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

WP3.1 – EVALUATION OF SHELL THICKNESSES

BACKGROUND DOCUMENT

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1. WORK PACKAGE DESCRIPTION

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2. BACKGROUND DOCUMENT

2.1. General

The evaluation of the shell thicknesses of the tower requires the development of a Finite Element model [1] . The tower geometry and the materials, as described in [§2.2], conform with the REPOWER Systems drawings: [R050405- -SZ (B)] & [R050405-EZ (D)].

An additional structural model [2] was built, mainly to check the analysis results of model [1]. In this case the foundation has been modeled using linear beam grid. The sections of the tower are simulated via linear beam elements.

Lastly, a simplified hand-calculation has also been performed. In this calculation the tower is assumed clamped to the base.

2.2. Description of the FE structural model [1]

1. The present structural design is based on linear elastic analysis [LA] for the wind loading and on spectral response analysis, for the seismic loading. The engineering software used is [Strand7 / Straus7].

2. The overall FE tower model (see Figure [F-2.2-1]) is constructed by the shell, the intermediate flanges and the embedded to the foundation skirt. The reinforced concrete foundation is also modeled.

3. The structural model becomes denser in the vicinity of the flanges, the door opening and the base ring, in order to describe more accurately the local concentration of the stresses. A cylindrical coordinate system is used.

4. The shell is divided along the height into 35 skirts, each of which constitutes an individual FE group. The element arrangement along the circumference is determined by the number of bolts of the connection flanges. Doing so, there is a node at each bolt position.

5. The intermediate flanges (see Figure [F-2.2-2]) are modeled by the use of brick elements. The interfaces of the flanges are connected by means of frictional unilateral contact elements, active only in compression. On bolt positions, the upper node of the upper flange is attached to the lower node of the lower flange (points 1 & 2), via the prestressed linear elements of cable type, active only in tension. Especially for the top flange, the contribution of the nacelle to the horizontal rigidness of the section is achieved by the introduction of master – slave links, converging at center of the circle

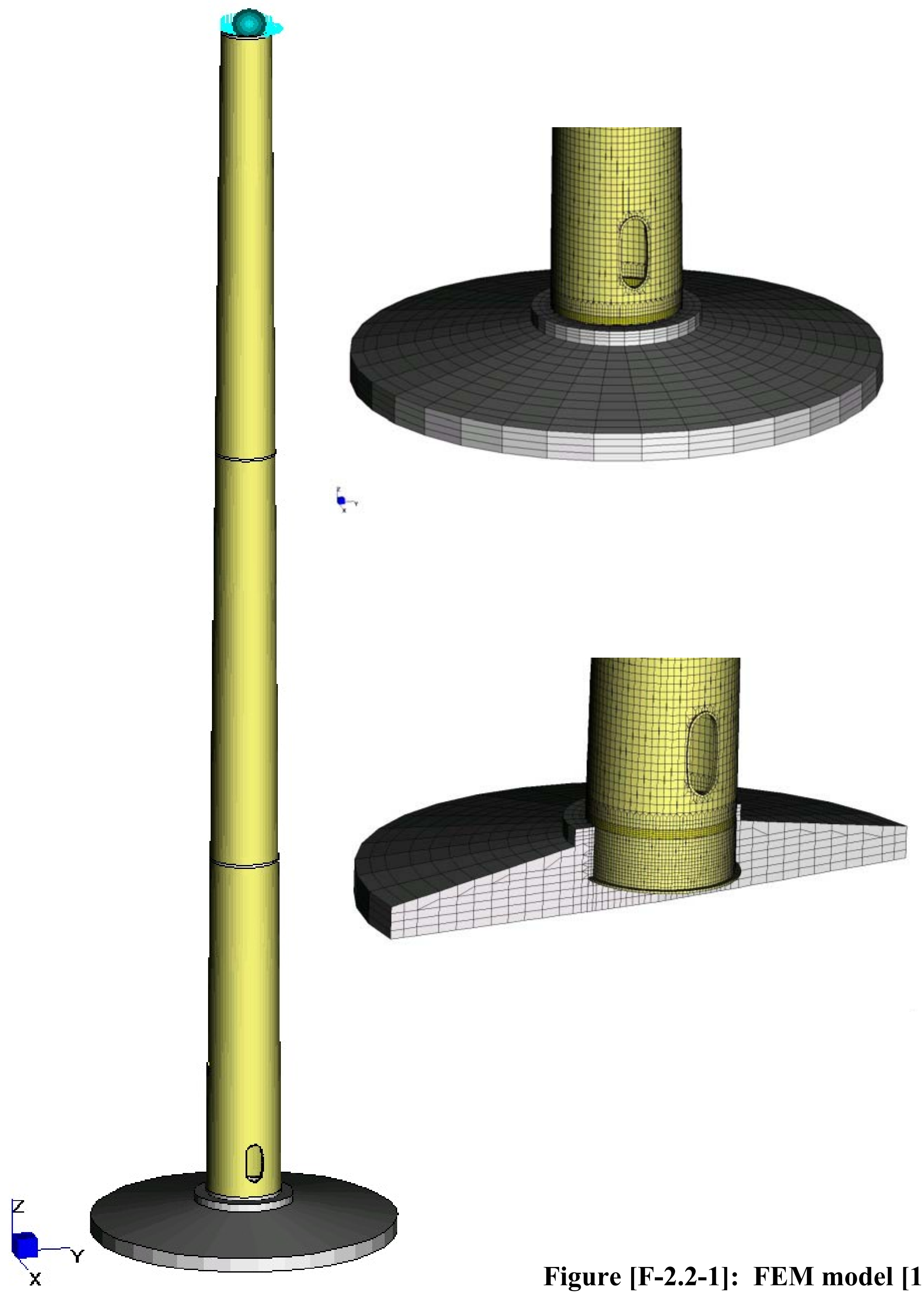
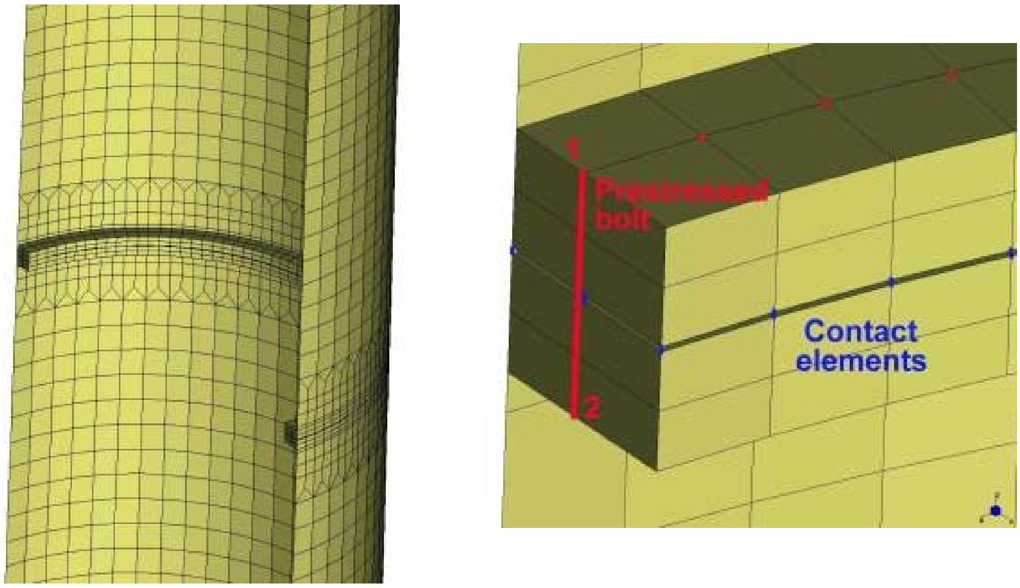


