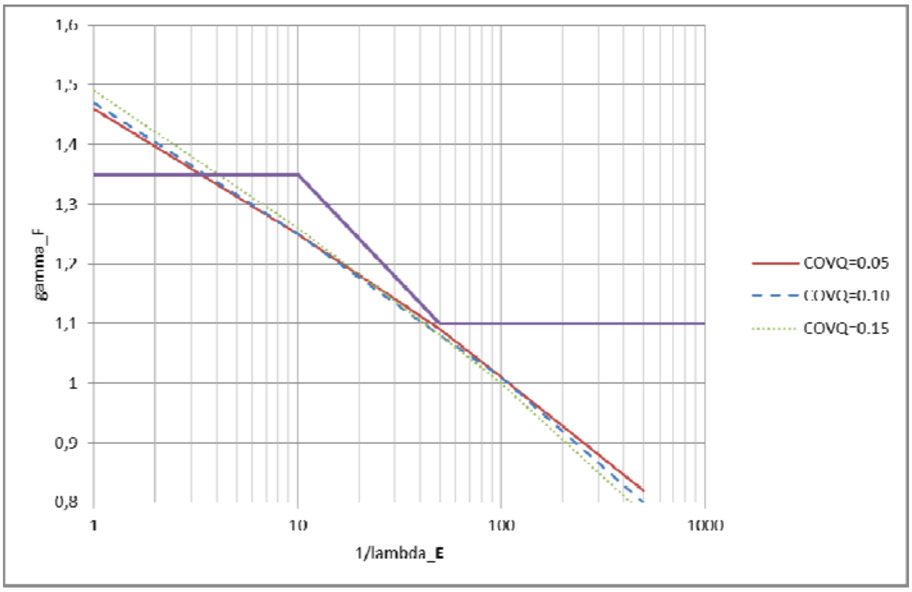

Figure 4.1. γ_F as function of $1/\lambda_E$ [year] for V_Q = 0.05, 0.10 and 0.15 and (V_R, V_δ) = (0.05, 0.10).

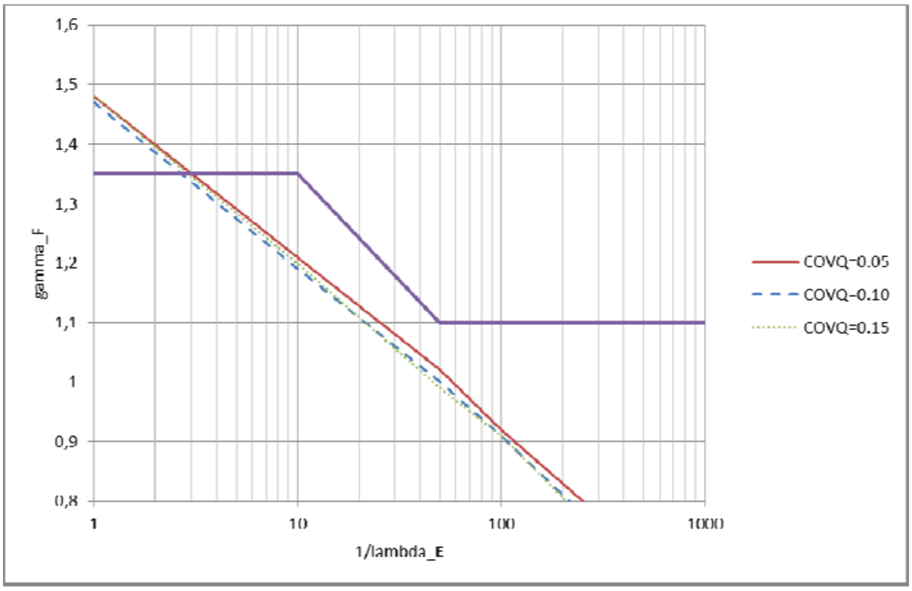

Figure 4.2. γ_F as function of $1/\lambda_E$ [year] for V_Q = 0.05, 0.10 and 0.15 and (V_R, V_δ) = (0.15, 0.15).

Figures 4.1 and 4.2 show the results for V_Q = 0.05, 0.10 and 0.15 for the cases

1. $V_R = 0.05$ and $V_\delta = 0.10$
2. $V_R = 0.15$ and $V_\delta = 0.15$

It is seen that a load partial safety factor equal to 1.35 for $1/\lambda_E < 10$ year, equal to 1.10 for $1/\lambda_E > 50$ year and linear interpolation in between (in logarithmic scale) results in a satisfactory reliability level for $1/\lambda_E > 3$ year and for the coefficient of variation (COV) for the load effect given a fault between 5% and 15%. The example in Figure 4.3 shows that typically the COV for the load effect in case of a control system fault is in that range. This example is based on the consequence of a control system failure, see Hammerum [15]. Emergency stops have been simulated, which traditionally results in high tower loads. 120 simulations were made for each wind 2 m/s bin in the range 4-24 m/s, see Figure 4.3.

It is noted that if the load effect is assumed to be Normal or LogNormal distributed then only very small changes in the load partial safety factor is needed.

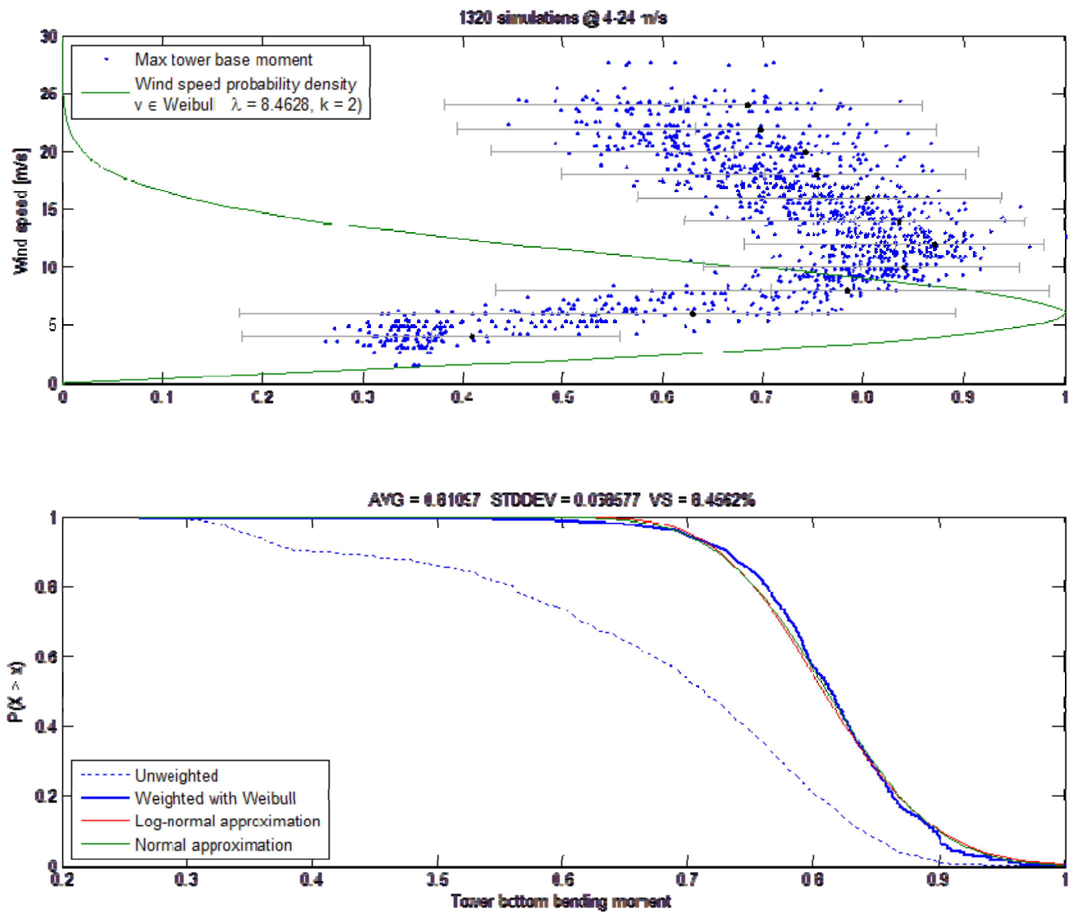

Figure 4.3. Example of simulation results for maximum tower bottom bending moments in case of control system failure from Hammerum [15]. The upper part shows the mean wind speed plotted against the maximum tower bottom bending moments. The lower plot shows the empiric exceedance function for the maximum tower bending moment, weighted with the Weibull distribution of the wind speed shown in the upper plot.

4.5 Reliability analysis of influence of component class partial safety factor γ_c

This section describes reliability based investigations related to the consequence of failure factor γ_c which is introduced to distinguish between:

- Component class 1: used for "fail-safe" structural components whose failure does not result in the failure of a major part of a wind turbine, for example replaceable bearings with monitoring. Component class 1 is considered to correspond to low consequence of failure.
- Component class 2: used for "non fail-safe" structural components whose failures may lead to the failure of a major part of a wind turbine. Component class 2 is considered to correspond to middle consequence of failure.
- Component class 3: used for “non fail-safe” mechanical components that link actuators and brakes to main structural components for the purpose of implementing non-redundant wind turbine protection functions. Component class 3 is considered to correspond to high consequence of failure.

The generic limit state equation (4.12) for DLC 1.1 and DLC 6.1 is used. The consequence of failure factor γ_C is introduced in the corresponding design equation:

$$\frac{z\,R_k}{\gamma_M} - \gamma_C \gamma_f L_k \geq 0 \qquad (4.21)$$

where

R_k	characteristic value of load bearing capacity
L_k	characteristic value of variable load
γ_M	partial safety factor for load bearing capacity
γ_f	partial safety factor for load effect = 1.35

Table 4.10 shows the influence of γ_C on the reliability index β and annual probability of failure P_f for DLC 1.1 for the coefficient of variation for strength parameter V_R=0.05 and the coefficient of variation for model uncertainty for resistance model V_δ=0.15. Also shown is the ratio between the failure probability for a given γ_C and for γ_C =1.0.

Table 4.10. Reliability index, annual probability of failure and $P_f / P_f(\gamma_c = 1.0)$ as function of γ_C for DLC 1.1.

γ_C	β	P_f	$P_f / P_f(\gamma_c = 1.0)$
0.9	2.82	2.4 10^{-3}	0.3
1.0	3.22	6.5 10^{-4}	1.0
1.1	3.58	1.8 10^{-4}	3.7
1.2	3.90	4.7 10^{-5}	13.7
1.3	4.21	1.3 10^{-5}	50.0

Table 4.11 shows the influence of γ_C on reliability index β and annual probability of failure P_f for DLC 6.1 for V_R=0.05 and V_δ=0.15. Also shown is the ratio between the failure probability for a given γ_C and for γ_C =1.0.

Table 4.11. Reliability index, annual probability of failure and $P_f / P_f(\gamma_c = 1.0)$ as function of γ_C for DLC 6.1.

γ_C	β	P_f	$P_f / P_f(\gamma_c = 1.0)$
0.9	3.04	1.2 10^{-3}	0.4
1.0	3.34	4.2 10^{-4}	1.0
1.1	3.61	1.6 10^{-4}	2.7
1.2	3.85	6.0$10^{-5}$	7.0
1.3	4.07	2.4 10^{-5}	17.9

Table 4.12 shows the influence of γ_C on the reliability index β and the annual probability of failure P_f for fatigue using the stochastic model in section 4.3. γ_C is multiplied to the partial safety factor for

fatigue load, γ_f. Also shown is the ratio between the failure probability for a given γ_C and for γ_C =1.0.

Table 4.12. Reliability index, annual probability of failure and $P_f / P_f(\gamma_c = 1.0)$ as function of γ_C for fatigue.

γ_C	β	P_f	$P_f / P_f(\gamma_c = 1.0)$
0.9	2.95	1.6 10^{-3}	0,3
1.0	3.29	4.9 10^{-4}	1.0
1.1	3.64	1.4 10^{-4}	3.6
1.2	3.98	3.4 10^{-5}	14.4
1.3	4.32	7.9 10^{-6}	62.6

The results in tables 4.10-4.12 indicates that a consequence of failure factor γ_C multiplied to the load partial safety factor can be used with the following values for different consequence / component classes:

Component class 1 - low consequence: γ_C =0.9
Component class 2 - middle consequence: γ_C =1.0
Component class 3 - high consequence: γ_C =1.2

corresponding to a difference in probability of failure equal to a factor 10 between ‘low’ and ‘middle’ and between ‘middle’ and ‘high’.

4.6 Load partial safety factor calibration for typhoon conditions

For reliability analysis of wind turbine components, typically a COV of 20 % for extreme wind induced pressures is used. While this value is accepted as representative for a majority of sites, certain locations are characterized by much higher uncertainty levels, such as those exposed to tropical cyclones and hurricanes. In this section load partial safety factors are calibrated for wind pressures with higher uncertainties such that the minimum acceptable reliability level is achieved.

The limit state equation used for the analysis is shown in expression

$$g(X) = z\,\delta\,R - X_{dyn} X_{\exp} X_{aero} X_{str} Q \tag{4.22}$$

where

z	design parameter, e.g. cross-sectional area
δ	model uncertainty for resistance model
R	uncertainty in dominating strength parameter in resistance model
X_{dyn}	uncertainty related to modeling of the dynamic response, including uncertainty in damping ratios and eigenfrequencies
X_{exp}	uncertainty related to the modeling of the exposure (site assessment) - such as the terrain roughness and the landscape topography
X_{aero}	uncertainty in assessment of lift and drag coefficients and additionally utilization of BEM, dynamic stall models, etc.
X_{str}	uncertainty related to the computation of the load-effects given external load
Q	uncertainty related to the load-effect given faults

The resistance model includes both model and inherent physical uncertainties related to the strength parameter, while the load model contains the wind pressure, together with a set of general uncertainties. The properties of the *X* variables are shown in Table 4.13.

Table 4.13. Stochastic model for model uncertainties.

Variable	Distribution	Mean	COV	Quantile
X_{dyn}	Lognormal	1.00	0.05	Mean
X_{exp}	Lognormal	1.00	0.15	Mean
X_{aero}	Gumbel	1.00	0.10	Mean
X_{str}	Lognormal	1.00	0.03	Mean

The cross section parameter *z* is determined by using the general design equation as shown in the following expression,

$$\frac{z\,R_k}{\gamma_M} - \gamma_f Q_k \geq 0 \qquad (4.23)$$

where

z	design parameter
R_k	characteristic value of load bearing capacity taken as the 5% quantile
Q_k	characteristic value of variable load taken as the 98% quantile in the distribution function for the annual maximum load
γ_M	partial safety factor for load bearing capacity
γ_f	partial safety factor for load effect

Using a coefficient of variation for the annual maximum wind pressure, V_Q equal to 0.20 together with the material partial safety factors in section 4.2 the reliability levels in Table 4.14 are obtained. The applied safety factors are shown in Table 4.15.

Table 4.14. Annual reliability indices with V_Q = 0.20.

	V_R=0.05	V_R=0.10
V_δ=0.0	3.33	3.46
V_δ=0.05	3.41	3.55
V_δ=0.10	3.41	3.54

Table 4.15. Partial safety factors for V_Q=20%.

	V_R=0.05	V_R=0.10
V_δ=0.0	1.15	1.35
V_δ=0.05	1.20	1.35
V_δ=0.10	1.25	1.35

At typhoon prone areas V_Q is larger than 20% and can be up to 60%. Table 4.16-4.23 shows load partial safety factors, γ_f calibrated to give the same reliability level as obtained for the using V_Q = 20%.

Table 4.16. Partial safety factor γ_f for V_Q=25%.

	γ_M=0.05	γ_M=0.10
V_δ=0.0	1.39	1.40
V_δ=0.05	1.39	1.40
V_δ=0.10	1.38	1.39

Table 4.17. Partial safety factor γ_f for V_Q=30%.

	V_R=0.05	V_R=0.10
V_δ=0.0	1.42	1.43
V_δ=0.05	1.42	1.43
V_δ=0.10	1.41	1.42

Table 4.18. Partial safety factor γ_f for V_Q=35%.

	γ_M=0.05	γ_M=0.10
V_δ=0.0	1.45	1.46
V_δ=0.05	1.45	1.46
V_δ=0.10	1.44	1.45

Table 4.19. Partial safety factor γ_f for V_Q=40%.

	V_R=0.05	V_R=0.10
V_δ=0.0	1.47	1.49
V_δ=0.05	1.48	1.49
V_δ=0.10	1.47	1.48

Table 4.20. Partial safety factor γ_f for V_Q=45%.

	γ_M=0.05	γ_M=0.10
V_δ=0.0	1.50	1.51
V_δ=0.05	1.51	1.52
V_δ=0.10	1.49	1.51

Table 4.21. Partial safety factor γ_f for V_Q=50%.

	V_R=0.05	V_R=0.10
V_δ=0.0	1.52	1.53
V_δ=0.05	1.53	1.54
V_δ=0.10	1.51	1.53

Table 4.22. Partial safety factor γ_f for V_Q=55%.

	γ_M=0.05	γ_M=0.10
V_δ=0.0	1.54	1.55
V_δ=0.05	1.55	1.56
V_δ=0.10	1.53	1.55

Table 4.23. Partial safety factor γ_f for V_Q=60%.

	V_R =0.05	V_R =0.10
V_δ =0.0	1.56	1.57
V_δ =0.05	1.57	1.58
V_δ =0.10	1.55	1.57

The increase of the load partial safety factor γ_f as function of V_Q is shown in Figure 4.4.

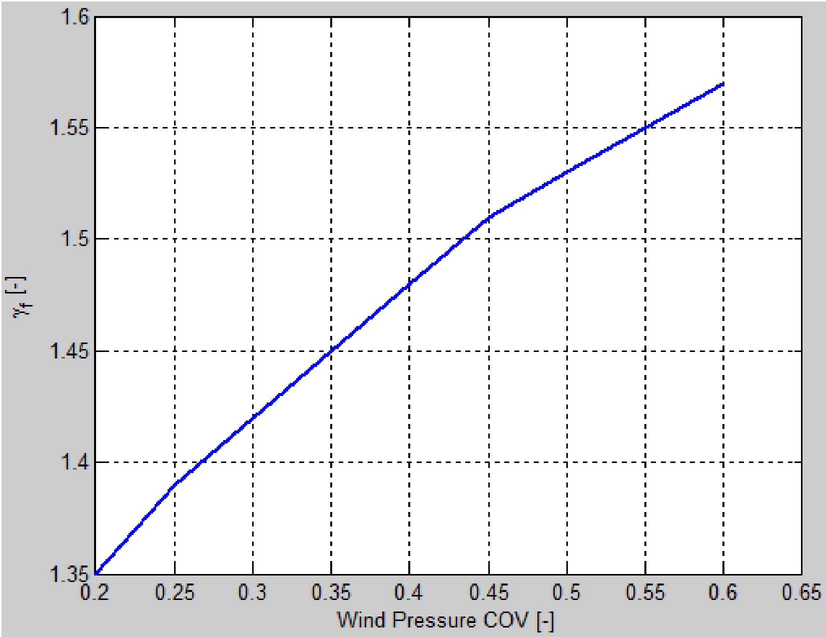

Figure 4.4. Load partial safety factor γ_f as function of V_Q for V_δ =0.05 and V_R =0.05.

