

WEC Development Project

Winter Semester 2025/2026

Final Design Report



OPTIMUS
SYRIA
160/5.0



Project Name: Optimus Syria

Sub Project: Foundations

Authors:

Jehad Albaali

Amgad Gerges

Reda Deif

Matriculation Number:

780109

770563

780081

Supervisor:

Prof. Dr.-Ing. Torsten Faber

Date: 29.01.2026

Table Of Content

1.	Introduction	1
2.	Design Criteria and Codes	2
2.1.	Applicable Codes and Standards	2
2.2.	Design Philosophy	2
2.3.	Safety Factors	2
3.	Soil Profile for Selected site (Qattinah)	3
4.	Selection of foundation type	4
4.1.	Reasons for estimated shallow foundation type	4
4.2.	Reasons for estimated round shape type	4
5.	Loads at Tower Base	4
6.	Estimated Concrete Dimensions of Circular Gravity Foundation	4
7.	Vertical Loads	5
7.1.	Weight of Raft	5
7.2.	weight of Backfill	6
7.3.	Total Vertical Load	6
8.	Geotechnical design	6
8.1.	Load Eccentricity	6
8.2.	Bearing Capacity	7
8.3.	Verification against Ground Failure	11
8.4.	Verification against Uplift	12
8.5.	Verification against Overturning Moment	12
8.6.	Verification against Sliding	12
9.	Structure Design	13
9.1.	Concrete Cover	13
9.2.	Design For Bending Moment and shear	14
9.3.	Wind Turbine Foundation Reinforcement: Types and Drawings	15
10.	Connection Between Tower and Foundation: Types and Design	19

11.	Check punching Shear _____	22
12.	Seismic Analysis _____	23
12.1.	Seismic Forces & Moments _____	24
12.2.	Global Stability Verification _____	24
12.3.	Soil-Structure Interaction Analysis _____	25
13.	3D Modelling of Wind turbine Foundation _____	26
14.	Estimated Cost of Wind Turbine Foundation _____	28
15.	References _____	29

List Of Figures

Figure 1-	Soil Profile For Qattinah _____	3
Figure 2-	Estimated Concrete Dimensions for Raft _____	4
Figure 3 -	Cross-section of raft foundation showing ground level and backfill _____	5
Figure 4 -	Effective area for a circular foundation. _____	7
Figure 5 -	Soil pressure distribution for raft foundation _____	11
Figure 6 –	Reinforcement details for the Bottom mesh of a wind turbine foundation _____	16
Figure 7 -	Reinforcement details for the Top mesh of a wind turbine foundation _____	16
Figure 8 -	Reinforcement details of the pedestal and anchor bolts layout for the wind turbine foundation _____	17
Figure 9 -	Pedestal reinforcement, link shear, and top & bottom mesh details _____	17
Figure 10 –	Details of reinforcement chairs for the top mesh (Link shear) _____	18
Figure 11 –	Anchor Bolts System _____	19
Figure 12 -	Embedded Steel Ring System _____	20
Figure 13 –	Anchor Bolts Distribution _____	21
Figure 14 –	Seismic Load _____	23
Figure 15 –	3D Model of wind turbine foundation _____	26
Figure 16 –	3D Model for Anchor Bolts _____	26
Figure 17 -	3D Model of the flange assembly illustrating the distribution of anchor bolt holes. _	27

1. Introduction

Onshore wind energy projects play a vital role in meeting the growing demand for sustainable power generation, particularly in regions with high wind potential such as Syria. The increasing size and capacity of modern wind turbines, including 5 MW units, impose significant structural and geotechnical challenges on foundation design. Wind turbine foundations must safely resist large vertical loads, horizontal forces, and overturning moments generated by wind action, rotor operation, and dynamic effects throughout the service life of the structure.

Gravity foundations are widely adopted for onshore wind turbines due to their structural simplicity, economic efficiency, and effectiveness in resisting overturning moments through self-weight and soil bearing resistance. In this project, **a circular gravity foundation** is selected to support a **5 MW onshore** wind turbine, considering its suitability for local soil conditions and its ability to provide adequate stability and load transfer to the underlying ground.

The foundation design process involves a comprehensive geotechnical assessment to evaluate the soil profile and bearing capacity, followed by the determination of loads at the tower base. Stability checks are performed against overturning, sliding, excessive settlement, and ground failure, while eccentricity effects are carefully assessed to ensure uniform stress distribution beneath the foundation. Both ultimate and serviceability limit states are considered in accordance with Eurocode provisions to achieve the required levels of safety and structural performance.

This report presents a detailed and systematic approach to the structural and geotechnical design of the wind turbine foundation, combining analytical calculations with practical engineering considerations. Emphasis is placed on durability, constructability, and cost-effective design solutions suitable for onshore wind energy development in **(Qattinah) Syria**.

2. Design Criteria and Codes

The design of the wind turbine foundation is carried out in accordance with internationally recognized standards to ensure structural safety, serviceability, and durability throughout the intended service life of the turbine. The adopted design methodology is based on the limit state approach, considering both ultimate and serviceability requirements in compliance with the Eurocode framework.

2.1. Applicable Codes and Standards

The following codes and standards are used in the design of the gravity foundation for the onshore 5 MW wind turbine:

- **EN 1990 (Eurocode 0):** Basis of Structural Design
- **EN 1991 (Eurocode 1):** Actions on Structures, including wind and imposed loads
- **EN 1992 (Eurocode 2):** Design of Concrete Structures
- **EN 1997 (Eurocode 7):** Geotechnical Design

2.2. Design Philosophy

The foundation design follows the **Limit State Design (LSD)** approach as specified in the Eurocodes. The following limit states are considered:

- **Ultimate Limit State (ULS):**
To ensure adequate safety against structural failure, loss of equilibrium, excessive deformation, and soil bearing capacity failure.
- **Serviceability Limit State (SLS):**
To control settlements, rotations, and crack widths within acceptable limits to maintain proper turbine operation and durability.

2.3. Safety Factors

Partial safety factors are applied to loads, material strengths, and soil parameters in accordance with Eurocode requirements. These factors account for uncertainties in loading conditions, material properties, and geotechnical behavior, ensuring a sufficient margin of safety in the design.

3. Soil Profile for Selected site (Qattinah)

Depth	Soil Type	Modulus of Elasticity	Friction Angle	Cohesion	Density	Poisson Ratio
(m)	Soft to medium clay (CL / CL-OL)	KN/m ²	[°]	KN/m ²	KN/m ³	
0 - 18	Medium cohesive clay	7000	20°	15	18	0.45
18 - 45	Stiff to very stiff clay (CL-CH)	11,000	22°	20	19	0.45
45 – 80	Very stiff clay	20,000	24°	25	20	0.45
80 – 100	Soft to medium clay (CL / CL-OL)	25,000	26°	30	21	0.40

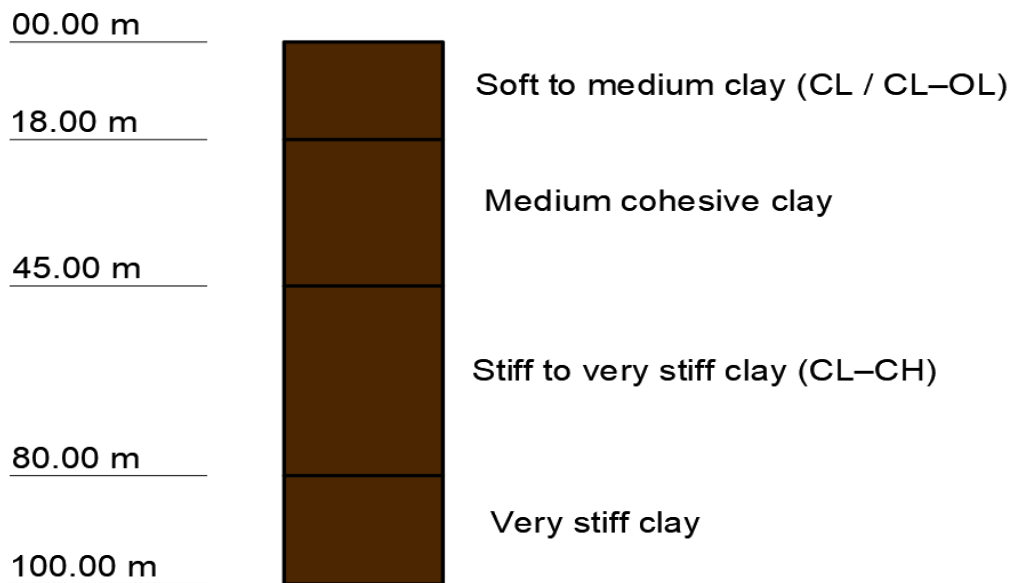


Figure 1- Soil Profile For Qattinah

➤ Soil Properties:

Friction Angle (ϕ) = 20°

Cohesion(C) = 15 KN/m²

Soil unit Weight γ_s = 18 KN/m³

Backfill unit Weight γ_s = 18 KN/m³

4. Selection of foundation type

The selection of the foundation type is a key aspect of wind turbine design, as it affects structural safety, constructability, and project economy. Considering the loading characteristics of the 5 MW wind turbine and the site conditions, a shallow circular gravity foundation is adopted as an efficient and practical solution. This choice ensures adequate load transfer, structural stability, and cost-effective construction.

