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iFT STATIK

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 he 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:

[1] proofed structural analysis Windkraftanlage VESTAS V27/ 225 dated 04.09.1991 by Ingenieurbüro Otte und Bergmann, Flensburg

[2] NA to BS EN 1991-1-4: 2005 + A1:2010 (Windloads)

[3] ‚Ground Investigation Report' job-ref.: 15-215 dated January 2015

[4] General Specification VESTAS V27-225 Windturbine, Item-no.: 941129

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 therefor calculated as follows:

1. Comparison of he 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_1=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 $E_{s,dyn} \geq$ 75.000 kN/m² 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.

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COMPARISON OF WIND LOADS

Hub height $z = 30{,}00$ m

SITE IRELAND:

(acc. to BS EN 1991-1-4: 2005 + A1: 2010)

Fig. NA.1	$v_{b,map}$	=	26,00 m/s
Height above sea level	A	=	280,00 m
NA.2.5	$c_{alt} = 1+0{,}001x280x(10/30)^{0,20}$	=	1,22
	$c_{dir} = c_{season} = c_{prob}$	=	1,00
	$v_b = 26{,}00x1{,}00x1{,}00x1{,}00x1{,}22$	=	31,84 m/s
	$q_b = 0{,}613x31{,}84^2x10^{-3}$	=	0,62 kN/m²
NA.2.17: distance to shoreline		≥	15,00 km
Fig. NA.7	$c_e(z=30\text{ m})$	=	3,20
NA.3a	$\mathbf{q_p} = 3{,}20 \times 0{,}62$	=	**1,98 kN/m²**

STRUCTURAL ANALYSIS:

(acc. to structural analysis [1])

location Zone IV			
DIN 4133, Appendix A	h	=	30,00 m < 50
(a)	q_0	=	1,70 kN/m²
(A.3)	$q = 0{,}75x(1+h/100)xq0$	=	1,66 kN/m²
addition elevation		=	0,15 kN/m²
	q	=	**1,81 kN/m²**

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

$\mathbf{f_1} = 1{,}98/\ 1{,}81 = \mathbf{1{,}10}$

CALCULATION OF FOUNDATION LOADS

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

increase safety factor $f_{ges} = f_1 \times f_2 = 1{,}10 \times 1{,}20 =$ **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 $\gamma_{bem} = 1{,}40$.

LOADCASE 1: extreme operation mode, no gaping

vertical	$F_{z,k}$	=	252,00 kN
soil on top of foundation	h	=	0,80 m
weight	γ	=	18,00 kN/m²

loads in direction of symmetry axis:

horizontal	$H_d = 1{,}30 \times 1{,}40 \times 72{,}10$	=	131,25 kN
moment	$M_d = 1{,}30 \times 1{,}40 \times 1957{,}25$	=	3562,20 kNm

loads across:

horizontal	$H_{xk} = H_{yk} = 131{,}25 / \sqrt{2}$	=	92,80 kN
moment	$M_{yk} = M_{xk} = 3562{,}20 / \sqrt{2}$	=	2518,90 kNm

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

vertical	$F_{z,k}$	=	252,00 kN
soil on top of foundation	h	=	0,80 m
	γ	=	18,00 kN/m²

loads in direction of symmetry axis:

$H_k = 1{,}30 \times 1{,}40 \times 164{,}00$	=	298,50 kN
$M_k = 1{,}30 \times 1{,}40 \times 4273{,}90$	=	7778,50 kNm

loads across:

$H_{xk} = H_{yk} = 298{,}50 / \sqrt{2}$	=	211,10 kN
$M_{yk} = M_{xk} = 7778{,}5 / \sqrt{2}$	=	5500,25 kNm

LOADCASE 3: fatigue strength ($\gamma_{bem} = 1{,}00$)

vertical	$F_{z,k}$	=	252,00 kN
soil on top of foundation	h	=	0,80 m
	γ	=	18,00 kN/m²

loads in direction of symmetry axis:

$H_k = 1{,}30 \times 31{,}40$	=	40,80 kN
$M_k = 1{,}30 \times 1083{,}70$	=	1408,80 kNm

loads across:

$H_{xk} = H_{yk} = 70{,}80 / \sqrt{2}$	=	28,90 kN
$M_{yk} = M_{xk} = 1408{,}80 / \sqrt{2}$	=	996,20 kNm

