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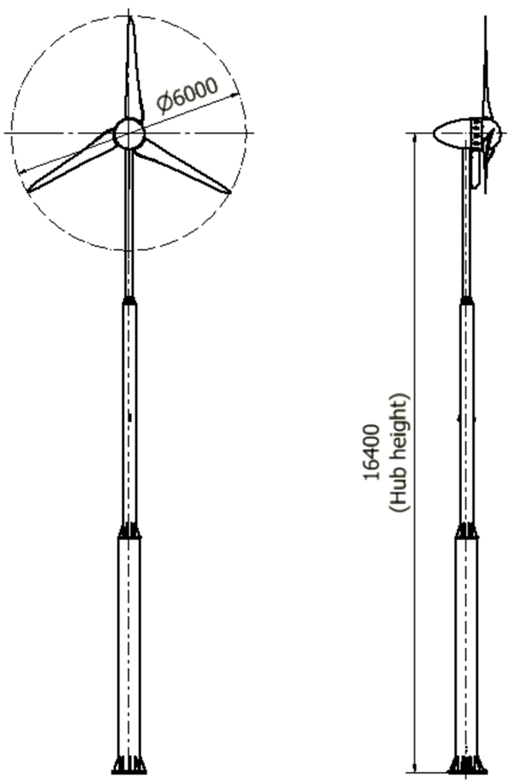
5 kW wind turbine

Static Stability Specification



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Contents

List of Changes

Rev	Modified parameter	Modified by	Date of modification
00	Report written	Pabut	31.03.2014

Appendixes

No.	Name	Revision
01	ICE 61400-2 2006 Wind turbines part 2; Design requirements for small wind turbines	-
02	5KW-MD-01.08.00.00.00-0 Tower selfstanding 16m_merged	0
03	5KW-MD-01.07.00.00.00-0 Selfstanding foundation 16m_merged	0
04	EN 1993-1-3: 2006 Eurocode 3: Design of steel structures – Part 1-8: Design of joints	-

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1. Introduction

Overview of the general stability of the my!Wind 5 kW turbine is given in this document. The acting loads of the system are based on appendix 01 and designate the turbine to wind class IIa. The calculations are based on the simplified load equations model approved by the ICE.

The turbine is erected on a self-standing steel tubular mast and attached to foundation block that is located on the ground. In order to fulfil the stability requirements according to appendix 01 the strength characteristics of the self-standing mast have to exceed the stresses introduced by the acting ultimate loads. The foundation block must remain stable against tilting, up to designated wind loads. The used bolts have to withstand stresses induced by the acting loads and allow no slipping of the bolted joints. In addition the embedded foundation bolts have to withstand the tear out force.

FEM analysis and empirical formulas are used to determine the final stability of the turbine and foundation system. The reaction forces for the bolted connections are taken from the FEM model and the strength of the connections is calculated according to the empirical formulas from appendix 04.

2. Geometry

The turbine consists of the on ground foundation block, self-standing mast and a tower top. All the acting loads are calculated according to the turbine hub height of 16,4 m.

The tower consists of a base plate, lower segment, middle segment and upper segment. All of these are welded assemblies of regular strength steel, which are connected by regular 8.8 bolts with electro galvanic zinc coating (appendix 02).

Tower specification:

First segment diameter	559 mm
First segment wall thickness	6,3 mm
Second segment diameter	323,9 mm
Second segment wall thickness	6 mm
Third segment diameter	193,7 mm
Third segment wall thickness:	6 mm
Overall weight	1 280 kg

The foundation block consists of a casted concrete body with inner load bearing structure from steel and steel reinforcement bars. The purpose of the concrete is to act only as a weight to counter the

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tilting moment form the trust forces. The unit is designed in a way that all the directly acting loads are received by the steel structure (appendix 03).

Foundation block specification:

Length	3,1 m
Width	3,1 m
Height	0,4 m
Weight (including reinforcement)	9,97 t

3. Materials

Tower – Steel S355J2H:

Young's modulus	200 GPa
Poisson's ratio	0,3
Tensile yield strength	355 MPa

Foundation body – Concrete C20/25:

Density	2500 kg/m3
Compressive strength	20 MPa

Foundation inner structure – Steel S235JR:

Young's modulus	200 GPa
Poisson's ratio	0,3
Tensile yield strength	235 MPa

Foundation reinforcement – ASTM A615 grade 60:

Young's modulus	200 GPa
Poisson's ratio	0,3
Tensile yield strength	420 MPa

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4. Loads

The acting loads on the tower and the foundation are calculated according to appendix 01 point 7. Load assumptions are derived from so called simplified load model as the turbine fulfils all the requirements to fall under described category. In this instance only the load cases that have influence on the tower and foundation stability described in the report. For some load cases it is easy to see from the simplified formulas that in terms of stability for the carrier structure, they are of minor significance. For load case "I: Parked wind loading, maximum exposure" only the most critical situation is analysed. In order to calculate the tilting moment at tower base, each load is multiplied by its arm. Summary of the acting loads for each load case and corresponding tilting moments together with induced stresses are represented below.

