Statische Berechnung

Customer
Green Wind Energy Ltd.
103 Cregagh Road
Belfast BT6 8PY
Irland

Project
Wind Energy Plant VESTAS V27 'Adrian'
hub height 30 m
Foundation

Site
near 32 Omerbane Road
Cloughmills
County Antrim

Projectnumber
216041
PRELIMINARY REMARKS

Subject to the following static analysis is the proof of a foundation for a wind energy plant type VESTAS V27 with a hub height of 30 m.

It is planned to install and operate a used windmill-power plant, which will be mounted on a new foundation at the site in Ireland.

The dimensioning of the foundation is based on the following documents:


Loads for the planned location are not given in [1]. Furthermore the engineer standards for calculating the wind loads changed since 1991.

A recalculation of the wind energy plant VESTAS V27 fails due to lack of relevant information on behalf of the manufacturer. The foundation load is therefore calculated as follows:

1. Comparison of the calculated wind loads in [1] with the on-site wind loads according to BS EN 1991-1-4. The comparison shows, that the on-site wind loads are slightly higher (~10%). The foundation loads given in [1] will therefore be multiplied by the factor $f_a = 1.10$.

2. At the request of the client, the foundation loads are then multiplied by the factor $f_2 = 1.20$ in order to cover the risks of dynamic effects on the foundation soil.

According to the ground investigation report [3], the foundation depth must be at least 2.50 m. The foundation area has to be drained to avoid an accumulation of water above foundation level.

According to [1] the dynamic modulus of elasticity of the foundation soil must be $E_s,_{\text{dyn}} \geq 75.000 \text{ kN/m}^2$ in order to avoid inappropriate dynamic effects on the wind energy plant. The dynamic modulus of elasticity has to be checked and confirmed on site by a geological survey.

Proofs of the windmill itself and its anchoring on the foundation are not subject to the following static analysis and have to be done by others.
COMPARISON OF WIND LOADS

Hub height

\[ z = 30.00 \text{ m} \]

SITE IRELAND:

Fig. NA.1
\[ v_{b,\text{map}} = 26.00 \text{ m/s} \]

Height above sea level
\[ A = 280.00 \text{ m} \]

NA.2.5
\[ c_{\text{at}} = 1 + 0.001 \times 280 \times (10/30)^{0.20} = 1.22 \]
\[ c_{\text{sr}} = c_{\text{season}} = c_{\text{prob}} = 1.00 \]

\[ v_b = 26.00 \times 1.00 \times 1.00 \times 1.00 \times 1.22 = 31.84 \text{ m/s} \]
\[ q_b = 0.613 \times 31.84 \times 10^{-3} = 0.21 \text{ kN/m}^2 \]
NA.2.17: distance to shoreline
\[ \geq 15.00 \text{ km} \]
Fig. NA.7
\[ c_d(z=30 \text{ m}) = 3.20 \]
NA.3a
\[ q_d = 3.20 \times 0.62 = 1.98 \text{ kN/m}^2 \]

STRUCTURAL ANALYSIS:
(acc. to structural analysis [1])

<table>
<thead>
<tr>
<th>location</th>
<th>Zone IV</th>
</tr>
</thead>
<tbody>
<tr>
<td>DIN 4133, Appendix A</td>
<td>h = 30.00 m &lt; 50</td>
</tr>
<tr>
<td>(a)</td>
<td>( q_0 = 1.70 \text{ kN/m}^2 )</td>
</tr>
<tr>
<td>(A.3)</td>
<td>( q = 0.75 \times (1 + h/100) \times q_0 = 1.66 \text{ kN/m}^2 )</td>
</tr>
</tbody>
</table>

addition elevation
\[ q = 0.15 \text{ kN/m}^2 \]

COMPARISON:
Wind loads according to BS EN 1991-1-4 are higher than the calculated values according to [1]. The windloads given in [1] will therefore be multiplied by
\[ f_1 = 1.98 / 1.81 = 1.10 \]
CALCULATION OF FOUNDATION LOADS

The foundation loads given in [1] will be increased by a safety margin (see preliminary remarks)

increase safety factor \( f_{\text{ges}} = f_1 \times f_2 = 1,10 \times 1,20 = 1,30 \)

Structural analysis [1], page 38 ff

Design foundation loads are calculated by multiplying the characteristic foundation loads [1] with the global safety factor \( \gamma_{\text{ben}} = 1,40 \).

