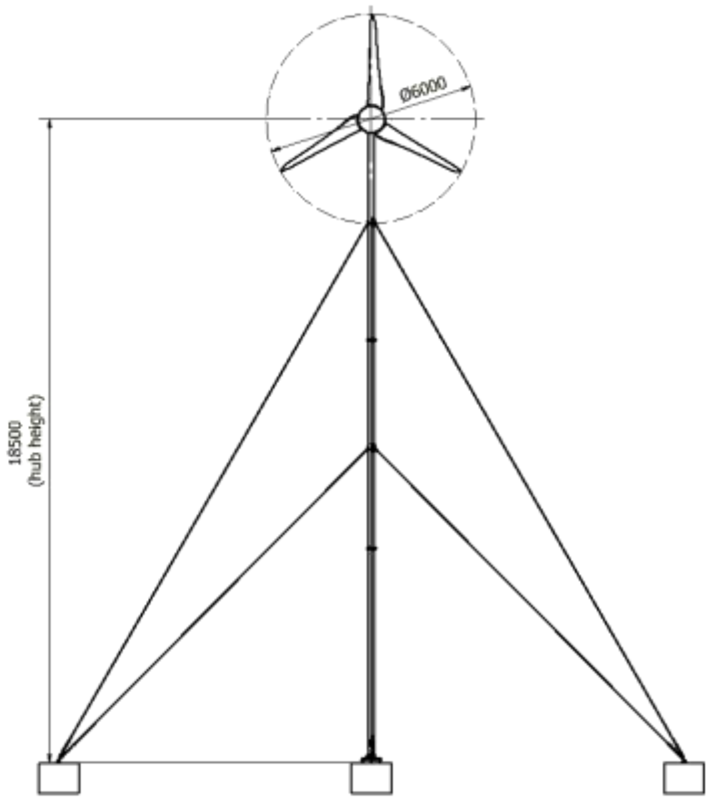
my!WIND Ltd.
Soola 1a
51013 Tartu, Estonia
Email: info@mywind.ee
Website: www.mywind.ee

# my!Wind

# 10 kW wind turbine generator, 6 m rotor

# Static stability
# with 18 m guy-wired mast



my!*WIND* Ltd.
Soola 1a
51013 Tartu, Estonia
Email: info@mywind.ee
Website: www.mywind.ee

## Contents



## List of Changes

| Rev | Modified parameter | Modified by | Date of modification |
|---|---|---|---|
| 00 | Report written | Pabut | 05-25-2013 |
| 01 | Text corrections | Mach | 01-23-2017 |
| 02 | Upgrade to 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-01.04.00.00.00-0 Tower guy-wired 18m_merged | 0 |
| 03 | 5KW-MD-01.06.01.00.00-0 Guy-wired foundation centre | 0 |
| 04 | 5KW-MD-01.06.02.00.00-0 Guy-wired foundation side | 0 |
| 05 | EN 1993-1-3: 2006 Eurocode 3: Design of steel structures – Part 1-8: Design of joints | - |

my!WIND Ltd.
Soola 1a
51013 Tartu, Estonia
Email: info@mywind.ee
Website: www.mywind.ee

## General remarks

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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.

my!*WIND* Ltd.
Soola 1a
51013 Tartu, Estonia
Email: info@mywind.ee
Website: www.mywind.ee

## 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 guy-wired steel tubular mast and attached to concrete foundation block. The guywires holding the mast upright are also attached to four foundation blocks. In order to fulfil the stability requirements according to appendix 01 the strength characteristics of the guy-wires mast have to exceed the stresses introduced by the acting ultimate loads. The guy-wire foundation blocks must remain stable against pull out and sliding. 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 and the guy-wires and associated equipment has to withstand the acting ultimate loads.

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 05. The reaction forces for the guy-wires are also taken from the FEM model and the strength of the equipment is calculated according to empirical formulas available from equipment providers. The stability of the foundation units is calculated with empirical formulas available from common literature.

## 2. Geometry

The turbine consists of one central foundation block, four side foundation blocks, cables and associated equipment, guy-wired mast and a tower top. All the acting loads are calculated according to the turbine hub height of 18.5 m.

The tower consists of a base plate, base plate angles, 1$^{st}$ segment, 2$^{nd}$ segment and 3$^{rd}$ 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 | 168.3 mm |
| First segment wall thickness | 4 mm |
| Second segment diameter | 168.3 mm |
| Second segment thickness: | 4 mm |
| Third segment diameter: | 168.3 mm |
| Third segment wall thickness: | 4 mm |
| Overall weight | 389 kg |

The foundation blocks consist of a casted concrete body with inner load bearing structure from steel. The purpose of the side foundation blocks is to act only as weights to counter the pulling forces from the wind thrust in the guy-wires. The central foundation block acts as a base for lifting and receives the compression force generated by the wind thrust. The units are designed in a way that all the directly acting loads are received by the steel structure (appendixes 03 and 04).

Side foundation block specification:

| | |
|---|---|
| Length (l) | 1.15 m |
| Width (w) | 1.15 m |
| Height (h) | 0.85 m |
| Weight (W) | 2 585 kg |

Central foundation block specification:

| | |
|---|---|
| Length (l) | 1.15 m |
| Width (w) | 1.15 m |
| Height (h) | 0.85 m |
| Weight (W) | 2 585 kg |

## 3. Materials

Tower – Steel S355J2H:

| | |
|---|---|
| Young's modulus | 200 GPa |
| Poisson's ratio | 0.3 |
| Tensile yield strength | 355 MPa |

Foundation blocks – 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 the 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 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. As the tower with two sets of guywires is statically undefined construction, the stresses and forces in the wires are taken from the FEM model. This is explained in detail in the following chapter. Summary of the acting loads for each load case and corresponding forces in the wires together with induced stresses are represented below. For this type of a construction, the most critical points are the points were the wires connect to the tower (see appendix 01).

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** | 3 816 N | 3 816 N | 3 816 N | 3 816 N |
| **Third segment wind** | 0 N | 0 N | 1 533 N | 782 N |
| **Second segment wind** | 0 N | 0 N | 1 533 N | 782 N |
| **First segment wind** | 0 N | 0 N | 1 533 N | 782 N |
| **Nacelle cover wind** | 0 N | 0 N | 738 N | 377 N |
| Results | | | | |
| **Reaction force at first wire connection** | 1 142 N | 3 643 N | 456 N | 1 890 N |
| **Reaction force at second wire connection** | 3 136 N | 12 511 N | 12 616 N | 13 644 N |
| **Reaction force at tower base** | 9 595 N | 19 504 N | 16 790 N | 19 253 N |
| **Stress at first wire connection** | 16 MPa | 56 MPa | 46 MPa | 56 MPa |
| **Stress at second wire connection** | 31 MPa | 235 MPa | 207 MPa | 236 MPa |

## 5. Calculations

### 5.1 Tower strength

The FEM calculations are carried out in 3 parts. At first, the tower segments and wires are analysed. Then the obtained reaction forces are used to analyse the tower base assembly and the intermediate plate for wire connections. This is done in order to keep all calculation models as simple as possible and to obtain most realistic result with small computational power.

The tower FEM model consists of four unique main solid parts: 1st segment, 2nd segment, 3rd segment and all holders that connect the wires to the segments. The cables are modelled as rigid bodies in order to eliminate their deformational influence. Cables are placed in a way that they resist only to tensional forces. In current model the upper cable is from one side of the tower and the lower cable from another side of the tower. This results from the fact that due to the high deformations the lower section of the tower is bent into the wind direction. The theory was verified by determining the direction of reaction forces in a way that the tower cables act for tension only.

The turbine head weight is represented by a point mass which is located 314 mm from the tower centre in horizontal direction (y axis, see Figure 1). The tower segments 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 segments to each other. The areas where the segments touch but are not connected in any way have frictionless supports. The holders are connected to the mast with rotational joints (see Figure 2) and the cables as rigid bodies are connected to the holders with fixed joints. The tower 1st segment is connected to the ground with a rotational joint. These joints are used to derive the reaction forces for the cables and other associated equipment.

In order to achieve more realistic result in the highly stressed are where the lifting ribs connect to the tower tubes, 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 the 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 3rd segment and the upper wire connection.

