RFS-CT-2006-00031 - HISTWIN
High-Strength Steel Tower for Wind Turbine

WP1.5 – SEISMIC DESIGN

BACKGROUND DOCUMENT

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1. WORK PACKAGE DESCRIPTION

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2. BACKGROUND DOCUMENT

2.1. General

2.1.1 Fundamental requirements

Steel towers designed to withstand the actions of the earthquake shall conform to the following European Codes:

EC 8-1 : Design of structures for earthquake resistance – General rules, seismic actions and rules for buildings

EC 8-6 : Design of structures for earthquake resistance – Towers, masts and chimneys

The relevant clauses of [GL Wind 2003 IV – Part1] may also be observed.

Both requirements of [EC 8-1 §2.1], no–collapse (ultimate limit state) and damage limitation must be met. The later is considered to have been satisfied if, under a seismic action having a larger probability of occurrence than the design seismic action corresponding to the "no-collapse requirement", the displacements, as calculated according to [EC 8-1 §4.4.3], are limited, in order to prevent permanent damage of the equipment. If no specific information is available by the relevant National Annexes, the owner or the rotor supplier, a reduction factor: ν=0,5 for structures of high importance is recommended by the above mentioned Code. It must be noted though that in practice, a wind turbine tower designed according to the relevant set of Eurocodes (limitation of the 2nd order effects, competent fatigue assessment, consideration of the aeroelastic actions, adequate distance of the natural frequencies of the tower from the excitation frequencies of the rotor blades etc.), no special restrictions to the magnitude of the displacements are usually needed.

2.1.2 Methods of analysis

The seismic actions may be determined on the basis of the linear-elastic behavior of the structure and the reference method shall be the modal response spectrum analysis. As regards the alternative approaches mentioned in [EC 8-1 §4.3.3], it is remarked that:

- Lateral force method of analysis in not applicable in the case of the wind towers, since the contributions from modes of vibration higher than the fundamental are always significant.
- Non-linear time history (dynamic) analysis is predicated on artificial, recorded or simulated accelerograms, which are not always available, at least to the required degree of accuracy and completeness. This method though can be used supplementary as a verification of the modal response spectrum analysis and for the investigation of the response of the structural elements with unilateral behavior (preloaded base anchors, footings susceptible to local uplift etc.).

- Non-linear static (pushover) analysis may be applied for verification purposes only (see [EC 8-1 §4.3.3.4.2.1].

2.1.3 Earthquake motion representation

The earthquake motion is represented by an elastic ground acceleration response spectrum, defined as “elastic response spectrum”. As an alternative, the "design spectrum" may be used, when the capacity of the structure to dissipate energy, through mainly ductile behavior of its elements and / or other mechanisms, is taken into account. It must be noted though that the required compliance with the additional rules for dissipative behavior of [EC 8-1 §6.9] concerning inverted pendulum structures is rather problematic, since the procedure indicated in [§6.5.5-1] for the estimation of the seismic action magnifying factor [Ω] makes no sense in the case of single column structures, in contradiction with the relevant provisions of the previous edition of the Code [preEN 1998-1-3:2000 §3.9-3]. At any rate, this does not seem to be a design issue, since the wind as a rule is the dominant load case [11]. This statement is demonstrated by a working example, given in the table below, where the relevant load combination assumptions are as follows:

- Wind loading as in WP1.3
- Earthquake data: a=0,36, T_B =0.15, T_C=0,60, γ_I=1.40, ζ=2%)

No	Description	G+W	G+1.5W	G+E
1.	**$f_{,top}$** (mm)	845		687
2.	**$V_{,Base}$** (kN)		1,051	665
3.	**$M_{,Base}$** (kNm)		67,652	38,080

Therefore, this fact allows the adoption of the elastic response spectrum for the seismic excitation, where no ductility requirements are involved. However, the seismic analysis must be carried out in any case, especially to areas with extreme seismic data (high risk seismic zone, weak soil, etc).