LOADCASE 1: extreme operation mode, no gaping

- vertical loads
  \[ F_{z,k} = 252,00 \text{ kN} \]
- soil on top of foundation
  \[ h = 0,80 \text{ m} \]
- weight
  \[ \gamma = 18,00 \text{ kN/m}^2 \]

loads in direction of symmetry axis:
- horizontal
  \[ H_z = 1,30 \times 1,40 \times 72,10 = 131,25 \text{ kN} \]
- moment
  \[ M_z = 1,30 \times 1,40 \times 1957,25 = 3562,20 \text{ kNm} \]

loads across:
- horizontal
  \[ H_x = H_y = 131,25 / \sqrt{2} = 92,80 \text{ kN} \]
- moment
  \[ M_{xy} = M_{xy} = 3562,20 / \sqrt{2} = 2518,90 \text{ kNm} \]

LOADCASE 2: wind for Staudruckzone IV [1], gap up to balance point

- vertical loads
  \[ F_{z,k} = 252,00 \text{ kN} \]
- soil on top of foundation
  \[ h = 0,80 \text{ m} \]
- weight
  \[ \gamma = 18,00 \text{ kN/m}^2 \]

loads in direction of symmetry axis:
- horizontal
  \[ H_z = 1,30 \times 1,40 \times 164,00 = 298,50 \text{ kN} \]
- moment
  \[ M_z = 1,30 \times 1,40 \times 4273,90 = 7778,50 \text{ kNm} \]

loads across:
- horizontal
  \[ H_x = H_y = 298,50 / \sqrt{2} = 211,10 \text{ kN} \]
- moment
  \[ M_{xy} = M_{xy} = 7778,5 / \sqrt{2} = 5500,25 \text{ kNm} \]

LOADCASE 3: fatigue strength \( \gamma_{\text{ben}} = 1,00 \)

- vertical loads
  \[ F_{z,k} = 252,00 \text{ kN} \]
- soil on top of foundation
  \[ h = 0,80 \text{ m} \]
- weight
  \[ \gamma = 18,00 \text{ kN/m}^2 \]

loads in direction of symmetry axis:
- horizontal
  \[ H_z = 1,30 \times 31,40 = 40,80 \text{ kN} \]
- moment
  \[ M_z = 1,30 \times 1083,70 = 1408,80 \text{ kNm} \]

loads across:
- horizontal
  \[ H_x = H_y = 70,80 / \sqrt{2} = 28,90 \text{ kN} \]
- moment
  \[ M_{xy} = M_{xy} = 1408,80 / \sqrt{2} = 996,20 \text{ kNm} \]
Position: 01.01 Nachweis Lagesicherheit LF1+2

Fundament FD 02/2012/G (Friso R-2016-1/P6)

Scale 1 : 200

C25/30 XCY, XFY, WFC, UwF UGRE = 45mm

dimensions side longitudinals height

foundation bx = 9.00 m by = 9.00 m d = 1.20 m
pedestal cx = 2.47 m cy = 2.47 m h = 1.00 m

Lagesicherheit EQU für Einzelastfälle bei y = 4.50
η = 8435.2 kNm / 16446.7 kNm = 0.51

loads : loading case 1 RESULT-LC with axial eccentric

all load inputs of these loadcase are already y-times!

height of earth load = 0.80 m
specific weight = 18.00 kN/m³

total foundation with pedestal Gk = 3352.81 kN (for design) F = 1.35
moment I. Ord. Mx = 3622.00 kNm for press. (DIN1054) and gapping
moment II. Ord. MxII = 35820 kNm for design
HForce I. Ord. Hy1 = 131.25 kN for press. (DIN1054) and gapping
HForce II. Ord. HyII = 131.25 kN for design

positive moments Mx und My generate positive pressure stresses in-
in the corner right above of sole joint.

vertical loads: column N = 252.00 kN ax = 0.00 m ay = 0.00 m
y-times column load by reductionfactor = 1.00 divided.
y-times further loads by reductionfactor = 1.40 divided.
sole pressure y-fach und klaffende Fuge 1.0-fach.
total load tot N = 3604.91 kN ex = 0.00 m ey = 0.76 m

SOIL PRESSURE : load case No.: 1 Perm. Sigma = 210 kN/m²
y-times column load by reductionfactor = 1.00 divided.
y-times further loads by reductionfactor
sole pressure γ-fach und klaffende Fuge 1,0-fach.
without gaping
(by II.Ord.)