Figure [F-2.2-1]: FEM model [1]

Figure [F-2.2-2]: Detail to the flanges



6. The foundation is modeled together with the tower body. The whole system is assumed to be elastically supported to the foundation base, taking account of the soil-structure interaction. The foundation has been introduced by means of brick elements, elastically supported to the ground, through unilateral contact and friction conditions.

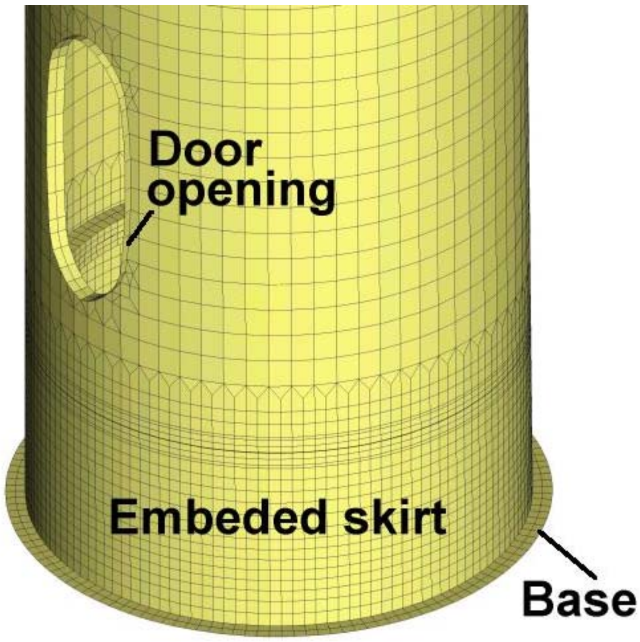


Figure [F-2.2-3]: Detail to the door & embeded skirt

7. Analysis has been performed taking into account material and geometric nonlinearities together with local imperfections (GMNIA analysis, EC 3-1-6 §8.8).

2.3. Loading

2.3.1. Vertical loads [G]

1. The self weight of the shell is estimated directly by the FE software, as a function of the geometry and the unit mass (γ_s=7.850kg/m^3) of the steel elements. The contribution of the platforms and the ancillary equipment (ladders, cable racks etc.) to the total weight of the tower is disregarded at this stage.

2. As provided by REPOWER, the weight of the nacelle, including the blades and the rotor, is equal to: G_r=1067,00kN, having the center of gravity shifted horizontally +0,7250m from the axis of the tower and vertically +0.500 to +1.00 m above the upper flange level (+76.15 m).