Table 1 – Acting loads and resulting reactions

Considered load cases / Acting loads	A: Normal operation	B: Maximum thrust	H: Parked wind loading	I: Parked wind loading max exp.
Thrust force	801 N	3 910 N	2 351 N	3 472 N
Turbine head weight	2 570 N	2 570 N	2 570 N	2 570 N
Tower weight	12 557 N	12 557 N	12 557 N	12 557 N
Third segment wind	0 N	0 N	1 176 N	600 N
Second segment wind	0 N	0 N	2 950 N	1 505 N
First segment wind	0 N	0 N	5 091 N	2 597 N
Nacelle cover wind	0 N	0 N	738 N	377 N
Results				
Tilting moment at third segment	4 256 Nm	17 652 Nm	16 240 Nm	17 779 Nm
Tilting moment at second segment	9 060 Nm	41 112 Nm	50 909 Nm	48 984 Nm
Tilting moment at tower base	13 864 Nm	64 573 Nm	109 473 Nm	92 497 Nm
Tilting moment at foundation base	14 184 Nm	66 137 Nm	114 396 Nm	95 918 Nm
Nominal stress at third segment base	26 MPa	110 MPa	101 MPa	110 MPa
Nominal stress at second segment base	19 MPa	88 MPa	109 MPa	105 MPa
Nominal stress at first segment base	9 MPa	43 MPa	73 MPa	62 MPa

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5. Calculations

5.1 Tower strength

FEM model is modelled with 4 solid parts: base, first segment, second segment and third segment. The turbine head weight is represented by a point mass which is located 314 mm from the tower centre in horizontal direction (z axis). The parts are connected to each other with bonded contacts that have bolt washer sizes. These connection areas are used to derive the reaction forces for all of the bolts that connect the bodies to each other. The areas where bodies touch but are not connected in any way have frictionless support. The base plate is grounded with 5 fixed supports that are of bolt washer size. These areas are used to derive the reaction forces for the embedded foundation block bolts. In order to achieve more realistic results in the highly stressed areas where the segments meet, welding seams have been added to the model in order to avoid singularity effects. This is used to provide a more even load distribution at intersection of elements. The acting loads are taken from Table 1 and applied considering their nature and direction (see Figure 1).

Only load case H: Parked wind loading is presented in the report, as it results in the highest overall stresses for the connection areas of the segments and the connection area for the base plate.

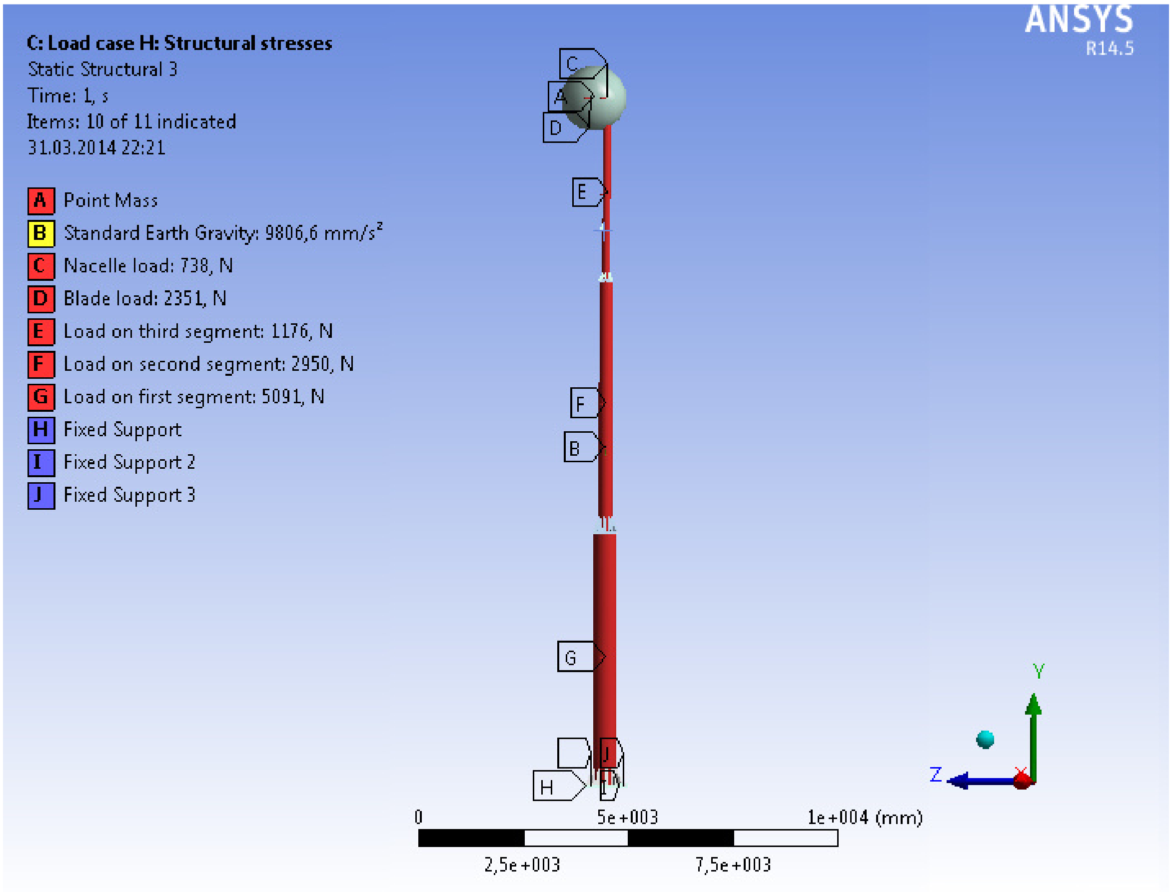

Figure 1 - Boundary conditions

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Tetrahedron mesh is used with a dominant element size of 15 mm which results in a collective mesh of 780 512 elements and 1 501 674 nodes. On the strengthening ribs, the mesh is refined with element size of 8 mm (see Figure 2).

