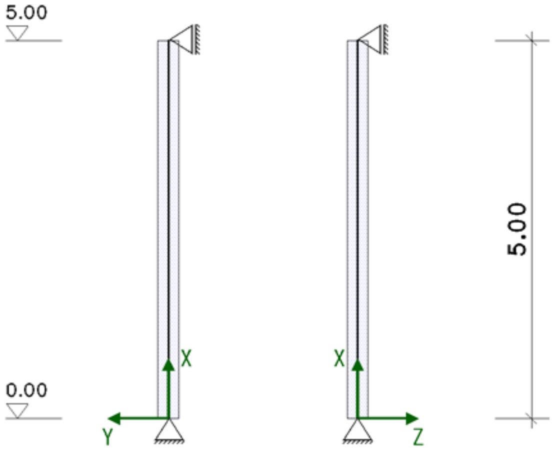
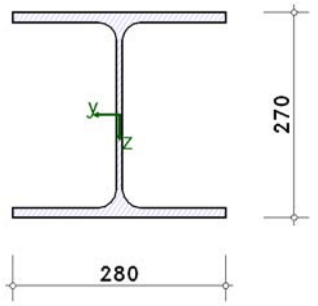
steel column

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

System



cross-section, scale 1:10



steel grade S235
section HE280A

support situation at head and foot end

supp.	shear force		moment	
	C_{QY}	C_{QZ}	C_{MY}	C_{MZ}
-	kN/m	kN/m	kNm/-	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

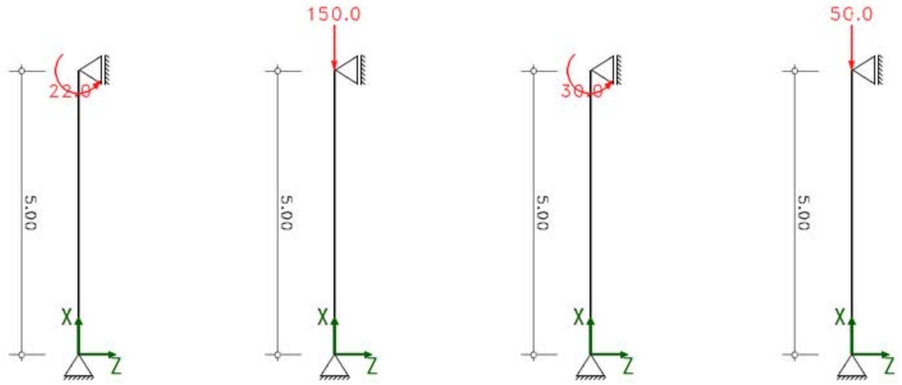
1: permanent loads	**permanent loads**
1: dead load	additive
4: live load	**transient live loads of storage rooms**
2: live load	additive
9: wind	**transient wind load**
3: Y-direction	alternative in group A
4: Z-direction	additive

1.2. table of load pictures

loadc.	loadpic.	introduct.	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

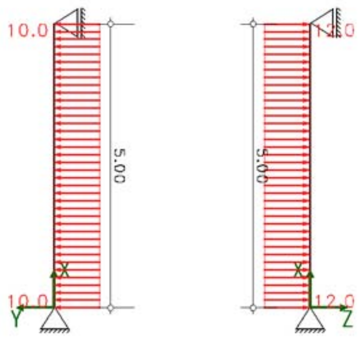
1.2.1. graphics of point loads

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: γ_{M1} = 1.10

perm. normal, shear, equivalent stresses: $\sigma_{x,Rd}$ = 213.6 N/mm², τ_{Rd} = 123.3 N/mm², $\sigma_{v,Rd}$ = 213.6 N/mm²

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

direct.	form	β -	l_0 m	Φ -	e cm	EI MNm²	N_{Ki} MN
Y	curvat.	1.00	5.00	0.0050	0.000	10.00	3.95
Z	curvat.	1.00	5.00	0.0050	0.000	28.71	11.35

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

2.1.2. direction of imperfection

$\alpha_{imp} = (\alpha_{load} - \alpha_{bend})/(3-1) \cdot (N_{Ki,z}/N_{Ki,y} - 1) + \alpha_{bend}$
with α_{load} : direction due to load, α_{bend} : direction of the weak axis

$1 < N_{Ki,z}/N_{Ki,y} = 2.87 < 3 \Rightarrow$ interpolation between deformation direction and weak axis:

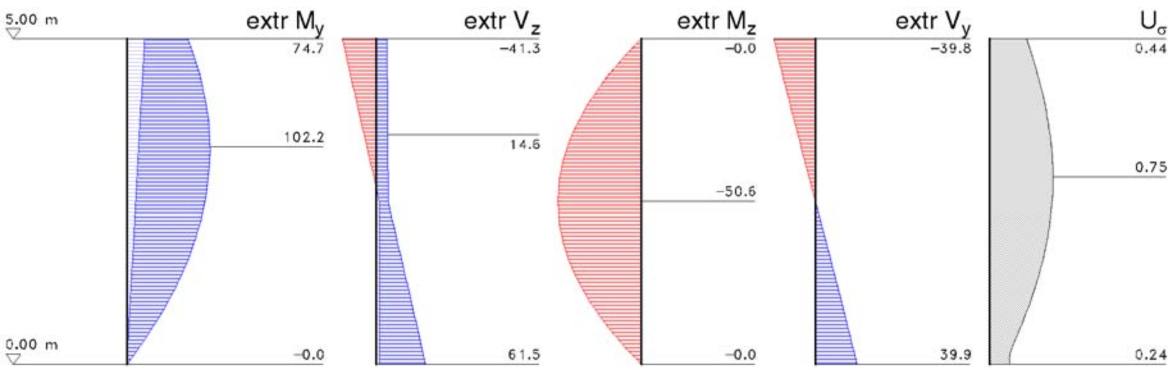
the decisive direction is interpolated between the direction of deformation due to the design load and the direction of least buckling safety.