The load partial safety factors for DLC 6.1 and DLC 6.2 are derived by assuming that the coefficient of variation of the annual maximum wind speed, COV, is 15-20%. The above calculations are performed for COV of the annual maximum wind speed in the interval from 10% to 30% (corresponding to COV for the annual maximum wind pressure from 20% to 60%). If COV of the annual maximum wind pressure is larger than 15%, the above results indicate to increase the load partial safety factor at least by a factor varying linearly from 1.0 at COV of the annual maximum wind speed ≤ 15% to 1.15 at COV = 30% (or equivalently from 1.0 at COV of the annual maximum wind pressure ≤ 30% to 1.15 at COV = 60%).

It is noted that the COV of the annual maximum wind speed can approximately be obtained assuming a Gumbel distribution and assuming that e.g. 50 and 100 year return values of the wind speed, U_{50} and U_{100} are available. The parameters α and β in the Gumbel distribution are obtained from:

$$\alpha = \frac{U_{100} - U_{50}}{p_{100} - p_{50}} \qquad \beta = U_{50} + \alpha\, p_{50} \qquad (4.24)$$

with $p_{100} = \ln\left(-\ln\left(1 - \frac{1}{100}\right)\right)$ and $p_{50} = \ln\left(-\ln\left(1 - \frac{1}{50}\right)\right)$

COV for the annual maximum wind speed is then determined from:

$$COV = \frac{\sigma}{\mu} = \frac{\pi}{\sqrt{6}} \frac{1}{\frac{\beta}{\alpha} + 0{,}5772} \qquad (4.25)$$

5 MODIFICATION OF PARTIAL SAFETY FACTORS WHEN 'BETTER' MODELS/INFORMATION ARE AVAILABLE

Table 5.1-5.6 give some general examples of sources of uncertainties concerning loads, which are dependent on the "quality" of the models or information available. The "quality" of the model/information would affect the definition of the stochastic variables (mainly the COV) and, hence, the safety factor can be re-calibrated based on it as described in some cases in section 4.

For uncertainty modelling of wind, as an example, two additional columns are included, showing a Worst case and a Best case scenario. The worst scenario implies larger uncertainties, so the corresponding stochastic variables would have a larger COV, leading to larger safety factors. The best scenario implies less uncertainties, so the corresponding stochastic variables would have a smaller COV (assuming same distribution), leading to smaller safety factors. Of course, in practice, there could be intermediate scenarios.

Table 5.1 Uncertainty related to modelling of wind.

Uncertainty sources	Worst scenario	Best scenario
Intra-annual variations (seasonal variations) and inter-annual variations, directional variations	Data not covering all seasons and directions	Data from all seasons and directions and along several years
Quality of anemometers	Non Calibrated, standard cup anemometer	Calibrated 1st class or sonic anemometers
Quality of met mast mounting	Anemos at mid height, with bad mounting	Anemos at the top, with good mounting
Number of measurements at met mast	Less than 1 year	Several years
MCP	No MCP applied	MCP with more than 30 years at reference mast
Horizontal extrapolation	Curves lines more than 20m. Unkown Roughness Complex terrain	Curves lines less than 10m. Low Roughness Flat terrain
Vertical extrapolation	Simple exponential model. Measurements below hub height.	Measurements at several heights within rotor size
Wind field and turbulence model	Use a basic standard wind model (Kaimal, Mann)	Detailed characterization of spectra and coherence, based on measurements
Wake models	Effective turbulence model	DWM or CFD analysis
Determination of Long-term wind speeds	EWS2 method (V_{ref}=5·V_{ave})	Extrapolation based on several years of measurements

Table 5.2 Uncertainty related to modelling of aerodynamics.

Uncertainty sources	Worst scenario	Best scenario
Blade geometric properties (roughness, airfoil shape)	Poor manufacturing quality control	Very good manufacturing quality control
Aerodynamic coefficients	Based on simple fluid dynamics formulation	Based on measurements at different Re and several aoa.
Rotor aerodynamic models	Simple BEM model	Complete CFD

Table 5.3 Uncertainty related to modelling of structural dynamics.

Uncertainty sources	Worst scenario	Best scenario
Structural properties (masses, stiffness's, frequencies…)	Data estimated from design. Poor manufacturing quality control	Real data measured. Very good manufacturing quality control
Structural models (degrees of freedom, coupling of modes…)	Modal synthesis with simple beam models, few dof	Complete 3D FEM

Table 5.4 Uncertainty related to modelling of wind turbine actuation systems.

Uncertainty sources	Worst scenario	Best scenario
Control parameters	Predefined parameters, from simulation environment	Parameters as in field
Control algorithms	Simplified algorithms, similar to PLC (but not the same)	Algorithms exactly as in field
Actuation systems models	1st order system	Complete validated system model
Actuation systems properties	Estimated from design	Measured on real equipment

Table 5.5 Uncertainty related to modelling of fatigue.

Uncertainty sources	Worst scenario	Best scenario
Number and chronology of events (Cycle history)	Consider estimated number of events and chronology	Consider actual number of events and chronology
Simplified equivalent damage loads (e.g.- Miner's rule)	Consider only Damage Equivalent Load, using Miner's Rule	Consider full time series for damage evaluation

Table 5.6 Uncertainty related to modelling of extreme load response.

Uncertainty sources	Worst scenario	Best scenario
Probability of load cases	Probability of wind, turbine response (e.g.- alignment, azimuth) and eventual failures	Use actual data about recurrence of events
Load response distribution	Load response estimated from characteristic load and some assumptions (extrapolation model)	Actual distribution obtained from complete 50 year simulation

In section 4.3 and Annex C calibration of partial safety factors for fatigue is described. The fatigue load partial safety factor is dependent on the uncertainty of the fatigue stresses which is assumed to have two contributions:

- Uncertainty related to estimation of the fatigue stress given the fatigue load – modelled by a stochastic variable X_{wind} with coefficient of variation COV_{Wind}
- Uncertainty related to the fatigue load – modelled by a stochastic variable X_{SCF} with coefficient of variation COV_{SCF}

The total coefficient of variation of the fatigue load becomes $COV_{load} = \sqrt{COV_{Wind}^2 + COV_{SCF}^2}$.

Table 5.7. Examples of COV_{Wind}.

COV_{Wind}	Uncertainty is assessment of fatigue wind load
0.10-0.15	*Site assessment:* • More than 2 years of climatic data, corrected with MCP techniques. • Wind measurements above and below wind turbine hub height. • Flat terrain with low roughness *Dynamic response:* • Structural dynamic effects through modal analysis, with at least 4 modes considered for blade and tower. • Mass and stiffness properties defined with FEM and validated with real scale specimens. • Eigenvalues and damping validated with real scale tests. *Aerodynamic coefficients:* • Airfoil data experimentally validated in wind tunnel at different Re numbers • Airfoil data including 3D effects • Attached flow in all operating regimes • BEM, including Dynamic stall and Tip and hub loss included • Dynamic wake inflow model • Quality control of shape of manufactured blades
0.15-0.20	*Site assessment:* • Minimum 1 year of climatic data. • Wind measurements at hub height and below. • Non-complex site with medium roughness. *Dynamic response:* • Structural dynamic effects through modal analysis, with 2 modes considered for blade and tower. • Mass and stiffness properties defined with FEM but not validated with real scale specimens. • Eigenvalues and damping not validated with real scale tests. *Aerodynamic coefficients:* • Airfoil data based on CFD, but not measured in wind tunnel. • 3D effects not included in airfoil data • Attached flow in all operating regimes • BEM, but not including dynamic stall effects nor tip and hub losses • Static wake inflow model
0.20-0.25	*Site assessment:* • Less than 1 year of data, not corrected with MCP techniques Wind measurements below hub height. • Complex terrain. *Dynamic response:* • Structural dynamic effects not considered *Aerodynamic coefficients:* • Airfoil data based on similar airfoils or for a single Re number. • 3D effects not included in airfoil data • Stall flow in relevant operating regimes • BEM, but not including dynamic stall effects nor tip and hub losses • No model for wake effects • Dirt and erosion on blades

The uncertainties related to wind load assessment, X_{wind} in relation to fatigue can be divided in:

- modeling of the exposure (site assessment) – incl. assessment of terrain roughness, landscape topography, annual mean wind speed, turbulence intensity, density, shear and veer
- modeling of the dynamic response, including uncertainty in damping ratios and eigenfrequencies
- assessment of lift and drag coefficients and additionally utilization of BEM, dynamic stall models, etc.

Table 5.7 shows examples of how to model the uncertainty related to X_{wind}. The contribution of the different sources of uncertainties to the total X_{wind} could be evaluated with sensitivity analysis. X_{wind} could then be defined as a response surface dependent on several stochastic variables, each of them accounting for a specific effect described in Table 5.7.

Table 5.8 shows examples of how to model the uncertainty related to X_{SCF} (partly based on Sørensen [16]). Five values of COV_{SCF} are used to model different levels of analysis and complexity.

Table 5.8. Examples of COV_{SCF}.

COV_{SCF}	Fatigue critical detail
0.00	Statically determinate systems with simple fatigue critical details (e.g. girth welds) where FEM analyses are performed
0.05	Statically determinate systems with complex fatigue critical details (e.g. multi-planar joints) where FEM analyses are performed
0.10	Statically in-determinate systems with complex fatigue critical details (e.g. doubler plates) where FEM analyses are performed
0.15	2 dimensional tubular joints using SCF parametric equations
0.20	Tubular joints in structures where tubular stiffness is modeled by Local Joint Flexibility (LJF) models and SCF parametric equations are used

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ANNEX A. RELIABILITY AND PARTIAL SAFETY FACTORS - TOWER BUCKLING

This section describes a detailed reliability assessment for the wind turbine tower related to the buckling failure mode. In the wind turbine standard IEC 61400-1 [1] detailed requirements related to the loading conditions are described. However, only general requirements with respect to the materials and design are given for which reason the IEC-standard often is used together with a 'recognized' standard for the materials. For the case considered in this section it is assumed that the wind turbine tower is design according to Eurocode 3: Design of Steel Structures, Part 1-6: Strength and Stability of Shell Structures [2].

A1 Wind Turbine Tower

The wind turbine considered as a representative example in the present section is the NREL 5MW reference wind turbine which is pitch controlled [3]. The tubular steel tower has a height of 87.6m which in the present study is assumed divided into three sections each having a length on 29.2m.

The tower diameter is according to NREL 6.00m at the tower bottom and 3.87m at the tower top. Similarly, the tower thickness is 27mm at the bottom and 19mm at the top. In the present annex a section of the tower is considered that has a constant diameter on 6.00m and the thickness is adjusted in order that the design meets the requirements in Eurocode 3.

It is assumed that no stiffeners are inserted in the tower. The boundary conditions for both ends of a tower section are assumed to be BC2f 'pinned' (radially restrained, meridionally free, rotation free). The boundary conditions are shown on figure A1.

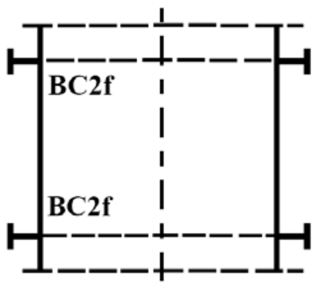

Figure A1: Boundary conditions for wind turbine tower section.

A2 Load Modelling

The loads correspond to normal operation of the wind turbine (DLC 1.1 in IEC 61400-1) and contain the following two loads:

- Gravity loading: Estimated from weight of wind turbine above mudline.
- Wind loading: Estimated from mudline bending moment (fore-after)

The torsional moment and side-to-side moment in the tower and the pressure on the tower from wind loading is therefore not taken into account in the reliability assessment. However, these three load-effects will in general be small.

The uncertainty related to load-effects on wind turbine components has been considered in [4] where the physical, model and statistical uncertainties are split into their respective components. It is assumed that the load-effect on a cross-section in the wind turbine tower can be modelled by:

$$F = X_{dyn} X_{\exp} X_{aero} X_{str} L + G \qquad \text{(A1)}$$

where X_{dyn} is the uncertainty related to modelling of the dynamic response for the wind turbine, including uncertainty in damping ratios and eigenfrequencies. X_{exp} is the uncertainty related to the modelling of the exposure (site assessment) - such as the terrain roughness and the landscape topography. X_{aero} is related to the uncertainty in assessment of lift and drag coefficients. X_{str} accounts for the uncertainty related to the computation of the load-effect. The physical uncertainty of the extreme load-effect due to wind loads is modelled by the stochastic variable L. The stochastic variable G models the uncertainty related to load-effects from gravity loading (weight of wind turbine components). The uncertainties will in general be dependent on the considered load case.

Stochastic variables for the uncertainties are adopted from [4] and shown in table A1.

Table A1. Stochastic models for physical, model and statistical uncertainty on load-effects.

Variable	Distribution	Mean	COV
X_{dyn}	Lognormal	1.00	0.05
X_{exp}	Lognormal	1.00	0.15
X_{aero}	Gumbel	1.00	0.10
X_{str}	Lognormal	1.00	0.03
L	Weibull	-	0.15
G	Normal	-	0.05

The gravity loading is assumed Normal distributed with a coefficient of variation on 5% based on engineering judgment and recommendations in JCSS [7]. A stochastic model for the physical uncertainty on the extreme load-effect L during normal operation is estimated by statistical extrapolation for the tower mudline bending moment. The distribution function for the annual maximum load-effect is shown in figure A2 together with a Weibull distribution with a coefficient of variation on 15%. A reasonable agreement between the two distributions is observed in the tail region.

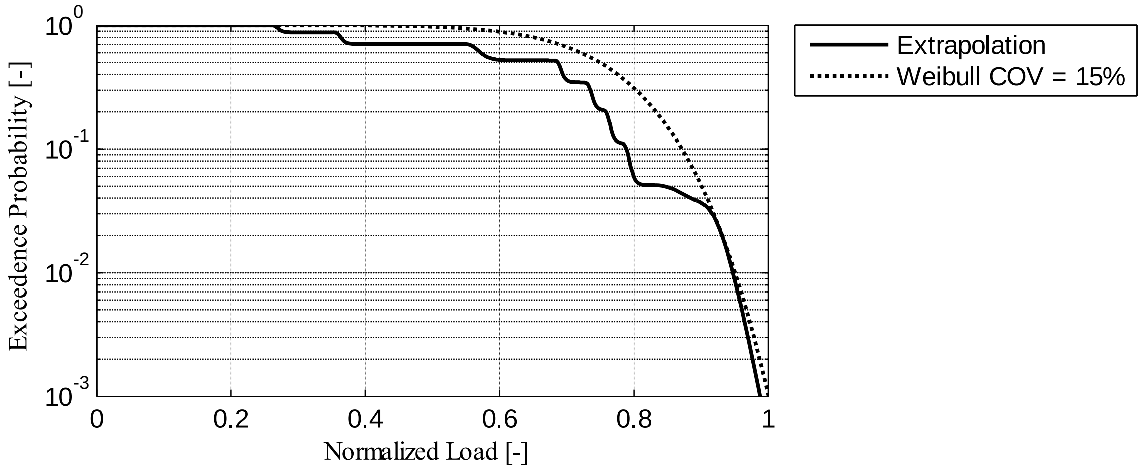

Figure A2: Extrapolated mudline bending moment and Weibull distribution (distribution function for annual maximum).

Table A2. Characteristic values for gravity and wind loading.

Load	Load case	Characteristic load
Gravity	Axial force $F_{x,k}$	6849 kN
Wind	Mudline bending moment $M_{x,k}$	112091 kNm

The characteristic value for the load-effect due to gravity loading is assumed to be the mean value. The characteristic value for the load-effect due to wind loading is estimated as the 98% quantile in the distribution function for the annual maximum loading corresponding to a return period of 50 years. The characteristic loads are shown in table A2.

The partial safety factors for loading are for DLC 1.1 in IEC 61400-1, [1]:

$$\gamma_{f,Gravity} = 1.10 \tag{A2}$$

$$\gamma_{f,Wind} = 1.25 \tag{A3}$$

The stresses in the tower shell are calculated according to membrane theory based on unstiffened cylindrical shells with a constant wall-thickness and shell-radius. The design values of the axial stresses in the tower are calculated from the design axial force, $F_{x,d}$ and design bending moment, $M_{x,d}$ according to:

$$\sigma_{x,Nd} = \frac{-F_{x,d}}{2\pi rt} \tag{A4}$$

$$\sigma_{x,Md} = \pm\frac{M_{x,d}}{\pi r^2 t} \tag{A5}$$

where r is the radius of the tower and t is the thickness of the tower. The design value of the axial stress is then obtained by:

$$\sigma_{x,Ed} = \sigma_{x,Nd} + \sigma_{x,Md} \tag{A6}$$

A3 Deterministic Design Procedure according to Eurocode 3 part 1-6

The design procedure based on membrane theory for buckling of shell structures in Eurocode 3 is in the following briefly outlined.

The dimensionless length parameter ω is given by:

$$\omega = \frac{l}{\sqrt{rt}} \tag{A7}$$

The elastic critical buckling stress $\sigma_{x,Rcr}$ is calculated from:

$$\sigma_{x,Rcr} = 0.605 E C_x \frac{t}{r} \tag{A8}$$

where the factor C_x is determined from table A3.

Table A3. Factor C_x dependent on dimensional length parameter ω.

Cylinder length	Interval for ω	Factor C_x
Short	$\omega < 1.7$	$C_x = 1.36 - \frac{1.83}{\omega} + \frac{2.07}{\omega^2}$
Medium	$1.7 \le \omega \le 0.5\frac{r}{t}$	$C_x = 1.0$
Long	$\omega > 0.5\frac{r}{t}$	$C_x = \max\left\{0.60;\ 1 + \frac{0.2}{C_{xb}}\left[1 - 2\omega\frac{t}{r}\right]\right\}$

Table A4. Parameter C_{xb} dependent on boundary conditions.

Case	Cylinder end	Boundary Condition	C_{xb}
1	end 1	BC 1	6
	end 2	BC 1	
2	end 1	BC 1	3
	end 2	BC 2	

3	end 1	BC 2	1
	end 2	BC 2	

The parameter C_{xb} is dependent on the boundary conditions as shown in table A4. The boundary conditions used in the present study corresponds to case 3 in the table.

The relative slenderness for normal stresses λ_x is given by:

$$\lambda_x = \sqrt{\frac{f_{yk}}{\sigma_{x,Rcr}}} \quad \text{(A9)}$$

The elastic imperfection factor α_x is given by:

$$\alpha_x = \frac{0.62}{1+1.91\left(\frac{1}{Q}\sqrt{\frac{r}{t}}\right)^{1.44}} \quad \text{(A10)}$$

The parameter Q is dependent on the fabrication tolerances as specified in table A5.