In chapters [2.2.1] & [2.2.2] the parameters involved to the above mentioned translational spectra, as given by [EC 8-1 §3.2.2], are described. Regarding the rotational component of the ground motion, [EC 8-6 §3.1] states that it must be taken into account for tall structures in regions of high seismicity, as specified by the National Annexes. The suggested by the Code field of application includes the following types of structures and site characteristics:

- [§3.1]: Structures taller than 80m in regions where the peak ground acceleration times the soil factor exceeds the 25% of the gravity acceleration: a_g•S ≥ 0,25•g.
- [§4.2.5]: Structures with height greater than five times the maximum base dimension. Obviously this criterion does not apply to pile foundations.

The method for the quantification of rotational spectra components is described in [EC 8-6 Annex A]. As a justified simplification, the vertical component [$R_z^\theta(T)$] may be disregarded, while the horizontal [$R_h^\theta(T)$] could conservatively be combined with the transitional component (SRSS procedure – see [§2.3]) by means of the relation:

$\sqrt{S_e(T)^2 + \left(R_d^\theta(T) \bullet h_r\right)^2}$ where [h_r] is the hub height

With the exception of towers erected on rock or very stiff soil, the structural model of the steel shell must be built together with the one of the foundation, so that the 2nd order effects and the contribution of the soil–structure interaction are properly taken into account (see [EC 8-1 §4.3.1] & [EC 8-6 §4.2.5]).

2.2. Response spectra

2.2.1 Elastic response spectrum

The shape of the elastic response spectrum [$S_e(T)$] is presented in Figure [F-2.2.1] at the next page. The horizontal translation components of the seismic action are determined by the formulas:

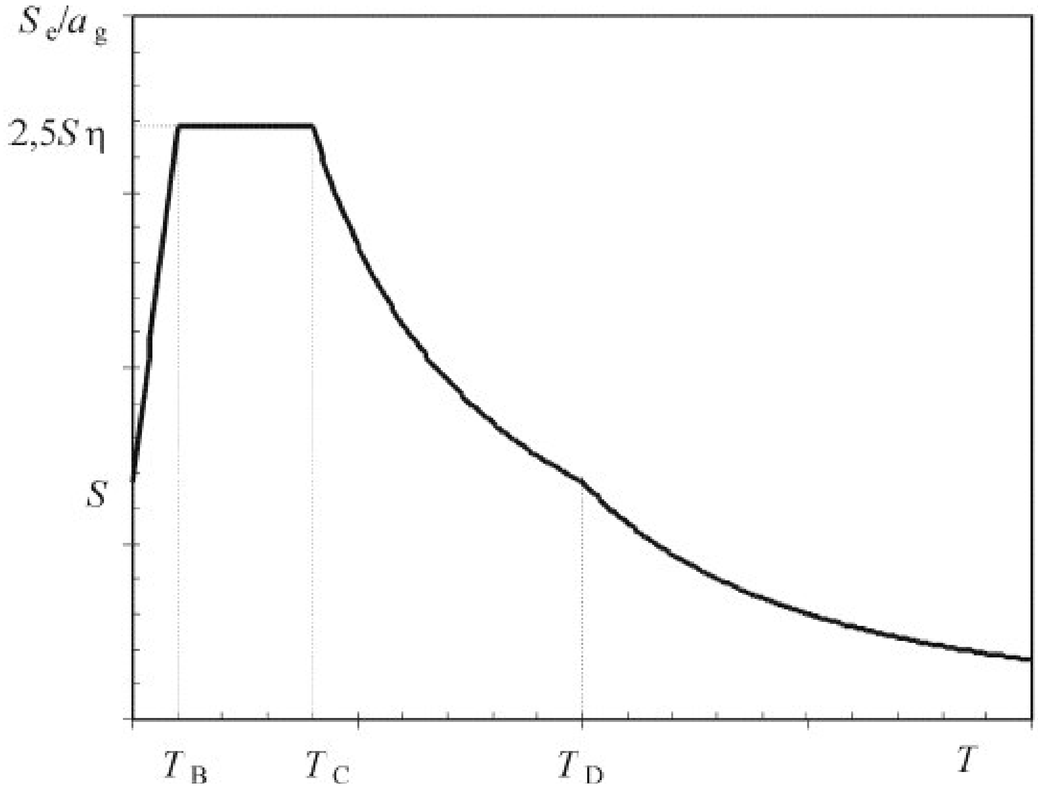

Figure 2.2.1: Elastic response spectrum

$$S_e(T) = \alpha_g \bullet S \bullet \left[1 + \frac{T}{T_B} \bullet (\eta \bullet \beta_o - 1)\right] \qquad \text{when: } 0 \leq T < T_B$$

$$S_e(T) = \alpha_g \bullet S \bullet \eta \bullet \beta_o \qquad \text{when: } T_B \leq T \leq T_C$$

$$S_e(T) = \alpha_g \bullet S \bullet \eta \bullet \beta_o \bullet \left(\frac{T_C}{T}\right) \qquad \text{when: } T_C \leq T \leq T_D$$

$$S_e(T) = \alpha_g \bullet S \bullet \eta \bullet \beta_o \bullet \left(\frac{T_C \bullet T_D}{T^2}\right) \qquad \text{when: } T_D \leq T \leq 4s$$

The damping correction factor [η] and the components of the horizontal design ground acceleration [α_g] can be derived by the relations:

$$\eta = \sqrt{\frac{10}{5+\xi}} \geq 0{,}55 \qquad \alpha_g = \gamma_I \bullet \alpha_{gR}$$

The values of the factors involved in the formulas, are:

α_{gR} : Reference peak ground acceleration on type [A] ground, specified by the National Annexes.

γ_I : Importance factor. For the electric power plants it is recommended by [EC 8-6 §4.1]: γ_I=1,40.

β_o : Coefficient of spectral amplification: β_0=2,50

ξ : Critical damping factor ratio. For welded steel structures the value: ζ=2% is adopted as a rule.

S : Soil factor, given in Tables [3.2] & [3.3] of [EC 8-1] for the specific ground type as classified in Table [3.1] and for the spectra type, depending on the surface–wave magnitude of the earthquakes that contribute most to the seismic hazard defined for the site.

T : Vibration period of a linear single-degree-of-freedom system.

T_B, T_C, T_D : Characteristic values of spectrum, defined in the same way as the soil factor [S].

The vertical component of the seismic action is determined by the same expressions, with the following modifications:

- S = 1,00 βo = 3,00
- T_B = 0,05s T_C = 0,15s T_D = 1,05s
- The horizontal design ground acceleration [α_g] is replaced by the vertical [α_{vg}], as given in Table [3.4] of [EC 8-1].

2.2.2 Design spectrum

The horizontal translation components of the design spectrum [$S_d(T)$] are defined by the following expressions:

$$S_d(T) = \alpha_g \bullet S \bullet \left[\frac{2}{3} + \frac{T}{T_B} \bullet \left(\frac{2,5}{q} - \frac{2}{3}\right)\right] \qquad \text{when: } 0 \le T < T_B$$

$$S_d(T) = \alpha_g \bullet S \bullet \frac{2,5}{q} \qquad \text{when: } T_B \le T \le T_C$$

$$S_d(T) = \alpha_g \bullet S \bullet \frac{2,5}{q} \bullet \left(\frac{T_C}{T}\right) \ge \beta \bullet a_g \qquad \text{when: } T_C \le T \le T_D$$

$$S_d(T) = \alpha_g \bullet S \bullet \frac{2,5}{q} \bullet \left(\frac{T_C \bullet T_D}{T^2}\right) \ge \beta \bullet a_g \qquad \text{when: } T_D \le T \le 4s$$

It is noted that:

- The parameters [a_g], [S], [T_C] & [T_D] are as defined in [§2.2.1].
- The lower bound factor [β] may be taken as equal to 0,2, when not determined by its National Annex.
- The behavior (shape) factor [q] can be assigned with the value: q = 1,50 (see [EC 8-6 §4.7.5, §4.10 & 6.1]), relevant to the cross–sectional Class [4] of the tower shell, according to the categorization of [EC 3-1-1 Table 5.2].

