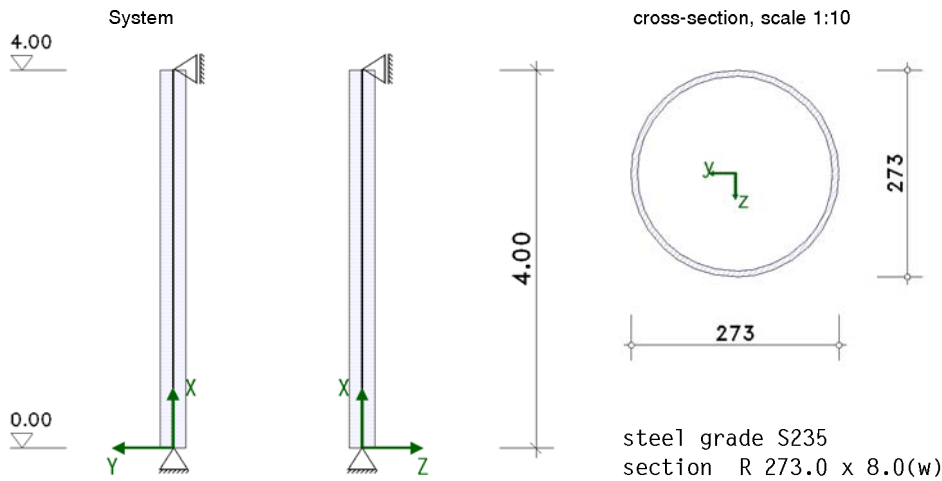


steel column

design calculation acc. to DIN EN 1993-1-1:2010-12 with NA-Deutschland(DIN EN 1993-1-1/NA:2022-10)



support situation at head and foot end

supp.	shear force		moment	
	C _{QY} kN/m	C _{QZ} kN/m	C _{MY} kNm/-	C _{MZ} kNm/-
head	fix	fix	----	----
foot	fix	fix	----	----

1. loading

1.1. structure of action effects

on the left-hand side, the action effects and load cases are shown in a tree structure. the right-hand side shows their characteristics of the superposition.

applied symbols: action effect load case

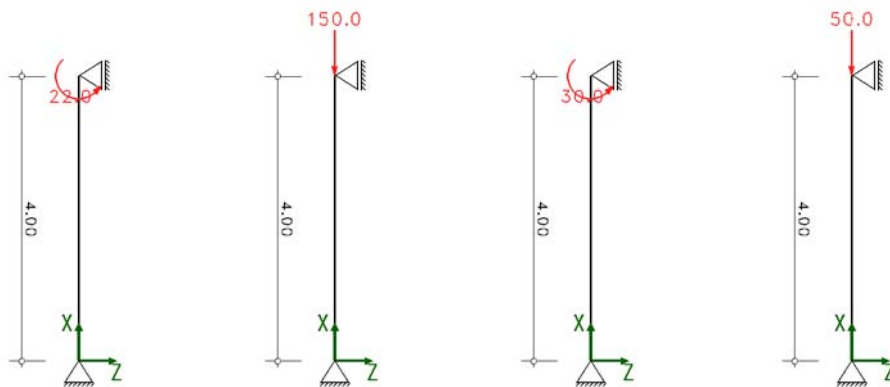
1: permanent loads	permanent loads
1: dead load	additive
4: live load	transient live loads of storage rooms
2: live load	additive
9: wind	transient wind load
3: Y-direction	alternative in group A
4: Z-direction	additive

1.2. table of load pictures

loadc.	loadpic.	introduc.	direction	value	entity
1	pointload	head	N	150.00	kN
			M _y	22.00	kNm
2	pointload	head	N	50.00	kN
			M _y	30.00	kNm
3	line load full height		q _y	10.00	kN/m
4	line load full height		q _z	12.00	kN/m

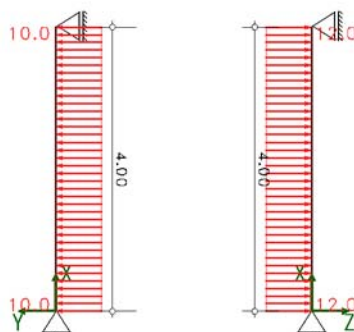
1.2.1. graphics of point loads

load case 1 (figure 1) load case 1 (figure 2) load case 2 (figure 3) load case 2 (figure 4)



1.2.2. graphics of line loads

load case 3 (figure 1) load case 4 (figure 2)



1.3. dead load of the column

the weight of the column is taken into account with 78.50 kN/m³ in load case 1.

2. design resistance acc. to Th.II.O.

cross sectional check: plastic stress verification incl. c/t-verification

material safety: $\gamma_{M1} = 1.10$

perm. normal, shear, equivalent stresses: $\sigma_{x,Rd} = 213.6 \text{ N/mm}^2$, $\tau_{Rd} = 123.3 \text{ N/mm}^2$, $\sigma_{v,Rd} = 213.6 \text{ N/mm}^2$

2.1. consideration of structural imperfections

2.1.1. imperfection figures

an imperfection figure is determined for each axis direction acc. to [2] resp. [1], section 5.3.

replacement length $\beta = l_0/l$

global initial obliquity $\Phi = \Phi_0 \cdot \alpha_h \cdot \alpha_m$

buckling load $N_{Ki} = (\pi/l_0)^2 EI$

direct.	form	β -	l_0 m	Φ -	e cm	EI MNm ²	N_{Ki} MN
Y	curvat.	1.00	4.00	0.0050	0.000	12.29	7.59
Z	curvat.	1.00	4.00	0.0050	0.000	12.29	7.59

l_0 - buckling length e - max. pre-deformation EI - bending stiffness

2.1.2. direction of imperfection

$N_{Ki,y}/N_{Ki,z} = 1.00 \Rightarrow$ immer in direction der Verformung aus planmäßiger load.

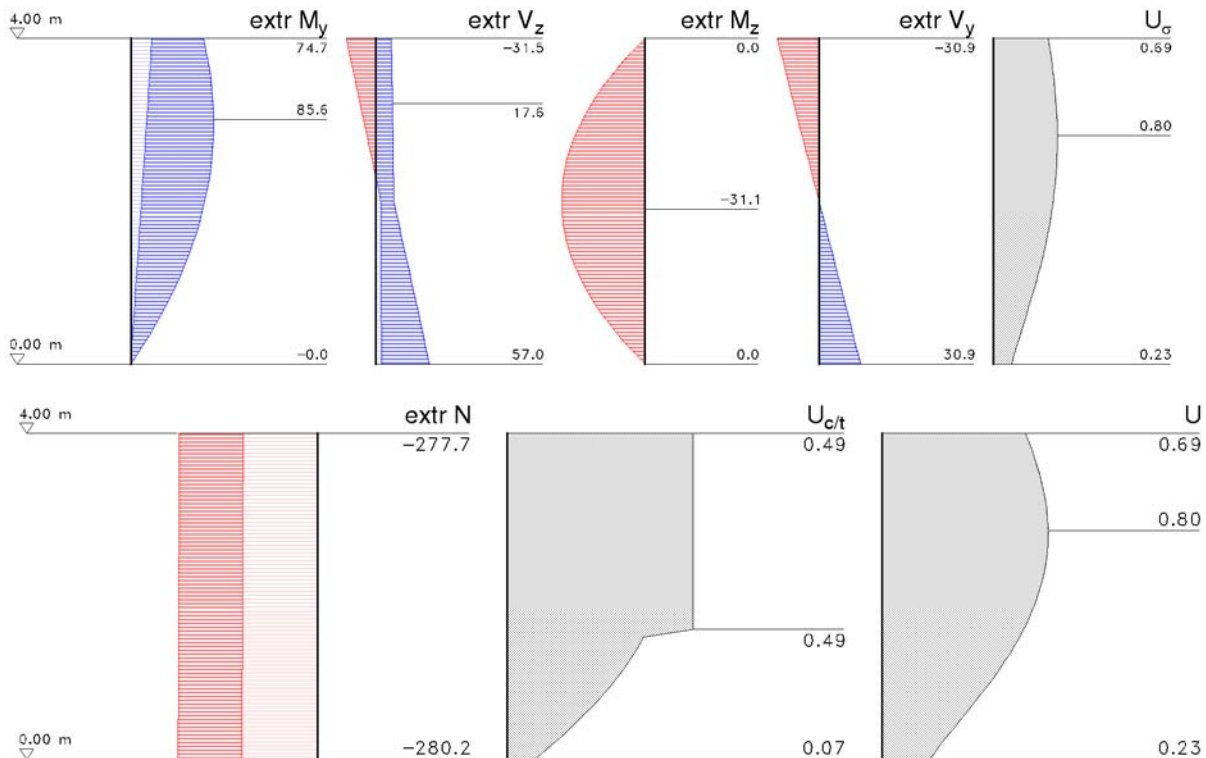
2.2. factorization of load case combinations

LS	design situat.	factorization
1	perman.	$Lc1+I_z$
2	perman.	$1.35 \cdot Lc1+I_z$
3	perman.	$Lc1+1.5 \cdot Lc2+I_z$
4	perman.	$1.35 \cdot Lc1+1.5 \cdot Lc2+I_z$
5	perman.	$Lc1+0.6 \cdot 1.5 \cdot Lc3+0.8 \cdot I_y+0.6 \cdot I_z$
6	perman.	$1.35 \cdot Lc1+0.6 \cdot 1.5 \cdot Lc3+0.7 \cdot I_y+0.71 \cdot I_z$
7	perman.	$Lc1+1.5 \cdot Lc2+0.6 \cdot 1.5 \cdot Lc3+0.39 \cdot I_y+0.92 \cdot I_z$
8	perman.	$1.35 \cdot Lc1+1.5 \cdot Lc2+0.6 \cdot 1.5 \cdot Lc3+0.36 \cdot I_y+0.93 \cdot I_z$
9	perman.	$Lc1+0.6 \cdot 1.5 \cdot Lc4+I_z$

LS	design situat.	factorization
10	perman.	1.35·Lc1+0.6·1.5·Lc4+I _z
11	perman.	Lc1+1.5·Lc2+0.6·1.5·Lc4+I _z
12	perman.	1.35·Lc1+1.5·Lc2+0.6·1.5·Lc4+I _z
13	perman.	Lc1+0.6·1.5·(Lc3+Lc4)+0.46·I _y +0.89·I _z
14	perman.	1.35·Lc1+0.6·1.5·(Lc3+Lc4)+0.41·I _y +0.91·I _z
15	perman.	Lc1+1.5·Lc2+0.6·1.5·(Lc3+Lc4)+0.27·I _y +0.96·I _z
16	perman.	1.35·Lc1+1.5·Lc2+0.6·1.5·(Lc3+Lc4)+0.26·I _y +0.97·I _z
17	perman.	Lc1+1.5·Lc3+0.92·I _y +0.4·I _z
18	perman.	1.35·Lc1+1.5·Lc3+0.86·I _y +0.52·I _z
19	perman.	Lc1+1.5·(Lc2+Lc3)+0.58·I _y +0.81·I _z
20	perman.	1.35·Lc1+1.5·(Lc2+Lc3)+0.54·I _y +0.84·I _z
21	perman.	Lc1+1.5·Lc4+I _z
22	perman.	1.35·Lc1+1.5·Lc4+I _z
23	perman.	Lc1+1.5·(Lc2+Lc4)+I _z
24	perman.	1.35·Lc1+1.5·(Lc2+Lc4)+I _z
25	perman.	Lc1+1.5·(Lc3+Lc4)+0.52·I _y +0.85·I _z
26	perman.	1.35·Lc1+1.5·(Lc3+Lc4)+0.48·I _y +0.87·I _z
27	perman.	Lc1+1.5·(Lc2+Lc3+Lc4)+0.36·I _y +0.93·I _z
28	perman.	1.35·Lc1+1.5·(Lc2+Lc3+Lc4)+0.35·I _y +0.94·I _z

2.3. extreme results

the internal forces are related to the deformed system axis



x m	N		My		Vz		Mz		Vy		U
	min kN	max kN	min kNm	max kNm	min kN	max kN	min kNm	max kNm	min kN	max kN	
4.00	-277.66	-149.71	22.00	74.70	-31.46	16.37	-0.00	0.00	-30.94	-0.00	0.69
3.00	-278.26	-150.44	16.74	85.56	-13.04	17.86	-23.34	0.00	-15.65	-0.00	0.80
2.80	-278.38	-150.58	15.66	85.34	-9.33	18.11	-26.16	0.00	-12.53	-0.00	0.80
2.30	-278.71	-150.89	12.93	81.32	-0.04	18.67	-30.47	0.00	-4.71	-0.00	0.79
2.20	-278.77	-150.94	12.38	79.92	1.82	18.77	-30.86	0.00	-3.15	-0.00	0.78
2.00	-278.89	-151.04	11.28	76.53	5.54	18.96	-31.18	0.00	-0.00	-0.00	0.75
1.90	-278.96	-151.09	10.72	74.53	5.56	20.93	-31.10	0.00	-0.00	1.57	0.74
0.10	-280.17	-151.74	0.57	5.61	5.68	55.18	-3.02	0.00	-0.00	29.44	0.26
0.00	-280.24	-151.78	-0.00	0.00	5.69	56.99	-0.00	0.00	-0.00	30.94	0.23

relevant utilisation:

from load spectrum 28 at the location $x = 2.80$ m, with the internal forces and moments:

$N = -278.34$ kN, $M_y = 85.34$ kNm, $V_z = 3.07$ kN, $M_z = -26.16$ kNm, $V_y = -12.53$ kN

$U_{c/t} = 0.49$, $U_{\sigma} = 0.80 \Rightarrow \max U = 0.80 < 1.0 \Rightarrow$ permissible utilisation is complied with

3. serviceability

calculation acc. to theory II. Order for the quasi-permanent combination acc. to [3], eq. 6.16

verification of the permissible horizontal deformation acc. to [2], section 7.2.2

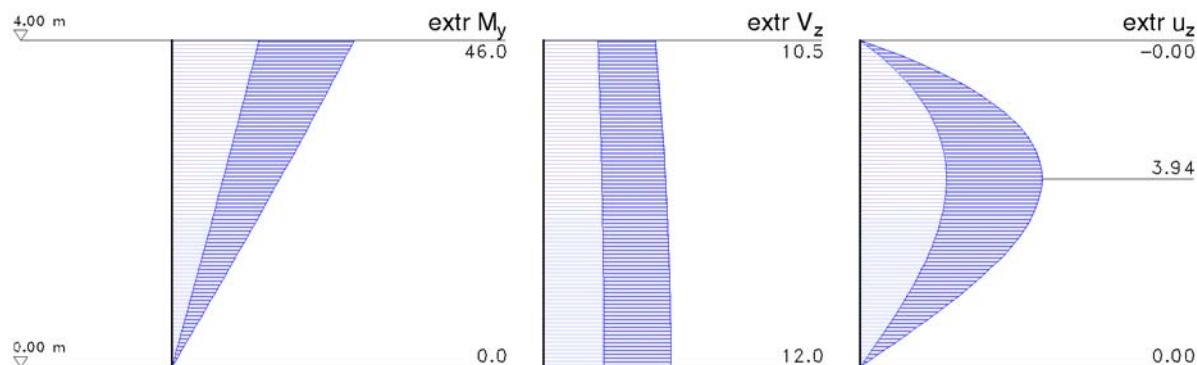
3.1. factorization of load case combinations

LS	design situat.	factorization
1	perman.	Lc1
2	perman.	Lc1+0.8·Lc2

3.2. extreme results

the internal forces are related to the deformed system axis

3.2.1. deformation in Z-direction



x m	My		Vz		uz	
	min kNm	max kNm	min kN	max kN	min mm	max mm
4.00	22.00	46.00	5.14	10.53	-0.00	-0.00
2.30	12.93	27.20	5.50	11.50	1.87	3.94
0.00	0.00	0.00	5.69	11.99	0.00	0.00

relevant deformation in Z-direction:

from load spectrum 2 at the location $x = 2.30$ m, with the internal forces and moments:

$N = -190.89$ kN, $M_y = 27.20$ kNm, $V_z = 11.50$ kN, $M_z = 0.00$ kNm, $V_y = -0.00$ kN

$u_z = 3.94 < 6.00$ mm $\Rightarrow$ permissible deformation is complied with

4. support reaction

4.1. characteristic support reactions of load cases at the column base

Lc	N _k kN	H _{k,y} kN	H _{k,z} kN	M _{k,y} kN	M _{k,z} kN
1	152.09	-0.00	5.50	0.00	-0.00
2	50.00	-0.00	7.50	-0.00	0.00
3	0.00	20.00	-0.00	-0.00	0.00
4	0.00	-0.00	24.00	-0.00	-0.00
Σ	202.09	20.00	37.00	-0.00	0.00

4.2. extreme design values of support reactions (design resistance Th.II.O.) at the column base

LS	N _{Ed} kN	H _{Ed,y} kN	H _{Ed,z} kN	M _{Ed,y} kN	M _{Ed,z} kN
1	152.09	-0.00	5.50	0.00	-0.00
4	280.32	-0.00	18.67	0.00	0.00
9	152.09	-0.00	27.10	-0.00	0.00
11	227.09	-0.00	38.35	0.00	0.00
17	152.09	30.00	5.50	0.00	-0.00
18	205.32	30.00	7.42	-0.00	0.00
20	280.32	30.00	18.67	0.00	-0.00
24	280.32	-0.00	54.67	-0.00	0.00

5. summary

all verifications were carried out successfully.

maximum utilisation of the verifications:

design resist Th.II.O	80 %
deformation in Z-direct.	66 %

literature and standards:

[1] DIN EN 1993-1-1/NA: Nationaler Anhang - National festgelegte Parameter - Eurocode 3: Teil 1-1, Okt. 2022

[2] DIN EN 1993-1-1: Eurocode 3: Bem. und Konstr. von Stahlbauten - Teil 1-1: Allg. Bem.regeln u. Regeln für den Hochbau, Dez. 2010

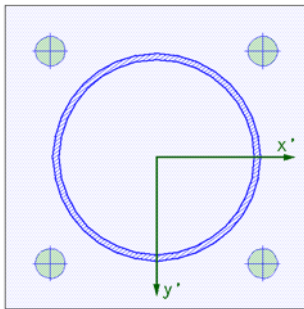
[3] DIN EN 1990: Grundlagen der Tragwerksplanung, Oktober 2021

BASE PLATE WITH ISOLATED FOUNDATION

4H-FUND version: 12/2022-1u

steel code verifications acc. to DIN EN 1993-1-2:2010-12 with NA-Deutschland
reinf. concr. design acc. to DIN EN 1992-1-1:2011-01 with NA-Deutschland(DIN EN 1992-1-1/NA:2013-04)
external stability acc. to DIN EN 1997-1:2014-03 with NA-Deutschland
additional rules acc. to DIN 1054:2021-04 , DIN 4017:2006-03 and DIN 4019:2015-05

top view base plate
scale 1:10



column cross section

standardized profile: R 273.0 x 8.0(w), of quality S235

base plate

$b_x = 400 \text{ mm}$ $b_y = 400 \text{ mm}$ $t = 30 \text{ mm}$, of quality S235

mortar joint

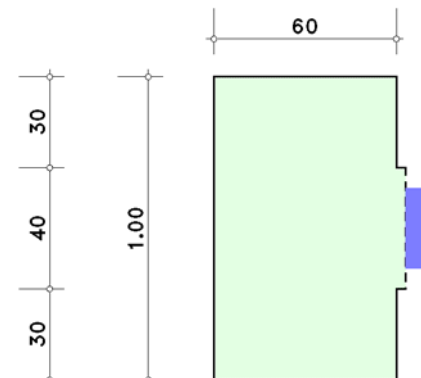
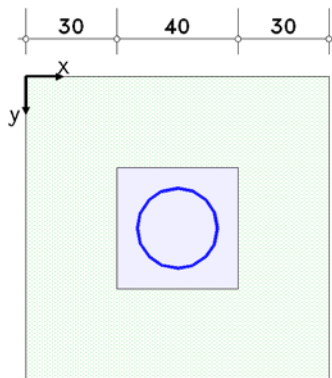
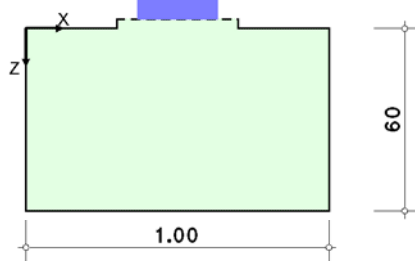
$t_F = 20 \text{ mm}$

anchors

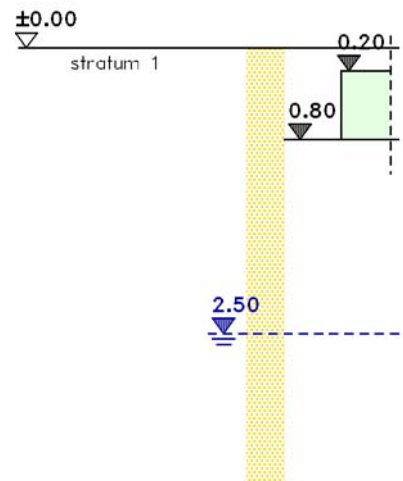
4 anchors, FK 8.8, M22, without shaft
with a length of 450 mm

edge distances $a_x/a_y = 60/60 \text{ mm}$

elevation, top view isolated foundation
scale 1:25



soil profile



concrete strength class C25/30

steel class B500A

1. soil situation

the anchoring depth of the foundation is $t = 0.80 \text{ m}$.

the ground water level (below top edge soil) is at $t_w = 2.50 \text{ m}$.

1.1. designation and characteristic values of soil strata

stratum	z m	γ kN/m ³	γ' kN/m ³	ϕ °	c_k kN/m ²	E_m MN/m ²	δ_p °
stratum 1	0.00	19.00	11.00	32.5	---	10.00	auto

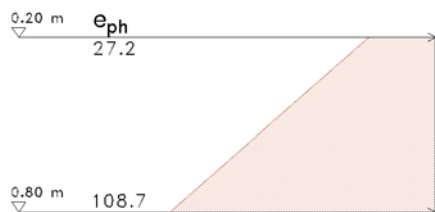
z - altitude at the top of the layer γ - unit weight γ' - unit weight of submerged soil ϕ - friction angle

c_k - char. cohesion of the drained soil E_m - mean compression modulus δ_p - angle of wall friction on the passive side

1.2. char. passive earth pressure

the passive earth pressure is determined for flat sliding surfaces

For a rough wall surface, with a wall friction angle $\delta = 2/3 \cdot \phi'_k$



$\Sigma(\gamma \cdot h)$ total soil weight at the considered depth
 $\Sigma(\gamma \cdot h)_{cal}$ total soil weight at the considered depth plus influence of the slope
 K_{pgh} coeff. of earth pressure acc. to [1] section 6.2.1, eq.(7) (formulation acc. to Müller-Breslau)
 e_{ph} ordinate of horiz. earth pressure

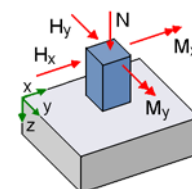
z m	$\Sigma(\gamma \cdot h)$ kN/m ²	$\Sigma(\gamma \cdot h)_{cal}$ kN/m ²	K_{pgh} -	e_{ph} kN/m ²
0.20	3.80	6.65	7.152	27.18
0.80	15.20	15.20	7.152	108.71

the resultant maximum passive earth pressure is $E_{phg} = 40.77$ kN/m, at $z_s = 0.56$ m.

2. loading

2.1. design calculation situation of load cases for external stability

loadc.	notation	BS-P	BS-T
1	dead load	x	
2	live load	x	
3	Y-direction	x	
4	Z-direction	x	



2.2. design values column load from 'EC 3 design resistance Th.II.O.' for reinforced concrete design

point of application in centroid of the column on the top of the foundation slab

LS	notation	design situat.	N _{st,d} kN	H _{x,St,Ed} kN	H _{y,St,Ed} kN	M _{x,St,Ed} kNm	M _{y,St,Ed} kNm
5	LS 1 (auto)	perman. a.v.	152.09	5.50	-0.00	-0.00	0.00
6	LS 2 (auto)	perman. a.v.	205.32	7.42	-0.00	-0.00	-0.00
7	LS 3 (auto)	perman. a.v.	227.09	16.75	-0.00	0.00	-0.00
8	LS 4 (auto)	perman. a.v.	280.32	18.67	-0.00	0.00	0.00
9	LS 5 (auto)	perman. a.v.	152.09	5.50	18.00	0.00	0.00
10	LS 6 (auto)	perman. a.v.	205.32	7.42	18.00	0.00	-0.00
11	LS 7 (auto)	perman. a.v.	227.09	16.75	18.00	-0.00	-0.00
12	LS 8 (auto)	perman. a.v.	280.32	18.67	18.00	-0.00	0.00
13	LS 9 (auto)	perman. a.v.	152.09	27.10	-0.00	0.00	-0.00
14	LS 10 (auto)	perman. a.v.	205.32	29.02	-0.00	-0.00	-0.00
15	LS 11 (auto)	perman. a.v.	227.09	38.35	-0.00	0.00	0.00
16	LS 12 (auto)	perman. a.v.	280.32	40.27	-0.00	0.00	-0.00
17	LS 13 (auto)	perman. a.v.	152.09	27.10	18.00	0.00	-0.00
18	LS 14 (auto)	perman. a.v.	205.32	29.02	18.00	0.00	-0.00
19	LS 15 (auto)	perman. a.v.	227.09	38.35	18.00	-0.00	0.00
20	LS 16 (auto)	perman. a.v.	280.32	40.27	18.00	-0.00	-0.00
21	LS 17 (auto)	perman. a.v.	152.09	5.50	30.00	-0.00	0.00
22	LS 18 (auto)	perman. a.v.	205.32	7.42	30.00	0.00	-0.00
23	LS 19 (auto)	perman. a.v.	227.09	16.75	30.00	-0.00	-0.00
24	LS 20 (auto)	perman. a.v.	280.32	18.67	30.00	-0.00	0.00
25	LS 21 (auto)	perman. a.v.	152.09	41.50	-0.00	-0.00	-0.00
26	LS 22 (auto)	perman. a.v.	205.32	43.42	-0.00	-0.00	-0.00
27	LS 23 (auto)	perman. a.v.	227.09	52.75	-0.00	-0.00	0.00
28	LS 24 (auto)	perman. a.v.	280.32	54.67	-0.00	0.00	-0.00
29	LS 25 (auto)	perman. a.v.	152.09	41.50	30.00	-0.00	-0.00
30	LS 26 (auto)	perman. a.v.	205.32	43.42	30.00	0.00	-0.00
31	LS 27 (auto)	perman. a.v.	227.09	52.75	30.00	-0.00	0.00
32	LS 28 (auto)	perman. a.v.	280.32	54.67	30.00	-0.00	-0.00

2.3. dead load

the weight of the foundation plate is taken into account with $\gamma_E = 25.00$ kN/m³.

the height of the earth load is $h_A = 0.20$ m.

the mean unit weight of the earth load is $\gamma_A = 19.00$ kN/m³.

the resultant of dead load in the floor joint is $N_{0,dead1,k} = 18.19$ kN.

the dead weight is taken into account in load case 1.