Table A5. Parameter Q dependent on fabrication tolerances / quality class.

Fabrication tolerance quality class	Description	Q
Class A	Excellent	40
Class B	High	25
Class C	Normal	16

The squash limit slenderness for normal stress λ_{x0}, the plastic range factor β_x and the interaction exponent η_x are given by:

$$\lambda_{x0} = 0.20 \quad \text{(A11)}$$

$$\beta_x = 0.60 \quad \text{(A12)}$$

$$\eta_x = 1.00 \quad \text{(A13)}$$

The plastic limit relative slenderness λ_{xp} is given by:

$$\lambda_{xp} = \sqrt{\frac{\alpha_x}{1-\beta_x}} \quad \text{(A14)}$$

The buckling reduction factor for normal stress χ_x is given by:

$$\chi_x = 1 \qquad \text{when} \qquad \lambda_x \le \lambda_{x0} \quad \text{(A15)}$$

$$\chi_x = 1-\beta_x\left(\frac{\lambda_x-\lambda_{x0}}{\lambda_{xp}-\lambda_{x0}}\right)^{\eta_x} \qquad \text{when} \qquad \lambda_{x0} < \lambda_x < \lambda_{xp} \quad \text{(A16)}$$

$$\chi_x = \frac{\alpha_x}{\lambda_x^2} \qquad \text{when} \qquad \lambda_{xp} \le \lambda_x \quad \text{(A17)}$$

The design value for the buckling resistance for normal stress $\sigma_{x,Rd}$ is obtained from:

$$\sigma_{x,Rd} = \frac{1}{\gamma_n}\frac{\chi_x f_{yk}}{\gamma_{M1}} \quad \text{(A18)}$$

which should be larger or equal to the stresses determined for the design load:

$$\sigma_{x,Ed} \le \sigma_{x,Rd} \quad \text{(A19)}$$

The partial safety factor γ_{M1} should according to Eurocode 3 part 1-6 not be smaller than:

$$\gamma_{M1} = 1.1 \quad \text{(A20)}$$

However, in IEC 61400-1 a minimum partial safety factor for failure due to buckling is specified:

$$\gamma_{M1} = 1.2 \quad \text{(A21)}$$

which also corresponds to the partial safety factor specified in the Danish National Annex to Eurocode 3 part 1-6. In the following the partial safety factor γ_{M1}=1.20 is used as reference.

The partial safety factor γ_n (consequences of failure) is according to IEC 61400-1 for 'non-fail safe' components equal to:

$$\gamma_n = 1.0 \tag{A22}$$

A4 Uncertainty Buckling Strength

The buckling strength of shells has been studied in several test programs where cylindrical shells are loaded in axial compression. In [5] test results from the literature are compared to the buckling curves used in Eurocode 3 as shown in figure A3. From the figure it is seen that the buckling reduction factor χ_x contains a significant uncertainty dependent on the relative slenderness λ_x. It is also seen from figure A3 that the buckling curves used in Eurocode 3 are not specified as mean curves and the bias introduced by using these buckling curves should therefore be taken into account in the reliability assessment.

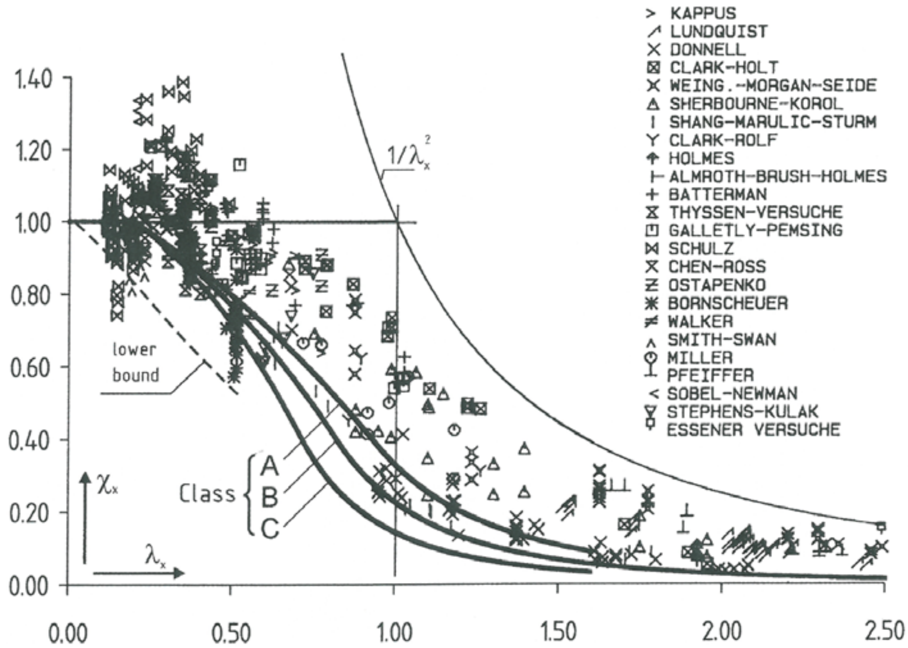

Figure A3: Axial compression cylinder tests compared to the Eurocode 3 buckling curves. [5]

In order to estimate the uncertainty and bias related to the buckling reduction factor χ_x the test results in figure A3 have been digitalized. The digitalized test data are shown in figure A4 along with the mean buckling curve estimated using the Maximum-Likelihood method. The parameters in the mean buckling curve are shown in equation (A23) to (A25). For λ_x = 0.51 many tests results from the same test series (publication) with low buckling strengths are observed, see figure A3. These test results fall significantly below the other test results for which reason they are not taken into account as shown in figure A4.

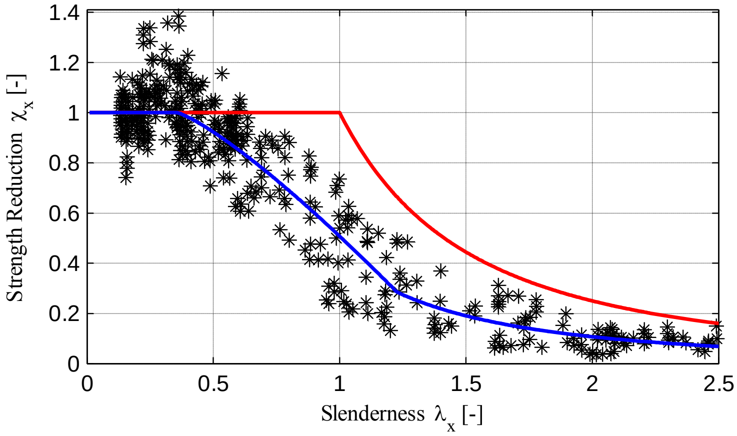

Figure A4: Digitalized axial compression cylinder tests with mean buckling curve (blue).

$$\chi_x = 1 \quad \text{when} \quad \lambda_x \le 0.36 \tag{A23}$$

$$\chi_x = 1 - 0.72\left(\frac{\lambda_x - 0.36}{1.23 - 0.36}\right)^{1.23} \quad \text{when} \quad 0.36 < \lambda_x < 1.23 \tag{A24}$$

$$\chi_x = \frac{0.43}{\lambda_x^2} \quad \text{when} \quad 1.23 \le \lambda_x \tag{A25}$$

The design of wind turbine towers normally corresponds to a relative slenderness in the range λ_x = 0.35 to 0.70. The uncertainty related to the buckling reduction factor is estimated using the method described in Eurocode 0 annex D [6]. In this approach the buckling reduction factor is assumed to follow a Lognormal distribution with mean value equal to the mean buckling curve. The coefficient of variation for the buckling reduction factor is estimated to 13% in the considered interval. In the present annex it assumed that the coefficient of variation is constant and independent of the slenderness. This is a reasonable approximation for the considered interval. However for larger slenderness it seems that the standard deviation is independent of the slenderness rather than the coefficient of variation, see figure A4.

In a reliability assessment also material uncertainty related to the yield stress f_y and modulus of elasticity E has to be taken into account. For the geometry, the uncertainty related to the shell radius r and shell thickness t is considered. However, the shell length l is modelled as a deterministic parameter. The stochastic models used for the materials and geometry are given in table A6 and in general based on recommendations from Joint Committee on Structural Safety [7].

Table A6. Stochastic models for material properties and geometry.

Variable	Distribution	Mean	COV
f_y	Lognormal	264 MPa	0.07
E	Lognormal	210000 MPa	0.03
R	Normal	3000 mm	0.01
T	Normal	-	0.01
χ_x	Lognormal	-	0.13

A5 Reliability assessment

The reliability of the wind turbine tower with respect to buckling is in the following estimated using the First Order Reliability Method (FORM). The wind turbine tower is assumed designed according to Eurocode 3 and IEC 61400-1 as described in the previous sections. It is assumed that the fabrication tolerances / quality class corresponds to high (class B in table A5). The limit state equation is formulated based on the stochastic models in table A1 and A6 along with the mean buckling curve.

Table A7. Reliability for buckling of wind turbine tower.

Case	Partial Safety Factors	Annual Reliability Index β	Annual Probability of Failure P_F
1	γ_{M1}=1.20; $\gamma_{f,Wind}$=1.25	3.39	$3.48 \cdot 10^{-4}$
2	γ_{M1}=1.10; $\gamma_{f,Wind}$=1.25	3.08	$1.04 \cdot 10^{-3}$
3	γ_{M1}=1.20; $\gamma_{f,Wind}$=1.35	3.65	$1.33 \cdot 10^{-4}$
4	γ_{M1}=1.10; $\gamma_{f,Wind}$=1.35	3.34	$4.25 \cdot 10^{-4}$

It is assumed that the wind turbine tower is designed to the limit with respect to the failure mode buckling. Wind turbines are normally not designed to a specific site but for predefined wind turbine classes. This implies that only a limited number of wind turbines are exploited to the limit given that the wind conditions at the specific site are determined properly. Due to the division into wind turbine classes the reliability estimated in the following should be regarded as a minimum reliability level rather than an average reliability level.

The estimated reliability is shown in table A7 for different values of the partial safety factors. For case 1 the partial safety factors corresponds to the minimum values specified in IEC 61400-1. For case 2 are the material partial safety factor reduced to γ_{M1}=1.1 corresponding to the minimum value specified in Eurocode 3 part 1-6. In case 3 is the partial safety factor for the wind loading increased to $\gamma_{f,Wind}$=1.35. This corresponds to the partial safety factor used in several design load cases in IEC 61400-1. However, in the present section design load case 1.1 is used as reference.

The target annual reliability index on $\beta = 3.3$ corresponding to an annual probability of failure equal to $P_F = 5 \cdot 10^{-4}$ is corresponding approximately to case 1 and 4 in table A7. The partial safety factor γ_{M1}=1.1 in Eurocode can therefore be used in IEC 61400-1 together with a load partial safety factor on $\gamma_{f,Wind}$=1.35. The reduced partial safety factor is accepted for design according to Eurocode 3 part 1-6 since this this standard contains hidden safety / bias in the buckling curves. Other standards do not necessary have this hidden safety / bias for which reason the lower partly safety factor cannot be generally adopted. It is also noted that the γ_{M1}=1.2 specified in IEC 61400-1 is used for other components and materials than considered in this annex.

In the present example only one failure mode is considered and no system effects are taken into account – but this corresponds to the basic approach in structural codes based on checking single failure modes one at the time.

The sensitivities with respect to the different stochastic variables are assessed from the α^2-vector determined by FORM, see table A8. The most important stochastic variable is the model uncertainty on exposure X_{exp} followed by the uncertainty related to the buckling reduction factor χ_x.

Table A8. Sensitivities for the stochastic variables (α^2-vector) for case 1.

Variable	α^2-vector	Variable	α^2-vector
f_y	0.05	X_{dyn}	0.03
E	0.00	X_{exp}	0.28
R	0.00	X_{aero}	0.25

T	0.00	X_{str}	0.01
χ_x	0.24	*L*	0.12
		G	0.00

A6 Further Investigations of the reliability level for buckling

In the present section the reliability is estimated for DLC 6.1 in IEC 61400-1: Parked wind turbine and extreme mean wind speed. The deterministic and stochastic model for the load bearing capacity of the cylinder with respect to buckling is the same as used above.

The design load–effect on the wind turbine is given by:

$$F_d = \gamma_{f,Wind} P_c \left(1 + 2k_p I_c\right) + \gamma_{f,Gravity} G_c \qquad \text{(A26)}$$

where $\gamma_{f,Wind}$ = 1.35 and $\gamma_{f,Gravity}$ = 1.1 according to IEC 61400-1. The characteristic wind pressure P_c corresponds to a return period on 50years. The characteristic turbulence intensity I_c corresponds to the mean value according to the extreme wind model in IEC 61400-1. The characteristic gravity load G_c corresponds also to the mean value. The peak-factor k_p is assumed to 3.5.

The stochastic model for the load-effect is defined as:

$$F = P\left(1 + 2k_p I \, X_{dyn}\right) X_{\exp} X_{aero} X_{str} + G \qquad \text{(A27)}$$

where the stochastic models for *X* and *G* are defined in table A1. The wind pressure *P* is assumed Gumbel distributed with a COV on 0.20. The turbulence intensity is assumed LogNormal distributed with a mean value on 0.11 and a COV on 0.05.

The estimated reliability level is given in table A9. From the table it is seen that the reliability level for this load case is slightly higher than the reliability levels estimated for DLC 1.1.

Table A.9. Reliability for buckling of wind turbine tower and DLC 6.1 in IEC 61400-1.

Case	Partial Safety Factors	Annual Reliability Index β	Annual Probability of Failure P_F
1	γ_{M1}=1.10; $\gamma_{f,Wind}$=1.35	3.47	$2.60 \cdot 10^{-4}$
2	γ_{M1}=1.20; $\gamma_{f,Wind}$=1.35	3.71	$1.04 \cdot 10^{-4}$

A7 Variation of tower geometry

In order to investigate the partial safety factor for different geometries of the wind turbine tower six different combinations of radius and thickness have been investigated. The tower is in the following only assumed to be loaded by a bending moment from the wind and the gravity loading is neglected. In table A10 are the results shown.

Table A.10. Reliability for buckling of wind turbine tower for different geometries. DLC 1.1 in IEC 61400-1. $\gamma_{f,Wind}$ = 1.35 and γ_n = 1.00.

Case	1	2	3	4	5	6
Length l [m]	30	30	30	30	30	30
Radius r [m]	3	3	2	2	1	1
Thickness t [mm]	30	20	30	20	20	10
Test: λ_{xk}	0.481	0.564	0.453	0.555	0.393	0.555
Test: χ_{xk}	0.935	0.878	0.952	0.884	0.986	0.884

Test: $f_{x,Rk}$ [MPa]	219.76	206.41	223.80	207.83	231.67	207.83
EN 1993: λ_{xk}	0.481	0.564	0.453	0.555	0.393	0.555
EN 1993: χ_{xk}	0.793	0.713	0.823	0.738	0.870	0.738
EN 1993: $\sigma_{x,Rk}$ [MPa]	186.29	167.50	193.43	173.39	204.41	173.39
EN 1993: $\eta_{EN1990,char}=\sigma_{x,Rk}/f_{x,Rk}$	0.847	0.812	0.864	0.834	0.883	0.834
γ_{M1} (COV_{XR}=0.13) Full Prob.	1.10	1.05	1.11	1.08	1.13	1.08
γ_{M1} (COV_{XR}=0.13) Simpel table	1.08	1.03	1.10	1.06	1.12	1.06

Table A10 shows that the bias in the buckling curves in Eurocode on average is 0.85. The partial safety factors is on average estimated to γ_{M1} = 1.09. The simple methods where the partial safety factor is estimated from the COV on X_R and R estimates the partial safety factor appropriately when the bias is taken into account.

A8 Reliability for yielding of steel

The reliability is in the following estimated for yielding of steel in wind turbine towers. The design resistance for the structure is given by:

$$R_d = z \frac{1}{\gamma_n} \frac{f_{yc}}{\gamma_m} \tag{A28}$$

where γ_n = 1.00 and γ_m = 1.10 are defined according to IEC 61400-1. The characteristic yield strength f_{yc} is defined as a 5% quantile. The design parameter z is scaled in order to obtain a structure which is designed to the limit.

The stochastic model for the resistance is defined as:

$$R = z X_R f_y \tag{A29}$$

where the stochastic model for the model uncertainty X_R is assumed Lognormal distributed with a COV on 0.05 and a characteristic value equal to unity. The yield strength is also assumed Lognormal distributed but with a COV on 0.07 according to JCCS. In the following are the reliability estimated for different load cases and values of the partial safety factor γ_m.

Table A.11. Reliability for yielding of wind turbine tower dependent on the amount of gravity loading. DLC 1.1 in IEC 61400-1. $\gamma_{f,Wind}$ = 1.35, $\gamma_{f,Gravity}$ = 1.1 and γ_n = 1.00.

α	0.00	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00
γ_m =1.0	2.12	2.66	2.97	3.04	3.00	2.94	2.89	2.84	2.79	2.75	2.72
γ_m =1.1	3.09	3.63	3.83	3.76	3.62	3.49	3.38	3.29	3.22	3.15	3.10
γ_m =1.2	3.98	4.51	4.59	4.37	4.15	3.96	3.81	3.69	3.60	3.51	3.45

Table A.12. Reliability for yielding of wind turbine tower dependent on the amount of gravity loading. DLC 6.1 in IEC 61400-1. $\gamma_{f,Wind}$ = 1.35, $\gamma_{f,Gravity}$ = 1.1 and γ_n = 1.00.

α	0.00	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00
γ_m =1.0	2.12	2.97	3.29	3.25	3.17	3.11	3.06	3.02	2.98	2.95	2.93
γ_m =1.1	3.09	3.92	3.96	3.76	3.61	3.50	3.41	3.35	3.29	3.25	3.21
γ_m =1.2	3.98	4.76	4.51	4.20	3.99	3.84	3.73	3.64	3.57	3.51	3.47

A9 Summary

This annex describes the design rules in Eurocode 3 part 1-6 for design for buckling of wind turbine towers. The design rules in the Eurocode are based on a large number of tests. A statistical analysis of the tests relevant for design of wind turbine towers shows that the coefficient of variation is 13% for the model uncertainty of the load bearing capacity. This large model uncertainty is 'taken care of' in EN 1991-1-6 by a partial safety factor on the load bearing capacity and by specifying a design equation which results in a characteristic load bearing capacity significantly lower than the mean load bearing capacity obtained from the tests. Using a stochastic model for the loads relevant for wind turbines a reliability analysis shows that using a material safety factor equal to 1.1 results in a reliability level consistent with the basic reliability requirements in IEC 61400-1 [1].