2.3.2. Wind loads [W]

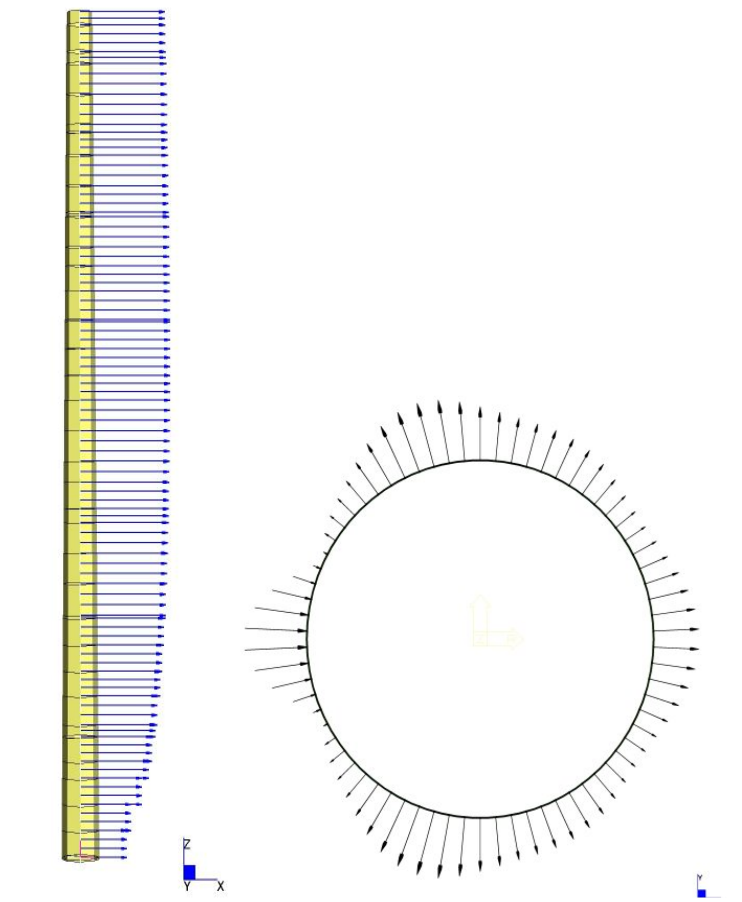


Figure [F-2.3.2]: Wind load distribution over the tower height & circumference

1. The wind loads of the tower turbine are divided into two parts; the loads at the top of the tower and the loads over the tower stem (see Figure [F-2.3.2]).

2. The loads at the top of the tower, transferred by the nacelle, are the upwind horizontal force [F_{wr}] and the overturning moment [M_{wr}] about the coplanar to the blades horizontal axis. These loads are provided by the manufacturer (see [WP 1.1]).

3. The loads over the tower stem are calculated, for the specific dynamic characteristics and geometry of the structure, according to [EC 1-1-4] for a basic wind velocity at 10m

above ground of: v_b=27,00 m/sec and for a terrain of category [II].

4. The distribution of the wind forces along the height [z] of the shell is given as a function of the diameter [D], by the equations (z, D in [m], F_W in [kN/m]):

   D = -0,01775•z+4,30266

   $z \leq 2{,}00m$: $F_W = 0{,}51 \cdot D$

   $z > 2{,}00m$: $F_W = 0{,}013 \cdot \ln(20 \cdot z) \cdot [\ln(20 \cdot z) + 7] \cdot D$

5. The distribution of the wind forces along the circumference of the shell can be expressed as a function of the angle [α], by the equation:

   $$c_f(\alpha) = -0{,}226 - 0{,}180 \cdot \cos(\alpha) + 0{,}240 \cdot \cos(2\alpha) - 0{,}171 \cdot \cos(3\alpha) + 0{,}027 \cdot \cos(4\alpha) - 0{,}042 \cdot \cos(5\alpha) + 0{,}007 \cdot \cos(6\alpha) - 0{,}051 \cdot \cos(7\alpha) + 0{,}026 \cdot \cos(8\alpha)$$

2.4. Simplified Structural model

1. The simplified model is actually a cantilever beam (see Figure [F-2.4]). Since this is a statically determinate system, we can easily calculate by hand the shear force and the bending moment at every point. Calculation of tower deformations is quite difficult to be performed by hand, due to the varying moment of inertia along the height of the tower.

2. For the self-weight, the reactions V,M at the base are:

   Load eccentricity: e_x=0,725 m, e_z=+0,50 m

   V=0,0 kN

   M=1.067•0,725=773,60 kNm (due to the eccentricity of the load)

3. The total weight of the tower (including flanges & stiffeners) is: 1.422 kN

4. Wind loads on the tower top are producing the following reactions:
   V=598,74 kN , M=1.665,41+598,74*(76,15+e_z)=47.558,83 kNm

5. The corresponding wind forces over the height are (as above):

   $F_{W1}(z) = 0{,}51 \cdot (-0{,}01775 \cdot z + 4{,}30266)$, z < 2,0 m

   $F_{W2}(z) = 0{,}013n(20 \cdot z) \cdot [\ln(20 \cdot z) + 7] \cdot (-0{,}01775 \cdot z + 4{,}30266)$, z >= 2,0 m

6. Base reactions are resulting from the integration of the load functions:

   $$V = \int_{z=0}^{2.0} Fw1(z) \cdot dz + \int_{z=2}^{76.15} Fw2(z) \cdot dz = 4{,}37+298{,}19=302{,}56 \text{ kN}$$

   $$M = \int_{z=0}^{2.0} Fw1(z) \cdot z \cdot dz + \int_{z=2}^{76.15} Fw2(z) \cdot z \cdot dz = 4{,}36+12.023=12.027{,}36 \text{ kNm}$$