4.1. Reasons for estimated shallow foundation type

- More effective cost than deep foundation
- Simple and Fast to Construct
- we don't need specialized equipment , which reduces construction time and cost
- ground water is at depth 74.5 m so it will not be affected by ground water.

4.2. Reasons for estimated round shape type

- Uniform Load and Moment Distribution
- Smaller Perimeter for the Same Area (amount of concrete and steel Rebar decrease)

5. Loads at Tower Base

- Vertical Load at Tower Base (V_{Base}) = 9,807 kN
- Horizontal Load (H) = 2,368 kN
- Overturning Moment (M): 236,822 kN.m

6. Estimated Concrete Dimensions of Circular Gravity Foundation

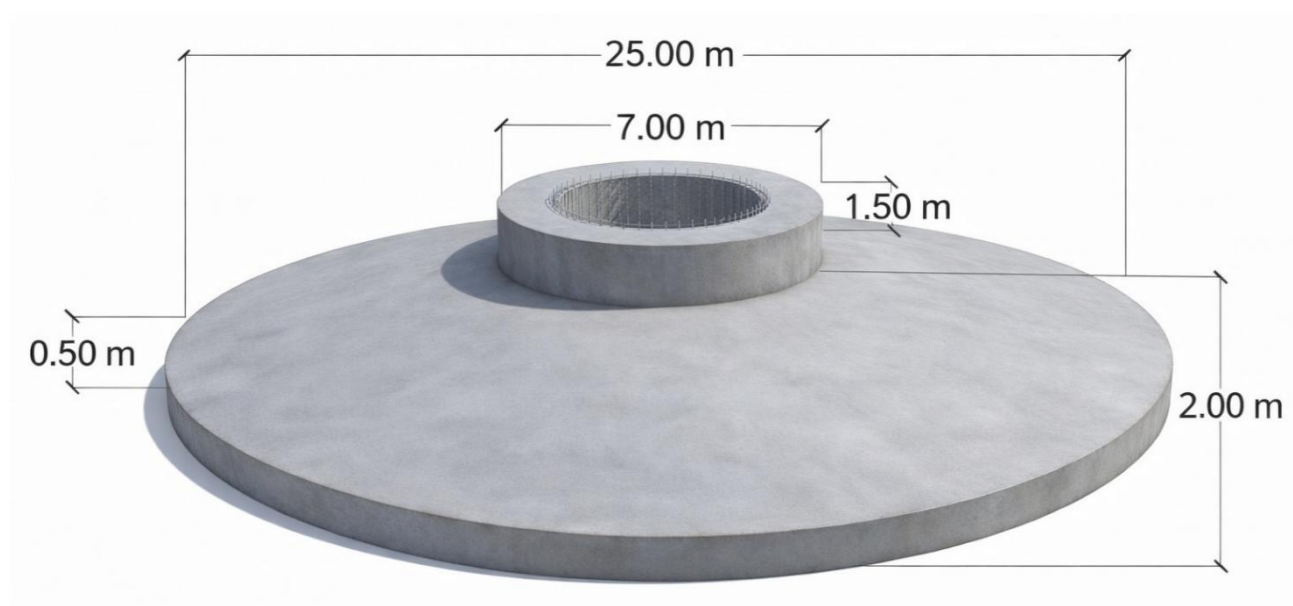


Figure 2- Estimated Concrete Dimensions for Raft

- Diameter of Base Plate (BP) = 25 m
- Diameter of Pedestal = 7 m
- Height Pedestal = 1.5 m
- Height of Middle slab = 2 m
- Height of Frustum (HF) = 1.5 m
- Height of Edge slab (HE) = 0.5 m

7. Vertical Loads

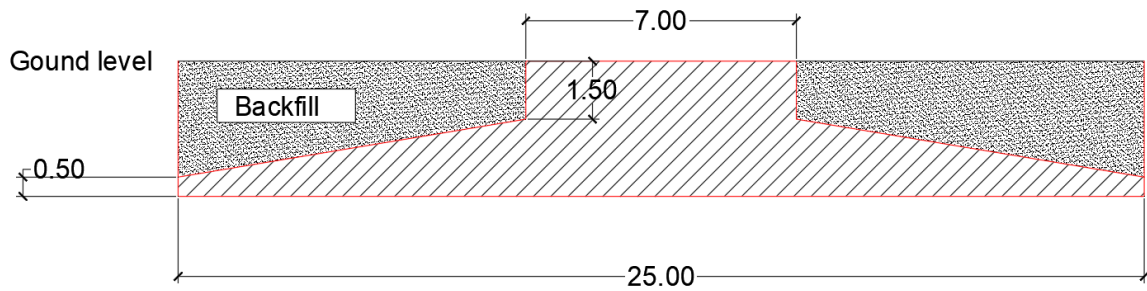


Figure 3 - Cross-section of raft foundation showing ground level and backfill

7.1. Weight of Raft

$$A_{BP} = \frac{\pi}{4} * D^2 = \frac{\pi}{4} * (25)^2 = 490.9 \text{ m}^2$$

$$A_{Pedestal} = \frac{\pi}{4} * (D_{Pedestal})^2 = \frac{\pi}{4} * (7)^2 = 38.5 \text{ m}^2$$

$$V_{concrete} = A_{BP} * h_{edge} + A_{Pedestal} * h_{Pedestal}$$

$$+ \frac{1}{3} \pi * h_{Frustum} * \left[\left(\frac{D_{BP}}{2} \right)^2 + \left(\frac{D_{BP}}{2} \right) * \left(\frac{D_{Pedestal}}{2} \right) + \left(\frac{D_{Pedestal}}{2} \right)^2 \right]$$

$$V_{concrete} = 490.9 * 0.5 + 38.5 * 1.5 + \frac{1}{3} \pi * 1.5 * \left[\left(\frac{25}{2} \right)^2 + \left(\frac{25}{2} \right) * \left(\frac{7}{2} \right) + \left(\frac{7}{2} \right)^2 \right] = 637 \text{ m}^3$$

$$\text{Unit weight for reinforced concrete} = \gamma_c = 25 \text{ KN/m}^3$$

$$G_c = \gamma_c * V_{concrete} = 25 * 637 = 15925 \text{ KN}$$

7.2. weight of Backfill

$$V_{\text{backfilling}} = \frac{1}{3} \pi * h_{\text{Frustum}} * \left[\left(\frac{D_{\text{BP}}}{2} \right)^2 + \left(\frac{D_{\text{BP}}}{2} \right) * \left(\frac{D_{\text{Pedestal}}}{2} \right) + \left(\frac{D_{\text{Pedestal}}}{2} \right)^2 \right]$$

$$V_{\text{backfilling}} = \frac{1}{3} \pi * 1.5 * \left[\left(\frac{25}{2} \right)^2 + \left(\frac{25}{2} \right) * \left(\frac{7}{2} \right) + \left(\frac{7}{2} \right)^2 \right] = 333.4 \text{ m}^3$$

$$\text{unit weight for the backfilling material} = \gamma_s = 18 \text{ KN/m}^3$$

$$G_s = \gamma_s * V_{\text{backfilling}} = 18 * 333.4 = 6002 \text{ KN}$$

$$G_{\text{Total}} = G_c + G_s = 15925 + 6002 = 21927 \text{ KN}$$

$$V_{\text{Base}} = 9807 \text{ KN}$$

7.3. Total Vertical Load

$$V_{\text{Total}} = G_{\text{Total}} + V_{\text{Base}} = 21927 + 9807 = 31734 \text{ KN}$$

8. Geotechnical design

Geotechnical design focuses on the study of soil and rock properties that support civil engineering structures. It aims to ensure the safety and stability of structures by analyzing soil strength, deformation, and interaction with foundations. Different potential failure modes are considered during the design process to prevent structural or soil failure. Geotechnical design is carried out in accordance with established engineering codes and reliable ground investigation data.