A10 References

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2. EN 1993-1-6. Design of steel structures – Part 1-6: Strength and stability of shell structures. *CEN* 2006.
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ANNEX B. RELIABILITY OF CONCRETE STRUCTURES FOR WIND TURBINES

B1 Introduction

In the present annex the reliability of a reinforced concrete beam and a short concrete column (without stability failure mode) is estimated using material partial safety factors from EN 1992 [1] and load partial safety factors from IEC 61400-1. The considered concrete elements are assumed representative for failure modes where the reinforcement strength and the concrete compression strength are dominating the uncertainty related to the load bearing capacity, respectively. Thus, failure modes for e.g. high-strength and pre-stressed concrete are not considered.

The reinforced concrete structures are shown on figure B1. The reinforcement in the concrete beam consists of 4x25mm bars placed 40mm from the bottom of the beam and for the column 4x16mm bars are used. Only the bending moment capacity of the beam and the compression strength for the column is studied in the present annex for which reason failures due to e.g. shear forces are not considered.

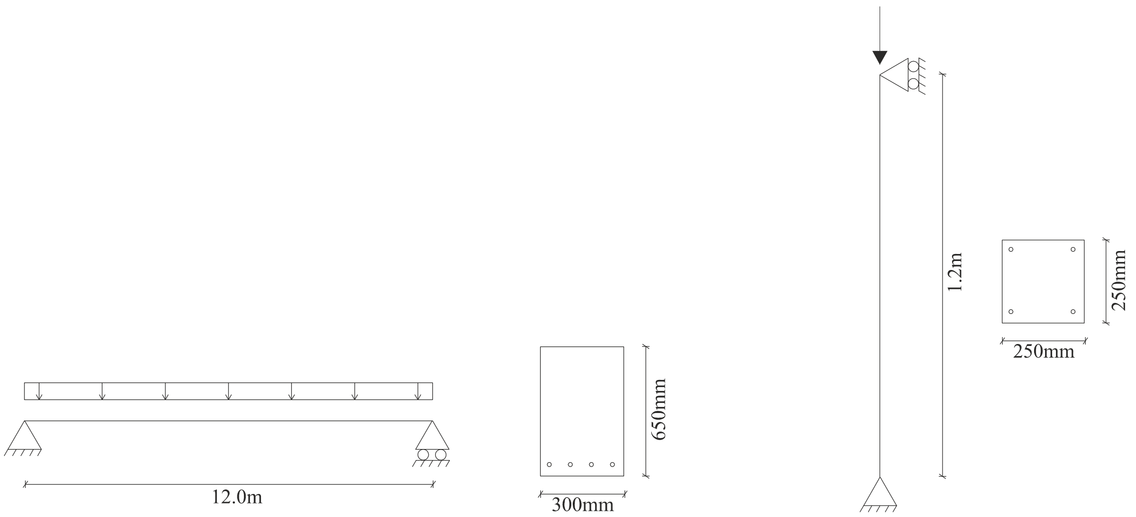

Figure B1. Left: Concrete beam. Right: Concrete Column.

The characteristic concrete and reinforcement strengths are specified as 5% quantiles according to EN 1992. The design values of the concrete strength f_{cd} and the reinforcement strength f_{yd} are given by:

$$f_{cd} = \frac{f_{ck}}{\gamma_C} \tag{B1}$$

$$f_{yd} = \frac{f_{yk}}{\gamma_S} \tag{B2}$$

where f_{ck} and f_{yk} are the characteristic value for the concrete and reinforcement strength, respectively. The partial safety factor for the concrete strength is according to EN 1992 equal to γ_C=1.50. The partial safety factor for the reinforcement strength is according to EN 1992 equal to γ_S=1.15.

In the present annex the characteristic concrete strength is assumed equal to f_{ck}=25MPa and the characteristic reinforcement strength is assumed equal to f_{yk}=550MPa. The modulus of elasticity for the reinforcement is assumed to be E_s=200GPa.

B2 Concrete Beam

The concrete beam is designed according to EN 1992 by first estimating the ratio of reinforcement ω:

$$\omega = \frac{A_s f_{yd}}{b\,d\,f_{cd}} \tag{B3}$$

where the area of the reinforcement is A_s=1963mm^2 and b is the width of the beam, see figure B1. The length d is defined as the distance from the reinforcement to the top of the beam (d=610mm).

The design bending moment capacity M_{Rd} of the beam is then given by:

$$M_{Rd} = \left(1 - \frac{1}{2}\omega\right)\omega b d^2 f_{cd} \tag{B4}$$

The design bending moment in the beam is in design load case 1.1 calculated by:

$$M_{Ed} = \frac{1}{8} z\left((1-\alpha)\gamma_{f,Gravity} G_c + \alpha\gamma_{f,Wind} Q_c\right)L^2 \tag{B5}$$

where L is the beam length, G_c is the characteristic gravity load and Q_c is the characteristic wind load. The partial safety factor for gravity loading is $\gamma_{f,Gravity}$=1.1 and the partial safety factor for wind loading is $\gamma_{f,Wind}$=1.25/1.35 according to IEC 61400-1:2005. The factor α determines the ratio of gravity forces and the factor z is a design parameter calibrated in order to scale the load until failure, defined by the design equation.

The design bending moment on the beam is in design load case 6.1 calculated by:

$$M_{Ed} = \frac{1}{8} z\left((1-\alpha)\gamma_{f,Gravity} G_c + \alpha\gamma_{f,Wind} P_c\left(1 + 2k_p I_c\right)\right)L^2 \tag{B6}$$

where the partial safety factor for wind loading is $\gamma_{f,Wind}$=1.35 according to IEC 61400-1. P_c is the characteristic wind load, k_p=3.5 is the peak factor and I_c is the characteristic turbulence intensity.

The design equation for the beam is then given by:

$$G = M_{Rd} - M_{Ed} \geq 0 \tag{B7}$$

The limit state equation for the concrete beam is formulated based on the ratio of reinforcement:

$$\omega = \frac{A_s f_y}{b\,d\,f_c} \tag{B8}$$

where f_c and f_y are realizations of then concrete and reinforcement strength, respectively. The bending moment capacity of the beam is then given by:

$$M_R = \left(1 - \frac{1}{2}\omega\right)\omega b d^2 f_c \tag{B9}$$

The stochastic bending moment in the beam in design load case 1.1 is given by:

$$M_E = \frac{1}{8} z\left(\left(1-\alpha\right)G + \alpha X_{dyn} X_{\exp} X_{aero} X_{str} Q\right) L^2 \quad \text{(B10)}$$

where X_{dyn} is the uncertainty related to modelling of the dynamic response for the wind turbine, including uncertainty in damping ratios and eigenfrequencies. X_{exp} is the uncertainty related to the modelling of the exposure (site assessment) - such as the terrain roughness and the landscape topography. X_{aero} is related to the uncertainty in assessment of lift and drag coefficients. X_{str} accounts for the uncertainty related to the computation of the load-effect. The physical uncertainty of the extreme load-effect due to wind loads is modelled by the stochastic variable Q.

The stochastic bending moment in the beam in design load case 6.1 is given by:

$$M_E = \frac{1}{8} z\left(\left(1-\alpha\right)G + \alpha P\left(1+2k_p I X_{dyn}\right) X_{\exp} X_{aero} X_{str}\right) L^2 \quad \text{(B11)}$$

where P and I are the stochastic wind load and turbulence intensity, respectively. The limit state function is then defined by:

$$g = X_R M_R - M_E \quad \text{(B12)}$$

where X_R is the model uncertainty related the bending moment resistance. The individual stochastic variables are given in table B.1 and in table B.2 and B.3 the estimated reliabilities are shown for design load cases 1.1 and 6.1, respectively.

Table B.1. Stochastic variables for concrete beam and column.

Variable	Distribution	Mean	COV	Char.
f_c	Lognormal	29.5MPa	0.10	5%
f_y	Lognormal	597.1MPa	0.05	5%
H	Normal	650mm	0.02	-
X_R	Lognormal	1.00	0.05	-
X_{dyn}	Lognormal	1.00	0.05	-
X_{exp}	Lognormal	1.00	0.15	-
X_{aero}	Gumbel	1.00	0.10	-
X_{str}	Lognormal	1.00	0.03	-
Q	Weibull	1.00	0.15	98%
P	Gumbel	1.00	0.20	98%
I	Lognormal	0.11	0.05	-
G	Normal	1.00	0.05	-

Table B.2. Annual reliability index for concrete beam in DLC 1.1 in IEC 61400-1.

α	0.00	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00
γ_f =1.25 γ_S =1.10	3.85	4.32	4.27	3.97	3.71	3.51	3.35	3.22	3.11	3.03	2.96
γ_f =1.35 γ_S =1.10	3.85	4.45	4.46	4.19	3.95	3.77	3.62	3.51	3.41	3.33	3.27
γ_f =1.25 γ_S =1.15	4.28	4.74	4.60	4.23	3.94	3.71	3.53	3.39	3.28	3.18	3.10
γ_f =1.35 γ_S =1.15	4.28	4.87	4.79	4.45	4.17	3.96	3.80	3.67	3.57	3.48	3.41
γ_f =1.25 γ_S =1.20	4.69	5.14	4.91	4.48	4.15	3.90	3.71	3.56	3.43	3.33	3.25
γ_f =1.35	4.69	5.27	5.09	4.69	4.38	4.15	3.97	3.84	3.72	3.63	3.55

γ_S =1.20											

Table B.3. Annual reliability index for concrete beam in DLC 6.1 in IEC 61400-1.

α	0.00	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00
γ_f =1.35 γ_S =1.10	3.85	4.49	4.02	3.78	3.64	3.55	3.49	3.44	3.40	3.37	3.35
γ_f =1.35 γ_S =1.15	4.28	4.74	4.20	3.94	3.78	3.68	3.61	3.56	3.52	3.48	3.46
γ_f =1.35 γ_S =1.20	4.69	4.97	4.37	4.08	3.91	3.80	3.73	3.69	3.62	3.59	3.56

If it is assumed that typical α-values are in the range from 0.1 to 0.7 then it is seen from Table B.2 and B.3 that a partial safety factor γ_S =1.1-1.2 is appropriate to satisfy a reliability requirement corresponding to an annual reliability index equal to 3.3.

B3 Concrete Column

The design load bearing capacity of the short concrete column (assuming that stability failure is not important) is given by:

$$N_{crd} = A_c f_{cd} + A_s f_{yd} \quad \text{(B13)}$$

where A_s and A_c are the area of reinforcement and concrete, respectively. The design force applied to the column in design load case 1.1 and design load case 6.1 is given by equation (B14) and (B15), respectively:

$$N_{Ed} = z\left(\left(1-\alpha\right)\gamma_{f,Gravity} G_c + \alpha\,\gamma_{f,Wind} Q_c\right) \quad \text{(B14)}$$

$$N_{Ed} = z\left(\left(1-\alpha\right)\gamma_{f,Gravity} G_c + \alpha\,\gamma_{f,Wind} P_c\left(1+2k_p I_c\right)\right) \quad \text{(B15)}$$

In limit state equation the load bearing capacity is estimated from:

$$N_{cr} = A_c f_c + A_s f_y \quad \text{(B16)}$$

The stochastic force applied to the column in design load case 1.1 and design load case 6.1 is given by:

$$N_E = z\left(\left(1-\alpha\right)G + \alpha\, X_{dyn} X_{\exp} X_{aero} X_{str} Q\right) \quad \text{(B17)}$$

$$N_E = z\left(\left(1-\alpha\right)G + \alpha\, P\left(1+2k_p I\, X_{dyn}\right) X_{\exp} X_{aero} X_{str}\right) \quad \text{(B18)}$$

The limit state function is then defined by:

$$g = X_R N_{cr} - N_E \quad \text{(B19)}$$

The stochastic models given in table B.1 are also used for the concrete column. The estimated reliabilities are shown in table B.4 and B.5 for design load cases 1.1 and 6.1, respectively.

Table B.4. Annual reliability index for concrete column in DLC 1.1 in IEC 61400-1.

α	0.00	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00
γ_f =1.25 γ_C =1.30	4.53	4.93	4.89	4.58	4.30	4.07	3.89	3.74	3.62	3.51	3.43
γ_f =1.35 γ_C =1.30	4.53	5.03	5.06	4.78	4.52	4.31	4.14	4.01	3.90	3.80	3.72
γ_f =1.25 γ_C =1.40	5.08	5.47	5.34	4.94	4.61	4.36	4.15	3.99	3.85	3.74	3.64
γ_f =1.35 γ_C =1.40	5.08	5.58	5.50	5.13	4.82	4.59	4.40	4.25	4.13	4.02	3.94
γ_f =1.25 γ_C =1.50	5.58	5.97	5.73	5.26	4.89	4.61	4.39	4.21	4.06	3.94	3.84
γ_f =1.35 γ_C =1.50	5.58	6.07	5.88	5.44	5.10	4.83	4.63	4.47	4.33	4.22	4.13

Table B.5. Annual reliability index for concrete column in DLC 6.1 in IEC 61400-1.

α	0.00	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00
γ_f =1.35 γ_C =1.30	4.53	5.06	4.52	4.24	4.07	3.96	3.88	3.82	3.77	3.74	3.70
γ_f =1.35 γ_C =1.40	5.08	5.40	4.77	4.46	4.27	4.15	4.06	3.99	3.94	3.90	3.87
γ_f =1.35 γ_C =1.50	5.58	5.69	4.99	4.65	4.45	4.32	4.22	4.15	4.09	4.05	4.01

It is seen from Table B.4 and B.5 that a partial safety factor γ_C =1.3 is appropriate to satisfy a reliability requirement corresponding to an annual reliability index equal to 3.3.

B5 References

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ANNEX C. FATIGUE RELIABILITY AND SAFETY FACTORS FOR WELDED DETAILS

C1 Introduction

For wind turbine steel substructures fatigue can be a critical failure mode for welded details, especially if joints with high stress concentrations are used. This annex describes reliability-based investigations on the required safety factors to be used for design for fatigue.

Design equations to be used for deterministic, code-based design and corresponding limit state equations to be used for reliability assessment are formulated. In the limit state equations uncertain parameters are modelled as stochastic variables. In the design equations partial safety factors for fatigue strength and load or equivalently Fatigue Design Factors (FDF) are used to secure the required reliability level.

Since design and limit state equations are closely related a detailed model of the fatigue damage is generally not needed for reliability-based assessment of fatigue safety factors. It is 'only' important to model the dependency on the uncertain parameters and the uncertain parameters themselves carefully. In this annex is considered the case with wind load dominating and no wake effects taken into account. In the UpWind report [1] and Sørensen [2] more detailed models and results are shown for the cases: 1) wave load dominating; 2) wind load dominating for a single wind turbine; 3) wind load dominating for a wind turbine in a wind farm. SN-curves and Miner's rule with linear damage accumulation are used as recommended in most relevant standards, e.g. IEC 61400-1 [3], ISO 19902 [4] and Eurocodes [5].

C2 Stochastic modelling

In this section probabilistic models are described for reliability assessment of wind turbines where wind load is dominating (over wave loads). The models are mainly based on Sørensen et al. [6].

If a linear SN-curves is considered the SN relation is written

$$N = K(\Delta\sigma)^{-m} \tag{C1}$$

where N is the number of stress cycles to failure with constant stress ranges $\Delta\sigma$. K and m are dependent on the fatigue critical detail.

For a wind turbine in free wind flow the design equation in deterministic design is written

$$G(z) = 1 - \int_{U_{in}}^{U_{out}} \frac{\nu \cdot FDF \cdot T_L}{K_C} D_L\left(m; \alpha_{\Delta\sigma}(U)\hat{\sigma}_u(U)/z\right) f_U(U)\, dU = 0 \tag{C2}$$

where

z is a design parameter (e.g. proportional to cross sectional area)

$$D_L(m;\sigma_{\Delta\sigma}) = \int_0^\infty s^m f_{\Delta\sigma}\left(s \mid \sigma_{\Delta\sigma}(U)\right) ds \tag{C3}$$

is the expected value of $\Delta\sigma^m$ given standard deviation $\sigma_{\Delta\sigma}$ and mean wind speed U

ν is the total number of fatigue load cycles per year (determined by e.g. rainflow counting)

T_L is the design life time

FDF is the Fatigue Design Factor (equal to $(\gamma_f \gamma_m)^m$ where γ_f and γ_m are partial safety factors for fatigue load and fatigue strength)

K_C is the characteristic value of K (here assumed to be obtained from $\log K_C$ as mean of $\log K$ minus two standard deviations)

U_{in} is the cut-in wind speed (typically 5 m/s)

U_{out} is the cut-out wind speed (typically 25 m/s)

$f_{\Delta\sigma}(s|\sigma_{\Delta\sigma}(U))$ is the density function for stress ranges given standard deviation of $\sigma_{\Delta\sigma}(U)$ at mean wind speed U. This distribution function can be obtained by e.g. rainflow counting of response, and can generally be assumed to be Weibull distributed, see below.

It is assumed that the standard deviation of the stress ranges, $\sigma_{\Delta\sigma}(U)$ can be written:

$$\sigma_{\Delta\sigma}(U) = \alpha_{\Delta\sigma}(U)\frac{\sigma_u(U)}{z} \tag{C4}$$

where

$\alpha_{\Delta\sigma}(U)$ is the influence coefficient for stress ranges given mean wind speed U

$\sigma_u(U)$ is the standard deviation of turbulence given mean wind speed U.

$\sigma_u(U)$ is modelled as LogNormal distributed with characteristic value $\hat{\sigma}_u(U)$ defined as the 90% quantile and standard deviation equal to $I_{ref} \cdot 1.4$ [m/s]. The characteristic value of the standard deviation of turbulence, $\hat{\sigma}_u(U)$ given average wind speed U is modelled by, see IEC 61400-1 [3]:

$$\hat{\sigma}_u(U) = I_{ref} \cdot (0.75 \cdot U + b) \quad ; \ b = 5.6 \text{ m/s} \tag{C5}$$

where I_{ref} is the reference turbulence intensity (equal to 0.14 for medium turbulence characteristics) and $\hat{\sigma}_u$ is denoted the ambient turbulence.

The corresponding limit state equation is written

$$g(t) = \Delta - \int_{U_{in}}^{U_{out}} \int_0^{\infty} \frac{\nu \cdot t}{K} (X_{Wind} X_{SCF})^m D_L(m; \alpha_{\Delta\sigma}(U)\sigma_u(U)/z) f_{\sigma_u}(\sigma_u|U) f_U(U) d\sigma_u dU \tag{C6}$$

where

Δ is a stochastic variable modelling the model uncertainty related to the Miner rule for linear damage accumulation

t is time in years

X_{Wind} is the model uncertainty related to assessment of the fatigue wind load effects and is due to uncertainties related to site assessment, assessment of lift and drag coefficients, dynamic response calculations,

X_{SCF} is the model uncertainty related to local stress analysis given global fatigue load effects

$\sigma_u(U)$ standard deviation of turbulence given average wind speed U.

The model uncertainties X_{Wind} and X_{SCF} are discussed in more details in e.g. Tarp-Johansen et al. [7].

The design parameter z is determined from the design equation (C2) and next used in the limit state equation (C6) to estimate the reliability index or probability of failure with the reference time interval $[0;t]$.