Sohldruck nach
DIN EN 1997-1 6.5.2.4 σ =
(Allgemein anerkanntes Verfahren : DIN 1054:2010)
Sohldruck nach
DIN EN 1997-1 6.5.2.4 σ a =
(Allgemein anerkanntes Verfahren : DIN 1054:2010)
for γ-times loads for calculation of designmoments
edge pressure
max p = 72.32 kN/m² by I.Ord.
min p = 16.61 kN/m² by II.Ord f. desgn.
below column centre
p = 48.31 kN/m² by II.Ord f. desgn.

Gleitsicherheit nach DIN EN 1997-1 (Φ = 30 deg)
Rd =Rtk/1.1 =189209 kN > Td = 131.25 kN

design moments for y-times loads

design moment
MxEd = 2156.83 kNm (around x-axis)
design moment
MyEd = -1509.17 kNm (around y-axis)
design moment
MxyEd = 221.35 kNm (around y-axis)

loads : loading case 2 RESULT- LC with biaxial eccentric

all load inputs of these loadcase are already y-times!

height of earth load=
0.80 m
specific weight = 18.00 kN/m³

total foundation
with pedestal Gb = 353291 kN (for designγF = 1.35)
further loads are already y-times put in.

moment 1. Ord Mx 1 = 2518.90 kNm for press. (DIN1054) and gapping
moment 1. Ord My 1 = 2518.90 kNm for press. (DIN1054) and gapping
moment 1. Ord Mxy 1 = 2518.90 kNm for design
moment II. Ord MxI 2 = 92.80 kN for press. (DIN1054) and gapping
moment II. Ord MyI 2 = 92.80 kN for press. (DIN1054) and gapping
moment II. Ord MxyI 2 = 92.80 kN for design
HForce 1. Ord Hx 1 = 92.80 kN for design
HForce 1. Ord Hy 1 = 92.80 kN for press. (DIN1054) and gapping
HForce 1. Ord Hxy 1 = 92.80 kN for design

positive moments Mx und My generate positive pressure stresses in
in the corner right above of sole joint.

vertical loads : e c c e n t r i c i t i e s

column N = 252.00 kN ex = 0.00 m ey = 0.00 m

γ-times column load by reductionfactor : 1.00 divided.
γ-times further loads by reductionfactor : 1.40 divided.

SOIL PRESSURE : load case No.12
Perm. Sigma = 210 kN/m²

γ-times column load by reductionfactor : 1.00 divided.
γ-times further loads by reductionfactor : 1.40 divided.
sole pressure γ-fach und klaffende Fuge 1,0-fach.
without gaping
(by II.Ord.)

Sohldruck nach
DIN EN 1997-1 6.5.2.4 σ a =
(Allgemein anerkanntes Verfahren : DIN 1054:2010)
pressure at the edges of the pressed area
by I.Ord.

for γ-times loads :

point x (m) y (m) pressure (kN/m²)
1 4.500 4.500 104.29
2 4.500 -4.500 59.47
3 -4.500 -4.500 14.64
4 -4.500 4.500 59.47

Sohldruck nach
DIN EN 1997-1 6.5.2.4 σ a =
(Allgemein anerkanntes Verfahren : DIN 1054:2010)
pressure at the edges of the pressed area
by II.Ord.
for γ-times loads:
point x (m) y (m) pressure (kN/m²)
1 4.500 4.500 104.29
2 4.500 4.500 59.47
3 -4.500 -4.500 14.64
4 -4.500 -4.500 59.47

for γ-times loads for calculation of design moments:
max p = 104.29 kN/m² by II.Ord f. dsgn.
min p = 14.64 kN/m² by II.Ord f. dsgn.
below column centre p = 59.47 kN/m² by II.Ord f. dsgn.

Gleitsicherheit nach DIN EN 1997-1; (Δϕ = 30 deg)
Rtd = Rtk/1.1 = 1892.09 kN > Td = 131.24 kN

design moments for γ-times loads:
design moment MxEd = 1633.06 kNm (around x-axis)
design moment MxEd = -491.82 kNm (around x-axis) top
design moment MyEd = 1633.06 kNm (around y-axis)
design moment MyEd = -491.82 kNm (around y-axis) top

loads: loading case 3 RESULT- LC with axial eccentric

all load inputs of these load cases are already γ-times:

height of earth load = 0.80 m
specific weight = 18.00 kN/m³
total foundation
with pedestal Gₚ = 3352.91 kN (for design γF = 1.35)
further loads are already γ-times put in.
moment 1. Ord M₀ = 7778.50 kNm for press. (DIN1054) and gapping
moment II. Ord M₀II = 7778.50 kNm for design
HForce 1. Ord H₀ = 298.50 kN for press. (DIN1054) and gapping
HForce II. Ord H₀II = 298.50 kN for design
positive moments Mx und My generate positive pressure stresses in
the corner right above of sole joint.

vertical loads:
column N = 252.00 kN
ax = 0.00 m
ay = 0.00 m
eccentricities

γ-times column load by reduction factor:
y = 1.00 divided.