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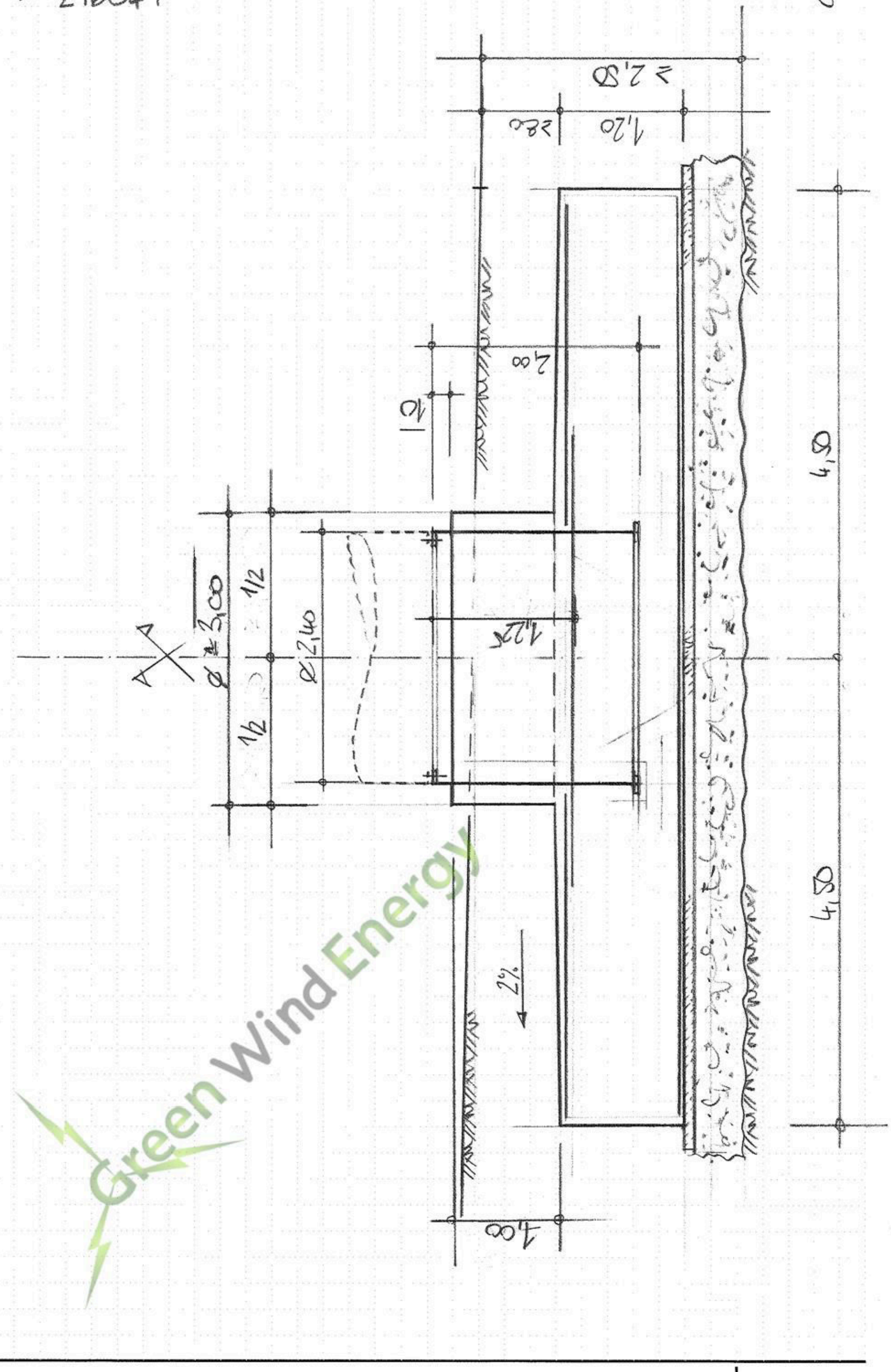
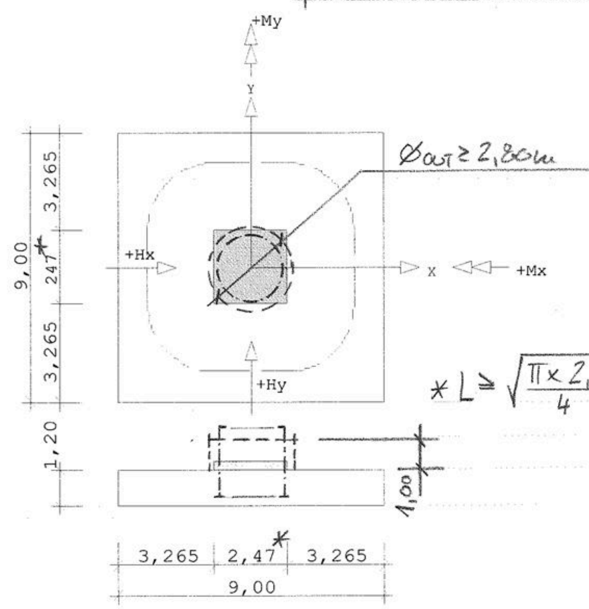