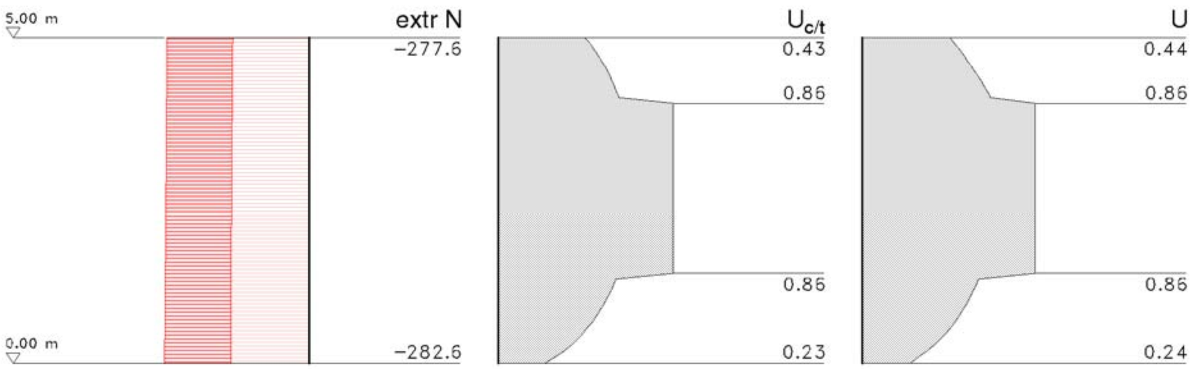
2.2. factorization of load case combinations

LS	design situat.	factorization
1	perman.	Lc1+0.99· I_y+0.1· I_z
2	perman.	1.35· Lc1+0.99· I_y+0.1· I_z
3	perman.	Lc1+1.5· Lc2+0.99· I_y+0.1· I_z
4	perman.	1.35· Lc1+1.5· Lc2+0.99· I_y+0.1· I_z
5	perman.	Lc1+0.6· 1.5· Lc3+I_y+0.01016· I_z
6	perman.	1.35· Lc1+0.6· 1.5· Lc3+I_y+0.0135· I_z
7	perman.	Lc1+1.5· Lc2+0.6· 1.5· Lc3+I_y+0.02904· I_z
8	perman.	1.35· Lc1+1.5· Lc2+0.6· 1.5· Lc3+I_y+0.03162· I_z
9	perman.	Lc1+0.6· 1.5· Lc4+0.99· I_y+0.1· I_z
10	perman.	1.35· Lc1+0.6· 1.5· Lc4+0.99· I_y+0.1· I_z
11	perman.	Lc1+1.5· Lc2+0.6· 1.5· Lc4+0.99· I_y+0.1· I_z
12	perman.	1.35· Lc1+1.5· Lc2+0.6· 1.5· Lc4+0.99· I_y+0.1· I_z
13	perman.	Lc1+0.6· 1.5· (Lc3+Lc4)+I_y+0.03315· I_z
14	perman.	1.35· Lc1+0.6· 1.5· (Lc3+Lc4)+I_y+0.03553· I_z
15	perman.	Lc1+1.5· Lc2+0.6· 1.5· (Lc3+Lc4)+I_y+0.04665· I_z
16	perman.	1.35· Lc1+1.5· Lc2+0.6· 1.5· (Lc3+Lc4)+I_y+0.04829· I_z
17	perman.	Lc1+1.5· Lc3+I_y+0.00613· I_z
18	perman.	1.35· Lc1+1.5· Lc3+I_y+0.00818· I_z
19	perman.	Lc1+1.5· (Lc2+Lc3)+I_y+0.018· I_z
20	perman.	1.35· Lc1+1.5· (Lc2+Lc3)+I_y+0.01976· I_z
21	perman.	Lc1+1.5· Lc4+0.99· I_y+0.1· I_z
22	perman.	1.35· Lc1+1.5· Lc4+0.99· I_y+0.1· I_z
23	perman.	Lc1+1.5· (Lc2+Lc4)+0.99· I_y+0.1· I_z
24	perman.	1.35· Lc1+1.5· (Lc2+Lc4)+0.99· I_y+0.1· I_z
25	perman.	Lc1+1.5· (Lc3+Lc4)+I_y+0.02996· I_z
26	perman.	1.35· Lc1+1.5· (Lc3+Lc4)+I_y+0.03141· I_z
27	perman.	Lc1+1.5· (Lc2+Lc3+Lc4)+I_y+0.03881· I_z
28	perman.	1.35· Lc1+1.5· (Lc2+Lc3+Lc4)+I_y+0.0401· I_z

2.3. extreme results

the internal forces are related to the deformed system axis





x	N		My		Vz		Mz		Vy		U
	min	max	min	max	min	max	min	max	min	max	
m	kN	kN	kNm	kNm	kN	kN	kNm	kNm	kN	kN	-
5.00	-277.57	**-149.51**	22.00	74.70	**-41.29**	13.71	-0.00	0.00	**-39.84**	0.00	0.44
4.91	-277.66	-149.60	21.61	77.60	-39.61	13.78	-3.62	0.00	-38.45	0.00	0.46
3.33	-279.23	-151.20	14.85	**102.22**	-10.88	14.73	-44.86	0.00	-13.64	0.00	0.86
2.78	-279.80	-151.69	12.41	99.69	-0.68	14.98	-49.92	0.00	-4.57	0.00	0.86
2.69	-279.89	-151.77	12.00	98.70	1.02	15.02	-50.27	0.00	-3.05	0.00	0.86
2.50	-280.07	-151.91	11.18	96.23	4.42	15.09	**-50.56**	0.00	-0.01	0.00	0.86
2.41	-280.16	-151.98	10.77	94.75	4.43	16.84	-50.49	0.00	0.00	1.51	0.86
1.39	-281.20	-152.61	6.24	67.82	4.48	36.00	-40.46	0.00	0.00	18.13	**0.86**
0.00	**-282.62**	-153.32	**-0.00**	0.00	4.50	**61.51**	-0.00	**0.00**	0.00	**39.87**	0.24

relevant utilisation:

from load spectrum 28 at the location x = 1.39 m, with the internal forces and moments:

N = -281.00 kN, M_y = 67.82 kNm, V_z = 36.00 kN, M_z = -40.46 kNm, V_y = 18.13 kN

$U_{c/t}$ = 0.86, U_σ = 0.59 ⇒ max U = 0.86 < 1.0 ⇒ **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

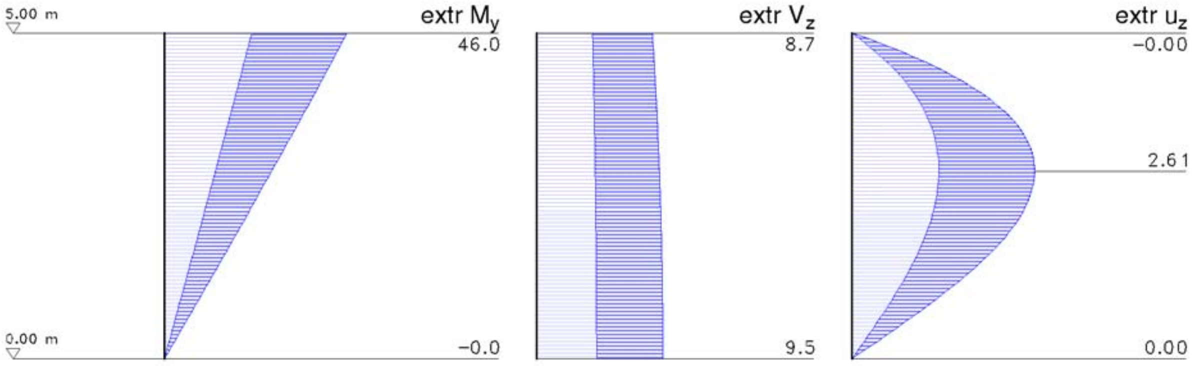
3.1. factorization of load case combinations