3. verification of steel column base

3.1. partial safety factors for material

design situat.	γ_{M1}	γ_{M2}	γ_c	γ_μ
permanent	1.10	1.25	1.50	1.20

the resistance values of the steel are determined for stability failure.

3.2. design values of steel verifications

3.2.1. factorization of load case combinations

LS	factorization	LS	factorization	LS	factorization	LS	factorization	LS	factorization
1	Lc5	7	Lc11	13	Lc17	19	Lc23	25	Lc29
2	Lc6	8	Lc12	14	Lc18	20	Lc24	26	Lc30
3	Lc7	9	Lc13	15	Lc19	21	Lc25	27	Lc31
4	Lc8	10	Lc14	16	Lc20	22	Lc26	28	Lc32
5	Lc9	11	Lc15	17	Lc21	23	Lc27		
6	Lc10	12	Lc16	18	Lc22	24	Lc28		

3.2.2. column load

LS	N _{st,d} kN	H _{x,St,d} kN	H _{y,St,d} kN	M _{x,St,d} kNm	M _{y,St,d} kNm	LS	N _{st,d} kN	H _{x,St,d} kN	H _{y,St,d} kN	M _{x,St,d} kNm	M _{y,St,d} kNm
1	152.09	5.50	-0.00	-0.00	0.00	15	227.09	38.35	18.00	-0.00	0.00
2	205.32	7.42	-0.00	-0.00	-0.00	16	280.32	40.27	18.00	-0.00	-0.00
3	227.09	16.75	-0.00	0.00	-0.00	17	152.09	5.50	30.00	-0.00	0.00
4	280.32	18.67	-0.00	0.00	0.00	18	205.32	7.42	30.00	0.00	-0.00
5	152.09	5.50	18.00	0.00	0.00	19	227.09	16.75	30.00	-0.00	-0.00
6	205.32	7.42	18.00	0.00	-0.00	20	280.32	18.67	30.00	-0.00	0.00
7	227.09	16.75	18.00	-0.00	-0.00	21	152.09	41.50	-0.00	-0.00	-0.00
8	280.32	18.67	18.00	-0.00	0.00	22	205.32	43.42	-0.00	-0.00	-0.00
9	152.09	27.10	-0.00	0.00	-0.00	23	227.09	52.75	-0.00	-0.00	0.00
10	205.32	29.02	-0.00	-0.00	-0.00	24	280.32	54.67	-0.00	0.00	-0.00
11	227.09	38.35	-0.00	0.00	0.00	25	152.09	41.50	30.00	-0.00	-0.00
12	280.32	40.27	-0.00	0.00	-0.00	26	205.32	43.42	30.00	0.00	-0.00
13	152.09	27.10	18.00	0.00	-0.00	27	227.09	52.75	30.00	-0.00	0.00
14	205.32	29.02	18.00	0.00	-0.00	28	280.32	54.67	30.00	-0.00	-0.00

3.3. weld between column shaft and base plate

design with direction oriented method acc. to clause 4.5.3.2

$$\sigma_{1,w,Ed} = (\sigma_{\perp}^2 + 3 \cdot \tau_{\perp}^2 + 3 \cdot \tau_{\parallel}^2)^{0.5}$$

$$\sigma_{2,w,Ed} = \sigma_{\perp}$$

$$f_{1,w,Rd} = f_u / (\beta_w \cdot \gamma_{M2})$$

$$f_{2,w,Rd} = 0.9 f_u / \gamma_{M2}$$

$$U = \max \{ \sigma_{1,w,Ed} / f_{1,w,Rd}, \sigma_{2,w,Ed} / f_{2,w,Rd} \}$$

connection designed with a circumferential fillet weld.

axial force transfer of 100 % by the weld.

minimum value of the weld thickness $a_{\min} = 5 \text{ mm}$

LS	a _w mm	$\sigma_{\perp}$ kN/cm ²	$\tau_{\perp}$ kN/cm ²	$\tau_{\parallel}$ kN/cm ²	$\sigma_{1,w,Ed}$ kN/cm ²	f _{1,w,Rd} kN/cm ²	$\sigma_{2,w,Ed}$ kN/cm ²	f _{2,w,Rd} kN/cm ²	U -
1	5	-2.51	-2.51	0.26	5.04	36.00	2.51	25.92	0.14
2	5	-3.39	-3.39	0.35	6.80	36.00	3.39	25.92	0.19
3	5	-3.74	-3.74	0.78	7.61	36.00	3.74	25.92	0.21
4	5	-4.62	-4.62	0.87	9.37	36.00	4.62	25.92	0.26
5	5	-2.51	-2.51	0.88	5.24	36.00	2.51	25.92	0.15
6	5	-3.39	-3.39	0.91	6.95	36.00	3.39	25.92	0.19
7	5	-3.74	-3.74	1.15	7.75	36.00	3.74	25.92	0.22
8	5	-4.62	-4.62	1.21	9.48	36.00	4.62	25.92	0.26
9	5	-2.51	-2.51	1.26	5.47	36.00	2.51	25.92	0.15
10	5	-3.39	-3.39	1.35	7.17	36.00	3.39	25.92	0.20
11	5	-3.74	-3.74	1.79	8.10	36.00	3.74	25.92	0.23
12	5	-4.62	-4.62	1.88	9.80	36.00	4.62	25.92	0.27
13	5	-2.51	-2.51	1.52	5.66	36.00	2.51	25.92	0.16
14	5	-3.39	-3.39	1.59	7.31	36.00	3.39	25.92	0.20
15	5	-3.74	-3.74	1.98	8.23	36.00	3.74	25.92	0.23
16	5	-4.62	-4.62	2.06	9.91	36.00	4.62	25.92	0.28
17	5	-2.51	-2.51	1.42	5.59	36.00	2.51	25.92	0.16
18	5	-3.39	-3.39	1.44	7.22	36.00	3.39	25.92	0.20
19	5	-3.74	-3.74	1.60	7.99	36.00	3.74	25.92	0.22
20	5	-4.62	-4.62	1.65	9.68	36.00	4.62	25.92	0.27
21	5	-2.51	-2.51	1.94	6.03	36.00	2.51	25.92	0.17
22	5	-3.39	-3.39	2.03	7.63	36.00	3.39	25.92	0.21
23	5	-3.74	-3.74	2.46	8.62	36.00	3.74	25.92	0.24
24	5	-4.62	-4.62	2.55	10.25	36.00	4.62	25.92	0.28
25	5	-2.51	-2.51	2.39	6.50	36.00	2.51	25.92	0.18

LS	a _w mm	σ _⊥ kN/cm ²	τ _⊥ kN/cm ²	τ kN/cm ²	σ _{1,w,Ed} kN/cm ²	f _{1,w,Rd} kN/cm ²	σ _{2,w,Ed} kN/cm ²	f _{2,w,Rd} kN/cm ²	U -
26	5	-3.39	-3.39	2.46	8.00	36.00	3.39	25.92	0.22
27	5	-3.74	-3.74	2.83	8.95	36.00	3.74	25.92	0.25
28	5	-4.62	-4.62	2.91	10.53	36.00	4.62	25.92	0.29

a_w - weld thickness σ_⊥ - normal stresses perpendicular to weld τ_⊥ - shear stresses perpendicular to weld
τ_{||} - shear stresses parallel to weld U - utilization

maximum weld thickness a_{w,max} = 5 mm

maximum utilization U = 0.29 < 1.00

3.4. FE-calculation

the calculation of pressures under the base plate and of the base plate decisive internal forces and moments is done by a FEM-calculation using constrained modulus method. the initial bedding of the plate results from the concrete modulus of elasticity under the base plate. tension springs are eliminated in elastic bedded areas. anchors are considered as point springs only acting in case of tension.

the plate is divided into 34 elements in X-direction and 34 elements in Y-direction.

the concrete compression is limited to the allowable partial area pressure with $\lim \sigma_{c,d} = f_{Rd,u}$.

the equivalent spring for the anchors is applied with $c = E \cdot A / l = 1414.00 \text{ kN/cm}$.

3.4.1. stresses in base plate (elast.-plast.)

internal forces and moments

LS	x _{Fp} cm	y _{Fp} cm	m _{xx} kNcm/cm	m _{yy} kNcm/cm	m _{xy} kNcm/cm	v _x kN/cm	v _y kN/cm
1	28.8	30.0	0.90	1.00	0.39	0.12	0.09
2	28.8	10.0	1.22	1.35	-0.53	0.16	-0.12
3	30.0	28.8	1.50	1.35	0.59	0.14	0.18
4	30.0	28.8	1.85	1.67	0.72	0.17	0.22
5	28.8	30.0	0.90	1.00	0.39	0.12	0.09
6	28.8	10.0	1.22	1.35	-0.53	0.16	-0.12
7	30.0	28.8	1.50	1.35	0.59	0.14	0.18
8	30.0	28.8	1.85	1.67	0.72	0.17	0.22
9	28.8	30.0	0.90	1.00	0.39	0.12	0.09
10	28.8	10.0	1.22	1.35	-0.53	0.16	-0.12
11	30.0	28.8	1.50	1.35	0.59	0.14	0.18
12	30.0	28.8	1.85	1.67	0.72	0.17	0.22
13	28.8	30.0	0.90	1.00	0.39	0.12	0.09
14	28.8	10.0	1.22	1.35	-0.53	0.16	-0.12
15	30.0	28.8	1.50	1.35	0.59	0.14	0.18
16	30.0	28.8	1.85	1.67	0.72	0.17	0.22
17	28.8	30.0	0.90	1.00	0.39	0.12	0.09
18	28.8	10.0	1.22	1.35	-0.53	0.16	-0.12
19	30.0	28.8	1.50	1.35	0.59	0.14	0.18
20	30.0	28.8	1.85	1.67	0.72	0.17	0.22
21	28.8	30.0	0.90	1.00	0.39	0.12	0.09
22	28.8	10.0	1.22	1.35	-0.53	0.16	-0.12
23	30.0	28.8	1.50	1.35	0.59	0.14	0.18
24	30.0	28.8	1.85	1.67	0.72	0.17	0.22
25	28.8	30.0	0.90	1.00	0.39	0.12	0.09
26	28.8	10.0	1.22	1.35	-0.53	0.16	-0.12
27	30.0	28.8	1.50	1.35	0.59	0.14	0.18
28	30.0	28.8	1.85	1.67	0.72	0.17	0.22

stresses and utilizations

$$\sigma_{Pl,V} = (\sigma_x^2 + \sigma_y^2 - \sigma_x \cdot \sigma_y + 3(\tau_{xy}^2 + \tau_{xz}^2 + \tau_{yz}^2))^{0.5}$$

$$\sigma_{Rd} = f_y / \gamma_{M0}$$

$$U = \sigma_{Pl,V} / \sigma_{Rd}$$

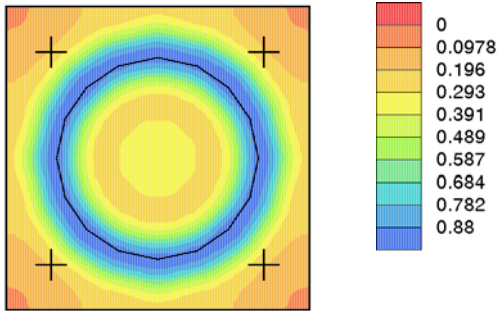
LS	x _{Fp} cm	y _{Fp} cm	σ _{Pl,V} kN/cm ²	σ _{Rd} kN/cm ²	U -	LS	x _{Fp} cm	y _{Fp} cm	σ _{Pl,V} kN/cm ²	σ _{Rd} kN/cm ²	U -
1	28.8	30.0	0.53	23.50	0.02	15	30.0	28.8	0.79	23.50	0.03
2	28.8	10.0	0.71	23.50	0.03	16	30.0	28.8	0.98	23.50	0.04
3	30.0	28.8	0.79	23.50	0.03	17	28.8	30.0	0.53	23.50	0.02
4	30.0	28.8	0.98	23.50	0.04	18	28.8	10.0	0.71	23.50	0.03
5	28.8	30.0	0.53	23.50	0.02	19	30.0	28.8	0.79	23.50	0.03
6	28.8	10.0	0.71	23.50	0.03	20	30.0	28.8	0.98	23.50	0.04
7	30.0	28.8	0.79	23.50	0.03	21	28.8	30.0	0.53	23.50	0.02
8	30.0	28.8	0.98	23.50	0.04	22	28.8	10.0	0.71	23.50	0.03
9	28.8	30.0	0.53	23.50	0.02	23	30.0	28.8	0.79	23.50	0.03
10	28.8	10.0	0.71	23.50	0.03	24	30.0	28.8	0.98	23.50	0.04
11	30.0	28.8	0.79	23.50	0.03	25	28.8	30.0	0.53	23.50	0.02
12	30.0	28.8	0.98	23.50	0.04	26	28.8	10.0	0.71	23.50	0.03
13	28.8	30.0	0.53	23.50	0.02	27	30.0	28.8	0.79	23.50	0.03
14	28.8	10.0	0.71	23.50	0.03	28	30.0	28.8	0.98	23.50	0.04

maximum utilization $U = 0.04 < 1.00$

x_{Fp}/y_{Fp} - coordinates on the base plate m_{xx}/m_{yy} - internal moments m_{xy} - torsional mom. v_x/v_y - shear force
 $\sigma_{Pl,V}$ - plastic equivalent stress σ_{Rd} - limit normal stress U - utilization

stress distribution - $\sigma_{Pl,V}$ [kN/cm²]