Position: 01.01 Nachweis Lagesicherheit LF1+2

Fundament FD 02/2012/O (Frilo R-2016-1/P8)

Scale 1 : 200

C25/30 XC4, XF1, WF nom c=45mm



$* L \geq \sqrt{\frac{\pi \times 2{,}80^2}{4}} \geq \underline{2{,}47m}$

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 Einzellastfälle: 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 γ-times!

height of earth load = 0.80 m
specific weight = 18.00 kN/m3
total foundation
with pedestal Gk= 3352.91 kN (for designγF = 1.35)
further loads are already γ-times put in.
moment I. Ord MxI = 3562.20 kNm for press. (DIN1054) and gaping
moment II. Ord MxII = 3562.20 kNm for design
HForce I. Ord HyI = 131.25 kN for press. (DIN1054) and gaping
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 : eccentricities
column N = 252.00 kN ax = 0.00 m ay = 0.00 m

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.
total load tot N = 3604.91 kN ex = 0.00 m ey = 0.76 m

$\frac{0{,}76}{9{,}00} = \underline{0{,}08} < \frac{1}{6} = 0{,}17.$

SOIL PRESSURE : load case No. : 1 Perm. Sigma = 210 kN/m2

γ-times column load by reductionfactor : 1.00 divided.

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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= 72.32 kN/m2 by I.Ord.
(Allgemein anerkanntes Verfahren : DIN 1054:2010)
Sohldruck nach DIN EN 1997-1 6.5.2.4 σ= 72.32 kN/m2 by II.Ord.
(Allgemein anerkanntes Verfahren : DIN 1054:2010)
for γ-times loads for calculation of designmoments :
edge pressure : max p = 80.00 kN/m2 by II.Ord f. dsgn.
min p = 16.61 kN/m2 by II.Ord f. dsgn.
below column centre p = 48.31 kN/m2 by II.Ord f. dsgn.

Gleitsicherheit nach DIN EN 1997-1 :(Phi = 30 deg)
Rtd =Rtk/1.1 =1892.09 kN > Td = 131.25 kN

design moments for γ-times loads
design moment MxEd = 2156.83 kNm (around x-axis)
design moment MxEd =-1509.17 kNm (around x-axis) top
design moment MyEd = 231.36 kNm (around y-axis)

loads : loading case 2 RESULT- LC with biaxial eccentric

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

height of earth load= 0.80 m
specific weight = 18.00 kN/m3
total foundation
with pedestal G_k= 3352.91 kN (for designγF = 1.35)
further loads are already γ-times put in.
moment I. Ord MxI = 2518.90 kNm for press. (DIN1054) and gaping
moment II. Ord MxII = 2518.90 kNm for design
moment I. Ord MyI = 2518.90 kNm for press. (DIN1054) and gaping
moment II. Ord MyII = 2518.90 kNm for design
HForce I. Ord HxI = 92.80 kN for press. (DIN1054) and gaping
HForce II. Ord HxII = 92.80 kN for design
HForce I. Ord HyI = 92.80 kN for press. (DIN1054) and gaping
HForce II. Ord HyII = 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 ax = 0.00 m ay = 0.00 m

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.
total load tot N = 3604.91 kN ex = 0.54 m ey = 0.54 m