γ-times further loads by reduction factor:
y = 1.40 divided.

γ-times load tot N = 3604.91 kN
ex = 0.00 m
ey = 1.67 m

total load

SOIL PRESSURE:
load case No.: 3 Perm. Sigma = 210 kN/m²

γ-times column load by reduction factor:
y = 1.00 divided.

γ-times further loads by reduction factor:
y = 1.40 divided.

γ-times load tot N = 3604.91 kN
ex = 0.00 m
ey = 1.67 m

total load

Sohldruck nach DIN EN 1997-1-6 5.2.4 α = 97.35 kN/m² by I.Ord.
(Allgemein anerkanntes Verfahren: DIN 1054:2010)
length of gapping l = 0.51 m by I.Ord.

Sohldruck nach DIN EN 1997-1-6 5.2.4 α = 97.35 kN/m² by II.Ord.
(Allgemein anerkanntes Verfahren: DIN 1054:2010)
length of gapping l = 0.75 m by II.Ord.

for γ-times loads for calculation of design moments:
max p = 129.80 kN/m² by II.Ord f. dsgn.
min p = 0.00 kN/m² by II.Ord f. dsgn.
below column centre p = 58.97 kN/m² by II.Ord f. dsgn.

Gleitsicherheit nach DIN EN 1997-1; (Δϕ = 30 deg)
Rtd = Rtk/1.1 = 1892.09 kN > Td = 298.50 kN
design moments for loads:
design moment \( M_{xEd} = 4532.72 \text{ kNm} \) (around \( x \)-axis)
design moment \( M_{xEd} = -3761.10 \text{ kNm} \) (around \( x \)-axis)
design moment \( M_{yEd} = 274.93 \text{ kNm} \) (around \( y \)-axis)

loads: loading case 4 RESULT - LC with biaxial eccentric

all load inputs of these loadcase are already adjustable

- height of earth load: 0.80 m
- specific weight: 18.00 kN/m³
- total foundation with pedestal \( G_k = 3352.91 \text{ kN} \) (for design \( F = 1.35 \))
- further loads are already y-times put in.

moment 1. Ord. \( M_{xH} = 5500.25 \text{ kNm} \) for press. (DIN 1054) and gaping
moment 1. Ord. \( M_{yH} = 5500.25 \text{ kNm} \) for design
moment 2. Ord. \( M_{xH} = 5500.25 \text{ kNm} \) for press. (DIN 1054) and gaping
moment 2. Ord. \( M_{yH} = 5500.25 \text{ kNm} \) for design
HF 1. Ord. \( H_{xH} = 211.10 \text{ kN} \) for press. (DIN 1054) and gaping
HF 1. Ord. \( H_{yH} = 211.10 \text{ kN} \) for design
HF 2. Ord. \( H_{xH} = 211.10 \text{ kN} \) for press. (DIN 1054) and gaping
HF 2. Ord. \( H_{yH} = 211.10 \text{ kN} \) for design

positive moments \( M_x \) and \( M_y \) generate positive pressure stresses in the corner right above of sole joint.

vertical loads:
- column \( N = 25200 \text{ kN} \) \( ax = 0.00 \text{ m} \) \( ay = 0.00 \text{ m} \)
- y-times column load by reductionfactor \( 1.00 \) divided.
- y-times further load by reductionfactor \( 1.40 \) divided.
- sole pressure \( y \)-fach und klaffende Fuge 1.0-fach.
- total load \( N = 360491 \text{ kN} \) \( ex = 1.18 \text{ m} \) \( ey = 1.18 \text{ m} \)

SOIL PRESSURE: load case No.: 4 Perm. Sigma = 210 kN/m²

y-times column load by reductionfactor \( 1.00 \) divided.