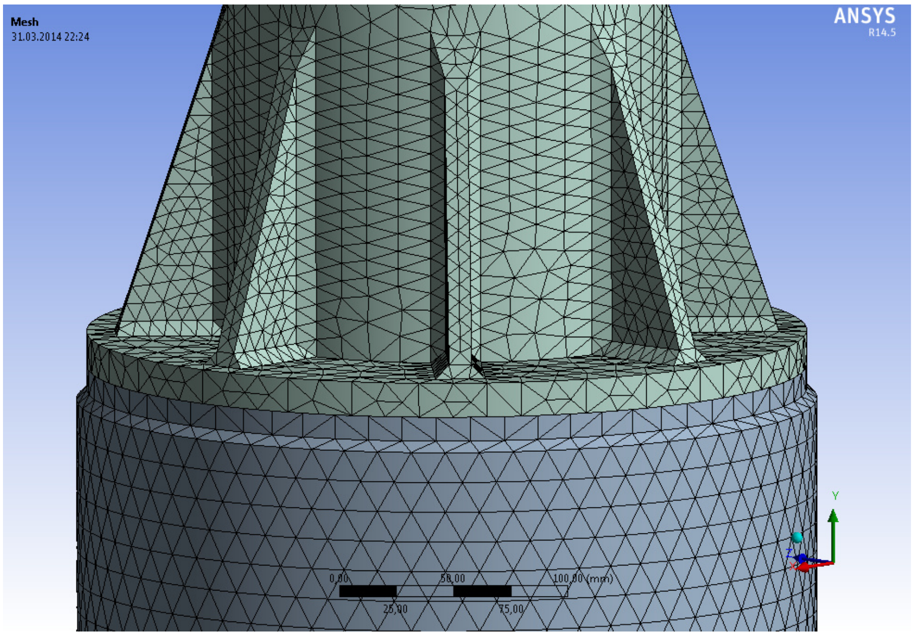

Figure 2 – Mesh

The highest stresses to be considered occur on the strengthening ribs of the first segment (see Figure 3). These values go up to 206 MPa. Two factors have to be taken into consideration when considering stresses at the particular point. Firstly, the high stress value is partially generated by the nature of FEM analysis. Secondly, in reality the spot will have a welded joint which is prone to defects. For safety reasons the highest occurring stress is considered in the safety factor calculation and no compensation for the singularity effect is used. In other components the occurring stresses are lower (see Figure 4 and Figure 5). The highest absolute stress occurs on the base plate near the bolted connection area (see Figure 6). This value goes up to 236 MPa. However, the high stress in this region is mainly caused by modelling the bolted connection with a fixed support (all six degrees of freedom removed). In reality the area would be subjected to lower stresses, as the material would be able to move in some directions and local stiffening effects would not take place. As the value of the stress is still in allowable limits, the area is not subjected to more thorough analysis.

The allowed stress according to appendix 01 is $\frac{355}{1{,}1\cdot 1{,}35} = 239\ \mathrm{MPa} > 206\ \mathrm{MPa}$ (OK). See appendix 01 – partial safety factor for materials 1,1; partial safety factor for loads 1,35.

Therefore the tower strength against ultimate load cases is guaranteed.

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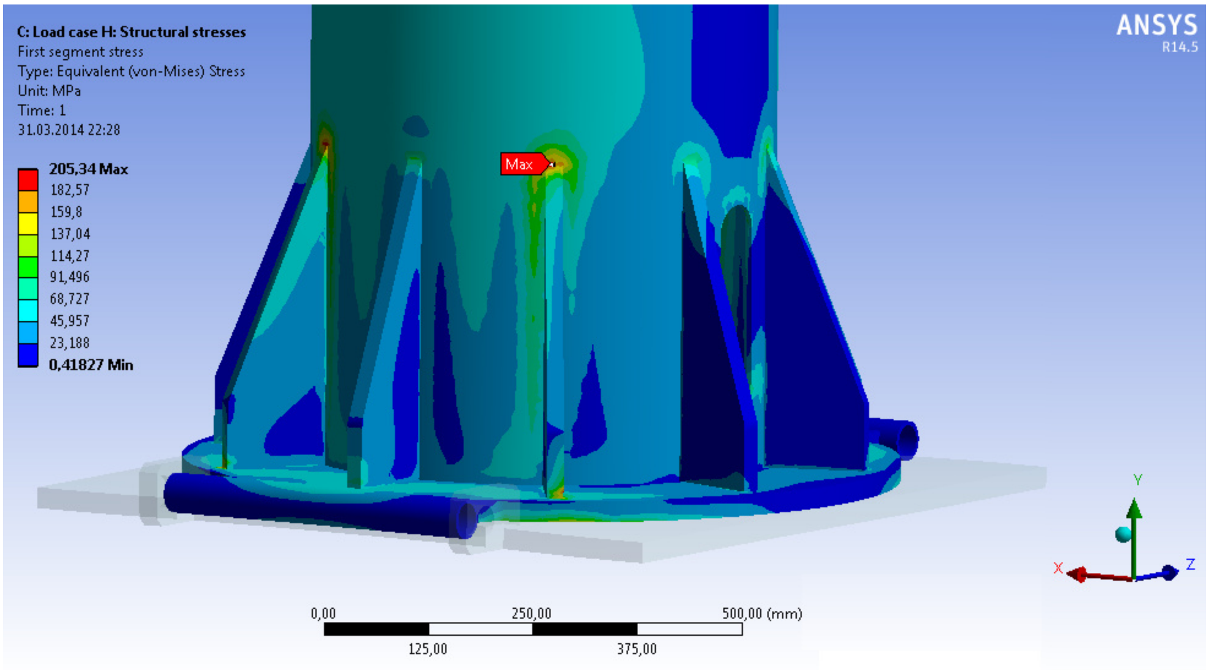