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_y min	M_y max	V_z min	V_z max	u_z min	u_z max
m	kNm	kNm	kN	kN	mm	mm
5.00	22.00	**46.00**	**4.21**	8.69	**-0.00**	-0.00
2.87	12.82	26.91	4.40	9.20	1.25	**2.61**
0.00	**-0.00**	0.00	4.50	**9.46**	0.00	0.00

relevant deformation in Z-direction:

from load spectrum 2 at the location x = 2.87 m, with the internal forces and moments:
N = -191.63 kN, M_y = 26.91 kNm, V_z = 9.20 kN, M_z = 0.00 kNm, V_y = 0.00 kN

u_z = 2.61 < 3.00 mm ⇒ **permissible deformation is complied with**

4. support reaction

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

Lc	N_k	$H_{k,y}$	$H_{k,z}$	$M_{k,y}$	$M_{k,z}$
	kN	kN	kN	kN	kN
1	153.82	0.00	4.40	-0.00	-0.00
2	50.00	0.00	6.00	-0.00	0.00
3	0.00	25.00	0.00	-0.00	0.00
4	-0.00	0.00	30.00	-0.00	-0.00
Σ	203.82	25.00	40.40	-0.00	0.00

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

LS	N_{Ed}	$H_{Ed,y}$	$H_{Ed,z}$	$M_{Ed,y}$	$M_{Ed,z}$
	kN	kN	kN	kN	kN
1	153.82	0.00	4.40	0.00	0.00
4	282.66	0.00	14.94	0.00	-0.00
11	228.82	0.00	40.40	-0.00	0.00
17	153.82	37.49	4.40	0.00	-0.00
20	282.66	37.49	14.94	0.00	0.00
21	153.82	0.00	49.40	-0.00	0.00
24	282.66	0.00	59.93	-0.00	0.00

5. summary

all verifications were carried out successfully.

maximum utilisation of the verifications:	
design resist Th.II.0	**86 %**
deformation in Z-direct.	**87 %**

literature and standards:

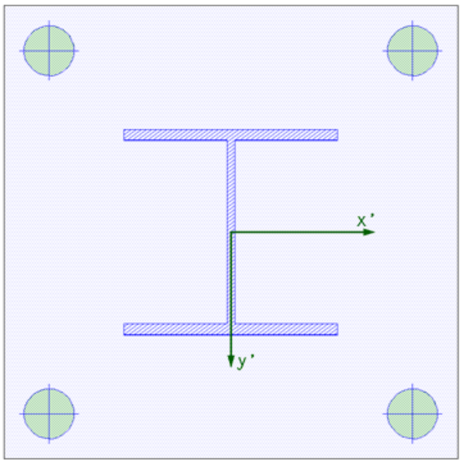
[1] DIN EN 1993-1-1/NA: Nationaler Anhang - National festgelegte Parameter - Eurocode 3: Teil 1-1, Okt. 2022
[2] DIN EN 1993-1-1: Eurocode 3: Bem. und Konstr. von Stahlbauten - Teil 1-1: Allg. Bem.regeln u. Regeln für den Hochbau, Dez. 2010
[3] DIN EN 1990: Grundlagen der Tragwerksplanung, Oktober 2021

BASE PLATE WITH ISOLATED FOUNDATION

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

steel code verifications acc. to DIN EN 1993-1:2010-12 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



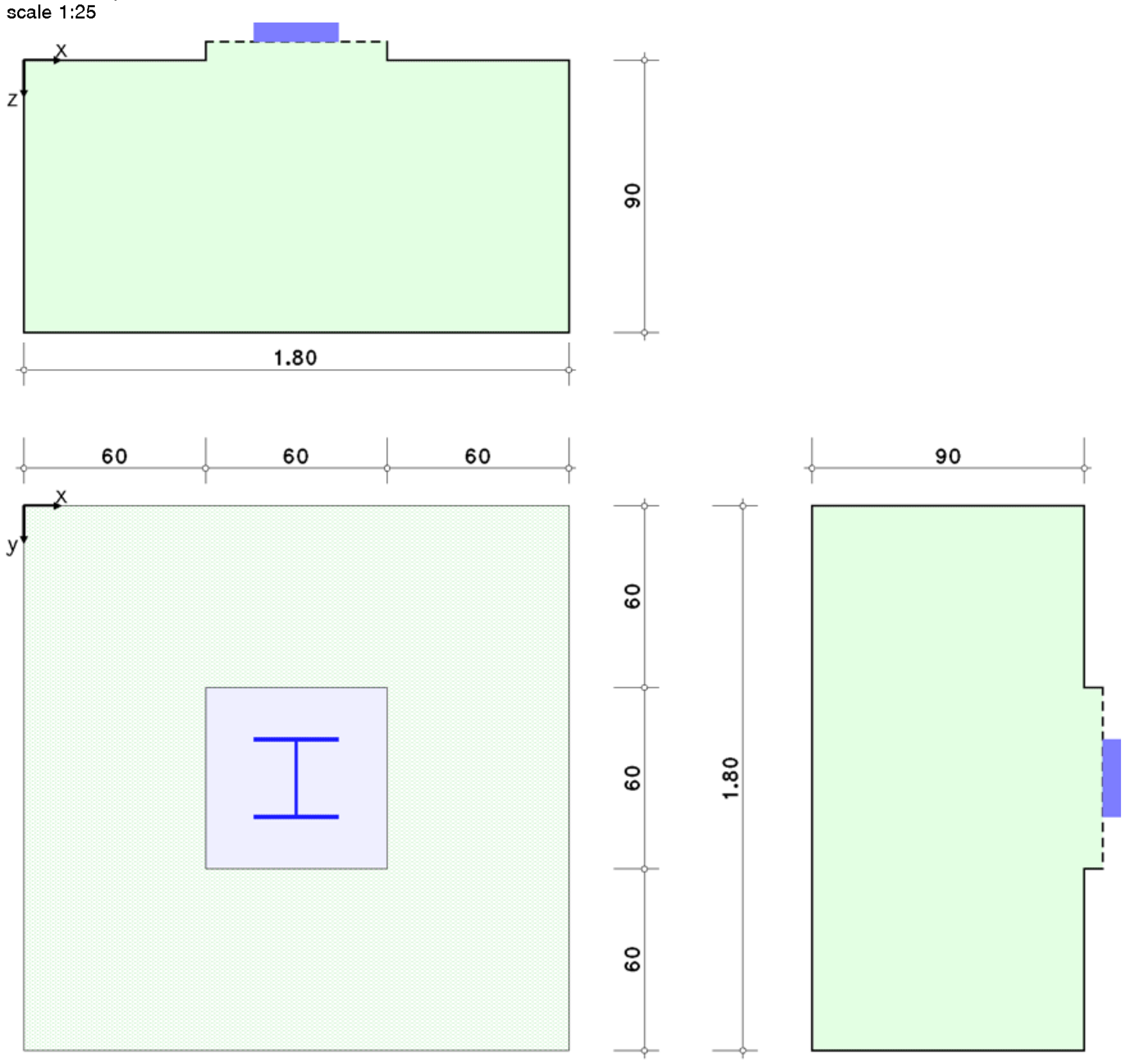
column cross section
standardized profile: HE280A, of quality S235