Next, it is assumed that the SN-curve is bilinear (thickness effect not included) with slope change at $N_D = 5 \cdot 10^6$:

$$N = K_1 S^{-m_1} \quad \text{for} \quad S \geq \Delta\sigma_D$$
$$N = K_2 S^{-m_2} \quad \text{for} \quad S < \Delta\sigma_D \qquad \text{(C7)}$$

where K_1, m_1 material parameters for $S \geq \Delta\sigma_D$ and K_2, m_2 material parameters for $S < \Delta\sigma_D$. In this section the quantile defining the characteristic values for K_1 and K_2 is chosen to 2.3%.

$$\Delta\sigma_D = \left(\frac{K_1}{5 \cdot 10^6} \right)^{1/m_1} \qquad \text{(C8)}$$

The fatigue strength $\Delta\sigma_F$ is defined as the value of S for $N_D = 2 \cdot 10^6$.

In case the SN-curve is bilinear $D_L(m; \sigma_{\Delta\sigma})$ in design equations and limit state equations is exchanged with

$$D_{BL}(m_1, m_2, \Delta\sigma_D; \sigma_{\Delta\sigma}) = \int_0^{\Delta\sigma_D} s^{m_2} f_{\Delta\sigma}(s|\sigma_{\Delta\sigma}(U))ds + \int_{\Delta\sigma_D}^{\infty} s^{m_1} f_{\Delta\sigma}(s|\sigma_{\Delta\sigma}(U))ds \qquad \text{(C9)}$$

(C9) can easily be modified to include a lower threshold $\Delta\sigma_{th}$. Further, the SN-curves can also be extended with a modification factor taking into account thickness effects.

C3 Reliability analysis and calibration of partial safety factors

If one fatigue critical detail is considered then the annual probability of failure is obtained from:

$$\Delta P_{F,t} = P_{\text{COL}|\text{FAT}} P(\text{Fatigue failure in year } t) \qquad \text{(C10)}$$

where $P(\text{Fatigue failure in year } t)$ is the probability of failure in year *t* and $P_{\text{COL}|\text{FAT}}$ is the probability of collapse of the structure given fatigue failure - modelling the importance of the detail.

The probability of failure in year *t* given survival up to year *t* is estimated by

$$\Delta P_{F,t} = P_{\text{COL}|\text{FAT}} \left(P(g(t) \leq 0) - P(g(t-1) \leq 0) \right) / P(g(t) \leq 0) \qquad \text{(C11)}$$

where the limit state equation is given in (C6).

Given a maximum acceptable probability of failure (collapse), $\Delta P_{F,\max}$ the maximum acceptable annual probability of fatigue failure (with one year reference time) and corresponding minimum reliability index become:

$$\Delta P_{F,\max,FAT} = \Delta P_{F,\max} / P_{\text{COL}|\text{FAT}} \qquad \text{(C12)}$$
$$\Delta\beta_{\min,FAT} = -\Phi^{-1}(\Delta P_{F,\max,FAT}) \qquad \text{(C13)}$$

where $\Phi(\ )^{-1}$ is the inverse standard Normal distribution function.

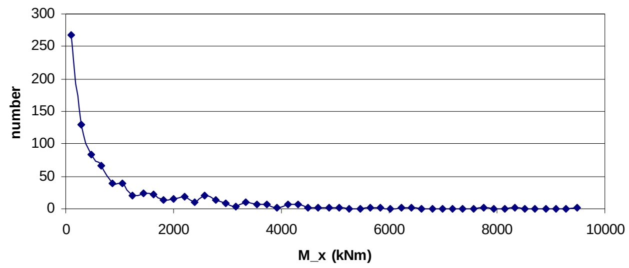

Figure C1. Number of load cycles in a 10 minutes period for mudline bending moment. Mean wind speed equal to 14 m/s.

Figure C1 shows a typical distribution of stress ranges for a pitch controlled wind turbine for tower bending moments, see Sørensen et al. [6]. The stress ranges can generally be modelled by a Weibull distribution. The Weibull shape coefficient k is typically in the range 0.8 – 1.0. These results are for cases where the response is dominated by the "background" turbulence in the wind load. For the results shown below it is assumed that the stress ranges are Weibull distributed with shape coefficient k = 0.8. The number of load cycles per year is $\nu = 10^7$.

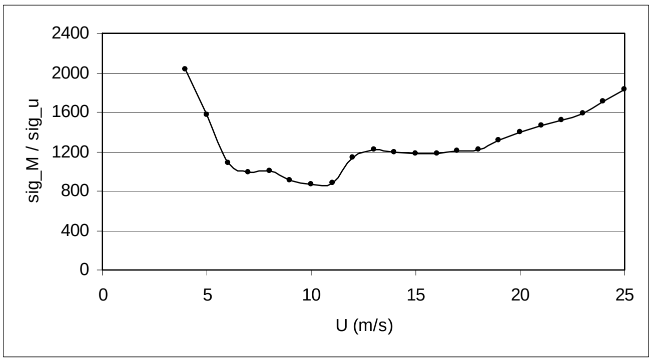

Figure C2. $\sigma_{\Delta\sigma}(U)/\sigma_u(U)$ for mudline bending moment – pitch controlled wind turbine.

In figure C2 is shown a typical example for a pitch controlled wind turbine of $\alpha_{\Delta\sigma}(U)/z$ = $\sigma_{\Delta\sigma}(U)/\sigma_u(U)$, see (C4). The ratio is seen to be non-linear due to the effect of the control system.

Table C1. Stochastic model.

Variable	Distribution	Expected value	Standard deviation / Coefficient Of variation	Comment
Δ	N	1	$COV_\Delta = 0.30$	Model uncertainty Miner's rule
X_{Wind}	LN	1	COV_{Wind}	Model uncertainty wind load
X_{SCF}	LN	1	COV_{SCF}	Model uncertainty stress concentration factor
m_1	D	3		Slope SN curve
$\log K_1$	N	determined from $\Delta\sigma_D$	$\sigma_{\log K_1} = 0.2$	Parameter SN curve
m_2	D	5		Slope SN curve
$\log K_2$	N	determined from $\Delta\sigma_D$	$\sigma_{\log K_2} = 0.2$	Parameter SN curve
$\Delta\sigma_F$	D	71 MPa		Fatigue strength
$\log K_1$ and $\log K_2$ are fully correlated				

The stochastic model shown in table C1 is considered as representative for a fatigue sensitive detail, see Sørensen [2]. It is assumed that the design lifetime is T_L = 25 year.

$\Delta P_{F,\max}$ = 5 10^{-4} (normal/high consequence of failure) and 5 10^{-3} (low consequence of failure) are used as annual maximum probabilities of failure. The corresponding annual reliability indices are 3.3 and 2.6.

The mean wind speed is assumed to be Weibull distributed:

$$F_U(u) = 1 - \exp\left(-\left(\frac{u}{A}\right)^k \right) \tag{C14}$$

with A = 9.0 m/s and k = 2.3. It is assumed that the reference turbulence intensity is I_{ref}=0.14.

Table C2 shows the required product of the partial safety factors $\gamma_f \gamma_m$ as function of the total coefficient of variation of the fatigue load: $COV_{load} = \sqrt{COV_{Wind}^2 + COV_{SCF}^2}$.

Table C2. Required partial safety factors $\gamma_f \gamma_m$ given $\Delta\beta_{\min,FAT}$ as function of COV for fatigue load.

$\Delta\beta_{\min,FAT}$ \ COV_{load}	0,00	0,05	0,10	0,15	0,20	0,25	0,30
2,6 (5 10^{-3})	0,91	0,92	0,94	0,98	1,01	1,04	1,06
3,3 (5 10^{-4})	1,04	1,06	1,12	1,21	1,32	1,43	1,56

Assuming that a coefficient of variation for the fatigue load ranges is typically within the interval 15-20% the partial safety factor γ_f in table C3 and

γ_m = 1.25

are recommended. It is noted that this partial safety factor corresponds to a fatigue design factor (FDF) equal to 3 if Wöhler exponent m = 5.

Table C3. Recommended partial safety factor for fatigue stress ranges, γ_f .

Coefficient of variation, COV_{load}	0-5 %	5-10 %	10-15 %	15-20 %	20-25 %	25-30 %
γ_f	0,85	0,90	0,95	1,00	1,10	1,20

Next, damage tolerant and safe life reliability assessment methods are introduced based on the following descriptions (from EN 1993-1-9:2005 [5]).

a) damage tolerant method

- selecting details, materials and stress levels so that in the event of the formation of cracks a low rate of crack propagation and a long critical crack length would result,
- provision of multiple load path
- provision of crack-arresting details,
- provision of readily inspectable details during regular inspections.

b) safe-life method

- selecting details and stress levels resulting in a fatigue life sufficient to achieve the target β – value at the end of the design service life. No inspections are required.

Generally, for the 'Damage tolerant' approach either the structure is redundant or inspections are performed (or a combination of these).

The fatigue strength partial safety factor is then generalised according to table C4. The partial safety factors are assumed to correspond to normal consequences of failure, i.e. component class 2 in IEC 61400-1.

Table C4. Recommended values for partial safety factor for fatigue strength, γ_m.

Assessment method	γ_m
Damage tolerant	1,10
Safe life	1,25

C4 Calibration of partial safety factors in case of inspections

In this section is investigated how much the partial safety factor for fatigue can be reduced if inspections are performed during the lifetime of a wind turbine. In order to model the influence of inspections a Fracture Mechanics model (FM) is needed for estimating the crack growth. The fracture mechanics model is calibrated to give the same reliability as function of time as obtained by the SN-approach.

The Fracture Mechanical (FM) modeling of the crack growth is applied assuming that the crack can be modeled by a 2-dimensional semi-elliptical crack, or simplified models where the ratio between crack width and depth is either a constant or the crack width is a given function of the crack depth. It is assumed that the fatigue life may be represented by a fatigue initiation life and a fatigue propagation life:

$$N = N_I + N_P \qquad \text{(C15)}$$

where

N number of stress cycles to failure

N_I number of stress cycles to crack propagation

N_P number of stress cycles from initiation to crack through.

The number of stress cycles from initiation to crack through is determined on the basis of a two-dimensional crack growth model. The crack is assumed to be semi-elliptical with length $2c$ and depth a, see Figure C3.

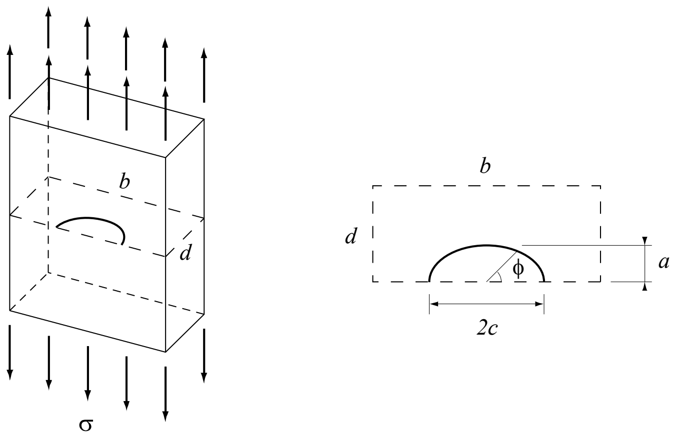

Figure C3. Semi-elliptical surface crack in a plate under tension or bending fatigue loads.

The crack growth can be described by the following two coupled differential equations.

$$\frac{da}{dN} = C_A (\Delta K_A)^m \qquad a(N_0) = a_0$$
$$\frac{dc}{dN} = C_C (\Delta K_C)^m \qquad c(N_0) = c_0 \tag{C16}$$

where C_A, C_C and m are material parameters, a_0 and c_0 describe the crack depth *a* and crack length *c*, respectively, after N_I cycles and where the stress intensity ranges are $\Delta K_A(\Delta\sigma)$ and $\Delta K_C(\Delta\sigma)$.

The stress range $\Delta\sigma$ is obtained from

$$\Delta\sigma = X_{Wind} X_{SCF} \cdot Y \cdot \Delta\sigma^e \tag{C17}$$

where

X_{Wave}, X_{SCF} model uncertainties, see section C3

Y model uncertainty related to geometry function

$\Delta\sigma^e$ equivalent stress range:

$$\Delta\sigma^e = \left[\frac{1}{n} \sum_{i=1}^{n_\sigma} n_i \Delta\sigma_i^m \right]^{1/m} \tag{C18}$$

The total number of stress ranges per year, n is

$$n = \sum_{i=1}^{n_\sigma} n_i \tag{C19}$$

The crack initiation time N_I is modeled as Weibull distributed with expected value μ_0 and coefficient of variation equal to 0.35, see e.g. Lassen [8].

Variable	Dist.	Expected value	Standard deviation
N_I	W	μ_0 (reliability based fit to SN approach)	0.35 μ_0
a_0	D	0.1 mm (high material control) / 0.5 mm (low material control)	
$\ln C_C$	N	$\mu_{\ln C_C}$ (reliability based fit to SN approach)	0.77
m	D	m-value (reliability based fit to SN approach)	
Z_{SCF}	LN	1	0.05
X_{Wave}	LN	1	0.20
n	D	Total number of stress ranges per year	

a_c	D	T (thickness)	
Y	LN	1	0.1
T	D	Thickness	
T_L	D	25 years	
T_F	D	Fatigue life	
$\ln C_C$ and N_I are correlated with correlation coefficient $\rho_{\ln(C_C),N_I}$ = -0.5			

Table C5. Uncertainty modelling used in the fracture mechanical reliability analysis. D: Deterministic, N: Normal, LN: LogNormal, W: Weibull.

The limit state function is written

$$g(\mathbf{X}) = N - n\,t \qquad \text{(C20)}$$

where t is time in the interval from 0 to the service life T_L.

To model the effect of different weld qualities, different values of the crack depth at initiation a_0 can be used. The corresponding assumed length is 5 times the crack depth. The critical crack depth a_c is taken as the thickness of the tubular member.

The parameters $\mu_{\ln C}$ and μ_0 are now fitted such that difference between the probability distribution functions for the fatigue live determined using the SN-approach and the fracture mechanical approach is minimized as illustrated in the example below.

Alternatively, or in addition to the above modeling the initial crack length can be modeled as a stochastic variable, for example by an exponential distribution function, and the crack initiation time N_I can be neglected.

The reliability of inspections can be modeled in many different ways. Often POD (Probability Of Detection) curves are used to model the reliability of the inspections, e.g. an exponential model:

$$POD(x) = 1 - \exp\left(-\frac{x}{\lambda}\right) \qquad \text{(C21)}$$

where λ is the expected value of the smallest detectable crack size.

The crack width $2c$ is obtained from the following model for $a/2c$ as a function of the relative crack depth a/B, where B is the thickness:

$$\frac{a}{2c} = 0.06 - 0.03\ln\left(\frac{a}{B}\right) \qquad \text{(C22)}$$

If an inspection has been performed at time T_I and no cracks are detected then the probability of failure can be updated by

$$P_F^U\left(t \middle| \text{no - detection at time } T_I\right) = P\left(g(t) \le 0 \middle| h(T_I) > 0\right) \quad , \; t > T_I \qquad \text{(C23)}$$

where $h(t)$ is a limit state modeling the crack detection. If the inspection technique is related to the crack length then $h(t)$ is written:

$$h(t) = c_d - c(t) \tag{C24}$$

where $c(t)$ is the crack length at time t and c_d is smallest detectable crack length. c_d is modelled by a stochastic variable with distribution function equal to the POD-curve:

$$F_{c_d}(x) = POD(x) \tag{C25}$$

Similarly if the inspection technique is related to the crack depth then $h(t)$ is written:

$$h(t) = a_d - a(t) \tag{C26}$$

where $a(t)$ is the crack length at time t and a_d is smallest detectable crack length. a_d is modelled by a stochastic variable with distribution function equal to the POD-curve:

$$F_{a_d}(x) = POD(x) \tag{C27}$$

If two independent inspections are performed at time T_I and no cracks are detected then the probability of failure can be updated by

$$P_F^U\left(t \middle| \text{no - detection at time } T_I\right) = P\left(g(t) \le 0 \middle| h_1(T_I) > 0 \cap h_2(T_I) > 0\right) \quad , t > T_I \tag{C28}$$

where $h_1(t) = a_{d_1} - a(T_I)$ and $h_2(t) = a_{d_2} - a(T_I)$ are the limit states modeling the inspections.

The inspection planning is based on the requirement that the annual probability of failure in all years has to satisfy the reliability constraint

$$\Delta P_F \le \Delta P_{F,MAX} \quad , t > T_I \tag{C29}$$

where $\Delta P_{F,MAX}$ is the maximum acceptable annual probability of failure.

Further, the planning is often made with the assumption that no cracks are found at the inspections. If a crack is found, then a new inspection plan has to be made based on the observation.

It is emphasized that the inspection planning is based on the no-find assumption. This way of inspection planning is the one which if most often used. Often this approach results in increasing time intervals between inspections.

Figure to C4 to C19 shows results for both accumulated and annual reliability indices for the following cases:

1. Inspection with time intervals 2, 3, 4, 5 and 10 years, λ=10 mm, partial safety factor γ_m = 1.00
2. Inspection with time intervals 2, 3, 4, 5 and 10 years, λ=10 mm, partial safety factor γ_m = 1.05

3. Inspection with time intervals 2, 3, 4, 5 and 10 years, λ=10 mm, partial safety factor γ_m = 1.10
4. Inspection with time intervals 2, 3, 4, 5 and 10 years, λ=10 mm, partial safety factor γ_m = 1.15
5. Inspection with time intervals 2, 3, 4, 5 and 10 years, λ=10 mm, partial safety factor γ_m = 1.20
6. Inspection with time intervals 2, 3, 4, 5 and 10 years, λ=10 mm, partial safety factor γ_m = 1.25
7. Inspection at year 13 and λ=2, 5 and 10 mm, partial safety factor γ_m = 1.00
8. Inspection with time intervals 2, 3, 4, 5 and 10 years, λ=5 mm, partial safety factor γ_m = 1.10
9. Inspection with time intervals 2, 3, 4, 5 and 10 years, λ=5 mm, partial safety factor γ_m = 1.10

The aspect ratio is 0.2 for all cases except case 9 where equation (C22) is applied.