8.1. Load Eccentricity

- $H_{\text{Found}} = 3.5 \text{ m}$
- $F_{\text{res}} = 2,400 \text{ KN}$
- $M_{\text{res}} = 228,422 \text{ KN.m}$
- $M_{\text{Total}} = M_{\text{res}} + F_{\text{res}} * H_{\text{Foundation}}$
- $M_{\text{Total}} = 228,422 + 2,400 * 3.536822 = 236822 \text{ KN.m}$
- $e = \frac{M_{\text{Total}}}{V_{\text{Total}}} = \frac{236,822}{31734} = 7.46 \text{ m}$

8.2. Bearing Capacity

- According to the effective area for a circular foundation can be expressed as a rectangular area that origins from an elliptical area see figure

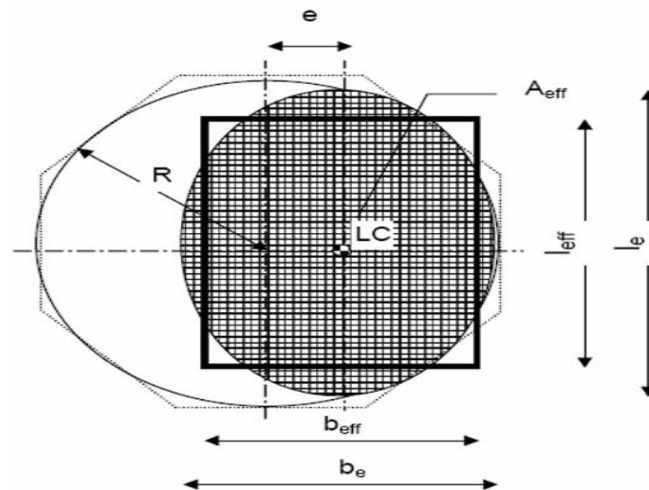


Figure 4 - Effective area for a circular foundation.

$$b_e = 2(R - e) = 2(12.5 - 6.6) = 11.8 \text{ m}$$

$$I_e = 2R\sqrt{1 - \left(1 - \frac{be}{2R}\right)^2} = 2 * 12.5\sqrt{1 - \left(1 - \frac{11.8}{2*12.5}\right)^2} = 21.23 \text{ m}$$

$$A_{eff} = 2 \left[R^2 * \arccos\left(\frac{e}{R}\right) - e\sqrt{R^2 - e^2} \right] = 2 \left[12.5^2 * \arccos\left(\frac{6.6}{12.5}\right) - 6.6\sqrt{12.5^2 - 6.6^2} \right]$$

$$A_{eff} = 176.92 \text{ m}^2$$

$$L_{eff} = \sqrt{A_{eff} \frac{I_e}{b_e}} = \sqrt{176.92 * \frac{21.23}{11.8}} = 17.84 \text{ m}$$

$$B_{eff} = \frac{L_{eff}}{I_e} b_e = \frac{17.84}{21.23} * 11.8 = 9.92 \text{ m}$$

if $e > 0.3 D \rightarrow$ we have to calculate q_{b2}

$$e = 7.46 \text{ m} < 0.3 D = 7.5 \text{ m} \rightarrow \text{neglect } q_{b2}$$

$$q_{b1} = cN_c s_c d_c i_c g_c b_c + qN_q s_q d_q i_q g_q b_q + 0.5 \gamma' B_{eff} N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma \rightarrow \text{first rupture}$$

Where

q_b is the bearing capacity for the plate

c is the cohesion

q is the surrounding load at the foundation level

γ' is the effective bulk density of the soil

B_{eff} is the effective width of the footing

N_c, N_q, N_γ are bearing capacity factors depending on the friction angle

s_c, s_q, s_γ are correction factors for the shape of the footing

d_c, d_q, d_γ are correction factors for the foundation depth

i_c, i_q, i_γ are correction factors for inclined loading

g_c, g_q, g_γ are correction factors for inclined adjacent ground surface

b_c, b_q, b_γ are correction factors for inclined base area of the footing

$$q = \gamma * d$$

where

d is depth of foundation $\rightarrow d = 3.5 \text{ m}$

for dry soil $\rightarrow \gamma' = \gamma$

$$q = \gamma * d = 18 * 3.5 = 63 \text{ KN/m}^2$$

$$\begin{array}{l} N_q = \frac{1 + \sin \phi}{1 - \sin \phi} e^{\pi \tan \phi} \\ N_c = \frac{N_q - 1}{\tan \phi} \rightarrow \text{for } \phi > 0 \end{array} \quad \left| \quad \begin{array}{l} N_q = \frac{1 + \sin(20)}{1 - \sin(20)} e^{\pi \tan 20} = 6.39 \\ N_c = \frac{6.39 - 1}{\tan 20} = 14.80 \end{array} \right.$$

$$F(\phi) = 0.08705 + 0.3231 \sin(2\phi) - 0.04836 \sin^2(2\phi)$$

$$F(\phi) = 0.08705 + 0.3231 \sin(2 * 20) - 0.04836 \sin^2(2 * 20) = 0.274$$

$$N_\gamma = F(\phi) \left[\frac{1 + \sin \phi}{1 - \sin \phi} e^{\frac{3\pi}{2} \tan \phi} \right] \rightarrow N_\gamma = 0.274 * \left[\frac{1 + \sin(20)}{1 - \sin(20)} e^{\frac{3\pi}{2} \tan 20} \right] = 3.10$$

$$S_c = 1 + \frac{N_q}{N_c} \frac{B_{eff}}{L_{eff}} \rightarrow \text{for } \phi > 0 \rightarrow S_c = 1 + \frac{6.39}{14.80} * \frac{9.92}{17.84} = 1.24$$

$$s_q = 1 + (\tan \phi) \frac{B_{eff}}{L_{eff}} \rightarrow s_q = 1 + (\tan 20) \frac{9.92}{17.84} = 1.2$$

$$s_\gamma = 1 - 0.4 \frac{B_{eff}}{L_{eff}} \rightarrow s_\gamma = 1 - 0.4 \frac{9.92}{17.84} = 0.8$$

$$d_c = d_q = 1 + 0.35 \frac{d}{B_{eff}} \rightarrow d_c, d_q < 1.7$$

where

***d* is depth of foundation $d = 3.5 \text{ m}$**

$$d_c = d_q = 1 + 0.35 \frac{3.5}{9.92} = 1.12$$

$$d_\gamma = 1 \rightarrow$$

$$m = \text{the smaller of } \begin{bmatrix} m_B = \frac{2 + B_{eff}/L_{eff}}{1 + B_{eff}/L_{eff}} \\ m_L = \frac{2 + L_{eff}/B_{eff}}{1 + L_{eff}/B_{eff}} \end{bmatrix}$$

$$m = \text{the smaller of } \begin{bmatrix} m_B = \frac{2 + 9.92/17.84}{1 + 9.92/17.84} \\ m_L = \frac{2 + 17.84/9.92}{1 + 17.84/9.92} \end{bmatrix} \begin{matrix} = 1.64 \\ = 1.36 \end{matrix}$$

$$i_q = \left(1 - \frac{H}{V + B_{eff} * L_{eff} * c * \cot \phi} \right)^m \rightarrow m = 1.36$$

$$i_q = \left(1 - \frac{4768}{35927 + 9.92 * 17.84 * 15 * \cot 20} \right)^{1.36} = 0.85$$

$$i_c = i_q - \frac{1 - i_q}{N_c \tan \phi} \rightarrow \text{for } \phi > 0$$

$$i_c = 0.85 - \frac{1 - 0.85}{14.80 * \tan 20} = 0.82$$

$$i_\gamma = \left(1 - \frac{H}{V + B_{eff} * L_{eff} * c * \cot \phi} \right)^{m+1}$$

Where

H is the horizontal load

V is the vertical load

$$i_{\gamma} = \left(1 - \frac{4768}{35927 + 9.92 * 17.84 * 15 * \cot 20}\right)^{1.36+1} = 0.76$$

$$g_c = e^{-2\beta \tan \phi} \rightarrow \text{for } \phi > 0$$

$$g_q = g_{\gamma} = 1 - 2\sin 2\beta$$

Where

β is the inclination of the ground regarding the horizontal plane \rightarrow in our case $\beta = 0$

$$g_c = g_q = g_{\gamma} = 1$$

$$b_q = b_{\gamma} = (1 - \alpha \tan \phi)^2$$

Where

α is the inclination of the base of the footing regarding the horizontal plane \rightarrow in our case $\alpha = 0$

$$b_q = b_{\gamma} = 1$$

$$b_c = b_q - \frac{1 - b_q}{N_c \tan \phi} \quad \phi > 0 \rightarrow b_c = 1$$

$$q_{b1} = cN_c s_c d_c i_c g_c b_c + qN_q s_q d_q i_q g_q b_q + 0.5 \gamma' B_{\text{eff}} N_{\gamma} s_{\gamma} d_{\gamma} i_{\gamma} g_{\gamma} b_{\gamma}$$

$$q_{b1} = 15 * 14.8 * 1.24 * 1.12 * 0.82 * 1 * 1 + 63 * 6.39 * 1.2 * 1.12 * 0.85 * 1 * 1 \\ + 0.5 * 18 * 9.92 * 3.10 * 0.8 * 1 * 0.76 * 1 * 1 = \mathbf{880 \text{ KN/} m^2}$$

8.3. Verification against Ground Failure

For onshore wind turbines supported by gravity foundations, verification against ground failure ensures that the soil can safely carry the applied vertical loads and moments. These foundations distribute loads over a large base area, resulting in significant contact pressures. Ground failure may occur in the form of bearing capacity failure or excessive settlement. This verification confirms that soil stresses remain within allowable limits defined by design standards. Adequate ground capacity is essential for the long-term stability of onshore wind turbines.