base plate
b_x = 600 mm b_y = 600 mm t = 60 mm, of quality S235

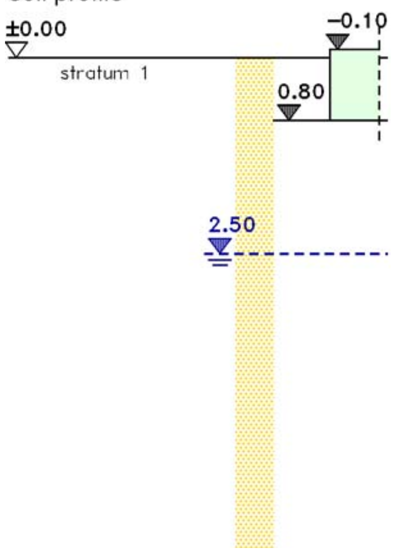
mortar joint
t_F = 20 mm

anchors
4 anchors, FK 8.8, M36, without shaft
with a length of 450 mm
edge distances a_x/a_y = 60/60 mm

elevation, top view isolated foundation
scale 1:25



soil profile



concrete strength class **C30/37**
steel class **B500A**

1. soil situation

the anchoring depth of the foundation is t = 0.80 m.
the ground water level (below top edge soil) is at t_w = 2.50 m.

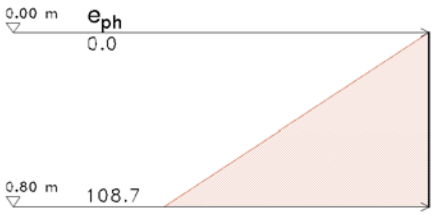
1.1. designation and characteristic values of soil strata

stratum	z m	γ kN/m³	γ' kN/m³	φ °	c_k kN/m²	E_m MN/m²	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 dained 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
fFor a **rough** wall surface, with a wall friction angle $\delta = 2/3\cdot\varphi'_k$



$\Sigma(\gamma\cdot h)$ total soil weight at the considered depth
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

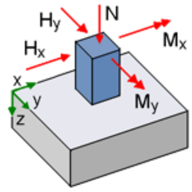
z m	$\Sigma(\gamma\cdot h)$ kN/m²	K_{pgh} -	e_{ph} kN/m²
0.00	0.00	7.152	0.00
0.80	15.20	7.152	108.71

the resultant maximum passive earth pressure is E_{phg} = 43.48 kN/m, at z_s = 0.53 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.	153.82	4.40	0.00	0.00	0.00
6	LS 2 (auto)	perman. a.v.	207.66	5.94	0.00	0.00	-0.00
7	LS 3 (auto)	perman. a.v.	228.82	13.40	0.00	0.00	-0.00
8	LS 4 (auto)	perman. a.v.	282.66	14.94	0.00	-0.00	0.00
9	LS 5 (auto)	perman. a.v.	153.82	4.40	22.50	-0.00	0.00
10	LS 6 (auto)	perman. a.v.	207.66	5.94	22.49	-0.00	-0.00
11	LS 7 (auto)	perman. a.v.	228.82	13.40	22.50	-0.00	-0.00
12	LS 8 (auto)	perman. a.v.	282.66	14.94	22.49	-0.00	0.00
13	LS 9 (auto)	perman. a.v.	153.82	31.40	0.00	0.00	-0.00
14	LS 10 (auto)	perman. a.v.	207.66	32.94	0.00	0.00	-0.00
15	LS 11 (auto)	perman. a.v.	228.82	40.40	0.00	0.00	-0.00
16	LS 12 (auto)	perman. a.v.	282.66	41.94	0.00	0.00	-0.00
17	LS 13 (auto)	perman. a.v.	153.82	31.40	22.50	-0.00	-0.00
18	LS 14 (auto)	perman. a.v.	207.66	32.94	22.49	-0.00	-0.00
19	LS 15 (auto)	perman. a.v.	228.82	40.40	22.50	-0.00	-0.00
20	LS 16 (auto)	perman. a.v.	282.66	41.94	22.49	-0.00	-0.00
21	LS 17 (auto)	perman. a.v.	153.82	4.40	37.49	-0.00	0.00
22	LS 18 (auto)	perman. a.v.	207.66	5.94	37.49	-0.00	-0.00
23	LS 19 (auto)	perman. a.v.	228.82	13.40	37.49	-0.00	-0.00
24	LS 20 (auto)	perman. a.v.	282.66	14.94	37.49	0.00	0.00
25	LS 21 (auto)	perman. a.v.	153.82	49.40	0.00	0.00	-0.00
26	LS 22 (auto)	perman. a.v.	207.66	50.94	0.00	0.00	-0.00
27	LS 23 (auto)	perman. a.v.	228.82	58.40	0.00	-0.00	-0.00
28	LS 24 (auto)	perman. a.v.	282.66	59.93	0.00	0.00	-0.00
29	LS 25 (auto)	perman. a.v.	153.82	49.40	37.49	-0.00	-0.00
30	LS 26 (auto)	perman. a.v.	207.66	50.94	37.49	-0.00	-0.00
31	LS 27 (auto)	perman. a.v.	228.82	58.40	37.49	-0.00	-0.00
32	LS 28 (auto)	perman. a.v.	282.66	59.93	37.49	0.00	-0.00

2.3. dead load

the weight of the foundation plate is taken into account with γ_E = 25.00 kN/m³.
no earth load in action.