LS 4 (max $\sigma_{Pl,V}$)



3.4.2. concrete compression under base plate

$$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c$$

$$U = \sigma_{c,m} / f_{jd}$$

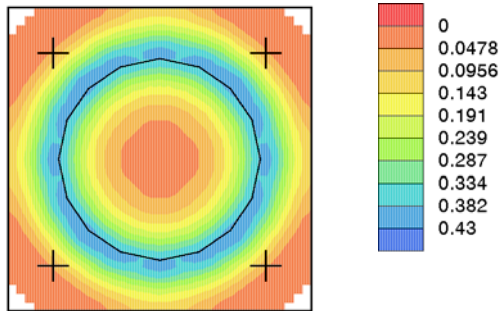
design value of concrete or mortar strength under bearing stress: $f_{jd} = 1.0 \cdot f_{cd}$

verification only for pressing areas larger than 5% of the base plate area ($A_{compr} > 80.0 \text{ cm}^2$)

LS	lim $\sigma_{c,d}$ kN/cm ²	A_{compr} cm ²	F_{compr} kN	$\sigma_{c,max}$ kN/cm ²	$\sigma_{c,m}$ kN/cm ²	f_{jd} kN/cm ²	U -
1	4.25	1537.7	152.09	0.23	0.099	1.42	0.07
2	4.25	1537.7	205.32	0.31	0.134	1.42	0.09
3	4.25	1537.7	227.09	0.35	0.148	1.42	0.10
4	4.25	1537.7	280.32	0.43	0.182	1.42	0.13
5	4.25	1537.7	152.09	0.23	0.099	1.42	0.07
6	4.25	1537.7	205.32	0.31	0.134	1.42	0.09
7	4.25	1537.7	227.09	0.35	0.148	1.42	0.10
8	4.25	1537.7	280.32	0.43	0.182	1.42	0.13
9	4.25	1537.7	152.09	0.23	0.099	1.42	0.07
10	4.25	1537.7	205.32	0.31	0.134	1.42	0.09
11	4.25	1537.7	227.09	0.35	0.148	1.42	0.10
12	4.25	1537.7	280.32	0.43	0.182	1.42	0.13
13	4.25	1537.7	152.09	0.23	0.099	1.42	0.07
14	4.25	1537.7	205.32	0.31	0.134	1.42	0.09
15	4.25	1537.7	227.09	0.35	0.148	1.42	0.10
16	4.25	1537.7	280.32	0.43	0.182	1.42	0.13
17	4.25	1537.7	152.09	0.23	0.099	1.42	0.07
18	4.25	1537.7	205.32	0.31	0.134	1.42	0.09
19	4.25	1537.7	227.09	0.35	0.148	1.42	0.10
20	4.25	1537.7	280.32	0.43	0.182	1.42	0.13
21	4.25	1537.7	152.09	0.23	0.099	1.42	0.07
22	4.25	1537.7	205.32	0.31	0.134	1.42	0.09
23	4.25	1537.7	227.09	0.35	0.148	1.42	0.10
24	4.25	1537.7	280.32	0.43	0.182	1.42	0.13
25	4.25	1537.7	152.09	0.23	0.099	1.42	0.07
26	4.25	1537.7	205.32	0.31	0.134	1.42	0.09
27	4.25	1537.7	227.09	0.35	0.148	1.42	0.10
28	4.25	1537.7	280.32	0.43	0.182	1.42	0.13

maximum utilization $U = 0.13 < 1.00$

A_{compr} - area with concrete compressions F_{compr} - res. compressive force on concrete $\sigma_{c,max}$ - max. concrete compression
 $\sigma_{c,m}$ - mean concrete compression U - utilization



3.4.3. anchor tensile forces

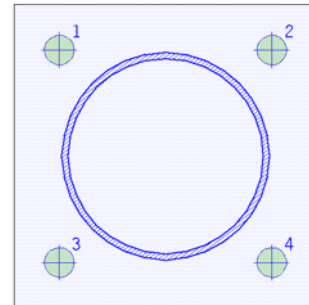
$$F_{t,Rd} = k_2 \cdot f_{ub} \cdot A_s / \gamma_{M2}$$

$$U = F_{t,Ed,max} / F_{t,Rd}$$

stress area of M22: $A_s = 3.03 \text{ cm}^2$

no countersunk bolts used: $k_2 = 0.90$

numeration



LS	$F_{t,Ed,1}$ kN	$F_{t,Ed,2}$ kN	$F_{t,Ed,3}$ kN	$F_{t,Ed,4}$ kN	$F_{t,Rd}$ kN	U_{max} -
1	---	---	---	---	174.53	0.00
2	---	---	---	---	174.53	0.00
3	---	---	---	---	174.53	0.00
4	---	---	---	---	174.53	0.00
5	---	---	---	---	174.53	0.00
6	---	---	---	---	174.53	0.00
7	---	---	---	---	174.53	0.00
8	---	---	---	---	174.53	0.00
9	---	---	---	---	174.53	0.00
10	---	---	---	---	174.53	0.00
11	---	---	---	---	174.53	0.00
12	---	---	---	---	174.53	0.00
13	---	---	---	---	174.53	0.00
14	---	---	---	---	174.53	0.00
15	---	---	---	---	174.53	0.00
16	---	---	---	---	174.53	0.00
17	---	---	---	---	174.53	0.00
18	---	---	---	---	174.53	0.00
19	---	---	---	---	174.53	0.00
20	---	---	---	---	174.53	0.00
21	---	---	---	---	174.53	0.00
22	---	---	---	---	174.53	0.00
23	---	---	---	---	174.53	0.00
24	---	---	---	---	174.53	0.00
25	---	---	---	---	174.53	0.00
26	---	---	---	---	174.53	0.00
27	---	---	---	---	174.53	0.00
28	---	---	---	---	174.53	0.00

maximum utilization $U = 0.00 < 1.00$

f_{ub} - tensile strength of bolt material $F_{t,Ed,i}$ - anchor tension force $F_{t,Rd}$ - design tension resistance of anchors

U_{max} - max. utilization

3.5. slippage verification of base plate

$$H_{res,d} = (H_{x,St,d}^2 + H_{y,St,d}^2)^{0.5}$$

$$N_{z,d} = A_{comp} \cdot \sigma_{c,m}$$

$$H_{Rd} = \mu_k / \gamma_{Rd} \cdot N_{z,d} \text{ with } \mu_k = 0.6 \text{ acc. to [2]}$$

LS	H _{res,d} kN	N _{z,d} kN	H _{Rd} kN	U -	LS	H _{res,d} kN	N _{z,d} kN	H _{Rd} kN	U -
1	5.50	152.09	76.05	0.07	15	42.36	227.09	113.55	0.37
2	7.42	205.32	102.66	0.07	16	44.11	280.32	140.16	0.31
3	16.75	227.09	113.55	0.15	17	30.50	152.09	76.05	0.40
4	18.67	280.32	140.16	0.13	18	30.90	205.32	102.66	0.30
5	18.82	152.09	76.05	0.25	19	34.36	227.09	113.55	0.30
6	19.47	205.32	102.66	0.19	20	35.33	280.32	140.16	0.25
7	24.59	227.09	113.55	0.22	21	41.50	152.09	76.05	0.55
8	25.93	280.32	140.16	0.19	22	43.42	205.32	102.66	0.42
9	27.10	152.09	76.05	0.36	23	52.75	227.09	113.55	0.46
10	29.02	205.32	102.66	0.28	24	54.67	280.32	140.16	0.39
11	38.35	227.09	113.55	0.34	25	51.21	152.09	76.05	0.67
12	40.27	280.32	140.16	0.29	26	52.78	205.32	102.66	0.51
13	32.53	152.09	76.05	0.43	27	60.68	227.09	113.55	0.53
14	34.15	205.32	102.66	0.33	28	62.36	280.32	140.16	0.44

maximum utilization $U = 0.67 < 1.00$

H_{res,d} - design value of resultant sliding force N_{z,d} - design value of compression force in sliding join
 μ_k - characteristic value of slip factor H_{Rd} - design value of sliding force U - utilization

4. design calculation of foundation slab

4.1. partial safety factors for material

design situat.	γ_c	γ_s
permanent and transient	1.50	1.15

4.2. design values of reinforced concrete design

the mobilisation of the passive earth pressure is neglected

4.2.1. factorization of load case combinations

LS	design situat.	factorization	LS	design situat.	factorization
1	permanent and transient	Lc1'+Lc5	29	permanent and transient	Lc1'+Lc19
2	permanent and transient	1.35·Lc1'+Lc5	30	permanent and transient	1.35·Lc1'+Lc19
3	permanent and transient	Lc1'+Lc6	31	permanent and transient	Lc1'+Lc20
4	permanent and transient	1.35·Lc1'+Lc6	32	permanent and transient	1.35·Lc1'+Lc20
5	permanent and transient	Lc1'+Lc7	33	permanent and transient	Lc1'+Lc21
6	permanent and transient	1.35·Lc1'+Lc7	34	permanent and transient	1.35·Lc1'+Lc21
7	permanent and transient	Lc1'+Lc8	35	permanent and transient	Lc1'+Lc22
8	permanent and transient	1.35·Lc1'+Lc8	36	permanent and transient	1.35·Lc1'+Lc22
9	permanent and transient	Lc1'+Lc9	37	permanent and transient	Lc1'+Lc23
10	permanent and transient	1.35·Lc1'+Lc9	38	permanent and transient	1.35·Lc1'+Lc23
11	permanent and transient	Lc1'+Lc10	39	permanent and transient	Lc1'+Lc24
12	permanent and transient	1.35·Lc1'+Lc10	40	permanent and transient	1.35·Lc1'+Lc24
13	permanent and transient	Lc1'+Lc11	41	permanent and transient	Lc1'+Lc25
14	permanent and transient	1.35·Lc1'+Lc11	42	permanent and transient	1.35·Lc1'+Lc25
15	permanent and transient	Lc1'+Lc12	43	permanent and transient	Lc1'+Lc26
16	permanent and transient	1.35·Lc1'+Lc12	44	permanent and transient	1.35·Lc1'+Lc26
17	permanent and transient	Lc1'+Lc13	45	permanent and transient	Lc1'+Lc27
18	permanent and transient	1.35·Lc1'+Lc13	46	permanent and transient	1.35·Lc1'+Lc27
19	permanent and transient	Lc1'+Lc14	47	permanent and transient	Lc1'+Lc28
20	permanent and transient	1.35·Lc1'+Lc14	48	permanent and transient	1.35·Lc1'+Lc28
21	permanent and transient	Lc1'+Lc15	49	permanent and transient	Lc1'+Lc29
22	permanent and transient	1.35·Lc1'+Lc15	50	permanent and transient	1.35·Lc1'+Lc29
23	permanent and transient	Lc1'+Lc16	51	permanent and transient	Lc1'+Lc30
24	permanent and transient	1.35·Lc1'+Lc16	52	permanent and transient	1.35·Lc1'+Lc30
25	permanent and transient	Lc1'+Lc17	53	permanent and transient	Lc1'+Lc31
26	permanent and transient	1.35·Lc1'+Lc17	54	permanent and transient	1.35·Lc1'+Lc31
27	permanent and transient	Lc1'+Lc18	55	permanent and transient	Lc1'+Lc32
28	permanent and transient	1.35·Lc1'+Lc18	56	permanent and transient	1.35·Lc1'+Lc32

