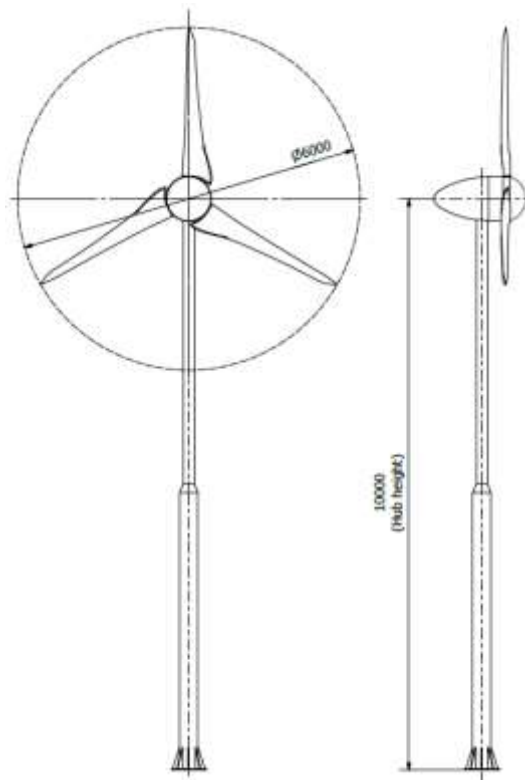


my!Wind

10 kW wind turbine, 6 m rotor

Static stability specification with 10 m mast



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|---------------------------------|----|
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List of Changes

| Rev | Modified parameter | Modified by | Date of modification |
|-----|---|-------------|----------------------|
| 00 | Report written | Pabut | 09-12-2011 |
| 01 | Explanations about foundation tilt, bolt loads and fixing torques | Pabut | 04-18-2013 |
| 02 | Review with minor corrections | Mach | 11-01-2013 |
| 03 | Text corrections | Mach | 01-24-2017 |
| 04 | Upgrade 10 kW | Mach | 09-10-2019 |

Appendixes

| No. | Name | Revision |
|-----|---|----------|
| 01 | IEC 61400-2 2006 Wind turbines part 2; Design requirements for small wind turbines | - |
| 02 | 5KW-MD-01.02.00.00.00-3 Tower selfstanding_merged | 3 |
| 03 | 5KW-MD-01.05.00.00.00-0 Selfstanding foundation | 0 |
| 04 | EN 1993-1-3: 2006 Eurocode 3: Design of steel structures – Part 1-8: Design of joints | - |

General remarks

Due care has been taken in preparation of the information available in this and all referenced documents issued by my!WIND. However, no guarantee is assumed by my!WIND for the correct interpretation and application of the information by the customer. In case any incomplete, incomprehensible or erroneous information is detected, the circumstances must be clarified immediately with the author.

With the application of these documents, the customer agrees that my!WIND can only be held liable for intentional acts and gross negligence.

This document supersedes any previously released version.

1. Introduction

Overview of the general stability of the my!Wind 10 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 IEC.

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 10 m.

The tower consists of a base plate, lower 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 | 323.9 mm |
| First segment wall thickness | 6 mm |
| Second segment diameter | 193.7 mm |
| Second segment thickness: | 6 mm |
| Overall weight | 485 kg |

The foundation block consists of a casted concrete body with inner load bearing structure from steel. The purpose of the concrete is to act only as a weight to counter the tilting moment from the thrust 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 | 2.5 m |
| Width | 2.5 m |
| Height | 0.4 m |
| Weight | 5.75 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 | 2300 kg/m ³ |
| Compressive strength | 20 MPa |

Foundation inner structure – Steel S235JR:

| | |
|------------------------|---------|
| Young's modulus | 200 GPa |
| Poisson's ratio | 0.3 |
| Tensile yield strength | 235 MPa |