$\left(\frac{0{,}54}{9{,}00}\right) \times 2 = \underline{0{,}12} < 0{,}A$ ✓

SOIL PRESSURE : load case No. : 2 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= 77.78 kN/m2 by I.Ord.
(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= 77.78 kN/m2 by II.Ord.
(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 designmoments :
edge pressure : 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 :(Phi = 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 = -941.82 kNm (around x-axis) top
design moment MyEd = 1633.06 kNm (around y-axis)
design moment MyEd = -941.82 kNm (around y-axis) top

loads : loading case 3 RESULT- LC with axial eccentric

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

height of earth load= 0.80 m
specific weight = 18.00 kN/m3
total foundation
with pedestal G_k= 3352.91 kN (for designγF = 1.35)
further loads are already γ-times put in.
moment I. Ord MxI = 7778.50 kNm for press. (DIN1054) and gaping
moment II. Ord MxII = 7778.50 kNm for design
HForce I. Ord HyI = 298.50 kN for press. (DIN1054) and gaping
HForce II. Ord HyII = 298.50 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 ax = 0.00 m ay = 0.00 m

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.
total load tot N = 3604.91 kN ex = 0.00 m ey = 1.67 m

$\left(\frac{1{,}67}{9{,}00}\right)^2 = 0{,}03 < \frac{1}{9} = 0{,}11$ ✓

SOIL PRESSURE : load case No. : 3 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.
with gaping (by II.Ord.)
gaping, not until center of gravity
for LC H not permitted, for LC HZ permitted !

Sohldruck nach DIN EN 1997-1 6.5.2.4 σ= 97.35 kN/m2 by I.Ord.
(Allgemein anerkanntes Verfahren : DIN 1054:2010)
length of gaping l = 0.51 m by I.Ord.

Sohldruck nach DIN EN 1997-1 6.5.2.4 σ= 97.35 kN/m2 by II.Ord.
(Allgemein anerkanntes Verfahren : DIN 1054:2010)
length of gaping l = 0.75 m by II.Ord.

for γ-times loads for calculation of designmoments :
edge pressure : 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 :(Phi = 30 deg)
Rtd =Rtk/1.1 =1892.09 kN > Td = 298.50 kN

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design moments for	γ-times loads			
design moment	MxEd = 4532.72	kNm	(around x-axis)	
design moment	MxEd =-3761.10	kNm	(around x-axis)	top
design moment	MyEd = 274.93	kNm	(around y-axis)	

loads : loading case 4 RESULT- LC with biaxial eccentric

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

height of earth load=		0.80	m	
specific weight	=	18.00	kN/m3	
total foundation				
with pedestal	G_k=	3352.91	kN (for designγF = 1.35)	
further loads are already γ-times put in.				
moment I. Ord MxI	=	5500.25	kNm	for press. (DIN1054) and gaping
moment II. Ord MxII	=	5500.25	kNm	for design
moment I. Ord MyI	=	5500.25	kNm	for press. (DIN1054) and gaping
moment II. Ord MyII	=	5500.25	kNm	for design
HForce I. Ord HxI	=	211.10	kN	for press. (DIN1054) and gaping
HForce II. Ord HxII	=	211.10	kN	for design
HForce I. Ord HyI	=	211.10	kN	for press. (DIN1054) and gaping
HForce II. Ord HyII	=	211.10	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 ax = 0.00 m ay = 0.00 m

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.
total load tot N = 3604.91 kN ex = 1.18 m ey = 1.18 m

$\left(\frac{1{,}18}{9{,}00}\right)^2 \times 2 = \underline{0{,}03} < 0{,}11$ ✓

SOIL PRESSURE : load case No. : 4 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.
with gaping (by II.Ord.)
gaping, not until center of gravity
for LC H not permitted, for LC HZ permitted !

Sohldruck nach DIN EN 1997-1 6.5.2.4 σ= 113.19 kN/m2 by I.Ord.
(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	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 σ= 113.19 kN/m2 by II.Ord.
(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	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 γ-times loads for calculation of designmoments :
edge pressure : max p = 163.64 kN/m2 by II.Ord f. dsgn.
min p = 0.00 kN/m2 by II.Ord f. dsgn.
below column centre p = 57.82 kN/m2 by II.Ord f. dsgn.