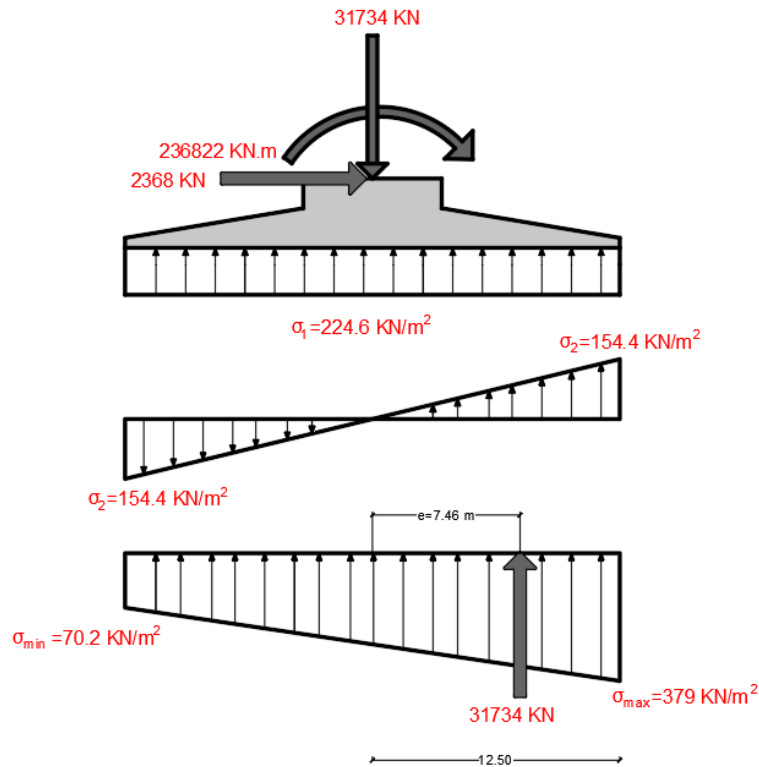


Figure 5 -Soil pressure distribution for raft foundation

$$\sigma_1 = \frac{V}{A_{\text{eff}}} = \frac{31734}{141.3} = 224.6 \text{ kN/m}^2$$

$$\sigma_2 = \frac{M}{Z} = \frac{32M}{\pi D^3} = \frac{32 * 236822}{\pi * 25^3} = 154.4 \text{ kN/m}^2$$

$$\sigma_{\text{Max}} = \sigma_1 + \sigma_2 = 224.6 + 154.4 = 379 \text{ kN/m}^2$$

$$\sigma_{\text{Max}} < q_{\text{all}} = 688.5 \text{ kN/m}^2 \rightarrow \text{ground failure is safe}$$

8.4. Verification against Uplift

Verification against uplift for onshore gravity foundations ensures that the foundation remains fully in contact with the ground under extreme wind loading. Overturning moments can generate uplift forces at the foundation edge despite the large self-weight. This verification considers the weight of the foundation, the soil cover, and load combinations. Loss of contact may reduce stability and increase structural stresses. Adequate uplift resistance is therefore essential for safe turbine operation.

$$\sigma_{\text{Min}} = \sigma_1 - \sigma_2 = 224.6 - 154.4 = 70.2 \text{ kN/m}^2 \rightarrow \text{positive}$$

$\sigma_{\text{Min}} \rightarrow \text{positive} \rightarrow \text{safe against uplift}$

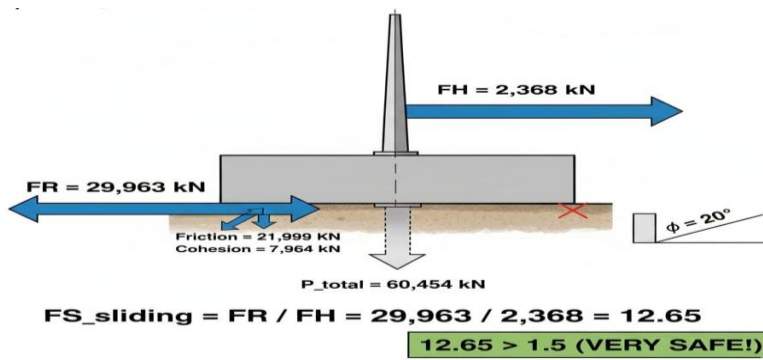
8.5. Verification against Overturning Moment

Verification against overturning moment is a governing design criterion for onshore wind turbine gravity foundations. Due to the large height of turbines, wind loads generate significant overturning moments at the base. The gravity foundation must provide sufficient restoring moment through its weight and base dimensions. This verification ensures that the foundation remains stable without rotation or loss of contact. Overturning stability is critical for the structural integrity of onshore wind turbines.

➤ The load eccentricity (e) lies within the foundation perimeter → **safe against overturning**

8.6. Verification against Sliding

Sliding verification for onshore gravity foundations addresses resistance to horizontal loads induced by wind action. These lateral forces are mainly resisted by the self-weight of the foundation and friction at the soil–foundation interface. Sliding may occur if the soil shear strength or interface friction is insufficient. This verification ensures that resisting forces exceed applied horizontal loads with an adequate safety margin. Preventing sliding is crucial to maintaining turbine position and operational reliability.



9. Structure Design

The structural design of wind turbine foundations aims to ensure that the structure can safely support the turbine throughout its service life. The main objective is to develop clear design criteria and structural details that can be effectively implemented during construction. For the foundation to perform as intended, the functional requirements and site conditions must be clearly defined before the design process begins. These requirements include the expected lifetime of the turbine, the loads induced by wind and turbine operation, soil characteristics, and environmental influences. The role of the structural designer is to ensure that all these conditions are satisfied, with particular emphasis on maintaining structural safety and stability.

9.1. Concrete Cover

The concrete cover is the distance between the surface of the reinforcement closest to the nearest concrete surface (including links and stirrups and surface reinforcement where relevant) and the nearest concrete surface.

Factors affecting concrete cover

- The design life length of the structure
- The quality of the concrete
- The exposure to chlorides
- The variation of wet- and dry state
- The exposure to chemical aggressive environment

In practice, 50 mm is used for foundations in most projects, **so we choose 50 mm as concrete cover for our project**

9.2. Design For Bending Moment and shear

$$F_{ck} = 35 \text{ MPa}$$

$$F_{yk} = 500 \text{ MPa}$$

$$\gamma_c = 1.5$$

$$\gamma_s = 1.25$$

$$\text{Concrete cover} = 50 \text{ mm}$$

$$\text{Effective depth (d)} = 1950 \text{ mm}$$

Concrete Grade	C 12/15	C 16/20	C 20/25	C 25/30	C 30/37	C 35/45	C 40/50	C 45/55	C 50/60
$f_{ck,cyl}^1 \text{ N/mm}^2$	12	16	20	25	30	35	40	45	50
$f_{ck,cube} \text{ N/mm}^2$	15	20	25	30	37	45	50	55	60
¹ $f_{ck,cyl} = f_{ck}$									

- Diameter **D = 25 m** → **r = 12.5 m**
- Ultimate overturning moment **M = 236,822 kN·m**
- Concrete thickness **h = 2.0 m**

$$A_{s,req} = \frac{M_{Ed}}{0.87 f_{yd} z}$$

$$A_{s,req} = \frac{3,016.84 \times 10^6}{0.87 \times 500 \times 1950} \approx 3556.54 \text{ mm}^2/\text{m}$$

Minimum Reinforcement Check

$$A_{s,min} = \rho_{min} b d = 0.0015 \times 1000 \times 1950 = 2925 \text{ mm}^2/\text{m}$$

$$A_{s,req} > A_{s,min}$$

Use $\Phi = 25\text{mm}$

$$A_{s(\Phi=25\text{mm})} = \frac{\pi}{4} 25^2 = 490.87 \text{ mm}^2$$

Number of bars = 8 Ø 25 bar/m

9.3. Wind Turbine Foundation Reinforcement: Types and Drawings

Types

➤ **Top Reinforcement – Tension Zone:**

Resists tensile stresses from wind, tower weight, and overturning moments, maintaining integrity of the upper concrete surface and preventing cracking.

➤ **Bottom Reinforcement – Compression Zone:**

Enhances the foundation's ability to resist compression and distribute loads evenly, preventing concrete crushing and supporting stability.

➤ **Shear Reinforcement – Stirrups / Links**

Resists shear forces from lateral loads and wind, preventing diagonal cracking and increasing shear capacity.

➤ **Vertical / Base Reinforcement**

Vertical bars are placed inside the foundation to **connect the top and bottom reinforcement** and transfer vertical and lateral loads from the tower to the concrete, enhancing foundation integrity and preventing separation between concrete layers.