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 are 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 shoulder. A summary of the acting loads for each load case and corresponding tilting moments together with induced stresses is 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 | 4 758 N | 4 758 N | 4 758 N | 4 758 N |
| Second segment wind | 0 N | 0 N | 1 466 N | 748 N |
| First segment wind | 0 N | 0 N | 2 451 N | 1 250 N |
| Nacelle cover wind | 0 N | 0 N | 738 N | 3 77 N |
| Results | | | | |
| Tilting moment at second segment | 5 045 Nm | 21 503 Nm | 20 005 Nm | 23 040 Nm |
| Tilting moment at tower base | 9 036 Nm | 40 995 Nm | 49 672 Nm | 49 069 Nm |
| Tilting moment at foundation base | 8 629 Nm | 42 143 Nm | 52 323 Nm | 51 185 Nm |
| Nominal stress at second segment base | 31 MPa | 134 MPa | 124 MPa | 143 MPa |
| Nominal stress at first segment base | 19 MPa | 88 MPa | 106 MPa | 105 MPa |

5. Calculations

5.1 Tower strength

FEM model is modelled with 3 solid parts: base, first segment and second segments. The turbine head weight is represented by a point mass which is located 314 mm from the tower centre in horizontal direction (y 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 4 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 two 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 I: Parked wind loading maximum exposure is presented in the report, as it results in the highest stresses at the intersection of the first and second segment. Although, the load case H: Parked wind loading provides the biggest tilting moment, the resulting overall stresses are lower due to more equal load distribution.

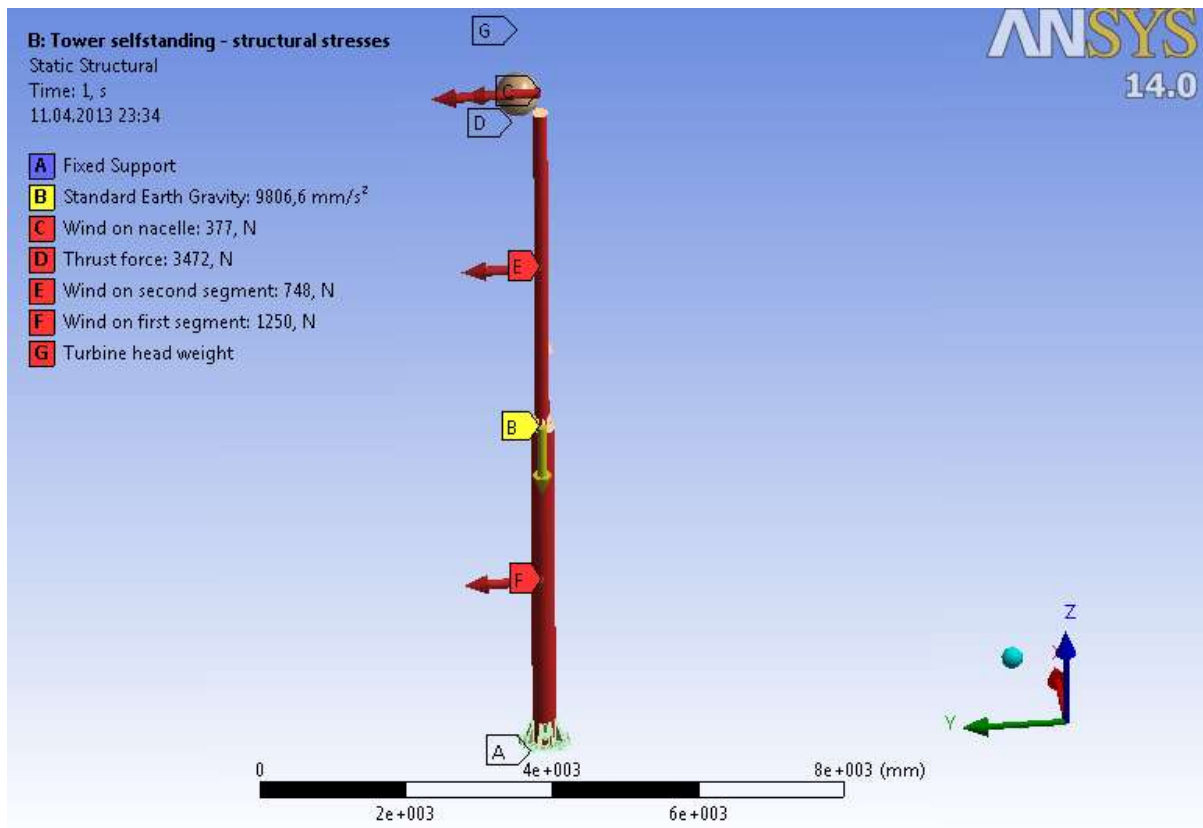


Figure 1 - Boundary conditions

Tetrahedron mesh is used with a dominant element size of 15 mm which results in a collective mesh of 375 271 elements and 715 974 nodes. On the strengthening ribs, the mesh is refined with element size of 8 mm (see Figure 2).