Gleitsicherheit nach DIN EN 1997-1 :(Phi = 30 deg)
Rtd =Rtk/1.1 =1892.09 kN > Td = 298.54 kN

design moments for	γ-times loads			
design moment	MxEd = 3331.17	kNm	(around x-axis)	
design moment	MxEd =-2481.84	kNm	(around x-axis)	top
design moment	MyEd = 3331.17	kNm	(around y-axis)	
design moment	MyEd =-2481.84	kNm	(around y-axis)	top

loads : loading case 5 RESULT- LC with axial eccentric

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

height of earth load= 0.80 m
specific weight = 18.00 kN/m3
total foundation
with pedestal Gk= 3661.07 kN (for designγF = 1.35)
further loads are already γ-times put in.
moment I. Ord MxI = 1408.80 kNm for press. (DIN1054) and gaping
moment II. Ord MxII = 1408.80 kNm for design
HForce I. Ord HyI = 40.80 kN for press. (DIN1054) and gaping
HForce II. Ord HyII = 40.80 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 ax = 0.00 m ay = 0.00 m

γ-times column load by reductionfactor : 1.00 divided.
γ-times further loads by reductionfactor : 1.00 divided.
sole pressure γ-fach und klaffende Fuge 1,0-fach.
total load tot N = 3913.07 kN ex = 0.00 m ey = 0.38 m

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

γ-times column load by reductionfactor : 1.00 divided.
γ-times further loads by reductionfactor : 1.00 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= 63.89 kN/m2 by I.Ord.
(Allgemein anerkanntes Verfahren : DIN 1054:2010)
Sohldruck nach DIN EN 1997-1 6.5.2.4 σ= 63.89 kN/m2 by II.Ord.
(Allgemein anerkanntes Verfahren : DIN 1054:2010)
for γ-times loads for calculation of designmoments :
edge pressure : max p = 71.80 kN/m2 by II.Ord f. dsgn.
min p = 47.13 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 :(Phi = 30 deg)
Rtd =Rtk/1.1 =2053.83 kN > Td = 40.80 kN

design moments for	γ-times loads			
design moment	MxEd = 1024.21	kNm	(around x-axis)	
design moment	MxEd = -332.97	kNm	(around x-axis)	top
design moment	MyEd = 274.93	kNm	(around y-axis)	

loads : loading case 6 RESULT- LC with biaxial eccentric

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

height of earth load=		0.80	m	
specific weight	=	18.00	kN/m3	
total foundation				
with pedestal	G_k=	3661.07	kN (for designγF = 1.35)	
further loads are already γ-times put in.				
moment I. Ord	MxI =	996.20	kNm	for press. (DIN1054) and gaping
moment II. Ord MxII	=	996.20	kNm	for design
moment I. Ord	MyI =	996.20	kNm	for press. (DIN1054) and gaping
moment II. Ord MyII	=	996.20	kNm	for design
HForce I. Ord	HxI =	28.90	kN	for press. (DIN1054) and gaping
HForce II. Ord HxII	=	28.90	kN	for design
HForce I. Ord	HyI =	28.90	kN	for press. (DIN1054) and gaping
HForce II. Ord HyII	=	28.90	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 ax = 0.00 m ay = 0.00 m

γ-times column load by reductionfactor : 1.00 divided.
γ-times further loads by reductionfactor : 1.00 divided.
sole pressure γ-fach und klaffende Fuge 1,0-fach.
total load tot N = 3913.07 kN ex = 0.27 m ey = 0.27 m

SOIL PRESSURE : load case No. : 6 Perm. Sigma = 210 kN/m2

γ-times column load by reductionfactor : 1.00 divided.
γ-times further loads by reductionfactor : 1.00 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= 65.74 kN/m2 by I.Ord.
(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	76.91
2	4.500	-4.500	59.47
3	-4.500	-4.500	42.02
4	-4.500	4.500	59.47

Sohldruck nach DIN EN 1997-1 6.5.2.4 σ= 65.74 kN/m2 by II.Ord.
(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	76.91
2	4.500	-4.500	59.47
3	-4.500	-4.500	42.02
4	-4.500	4.500	59.47

for γ-times loads for calculation of designmoments :
edge pressure : max p = 76.91 kN/m2 by II.Ord f. dsgn.
min p = 42.02 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 :(Phi = 30 deg)
Rtd =Rtk/1.1 =2053.83 kN > Td = 40.87 kN

design moments for γ-times loads

design moment	MxEd = 803.50	kNm	(around x-axis)	
design moment	MxEd = -112.26	kNm	(around x-axis)	top
design moment	MyEd = 803.50	kNm	(around y-axis)	
design moment	MyEd = -112.26	kNm	(around y-axis)	top

loads : loading case 7 RESULT- LC with central loading

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

height of earth load= 0.80 m
specific weight = 18.00 kN/m3
total foundation
with pedestal Gk= 3661.07 kN (for designγF = 1.35)
further loads are already γ-times put in.
vertical loads : e c c e n t r i c i t i e s
column N = 252.00 kN ax = 0.00 m ay = 0.00 m