The results show among others that

- if the fatigue partial safety factor is chosen to 1.0 then one inspection is needed at year 13 with at least a reliability which corresponds to an expected value of the smallest detectable crack equal to 2 mm
- if the fatigue partial safety factor is chosen to 1.0 then inspection intervals of maximum 5 years should be performed with at least a reliability which corresponds to an expected value of the smallest detectable crack equal to 10 mm
- if the fatigue partial safety factor is chosen to 1.1 then inspection intervals of maximum 10 years should be performed with at least a reliability which corresponds to an expected value of the smallest detectable crack equal to 10 mm
- if the aspect ratio given by (C22) is used then slightly larger inspection intervals can be used

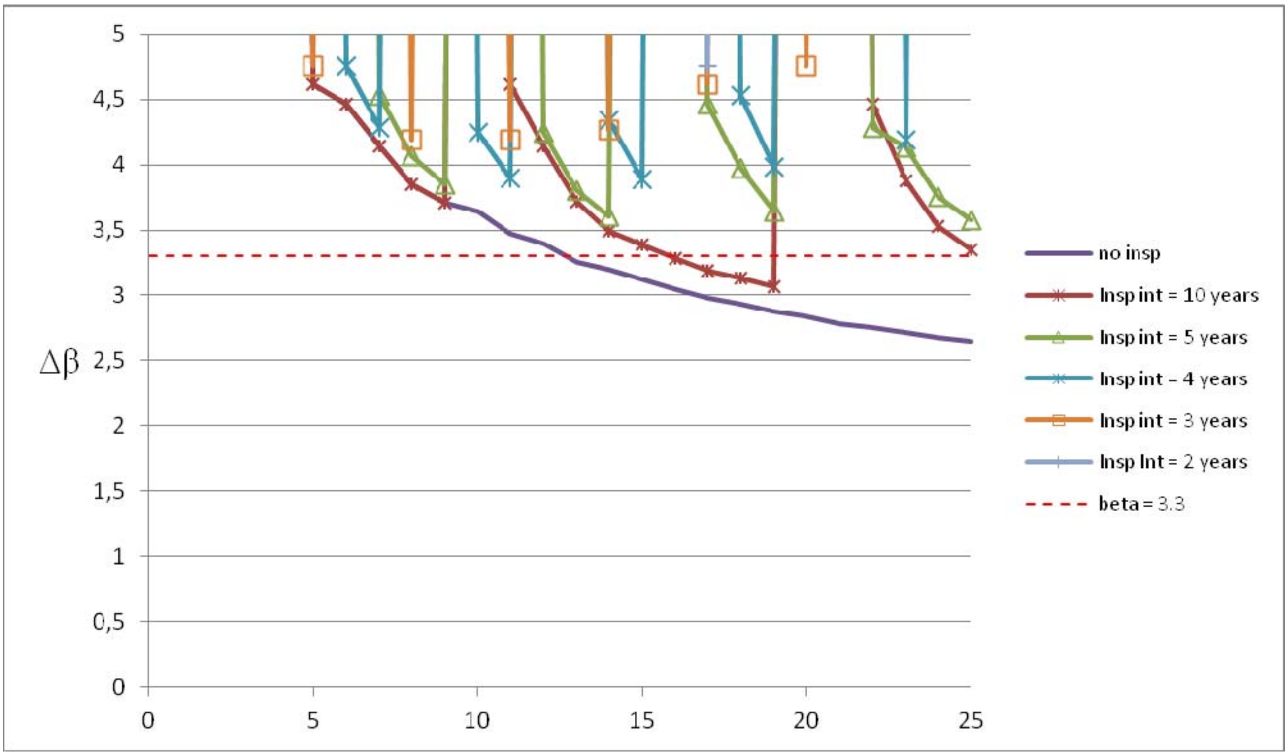

Figure C4. Annual reliability index without and with inspections. Inspection time intervals 2, 3, 4, 5 and 10 years and λ=10 mm, partial safety factor γ_m = 1.00 and aspect ratio = 0.2.

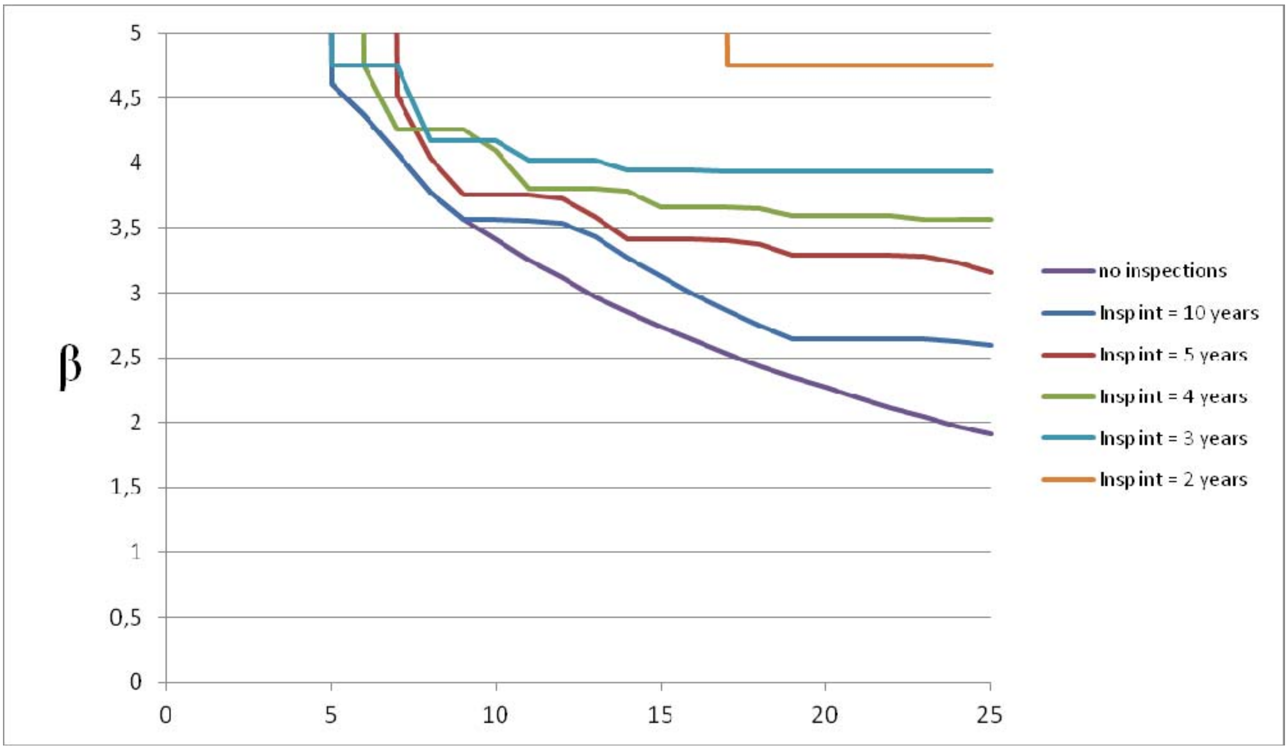

Figure C5. Accumulated reliability index without and with inspections. Inspection time intervals 2, 3, 4, 5 and 10 years and λ=10 mm, partial safety factor γ_m = 1.00 and aspect ratio = 0.2.

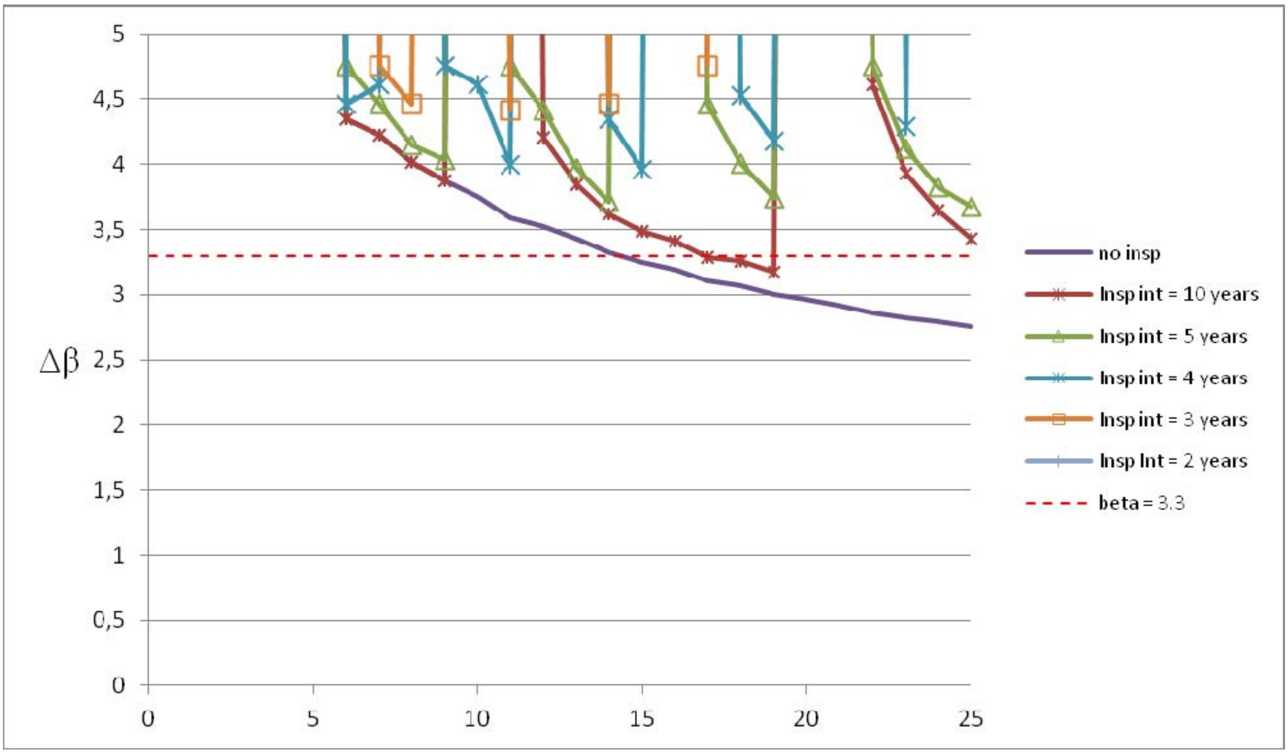

Figure C6. Annual reliability index without and with inspections. Inspection time intervals 2, 3, 4, 5 and 10 years and λ=10 mm, partial safety factor γ_m = 1.05 and aspect ratio = 0.2.

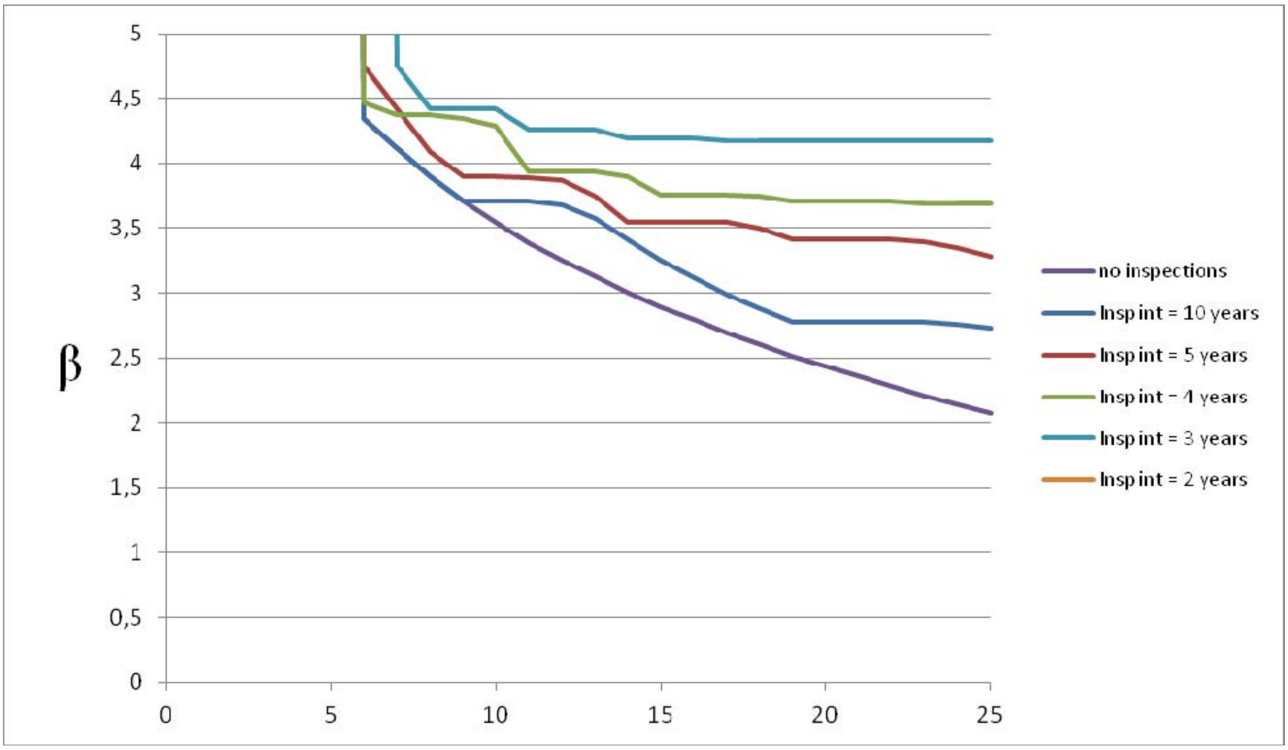

Figure C7. Accumulated reliability index without and with inspections. Inspection time intervals 2, 3, 4, 5 and 10 years and λ=10 mm, partial safety factor γ_m = 1.05 and aspect ratio = 0.2.

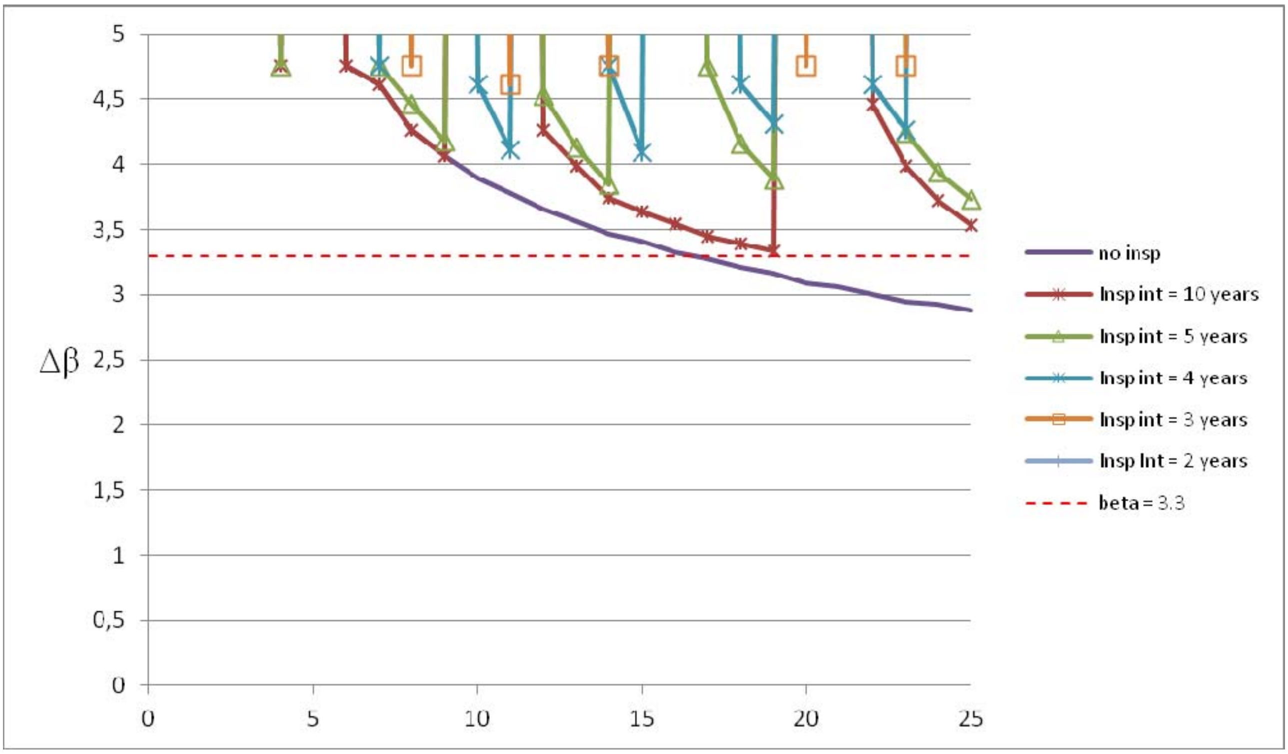

Figure C8. Annual reliability index without and with inspections. Inspection time intervals 2, 3, 4, 5 and 10 years and λ=10 mm, partial safety factor γ_m = 1.10 and aspect ratio = 0.2.

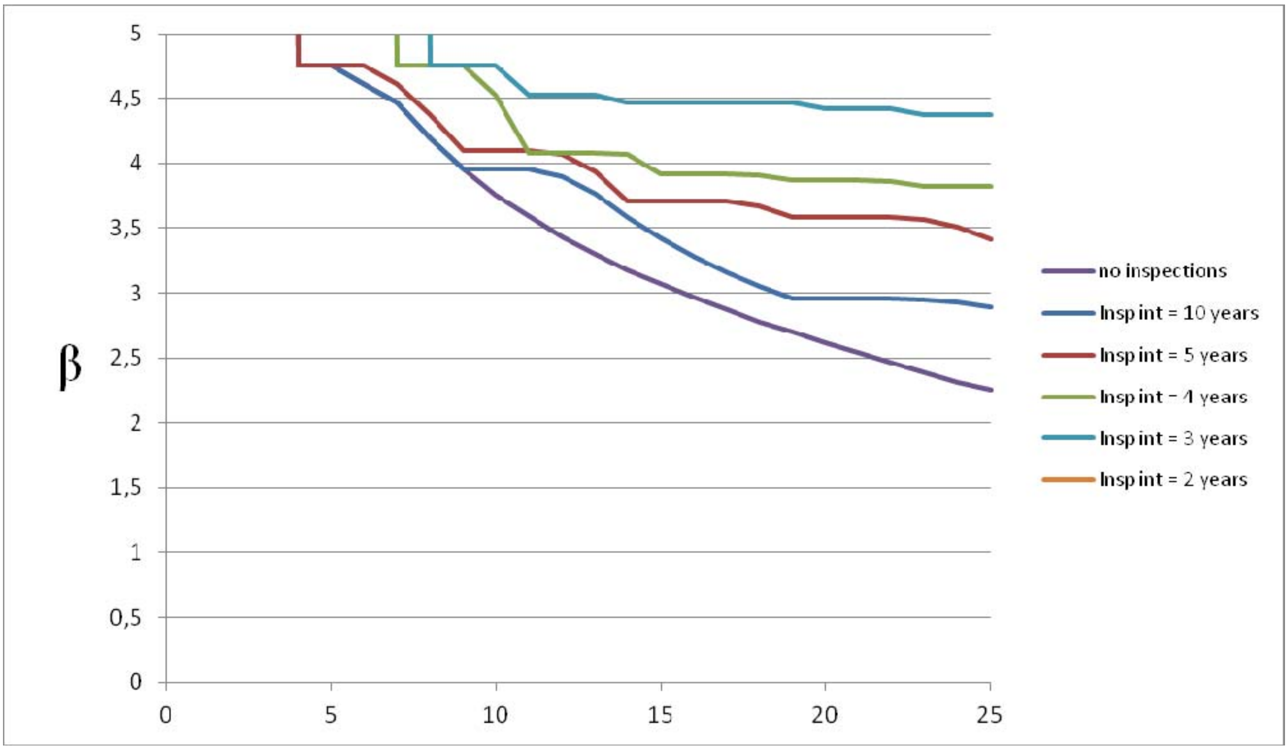

Figure C9. Accumulated reliability index without and with inspections. Inspection time intervals 2, 3, 4, 5 and 10 years and λ=10 mm, partial safety factor γ_m = 1.10 and aspect ratio = 0.2.