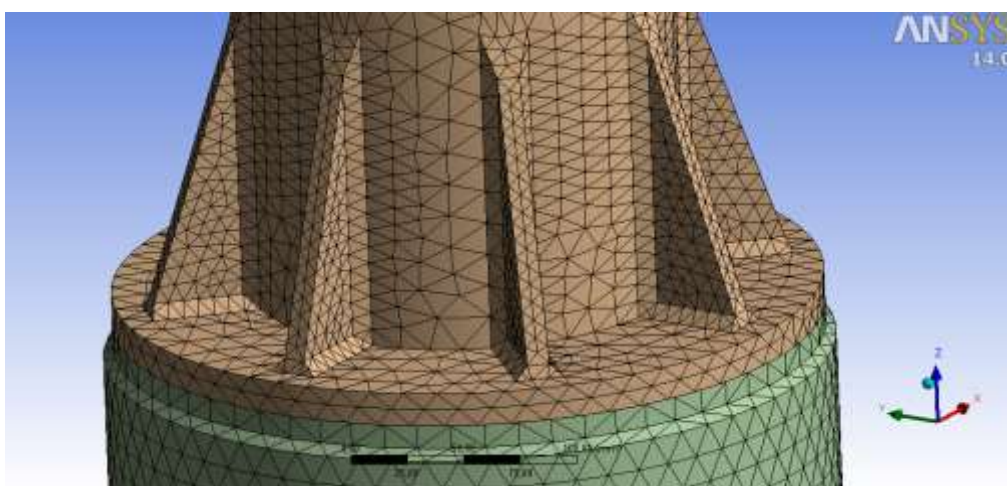


Figure 2 – Mesh

The highest stresses occur on the strengthening ribs of the second segment (see Figure 3). These values go up to 233 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 allowed stress according to appendix 01 is $\frac{355}{1.1 \cdot 1.35} = 239 \text{ MPa} > 233 \text{ MPa}$ (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.