the resultant of dead load in the floor joint is $N_{0,deadl,k}$ = 72.90 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
1	153.82	4.40	0.00	0.00	0.00
2	207.66	5.94	0.00	0.00	-0.00
3	228.82	13.40	0.00	0.00	-0.00
4	282.66	14.94	0.00	-0.00	0.00
5	153.82	4.40	22.50	-0.00	0.00
6	207.66	5.94	22.49	-0.00	-0.00
7	228.82	13.40	22.50	-0.00	-0.00
8	282.66	14.94	22.49	-0.00	0.00
9	153.82	31.40	0.00	0.00	-0.00
10	207.66	32.94	0.00	0.00	-0.00
11	228.82	40.40	0.00	0.00	-0.00
12	282.66	41.94	0.00	0.00	-0.00
13	153.82	31.40	22.50	-0.00	-0.00
14	207.66	32.94	22.49	-0.00	-0.00
15	228.82	40.40	22.50	-0.00	-0.00
16	282.66	41.94	22.49	-0.00	-0.00
17	153.82	4.40	37.49	-0.00	0.00
18	207.66	5.94	37.49	-0.00	-0.00
19	228.82	13.40	37.49	-0.00	-0.00
20	282.66	14.94	37.49	0.00	0.00
21	153.82	49.40	0.00	0.00	-0.00
22	207.66	50.94	0.00	0.00	-0.00
23	228.82	58.40	0.00	-0.00	-0.00
24	282.66	59.93	0.00	0.00	-0.00
25	153.82	49.40	37.49	-0.00	-0.00
26	207.66	50.94	37.49	-0.00	-0.00
27	228.82	58.40	37.49	-0.00	-0.00
28	282.66	59.93	37.49	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

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

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

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

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

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

connection designed with a **full-size double fillet weld.**
axial force transfer of 100 % by the weld.

minimum value of the weld thickness a_{min} = 8 mm

LS	$a_{w,Fl}$ mm	$a_{w,S}$ mm	$\sigma_\perp$ kN/cm²	$\tau_\perp$ kN/cm²	τ_{II} kN/cm²	$\sigma_{1,w,Ed}$ kN/cm²	$f_{1,w,Rd}$ kN/cm²	$\sigma_{2,w,Ed}$ kN/cm²	$f_{2,w,Rd}$ kN/cm²	decisiv	U -
1	**8**	**8**	-0.97	-0.97	-0.05	1.94	36.00	0.97	25.92	-	0.05
2	8	8	-1.31	-1.31	-0.07	2.63	36.00	1.31	25.92	-	0.07
3	8	8	-1.44	-1.44	-0.17	2.90	36.00	1.44	25.92	-	0.08
4	8	8	-1.78	-1.78	-0.19	3.58	36.00	1.78	25.92	-	0.10
5	8	8	-0.97	-0.97	0.72	2.31	36.00	0.97	25.92	web	0.06
6	8	8	-1.31	-1.31	0.72	2.90	36.00	1.31	25.92	web	0.08
7	8	8	-1.44	-1.44	0.72	3.15	36.00	1.44	25.92	web	0.09
8	8	8	-1.78	-1.78	0.72	3.78	36.00	1.78	25.92	web	0.10
9	8	8	-0.97	-0.97	-0.39	2.06	36.00	0.97	25.92	-	0.06
10	8	8	-1.31	-1.31	-0.41	2.72	36.00	1.31	25.92	-	0.08
11	8	8	-1.44	-1.44	-0.50	3.02	36.00	1.44	25.92	-	0.08
12	8	8	-1.78	-1.78	-0.52	3.68	36.00	1.78	25.92	-	0.10
13	8	8	-0.97	-0.97	0.72	2.31	36.00	0.97	25.92	web	0.06
14	8	8	-1.31	-1.31	0.72	2.90	36.00	1.31	25.92	web	0.08
15	8	8	-1.44	-1.44	0.72	3.15	36.00	1.44	25.92	-	0.09
16	8	8	-1.78	-1.78	0.72	3.78	36.00	1.78	25.92	-	0.10
17	8	8	-0.97	-0.97	1.20	2.84	36.00	0.97	25.92	web	0.08
18	8	8	-1.31	-1.31	1.20	3.34	36.00	1.31	25.92	web	0.09
19	8	8	-1.44	-1.44	1.20	3.55	36.00	1.44	25.92	web	0.10
20	8	8	-1.78	-1.78	1.20	4.13	36.00	1.78	25.92	web	0.11
21	8	8	-0.97	-0.97	-0.61	2.21	36.00	0.97	25.92	flange	0.06
22	8	8	-1.31	-1.31	-0.63	2.84	36.00	1.31	25.92	flange	0.08
23	8	8	-1.44	-1.44	-0.72	3.15	36.00	1.44	25.92	flange	0.09
24	8	8	-1.78	-1.78	-0.74	3.79	36.00	1.78	25.92	flange	0.11
25	8	8	-0.97	-0.97	1.20	2.84	36.00	0.97	25.92	web	0.08
26	8	8	-1.31	-1.31	1.20	3.34	36.00	1.31	25.92	web	0.09
27	8	8	-1.44	-1.44	1.20	3.55	36.00	1.44	25.92	web	0.10
28	8	8	-1.78	-1.78	1.20	4.13	36.00	1.78	25.92	web	**0.11**

$a_{w,Fl}$ - flange weld thickness $a_{w,S}$ - web weld thickness $\sigma_\perp$ - normal stresses perpendicular to weld
$\tau_\perp$ - shear stresses perpendicular to weld τ_{II} - shear stresses parallel to weld U - utilization

maximum weld thickness flange $a_{w,Fl,max}$ = 8 mm
maximum weld thickness of the web $a_{w,S,max}$ = 8 mm
maximum utilization U = 0.11 < 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 devided into 49 elements in X-direction and 54 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 $c = E\cdot A/l$ = 3810.80 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	44.7	42.8	2.56	2.83	-0.04	0.05	0.05
2	44.7	42.8	3.45	3.82	-0.06	0.06	0.07
3	44.7	42.8	3.81	4.20	-0.06	0.07	0.08
4	44.7	42.8	4.70	5.19	-0.08	0.09	0.09
5	44.7	42.8	2.56	2.83	-0.04	0.05	0.05
6	44.7	42.8	3.45	3.82	-0.06	0.06	0.07
7	44.7	42.8	3.81	4.20	-0.06	0.07	0.08
8	44.7	42.8	4.70	5.19	-0.08	0.09	0.09
9	44.7	42.8	2.56	2.83	-0.04	0.05	0.05
10	44.7	42.8	3.45	3.82	-0.06	0.06	0.07
11	44.7	42.8	3.81	4.20	-0.06	0.07	0.08
12	44.7	42.8	4.70	5.19	-0.08	0.09	0.09
13	44.7	42.8	2.56	2.83	-0.04	0.05	0.05
14	44.7	42.8	3.45	3.82	-0.06	0.06	0.07
15	44.7	42.8	3.81	4.20	-0.06	0.07	0.08
16	44.7	42.8	4.70	5.19	-0.08	0.09	0.09
17	44.7	42.8	2.56	2.83	-0.04	0.05	0.05
18	44.7	42.8	3.45	3.82	-0.06	0.06	0.07
19	44.7	42.8	3.81	4.20	-0.06	0.07	0.08
20	44.7	42.8	4.70	5.19	-0.08	0.09	0.09
21	44.7	42.8	2.56	2.83	-0.04	0.05	0.05
22	44.7	42.8	3.45	3.82	-0.06	0.06	0.07
23	44.7	42.8	3.81	4.20	-0.06	0.07	0.08
24	44.7	42.8	4.70	5.19	-0.08	0.09	0.09
25	44.7	42.8	2.56	2.83	-0.04	0.05	0.05
26	44.7	42.8	3.45	3.82	-0.06	0.06	0.07
27	44.7	42.8	3.81	4.20	-0.06	0.07	0.08
28	44.7	42.8	4.70	5.19	-0.08	0.09	0.09