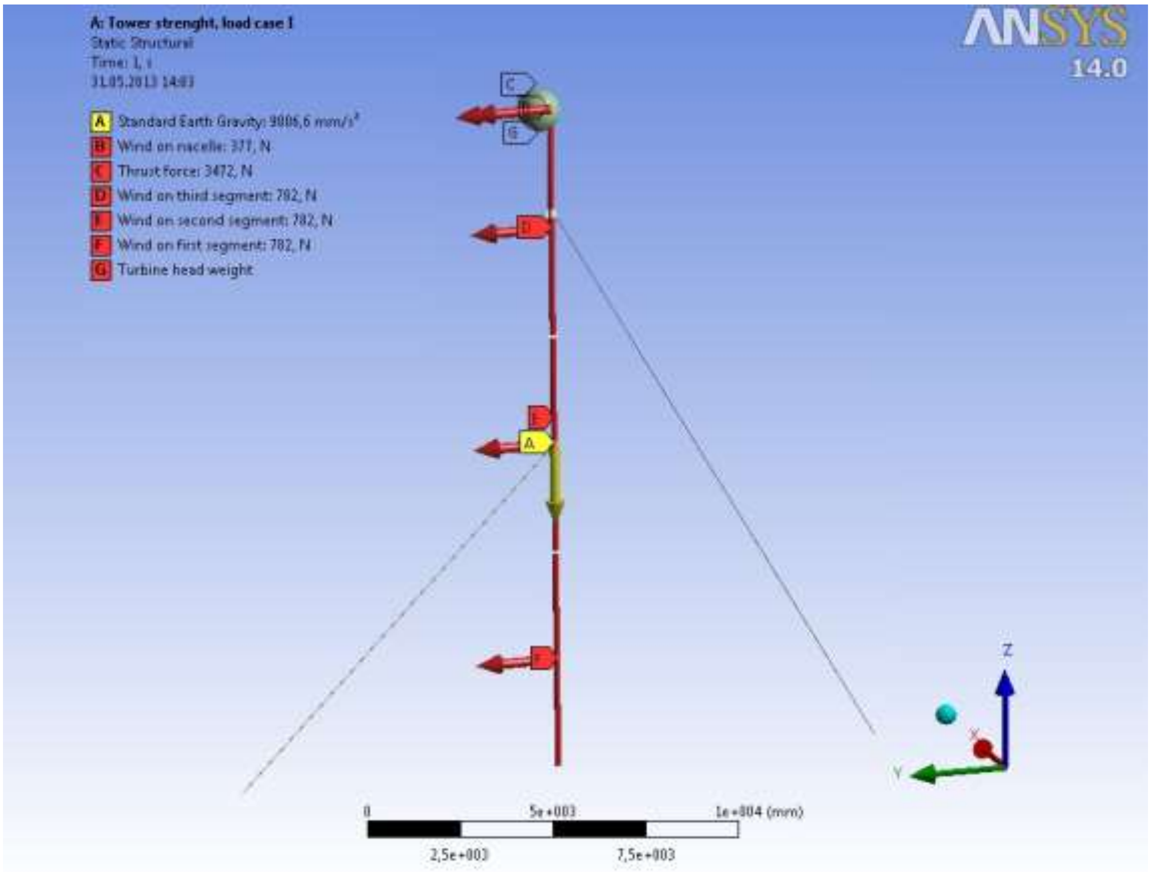


Figure 1 - Boundary conditions

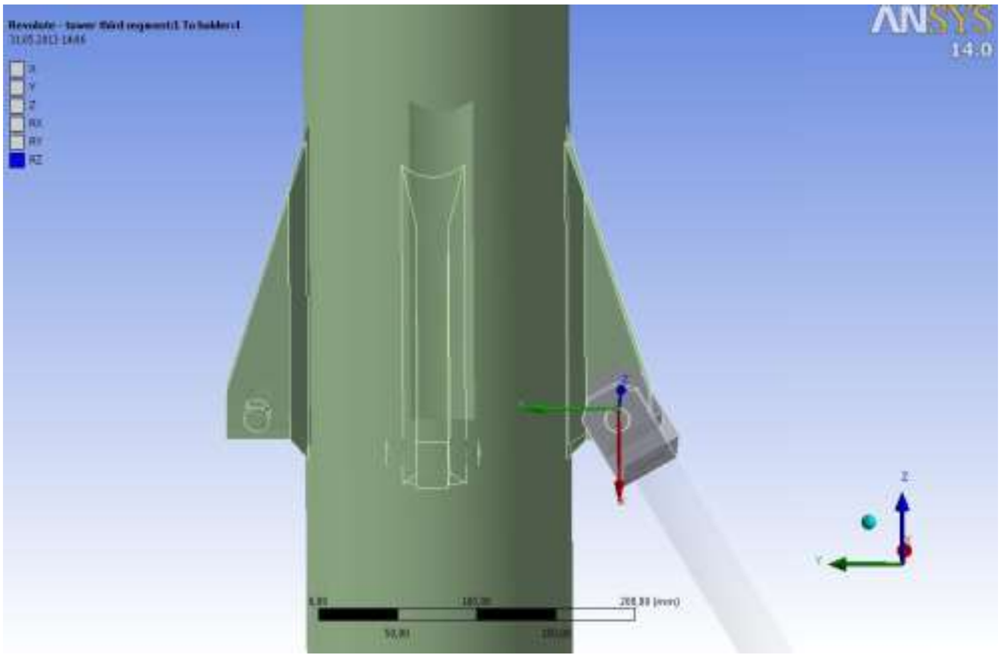


Figure 2 – Joint between cable and tower

Tetrahedron mesh is used with a dominant element size of 15 mm which results in a collective mesh of 321 780 elements and 642 737 nodes. On the lifting ribs, the mesh is refined with element size of 8 mm (see Figure 3).

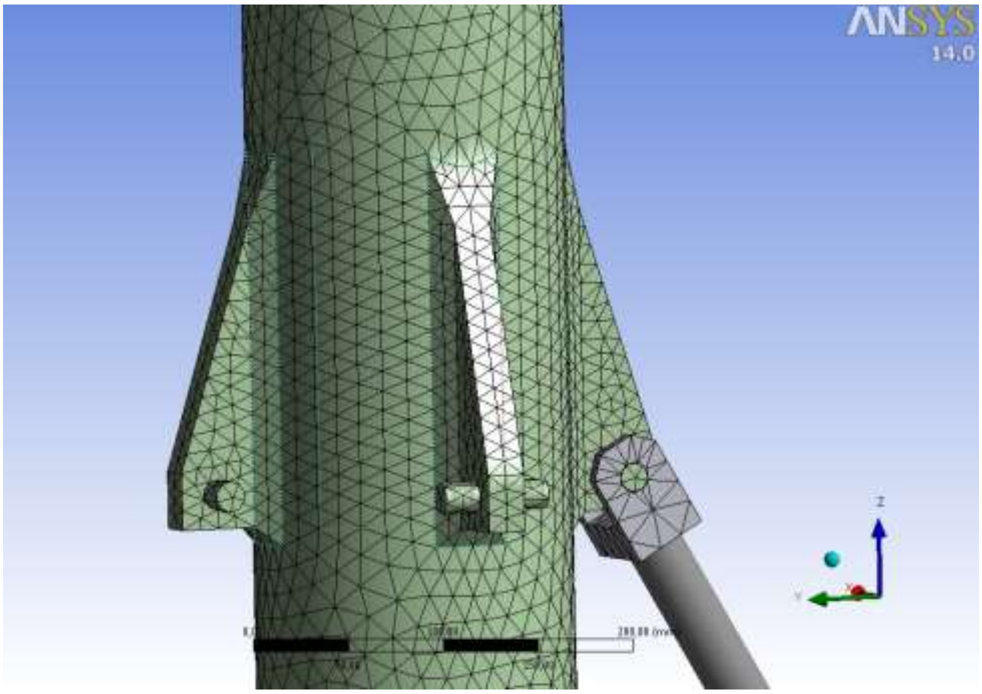

Figure 3 – Mesh

The highest stresses occur on the upper lifting rib and 3rd segment connection (see Figure 4). These values go up to 236 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 5 and Figure 6).

The allowed stress according to appendix 01 is $\frac{355}{1{,}1\cdot 1{,}35} = 239\ \mathrm{MPa} > 236\ \mathrm{MPa}$ (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.**

my!*WIND* Ltd.
Soola 1a
51013 Tartu, Estonia
Email: info@mywind.ee
Website: www.mywind.ee