Drawings

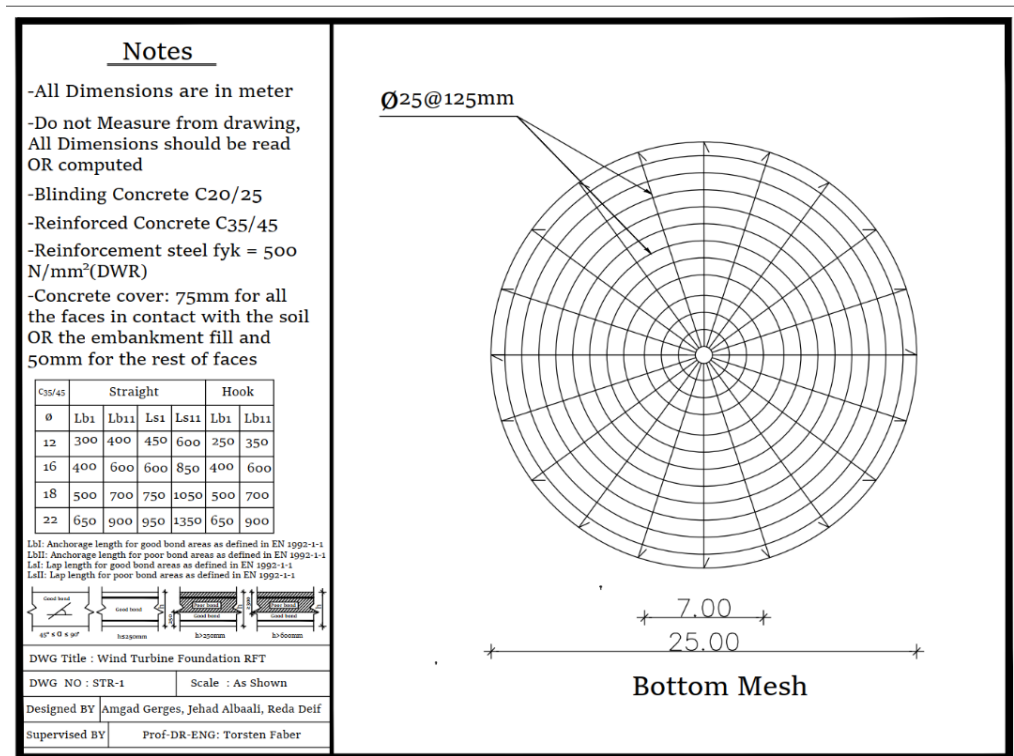


Figure 6 – Reinforcement details for the Bottom mesh of a wind turbine foundation

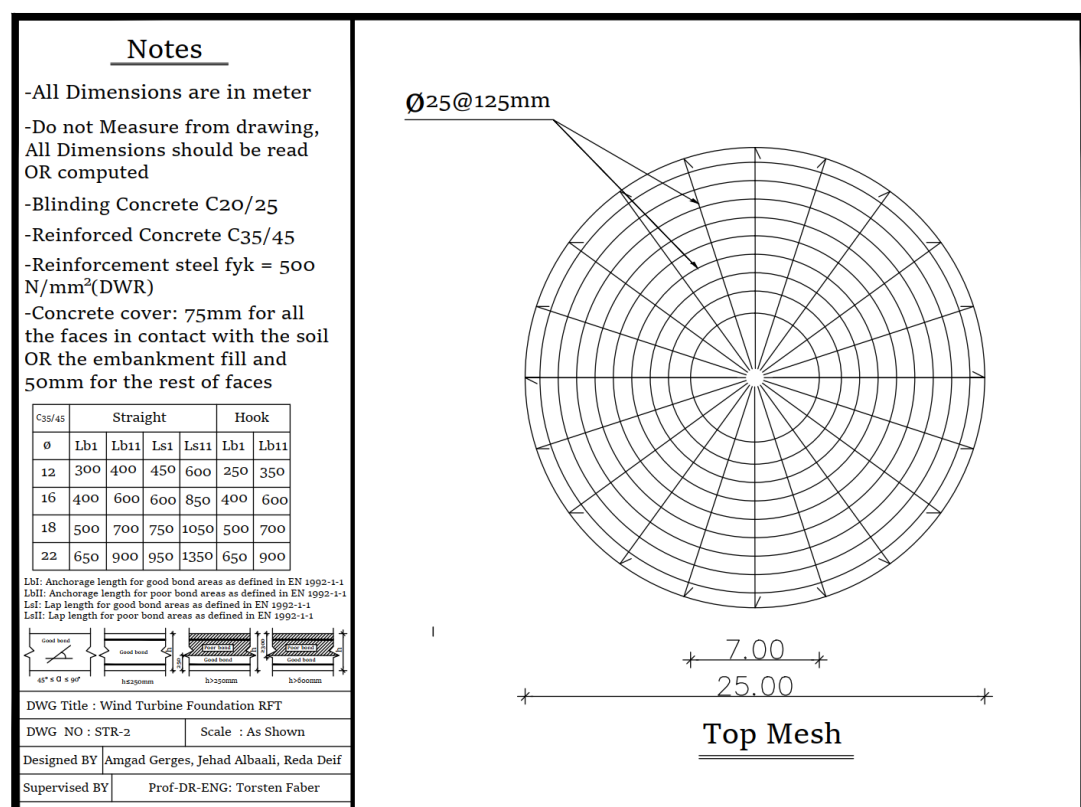


Figure 7 - Reinforcement details for the Top mesh of a wind turbine foundation

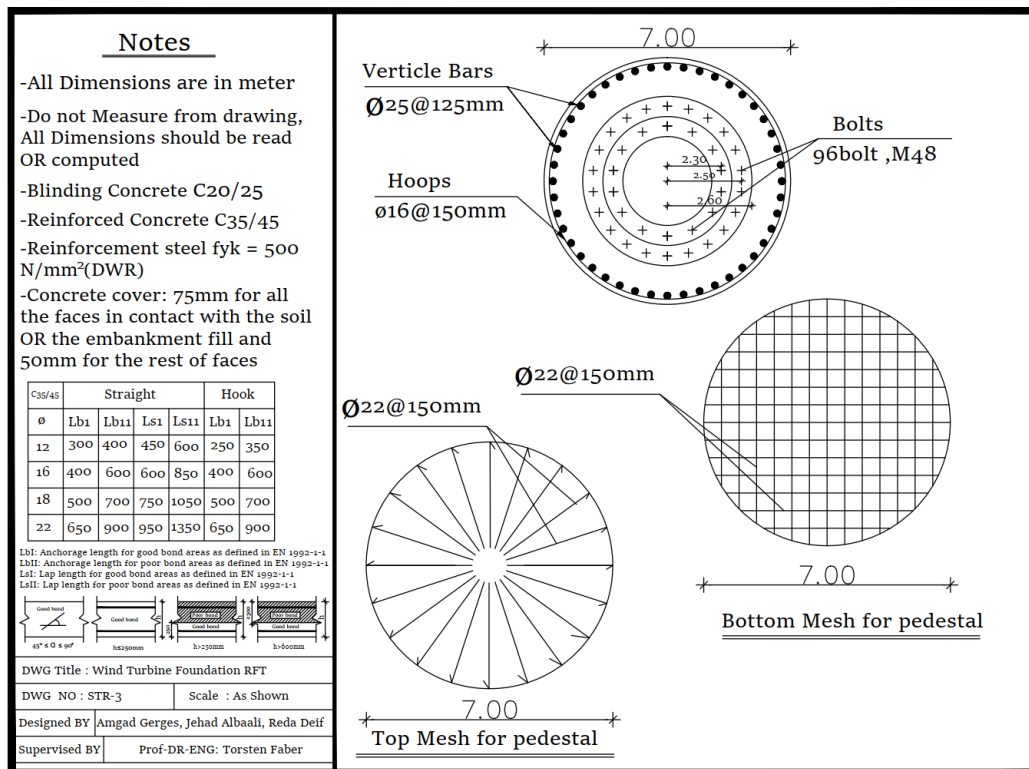


Figure 8 - Reinforcement details of the pedestal and anchor bolts layout for the wind turbine foundation