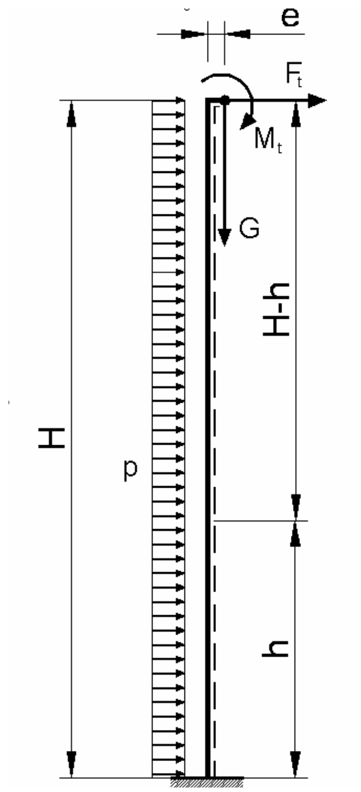


Figure [F-2.4]: Simplified structural model

7. The total wind forces at the tower base are:

   N=1422,00+1.067,00=**2.489,00 kN**

   V=598,74+302,56=**901,30 kN**

   M=47.558,83+12027.36=**59.587 kNm**

8. Similarly, for the load combination: [G+1.50•W] we have:

   N=2489,00 kN

   V=0,00+1,5• 901,3=**1.351.95 kN**

   M=1,5•59.586,19-773,58=**88.606 kNm**

9. For the comparison with the results given by the manufacturer, an overall safety factor 1.35 has been applied (see WP1.1, Extreme Load tables)

   N=1,35*2.489=**3.360,15** ***kN***

   V=0.00+1,35• 901,30=**1216,76 kN**

   M=1,35•(59.586,19-773,58)=**79.397 kNm**

10. The corresponding values provided by REPOWER (WP1.1) along the main loading direction are (including overall safety factor 1,35):

On tower top:

N=1.457,30 kN

V=808,30 kN

M=2.248,30 kNm

On tower bottom:

N=3.174,70 kN

V=873,20 kN

M=67.796,30 kNm

11. The deviations to the results arise mainly from the different assessment of the stem load. As provided by the manufacturer, the influence of the wind load on the tower stem to the total overturning moment at the tower base is about 5.5%. The corresponding influence according to the [EC1-1-4] approach to the stem load is about 20%.

    It is noted that the wind pressure as derived by [EC1-1-4] equates approximately: 1,23 kN/m^2 uniform loading to the tower stem.

2.5. Structural Analysis Results

2.5.1. *Eigenvalue analysis*

The governing eigenfrequencies for each model and for mass at the tower top as in [§2.3.1] are presented in Table [T-2.5.1] and Figures [F-2.5.1-1&2] .

Table [T-2.5.1]

Dynamic characteristics	FE Model		Linear Model	
	k=15400	k=14950	k=15400	k=14950
1st eigen frequency (sec^{-1})	0,357	0,357	0,324	0,322
3rd eigen frequency (sec^{-1})	2,820	2,822	2,626	2,610
9 th eigen frequency (sec^{-1})	7,520	7,490	7,850	7,833