Sohldruck nach DIN EN 1997-1 6.5.2.4 \( \sigma = 113.19 \text{ kN/m²} \) by 1.Ord.
(Allgemein anerkanntes Verfahren : DIN 1054:2010)
pressure at the edges of the pressed area by 1.Ord.

for y-times loads:
- point x (m) y (m) pressure (kN/m²)
  1  4.500  4.500  163.64
  2  4.500 -4.500  57.82
  3 -0.788 -4.500  0.00
  4 -4.500 -0.788  0.00
  5 -4.500  4.500  57.82

Sohldruck nach DIN EN 1997-1 6.5.2.4 \( \sigma = 113.19 \text{ kN/m²} \) by 1.Ord.
(Allgemein anerkanntes Verfahren : DIN 1054:2010)
pressure at the edges of the pressed area by 2.Ord.

for y-times loads:
- point x (m) y (m) pressure (kN/m²)
  1  4.500  4.500  163.64
  2  4.500 -4.500  57.82
  3 -0.418 -4.500  0.00
  4 -4.500 -0.418  0.00
  5 -4.500  4.500  57.82

for y-times loads for calculation of design moments:
- edge pressure \( p = 163.64 \text{ kN/m²} \) by 2.Ord. f. dsgn.

min \( p = 0.00 \text{ kN/m²} \) by 2.Ord. f. dsgn.
below column centre \( p = 57.82 \text{ kN/m²} \) by 2.Ord. f. dsgn.
Gleitsicherheit nach DIN EN 1997-1 : (\(\Phi = 30\) deg)

Rd = Rk/1.1 = 1892.09 kN > Td = 298.54 kN

design moments for y-times loads
- design moment: \(M_{Ed} = 3331.17\) kNm (around x-axis)
- design moment: \(M_{Ed} = 2481.84\) kNm (around y-axis)

loads: loading case 5 RESULT-LC with axial eccentric

all load inputs of these loadcase are already y-times!

- height of earth load = 0.80 m
- specific weight = 18.00 kN/m³
- total foundation with pedestal \(G_k = 3661.07\) kN (for design \(F = 1.35\))
- further loads are already y-times put in.
  - moment I. Ord \(M_{I} = 1408.80\) kNm for press. (DIN1054) and gapping
  - moment II. Ord \(M_{II} = 1408.80\) kNm for design
  - HForce I. Ord \(H_{I} = 40.80\) kN for press. (DIN1054) and gapping
  - HForce II. Ord \(H_{II} = 40.80\) kN for design

positive moments \(M_{x}\) und \(M_{y}\) generate positive pressure stresses in the corner right above of sole joint.

vertical loads:
- column \(N = 252.00\) kN
- \(\text{eccentricities}\)
  - \(m_x = 0.00\) m
  - \(m_y = 0.00\) m

y-times column load by reduction factor: 1.00 divided.
y-times further loads by reduction factor: 1.00 divided.
sole pressure y-fach und kllaffende Fuge 1,0-fach.
total load \(N = 3913.07\) kN

SOIL PRESSURE: load case No.: 5
Perm. Sigma = 210 kN/m²

y-times column load by reduction factor: 1.00 divided.
y-times further loads by reduction factor: 1.00 divided.
sole pressure y-fach und kllaffende Fuge 1,0-fach.
without gapping (by II.Ord.)

Sohldruck nach DIN EN 1997-1: 6.5.2.4 \(\sigma = 63.89\) kN/m² by I.Ord.
(Allgemein anerkanntes Verfahren : DIN 1054:2010)
Sohldruck nach DIN EN 1997-1: 6.5.2.4 \(\sigma = 63.89\) kN/m² by II.Ord.
(Allgemein anerkanntes Verfahren : DIN 1054:2010)
for y-times loads for calculation of designmoments
edge pressure: \(p_{max} = 71.80\) kN/m² by II.Ord f. dsgn.
\(p_{min} = 47.13\) kN/m² by II.Ord f. dsgn.
below column centre: \(p = 58.47\) kN/m² by II.Ord f. dsgn.

Gleitsicherheit nach DIN EN 1997-1 : (\(\Phi = 30\) deg)

Rd = Rk/1.1 = 2033.83 kN > Td = 4078.0 kN

design moments for y-times loads
- design moment: \(M_{Ed} = 1024.21\) kNm (around x-axis)
- design moment: \(M_{Ed} = 322.97\) kNm (around y-axis)
### Loads: Loading Case 6 Result - LC with Biaxial Eccentricity

All load inputs of these load case are already... y-times!