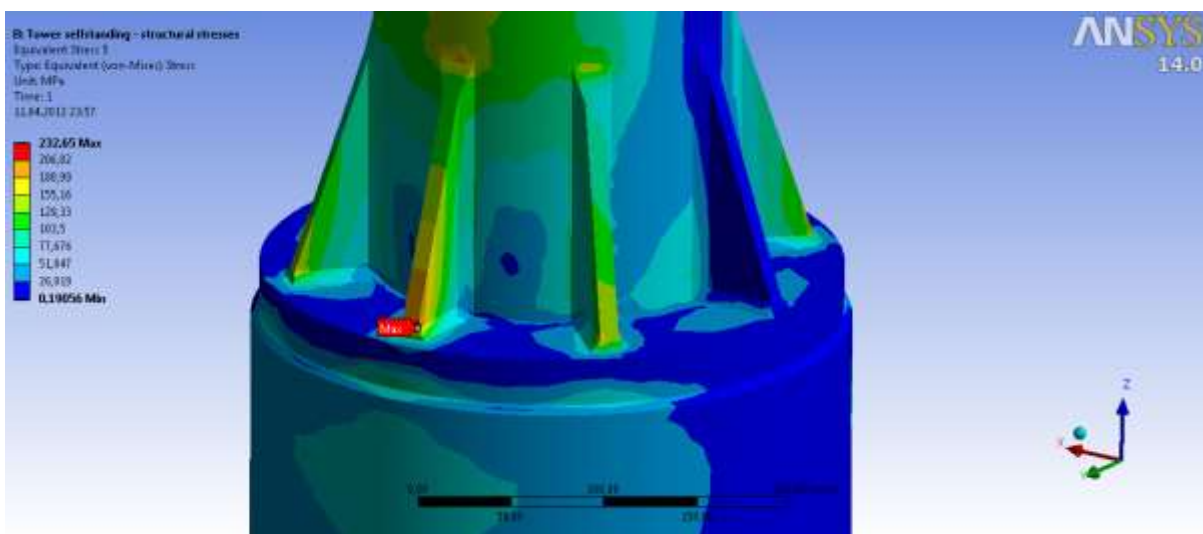


Figure 3 – Stress at second and first segment connection

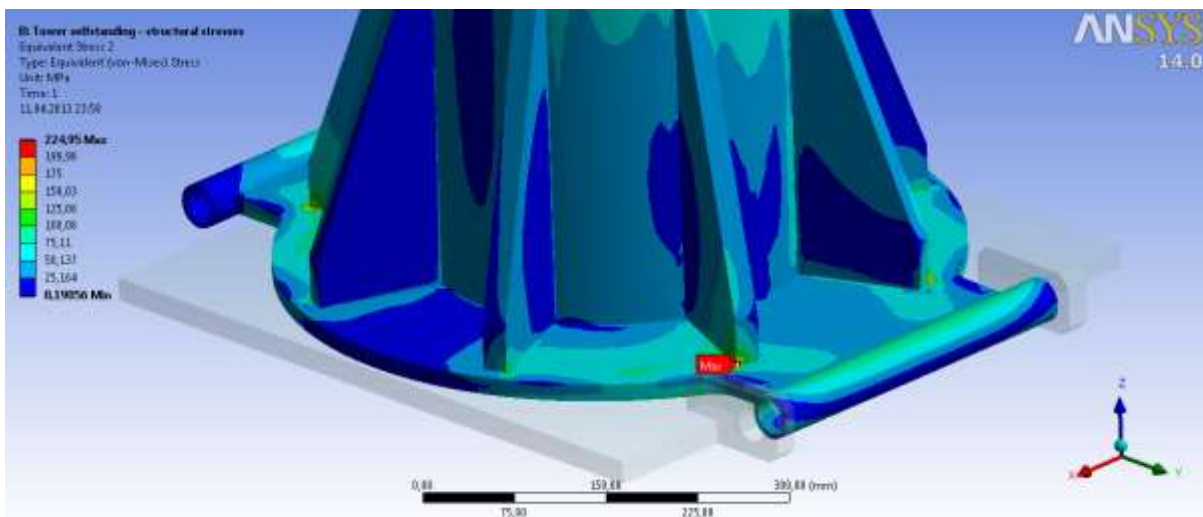


Figure 4 – Stress at first segment and base plate connection

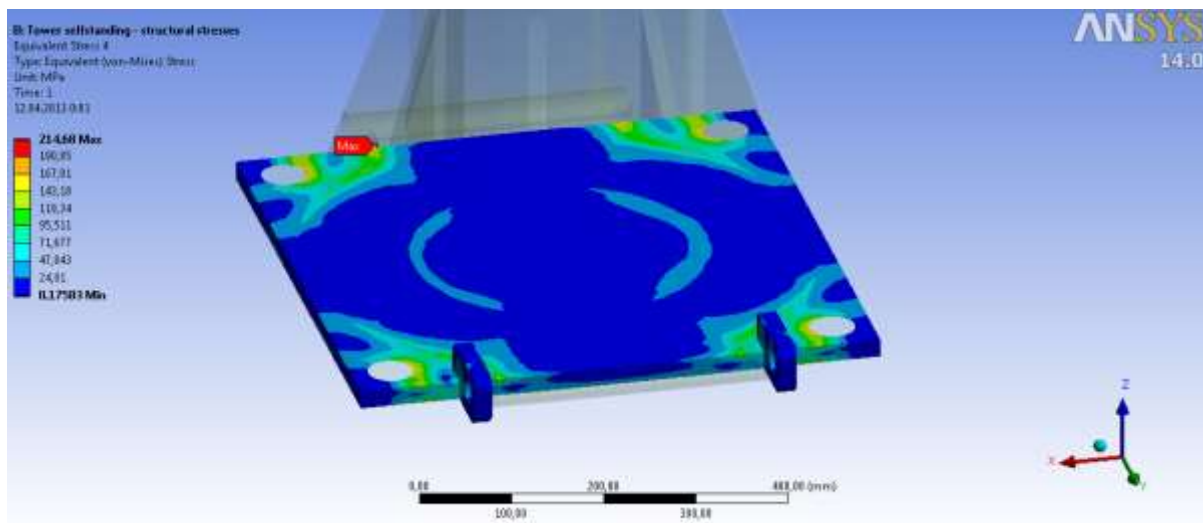


Figure 5 – Stress at the base plate

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.