For the vertical component of the seismic action the design spectrum is given by the same expressions, with the following modifications:

- $S = 1,00$
- $T_B = 0,05s$ $T_C = 0,15s$ $T_D = 1,05s$
- The horizontal design ground acceleration [α_g] is replaced by the vertical [α_{vg}], as given in Table [3.4] of [EC 8-1].

2.3. Combinations of seismic actions

The sum of the effective modal masses for the modes taken into account must amount to at least 90% of the total mass of the structure. The combination of the values of the seismic action effects for the governing modal responses will be assessed according to [EC 8-6 §4.3.3.3] and [EC 8-1 §4.3.3.3.2], by application of the SRSS method (square root of sum of squares), provided that the periods in any two vibration modes may be taken as independent of each other ($T_{min} \le 0,9 \bullet T_{max}$). If the above term is not satisfied, more accurate procedures for the combination of the modal maxima, such as the CQC (Complete Quadratic Combination) shall be adopted.

The effects of any rotational component of the ground motion may be combined, if significant, with those of the translational component via the SRSS procedure, since they are not generally in phase. Any rotational components about a horizontal direction should first be combined with those of the translational ones in the orthogonal horizontal direction.

The horizontal [E_{Edx}], [E_{Edy}] and the vertical [E_{Edz}] components of the seismic excitation may be combined in respect with the stipulations of [EC 8-1 §4.3.3.5]:

$\pm 1{,}0 \bullet E_{Edx} \pm 0{,}3 \bullet E_{Edy} \pm 0{,}3 \bullet E_{Edz}$

$\pm 0{,}3 \bullet E_{Edx} \pm 1{,}0 \bullet E_{Edy} \pm 0{,}3 \bullet E_{Edz}$

$\pm 0{,}3 \bullet E_{Edx} \pm 0{,}3 \bullet E_{Edy} \pm 1{,}0 \bullet E_{Edz}$

Regarding the combinations of the seismic action with other actions, it is considered that, instead of the general use approach of [EC 8-6 §4.5] & [EC 8-1 §3.2.4], the more specialized for wind towers provisions of [GL Wind 2003 IV – Part1 §4.4.3.3] should be applied.

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2.4. References

[1] EN 1993-1-1: Design of steel structures – General rules and rules for buildings, 2005

[2] preEN 1993-1-6: Design of steel structures – Strength and Stability of Shell Structures, 2006

[3] EN 1993-3-1: 2006: Design of steel structures – Towers, masts and chimneys – Towers and masts, 2006

[4] EN 1993-3-2: 2006: Design of steel structures – Towers, masts and chimneys – Chimneys, 2006

[5] EN 1998-1: Design of structures for earthquake resistance – General rules, seismic actions and rules for buildings, 2004

[6] preEN 1998-1: Design of structures for earthquake resistance – General rules, seismic actions and rules for buildings, 2000 (withdrawn)

[7] EN 1998-6: Design of structures for earthquake resistance – Towers, masts and chimneys, 2005

[8] GL Wind 2003 IV – Part1: Guideline for the Certification of Wind Turbines, 2004

[9] DIBt: Guideline for Wind Energy Plants, draft version, 2004

[10] ECCS-TC 13: Manual on design of Steel Structures in Seismic Zones, 1994

[11] Analysis and design of a prototype of a steel 1-MW wind turbine tower - I. Lavassas , G. Nikolaidis, P.Zervas, E. Efthimiou, I.N. Doudoumis – Engineering Structures 25 (2003) 1097-1106