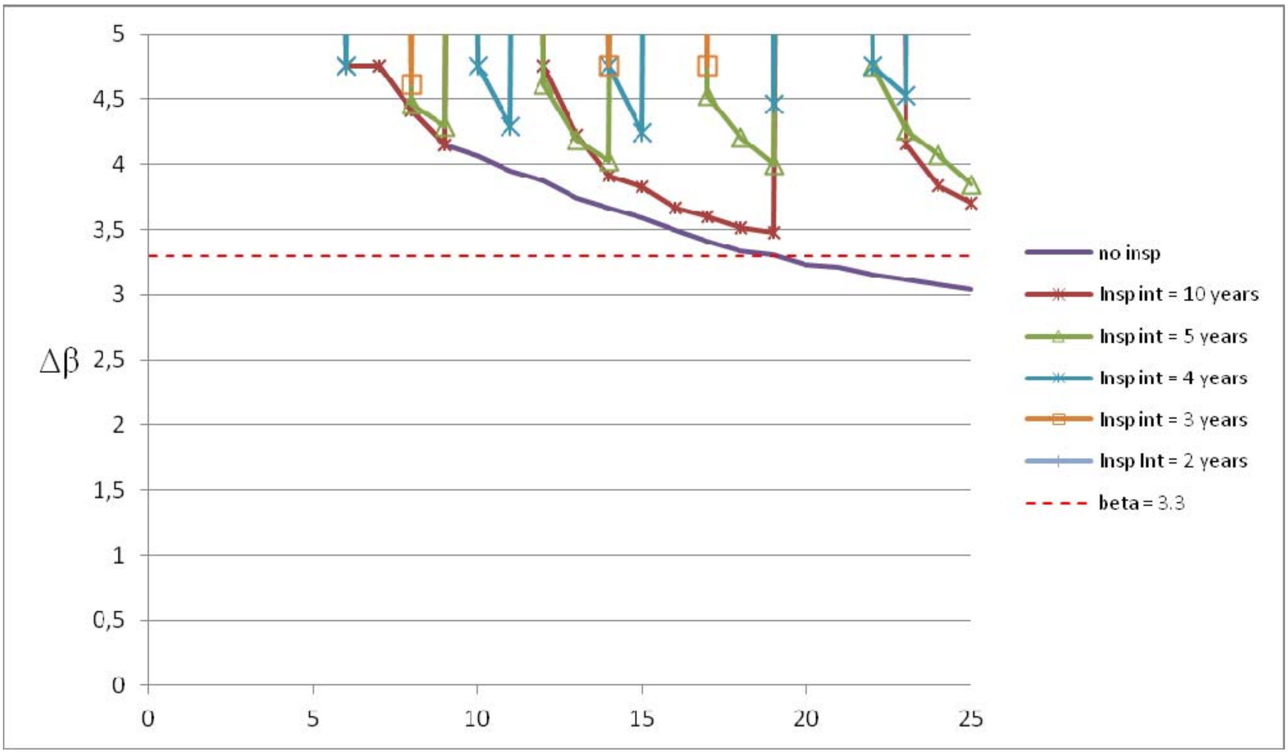

Figure C10. Annual reliability index without and with inspections. Inspection time intervals 2, 3, 4, 5 and 10 years and λ=10 mm, partial safety factor γ_m = 1.15 and aspect ratio = 0.2.

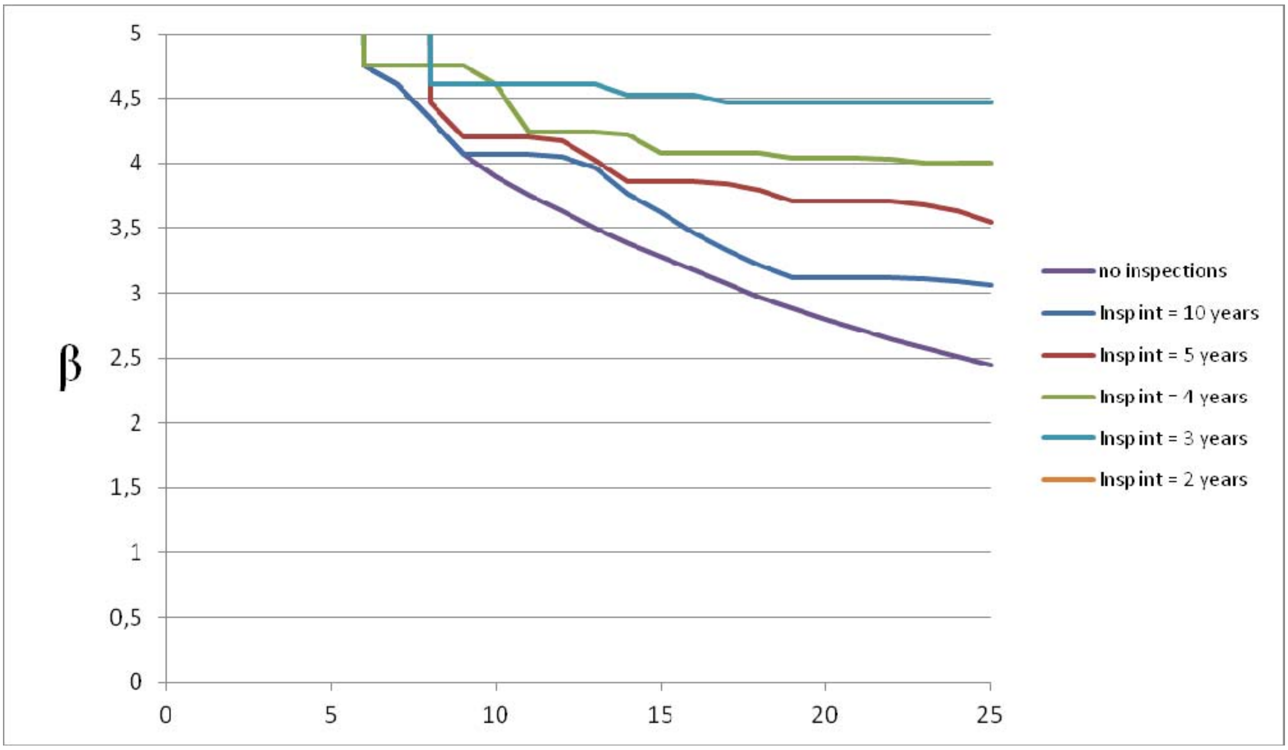

Figure C11. Accumulated reliability index without and with inspections. Inspection time intervals 2, 3, 4, 5 and 10 years and λ=10 mm, partial safety factor γ_m = 1.15 and aspect ratio = 0.2.

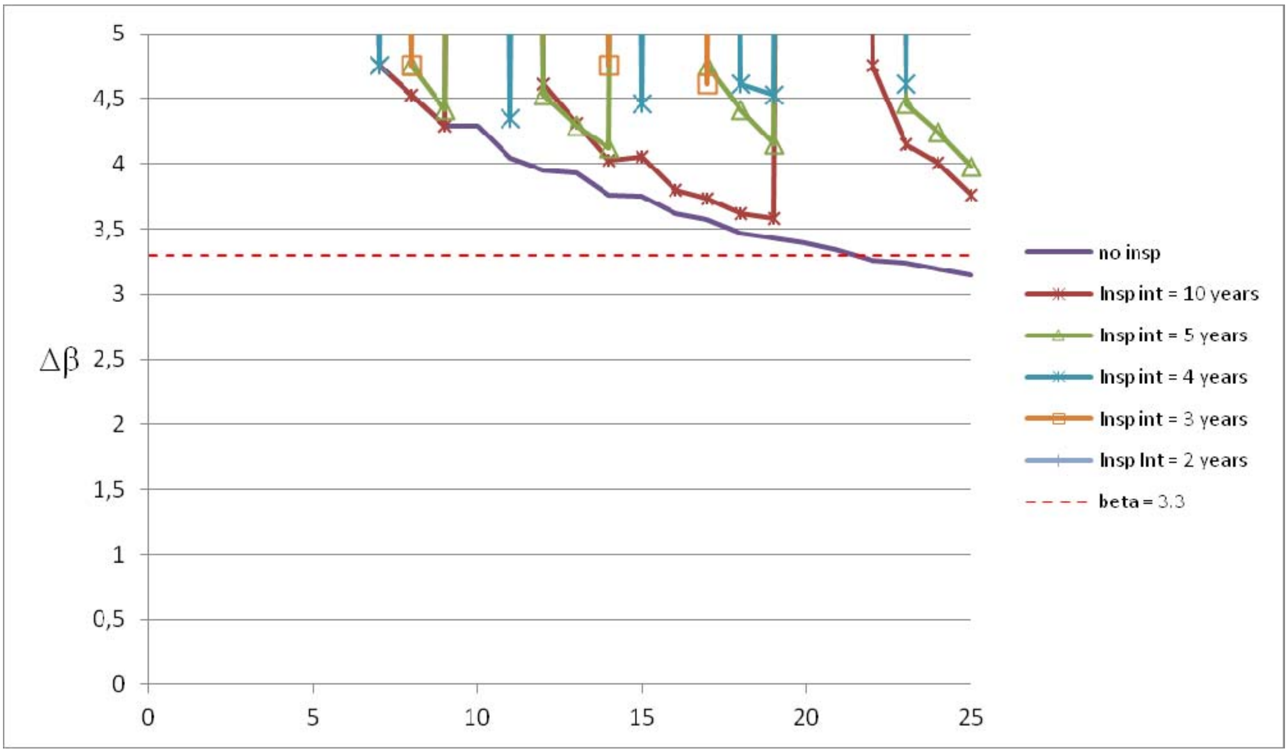

Figure C12. Annual reliability index without and with inspections. Inspection time intervals 2, 3, 4, 5 and 10 years and λ=10 mm, partial safety factor γ_m = 1.20 and aspect ratio = 0.2.

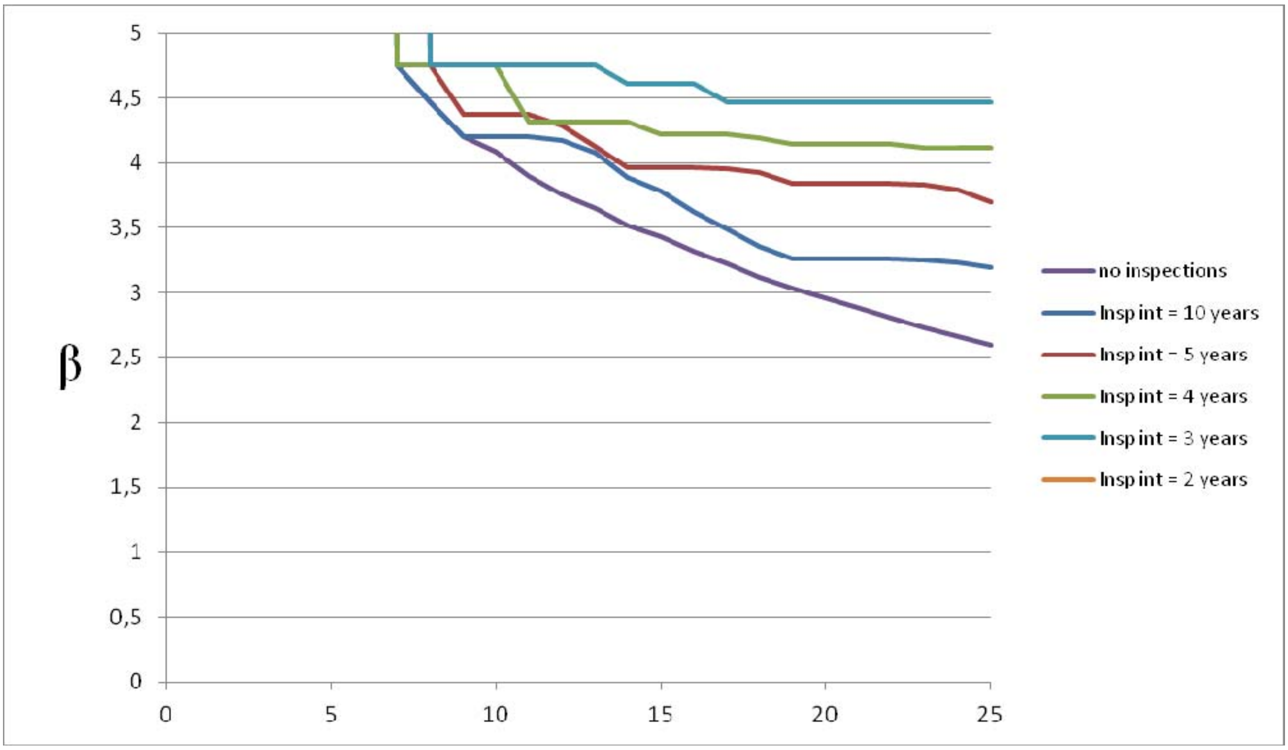

Figure C13. Accumulated reliability index without and with inspections. Inspection time intervals 2, 3, 4, 5 and 10 years and λ=10 mm, partial safety factor γ_m = 1.20 and aspect ratio = 0.2.

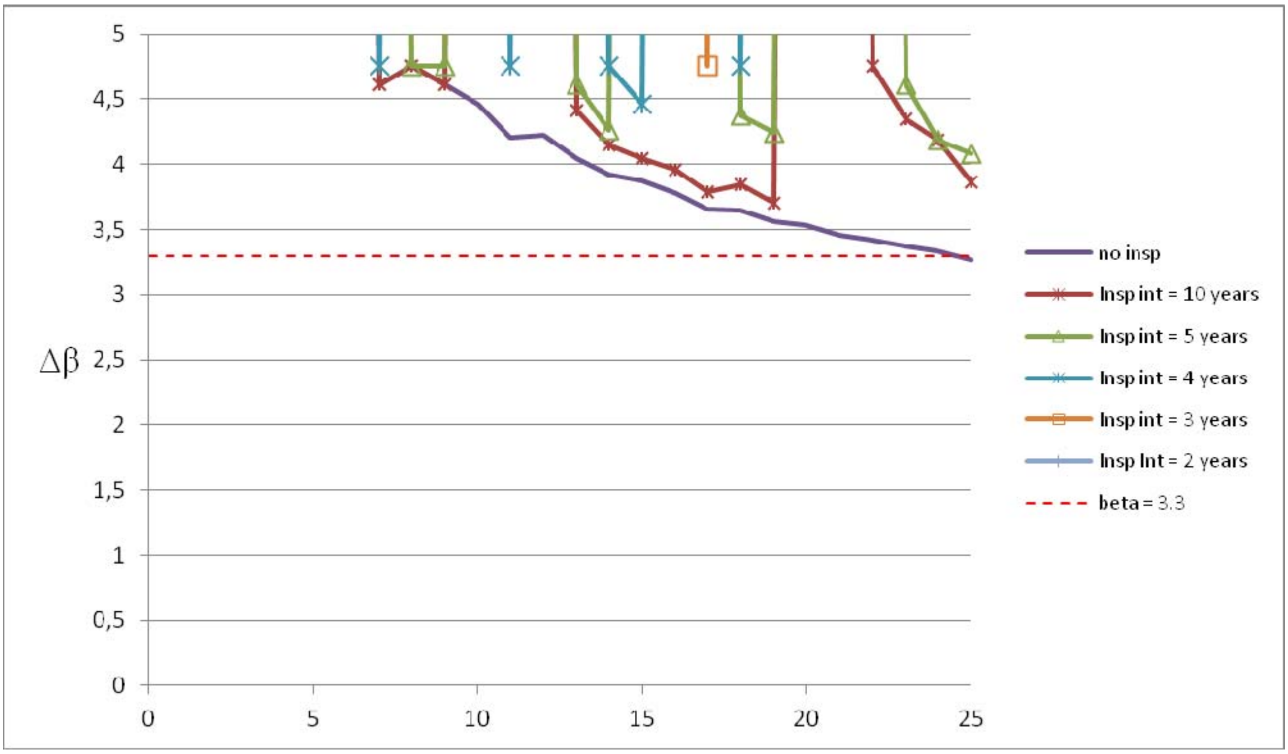

Figure C14. Annual reliability index without and with inspections. Inspection time intervals 2, 3, 4, 5 and 10 years and λ=10 mm, partial safety factor γ_m = 1.25 and aspect ratio = 0.2.

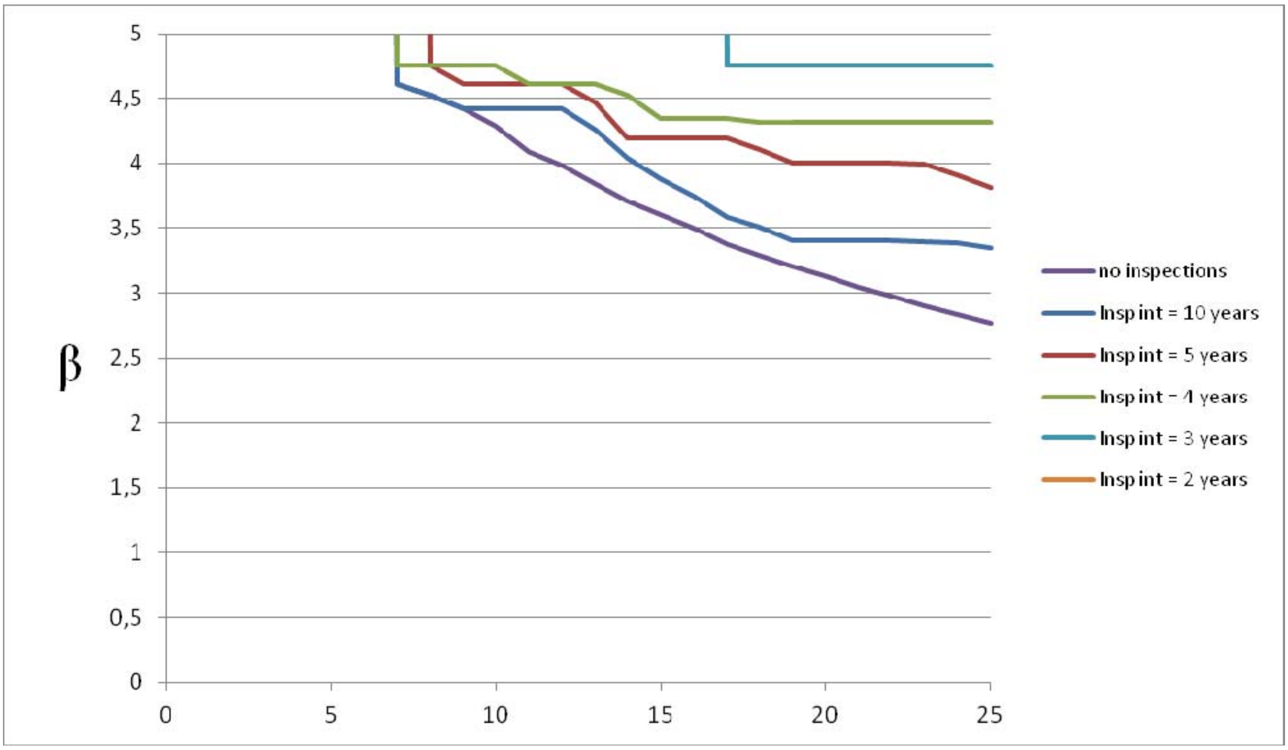

Figure C15. Accumulated reliability index without and with inspections. Inspection time intervals 2, 3, 4, 5 and 10 years and λ=10 mm, partial safety factor γ_m = 1.25 and aspect ratio = 0.2.