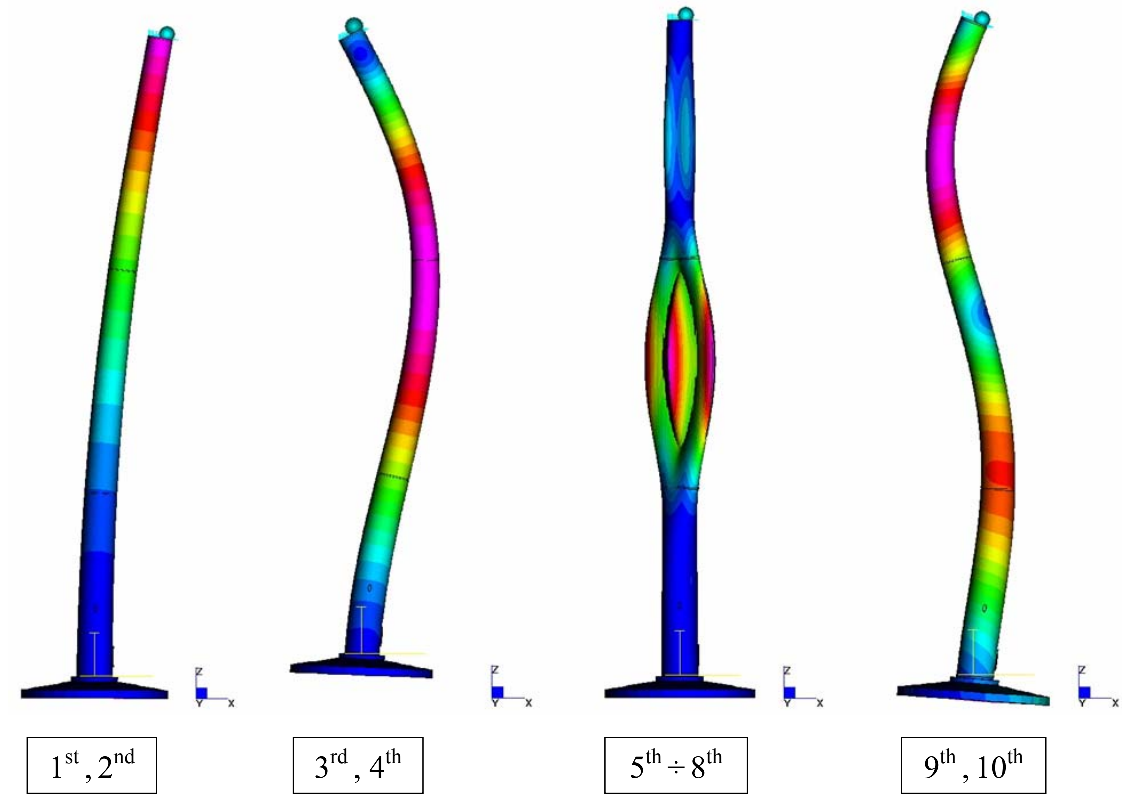


Figure [F-2.5.1-1]: FE Model: Mode shapes

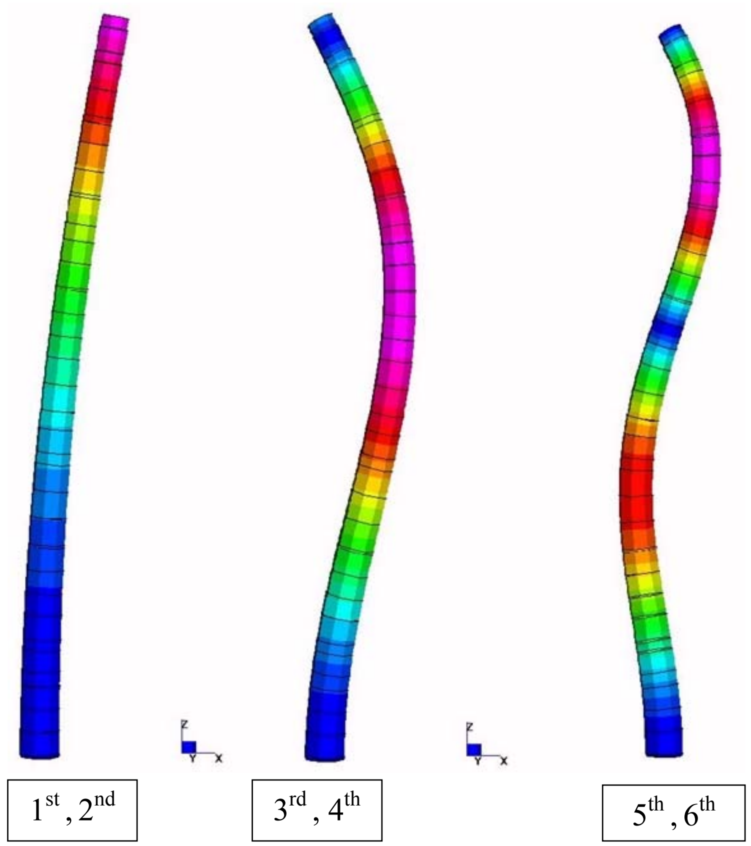
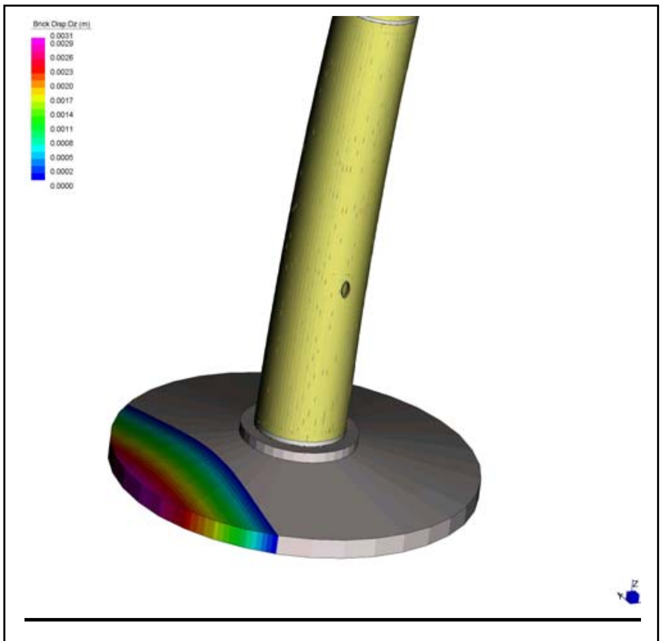


Figure [F-2.5.1-2]: Linear Model: Mode shapes

2.5.2. *Serviceability check*



1. The total horizontal displacements on the top of the tower, for the load combination [G+W] are (see Figure [F-2.5.2-2]:

   FE model: dx=1,195 m

   Linear model: dx=1,391 m

2. For the above mentioned combination an uplift of about 20% of the foundation is recorded, which leads to a maximum vertical displacement of 0,3mm (see Figure [F-2.5.2-1]).