*) without consideration of column load

4.2.2. column load

LS	N _{st,d} kN	H _{x,St,d} kN	H _{y,St,d} kN	M _{x,St,d} kNm	M _{y,St,d} kNm	LS	N _{st,d} kN	H _{x,St,d} kN	H _{y,St,d} kN	M _{x,St,d} kNm	M _{y,St,d} kNm
1	152.09	5.50	-0.00	-0.00	0.00	15	280.32	18.67	18.00	-0.00	0.00
2	152.09	5.50	-0.00	-0.00	0.00	16	280.32	18.67	18.00	-0.00	0.00
3	205.32	7.42	-0.00	-0.00	-0.00	17	152.09	27.10	-0.00	0.00	-0.00
4	205.32	7.42	-0.00	-0.00	-0.00	18	152.09	27.10	-0.00	0.00	-0.00
5	227.09	16.75	-0.00	0.00	-0.00	19	205.32	29.02	-0.00	-0.00	-0.00
6	227.09	16.75	-0.00	0.00	-0.00	20	205.32	29.02	-0.00	-0.00	-0.00
7	280.32	18.67	-0.00	0.00	0.00	21	227.09	38.35	-0.00	0.00	0.00
8	280.32	18.67	-0.00	0.00	0.00	22	227.09	38.35	-0.00	0.00	0.00
9	152.09	5.50	18.00	0.00	0.00	23	280.32	40.27	-0.00	0.00	-0.00
10	152.09	5.50	18.00	0.00	0.00	24	280.32	40.27	-0.00	0.00	-0.00
11	205.32	7.42	18.00	0.00	-0.00	25	152.09	27.10	18.00	0.00	-0.00
12	205.32	7.42	18.00	0.00	-0.00	26	152.09	27.10	18.00	0.00	-0.00
13	227.09	16.75	18.00	-0.00	-0.00	27	205.32	29.02	18.00	0.00	-0.00
14	227.09	16.75	18.00	-0.00	-0.00	28	205.32	29.02	18.00	0.00	-0.00

LS	N _{st,d} kN	H _{x,St,d} kN	H _{y,St,d} kN	M _{x,St,d} kNm	M _{y,St,d} kNm	LS	N _{st,d} kN	H _{x,St,d} kN	H _{y,St,d} kN	M _{x,St,d} kNm	M _{y,St,d} kNm
29	227.09	38.35	18.00	-0.00	0.00	43	205.32	43.42	-0.00	-0.00	-0.00
30	227.09	38.35	18.00	-0.00	0.00	44	205.32	43.42	-0.00	-0.00	-0.00
31	280.32	40.27	18.00	-0.00	-0.00	45	227.09	52.75	-0.00	-0.00	0.00
32	280.32	40.27	18.00	-0.00	-0.00	46	227.09	52.75	-0.00	-0.00	0.00
33	152.09	5.50	30.00	-0.00	0.00	47	280.32	54.67	-0.00	0.00	-0.00
34	152.09	5.50	30.00	-0.00	0.00	48	280.32	54.67	-0.00	0.00	-0.00
35	205.32	7.42	30.00	0.00	-0.00	49	152.09	41.50	30.00	-0.00	-0.00
36	205.32	7.42	30.00	0.00	-0.00	50	152.09	41.50	30.00	-0.00	-0.00
37	227.09	16.75	30.00	-0.00	-0.00	51	205.32	43.42	30.00	0.00	-0.00
38	227.09	16.75	30.00	-0.00	-0.00	52	205.32	43.42	30.00	0.00	-0.00
39	280.32	18.67	30.00	-0.00	0.00	53	227.09	52.75	30.00	-0.00	0.00
40	280.32	18.67	30.00	-0.00	0.00	54	227.09	52.75	30.00	-0.00	0.00
41	152.09	41.50	-0.00	-0.00	-0.00	55	280.32	54.67	30.00	-0.00	-0.00
42	152.09	41.50	-0.00	-0.00	-0.00	56	280.32	54.67	30.00	-0.00	-0.00

4.3. check, if slab can be designed non-reinforced

4.3.1. minimum foundation height

$$h_{\min} = a \cdot (3 \cdot \sigma_{bm} / f_{ctd})^{0.5} / 0.85 \geq a \quad \text{with } f_{ctd} = 0.85 \cdot f_{ctk} \cdot 0.05 / \gamma_c$$

determination of minimum foundation height acc. to [3], section 12.9.3:

distance between column and slab edge $a = 30.0$ cm

existing foundation height $h_{\text{exist}} = 60.0$ cm

LS	σ_{bm} kN/m ²	f_{ctd} N/mm ²	$h_{\min}$ cm	$h_{\min} / h_{\text{exist}}$ -	LS	σ_{bm} kN/m ²	f_{ctd} N/mm ²	$h_{\min}$ cm	$h_{\min} / h_{\text{exist}}$ -
1	152.09	1.03	30.0	0.50	29	227.09	1.03	30.0	0.50
2	152.09	1.03	30.0	0.50	30	227.09	1.03	30.0	0.50
3	205.32	1.03	30.0	0.50	31	280.32	1.03	31.9	0.53
4	205.32	1.03	30.0	0.50	32	280.32	1.03	31.9	0.53
5	227.09	1.03	30.0	0.50	33	152.09	1.03	30.0	0.50
6	227.09	1.03	30.0	0.50	34	152.09	1.03	30.0	0.50
7	280.32	1.03	31.9	0.53	35	205.32	1.03	30.0	0.50
8	280.32	1.03	31.9	0.53	36	205.32	1.03	30.0	0.50
9	152.09	1.03	30.0	0.50	37	227.09	1.03	30.0	0.50
10	152.09	1.03	30.0	0.50	38	227.09	1.03	30.0	0.50
11	205.32	1.03	30.0	0.50	39	280.32	1.03	31.9	0.53
12	205.32	1.03	30.0	0.50	40	280.32	1.03	31.9	0.53
13	227.09	1.03	30.0	0.50	41	152.09	1.03	30.0	0.50
14	227.09	1.03	30.0	0.50	42	152.09	1.03	30.0	0.50
15	280.32	1.03	31.9	0.53	43	205.32	1.03	30.0	0.50
16	280.32	1.03	31.9	0.53	44	205.32	1.03	30.0	0.50
17	152.09	1.03	30.0	0.50	45	227.09	1.03	30.0	0.50
18	152.09	1.03	30.0	0.50	46	227.09	1.03	30.0	0.50
19	205.32	1.03	30.0	0.50	47	280.32	1.03	31.9	0.53
20	205.32	1.03	30.0	0.50	48	280.32	1.03	31.9	0.53
21	227.09	1.03	30.0	0.50	49	152.09	1.03	30.0	0.50
22	227.09	1.03	30.0	0.50	50	152.09	1.03	30.0	0.50
23	280.32	1.03	31.9	0.53	51	205.32	1.03	30.0	0.50
24	280.32	1.03	31.9	0.53	52	205.32	1.03	30.0	0.50
25	152.09	1.03	30.0	0.50	53	227.09	1.03	30.0	0.50
26	152.09	1.03	30.0	0.50	54	227.09	1.03	30.0	0.50
27	205.32	1.03	30.0	0.50	55	280.32	1.03	31.9	0.53
28	205.32	1.03	30.0	0.50	56	280.32	1.03	31.9	0.53

$(h_{\min} / h_{\text{exist}})_{\max} = 0.53 < 1.0 \Rightarrow$ existing foundation height sufficient

4.3.2. permissible partial area load

determination of the max. resisting column load acc. to [3], section 6.7:

$$F_{Rdu} = A_{c1} \cdot f_{cd} \cdot (A_{c2} / A_{c1})^{0.5} \leq 3.0 \cdot f_{cd} \cdot A_{c1}$$

existing loading area (column cross section) $A_{c1} = 0.160$ m²

LS	A_{c2} m ²	F_{Rdu} kN	N_{st} / F_{Rdu} -	LS	A_{c2} m ²	F_{Rdu} kN	N_{st} / F_{Rdu} -
1	1.000	5666.67	0.03	12	1.000	5666.67	0.04
2	1.000	5666.67	0.03	13	1.000	5666.67	0.04
3	1.000	5666.67	0.04	14	1.000	5666.67	0.04
4	1.000	5666.67	0.04	15	1.000	5666.67	0.05
5	1.000	5666.67	0.04	16	1.000	5666.67	0.05
6	1.000	5666.67	0.04	17	1.000	5666.67	0.03
7	1.000	5666.67	0.05	18	1.000	5666.67	0.03
8	1.000	5666.67	0.05	19	1.000	5666.67	0.04
9	1.000	5666.67	0.03	20	1.000	5666.67	0.04
10	1.000	5666.67	0.03	21	1.000	5666.67	0.04
11	1.000	5666.67	0.04	22	1.000	5666.67	0.04

LS	Ac2 m ²	FRdu kN	Nst /FRdu -	LS	Ac2 m ²	FRdu kN	Nst /FRdu -
23	1.000	5666.67	0.05	40	1.000	5666.67	0.05
24	1.000	5666.67	0.05	41	1.000	5666.67	0.03
25	1.000	5666.67	0.03	42	1.000	5666.67	0.03
26	1.000	5666.67	0.03	43	1.000	5666.67	0.04
27	1.000	5666.67	0.04	44	1.000	5666.67	0.04
28	1.000	5666.67	0.04	45	1.000	5666.67	0.04
29	1.000	5666.67	0.04	46	1.000	5666.67	0.04
30	1.000	5666.67	0.04	47	1.000	5666.67	0.05
31	1.000	5666.67	0.05	48	1.000	5666.67	0.05
32	1.000	5666.67	0.05	49	1.000	5666.67	0.03
33	1.000	5666.67	0.03	50	1.000	5666.67	0.03
34	1.000	5666.67	0.03	51	1.000	5666.67	0.04
35	1.000	5666.67	0.04	52	1.000	5666.67	0.04
36	1.000	5666.67	0.04	53	1.000	5666.67	0.04
37	1.000	5666.67	0.04	54	1.000	5666.67	0.04
38	1.000	5666.67	0.04	55	1.000	5666.67	0.05
39	1.000	5666.67	0.05	56	1.000	5666.67	0.05

$(N_{st}/F_{Rdu})_{max} = 0.05 < 1.0 \Rightarrow$ permissible partial area load complied with.

$\Rightarrow$ the foundation slab may be unreinforced.

σ_{Bm} - mean ground pressure below slab f_{ctd} - design value of concrete tensile strength for non-reinforced members
 F_{Rdu} - perm partial area load A_{c1} - loading area A_{c2} - rech. distribution area

5. external stability - verification of design resistance (ULS)

5.1. partial safety factors on load side

acc. to [4] table A 2.1

5.2. partial safety factors on res1istance side

acc. to [4] tables A 2.2 and A 2.3

5.3. design values overturning (EQU)

the mobilisation of the passive earth pressure is neglected

5.3.1. factorization of load case combinations

LS	design situat.	factorization	LS	design situat.	factorization
1	BS-P	0.9 · Lc1	15	BS-P	0.9 · Lc1+1.5 · Lc2+0.6 · 1.5 · (Lc3+Lc4)
2	BS-P	1.1 · Lc1	16	BS-P	1.1 · Lc1+1.5 · Lc2+0.6 · 1.5 · (Lc3+Lc4)
3	BS-P	0.9 · Lc1+1.5 · Lc2	17	BS-P	0.9 · Lc1+1.5 · Lc3
4	BS-P	1.1 · Lc1+1.5 · Lc2	18	BS-P	1.1 · Lc1+1.5 · Lc3
5	BS-P	0.9 · Lc1+0.6 · 1.5 · Lc3	19	BS-P	0.9 · Lc1+1.5 · (Lc2+Lc3)
6	BS-P	1.1 · Lc1+0.6 · 1.5 · Lc3	20	BS-P	1.1 · Lc1+1.5 · (Lc2+Lc3)
7	BS-P	0.9 · Lc1+1.5 · Lc2+0.6 · 1.5 · Lc3	21	BS-P	0.9 · Lc1+1.5 · Lc4
8	BS-P	1.1 · Lc1+1.5 · Lc2+0.6 · 1.5 · Lc3	22	BS-P	1.1 · Lc1+1.5 · Lc4
9	BS-P	0.9 · Lc1+0.6 · 1.5 · Lc4	23	BS-P	0.9 · Lc1+1.5 · (Lc2+Lc4)
10	BS-P	1.1 · Lc1+0.6 · 1.5 · Lc4	24	BS-P	1.1 · Lc1+1.5 · (Lc2+Lc4)
11	BS-P	0.9 · Lc1+1.5 · Lc2+0.6 · 1.5 · Lc4	25	BS-P	0.9 · Lc1+1.5 · (Lc3+Lc4)
12	BS-P	1.1 · Lc1+1.5 · Lc2+0.6 · 1.5 · Lc4	26	BS-P	1.1 · Lc1+1.5 · (Lc3+Lc4)
13	BS-P	0.9 · Lc1+0.6 · 1.5 · (Lc3+Lc4)	27	BS-P	0.9 · Lc1+1.5 · (Lc2+Lc3+Lc4)
14	BS-P	1.1 · Lc1+0.6 · 1.5 · (Lc3+Lc4)	28	BS-P	1.1 · Lc1+1.5 · (Lc2+Lc3+Lc4)