Figure 3 – Stress at base plate and first segment connection

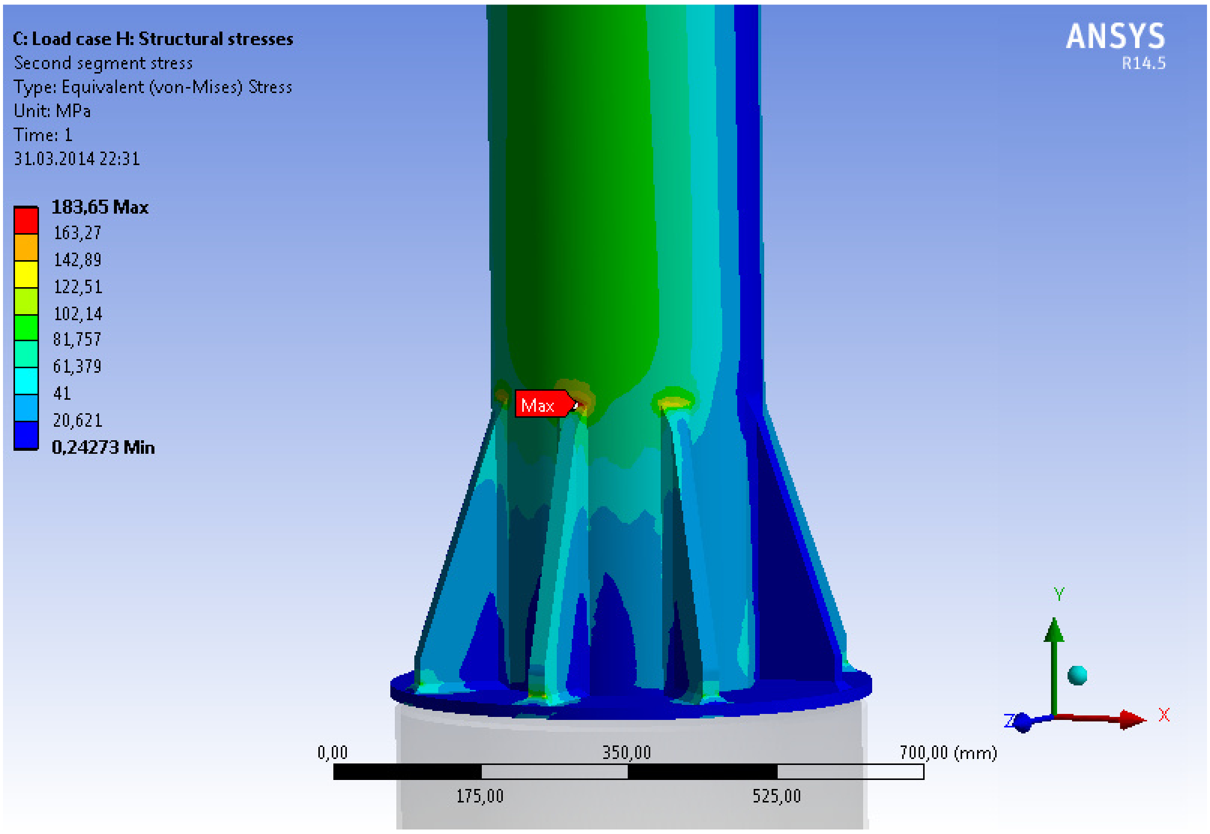

Figure 4 – Stress at first segment and second segment connection

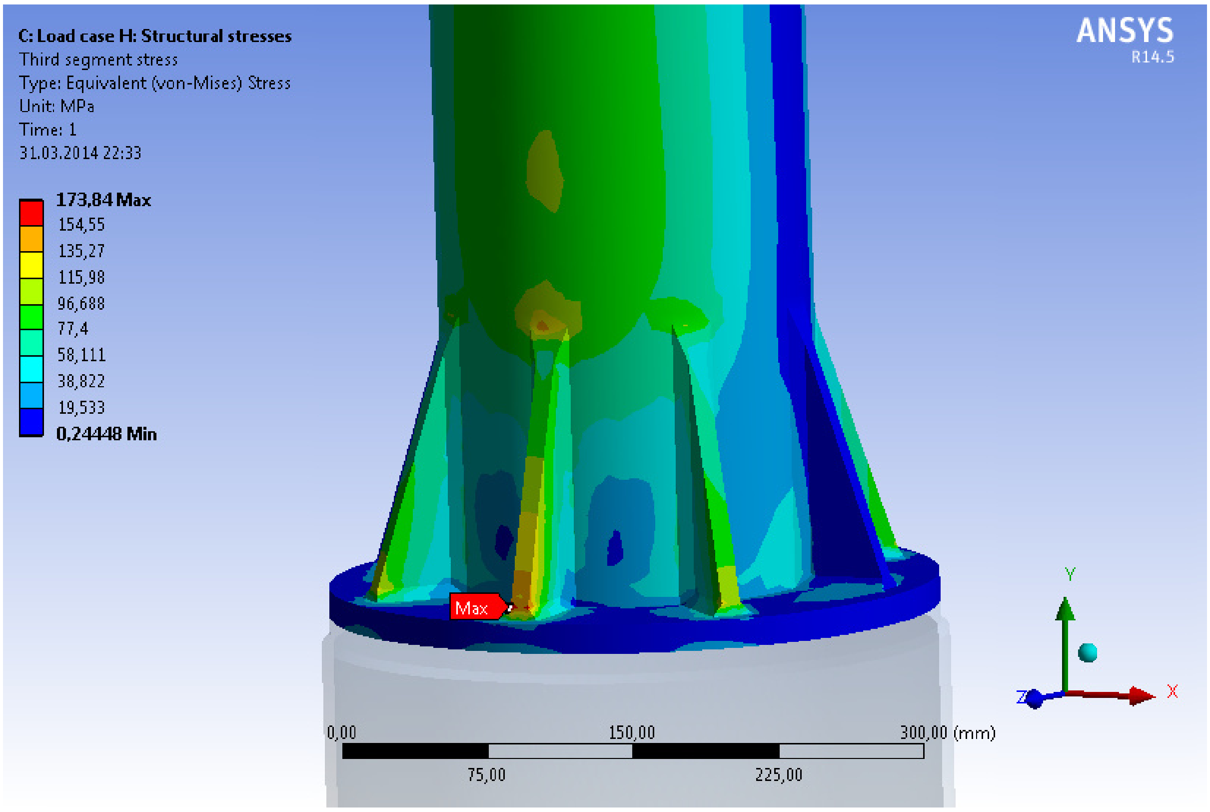

Figure 5 – Stress at second segment and third segment connection

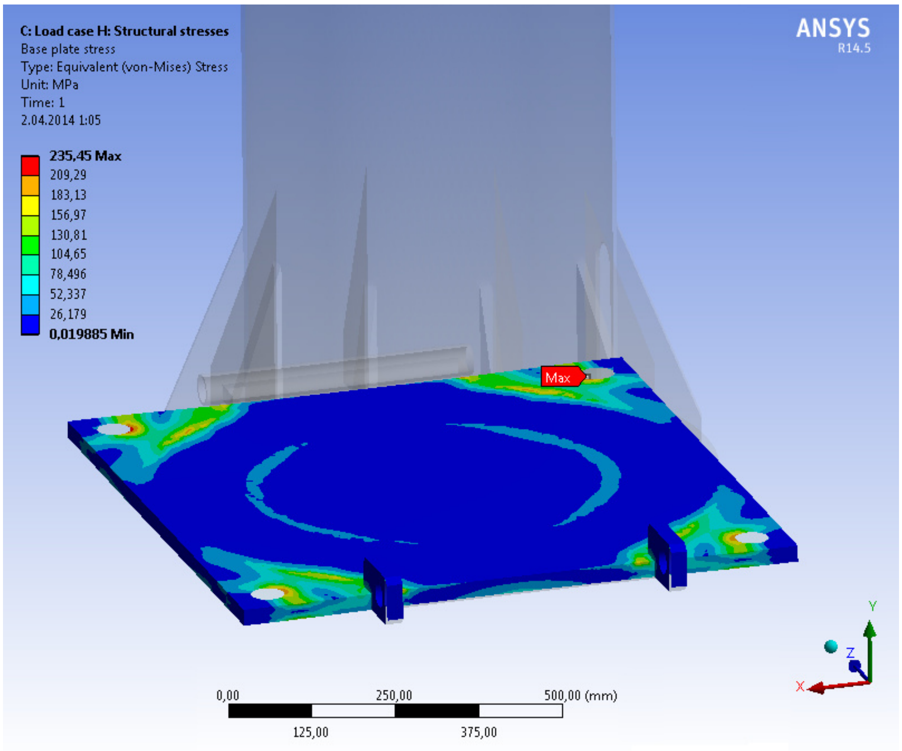

Figure 6 – Stress at base plate

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5.2 Bolted connections

The bolted connections are calculated according to appendix 04. The reaction forces for the bolts are obtained from the FEM model. For one connection interface only the highest forces obtained for one bolt are analysed, as it is assumed that all the bolts in that interface should withstand that force due to the circular nature of acting wind loads.

Third segment to second segment connection

Bolt size and strength class: M20x45 – 8.8

Assembly torque: 434 Nm

Tensile stress area of the bolt: 314 mm^2 (bruto), 245 mm^2 (neto)

Slip factor for preloaded bolts: 0,4