- **Height of Earth Load**: 0.00 m
- **Specific Weight**: 18.00 kN/m³
- **Total Foundation**: 3661.07 kN (for design y = 1.35)
- **Moment I. Ord. Mx**: 996.20 kNm for press. (DIN1054) and gaping
- **Moment II. Ord. My**: 996.20 kNm for design
- **Moment I. Ord. My**: 996.20 kNm for press. (DIN1054) and gaping
- **H-Force I. Ord. Hx**: 28.90 kN for press. (DIN1054) and gaping
- **H-Force II. Ord. Hy**: 28.90 kN for design
- **H-Force I. Ord. Hy**: 28.90 kN for press. (DIN1054) and gaping
- **H-Force II. Ord. Hy**: 28.90 kN for design

Positive moments Mx and My generate positive pressure stresses in the corner right above of sole joint.

- **Vertical Loads**:
  - Column: N = 252.00 kN ax = 0.00 m ay = 0.00 m

  - y-times further loads by reduction factor: 1.00 divided.
  - y-times further loads by reduction factor (sole pressure y-fach und klaffende Fuge 1.0-fach): 1.00 divided.

  - Total load: N = 3913.07 kN ex = 0.27 m ey = 0.27 m

### Soil Pressure:
- **Load Case No.: 6**
- **Perim. Sigma = 210 kN/m²**

- **y-times Column Load by Reduction Factor**: 1.00 divided.
- **y-times Further Loads by Reduction Factor**: 1.00 divided.

**Soil Druck nach DIN EN 1997-1.6.5.2.4**: 65.74 kN/m² by I.Ord.

**Pressure at the Edges of the Pressed Area** by I.Ord.

For y-times loads:
- **Point** x (m) y (m) Pressure (kN/m²)
  1. 4.500 4.500 76.91
  2. 4.500 4.500 59.47
  3. 4.500 4.500 42.02
  4. 4.500 4.500 59.47

**Soil Druck nach DIN EN 1997-1.6.5.2.4**: 65.74 kN/m² by II.Ord.

**Pressure at the Edges of the Pressed Area** by II.Ord.

For y-times loads:
- **Point** x (m) y (m) Pressure (kN/m²)
  1. 4.500 4.500 76.91
  2. 4.500 4.500 59.47
  3. 4.500 4.500 42.02
  4. 4.500 4.500 59.47

For y-times loads for calculation of design moments:
- **Edge pressure max p = 76.91 kN/m² by II.Ord f. dsgn.**
- **p = 42.02 kN/m² by II.Ord f. dsgn.**
- **Below Column Centre p = 59.47 kN/m² by II.Ord f. dsgn.**

**Geleitungsdichte nach DIN EN 1997-1.6.5.2.4**: 30 deg

Rtd = Rtk / 1.1 = 2053.83 kN > Td = 40.87 kN

Design Moments for y-times loads:
- **Design Moment MxEd = 803.50 kNm (around x-axis)**
- **Design Moment MxEd = -112.26 kNm (around x-axis)**
- **Design Moment MyEd = 803.50 kNm (around y-axis)**
- **Design Moment MyEd = -112.26 kNm (around y-axis)**
loads: loading case 7 RESULT: LC with central loading

all load inputs of these loadcase are already y-times!

height of earth load = 0.80 m
specific weight = 18.00 kN/m³

total foundation with pedestal Gk = 3661.07 kN (for design F = 1.35)
further loads are already y-times put in.

vertical loads:

<table>
<thead>
<tr>
<th>column</th>
<th>N</th>
<th>kN</th>
<th>ex</th>
<th>ay</th>
<th>m</th>
<th>m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>252.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

y-times column load by reductionfactor: 1.00 divided.
y-times further loads by reductionfactor: 1.00 divided.
ssole pressure y-fach und klaffende Fuge 1.0-fach.
total load: tot N = 3913.07 kN ex = 0.00 m ey = 0.00 m

SOIL PRESSURE: load case No.: 7, Perm. Sigma = 210 kN/m²

y-times column load by reductionfactor: 1.00 divided.
y-times further loads by reductionfactor: 1.00 divided.
ssole pressure y-fach und klaffende Fuge 1.0-fach.
without gaping (by II.Ord.)

centrical sole pressure: 59.47 kN/m² (nach DIN EN 1997-1 6.5.2.4 (II.Ord)
(Allgemein anerkanntes Verfahren: DIN 1054:2010)
centrical sole pressure: 59.47 kN/m² Söhlendruck nach DIN EN 1997-1 6.5.2.4 (II.Ord)
(Allgemein anerkanntes Verfahren: DIN 1054:2010)

design moments for y-times loads:
design moment: MxEd = 373.77 kNm (around x-axis)
design moment: MyEd = 373.77 kNm (around y-axis)

design: C 25/30 B 550(A) according EN 1992-1-1

without min. reinforcement according EN 1992-1-1 point 9.2.1.1 (1)!