5.3.2. column load

LS	Nst,d kN	Hx,st,d kN	Hy,st,d kN	Mx,st,d kNm	My,st,d kNm	LS	Nst,d kN	Hx,st,d kN	Hy,st,d kN	Mx,st,d kNm	My,st,d kNm
1	136.88	4.95	-0.00	-0.00	0.00	15	211.88	37.80	18.00	0.00	-0.00
2	167.30	6.05	-0.00	-0.00	0.00	16	242.30	38.90	18.00	0.00	-0.00
3	211.88	16.20	-0.00	0.00	-0.00	17	136.88	4.95	30.00	0.00	0.00
4	242.30	17.30	-0.00	0.00	-0.00	18	167.30	6.05	30.00	0.00	0.00
5	136.88	4.95	18.00	0.00	0.00	19	211.88	16.20	30.00	0.00	-0.00
6	167.30	6.05	18.00	0.00	0.00	20	242.30	17.30	30.00	0.00	-0.00
7	211.88	16.20	18.00	0.00	-0.00	21	136.88	40.95	-0.00	-0.00	-0.00
8	242.30	17.30	18.00	0.00	-0.00	22	167.30	42.05	-0.00	-0.00	-0.00
9	136.88	26.55	-0.00	-0.00	-0.00	23	211.88	52.20	-0.00	0.00	-0.00
10	167.30	27.65	-0.00	-0.00	-0.00	24	242.30	53.30	-0.00	0.00	-0.00
11	211.88	37.80	-0.00	0.00	-0.00	25	136.88	40.95	30.00	0.00	-0.00
12	242.30	38.90	-0.00	0.00	-0.00	26	167.30	42.05	30.00	0.00	-0.00
13	136.88	26.55	18.00	0.00	-0.00	27	211.88	52.20	30.00	0.00	-0.00
14	167.30	27.65	18.00	0.00	-0.00	28	242.30	53.30	30.00	0.00	-0.00

5.4. verification against overturning (EQU)

LS	decisive edge	Mdst kNm	Mstb kNm	μ -	LS	decisive edge	Mdst kNm	Mstb kNm	μ -
1	---	0.00	79.60	0.00	15	y (right)	12.96	104.41	0.12
2	---	0.00	97.29	0.00	16	y (right)	12.96	120.78	0.11
3	---	0.00	123.85	0.00	17	x (bottom)	18.00	76.63	0.23
4	---	0.00	141.54	0.00	18	x (bottom)	18.00	93.66	0.19
5	x (bottom)	10.80	76.63	0.14	19	x (bottom)	18.00	114.13	0.16
6	x (bottom)	10.80	93.66	0.12	20	x (bottom)	18.00	131.16	0.14
7	x (bottom)	10.80	114.13	0.09	21	y (right)	21.60	73.66	0.29
8	x (bottom)	10.80	131.16	0.08	22	y (right)	21.60	90.03	0.24
9	y (right)	12.96	73.66	0.18	23	y (right)	21.60	104.41	0.21
10	y (right)	12.96	90.03	0.14	24	y (right)	21.60	120.78	0.18
11	y (right)	12.96	104.41	0.12	25	y (right)	21.60	73.66	0.29
12	y (right)	12.96	120.78	0.11	26	y (right)	21.60	90.03	0.24
13	y (right)	12.96	73.66	0.18	27	y (right)	21.60	104.41	0.21
14	y (right)	12.96	90.03	0.14	28	y (right)	21.60	120.78	0.18

$\mu_{\max} = 0.29 < 1.0 \Rightarrow$ verification against overturning successful

5.5. design values base failure (GEO-2)

the assumed mobilised passive earth pressure is $e_{phg,mob} = 0.50 \cdot e_{phg}$.

5.5.1. factorization of load case combinations

LS	design situat.	factorization	LS	design situat.	factorization
1	BS-P	Lc1	15	BS-P	Lc1+1.5·Lc2+0.6·1.5·(Lc3+Lc4)
2	BS-P	1.35·Lc1	16	BS-P	1.35·Lc1+1.5·Lc2+0.6·1.5·(Lc3+Lc4)
3	BS-P	Lc1+1.5·Lc2	17	BS-P	Lc1+1.5·Lc3
4	BS-P	1.35·Lc1+1.5·Lc2	18	BS-P	1.35·Lc1+1.5·Lc3
5	BS-P	Lc1+0.6·1.5·Lc3	19	BS-P	Lc1+1.5·(Lc2+Lc3)
6	BS-P	1.35·Lc1+0.6·1.5·Lc3	20	BS-P	1.35·Lc1+1.5·(Lc2+Lc3)
7	BS-P	Lc1+1.5·Lc2+0.6·1.5·Lc3	21	BS-P	Lc1+1.5·Lc4
8	BS-P	1.35·Lc1+1.5·Lc2+0.6·1.5·Lc3	22	BS-P	1.35·Lc1+1.5·Lc4
9	BS-P	Lc1+0.6·1.5·Lc4	23	BS-P	Lc1+1.5·(Lc2+Lc4)
10	BS-P	1.35·Lc1+0.6·1.5·Lc4	24	BS-P	1.35·Lc1+1.5·(Lc2+Lc4)
11	BS-P	Lc1+1.5·Lc2+0.6·1.5·Lc4	25	BS-P	Lc1+1.5·(Lc3+Lc4)
12	BS-P	1.35·Lc1+1.5·Lc2+0.6·1.5·Lc4	26	BS-P	1.35·Lc1+1.5·(Lc3+Lc4)
13	BS-P	Lc1+0.6·1.5·(Lc3+Lc4)	27	BS-P	Lc1+1.5·(Lc2+Lc3+Lc4)
14	BS-P	1.35·Lc1+0.6·1.5·(Lc3+Lc4)	28	BS-P	1.35·Lc1+1.5·(Lc2+Lc3+Lc4)

5.5.2. column load

LS	Nst,d kN	Hx,St,d kN	Hy,St,d kN	Mx,St,d kNm	My,St,d kNm	LS	Nst,d kN	Hx,St,d kN	Hy,St,d kN	Mx,St,d kNm	My,St,d kNm
1	152.09	5.50	-0.00	-0.00	0.00	15	227.09	38.35	18.00	0.00	-0.00
2	205.32	7.43	-0.00	-0.00	0.00	16	280.32	40.28	18.00	0.00	-0.00
3	227.09	16.75	-0.00	0.00	-0.00	17	152.09	5.50	30.00	0.00	0.00
4	280.32	18.67	-0.00	0.00	-0.00	18	205.32	7.43	30.00	0.00	0.00
5	152.09	5.50	18.00	0.00	0.00	19	227.09	16.75	30.00	0.00	-0.00
6	205.32	7.43	18.00	0.00	0.00	20	280.32	18.67	30.00	0.00	-0.00
7	227.09	16.75	18.00	0.00	-0.00	21	152.09	41.50	-0.00	-0.00	-0.00
8	280.32	18.67	18.00	0.00	-0.00	22	205.32	43.42	-0.00	-0.00	-0.00
9	152.09	27.10	-0.00	-0.00	-0.00	23	227.09	52.75	-0.00	0.00	-0.00
10	205.32	29.03	-0.00	-0.00	-0.00	24	280.32	54.67	-0.00	0.00	-0.00
11	227.09	38.35	-0.00	0.00	-0.00	25	152.09	41.50	30.00	0.00	-0.00
12	280.32	40.28	-0.00	0.00	-0.00	26	205.32	43.42	30.00	0.00	-0.00
13	152.09	27.10	18.00	0.00	-0.00	27	227.09	52.75	30.00	0.00	-0.00
14	205.32	29.03	18.00	0.00	-0.00	28	280.32	54.67	30.00	0.00	-0.00

associated characteristic values

LS	Nst,k kN	Hx,St,k kN	Hy,St,k kN	Mx,St,k kNm	My,St,k kNm	LS	Nst,k kN	Hx,St,k kN	Hy,St,k kN	Mx,St,k kNm	My,St,k kNm
1	152.09	5.50	-0.00	-0.00	0.00	15	202.09	37.00	20.00	0.00	-0.00
2	152.09	5.50	-0.00	-0.00	0.00	16	202.09	37.00	20.00	0.00	-0.00
3	202.09	13.00	-0.00	-0.00	-0.00	17	152.09	5.50	20.00	0.00	0.00
4	202.09	13.00	-0.00	-0.00	-0.00	18	152.09	5.50	20.00	0.00	0.00
5	152.09	5.50	20.00	0.00	0.00	19	202.09	13.00	20.00	0.00	-0.00
6	152.09	5.50	20.00	0.00	0.00	20	202.09	13.00	20.00	0.00	-0.00
7	202.09	13.00	20.00	0.00	-0.00	21	152.09	29.50	-0.00	-0.00	-0.00
8	202.09	13.00	20.00	0.00	-0.00	22	152.09	29.50	-0.00	-0.00	-0.00
9	152.09	29.50	-0.00	-0.00	-0.00	23	202.09	37.00	-0.00	-0.00	-0.00
10	152.09	29.50	-0.00	-0.00	-0.00	24	202.09	37.00	-0.00	-0.00	-0.00
11	202.09	37.00	-0.00	-0.00	-0.00	25	152.09	29.50	20.00	0.00	-0.00
12	202.09	37.00	-0.00	-0.00	-0.00	26	152.09	29.50	20.00	0.00	-0.00
13	152.09	29.50	20.00	0.00	-0.00	27	202.09	37.00	20.00	0.00	-0.00
14	152.09	29.50	20.00	0.00	-0.00	28	202.09	37.00	20.00	0.00	-0.00

5.6. verification of safety against base failure

5.6.1. loading and substituting dimensions

LS	N _{0,k} kN	M _{0,x,k} kNm	M _{0,y,k} kNm	a' m	b' m	H _{a',k} kN	H _{b',k} kN
1	170.28	-0.00	-1.98	1.00	0.98	0.00	0.00
2	170.28	-0.00	-1.98	1.00	0.98	0.00	0.00
3	220.28	-0.00	-4.68	1.00	0.96	0.00	0.00
4	220.28	-0.00	-4.68	1.00	0.96	0.00	0.00
5	170.28	7.20	-1.98	0.98	0.92	0.00	0.00
6	170.28	7.20	-1.98	0.98	0.92	0.00	0.00
7	220.28	7.20	-4.68	0.96	0.93	0.00	0.00
8	220.28	7.20	-4.68	0.96	0.93	0.00	0.00
9	170.28	-0.00	-12.81	1.00	0.85	0.00	9.12
10	170.28	-0.00	-12.81	1.00	0.85	0.00	9.12
11	220.28	-0.00	-17.31	1.00	0.84	0.00	16.62
12	220.28	-0.00	-17.31	1.00	0.84	0.00	16.62
13	170.28	7.20	-12.81	0.92	0.85	0.00	9.12
14	170.28	7.20	-12.81	0.92	0.85	0.00	9.12
15	220.28	7.20	-17.31	0.93	0.84	0.00	16.62
16	220.28	7.20	-17.31	0.93	0.84	0.00	16.62
17	170.28	7.20	-1.98	0.98	0.92	0.00	0.00
18	170.28	7.20	-1.98	0.98	0.92	0.00	0.00
19	220.28	7.20	-4.68	0.96	0.93	0.00	0.00
20	220.28	7.20	-4.68	0.96	0.93	0.00	0.00
21	170.28	-0.00	-12.81	1.00	0.85	0.00	9.12
22	170.28	-0.00	-12.81	1.00	0.85	0.00	9.12
23	220.28	-0.00	-17.31	1.00	0.84	0.00	16.62
24	220.28	-0.00	-17.31	1.00	0.84	0.00	16.62
25	170.28	7.20	-12.81	0.92	0.85	0.00	9.12
26	170.28	7.20	-12.81	0.92	0.85	0.00	9.12
27	220.28	7.20	-17.31	0.93	0.84	0.00	16.62
28	220.28	7.20	-17.31	0.93	0.84	0.00	16.62