Figure [F-2.5.2-1]: Uplift of the foundation

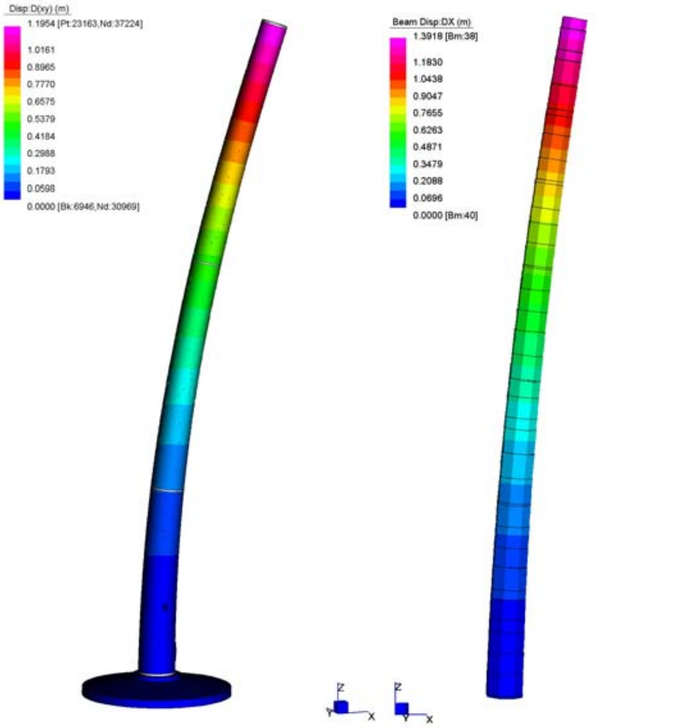


Figure [F-2.5.2-2]: Tower displacements – FE and linear model

2.5.3. *Bending moment & shear force distribution*

1. The cross sectional [N], [V], [M] diagrams are presented for the Linear model (see Figure F-2.5.3]). At the tower base the resultant values are:
   - Load combination [G+1,50•W]
     N=2.405,00 kN

     V=1.346,74 kN

     M=88.798,36 kNm
   - Load combination [1,35•G+1,35•W]
     N=3.246,90 kN

     V=1.212,06 kN

     M=79.574,85 kNm
2. It must be mentioned that the values above are a close match to the ones calculated by hand on the Engineering model.
3. For the Finite Element model it is not feasible to obtain directly the corresponding diagrams. For the sake of comparison, the values of [N], [V], [M] have been assessed at the tower base by integration of the resulting stresses:

$$N= \int_{\theta=0}^{2\bullet\pi} \sigma z \cdot t \cdot R \cdot d\theta = 2.453{,}40 \text{ kN}$$

$$V= \int_{\theta=0}^{2\bullet\pi} \sigma x \cdot t \cdot R \cdot d\theta = 1342.25 \text{ kN}$$

$$M= \int_{\theta=0}^{2\bullet\pi} \sigma z \cdot t \cdot x \cdot R \cdot d\theta = 88731.22 \text{ kNm}$$

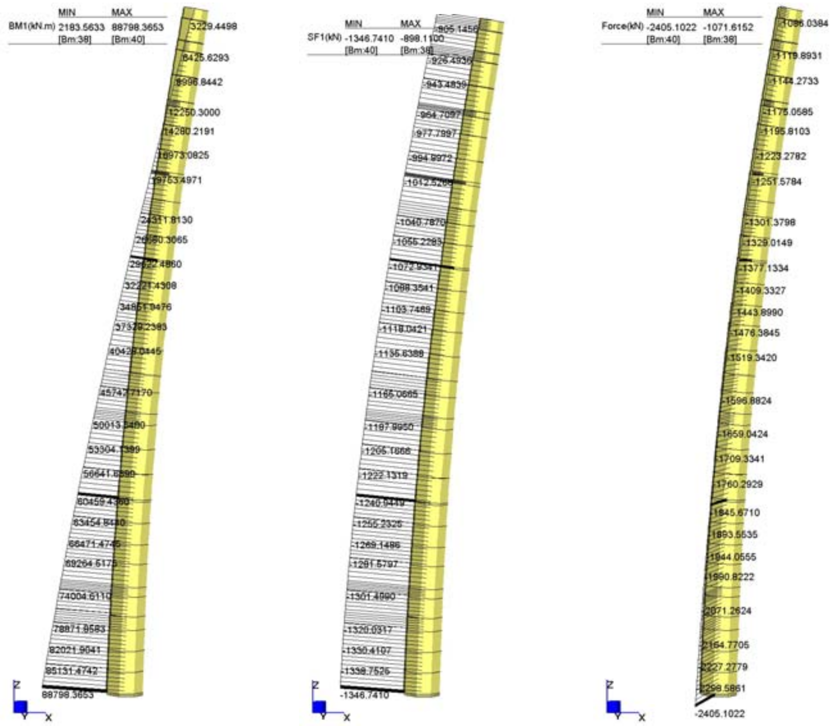


Figure [F-2.5.3]: Linear model – Moment, Shear & Axial Force diagrams to the tower

2.5.4. *Plastic limit state design*

1. High strength steel is used to the shell and flanges of the tower (f_y=355 MPa).

2. Investigating the Finite Element model results, the maximum Von Mises stress on the tower shell has been found equal to 348 MPa, at the vicinity of the door. For the main body of the tower, the maximum Von Mises stress is lower, reaching 293 MPa. It is worth mentioning that there is an almost uniform distribution along the height of the tower between courses [S5] and [S18].

3. Regarding the Linear Model, the maximum Von Mises stress is 251 MPa. As shown if Figure [F-2.5.4], there is a uniform distribution of the stresses for the two lower

courses of the tower, excluding the two first at the bottom, which have been stiffened because of the presence of the door. The maximum values are located between courses [S5] and [S12].