Shear resistance per shear plane: $F_{v,Rd} = \frac{\propto_v f_{ub} A}{\gamma_{M2}} = \frac{0{,}6 \cdot 800 \cdot 314}{1{,}1 \cdot 1{,}35} = 101\,494$ N

Tension resistance: $F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}} = \frac{0{,}9 \cdot 800 \cdot 245}{1{,}1 \cdot 1{,}35} = 118\,787$ N

Preload: $F_{p,C} = 0{,}7 f_{ub} A_s = 0{,}7 \cdot 800 \cdot 245 = 137\,200$ N

Maximum reaction forces (see Figure 7): x: 6 119 N (shear)

y: 33 802 N (tension)

z: 5 345 N (shear)

Shear resistance: $8\,125 \text{ N} < 101\,494 \text{ N}$ (OK)

Tension resistance: $33\,802 \text{ N} < 118\,787 \text{ N}$ (OK)

Combined shear and tension resistance: $\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1{,}4 \cdot F_{t,Rd}} = \frac{8\,125}{101\,494} + \frac{33\,802}{1{,}4 \cdot 118\,787} = 0{,}28 \ < 1$ (OK)

Slip resistance: $F_{s,Rd} = \frac{k_s n \mu (F_{p,c} - 0{,}8 F_{t,Ed})}{\gamma_{M3}} = \frac{1 \cdot 2 \cdot 0{,}4(137\,200 - 0{,}8 \cdot 33\,802)}{1{,}1 \cdot 1{,}35} = 59\,287 \text{ N} > 8\,125 \text{ N}$ (OK)

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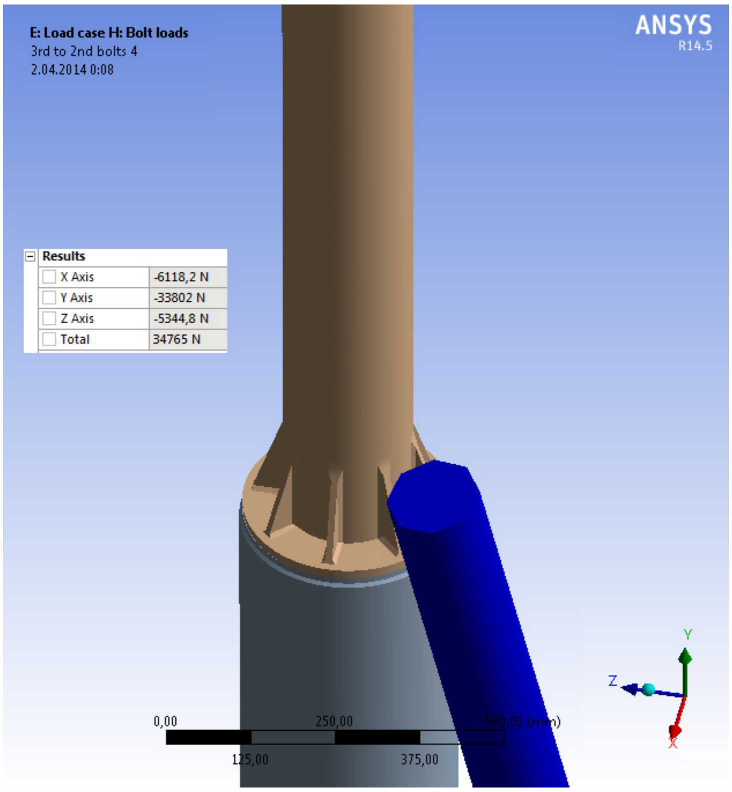