stresses and utilizations

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

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

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

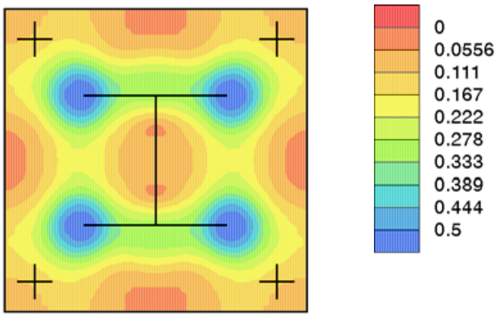
LS	x_{Fp} cm	y_{Fp} cm	$\sigma_{Pl,V}$ kN/cm²	σ_{Rd} kN/cm²	U -
1	44.7	42.8	0.30	21.50	0.01
2	44.7	42.8	0.41	21.50	0.02
3	44.7	42.8	0.45	21.50	0.02
4	44.7	42.8	0.55	21.50	**0.03**
5	44.7	42.8	0.30	21.50	0.01
6	44.7	42.8	0.41	21.50	0.02
7	44.7	42.8	0.45	21.50	0.02
8	44.7	42.8	0.55	21.50	0.03
9	44.7	42.8	0.30	21.50	0.01
10	44.7	42.8	0.41	21.50	0.02
11	44.7	42.8	0.45	21.50	0.02
12	44.7	42.8	0.55	21.50	0.03
13	44.7	42.8	0.30	21.50	0.01
14	44.7	42.8	0.41	21.50	0.02
15	44.7	42.8	0.45	21.50	0.02
16	44.7	42.8	0.55	21.50	0.03
17	44.7	42.8	0.30	21.50	0.01
18	44.7	42.8	0.41	21.50	0.02
19	44.7	42.8	0.45	21.50	0.02
20	44.7	42.8	0.55	21.50	0.03
21	44.7	42.8	0.30	21.50	0.01
22	44.7	42.8	0.41	21.50	0.02
23	44.7	42.8	0.45	21.50	0.02
24	44.7	42.8	0.55	21.50	0.03
25	44.7	42.8	0.30	21.50	0.01
26	44.7	42.8	0.41	21.50	0.02
27	44.7	42.8	0.45	21.50	0.02
28	44.7	42.8	0.55	21.50	0.03

maximum utilization U = 0.03 < 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} > 180.0 cm^2)

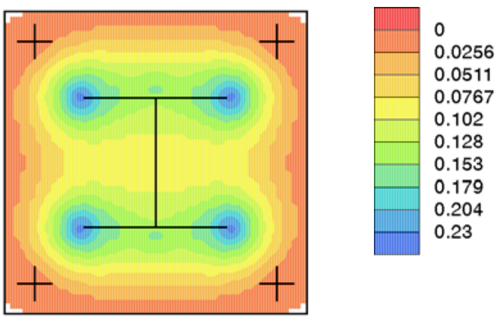
LS	lim $\sigma_{c,d}$ kN/cm²	A_{compr} cm²	F_{compr} kN	$\sigma_{c,max}$ kN/cm²	$\sigma_{c,m}$ kN/cm²	f_{jd} kN/cm²	U -
1	5.10	3546.9	153.82	0.13	0.043	1.70	0.03
2	5.10	3546.9	207.66	0.17	0.059	1.70	0.03
3	5.10	3549.7	228.82	0.19	0.064	1.70	0.04
4	5.10	3557.8	282.66	0.23	0.079	1.70	**0.05**
5	5.10	3546.9	153.82	0.13	0.043	1.70	0.03
6	5.10	3546.9	207.66	0.17	0.059	1.70	0.03
7	5.10	3549.7	228.82	0.19	0.064	1.70	0.04
8	5.10	3557.8	282.66	0.23	0.079	1.70	0.05
9	5.10	3546.9	153.82	0.13	0.043	1.70	0.03
10	5.10	3546.9	207.66	0.17	0.059	1.70	0.03
11	5.10	3549.7	228.82	0.19	0.064	1.70	0.04
12	5.10	3557.8	282.66	0.23	0.079	1.70	0.05
13	5.10	3546.9	153.82	0.13	0.043	1.70	0.03
14	5.10	3546.9	207.66	0.17	0.059	1.70	0.03
15	5.10	3549.7	228.82	0.19	0.064	1.70	0.04
16	5.10	3557.8	282.66	0.23	0.079	1.70	0.05
17	5.10	3546.9	153.82	0.13	0.043	1.70	0.03
18	5.10	3546.9	207.66	0.17	0.059	1.70	0.03
19	5.10	3549.7	228.82	0.19	0.064	1.70	0.04
20	5.10	3557.8	282.66	0.23	0.079	1.70	0.05
21	5.10	3546.9	153.82	0.13	0.043	1.70	0.03
22	5.10	3546.9	207.66	0.17	0.059	1.70	0.03
23	5.10	3549.7	228.82	0.19	0.064	1.70	0.04
24	5.10	3557.8	282.66	0.23	0.079	1.70	0.05
25	5.10	3546.9	153.82	0.13	0.043	1.70	0.03
26	5.10	3546.9	207.66	0.17	0.059	1.70	0.03
27	5.10	3549.7	228.82	0.19	0.064	1.70	0.04
28	5.10	3557.8	282.66	0.23	0.079	1.70	0.05

maximum utilization U = 0.05 < 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

pressure distribution [kN/cm²]

LS 4 (max $\sigma_{c,m}$)

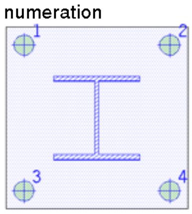


3.4.3. anchor tensile forces

$F_{t,Rd} = k2 \cdot f_{ub} \cdot A_s / \gamma_{M2}$
$U = F_{t,Ed,max} / F_{,Rd}$

stress area of M36: A_s = 8.17 cm²
no countersunk bolts used: k2 = 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	---	---	---	---	470.36	**0.00**
2	---	---	---	---	470.36	0.00
3	---	---	---	---	470.36	0.00
4	---	---	---	---	470.36	0.00
5	---	---	---	---	470.36	0.00
6	---	---	---	---	470.36	0.00