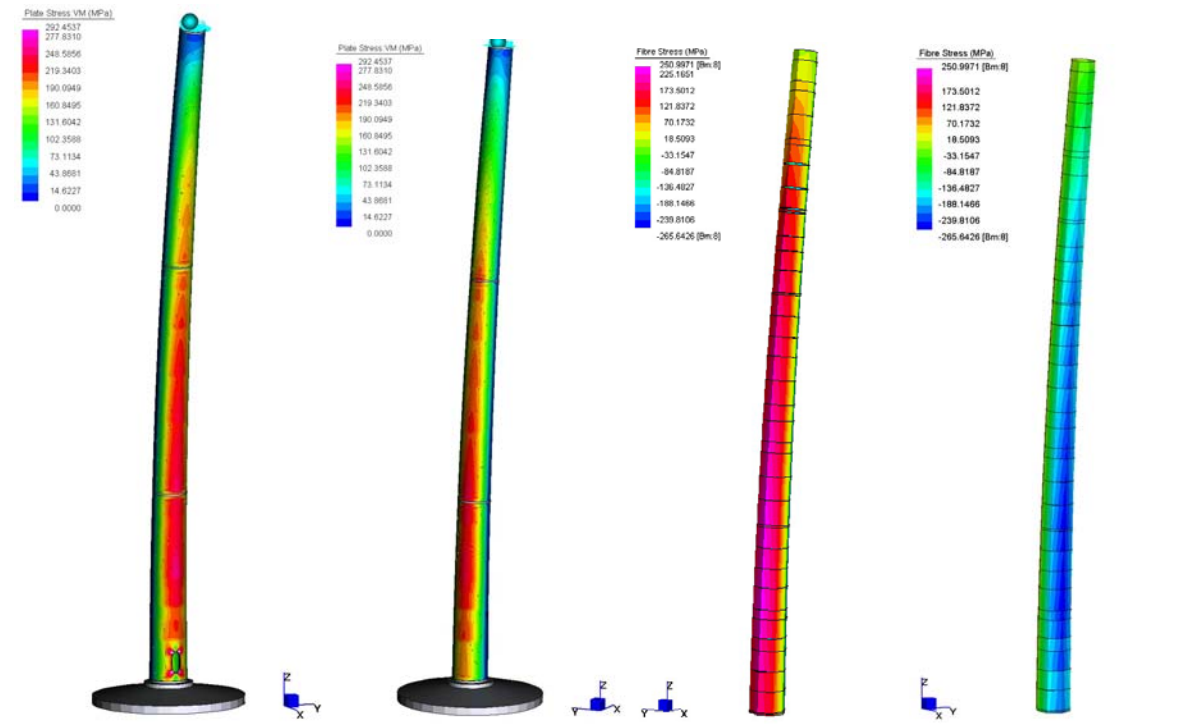


Figure [F-2.5.4]: Von Mises Stress (FEM & Linear model)

2.5.5. *Buckling analysis*

Linear buckling analysis has been performed for the tower, as described more analytically in [WP3.2]. The first 10 buckling eigenmodes are concentrated at the upper part of the tower, equally distributed between the courses [22] and [27] (see Figure [F-2.5.5]). The first buckling eigenvalue for the perfect shell is 3.44.

2.5.6. *Response spectrum analysis*

1. The Response Spectrum analysis for the tower has been performed, as presented in [WP1.5], for the following parameters:

   $a=0.24$, $T_B=0.10$ $T_C=0.40$, $\gamma_I=1.40$, $\zeta=2\%$

2. For the assessment of the effects due to the combination of the components of the seismic action, the SRSS procedure was adopted.