Figure 7 – Third segment second segment highest reaction forces

Second segment to first segment connection

Bolt size and strength class: M20x45 – 8.8

Assembly torque: 434 Nm

Tensile stress area of the bolt: 314 mm^2 (bruto), 245 mm^2 (neto)

Slip factor for preloaded bolts: 0,4

Shear resistance per shear plane: $F_{v,Rd} = \frac{\propto_v f_{ub} A}{\gamma_{M2}} = \frac{0,6 \cdot 800 \cdot 314}{1,1 \cdot 1,35} = 101\ 494$ N

Tension resistance: $F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}} = \frac{0,9 \cdot 800 \cdot 245}{1,1 \cdot 1,35} = 118\ 787$ N

Preload: $F_{p,C} = 0,7 f_{ub} A_s = 0,7 \cdot 800 \cdot 245 = 137\ 200$ N

Maximum reaction forces (see Figure 8): x: 13 044 N (shear)

y: 59 259 N (tension)

z: 12 959 N (shear)

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Shear resistance: $18\ 387\ \mathrm{N} < 101\ 494\ \mathrm{N}$ (OK)

Tension resistance: $59\ 259\ \mathrm{N} < 118\ 787\ \mathrm{N}$ (OK)

Combined shear and tension resistance: $\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1{,}4 \cdot F_{t,Rd}} = \frac{18\ 387}{101\ 494} + \frac{59\ 259}{1{,}4 \cdot 118\ 787} = 0{,}54 < 1$ (OK)

Slip resistance: $F_{s,Rd} = \frac{k_s n \mu (F_{p,c} - 0{,}8 F_{t,Ed})}{\gamma_{M3}} = \frac{1 \cdot 2 \cdot 0{,}4(137\ 200 - 0{,}8 \cdot 59\ 259)}{1{,}1 \cdot 1{,}35} = 48\ 315\ \mathrm{N} > 18\ 387\ \mathrm{N}$ (OK)

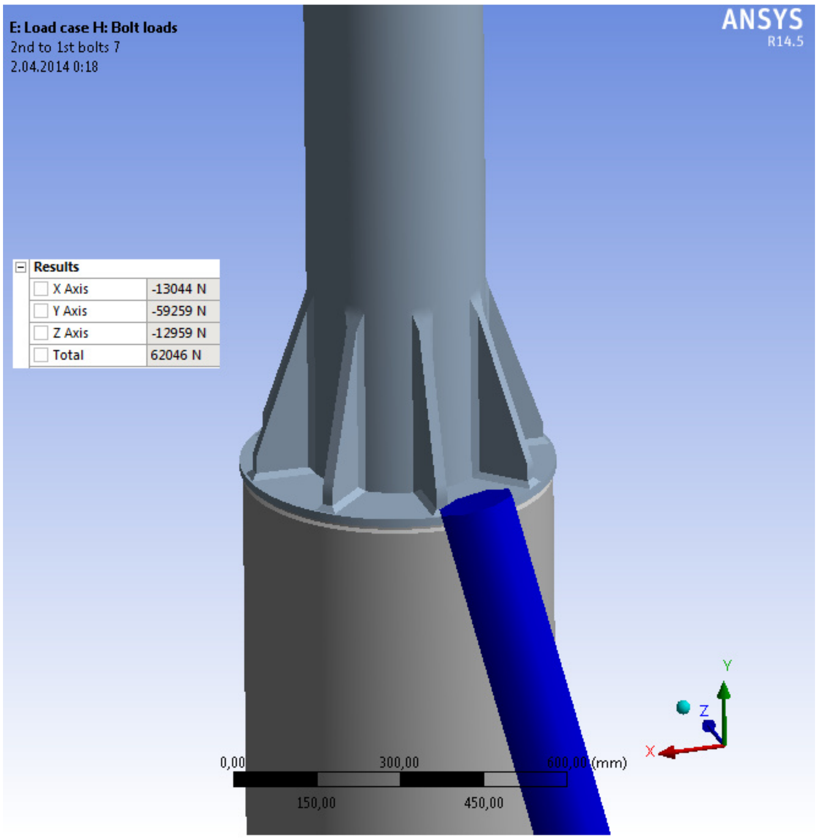

Figure 8 – Second segment first segment highest reaction forces

First segment to base plate connection

Bolt size and strength class: M20x45 – 8.8

Assembly torque: 434 Nm

Tensile stress area of the bolt: $314\ \mathrm{mm}^2$ (bruto), $245\ \mathrm{mm}^2$ (neto)

Slip factor for preloaded bolts: 0,4

Shear resistance per shear plane: $F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}} = \frac{0{,}6 \cdot 800 \cdot 314}{1{,}1 \cdot 1{,}35} = 101\ 494\ \mathrm{N}$

Tension resistance: $F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}} = \frac{0{,}9 \cdot 800 \cdot 245}{1{,}1 \cdot 1{,}35} = 118\ 787$ N

Preload: $F_{p,C} = 0{,}7 f_{ub} A_s = 0{,}7 \cdot 800 \cdot 245 = 137\ 200$ N

Maximum reaction forces (see Figure 9):x: 22 728 N (shear)

y: 59 406 N (tension)

z: 10 363 N (shear)

Shear resistance: $24\ 979\ \mathrm{N} < 101\ 494\ \mathrm{N}$ (OK)

Tension resistance: $59\ 406\ \mathrm{N} < 118\ 787\ \mathrm{N}$ (OK)