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} -
7	---	---	---	---	470.36	0.00
8	---	---	---	---	470.36	0.00
9	---	---	---	---	470.36	0.00
10	---	---	---	---	470.36	0.00
11	---	---	---	---	470.36	0.00
12	---	---	---	---	470.36	0.00
13	---	---	---	---	470.36	0.00
14	---	---	---	---	470.36	0.00
15	---	---	---	---	470.36	0.00
16	---	---	---	---	470.36	0.00
17	---	---	---	---	470.36	0.00
18	---	---	---	---	470.36	0.00
19	---	---	---	---	470.36	0.00
20	---	---	---	---	470.36	0.00
21	---	---	---	---	470.36	0.00
22	---	---	---	---	470.36	0.00
23	---	---	---	---	470.36	0.00
24	---	---	---	---	470.36	0.00
25	---	---	---	---	470.36	0.00
26	---	---	---	---	470.36	0.00
27	---	---	---	---	470.36	0.00
28	---	---	---	---	470.36	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

$H_{res,d} = (H_{x,St,d}^2+H_{y,St,d}^2)^{0.5}$
$N_{z,d} = A_{compr} \cdot \sigma_{c,m}$
$H_{Rd} = \mu_k/\gamma_\mu \cdot N_{z,d}$ with $\mu_k = 0.6$ acc. to [2]

LS	$H_{res,d}$ kN	$N_{z,d}$ kN	H_{Rd} kN	U -
1	4.40	153.82	76.91	0.06
2	5.94	207.66	103.83	0.06
3	13.40	228.82	114.41	0.12
4	14.94	282.66	141.33	0.11
5	22.92	153.82	76.91	0.30
6	23.27	207.66	103.83	0.22
7	26.18	228.82	114.41	0.23
8	27.00	282.66	141.33	0.19
9	31.40	153.82	76.91	0.41
10	32.94	207.66	103.83	0.32
11	40.40	228.82	114.41	0.35
12	41.94	282.66	141.33	0.30
13	38.63	153.82	76.91	0.50
14	39.89	207.66	103.83	0.38
15	46.24	228.82	114.41	0.40
16	47.59	282.66	141.33	0.34
17	37.75	153.82	76.91	0.49
18	37.96	207.66	103.83	0.37
19	39.82	228.82	114.41	0.35
20	40.36	282.66	141.33	0.29
21	49.40	153.82	76.91	0.64
22	50.94	207.66	103.83	0.49
23	58.40	228.82	114.41	0.51
24	59.93	282.66	141.33	0.42
25	62.01	153.82	76.91	**0.81**
26	63.25	207.66	103.83	0.61
27	69.40	228.82	114.41	0.61
28	70.69	282.66	141.33	0.50

maximum utilization U = 0.81 < 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
1	permanent and transient	Lc1* +Lc5
2	permanent and transient	1.35· Lc1* +Lc5
3	permanent and transient	Lc1* +Lc6
4	permanent and transient	1.35· Lc1* +Lc6
5	permanent and transient	Lc1* +Lc7
6	permanent and transient	1.35· Lc1* +Lc7
7	permanent and transient	Lc1* +Lc8
8	permanent and transient	1.35· Lc1* +Lc8
9	permanent and transient	Lc1* +Lc9
10	permanent and transient	1.35· Lc1* +Lc9
11	permanent and transient	Lc1* +Lc10
12	permanent and transient	1.35· Lc1* +Lc10
13	permanent and transient	Lc1* +Lc11
14	permanent and transient	1.35· Lc1* +Lc11
15	permanent and transient	Lc1* +Lc12
16	permanent and transient	1.35· Lc1* +Lc12
17	permanent and transient	Lc1* +Lc13
18	permanent and transient	1.35· Lc1* +Lc13
19	permanent and transient	Lc1* +Lc14
20	permanent and transient	1.35· Lc1* +Lc14

LS	design situat.	factorization
21	permanent and transient	Lc1* +Lc15
22	permanent and transient	1.35· Lc1* +Lc15
23	permanent and transient	Lc1* +Lc16
24	permanent and transient	1.35· Lc1* +Lc16
25	permanent and transient	Lc1* +Lc17
26	permanent and transient	1.35· Lc1* +Lc17
27	permanent and transient	Lc1* +Lc18
28	permanent and transient	1.35· Lc1* +Lc18
29	permanent and transient	Lc1* +Lc19
30	permanent and transient	1.35· Lc1* +Lc19
31	permanent and transient	Lc1* +Lc20
32	permanent and transient	1.35· Lc1* +Lc20
33	permanent and transient	Lc1* +Lc21
34	permanent and transient	1.35· Lc1* +Lc21
35	permanent and transient	Lc1* +Lc22
36	permanent and transient	1.35· Lc1* +Lc22
37	permanent and transient	Lc1* +Lc23
38	permanent and transient	1.35· Lc1* +Lc23
39	permanent and transient	Lc1* +Lc24
40	permanent and transient	1.35· Lc1* +Lc24
41	permanent and transient	Lc1* +Lc25
42	permanent and transient	1.35· Lc1* +Lc25
43	permanent and transient	Lc1* +Lc26
44	permanent and transient	1.35· Lc1* +Lc26
45	permanent and transient	Lc1* +Lc27
46	permanent and transient	1.35· Lc1* +Lc27
47	permanent and transient	Lc1* +Lc28
48	permanent and transient	1.35· Lc1* +Lc28
49	permanent and transient	Lc1* +Lc29
50	permanent and transient	1.35· Lc1* +Lc29
51	permanent and transient	Lc1* +Lc30
52	permanent and transient	1.35· Lc1* +Lc30
53	permanent and transient	Lc1* +Lc31
54	permanent and transient	1.35· Lc1* +Lc31
55	permanent and transient	Lc1* +Lc32
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
1	153.82	4.40	0.00	0.00	0.00
2	153.82	4.40	0.00	0.00	0.00
3	207.66	5.94	0.00	0.00	-0.00
4	207.66	5.94	0.00	0.00	-0.00
5	228.82	13.40	0.00	0.00	-0.00
6	228.82	13.40	0.00	0.00	-0.00
7	282.66	14.94	0.00	-0.00	0.00
8	282.66	14.94	0.00	-0.00	0.00
9	153.82	4.40	22.50	-0.00	0.00
10	153.82	4.40	22.50	-0.00	0.00
11	207.66	5.94	22.49	-0.00	-0.00
12	207.66	5.94	22.49	-0.00	-0.00
13	228.82	13.40	22.50	-0.00	-0.00
14	228.82	13.40	22.50	-0.00	-0.00
15	282.66	14.94	22.49	-0.00	0.00
16	282.66	14.94	22.49	-0.00	0.00
17	153.82	31.40	0.00	0.00	-0.00
18	153.82	31.40	0.00	0.00	-0.00
19	207.66	32.94	0.00	0.00	-0.00
20	207.66	32.94	0.00	0.00	-0.00
21	228.82	40.40	0.00	0.00	-0.00
22	228.82	40.40	0.00	0.00	-0.00
23	282.66	41.94	0.00	0.00	-0.00
24	282.66	41.94	0.00	0.00	-0.00
25	153.82	31.40	22.50	-0.00	-0.00
26	153.82	31.40	22.50	-0.00	-0.00
27	207.66	32.94	22.49	-0.00	-0.00
28	207.66	32.94	22.49	-0.00	-0.00
29	228.82	40.40	22.50	-0.00	-0.00
30	228.82	40.40	22.50	-0.00	-0.00
31	282.66	41.94	22.49	-0.00	-0.00
32	282.66	41.94	22.49	-0.00	-0.00
33	153.82	4.40	37.49	-0.00	0.00
34	153.82	4.40	37.49	-0.00	0.00
35	207.66	5.94	37.49	-0.00	-0.00
36	207.66	5.94	37.49	-0.00	-0.00
37	228.82	13.40	37.49	-0.00	-0.00
38	228.82	13.40	37.49	-0.00	-0.00
39	282.66	14.94	37.49	0.00	0.00
40	282.66	14.94	37.49	0.00	0.00
41	153.82	49.40	0.00	0.00	-0.00
42	153.82	49.40	0.00	0.00	-0.00
43	207.66	50.94	0.00	0.00	-0.00
44	207.66	50.94	0.00	0.00	-0.00
45	228.82	58.40	0.00	-0.00	-0.00
46	228.82	58.40	0.00	-0.00	-0.00
47	282.66	59.93	0.00	0.00	-0.00
48	282.66	59.93	0.00	0.00	-0.00
49	153.82	49.40	37.49	-0.00	-0.00
50	153.82	49.40	37.49	-0.00	-0.00
51	207.66	50.94	37.49	-0.00	-0.00
52	207.66	50.94	37.49	-0.00	-0.00
53	228.82	58.40	37.49	-0.00	-0.00
54	228.82	58.40	37.49	-0.00	-0.00
55	282.66	59.93	37.49	0.00	-0.00
56	282.66	59.93	37.49	0.00	-0.00