5.6.2. decisive soil parameters

determination of the decisive values by method of weighted average

values beyond the base to top edge of soil: γ_1 , ϕ_1 , c_1

values below the base up to depth (d_s) of the sliding clod: γ_2 , ϕ_2 , c_2

LS	γ_1 kN/m ³	ϕ_1 °	c_1 kN/m ²	d_s m	γ_2 kN/m ³	ϕ_2 °	c_2 kN/m ²
1	19.00	32.50	---	1.69	19.00	32.50	---
2	19.00	32.50	---	1.69	19.00	32.50	---
3	19.00	32.50	---	1.66	19.00	32.50	---
4	19.00	32.50	---	1.66	19.00	32.50	---
5	19.00	32.50	---	1.59	19.00	32.50	---
6	19.00	32.50	---	1.59	19.00	32.50	---
7	19.00	32.50	---	1.62	19.00	32.50	---
8	19.00	32.50	---	1.62	19.00	32.50	---
9	19.00	32.50	---	1.34	19.00	32.50	---
10	19.00	32.50	---	1.34	19.00	32.50	---
11	19.00	32.50	---	1.27	19.00	32.50	---
12	19.00	32.50	---	1.27	19.00	32.50	---
13	19.00	32.50	---	1.34	19.00	32.50	---
14	19.00	32.50	---	1.34	19.00	32.50	---
15	19.00	32.50	---	1.27	19.00	32.50	---
16	19.00	32.50	---	1.27	19.00	32.50	---
17	19.00	32.50	---	1.59	19.00	32.50	---
18	19.00	32.50	---	1.59	19.00	32.50	---
19	19.00	32.50	---	1.62	19.00	32.50	---
20	19.00	32.50	---	1.62	19.00	32.50	---
21	19.00	32.50	---	1.34	19.00	32.50	---
22	19.00	32.50	---	1.34	19.00	32.50	---
23	19.00	32.50	---	1.27	19.00	32.50	---
24	19.00	32.50	---	1.27	19.00	32.50	---
25	19.00	32.50	---	1.34	19.00	32.50	---
26	19.00	32.50	---	1.34	19.00	32.50	---
27	19.00	32.50	---	1.27	19.00	32.50	---
28	19.00	32.50	---	1.27	19.00	32.50	---

5.6.3. values of design resistance, shape, load inclination and depth

basic values of design resistance values N_{b0}, N_{d0}, N_{c0} acc. to [5]

shape factors v_b, v_d, v_c acc. to [5], tab.2

load inclination factors i_b, i_d, i_c acc. to [5], tab.3

LS	N_{b0}	N_{d0}	N_{c0}	v_b	v_d	v_c	i_b	i_d	i_c
	-	-	-	-	-	-	-	-	-
1	15.03	24.58	---	0.707	1.525	---	1.000	1.000	---
2	15.03	24.58	---	0.707	1.525	---	1.000	1.000	---
3	15.03	24.58	---	0.713	1.514	---	1.000	1.000	---
4	15.03	24.58	---	0.713	1.514	---	1.000	1.000	---
5	15.03	24.58	---	0.719	1.504	---	1.000	1.000	---
6	15.03	24.58	---	0.719	1.504	---	1.000	1.000	---
7	15.03	24.58	---	0.707	1.524	---	1.000	1.000	---
8	15.03	24.58	---	0.707	1.524	---	1.000	1.000	---
9	15.03	24.58	---	0.745	1.456	---	0.870	0.919	---
10	15.03	24.58	---	0.745	1.456	---	0.870	0.919	---
11	15.03	24.58	---	0.747	1.453	---	0.819	0.886	---
12	15.03	24.58	---	0.747	1.453	---	0.819	0.886	---
13	15.03	24.58	---	0.722	1.499	---	0.871	0.920	---
14	15.03	24.58	---	0.722	1.499	---	0.871	0.920	---
15	15.03	24.58	---	0.729	1.485	---	0.820	0.887	---
16	15.03	24.58	---	0.729	1.485	---	0.820	0.887	---
17	15.03	24.58	---	0.719	1.504	---	1.000	1.000	---
18	15.03	24.58	---	0.719	1.504	---	1.000	1.000	---
19	15.03	24.58	---	0.707	1.524	---	1.000	1.000	---
20	15.03	24.58	---	0.707	1.524	---	1.000	1.000	---
21	15.03	24.58	---	0.745	1.456	---	0.870	0.919	---
22	15.03	24.58	---	0.745	1.456	---	0.870	0.919	---
23	15.03	24.58	---	0.747	1.453	---	0.819	0.886	---
24	15.03	24.58	---	0.747	1.453	---	0.819	0.886	---
25	15.03	24.58	---	0.722	1.499	---	0.871	0.920	---
26	15.03	24.58	---	0.722	1.499	---	0.871	0.920	---
27	15.03	24.58	---	0.729	1.485	---	0.820	0.887	---
28	15.03	24.58	---	0.729	1.485	---	0.820	0.887	---

5.6.4. ultimate load and allowable load

characteristic design bearing capacity $R_{n,k} = a \cdot b' \cdot (\gamma_2 b' \cdot N_{b0} \cdot v_b \cdot i_b + \gamma_1 t \cdot N_{d0} \cdot v_d \cdot i_d + c_2 \cdot N_{c0} \cdot v_c \cdot i_c)$

design value of resistance $R_{n,d} = R_{n,k} / \gamma_{R,v}$

the degree of utilization is $\mu = N_d / R_{n,d}$

LS	$R_{n,k}$ kN	$\gamma_{R,v}$ -	$R_{n,d}$ kN	N_d kN	μ -	LS	$R_{n,k}$ kN	$\gamma_{R,v}$ -	$R_{n,d}$ kN	N_d kN	μ -
1	749.09	1.40	535.07	170.28	0.32	15	501.14	1.40	357.95	245.28	0.69
2	749.09	1.40	535.07	229.88	0.43	16	501.14	1.40	357.95	304.88	0.85
3	728.44	1.40	520.31	245.28	0.47	17	670.36	1.40	478.83	170.28	0.36
4	728.44	1.40	520.31	304.88	0.59	18	670.36	1.40	478.83	229.88	0.48
5	670.36	1.40	478.83	170.28	0.36	19	678.66	1.40	484.76	245.28	0.51
6	670.36	1.40	478.83	229.88	0.48	20	678.66	1.40	484.76	304.88	0.63
7	678.66	1.40	484.76	245.28	0.51	21	558.30	1.40	398.79	170.28	0.43
8	678.66	1.40	484.76	304.88	0.63	22	558.30	1.40	398.79	229.88	0.58
9	558.30	1.40	398.79	170.28	0.43	23	529.58	1.40	378.27	245.28	0.65
10	558.30	1.40	398.79	229.88	0.58	24	529.58	1.40	378.27	304.88	0.81
11	529.58	1.40	378.27	245.28	0.65	25	519.11	1.40	370.79	170.28	0.46
12	529.58	1.40	378.27	304.88	0.81	26	519.11	1.40	370.79	229.88	0.62
13	519.11	1.40	370.79	170.28	0.46	27	501.14	1.40	357.95	245.28	0.69
14	519.11	1.40	370.79	229.88	0.62	28	501.14	1.40	357.95	304.88	0.85

$\mu_{\max} = 0.85 < 1.0 \Rightarrow$ design bearing capacity sufficient

5.7. design values slippage (GEO-2)

the assumed mobilised passive earth pressure is $e_{phg,mob} = 1.00 \cdot e_{phg}$.

design values of applied loads see base failure.

5.8. verification of safety against sliding

slip resistance in case of consolidated soil $R_{t,k} = N_{0,k} \cdot \tan(\delta_s)$

design value of slip resistance $R_{t,d} = R_{t,k} / \gamma_{R,h}$

design value of mobilised passive earth pressure $E_{p,d} = E_{p,k,mob} / \gamma_{R,e}$

the degree of utilization is $\mu = (R_{t,d} + E_{p,d}) / H_{Res,d}$

angle of base friction (for rough base area) $\delta_s = 32.5^\circ$

LS	N _{0,k} kN	R _{t,k} kN	γ _{R,h} -	γ _{R,e} -	R _{t,d} kN	E _{p,d} kN	H _{Res,d} kN	μ -
1	170.28	108.48	1.10	1.40	98.62	3.93	5.50	0.05
2	170.28	108.48	1.10	1.40	98.62	5.30	7.43	0.07
3	220.28	140.34	1.10	1.40	127.58	11.96	16.75	0.12
4	220.28	140.34	1.10	1.40	127.58	13.34	18.67	0.13
5	170.28	108.48	1.10	1.40	98.62	13.44	18.82	0.17
6	170.28	108.48	1.10	1.40	98.62	13.91	19.47	0.17
7	220.28	140.34	1.10	1.40	127.58	17.56	24.59	0.17
8	220.28	140.34	1.10	1.40	127.58	18.53	25.94	0.18
9	170.28	108.48	1.10	1.40	98.62	19.36	27.10	0.23
10	170.28	108.48	1.10	1.40	98.62	20.73	29.03	0.24
11	220.28	140.34	1.10	1.40	127.58	27.39	38.35	0.25
12	220.28	140.34	1.10	1.40	127.58	28.77	40.28	0.26
13	170.28	108.48	1.10	1.40	98.62	23.24	32.53	0.27
14	170.28	108.48	1.10	1.40	98.62	24.40	34.15	0.28
15	220.28	140.34	1.10	1.40	127.58	30.26	42.36	0.27
16	220.28	140.34	1.10	1.40	127.58	31.51	44.11	0.28
17	170.28	108.48	1.10	1.40	98.62	21.79	30.50	0.25
18	170.28	108.48	1.10	1.40	98.62	22.08	30.91	0.26
19	220.28	140.34	1.10	1.40	127.58	24.54	34.36	0.23
20	220.28	140.34	1.10	1.40	127.58	25.24	35.34	0.23
21	170.28	108.48	1.10	1.40	98.62	29.12	41.50	0.32
22	170.28	108.48	1.10	1.40	98.62	29.12	43.42	0.34
23	220.28	140.34	1.10	1.40	127.58	29.12	52.75	0.34
24	220.28	140.34	1.10	1.40	127.58	29.12	54.67	0.35
25	170.28	108.48	1.10	1.40	98.62	36.58	51.21	0.38
26	170.28	108.48	1.10	1.40	98.62	37.62	52.78	0.39
27	220.28	140.34	1.10	1.40	127.58	36.98	60.68	0.37
28	220.28	140.34	1.10	1.40	127.58	36.84	62.36	0.38

$\mu_{\max} = 0.39 < 1.0 \Rightarrow$ slip resistance sufficient

6. external stability - verification of serviceability

6.1. design values limitation of gapping joint under permanent load

the mobilisation of the passive earth pressure is neglected

6.1.1. factorization of load case combinations

LS	factorization
1	Lc1

6.1.2. column load

LS	N _{St,d} kN	H _{x,St,d} kN	H _{y,St,d} kN	M _{x,St,d} kNm	M _{y,St,d} kNm
1	152.09	5.50	-0.00	-0.00	0.00

6.2. limitation of gapping joint under permanent load

int. f. and mom. in centroid of foundation base: $N_{0,k} = 170.28$ kN

$M_{0,x,k} = -0.00$ kNm

$M_{0,y,k} = -3.30$ kNm

resultant eccentricity: $e_x = -0.02$ m
 $e_y = -0.00$ m

$e_x/b_x + e_y/b_y = 0.02 < 1/6$

$\Rightarrow$ the resultant is located in the 1. core area

sc. no emerge of a gapping joint due to permanent load.

6.3. design values limitation of gapping joint under total load

the mobilisation of the passive earth pressure is neglected

6.3.1. factorization of load case combinations

LS	factorization	LS	factorization
1	Lc1	5	Lc1+Lc4
2	Lc1+Lc2	6	Lc1+Lc2+Lc4
3	Lc1+Lc3	7	Lc1+Lc3+Lc4
4	Lc1+Lc2+Lc3	8	Lc1+Lc2+Lc3+Lc4

6.3.2. column load

LS	N _{St,d} kN	H _{x,St,d} kN	H _{y,St,d} kN	M _{x,St,d} kNm	M _{y,St,d} kNm	LS	N _{St,d} kN	H _{x,St,d} kN	H _{y,St,d} kN	M _{x,St,d} kNm	M _{y,St,d} kNm
1	152.09	5.50	-0.00	-0.00	0.00	5	152.09	29.50	-0.00	-0.00	-0.00
2	202.09	13.00	-0.00	-0.00	-0.00	6	202.09	37.00	-0.00	-0.00	-0.00
3	152.09	5.50	20.00	0.00	0.00	7	152.09	29.50	20.00	0.00	-0.00
4	202.09	13.00	20.00	0.00	-0.00	8	202.09	37.00	20.00	0.00	-0.00

6.4. limitation of gaping joint under total load

LS	N _{0,k} kN/m	M _{0,x,k} kNm/m	M _{0,y,k} kNm/m	e _x m	e _y m	(e _x /b _x) ² + (e _y /b _y) ² -
1	170.28	-0.00	-3.30	-0.02	-0.00	0.000
2	220.28	-0.00	-7.80	-0.04	-0.00	0.001
3	170.28	12.00	-3.30	-0.02	0.07	0.005
4	220.28	12.00	-7.80	-0.04	0.05	0.004
5	170.28	-0.00	-17.70	-0.10	-0.00	0.011
6	220.28	-0.00	-22.20	-0.10	-0.00	0.010
7	170.28	12.00	-17.70	-0.10	0.07	0.016
8	220.28	12.00	-22.20	-0.10	0.05	0.013

$$((e_x/b_x)^2 + (e_y/b_y)^2)_{\max} = 0.016 < 1/9$$

⇒ the decisive resultant is located in the 2. core area,
sc. no gaping joint beyond centroid.

6.5. verification against displacement in base area

the verification is rated as successful, if the passive earth pressure remains unconsidered in the verification of safety against sliding (s.a.).