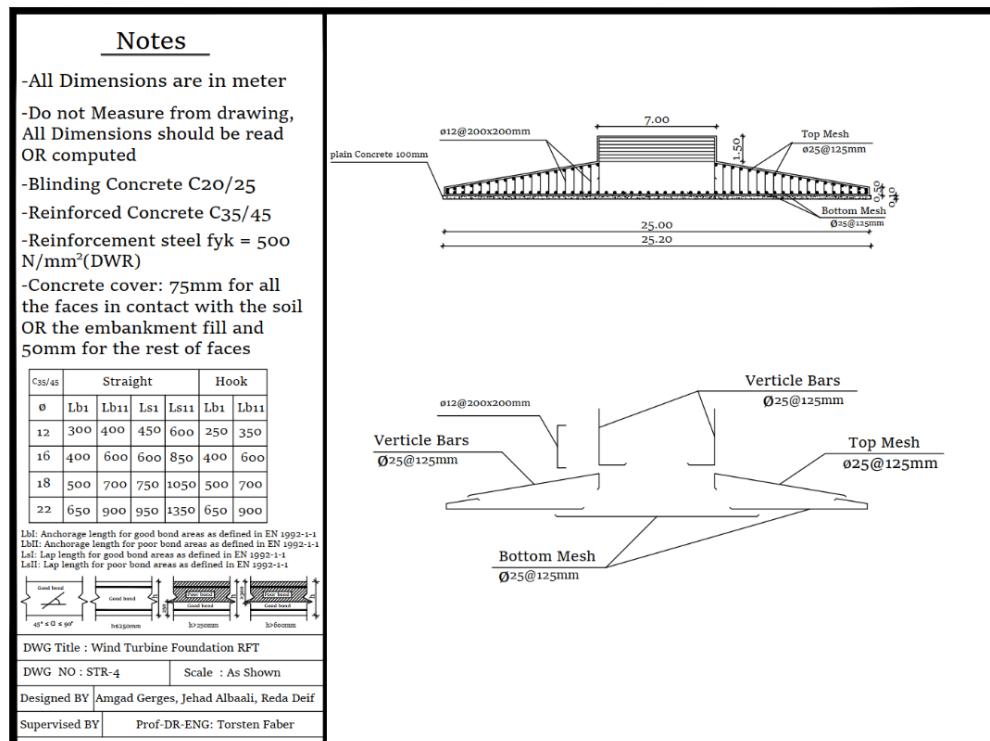


Figure 9 - Pedestal reinforcement, link shear, and top & bottom mesh details

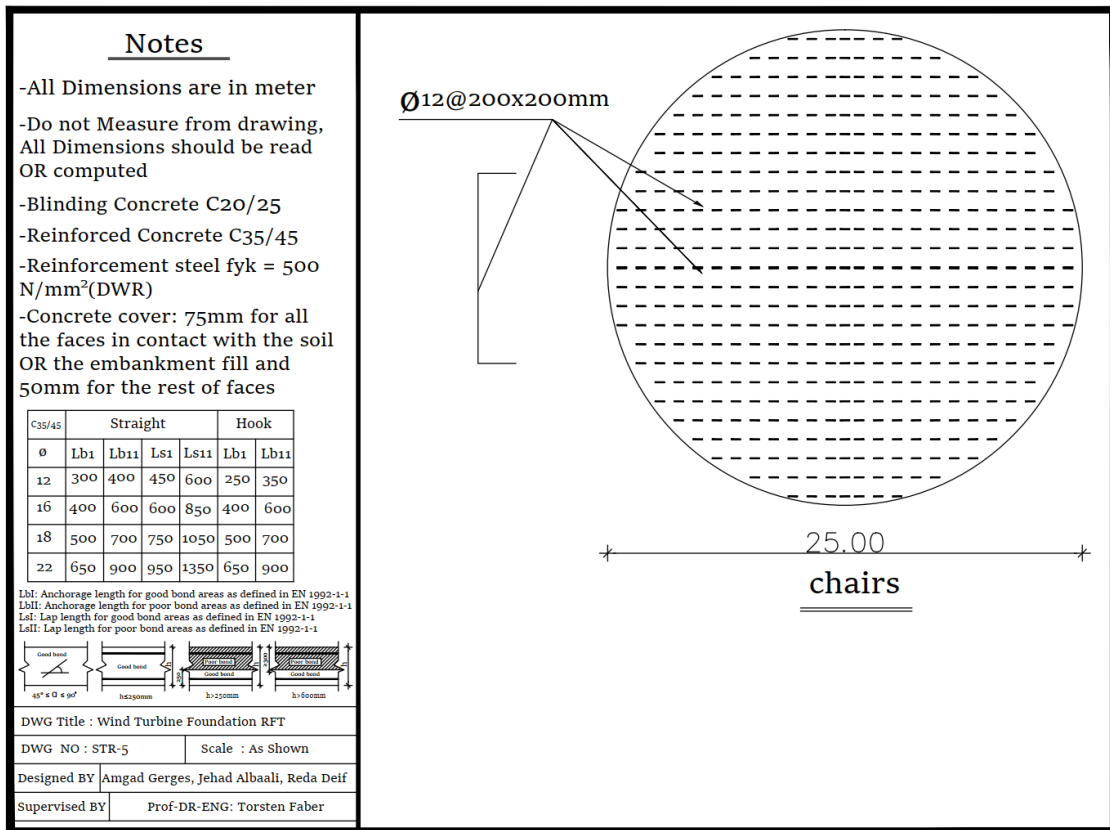


Figure 10 – Details of reinforcement chairs for the top mesh (Link shear)

10.Connection Between Tower and Foundation: Types and Design

➤ Types of Connection Between Tower and Foundation

1- Anchor cage System :

A large group of anchor bolts is connected to two steel rings (upper and lower), These bolts are pre-tensioned to resist tensile forces.



Figure 11 – Anchor Bolts System

Advantages

- Active system: Pre-tensioned bolts directly transfer tensile forces without relying on concrete cracking.
- Clear and predictable load path: bolts carry tension, slab reinforcement carries bending.
- Easy inspection and maintenance: since bolts and flange are accessible
- Simple installation and alignment with tower

Disadvantages

- Requires perfect bolt alignment and accurate pre-tensioning.
- Risk of bolt fatigue or corrosion over time.

More components → more maintenance

2- Embedded Steel Ring System

A steel ring section with holes for reinforcement is embedded 1.5–2 m into the concrete foundation as part of the tower



Figure 12 - Embedded Steel Ring System

Advantages

- No anchor bolts → eliminates bolt fatigue issues.
- Provides very rigid connection.
- Simple external geometry

Disadvantages

- Passive system → tensile forces require concrete cracking to activate reinforcement.
- Hard to inspect or repair if any defect occurs because the steel is embedded.
- Requires very high-quality concrete work around the steel section

➤ **Design of Connection Between Tower and Foundation :Anchor cage System**

Dimensions	Materials {EC2}
<ul style="list-style-type: none"> • Hub Height: 100 m • Tower Base Diameter: 4.8 m • Pedestal Diameter: 7.0 m • Raft Diameter: 25.0 m • Middle Slab Thickness: 2.0 m 	<ul style="list-style-type: none"> • Concrete Class: C35/45 • Cylinder Strength (f_{ck}): 35 MPa • Reinforcing Steel: 8500 ($f_{yk} = 500$ MPa) • Anchor Bolts: Grade 10.9 ($f_{ub} = 1000$ MPa)

Assumptions for Bolts

- Number of bolts (n) = 96
- Bolts Size M48, grade 10.9
- Effective Tensile Area for Bolt = 1533 mm^2

Loads

- $M=236822 \text{ KN.m}$
- Vertical load (N) = 31734 KN

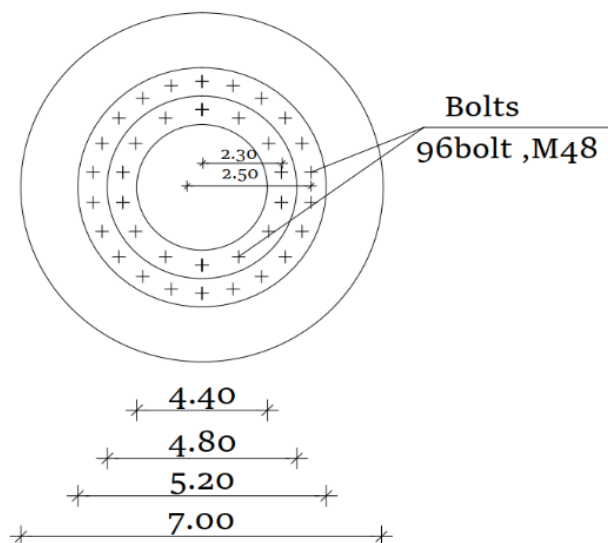


Figure 13 – Anchor Bolts Distribution

$$N_{M,Ed,bolt} = \frac{M}{n_{outer} * r_{outer}^2 + n_{inner} * r_{inner}^2} = \frac{236822}{48 * 2.6^2 + 48 * 2.4^2} = 394.1 \text{ KN}$$

$$N_{V,Ed,bolt} = \frac{T_{total}}{n} = \frac{56400}{96} = 588 \text{ KN}$$

$$T_{V,bolt} = \frac{\text{Total vertical load}}{\text{Total Number of bolts}} = \frac{31734}{96} = 330.6 \text{ KN}$$

$$N_{Total,Ed,bolt} = N_{M,Ed,bolt} + N_{V,Ed,bolt} = 394.1 + 330.6 = 724.7 \text{ KN}$$

Bolt Strength Verification

$$N_{Rd,bolt} = \frac{A_{bolt} * 0.9 * F_{yk}}{\gamma_{m2}} = \frac{1533 * 0.9 * 1000}{1.25} * 10^{-3} = 1103.76 \text{ KN}$$

$$T_{Total,bolt} = 724.7 \text{ KN} < N_{Rd,bolt} = 1103.76 \text{ KN} \rightarrow \text{safe}$$

Pretension Verification

$$N_{Ed,bolt(inner)} = 724.7 \text{ KN} = 72.47 \text{ ton}$$

$$N_{Ed,bolt(outer)} = 724.7 \text{ KN} = 72.47 \text{ ton}$$

N_{Ed,bolt}: is Tension Force applied on each bolt

$$F_b = 0.7 * F_{ub} * A_{bolt} = 0.7 * 1000 * 1533 = 1073100 \text{ N} = 107.31 \text{ ton}$$

N_{Ed,bolt}: is TPretension Force applied on each bolt

11. Check punching Shear

$$u_1 = \pi(D_{pedestal} + 4d) = \pi * (7 + 4 * 1.95) = 46.5 \text{ m}$$

Concrete punching resistance without shear reinforcement

$$V_{Rd,c} = C_{Rd,c} * K * (100\rho_l * f_{ck})^{\frac{1}{3}}$$

$$C_{Rd,c} = \frac{0.18}{\gamma_c} = \frac{0.18}{1.5} = 0.12$$

$$K = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{1950}} = 1.32 \text{ where } K \leq 2$$

Assume longitudinal reinforcement ratio $\rho_l \approx 0.5 \% = 0.005$

$$v_{Ed} < V_{Rd,c} \rightarrow \text{Safe against punching}$$

12. Seismic Analysis

The analysis employs the Equivalent Static Force Method (Pseudo-Static approach) in accordance with Eurocode 8 standards. This simplified method is appropriate for regular structures and provides conservative results for seismic design verification.