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 mm² (gross), 245 mm² (net)

Slip factor for preloaded bolts: 0.4

Shear resistance per shear plane: $F_{v,Rd} = \frac{\alpha_v f_{ub} A_s}{\gamma_{M2}} = \frac{0.6 \cdot 800 \cdot 314}{1.1 \cdot 1.35} = 101\,494 \text{ N}$

Tension resistance: $F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}} = \frac{0.9 \cdot 800 \cdot 245}{1.1 \cdot 1.35} = 118\,787 \text{ N}$

Preload: $F_{p,C} = 0.7 f_{ub} A_s = 0.7 \cdot 800 \cdot 245 = 137\,200 \text{ N}$

Maximum reaction forces (see Figure 6): x: 10 765 N (shear)

y: 8 893 N (shear)

z: 47 268 N (tension)

Shear resistance: 13 963 N < 101 494 N (OK)

Tension resistance: 47 268 N < 118 787 N (OK)

Combined shear and tension resistance: $\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4 \cdot F_{t,Rd}} = \frac{13\,963}{101\,494} + \frac{47\,268}{1.4 \cdot 118\,787} = 0.42 < 1$ (OK)

Slip resistance: $F_{s,Rd} = \frac{k_s n \mu (F_{p,c} - 0.8 F_{t,Ed})}{\gamma_{M3}} = \frac{1 \cdot 2 \cdot 0.4 (137\,200 - 0.8 \cdot 47\,268)}{1.1 \cdot 1.35} = 53\,541 \text{ N} > 19\,658 \text{ N}$ (OK)

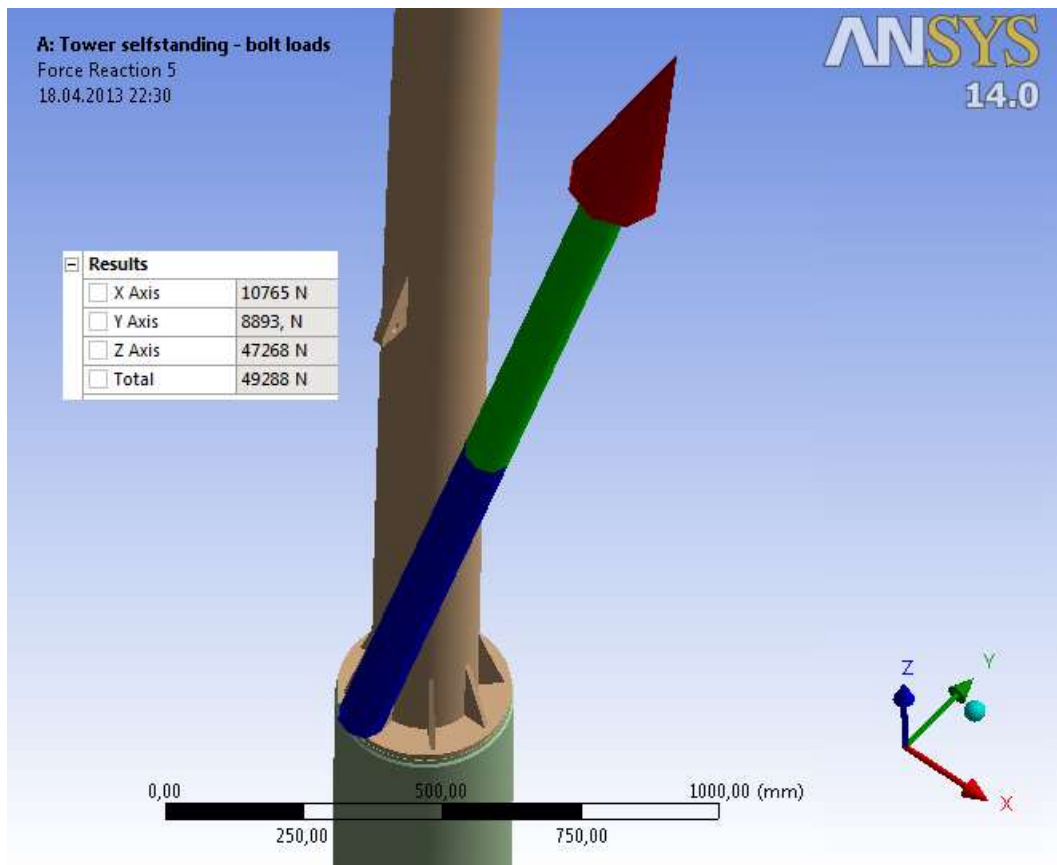


Figure 6 – Second segment first segment highest reaction forces

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 mm² (gross), 245 mm² (net)