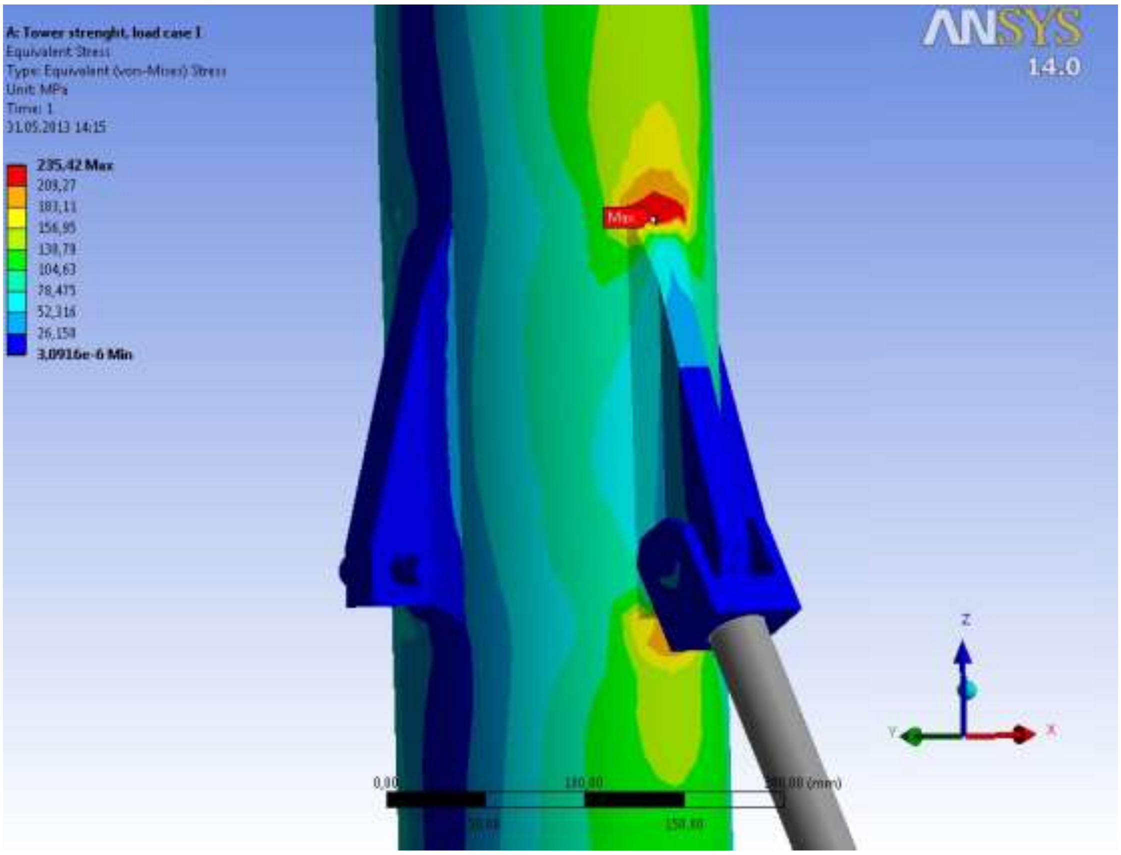


Figure 4 – Stress at 2nd and 1st segment connection

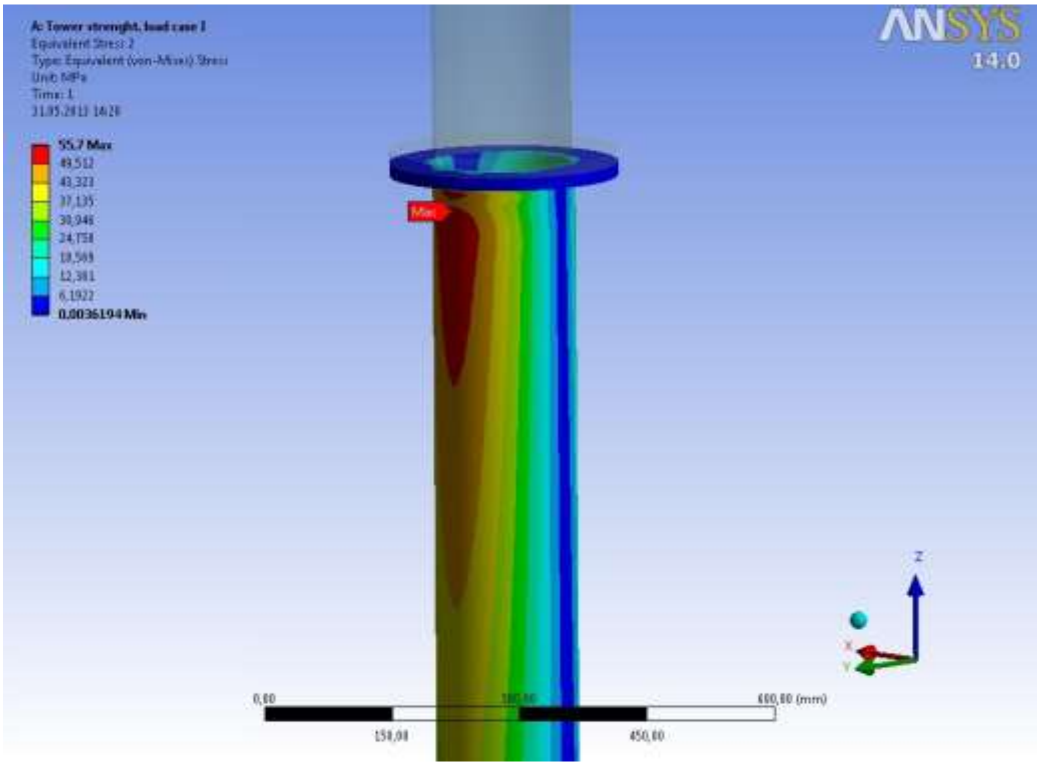


Figure 5 – Stress at 2nd segment

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Soola 1a
51013 Tartu, Estonia
Email: info@mywind.ee
Website: www.mywind.ee

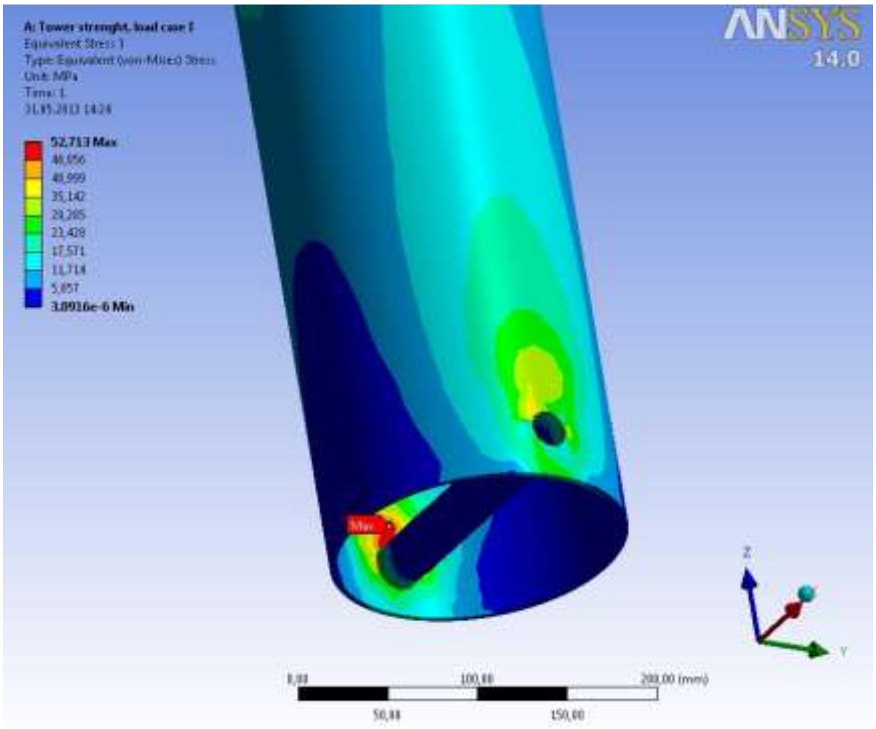


Figure 6 – Stress at 3rd segment

### 5.2 Base strength

The tower base assembly is analysed in a separate FEM model to save calculation resources. The model consists of three solid parts: two base angles and a base plate (see Figure 7). The angles are connected to the base plate with bonded contacts that have bolt washer sizes. These connections are used to derive the reactions forces for all of the bolts that connect the bodies. The areas where parts touch but are not connected in any way have frictionless supports. The base plate is grounded with four fixed supports that are of bolt washer sizes. These areas are used to derive the reaction forces for the embedded foundation block bolts.

Hexahedron mesh is used with a dominant element size of 5 mm which results in a collective mesh of 106 490 elements and 481 479 nodes.

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Soola 1a
51013 Tartu, Estonia
Email: info@mywind.ee
Website: www.mywind.ee

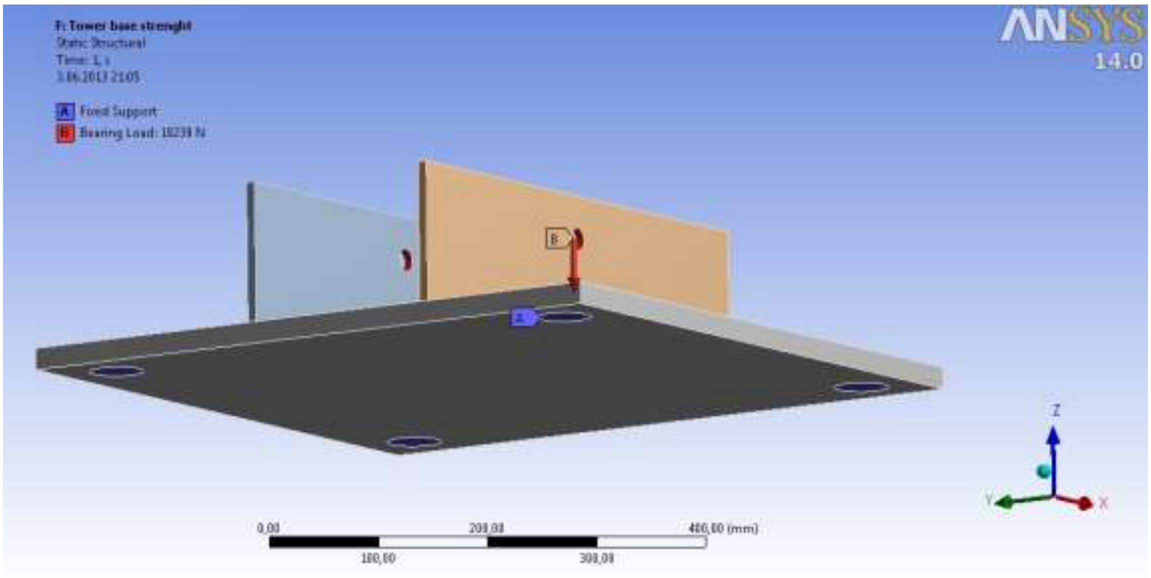


Figure 7 – Boundary conditions

The assembly is loaded with a bearing load type boundary condition. The magnitude and direction of load is derived from the tower strength calculation model (see Figure 8)