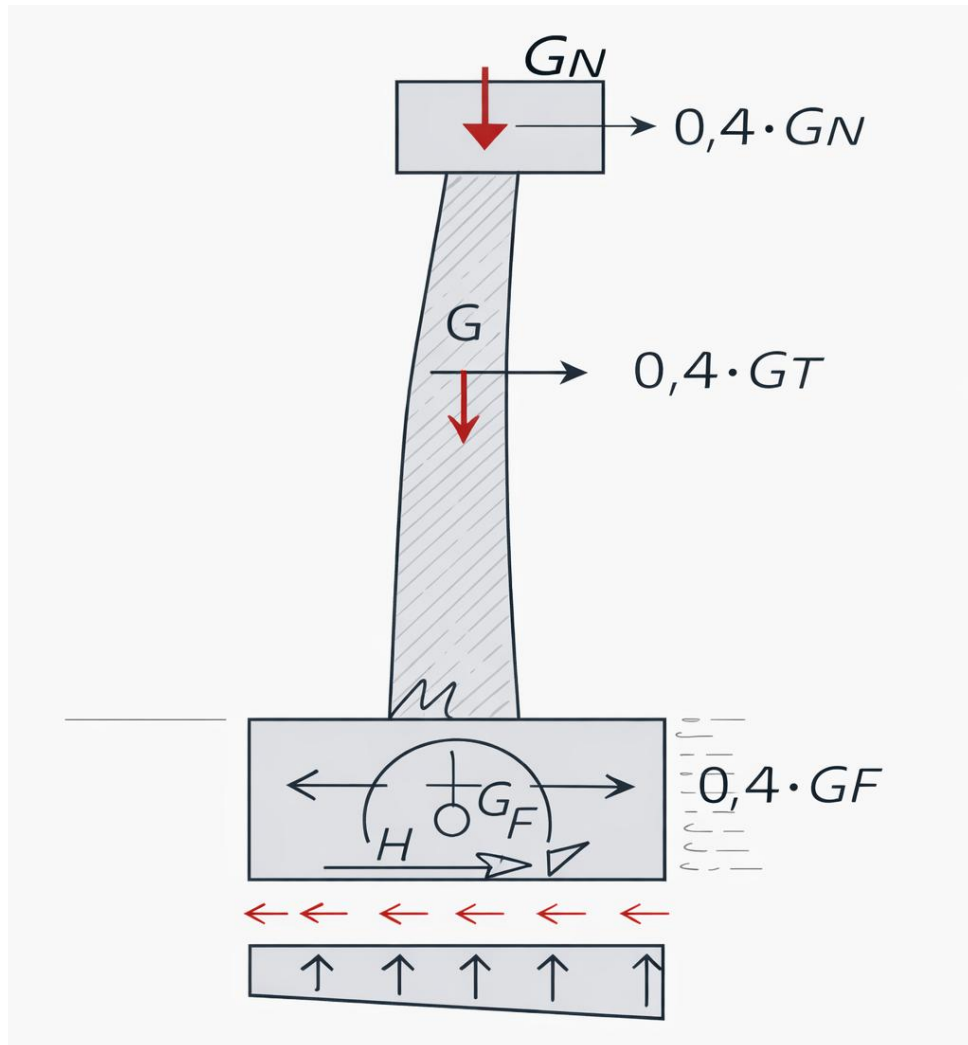


Figure 14 – Seismic Load

Seismic Coefficient: $a_g=0.4g$

This represents a high seismic zone classification, requiring rigorous stability verification across all failure modes

12.1. Seismic Forces & Moments

Applying the seismic coefficient (0.4) to the precise structural weights yields the following demand on the structure:

Component	Weight	Seismic Force	Lever Arm (H)
Nacelle	146.3 tons	58.5 tons	100.0 m
Tower	349.0 tons	139.6 tons	50.0 m
Foundation	2,856.0 tons	1,142.4 tons	1.0 m
TOTAL (Demand)	3,351.3 tons	1,340.5 tons	—

1,340	14K
Base Shear	Overturning Moment
Total horizontal force in tons	Total moment in ton.m

The seismic force calculation follows $V_i = 0.4 \times G$ for each component, with forces applied at respective centers of gravity to determine the total base shear and overturning moment acting on the foundation.

12.2. Global Stability Verification

Two critical stability checks ensure the foundation remains secure under seismic loading conditions. Both checks demonstrate adequate safety factors exceeding minimum code requirements.

Sliding Stability Check

This check verifies that horizontal seismic forces do not cause the foundation to slide across the soil interface.

Resisting Force (Friction + Cohesion): 2,372 tons

Driving Shear Force: 1,340 tons

Factor of Safety: $1.77 > 1.0 \rightarrow \text{SAFE}$

The resisting forces, generated through friction between the foundation base and soil plus soil cohesion, significantly exceed the applied horizontal seismic force.

Overturning Stability Check

This check ensures the foundation weight provides sufficient resistance against rotational failure.

Stabilizing Moment (Weight x Radius): 41,891 ton-m

Overturning Moment: $\sim 13,972$ ton-m

Factor of Safety: $2.30 > 1.0 \rightarrow$ SAFE

The stabilizing moment from the foundation's self-weight provides more than double the resistance needed to prevent overturning.

12.3. Soil-Structure Interaction Analysis

Bearing Capacity Verification

The bearing pressure check ensures soil beneath the foundation can support the combined static and seismic loads without failure.

Maximum Edge Pressure (q_{\max}): 356 kPa

Allowable Dynamic Bearing Capacity: > 400 kPa

The allowable capacity assumes compacted fill material with appropriate seismic increase factors per Eurocode 8 provisions.

Result: Safe against soil failure

The maximum bearing pressure remains below the soil's dynamic capacity, ensuring no bearing failure occurs during seismic loading.

Eccentricity & Gapping Analysis

Eccentricity measures how far the resultant force shifts from the foundation center, indicating potential edge uplift.

Calculated Eccentricity (e): 5.43 m

Critical Threshold: $R/4 = 3.12$ m

Foundation Radius (R): 12.5 m

Observation: Since $e > R/4$, partial gap opening occurs at the foundation edge under peak seismic loading.

Conclusion: Acceptable under ULS conditions

While edge gapping occurs, global stability is maintained with adequate safety factors.

This behaviour is acceptable for extreme seismic events at Ultimate Limit State.