Slip factor for preloaded bolts: 0.4

Shear resistance per shear plane: $F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}} = \frac{0.6 \cdot 800 \cdot 314}{1.1 \cdot 1.35} = 101\,494 \text{ N}$

Tension resistance:
$$F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}} = \frac{0.9 \cdot 800 \cdot 245}{1.1 \cdot 1.35} = 118\,787 \text{ N}$$

Preload:
$$F_{p,c} = 0.7 f_{ub} A_s = 0.7 \cdot 800 \cdot 245 = 137\,200 \text{ N}$$

Maximum reaction forces (see Figure 7):

- x: 24 110 N (shear)
- y: 7 390 N (shear)
- z: 50 208 N (tension)

Shear resistance: 25 217 N < 101 494 N (OK)

Tension resistance: 50 208 N < 118 787 N (OK)

Combined shear and tension resistance:
$$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4 \cdot F_{t,Rd}} = \frac{25\,217}{101\,494} + \frac{50\,208}{1.4 \cdot 118\,787} = 0.55 < 1 \text{ (OK)}$$

Slip resistance:
$$F_{s,Rd} = \frac{k_s n \mu (F_{p,c} - 0.8 F_{t,Ed})}{\gamma_{M3}} = \frac{1 \cdot 2 \cdot 0.4 (137\,200 - 0.8 \cdot 50\,208)}{1.1 \cdot 1.35} = 52\,273 \text{ N} > 25\,217 \text{ N (OK)}$$