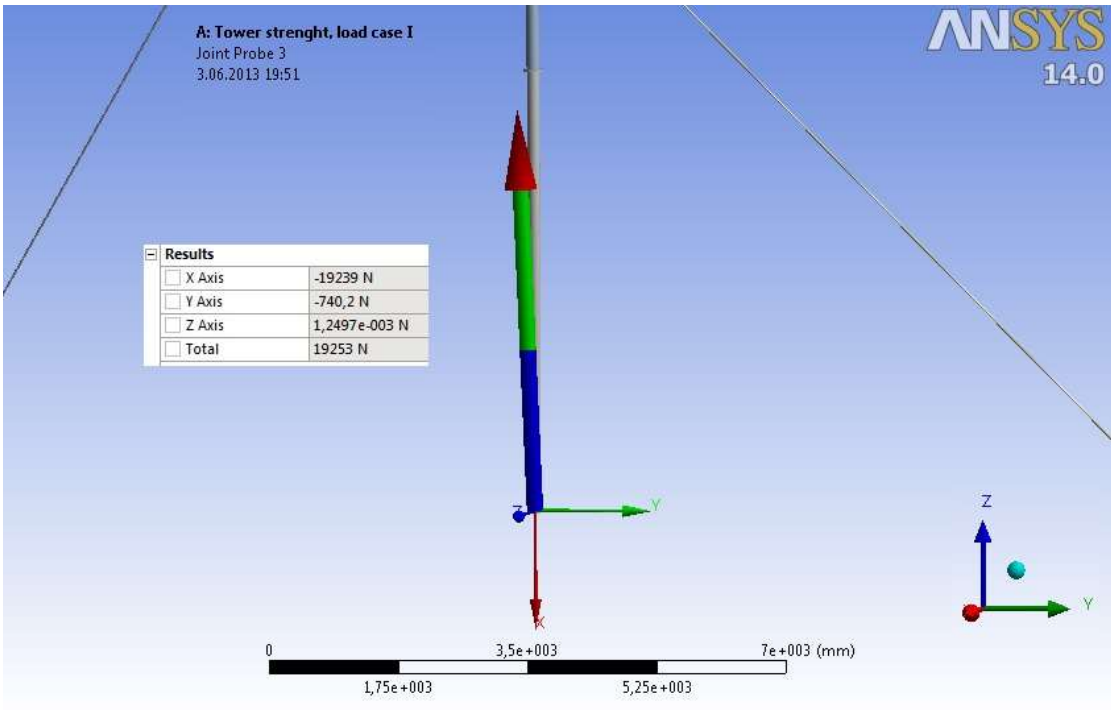


Figure 8 – Tower base reaction force

The highest stresses occur on bottom of the base plate, where the embedded foundation bolts connect to the base plate (see Figure 9). These values go up to 126 MPa. 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.

The allowed stress according to appendix 01 is $\frac{355}{1{,}1 \cdot 1{,}35} = 239\ \mathrm{MPa} > 126\ \mathrm{MPa}$ (OK). See appendix 01 – partial safety factor for materials 1.1; partial safety factor for loads 1.35.

**Therefore the tower base strength against ultimate load cases is guaranteed.**

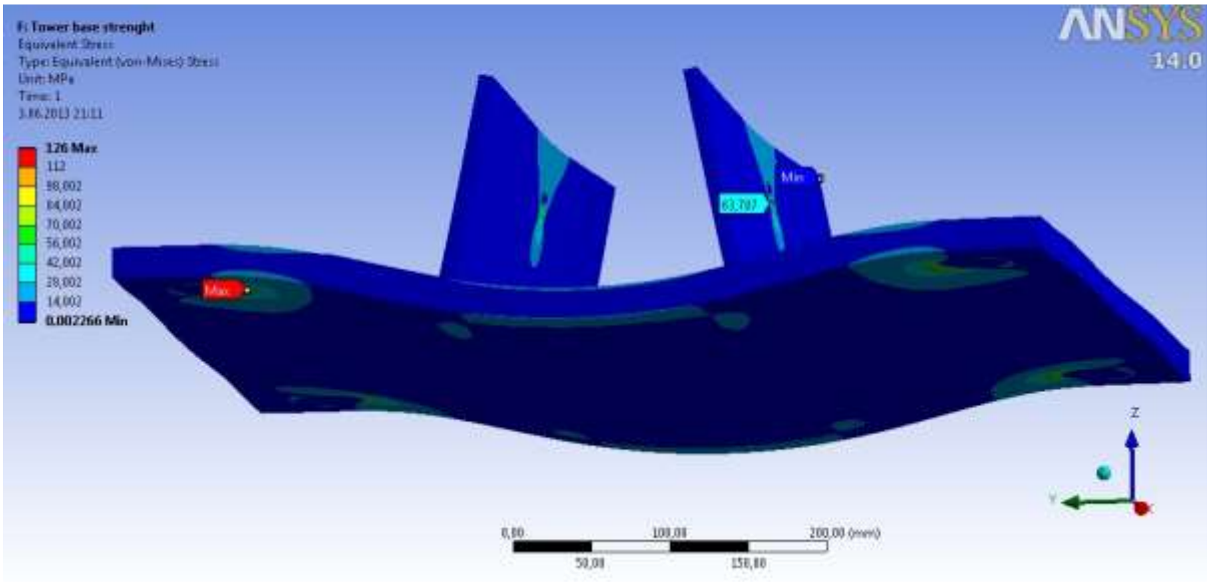


Figure 9 – Stress at tower base

## 5.3 Intermediate plate strength

The intermediate plate for wires is analysed in a separate FEM model to save calculation resources. The model consists of one solid part (see Figure 10). The part is loaded with a two bearing type boundary conditions. The magnitude and direction of the loads is derived from the tower strength calculation model (see Figures 11 and 12). For safety reasons it is assumed that under certain conditions the element is loaded with maximum forces from both wires. The part is grounded with a fixed support. This condition is used to derive reaction forces for the embedded foundation block bolt.

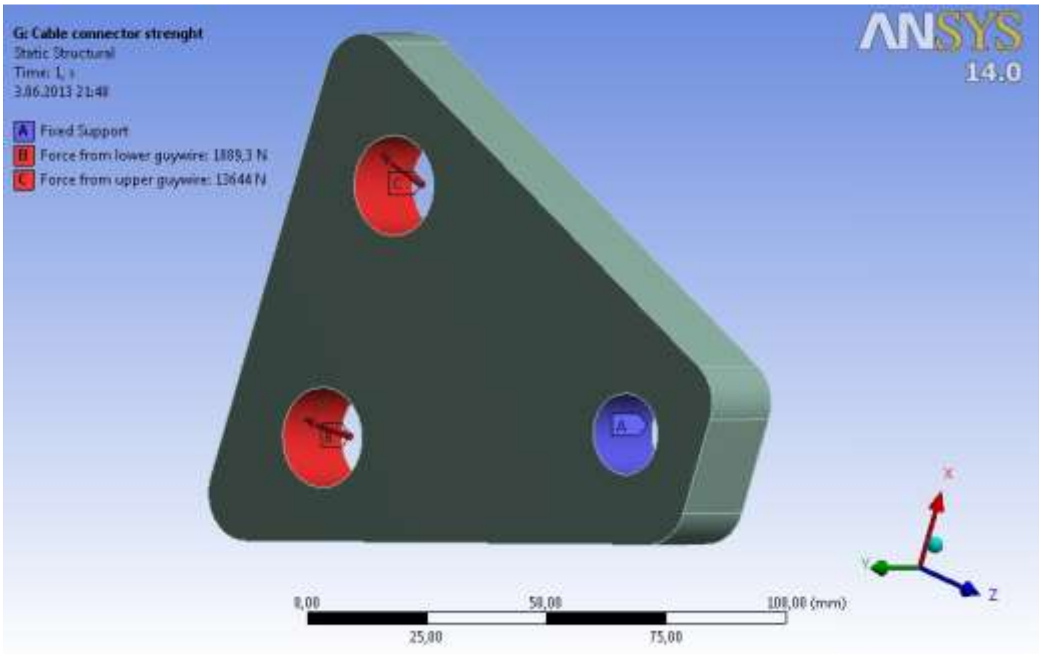


Figure 10 – Boundary conditions

Hexahedron mesh is used with a dominant element size of 5 mm which results in a collective mesh of 1 915 elements and 8 550 nodes.

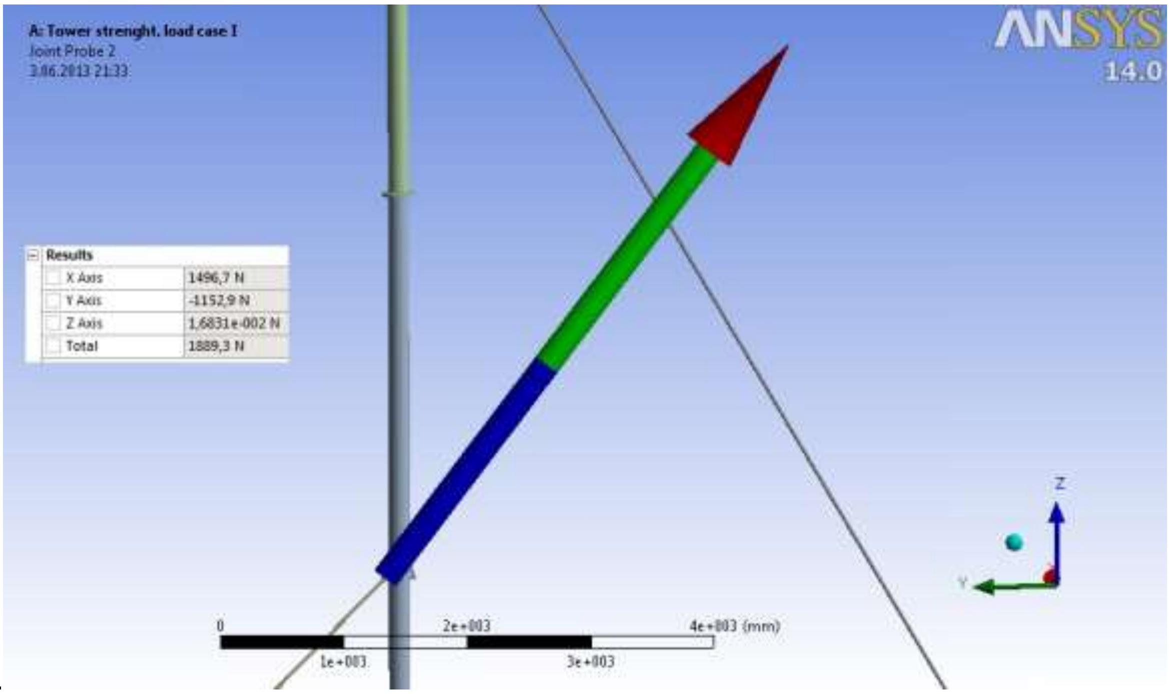


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Figure 11 – Lower wire reaction force