<table>
<thead>
<tr>
<th>lc</th>
<th>(rd x) MxEd</th>
<th>kNm</th>
<th>req. Ac</th>
<th>cm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2156.83</td>
<td></td>
<td>39.09</td>
<td></td>
</tr>
<tr>
<td></td>
<td>top Mx = 1509.17 kNm</td>
<td>rec.Ac = 27.82 cm²</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>(rd y) MyEd</td>
<td>231.36</td>
<td>7.82</td>
<td>cm²</td>
</tr>
<tr>
<td></td>
<td>top My = 1941.2 kNm</td>
<td>rec.Ac = 83.92 cm²</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>1633.06</td>
<td></td>
<td>28.96</td>
<td>cm²</td>
</tr>
<tr>
<td></td>
<td>top My = 1941.2 kNm</td>
<td>rec.Ac = 83.92 cm²</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>(rd y) MyEd</td>
<td>1633.06</td>
<td>29.32</td>
<td>cm²</td>
</tr>
<tr>
<td>3</td>
<td>4532.72</td>
<td></td>
<td>83.92</td>
<td>cm²</td>
</tr>
<tr>
<td></td>
<td>top Mx = 3761.10 kNm</td>
<td>rec.Ac = 67.11 cm²</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>(rd y) MyEd</td>
<td>274.95</td>
<td>16.78</td>
<td>cm²</td>
</tr>
<tr>
<td>4</td>
<td>3331.17</td>
<td></td>
<td>60.18</td>
<td>cm²</td>
</tr>
<tr>
<td></td>
<td>top Mx = 2481.84 kNm</td>
<td>rec.Ac = 44.13 cm²</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>(rd y) MyEd</td>
<td>3331.17</td>
<td>60.72</td>
<td>cm²</td>
</tr>
<tr>
<td>5</td>
<td>1024.21</td>
<td></td>
<td>18.12</td>
<td>cm²</td>
</tr>
<tr>
<td></td>
<td>top Mx = 3328.87 kNm</td>
<td>rec.Ac = 4.89 cm²</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>803.50</td>
<td></td>
<td>4.89</td>
<td>cm²</td>
</tr>
<tr>
<td></td>
<td>top Mx = 312.26 kNm</td>
<td>rec.Ac = 3.06 cm²</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>373.77</td>
<td></td>
<td>6.59</td>
<td>cm²</td>
</tr>
<tr>
<td></td>
<td>(rd y) MyEd</td>
<td>803.50</td>
<td>14.33</td>
<td>cm²</td>
</tr>
<tr>
<td>8</td>
<td>373.77</td>
<td></td>
<td>6.59</td>
<td>cm²</td>
</tr>
<tr>
<td></td>
<td>(rd y) MyEd</td>
<td>373.77</td>
<td>6.65</td>
<td>cm²</td>
</tr>
</tbody>
</table>
bending reinforcement: C 25/30, B 550/(A) according to EN 1992-1-1.

Distribution of the reinforcement according to DAFStb table 2.10.

<table>
<thead>
<tr>
<th>y-direction: static height</th>
<th>$h_y = 1.14$ m chosen</th>
</tr>
</thead>
<tbody>
<tr>
<td>top reinforcement, tot $A_s = 67.11$ cm$^2$</td>
<td></td>
</tr>
<tr>
<td>distribution $b/8$</td>
<td>$b/8$ $b/8$ $b/8$</td>
</tr>
<tr>
<td>$c(m^2)$</td>
<td>7.95</td>
</tr>
<tr>
<td>$c(m^2/m)$</td>
<td>6.71</td>
</tr>
<tr>
<td>x-direction: static height</td>
<td>$h_x = 1.13$ m chosen</td>
</tr>
<tr>
<td>top reinforcement, tot $A_s = 44.53$ cm$^2$</td>
<td></td>
</tr>
<tr>
<td>distribution $b/8$</td>
<td>$b/8$ $b/8$ $b/8$</td>
</tr>
<tr>
<td>$c(m^2)$</td>
<td>5.47</td>
</tr>
<tr>
<td>$c(m^2/m)$</td>
<td>4.86</td>
</tr>
</tbody>
</table>

By user selected reinforcement:

- Y-direction reinforcement:
  - Bottom $= 114.57$ cm$^2$
  - Distribution, sel. reinf. in y-direction $= 14.32$ $b/8$ $b/8$ $b/8$ $b/8$

- X-direction reinforcement:
  - Bottom $= 114.57$ cm$^2$
  - Distribution, sel. reinf. in x-direction $= 14.32$ $b/8$ $b/8$ $b/8$ $b/8$

Proof of punching considering the user selected reinforcement:

- Limit state the bearing capacity for punching shear according to EN 1992-1-1

- $k_r = 1.90 * d_m = 2.16$ m
- $u_{cr}$ = 23.43 m
- $A_{cr} = 42.02$ m$^2$
- Existing average Rho = 0.11 %
- Load increasing factor = 1.00
- Red VEd (without Beta) = 335.09 kN
- Beta = 1.15
- $Vd$ (Beta considered) = 0.014 N/mm$^2$
- $Vd_c$ = 0.253 N/mm$^2$ $Vd$

Not additional punching shear reinforcement necessary.

Connection reinforcement according to EN 1992-1-1 req $A_s = 139.58$ cm$^2$, 12 o 40.
PROOF OF FATIGUE STRENGTH
(acc. to EN 1992-1-1)

CONCRETE STEEL
As-values and loadcases: ref. printout

\[
\begin{align*}
\text{LC 5} & \quad A_{\text{as,req}} = 18,12 \text{ cm}^2 \\
\text{LC 7} & \quad A_{\text{as,req}} = 6,65 \text{ cm}^2 \\
A_{\text{as,prov}} &= 31,16+43,98+31,16 = 106,30 \text{ cm}^2 \\
\text{B500S} & \quad f_{\text{yk}} = 50,00 \text{ kN/cm}^2 \\
\gamma_{\text{M}} &= 1,15 \\
f_{\text{rd}} &= 50,00 / 1,15 = 43,50 \text{ kN/cm}^2 \\
\sigma_{\text{L5}} &= 18,12/106,30 \times 43,50 = 7,42 \text{ kN/cm}^2 \\
\sigma_{\text{L5}} &= 6,65/106,30 \times 43,50 = 2,72 \text{ kN/cm}^2 \\
\Delta \sigma &= 7,42 - 2,72 = 4,70 \text{ kN/cm}^2
\end{align*}
\]

\(< \Delta \sigma = 7,00 \text{ kN/cm}^2 \) (acc. EN 1992-1-1, 6.8.6 (1)) √

CONCRETE
Concrete C25/30

\[
\begin{align*}
f_{\text{ck}} &= 25,00 \text{ MN/m}^2 \\
\gamma_{\text{M}} &= 1,50 \\
f_{\text{cd}} &= 0,85 \times 25,00/1,50 = 14,17 \text{ MN/m}^2 \\
\text{EN 1992-1-1 (6.8.7)} & \quad k_1 = 1,00 \\
\text{first loading after} & \quad t_0 = 28 \text{ days} \\
\beta_{\text{cd}}(28d) &= 1,00 \\
(6.8.7 (6.76)) & \quad f_{\text{cd,fat}} = 12,75 \text{ MN/m}^2 \\
b &= 9,00 \text{ m} \\
h &= 1,20-0,05-0,02 = 1,13 \text{ m} \\
d_1 &= 0,07 \text{ m} \\
A_{\text{x1}} &= A_{\text{x2}} = 106,30 \text{ cm}^2 \\
\text{compression zone (acc. E. Grasser)} & \quad x = 14,54 \text{ cm} \\
\text{LC 5} & \quad \sigma_{\text{c,max}} = 803,50/4532,72 \times 14,17 = 2,51 \text{ MN/m}^2 \\
\text{LC 7} & \quad \sigma_{\text{c,min}} = 373,77/4532,72 \times 14,17 = 1,17 \text{ MN/m}^2 \\
\sigma_{\text{c,max}} / f_{\text{cd,fat}} &= 2,51 / 12,75 = 0,20 \\
\sigma_{\text{c,min}} / f_{\text{cd,fat}} &= 1,17 / 12,75 = 0,09
\end{align*}
\]

\[\sigma_{\text{c,max}} / f_{\text{cd,fat}} = 0,20 \leq \text{ min } \{ 0,50 + 0,45 \times 0,09 = 0,54 ; 0,90 \} \] √
Aufgestellt

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www.ift-statik.de  konstruktion@ift-statik.de

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Unterschrift

Für die Prüfung

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Der Bauherr

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