Combined shear and tension resistance: $\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1{,}4 \cdot F_{t,Rd}} = \frac{24\ 979}{101\ 494} + \frac{59\ 406}{1{,}4 \cdot 118\ 787} = 0{,}60 < 1$ (OK)

Slip resistance: $F_{s,Rd} = \frac{k_S n \mu (F_{p,c} - 0{,}8 F_{t,Ed})}{\gamma_{M3}} = \frac{1 \cdot 2 \cdot 0{,}4(137\ 200 - 0{,}8 \cdot 59\ 406)}{1{,}1 \cdot 1{,}35} = 48\ 252\ \mathrm{N} > 24\ 979\ \mathrm{N}$ (OK)

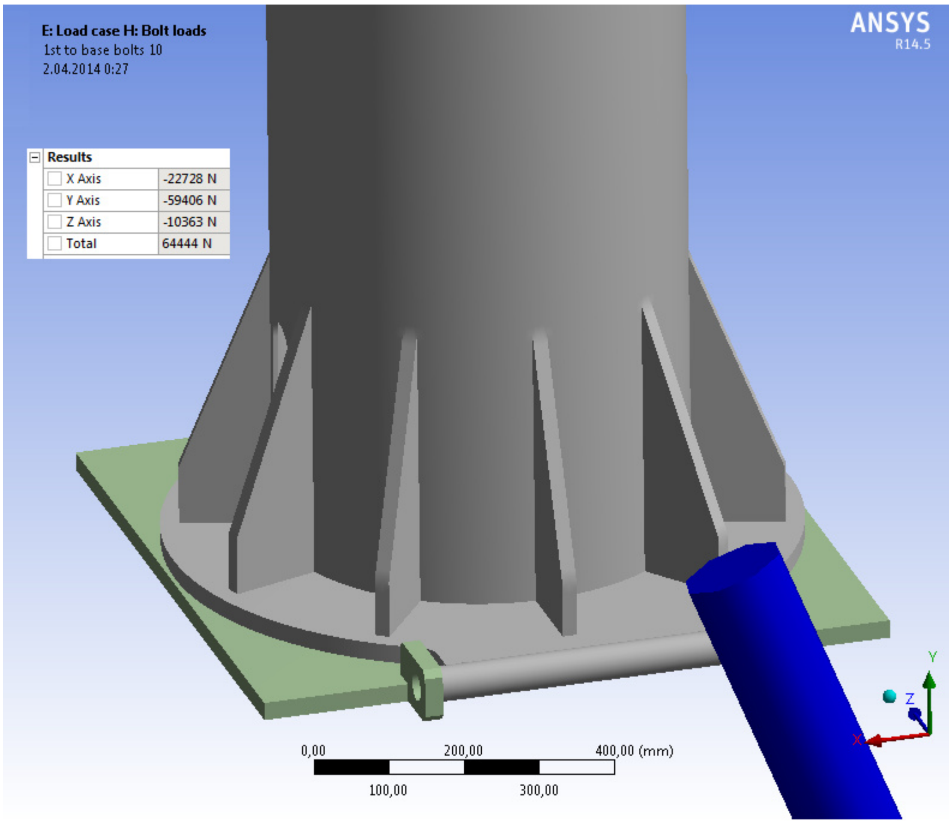

Figure 9 – First segment base reaction forces

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Base plate to foundation connection

Bolt size and strength class: M27x520 – 8.8

Assembly torque: 1080 Nm

Tensile stress area of the bolt: 572 mm^2 (bruto), 459 mm^2 (neto)

Slip factor for preloaded bolts: 0,4

Shear resistance per shear plane: $F_{v,Rd} = \frac{\propto_v f_{ub} A}{\gamma_{M2}} = \frac{0,6 \cdot 800 \cdot 572}{1,1 \cdot 1,35} = 184\,888$ N

Tension resistance: $F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}} = \frac{0,9 \cdot 800 \cdot 459}{1,1 \cdot 1,35} = 222\,545$ N

Tear out resistance: $B_{p,Rd} = \frac{\propto_v \pi d_m t_p f_u}{\gamma_{M2}} = \frac{0,6 \cdot 3,14 \cdot 44,1 \cdot 10 \cdot 640}{1,1 \cdot 1,35} = 358\,837$ N

Preload: $F_{p,C} = 0,7 f_{ub} A_s = 0,7 \cdot 800 \cdot 459 = 257\,040$ N

Maximum reaction forces (see Figure 10): x: 52 600 N (shear)

y: 85 632 N (tension)

z: 26 998 N (shear)

Shear resistance: $59\,124 \text{ N} < 184\,888 \text{ N}$ (OK)

Tension resistance: $85\,632 \text{ N} < 222\,545 \text{ N}$ (OK)

Tear out resistance: $85\,632 \text{ N} < 358\,837 \text{ N}$ (OK)

Combined shear and tension resistance: $\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1,4 \cdot F_{t,Rd}} = \frac{59\,124}{184\,888} + \frac{85\,632}{1,4 \cdot 222\,545} = 0,59 < 1$ (OK)

Slip resistance: $F_{s,Rd} = \frac{k_s n \mu (F_{p,c} - 0,8 F_{t,Ed})}{\gamma_{M3}} = \frac{1 \cdot 2 \cdot 0,4 (257\,040 - 0,8 \cdot 85\,632)}{1,1 \cdot 1,35} = 101\,697 \text{ N} > 59\,124 \text{ N}$ (OK)

Therefore the bolt strength and no slipping against ultimate load cases are guaranteed.