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

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

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

design moments for γ-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) !

lc	1	(rd x) MxEd	=	2156.83	kNm	req. Ac	=	39.09	cm2
		top Mx	=	-1509.17	kNm	rec.Actp	=	27.82	cm2
		(rd y) MyEd	=	231.36	kNm	req. Ac	=	7.82	cm2
lc	2	(rd x) MxEd	=	1633.06	kNm	req. Ac	=	28.96	cm2
		top Mx	=	-941.82	kNm	rec.Actp	=	17.73	cm2
		(rd y) MyEd	=	1633.06	kNm	req. Ac	=	29.22	cm2
		top My	=	-941.82	kNm	rec.Actp	=	17.89	cm2
lc	3	(rd x) MxEd	=	4532.72	kNm	req. Ac	=	83.92	cm2
		top Mx	=	-3761.10	kNm	rec.Actp	=	67.11	cm2
		(rd y) MyEd	=	274.93	kNm	req. Ac	=	16.78	cm2
lc	4	(rd x) MxEd	=	3331.17	kNm	req. Ac	=	60.18	cm2
		top Mx	=	-2481.84	kNm	rec.Actp	=	44.13	cm2
		(rd y) MyEd	=	3331.17	kNm	req. Ac	=	60.72	cm2
		top My	=	-2481.84	kNm	rec.Actp	=	44.53	cm2
lc	5	(rd x) MxEd	=	1024.21	kNm	req. Ac	=	18.12	cm2
		top Mx	=	-332.97	kNm	rec.Actp	=	6.93	cm2
		(rd y) MyEd	=	274.93	kNm	req. Ac	=	4.89	cm2
lc	6	(rd x) MxEd	=	803.50	kNm	req. Ac	=	14.21	cm2
		top Mx	=	-112.26	kNm	rec.Actp	=	3.04	cm2
		(rd y) MyEd	=	803.50	kNm	req. Ac	=	14.33	cm2
		top My	=	-112.26	kNm	rec.Actp	=	3.06	cm2
lc	7	(rd x) MxEd	=	373.77	kNm	req. Ac	=	6.59	cm2
		(rd y) MyEd	=	373.77	kNm	req. Ac	=	6.65	cm2

bending reinforcement :	C 25/30 B 550(A) according EN 1992-1-1							
Distribution of the reinforc. acc. to paper DAfStb 240 table 2.10.								
y-direction:	static height		hy =	1.14	m		chosen	
	top reinforcem.		tot As =	67.11	cm2			
	reinforc. bottom tot As		=	83.92	cm2		57 o	16
	distribution	bx/8	bx/8	bx/8	bx/8			
	(cm2)	7.55	9.23	11.75	13.43			
	(cm2/m)	6.71	8.21	10.44	11.94			
x-direction:	static height		hx =	1.13	m		chosen	
	top reinforcem.		tot As =	44.53	cm2			
	reinforc. bottom tot As		=	60.72	cm2		57 o	16
	distribution	by/8	by/8	by/8	by/8			
	(cm2)	5.47	6.68	8.50	9.72			
	(cm2/m)	4.86	5.94	7.56	8.64			

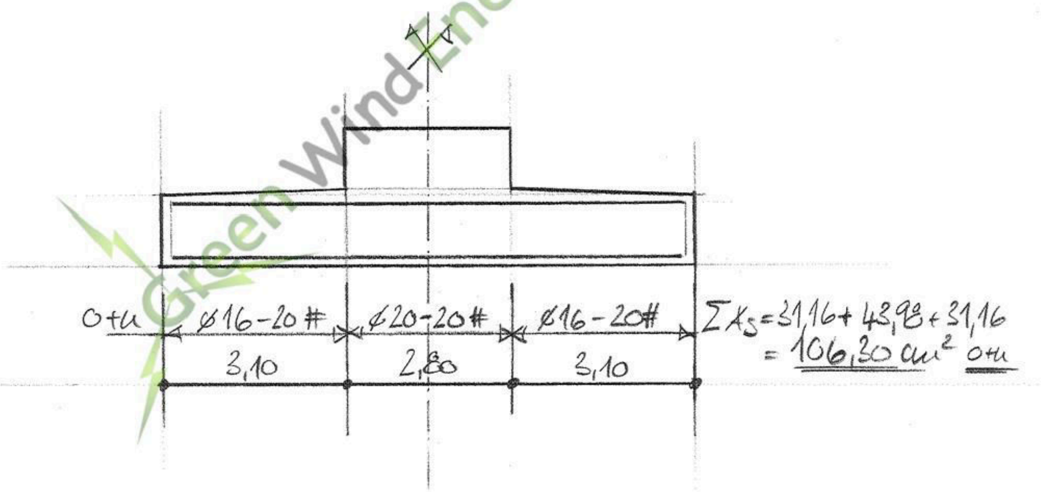
by user selected reinforcement :