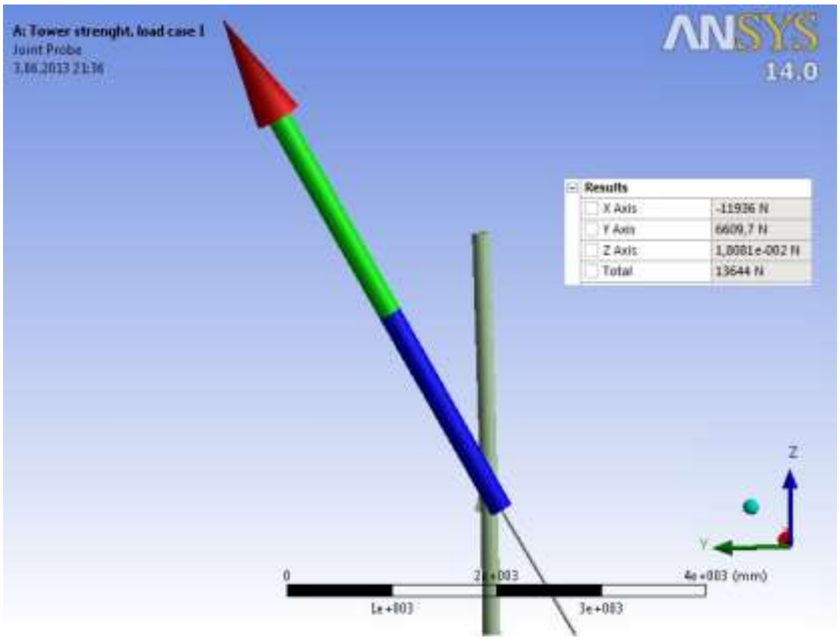


Figure 12 – Upper wire reaction

The highest stresses occur at the upper wire connection hole (see Figure 13). These values go up to 106 MPa. For safety reasons the highest occurring stress is considered in the safety factor calculation and no compensation for the singularity effect is used.

The allowed stress according to appendix 01 is $\frac{355}{1{,}1 \cdot 1{,}35} = 239 \text{ MPa} > 106 \text{ MPa}$ (OK). See appendix 01 – partial safety factor for materials 1.1; partial safety factor for loads 1.35.

**Therefore the intermediate plate strength against ultimate load cases is guaranteed.**

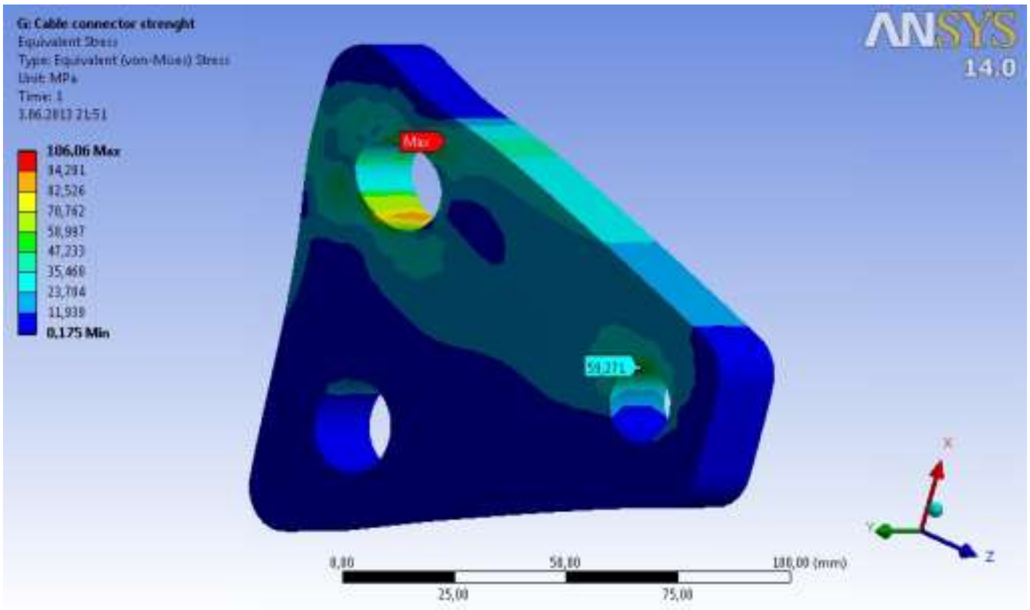

Figure 13 – Stress at wire connector

## 5.4 Cables and components

The cables and associated components are calculated according to empirical formulas available from equipment providers. As only standard components are used, then the results obtained for one manufacturer are also applicable to all other manufacturers that produce according to same standards. For the tension equipment only the highest forces obtained are analysed and it is assumed that all cables and other associated parts should withstand that force. Since same equipment is used for the upper and lower level guywires, only the upper level is analysed as it exhibits much greater forces (see Figures 11 and 12). The level consist of a screw pin shackle, steel cable, turnbuckle another screw pin shackle and an eye nut (see Figure 14). For safety reasons it is assumed that under certain conditions the elements that have two wires attached to them are loaded with maximum forces from both wires.

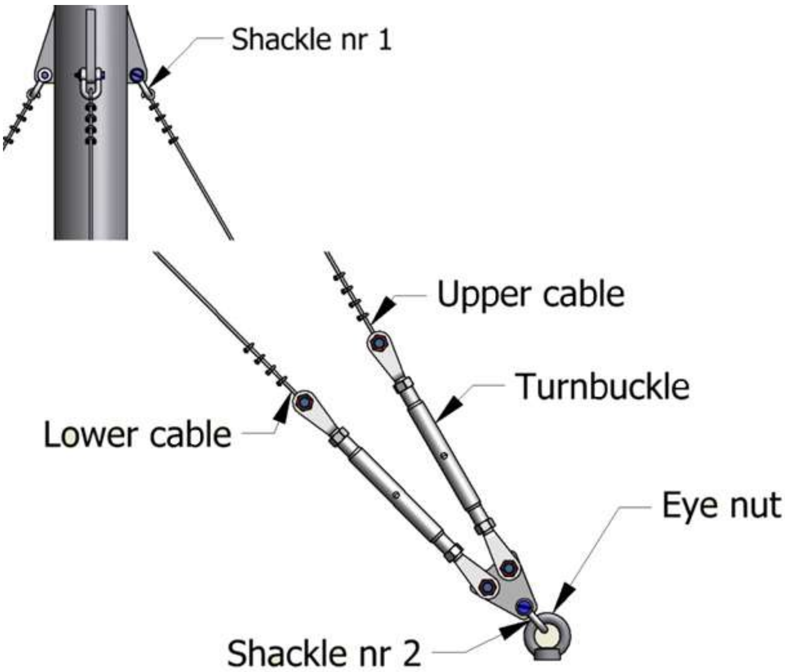


Figure 14 – Cables and associated equipment

**Shackle nr 1**

| | |
|---|---|
| Type: | Bow Shackle screw pin, forged, M16 |
| Standard: | DIN 82016 |
| Minimum breaking force: | 12 t; 117 720 N |
| Safety factor: | 6 |

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Soola 1a
51013 Tartu, Estonia
Email: info@mywind.ee
Website: www.mywind.ee

| | |
|---|---|
| Maximum tension force: | 13 644 N |
| Braking resistance: | $\frac{117\ 720}{6} = 19\ 620\ \mathrm{N} > 13\ 644\ \mathrm{N}$ (OK) |

**Steel cable**

| | |
|---|---|
| Type: | 6x19-FC Ordinary alt Seale, 8 mm |
| Standard: | EN 12385-4 |
| Minimum breaking force: | 37 400 N |
| Safety factor: | 2 |
| Maximum tension force: | 13 644 N |
| Braking resistance: | $\frac{37\ 400}{2} = 18\ 700\ \mathrm{N} > 13\ 644\ \mathrm{N}$ (OK) |

**Turnbuckle**

| | |
|---|---|
| Type: | Jaw/Jaw, M22 |
| Standard: | DIN 1478 |
| Working load limit: | 11 t; 107 910 N |
| Safety factor: | 5 |
| Maximum tension force: | 13 644 N |
| Braking resistance: | $\frac{107\ 910}{5} = 21\ 582\ \mathrm{N} > 13\ 644\ \mathrm{N}$ (OK) |

**Shackle nr 2**

| | |
|---|---|
| Type: | Bow Shackle screw pin, forged, M16 |
| Standard: | DIN 82016 |
| Working load limit: | 12 t; 117 720 N |
| Safety factor: | 6 |
| Maximum tension force: | 15 476 N (vector sum) |
| Braking resistance: | $\frac{117\ 720}{6} = 19\ 620\ \mathrm{N} > 15\ 476\ \mathrm{N}$ (OK) |