LS	N _{0,k} kN	R _{t,k} kN	γ _{R,h} -	R _{t,d} kN	H _{Res,d} kN	μ -
1	170.28	108.48	1.10	98.62	5.50	0.06
2	170.28	108.48	1.10	98.62	7.43	0.08
3	220.28	140.34	1.10	127.58	16.75	0.13
4	220.28	140.34	1.10	127.58	18.67	0.15
5	170.28	108.48	1.10	98.62	18.82	0.19
6	170.28	108.48	1.10	98.62	19.47	0.20
7	220.28	140.34	1.10	127.58	24.59	0.19
8	220.28	140.34	1.10	127.58	25.94	0.20
9	170.28	108.48	1.10	98.62	27.10	0.27
10	170.28	108.48	1.10	98.62	29.03	0.29
11	220.28	140.34	1.10	127.58	38.35	0.30
12	220.28	140.34	1.10	127.58	40.28	0.32
13	170.28	108.48	1.10	98.62	32.53	0.33
14	170.28	108.48	1.10	98.62	34.15	0.35
15	220.28	140.34	1.10	127.58	42.36	0.33
16	220.28	140.34	1.10	127.58	44.11	0.35
17	170.28	108.48	1.10	98.62	30.50	0.31
18	170.28	108.48	1.10	98.62	30.91	0.31
19	220.28	140.34	1.10	127.58	34.36	0.27
20	220.28	140.34	1.10	127.58	35.34	0.28
21	170.28	108.48	1.10	98.62	41.50	0.42
22	170.28	108.48	1.10	98.62	43.42	0.44
23	220.28	140.34	1.10	127.58	52.75	0.41
24	220.28	140.34	1.10	127.58	54.67	0.43
25	170.28	108.48	1.10	98.62	51.21	0.52
26	170.28	108.48	1.10	98.62	52.78	0.54
27	220.28	140.34	1.10	127.58	60.68	0.48
28	220.28	140.34	1.10	127.58	62.36	0.49

$$\mu_{\max} = 0.54 < 1.0 \Rightarrow \text{verification against displacement in base area successful}$$

6.6. design values settlement

the mobilisation of the passive earth pressure is neglected

6.6.1. factorization of load case combinations

LS	factorization	LS	factorization
1	Lc1	5	Lc1+Lc4
2	Lc1+Lc2	6	Lc1+Lc2+Lc4
3	Lc1+Lc3	7	Lc1+Lc3+Lc4
4	Lc1+Lc2+Lc3	8	Lc1+Lc2+Lc3+Lc4

6.6.2. column load

LS	N _{st,d} kN	H _{x,St,d} kN	H _{y,St,d} kN	M _{x,St,d} kNm	M _{y,St,d} kNm	LS	N _{st,d} kN	H _{x,St,d} kN	H _{y,St,d} kN	M _{x,St,d} kNm	M _{y,St,d} kNm
1	152.09	5.50	-0.00	-0.00	0.00	5	152.09	29.50	-0.00	-0.00	-0.00
2	202.09	13.00	-0.00	-0.00	-0.00	6	202.09	37.00	-0.00	-0.00	-0.00
3	152.09	5.50	20.00	0.00	0.00	7	152.09	29.50	20.00	0.00	-0.00
4	202.09	13.00	20.00	0.00	-0.00	8	202.09	37.00	20.00	0.00	-0.00

6.7. settlements

determination of settlement by use of closed formulas acc. to [6]

allowable maximum settlement perm $s_{\max} = 5.0$ cm

allowable obliquity about the x-axis perm $\alpha_x = 0.5$ °

allowable obliquity about the y-axis perm $\alpha_y = 0.5$ °

6.7.1. determination from settlement causing contact pressure and limiting depth

mean settlement causing contact pressure $\sigma_0' = \sigma_0 - \sigma_a$, if $2\sigma_a > \sigma_0$ then $\sigma_0' = \sigma_0$

the limiting depth d_s results from $d_s = z$, if $\sigma_B(z) = 0.2 \cdot \sigma_0'(z)$ below significant point.

unloading from excavation due to foundation depth $\sigma_a = 15.20$ kN/m²

LS	N _{0,k} kN	M _{0,x,k} kNm	M _{0,y,k} kNm	σ_0 kN/m ²	σ_0' kN/m ²	d_s m
1	170.28	-0.00	-3.30	170.28	155.08	2.38
2	220.28	-0.00	-7.80	220.28	205.08	2.70
3	170.28	12.00	-3.30	170.28	155.08	2.38
4	220.28	12.00	-7.80	220.28	205.08	2.70
5	170.28	-0.00	-17.70	170.28	155.08	2.38
6	220.28	-0.00	-22.20	220.28	205.08	2.70
7	170.28	12.00	-17.70	170.28	155.08	2.38
8	220.28	12.00	-22.20	220.28	205.08	2.70

6.7.2. determination of settlement values and settlement parts per soil stratum

coefficient f for settlement below significant point acc. to [7], vol. 2, tab. 4

coefficients f_x/f_y for obliquity of a rigid foundation acc. to [8], fig. 19

settlement parts from central load $s_{m,i} = \sigma_0' \cdot b_y \cdot (f_i - f_{i-1}) / E_{m,i}$

settlement parts from $M_{0,y}$ $s_{x,i} = b_x/2 \cdot M_{0,y} / (E_{m,i} \cdot b_y \cdot b_x^2) \cdot (f_{x,i} - f_{x,i-1})$

settlement parts from $M_{0,x}$ $s_{y,i} = b_y/2 \cdot M_{0,x} / (E_{m,i} \cdot b_x \cdot b_y^2) \cdot (f_{y,i} - f_{y,i-1})$

LS 1: $\sigma_0' = 155.08$ kN/m ² $M_{0,x} = -0.00$ kNm $M_{0,y} = -3.30$ kNm	level m	z m	f -	f_x -	f_y -	S_m cm	S_x cm	S_y cm
	2.50	1.70	0.607	3.814	3.814	0.94	-0.06	-0.00
	3.18	2.38	0.669	3.899	3.899	0.10	-0.00	-0.00

LS 2: $\sigma_0' = 205.08$ kN/m ² $M_{0,x} = -0.00$ kNm $M_{0,y} = -7.80$ kNm	level m	z m	f -	f_x -	f_y -	S_m cm	S_x cm	S_y cm
	2.50	1.70	0.607	3.814	3.814	1.24	-0.15	-0.00
	3.50	2.70	0.690	3.899	3.899	0.17	-0.00	-0.00

LS 3: $\sigma_0' = 155.08$ kN/m ² $M_{0,x} = 12.00$ kNm $M_{0,y} = -3.30$ kNm	level m	z m	f -	f_x -	f_y -	S_m cm	S_x cm	S_y cm
	2.50	1.70	0.607	3.814	3.814	0.94	-0.06	0.23
	3.18	2.38	0.669	3.899	3.899	0.10	-0.00	0.01

LS 4: $\sigma_0' = 205.08$ kN/m ² $M_{0,x} = 12.00$ kNm $M_{0,y} = -7.80$ kNm	level m	z m	f -	f_x -	f_y -	S_m cm	S_x cm	S_y cm
	2.50	1.70	0.607	3.814	3.814	1.24	-0.15	0.23
	3.50	2.70	0.690	3.899	3.899	0.17	-0.00	0.01

LS 5: $\sigma_0' = 155.08$ kN/m ² $M_{0,x} = -0.00$ kNm $M_{0,y} = -17.70$ kNm	level m	z m	f -	f_x -	f_y -	S_m cm	S_x cm	S_y cm
	2.50	1.70	0.607	3.814	3.814	0.94	-0.34	-0.00
	3.18	2.38	0.669	3.899	3.899	0.10	-0.01	-0.00

LS 6: $\sigma_0' = 205.08$ kN/m ² $M_{0,x} = -0.00$ kNm $M_{0,y} = -22.20$ kNm	level m	z m	f -	f_x -	f_y -	S_m cm	S_x cm	S_y cm
	2.50	1.70	0.607	3.814	3.814	1.24	-0.42	-0.00
	3.50	2.70	0.690	3.899	3.899	0.17	-0.01	-0.00

LS 7: $\sigma_0' = 155.08$ kN/m ² $M_{0,x} = 12.00$ kNm $M_{0,y} = -17.70$ kNm	level m	z m	f -	f_x -	f_y -	S_m cm	S_x cm	S_y cm
	2.50	1.70	0.607	3.814	3.814	0.94	-0.34	0.23
	3.18	2.38	0.669	3.899	3.899	0.10	-0.01	0.01

LS 8:

$$\sigma_0' = 205.08 \text{ kN/m}^2$$

$$M_{0,x} = 12.00 \text{ kNm}$$

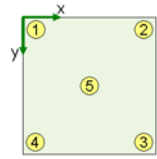
$$M_{0,y} = -22.20 \text{ kNm}$$

level m	z m	f -	f _x -	f _y -	S _m cm	S _x cm	S _y cm
2.50	1.70	0.607	3.814	3.814	1.24	-0.42	0.23
3.50	2.70	0.690	3.899	3.899	0.17	-0.01	0.01

6.7.3. resultant settlements and obliquity per LK

$$s_1 = \sum (s_{m,i} + s_{x,i} - s_{y,i}) \quad s_2 = \sum (s_{m,i} - s_{x,i} - s_{y,i}) \quad s_3 = \sum (s_{m,i} - s_{x,i} + s_{y,i}) \quad s_4 = \sum (s_{m,i} + s_{x,i} + s_{y,i}) \quad s_5 = \sum s_{m,i}$$

$$\tan \alpha_x = 2 \cdot \sum s_{y,i} / b_y \quad \tan \alpha_y = 2 \cdot \sum s_{x,i} / b_x$$



LS	S1 cm	S2 cm	S3 cm	S4 cm	S5 cm	S _{max} cm	α _x °	α _y °
1	1.0	1.1	1.1	1.0	1.0	1.1	-0.0	-0.1
2	1.3	1.6	1.6	1.3	1.4	1.6	-0.0	-0.2
3	0.7	0.9	1.3	1.2	1.0	1.3	0.3	-0.1
4	1.0	1.3	1.8	1.5	1.4	1.8	0.3	-0.2
5	0.7	1.4	1.4	0.7	1.0	1.4	-0.0	-0.4
6	1.0	1.8	1.8	1.0	1.4	1.8	-0.0	-0.5
7	0.5	1.1	1.6	0.9	1.0	1.6	0.3	-0.4
8	0.7	1.6	2.1	1.2	1.4	2.1	0.3	-0.5

$$\max S_{\max} = 2.1 < 5.0 \text{ cm} \quad \max |\alpha_x| = 0.3^\circ < 0.5^\circ \quad \max |\alpha_y| = 0.5^\circ = 0.5^\circ$$

⇒ allowable settlement and obliquity kept

M_{dst} - destabilising moment M_{stb} - stabilising moment N₀ - normal force in foundation joint

M₀ - moment load in centroid of foundation joint a'/b' - substituting widths due to eccentric load with a' > b'

H_a/H_b - horizontal loads in direction of the corresponding widths t - anchoring depth σ₀ - mean normal soil stress

σ_B - soil stress from structural load σ₀ - overburden stress from soil own weight

d_s - limiting depth resp. thickness of compressible stratum below base of foundation z - depth from foundation foot

7. torsion spring of the foundation-ground system

determination of the torsional spring constant with the aid of the subgrade modulus

$$c_{v,x} = k_s \cdot I_x$$

$$c_{v,y} = k_s \cdot I_y$$

estimation of the subgrade acc. to [9]

$$k_s = E_s / (f \cdot (b_x \cdot b_y)^{0.5})$$

with form factor f depending on aspect ratio: 1:1 → f = 0.45, 1:2 → f = 0.42, 1:4 → f = 0.35

assumption for correction factor κ = 1

$$\text{constrained modulus } E_s = 1 \cdot 10000.00 = 10000.00 \text{ kN/m}^2$$

$$\text{shape factor } f = 0.45$$

$$\text{foundation modulus } k_s = 22222.22 \text{ kN/m}^3$$

$$\text{moment of inertia } I_x / I_y = 0.08 / 0.08 \text{ kN/m}^3$$

$$\text{torsional spring about the x-axis } c_{v,x} = 1851.85 \text{ kNm}$$

$$\text{torsional spring about the y-axis } c_{v,y} = 1851.85 \text{ kNm}$$

8. Summary of foundation design

all executed verifications and design calculations successful.

overturning	μ _{max} = 0.29
base failure	μ _{max} = 0.85
slip	μ _{max} = 0.39
gaping joint under permant load	μ _{exist} = 0.12
gaping joint under total load	μ _{max} = 0.14
displacement in the base area	μ _{max} = 0.54
settlement	S _{max} = 2.1 < 5.0 cm
obliquity (about x-axis)	α _{max,x} = 0.3 < 0.5 °
obliquity (about y-axis)	α _{max,y} = 0.5 = 0.5 °
torsional spring (about the x-axis)	c _{v,x} = 1852 kNm
torsional spring (about the y-axis)	c _{v,y} = 1852 kNm

literature and standards:

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[5] DIN 4017: Baugrund, Berechnung des Grundbruchwiderstandes von Flächengründungen, März 2006

[6] DIN 4019: Baugrund - Setzungsberechnungen, Januar 2014

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