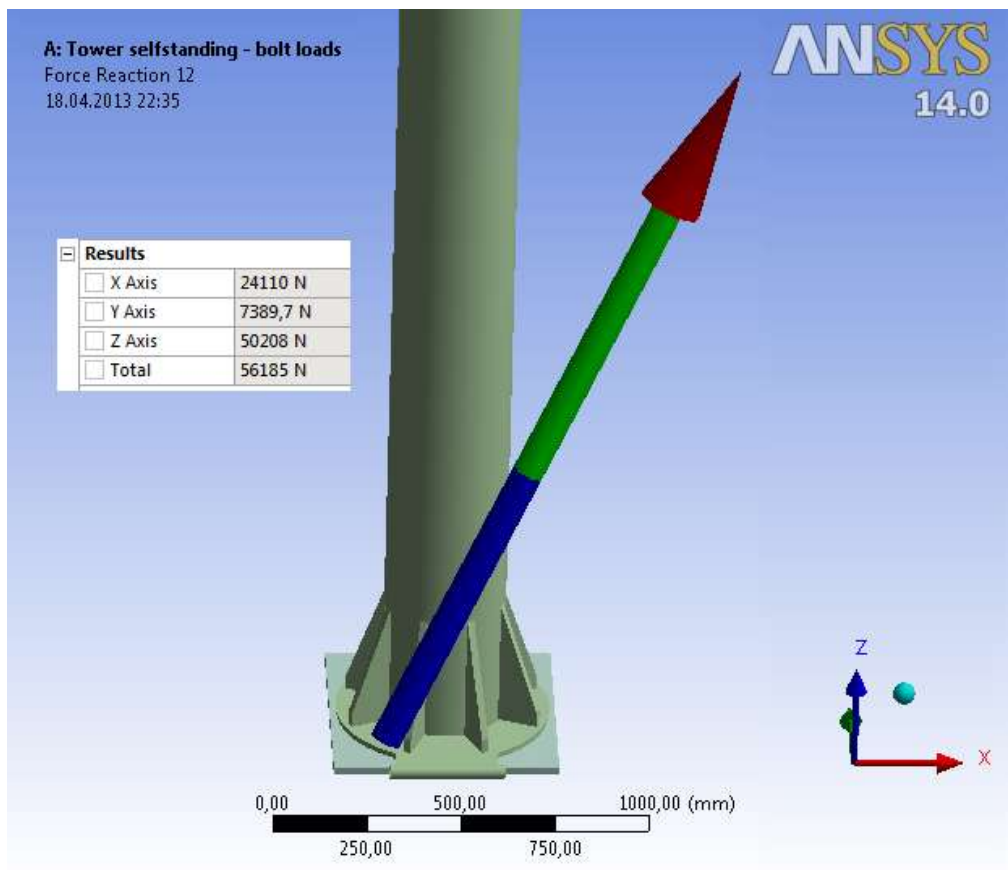


Figure 7 – First segment base reaction forces

Base plate to foundation connection

Bolt size and strength class: M27x550 – 8.8

Assembly torque: 1080 Nm

Tensile stress area of the bolt: 572 mm² (gross), 459 mm² (net)

Slip factor for preloaded bolts: 0.4

Shear resistance per shear plane: $F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}} = \frac{0.6 \cdot 800 \cdot 572}{1.1 \cdot 1.35} = 184\,888\text{ N}$

Tension resistance: $F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}} = \frac{0.9 \cdot 800 \cdot 459}{1.1 \cdot 1.35} = 222\,545\text{ N}$

Tear out resistance: $B_{p,Rd} = \frac{\alpha_v \pi d_m t p f_u}{\gamma_{M2}} = \frac{0.6 \cdot 3.14 \cdot 44.1 \cdot 8.5 \cdot 640}{1.1 \cdot 1.35} = 304\,363\text{ N}$

Preload: $F_{p,c} = 0.7 f_{ub} A_s = 0.7 \cdot 800 \cdot 459 = 257\,040\text{ N}$

Maximum reaction forces (see Figure 8): x: 32 091 N (shear)

y: 8 159 N (shear)

z: 54 113 N (tension)

Shear resistance: 33 112 N < 184 888 N (OK)

Tension resistance: 54 113 N < 222 545 N (OK)

Tear out resistance: 54 113 N < 304 363 N (OK)

Combined shear and tension resistance: $\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4 \cdot F_{t,Rd}} = \frac{33\,112}{184\,888} + \frac{54\,113}{1.4 \cdot 222\,545} = 0.35 < 1$ (OK)

Slip resistance: $F_{s,Rd} = \frac{k_s n \mu (F_{p,c} - 0.8 F_{t,Ed})}{\gamma_{M3}} = \frac{1.2 \cdot 0.4 (257\,040 - 0.8 \cdot 54\,113)}{1.1 \cdot 1.35} = 115\,151\text{ N} > 33\,112\text{ N}$ (OK)