**Eye nut**

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Soola 1a
51013 Tartu, Estonia
Email: info@mywind.ee
Website: www.mywind.ee

| | |
|---|---|
| Type: | Steel SS 1370 or equivalent, M27 |
| Standard: | DIN 582 |
| Working load limit: | 7.2 t; 70 632 N (loaded under an angle) |
| Safety factor: | 4 |
| Maximum tension force: | 15 476 N (vector sum) |
| Breaking resistance: | $\frac{70\ 632}{4} = 17\ 658\ \mathrm{N} > 15\ 476\ \mathrm{N}$ (OK) |

**Therefore the strength of cables and associated equipment against ultimate load cases is guaranteed.**

### 5.5 Bolted connections

The bolted connections are calculated according to appendix 05. 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: | M16x55 – 8.8 |
| Assembly torque: | 222 Nm |
| Tensile stress area of the bolt: | 200 mm$^2$ (gross), 157 mm$^2$ (net) |
| Slip factor for preloaded bolts: | 0.4 |
| Shear resistance per shear plane: | $F_{v,Rd} = \frac{\propto_v f_{ub} A}{\gamma_{M2}} = \frac{0.6 \cdot 800 \cdot 200}{1.1 \cdot 1.35} = 64\ 646$ N |
| Tension resistance: | $F_{t,Rd} = \frac{k_2 f_{ub} A_S}{\gamma_{M2}} = \frac{0.9 \cdot 800 \cdot 157}{1.1 \cdot 1.35} = 76\ 121$ N |
| Preload: | $F_{p,C} = 0.7 f_{ub} A_s = 0.7 \cdot 800 \cdot 157 = 87\ 920$ N |
| Maximum reaction forces (see Figure 15): | x: 81 N (shear) |
| | y: 198 N (shear) |
| | z: 8 802 N (tension) |

Shear resistance: $213\ \mathrm{N} < 64\ 646\ \mathrm{N}$ (OK)

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Soola 1a
51013 Tartu, Estonia
Email: info@mywind.ee
Website: www.mywind.ee

Tension resistance: $8\,802 \text{ N} < 76\,121 \text{ N}$ (OK)

Combined shear and tension resistance: $\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4 \cdot F_{t,Rd}} = \frac{213}{64\,646} + \frac{8\,802}{1.4 \cdot 76\,121} = 0.09 < 1$ (OK)

Slip resistance: $F_{s,Rd} = \frac{k_s n \mu (F_{p,c} - 0.8 F_{t,Ed})}{\gamma_{M3}} = \frac{1 \cdot 2 \cdot 0.4(87\,920 - 0.8 \cdot 8802)}{1.1 \cdot 1.35} = 43\,570 \text{ N} > 213 \text{ N}$ (OK)

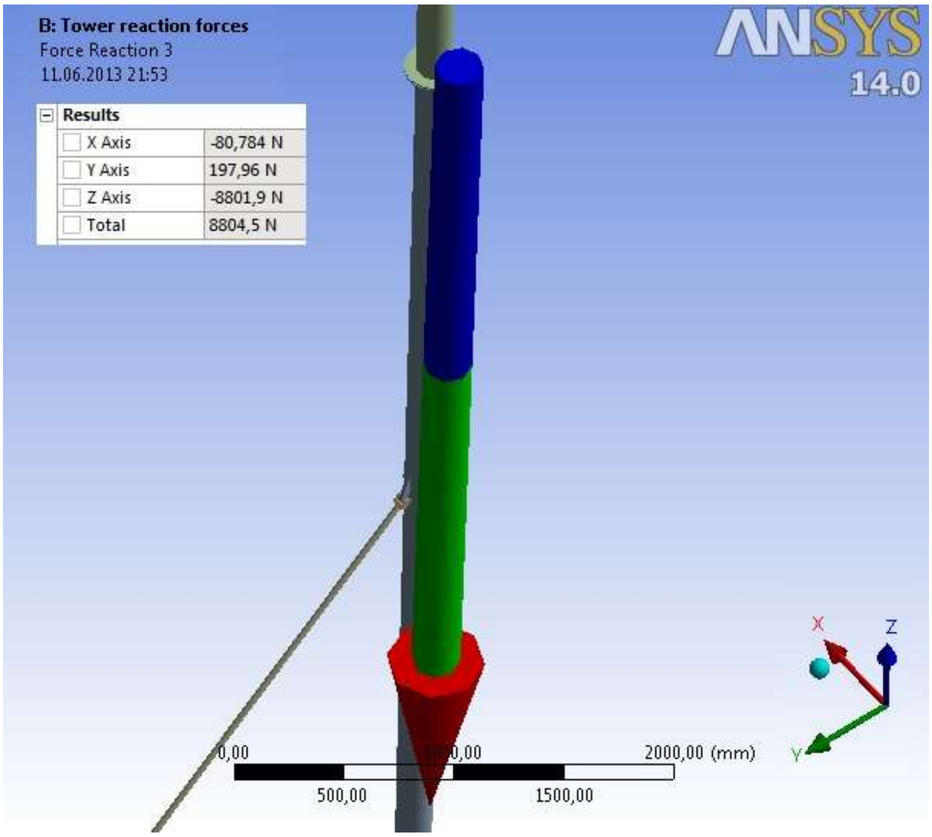


Figure 15 – Third segment second segment highest reaction forces

**Second segment to first segment connection**

| | |
|---|---|
| Bolt size and strength class: | M16x55 – 8.8 |
| Assembly torque: | 222 Nm |
| Tensile stress area of the bolt: | 200 mm$^2$ (gross), 157 mm$^2$ (net) |
| Slip factor for preloaded bolts: | 0.4 |
| Shear resistance per shear plane: | $F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}} = \frac{0.6 \cdot 800 \cdot 200}{1.1 \cdot 1.35} = 64\,646$ N |
| Tension resistance: | $F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}} = \frac{0.9 \cdot 800 \cdot 157}{1.1 \cdot 1.35} = 76\,121$ N |

Preload: $F_{p,C} = 0.7 f_{ub} A_s = 0.7 \cdot 800 \cdot 157 = 87\,920$ N

Maximum reaction forces (see Figure 16): x: 0 N (shear)

y: 7 N (shear)

z: 4 013 N (tension)

Shear resistance: $7 \text{ N} < 64\,646 \text{ N}$ (OK)

Tension resistance: $4\,013 \text{ N} < 76\,121 \text{ N}$ (OK)

Combined shear and tension resistance: $\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4 \cdot F_{t,Rd}} = \frac{7}{64\,646} + \frac{4\,013}{1.4 \cdot 76\,121} = 0.04 < 1$ (OK)

Slip resistance: $F_{s,Rd} = \frac{k_s n \mu (F_{p,c} - 0.8 F_{t,Ed})}{\gamma_{M3}} = \frac{1 \cdot 2 \cdot 0.4 (87\,920 - 0{,}8 \cdot 4013)}{1.1 \cdot 1.35} = 45\,634 \text{ N} > 7 \text{ N}$ (OK)

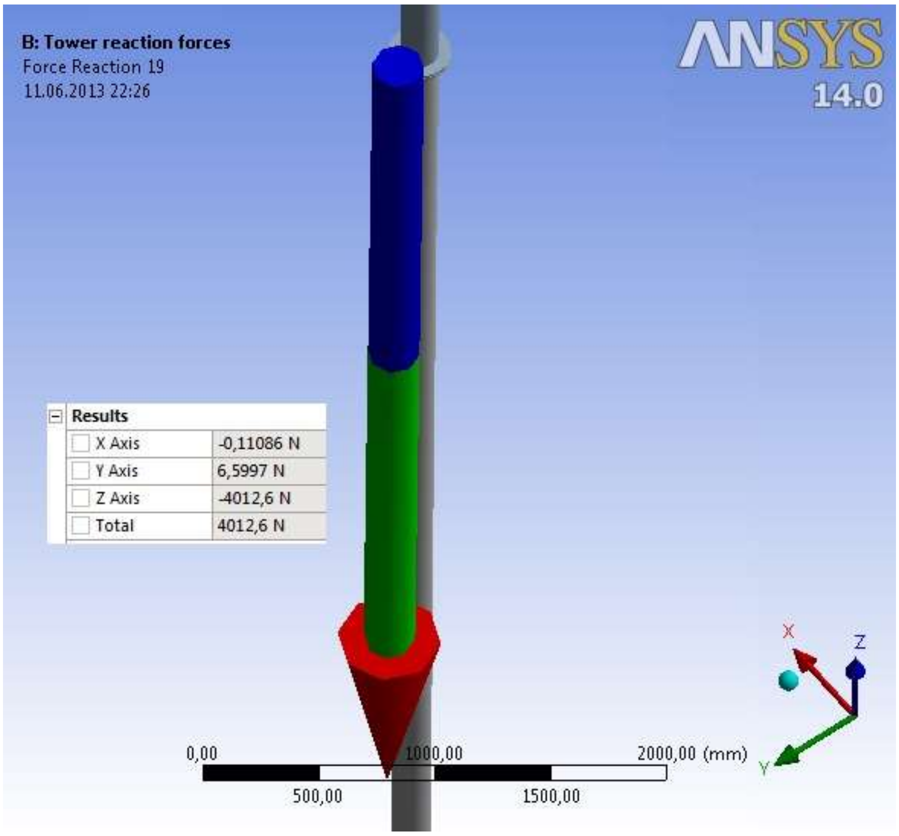


Figure 16 – Second segment first segment reaction forces

**First segment to base angle connection**

Bolt size and strength class: M20x240 – 8.8

Assembly torque: 434 Nm

Tensile stress area of the bolt: 314 mm$^2$ (gross), 245 mm$^2$ (net)