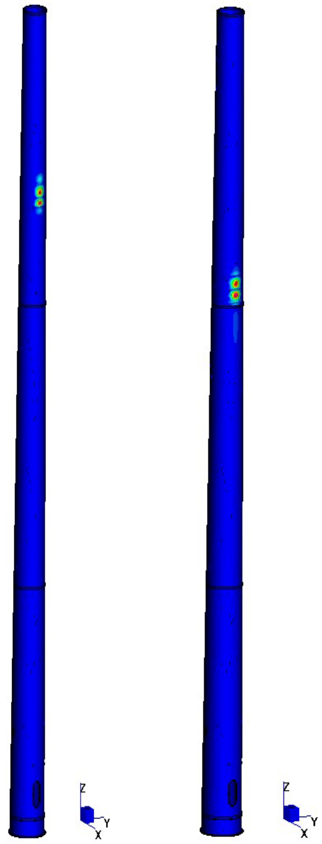


Figure [F-2.5.5]: Buckling modes [1] & [10] for the perfect shell

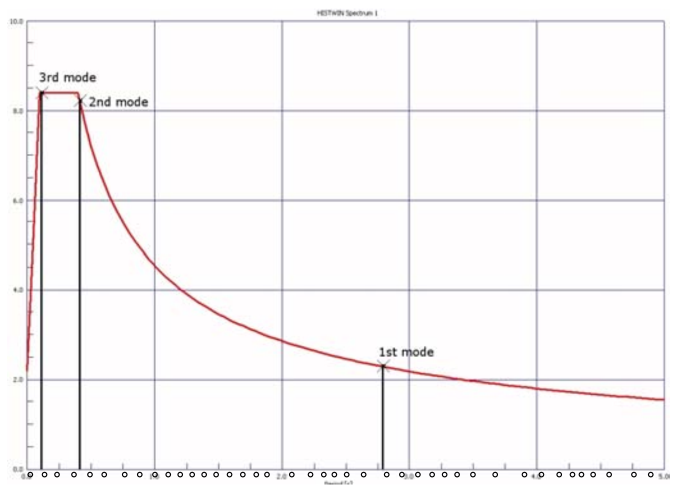


Figure [F-2.5.6-1]: Design spectrum applied on the tower

3. The total participating mass, not considering the contribution of the foundation, is about 93% in all directions. The governing eigenvalues are as follows:
   - Direction vector [1 0 0]:
     1st: 61,7%
     3rd: 14,9%
     5th: 4,9% (FE model: 9th)
   - Direction vector [0 1 0]:
     2nd: 67,2%
     4th: 16,3%
     6th: 6,0% (FE model: 10th)
4. The maximum seismic displacement at the top of the tower is 0,53m.

5. The total shear [V] and overturning moment [M] at the tower base are:

   V=499,33 kN

   M=25.449,32 kNm

   which correspond to approximately 30% of the relevant values of the extreme wind load combination.

6. The dominant stress components for the seismic loading are the meridional ones.

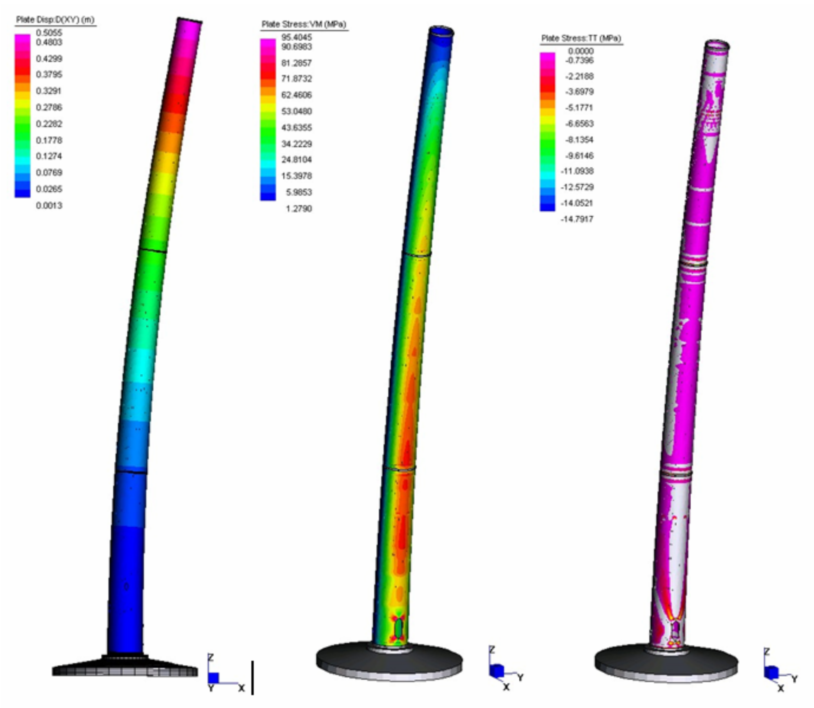


Figure [F-2.5.6-2]: Displacements, Von Mises stresses and compressive circumferential stresses (seismic loading – spectrum analysis)

2.6. Conclusive remarks

1) The methods used for the evaluation of the shell thickness were:

- Linear static analysis (LA)
- Static GMNA & GMNIA analysis
- Linear buckling analysis

- Eigenvalue analysis
- Response spectrum analysis

2) Linear model (as well as the calculation by hand, given that the wind tower is a statically determinate structure) are sufficient for estimating the forces on the tower, but they are inadequate to acess the total stress state on the tower shell and in detail, especially because of the complicated distribution of the wind force over the tower circumference. Stress state on the tower details may be evaluated by the use of additional Finite Element models together with hand calculation or a linear model, but in this case the boundary conditions on the additional FE models need to be estimated. A complete FE model for the tower may need more computation effort, but it provides a complete stress state, when accomplished with a simplified model and hand-calculation for the check of the results.

3) There is a difference on the participation of the tower stem forces to the total force acting on the tower between EC1 computing procedures (20%) and the manufacturer (5.5%) data.

4) Based on the previously presented analyses, the two lowest sections of the tower (courses [S5] to [S21] with thicknesses t=24 to 16 mm) are mainly designed against the plastic limit state, having a uniform distribution of maximum Von Mises stress. Courses [S1] & [S2] (t=30 mm) seem to be ovedesigned considering the tower as a linear model, or a FE model without openings, but their dimensioning is governed by the local stress concentration on the shell around the door. Courses [S3] & [S4] are commutative for the smooth distribution of stresses. Courses [S22] to [S33] with thicknesses t=15 to 12 mm are mainly designed for buckling. Top course [S35] has a thickness t=18 mm in order to provide smooth transfer of the top forces to the tower body, and [S34] (t=14 mm) is commutative.

5) The critical loading for the tower design is the extreme wind loading. The seismic loading lead to a stress-state of about 30% of the corresponding one extreme wind loading.

References

[1] EC 2-1-1: Design of concrete structures – General rules and rules for buildings, 2004

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

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

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

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

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