Y -direction reinf. bottom :	114.57	cm2				
distribution sel. reinf. in y-direction			bx/8	bx/8	bx/8	bx/8
		(cm2)	14.32	14.32	14.32	14.32
X -direction reinf. bottom :	114.57	cm2				
distribution sel. reinf. in x-direction			by/8	by/8	by/8	by/8
		(cm2)	14.32	14.32	14.32	14.32

proof of punching considering the user selected
bending reinforcement :

limit state the bearing capacity for punching shear according EN 1992-1-1			
rk = 1.90 * dm	=	2.16	m
u_crit	=	23.43	m
A_crit	=	42.02	m2
existing average Rho	=	0.11	%
load increasing factor	=	1.00	
red VEd (without Beta)	=	335.09	kN
Beta	=	1.15	
vEd (Beta considered)	=	0.014	N/mm2
vRd,c	=	0.253	N/mm2> vEd
not additional punching shear reinforcement necessary.			

connection reinforc. according EN 1992-1-1 req tot As =139.58 cm2 12 o 40



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PROOF OF FATIGUE STRENGTH

(acc. to EN 1992-1-1)

CONCRETE STEEL

As-values and loadcases: ref. printout

LC 5	$A_{s,req}$	=	18,12 cm²
LC 7	$A_{s,req}$	=	6,65 cm²
	$A_{s,prov}$ = 31,16+43,98+31,16	=	106,30 cm²
B500S	f_{yk}	=	50,00 kN/cm²
	γ_M	=	1,15
	f_{yd} = 50,00 / 1,15	=	43,50 kN/cm²
	σ_{LC7} = 18,12/106,30 x 43,50	=	7,42 kN/cm²
	σ_{LC5} = 6,65/106,30 x 43,50	=	2,72 kN/cm²
	$\Delta\sigma$ = 7,42 - 2,72	**=**	**4,70 kN/cm²**

< $\Delta\sigma$ = 7,00 kN/cm² (acc. EN 1992-1-1, 6.8.6 (1)) ✓

CONCRETE

Concrete C25/30	f_{ck}	=	25,00 MN/m²
	γ_M	=	1,50
	f_{cd} = 0,85 x 25,00/1,50	=	14,17 MN/m²
EN 1992-1-1 (6.8.7)	k_1	=	1,00
first loading after	t_0	=	28 days
	$\beta_{cc}(28d)$	=	1,00
(6.8.7 (6.76))	$f_{cd,\,fat}$	=	12,75 MN/m²
	b	=	9,00 m
	h = 1,20-0,05-0,02	=	1,13 m
	d_1	=	0,07 m
	$A_{s1} = A_{s2}$	=	106,30 cm²
compression zone (acc. E. Grasser)	x	=	14,54 cm
LC 5	$\sigma_{c,max}$ = 803,50/4532,72 x 14,17	=	2,51 MN/m²
LC 7	$\sigma_{c,min}$ = 373,77/4532,72 x 14,17	=	1,17 MN/m²
	$\sigma_{c,max} / f_{cd,fat}$ = 2,51 / 12,75	=	0,20
	$\sigma_{c,min} / f_{cd,fat}$ = 1,17 / 12,75	=	0,09

$\sigma_{c,max}$ / $f_{cd,fat}$ = 0,20 ≤ min { 0,50 + 0,45 x 0,09 = **0,54** ; 0,90} ✓

Aufgestellt

14. März 2016

Bearbeiter

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F. Schaefer
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Unterschrift

Für die Prüfung

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Der Bauherr

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