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 17): x: 19 239 N (shear)

y: 741 N (shear)

z: 0 (tension)

Shear resistance: $19\,254 \text{ N} < 101\,494 \text{ N}$ (OK)

Tension resistance: $0 \text{ N} < 118\,787 \text{ N}$ (OK)

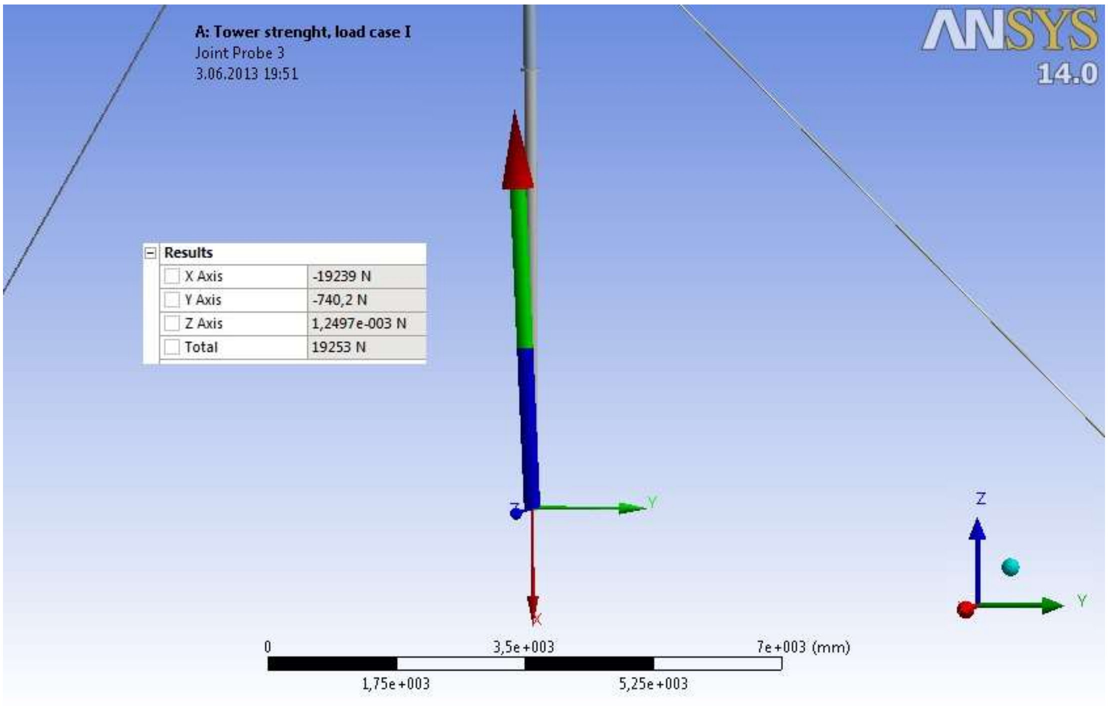


Figure 17 – First segment base angle reaction forces

my!*WIND* Ltd.
Soola 1a
51013 Tartu, Estonia
Email: info@mywind.ee
Website: www.mywind.ee

**Base angle to base plate connection**

Bolt size and strength class: M20x45 – 8.8

Assembly torque: 434 Nm

Tensile stress area of the bolt: 314 $mm^2$ (gross), 245 $mm^2$ (net)

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 18): x: 4 014 N (shear)

y: 1 603 N (shear)

z: 238 N (tension)

Shear resistance: $4\,322 \text{ N} < 101\,494 \text{ N}$ (OK)

Tension resistance: $238 \text{ N} < 118\,787 \text{ N}$ (OK)

Combined shear and tension resistance: $\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4 \cdot F_{t,Rd}} = \frac{4322}{101\,494} + \frac{238}{1.4 \cdot 118\,787} = 0.05 < 1$ (OK)

Slip resistance: $F_{s,Rd} = \frac{k_s n \mu (F_{p,c} - 0{,}8 F_{t,Ed})}{\gamma_{M3}} = \frac{1 \cdot 2 \cdot 0.4 (137\,200 - 0.8 \cdot 238)}{1.1 \cdot 1.35} = 73\,810 \text{ N} > 4\,014 \text{ N}$ (OK)

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Soola 1a
51013 Tartu, Estonia
Email: info@mywind.ee
Website: www.mywind.ee

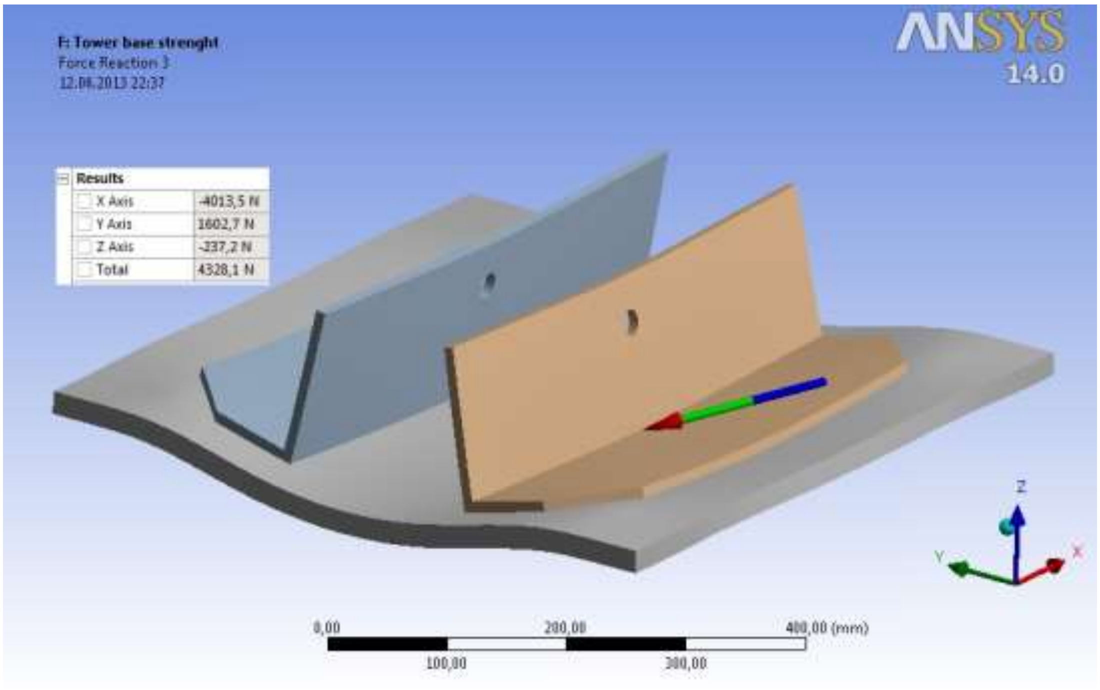


Figure 18 – Base angle to base plate reaction forces

**Base plate to foundation connection**

| | |
|---|---|
| Bolt size and strength class: | M27x550 – 8.8 |
| Assembly torque: | 1080 Nm |
| Tensile stress area of the bolt: | 572 $mm^2$ (gross), 459 $mm^2$ (net) |
| 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: | $B_{p,Rd} = \frac{\propto_v \pi d_m t_p f_u}{\gamma_{M2}} = \frac{0.6 \cdot 3.14 \cdot 44.1 \cdot 10 \cdot 640}{1.1 \cdot 1.35} = 358\,074$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 19): | x: 2 907 N (shear) |
| | y: 4 370 N (shear) |
| | z: 4 813 N (tension) |

Shear resistance: $5\ 249\ \mathrm{N} < 184\ 888\ \mathrm{N}$ (OK)

Tension resistance: $4\ 813\ \mathrm{N} < 222\ 545\ \mathrm{N}$ (OK)

Tear out resistance: $4\ 813\ \mathrm{N} < 304\ 363\ \mathrm{N}$ (OK)

Combined shear and tension resistance: $\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4 \cdot F_{t,Rd}} = \frac{5\ 249}{184\ 888} + \frac{4\ 813}{1.4 \cdot 222\ 545} = 0.04 < 1$ (OK)

Slip resistance: $F_{s,Rd} = \frac{k_s n \mu (F_{p,c} - 0.8 F_{t,Ed})}{\gamma_{M3}} = \frac{1 \cdot 2 \cdot 0.4 (257\ 040 - 0.8 \cdot 4\ 813)}{1.1 \cdot 1.35} = 136\ 399\ \mathrm{N} > 5\ 249\ \mathrm{N}$ (OK)