13.3D Modelling of Wind turbine Foundation

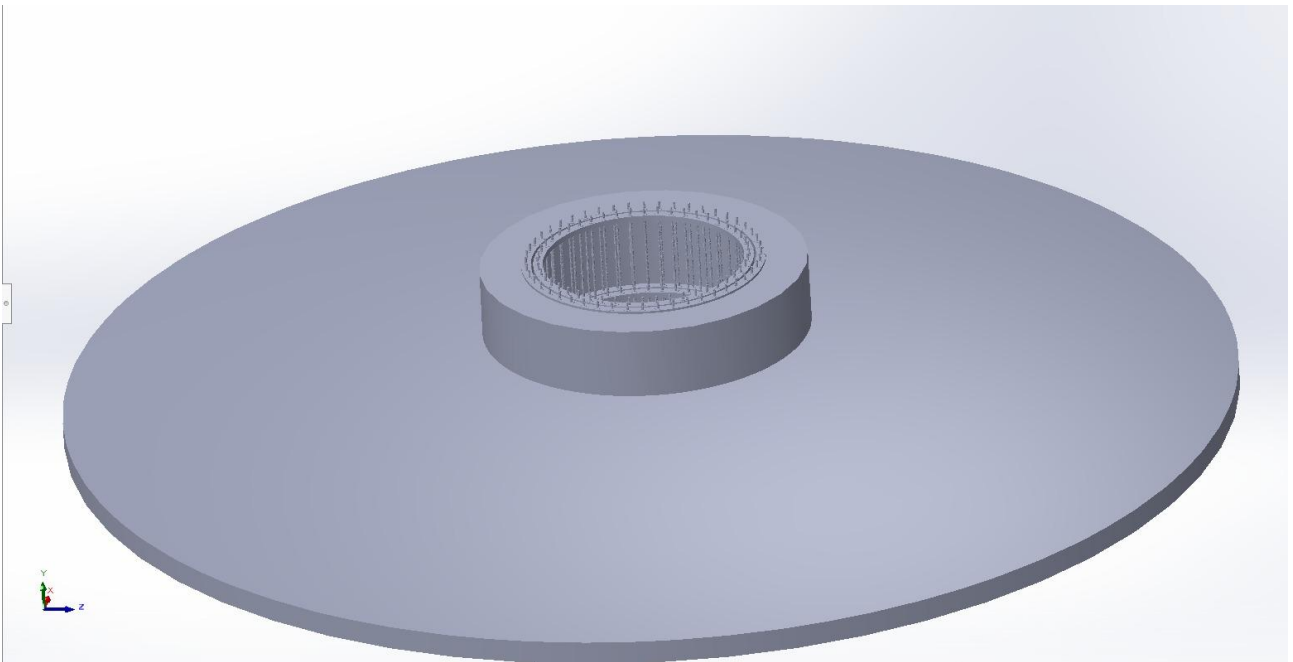


Figure 15 – 3D Model of wind turbine foundation

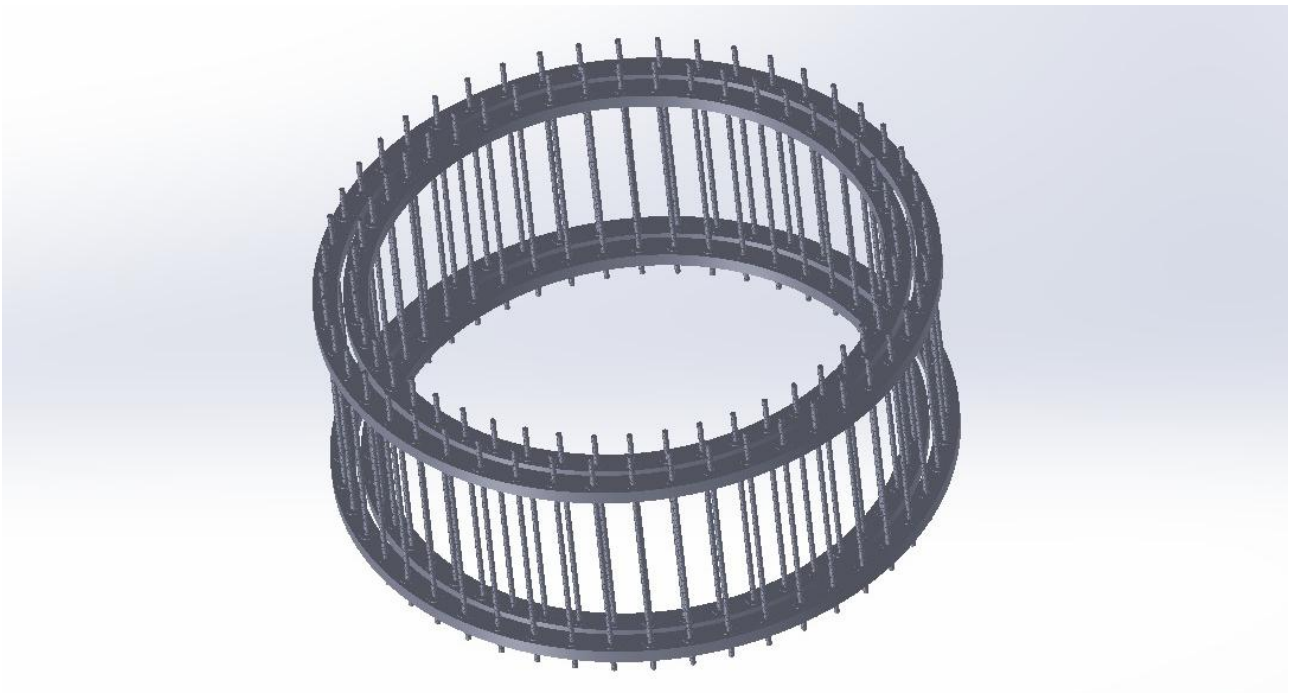


Figure 16 – 3D Model for Anchor Bolts

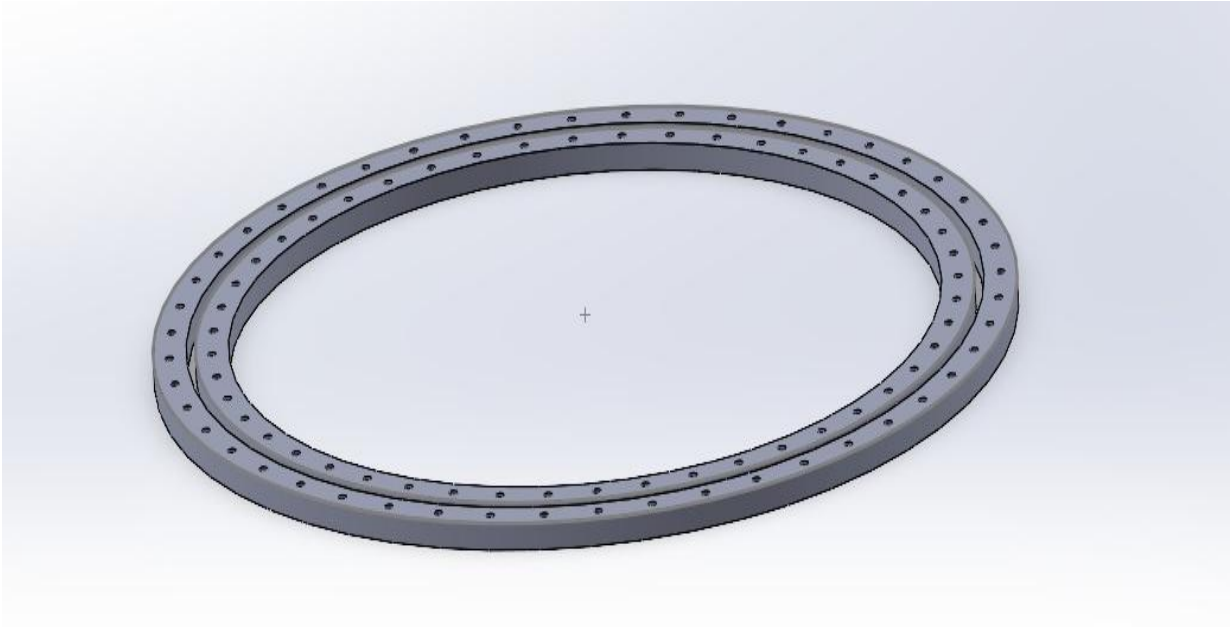


Figure 17 - 3D Model of the flange assembly illustrating the distribution of anchor bolt holes.

14.Estimated Cost of Wind Turbine Foundation

1- Costs Calculation

Item	Costs (EUR)
Concrete + Labor	138571
Reinforcement Steel Bars	29,393
Anchor bolt cage	9,092
Excavation	15393
Total Cost	192449

15. References

- 1- Characterization of Major Soil Orders in Syria, FAO AGRIS database, Food and Agriculture Organization of the United Nations. <https://agris.fao.org>
- 2- **Soils of the Southern Syria (2023)**
Soils of the Southern Syria – A big database for the future land management planning, Data in Brief, Elsevier. <https://doaj.org>
- 3- **FAO / ACSAD (n.d.)**

Representative Soil Profiles – Syrian Gypsiferous Soils, FAO & Arab Center for the Studies of Arid Zones and Drought <https://www.fao.org/4/t0323e/t0323e0a.htm>
- 4- **EN 1990:2002 + A1:2005**
Eurocode 0: Basis of Structural Design, European Committee for Standardization (CEN), Brussels.
- 5- **EN 1991-1-4:2005 + A1:2010**
Eurocode 1: Actions on Structures – Part 1-4: Wind Actions, CEN, Brussels.
- 6- **EN 1992-1-1:2004 + A1:2014**
Eurocode 2: Design of Concrete Structures – Part 1-1: General Rules and Rules for Buildings, CEN, Brussels.
- 7- **EN 1993-1-8:2005 + A1:2014**
Eurocode 3: Design of Steel Structures – Part 1-8: Design of Joints, CEN, Brussels.
- 8- **EN 1997-1:2004 + A1:2013**
Eurocode 7: Geotechnical Design – Part 1: General Rules, CEN, Brussels.
- 9- **EN 1997-2:2007**
Eurocode 7: Geotechnical Design – Part 2: Ground Investigation and Testing, CEN, Brussels.
- 10- **EN 1998-1:2004 + A1:2013**
Eurocode 8: Design of Structures for Earthquake Resistance – Part 1: General Rules, Seismic Actions and Rules for Buildings, CEN, Brussels.