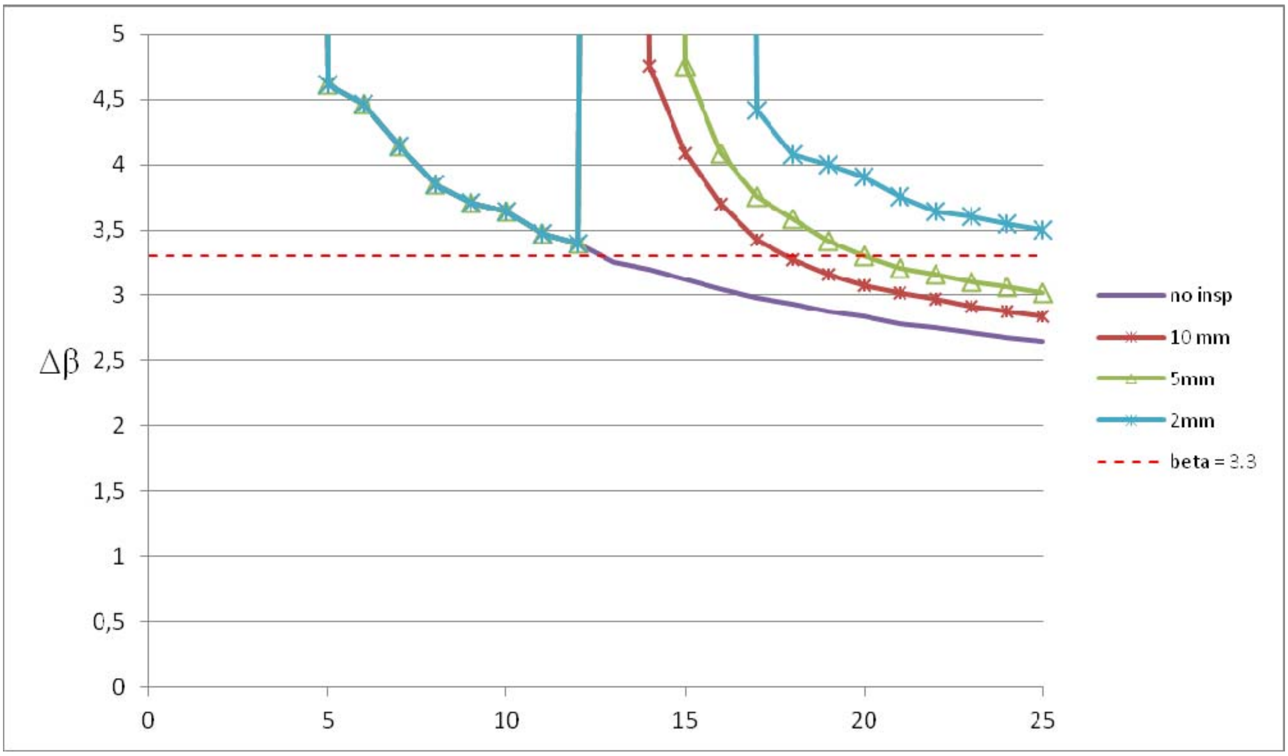

Figure C16. Annual reliability index without and with inspections. Inspection at year 13 with λ=2, 5 and 10 mm, partial safety factor γ_m = 1.00 and aspect ratio = 0.2.

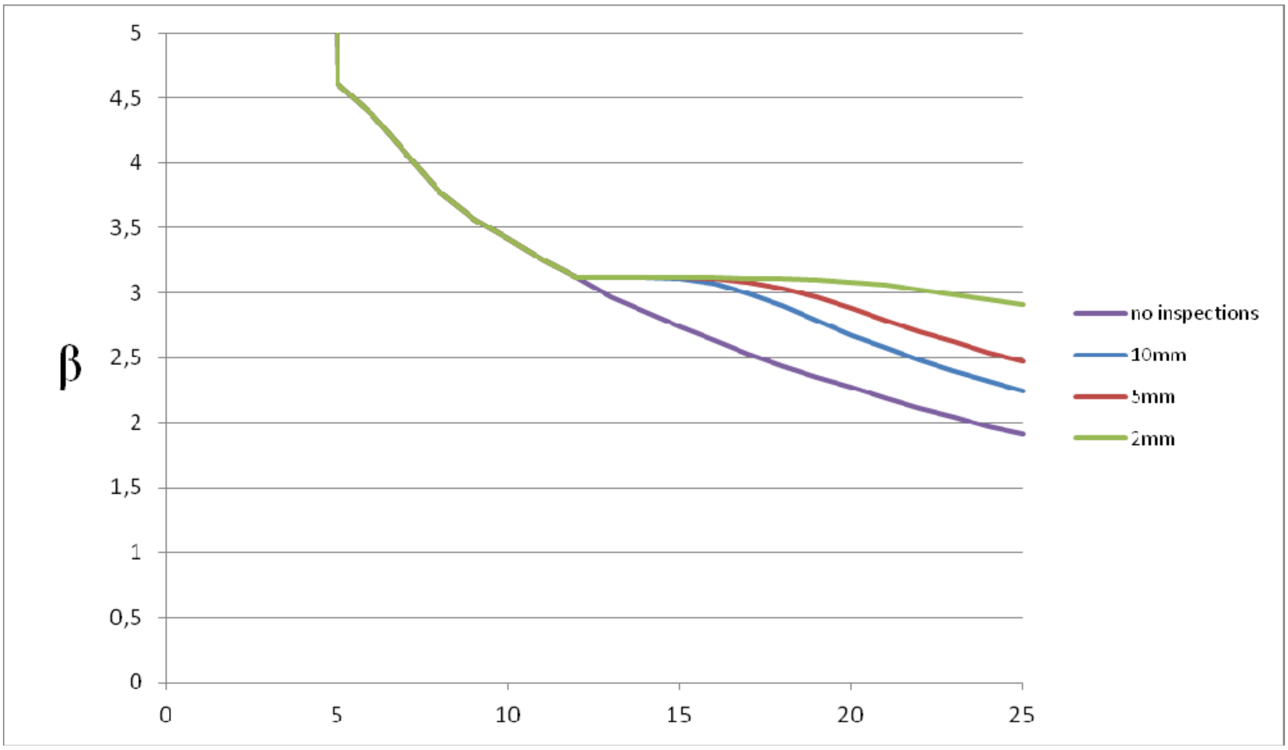

Figure C17. Accumulated reliability index without and with inspections. Inspection at year 13 with λ=2, 5 and 10 mm, partial safety factor γ_m = 1.00 and aspect ratio = 0.2.

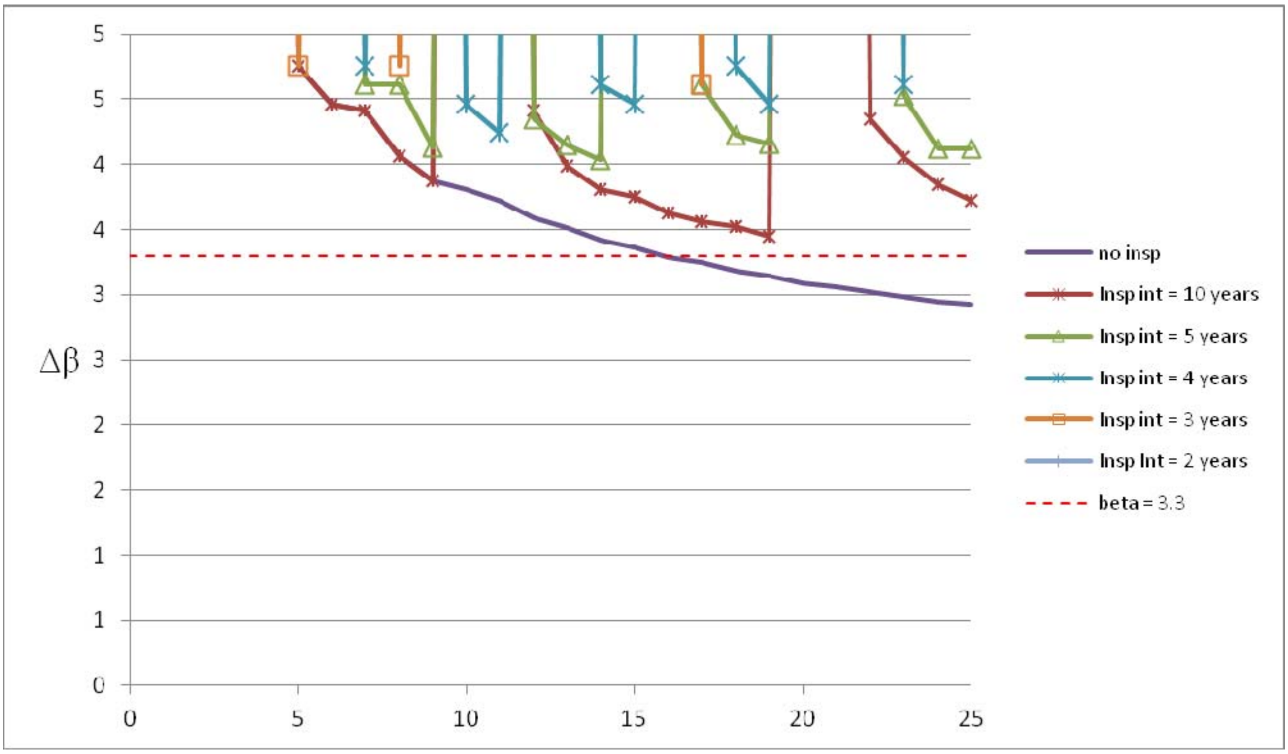

Figure C18. Annual reliability index without and with inspections. Inspection time intervals 2, 3, 4, 5 and 10 years and λ=5 mm, partial safety factor γ_m = 1.10 and aspect ratio = 0.2.

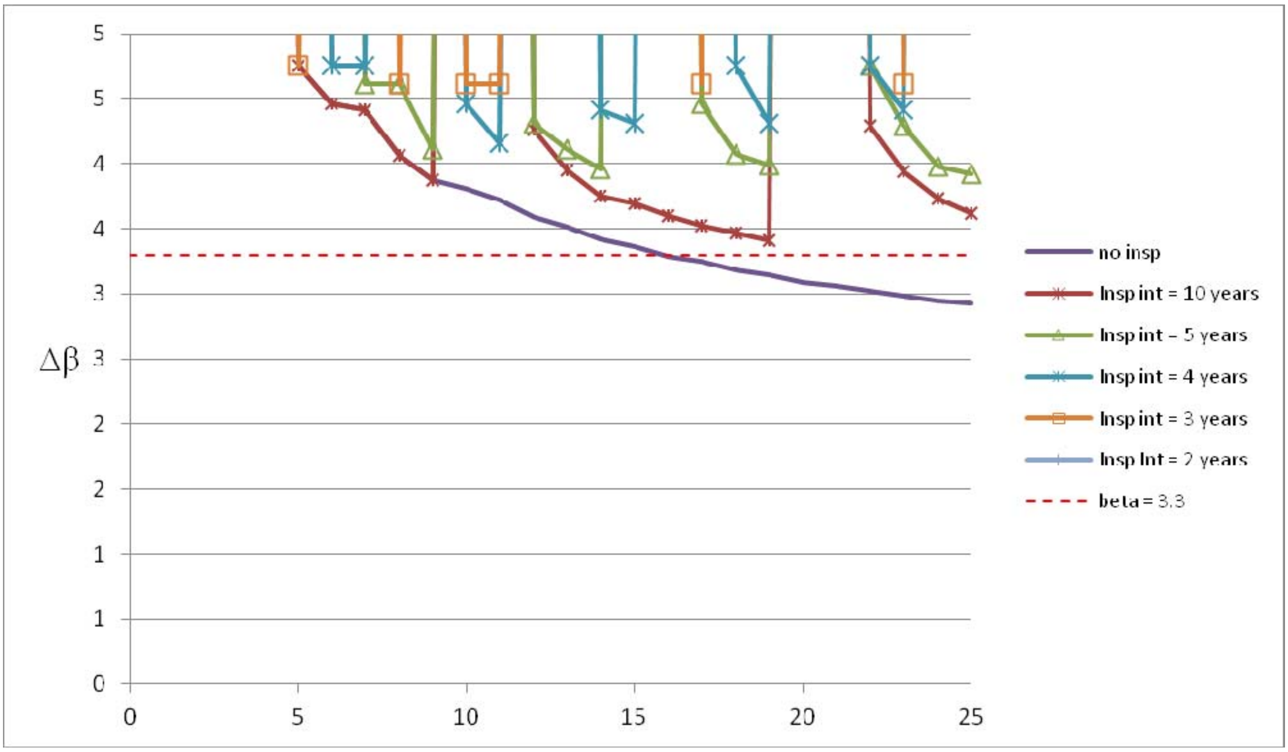

Figure C19. Annual reliability index without and with inspections. Inspection time intervals 2, 3, 4, 5 and 10 years and λ=5 mm, partial safety factor γ_m = 1.10 and aspect ratio defined by eq. (C22).

C5 References

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ANNEX D. CHANGES IN MATERIAL PARTIAL SAFETY FACTORS IN CD IEC 61400-1 ED. 4: 2014.

This annex gives an overview of the main changes in material partial factors in the CD IEC 61400-1:2014 ed. 4 compared to IEC 61400-1:2005 ed. 3.

Clause	IEC 61400-1:2005 ed. 3	CD IEC 61400-1 ed. 4
7.6.1.2	γ_C = 0,9 / 1,0 / 1,3	γ_C = 0,9 / 1,0 / 1,1
7.6.2.2 Ultimate strength analysis		
	$\gamma_m \geq 1{,}1$	$\gamma_M \geq 1{,}2$ Comment: Annex K is added with additional information on material partial safety factors. Comment: generally the material partial safety factor has at least to be equal to 1,2. But the value can be reduced due to additional safety in the failure mode considered. This is e.g. the case for yielding failure of steel (ductile failure with extra load bearing capacity), see below and buckling failure assessed by the parametric formulas in Eurocode 3 part 6 (conservative design equation), see below.
	γ_m = 1,2 for global buckling …	γ_M =1,2 for global buckling of curved shells such as tubular towers and blades Footnote 20 added: The parametric formulas based on membrane theory in Eurocode 3 part 6 (EN 1993-1-6) for shell buckling applicable to tubular steel towers with D/t < 300 includes a bias that may be accounted for by reducing the γ_M for buckling to 1,1.
	γ_m = 1,3 for rupture from exceeding tensile or compression strength.	γ_M =1,3 when materials with no distinct elastic limit (yield strength is more than 90% of the tensile or compression strength) are used
		Added: γ_M =1,1 when materials with a distinct elastic limit (yield strength is less than 90% of the tensile or compression strength) are used.
7.6.3.2 Fatigue	γ_C = 1,0 / 1,15 / 1,3	γ_C = 0,9 / 1,0 / 1,1 Comment: the value for component class 2 is reduced from 1.15 to 1.0 implying that most fatigue material partial factors are increased with a factor 1.15.
	The partial safety factor for	Footnote 21 added:

	loads, γ_f shall be 1,0	It is assumed that the coefficient of variation of the fatigue load stress ranges is less than 20 %. Comment: this implies that the fatigue load partial safety factor becomes dependent on the uncertainty related to the assessment of the fatigue load ranges. For the normal case the fatigue load partial safety factor is 1.0. more information can be found in Annex K.
	γ_m shall be at least 1,5 …	γ_M shall be at least 1,7 … Comment: this is a consequence of changing γ_C from 1,15 to 1,0 for consequence class 2.
	γ_m must be increased accordingly and at least to 1,7.	γ_M must be increased accordingly and at least to 2,0. Comment: this is a consequence of changing γ_C from 1,15 to 1,0 for consequence class 2.
	For welded and structural steel, traditionally the 97,7 % survival probability is used as basis for the SN curves. In this case γ_m may be taken as 1,1. In all cases, γ_m shall be larger than 0,9.	For welded and structural steel, traditionally the 97,7 % survival probability is used as basis for the SN curves. In this case γM may be taken as 1,25, corresponding to a safe-life assessment approach, see Annex K. In cases, where it is possible to detect critical crack development through introduction of a periodic inspection programme, a lower value of γM may be used, corresponding to a damage tolerant assessment approach, see Annex K. In all cases, γM shall be larger than 1,0. Comment: this is partly a consequence of changing γ_C from 1,15 to 1,0 for consequence class 2.
	For fibre composites, … In that case γ_m may be taken as 1,2.	For fibre composites, the strength distribution shall be established from test data for the actual material. The 95 % survival probability with a confidence level of 95 % shall be used as a basis for the SN-curve. In that case γ_M may be taken as 1,35. The same approach may be used for other materials. Comment: this is partly a consequence of changing γ_C from 1,15 to 1,0 for consequence class 2.
7.6.5 Critical deflection analysis	γ_C = 1,0 / 1,0 / 1,3	The partial safety factor for consequences of failure γ_C shall be 1,0 for normal (N) load cases. In order to ensure a sufficient safety margin of the tower clearance, and since sufficient tower clearance is regarded as a

		major criterion for the structural integrity of the entire wind turbine, the partial safety factor for consequences of failure γ_C shall be 1,1 for abnormal (A) load cases.
	The value of γ_m shall be 1,1 except when the elastic properties have been determined by full-scale testing in which case it may be reduced to 1,0.	The value of γ_M shall be 1,1 except when the elastic properties of the component in question have been determined by testing and monitoring in which case it may be reduced to 1,05.
7.6.5		Moreover for load case 1.1 a statistical analysis of maximum tip deflection or minimum tower clearance is mandatory according to clause 7.4.1. … … the ratio of the combined partial factors for loads, materials and consequences of failure minus one to the combined partial factor (i.e. $\frac{\gamma_f \gamma_C \gamma_M - 1}{\gamma_f \gamma_C \gamma_M}$

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We have more than 240 staff members of which approximately 60 are PhD students. Research is conducted within nine research programmes organized into three main topics: Wind energy systems, Wind turbine technology and Basics for wind energy.

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