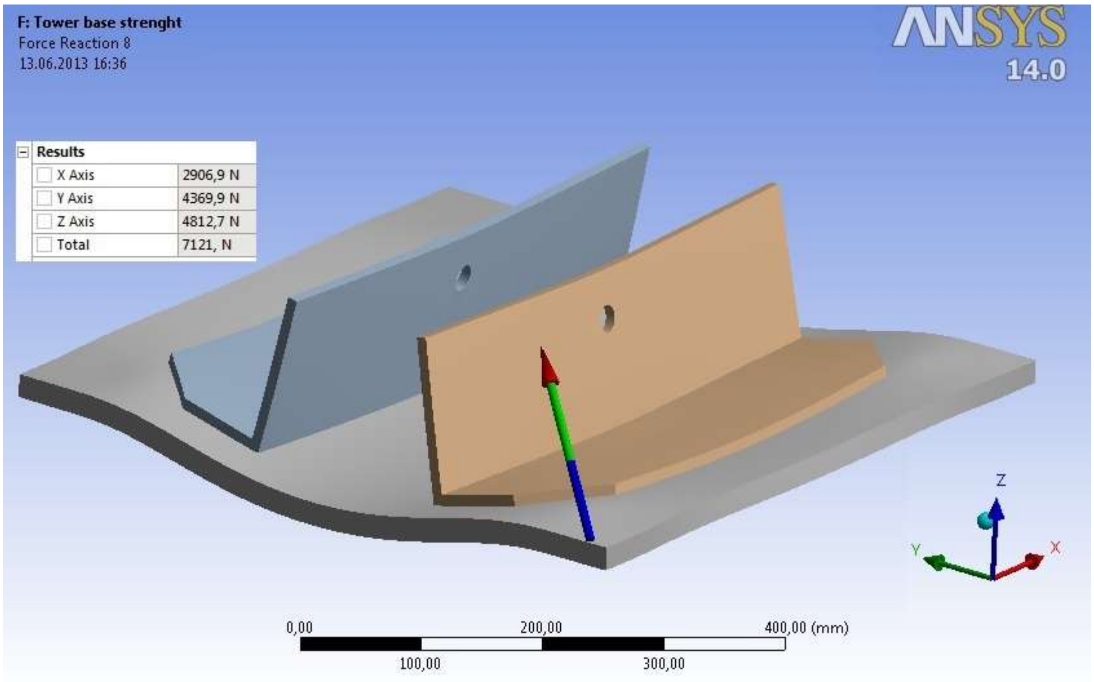


Figure 19 – Base plate to foundation reaction forces

**Intermediate plate to foundation connection**

Bolt size and strength class: M27x550 – 8.8

Assembly torque: 1080 Nm

Tensile stress area of the bolt: 572 mm$^2$ (gross), 459 mm$^2$ (net)

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: $B_{p,Rd} = \frac{\propto_v \pi d_m t_p f_u}{\gamma_{M2}} = \frac{0.6 \cdot 3.14 \cdot 44.1 \cdot 10 \cdot 640}{1.1 \cdot 1.35} = 358\ 074$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 20): x: 12 840 N (tension)

y: 0 N (shear)

z: 9 100 N (shear)

Shear resistance: $9\ 100\ \mathrm{N} < 184\ 888\ \mathrm{N}$ (OK)

Tension resistance: $12\ 840\ \mathrm{N} < 222\ 545\ \mathrm{N}$ (OK)

Tear out resistance: $12\ 840\ \mathrm{N} < 304\ 363\ \mathrm{N}$ (OK)

Combined shear and tension resistance: $\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4 \cdot F_{t,Rd}} = \frac{9\ 100}{184\ 888} + \frac{12\ 840}{1.4 \cdot 222\ 545} = 0.09 < 1$ (OK)

Slip resistance:$F_{s,Rd} = \frac{k_s n \mu (F_{p,c} - 0.8 F_{t,Ed})}{\gamma_{M3}} = \frac{1 \cdot 2 \cdot 0.4(257\ 040 - 0.8 \cdot 12\ 840)}{1.1 \cdot 1.35} = 132\ 939\ \mathrm{N} > 9\ 100\ \mathrm{N}$ (OK)

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

my!WIND Ltd.
Soola 1a
51013 Tartu, Estonia
Email: info@mywind.ee
Website: www.mywind.ee

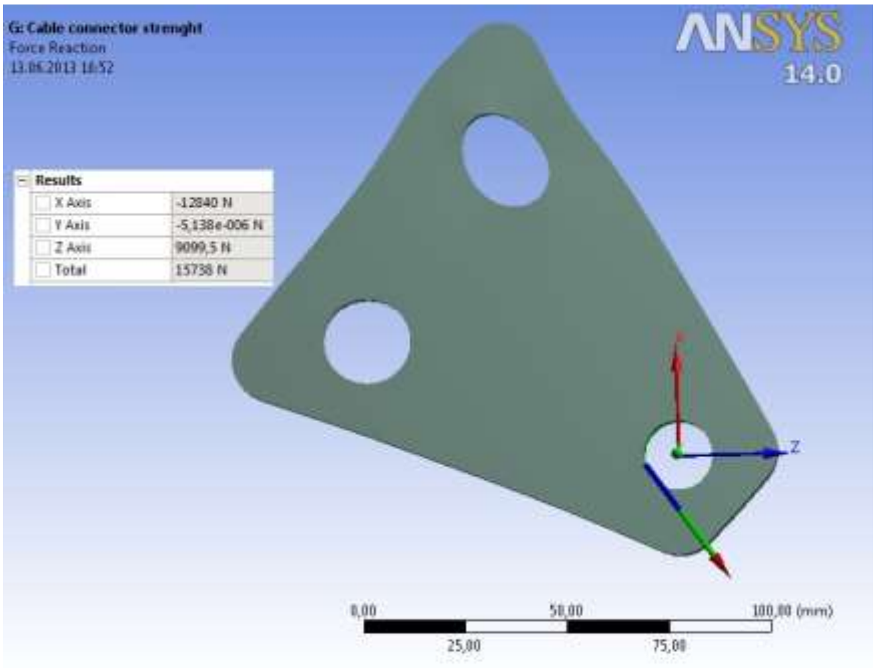


Figure 20 – Intermediate plate to foundation reaction forces

### 5.6 Foundation stability

The side foundation block acts as a counterweight for the forces in the cables. All the blocks are identical and therefore it is assumed that one block alone must be able to take the maximum forces from the wires. The foundation is designed in a manner that all the forces are received by the inner steel structure. Therefore, only the stability conditions against pulling out, tilting and pulling through the soil are analysed and no stress values are considered. The acting loads are taken from Table 1 and are applied considering their nature and direction.

The centre foundation block acts as a solid base for the tower and is subjected only to compressive forces. The foundation is designed in a manner that all the forces are received by the inner steel structure. Therefore, only the stability condition against sinking is analysed and no stress values are considered. The acting loads are taken from Table 1 and are applied considering their nature and direction.

Only load case I: Parked wind loading maximum exposure is presented in the report, as it results in the highest reaction forces for side foundation blocks.

Soil material is chosen for the worst soil conditions. Therefore, if the foundation fulfils the stability requirements for this soil, it simultaneously will fulfil them for stronger soils.

Soil material: clay, sandy clay, silty clay, clayey silt, silt and sandy silt.

Lateral bearing (below natural grade): 15 710 Pa/m

my!*WIND* Ltd.
Soola 1a
51013 Tartu, Estonia
Email: info@mywind.ee
Website: www.mywind.ee

Lateral sliding resistance: 6 220 Pa

Tension force upper cable: 13 644 N

Tension force lower cable: 1890 N

Maximum reaction forces (see Figure 21): x: 8159 N (horizontal)

y: 13152 N (vertical)

Lifting resistance: $\frac{W \cdot g}{\gamma_{M3}} = \frac{2365 \cdot 9.81}{1.1 \cdot 1.35} = 15\,623 > 13\,152$ (OK)

Sliding resistance: $\frac{h/3 \cdot S_1 wh + \mu wl}{1.1 \cdot 1.35} = \frac{\frac{0.85}{3} \cdot 15\,710 \cdot 1.15 \cdot 0.85 + 6220 \cdot 1.15 \cdot 1.15}{1.1 \cdot 1.35} = 8468\text{ N } > 8159\text{ N}$ (OK)

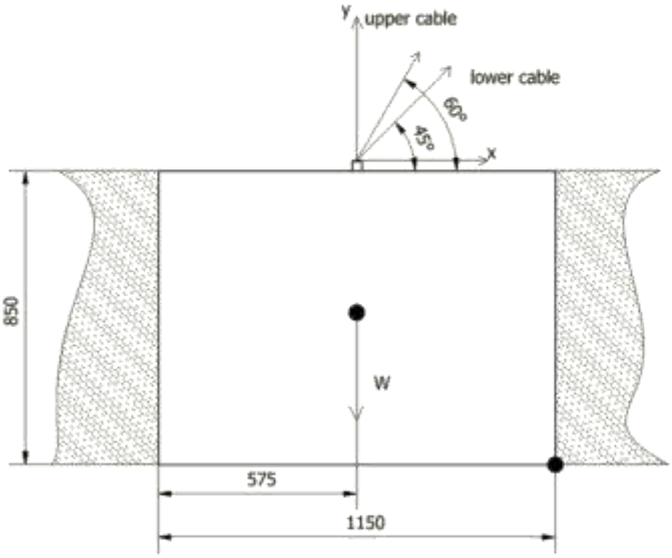


Figure 21 – Side foundation stability calculation

Soil material: clay, sandy clay, silty clay, clayey silt, silt and sandy silt.

Allowable foundation pressure 71 820 Pa

Maximum reaction forces (see Figure 8): x: 19 239 N (vertical)

y: 740,2 N (horizontal)

z: 0 N (horizontal)

Pressure resistance: $\frac{Plw}{\gamma_{M3}} = \frac{71\,820 \cdot 1.15 \cdot 1.15}{1.1 \cdot 1.35} = 64\,405 > 13\,152$ (OK)

**Therefore the foundation stability against ultimate load cases is guaranteed.**

## 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: $\frac{355}{236} = 1.50$ (load case I: Parked wind loading maximum exposure).

Lowest safety factor for cable braking: $\frac{37\,400}{13\,644} = 2.74$ (load case I: Parked wind loading maximum exposure)

Lowest safety factor for bolted connections appears for first segment to base angle connection for shear resistance: $\frac{150\,719}{19\,254} = 7.8$ (load case I: Parked wind loading maximum exposure).

Lowest safety factor for foundation slipping: $\frac{12\,574}{8159} = 1.54$ (load case I: Parked wind loading maximum exposure)