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

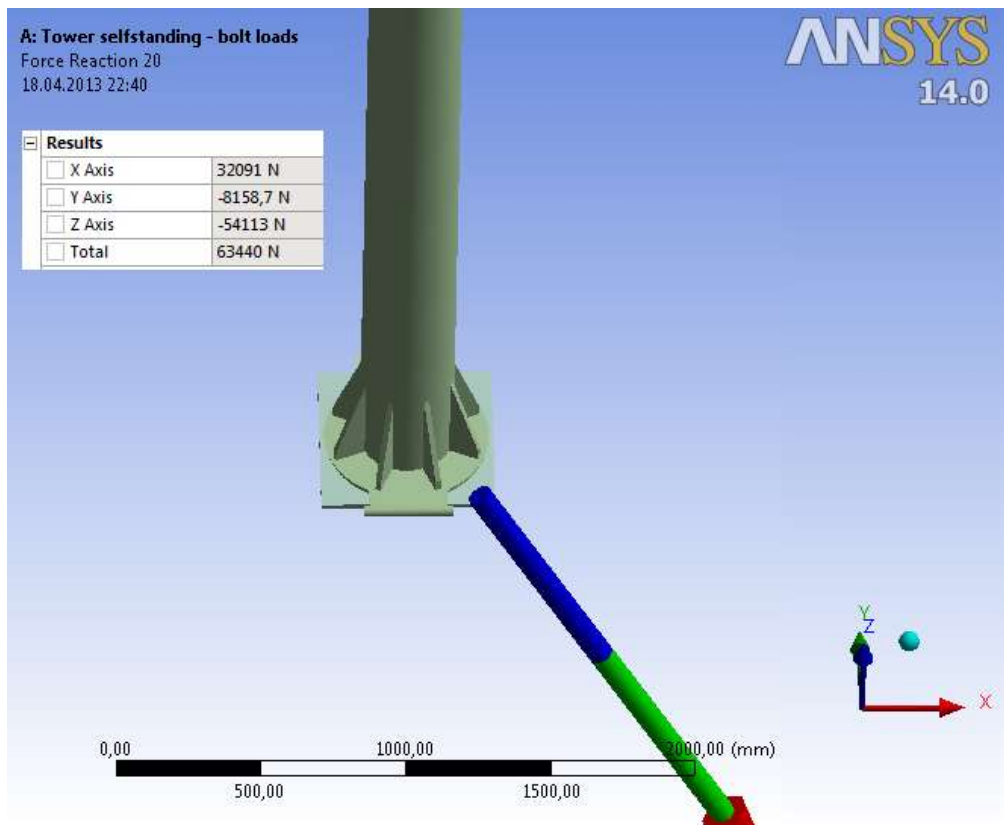


Figure 8 – Base plate foundation reaction forces

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 9 where T_w represents the tilting moment and T_v 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 52,323 Nm. The stabilizing moment resulting from the foundation, turbine and tower weight considering the foundation length results in 78,862 Nm.

The allowed tilting moment according to appendix 01 is $\frac{78\,862}{1.1 \cdot 1.35} = 53\,105 \text{ Nm} > 52\,323 \text{ Nm}$ (OK).

Therefore the tower strength against tilting is guaranteed.

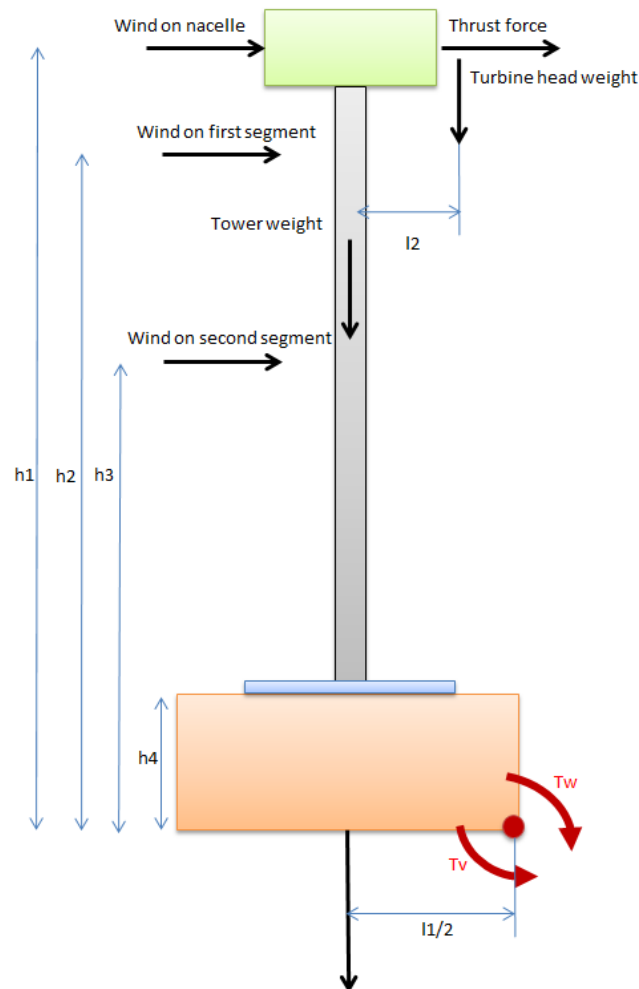


Figure 9 – Stability calculation schematic

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: $\frac{355}{233} = 1.53$ (load case I: Parked wind loading maximum exposure).

Lowest safety factor for bolted connections appears for first segment to base connection for combined shear and tension resistance: $\frac{1}{0.55} = 1.8$ (load case I: Parked wind loading maximum exposure).

Lowest safety factor for foundation tilting: $\frac{78\,862}{52\,323} = 1.5$ (load case H: Parked wind loading)