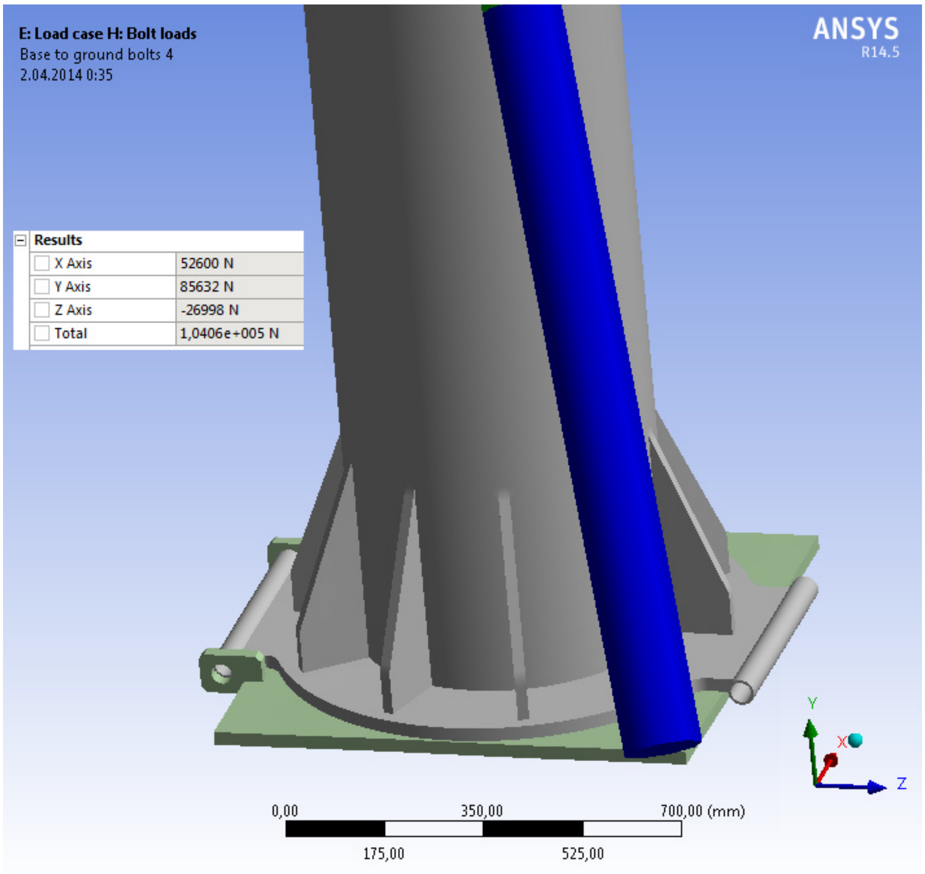

Figure 10 – Base plate foundation reaction forces

5.3 Foundation stability

The foundation block acts as a counterweight for tilting the entire turbine assembly in case of ultimate load cases. Therefore, only the stability condition for the foundation is analysed and no stress values are considered. The acting loads are taken from Table 1 and are applied considering their nature and direction. The calculation schematic is presented on Figure 11 where *Tw* represents the tilting moment and *Tv* the stabilizing moment.

Only load case H: Parked wind loading is presented in the report, as it results in the highest tilting moment for the turbine assembly. In that particular case the tilting moment adds up to 114,396 Nm. The stabilizing moment resulting from the foundation, turbine and tower weight considering the foundation length results in 171 675 Nm.

The allowed tilting moment according to appendix 01 is $\frac{171\,675}{1{,}1 \cdot 1{,}35} = 115\,606\ \mathrm{Nm}\ > 114\,396\ \mathrm{Nm}$ (OK).

Therefore the tower strength against tilting is guaranteed.

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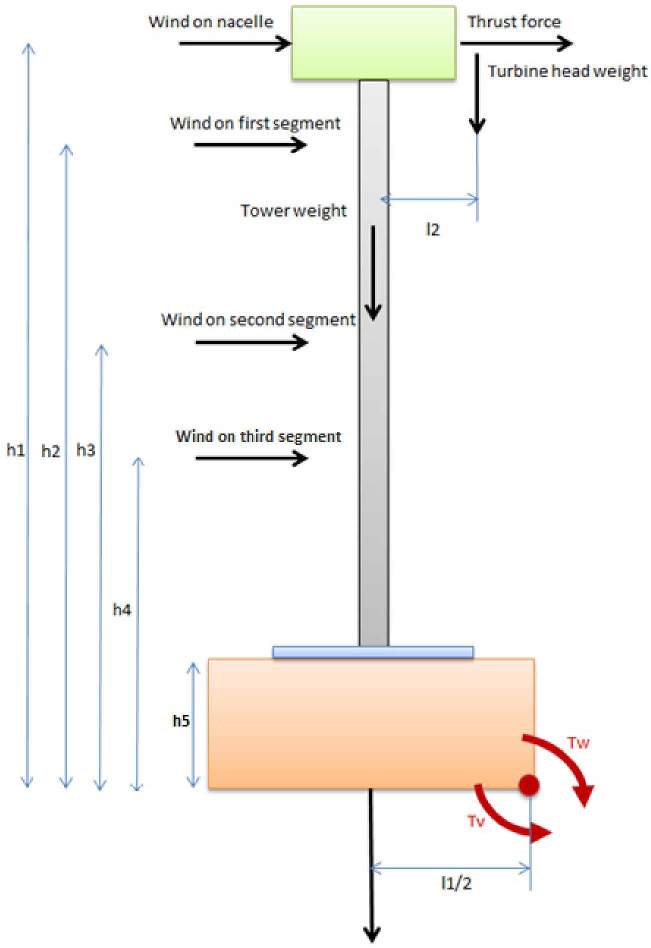

Figure 11 – Stability calculation schematic

6. Conclusion and results

According to the turbine specification, presented calculations and boundary conditions the turbine fulfils all the static stability requirements according to appendix 01.

Lowest safety factor for combined stresses on the tower structure: $\frac{355}{206} = 1{,}72$ (load case H: Parked wind loading).

Lowest safety factor for bolted connections appears for first segment to base connection for combined shear and tension resistance: $\frac{1}{0{,}60} = 1{,}67$ (load case H: Parked wind loading).

Lowest safety factor for foundation tilting: $\frac{171\,675}{114\,396} = 1{,}5$ (load case H: Parked wind loading)