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

4.3.1. minimum foundation height

$h_{min} = a \cdot (3 \cdot \sigma_{bm}/f_{ctd})^{0.5} \cdot 1/0.85 \geq a$ with $f_{ctd} = 0.85 \cdot f_{ctk;0.05}/\gamma_c$

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

distance between column and slab edge a = 60.0 cm

existing foundation height h_{exist} = 90.0 cm

LS	σ_{Bm} kN/m²	f_{ctd} N/mm²	h_{min} cm	h_{min}/h_{exist} -
1	47.47	1.15	60.0	**0.67**
2	47.47	1.15	60.0	0.67
3	64.09	1.15	60.0	0.67
4	64.09	1.15	60.0	0.67
5	70.62	1.15	60.0	0.67
6	70.62	1.15	60.0	0.67
7	87.24	1.15	60.0	0.67
8	87.24	1.15	60.0	0.67
9	47.47	1.15	60.0	0.67
10	47.47	1.15	60.0	0.67
11	64.09	1.15	60.0	0.67
12	64.09	1.15	60.0	0.67
13	70.62	1.15	60.0	0.67
14	70.62	1.15	60.0	0.67
15	87.24	1.15	60.0	0.67
16	87.24	1.15	60.0	0.67
17	47.47	1.15	60.0	0.67
18	47.47	1.15	60.0	0.67
19	64.09	1.15	60.0	0.67
20	64.09	1.15	60.0	0.67
21	70.62	1.15	60.0	0.67
22	70.62	1.15	60.0	0.67
23	87.24	1.15	60.0	0.67
24	87.24	1.15	60.0	0.67
25	47.47	1.15	60.0	0.67
26	47.47	1.15	60.0	0.67
27	64.09	1.15	60.0	0.67
28	64.09	1.15	60.0	0.67
29	70.62	1.15	60.0	0.67
30	70.62	1.15	60.0	0.67

LS	σ_{Bm} kN/m²	f_{ctd} N/mm²	h_{min} cm	h_{min}/h_{exist} -
31	87.24	1.15	60.0	0.67
32	87.24	1.15	60.0	0.67
33	47.47	1.15	60.0	0.67
34	47.47	1.15	60.0	0.67
35	64.09	1.15	60.0	0.67
36	64.09	1.15	60.0	0.67
37	70.62	1.15	60.0	0.67
38	70.62	1.15	60.0	0.67
39	87.24	1.15	60.0	0.67
40	87.24	1.15	60.0	0.67
41	47.47	1.15	60.0	0.67
42	47.47	1.15	60.0	0.67
43	64.09	1.15	60.0	0.67
44	64.09	1.15	60.0	0.67
45	70.62	1.15	60.0	0.67
46	70.62	1.15	60.0	0.67
47	87.24	1.15	60.0	0.67
48	87.24	1.15	60.0	0.67
49	47.47	1.15	60.0	0.67
50	47.47	1.15	60.0	0.67
51	64.09	1.15	60.0	0.67
52	64.09	1.15	60.0	0.67
53	70.62	1.15	60.0	0.67
54	70.62	1.15	60.0	0.67
55	87.24	1.15	60.0	0.67
56	87.24	1.15	60.0	0.67

$(h_{min}/h_{exist})_{max}$ = **0.67 < 1.0** ⇒ 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} = 0.360 m²

LS	A_{c2} m²	F_{Rdu} kN	N_{st}/F_{Rdu} -
1	2.250	15300.00	0.01
2	2.250	15300.00	0.01
3	2.250	15300.00	0.01
4	2.250	15300.00	0.01
5	2.250	15300.00	0.01
6	2.250	15300.00	0.01
7	2.250	15300.00	0.02
8	2.250	15300.00	0.02
9	2.250	15300.00	0.01
10	2.250	15300.00	0.01
11	2.250	15300.00	0.01
12	2.250	15300.00	0.01
13	2.250	15300.00	0.01
14	2.250	15300.00	0.01
15	2.250	15300.00	0.02
16	2.250	15300.00	0.02
17	2.250	15300.00	0.01
18	2.250	15300.00	0.01
19	2.250	15300.00	0.01
20	2.250	15300.00	0.01
21	2.250	15300.00	0.01
22	2.250	15300.00	0.01
23	2.250	15300.00	0.02
24	2.250	15300.00	0.02
25	2.250	15300.00	0.01
26	2.250	15300.00	0.01
27	2.250	15300.00	0.01
28	2.250	15300.00	0.01
29	2.250	15300.00	0.01
30	2.250	15300.00	0.01
31	2.250	15300.00	0.02
32	2.250	15300.00	0.02
33	2.250	15300.00	0.01
34	2.250	15300.00	0.01
35	2.250	15300.00	0.01
36	2.250	15300.00	0.01
37	2.250	15300.00	0.01
38	2.250	15300.00	0.01
39	2.250	15300.00	0.02
40	2.250	15300.00	0.02
41	2.250	15300.00	0.01
42	2.250	15300.00	0.01
43	2.250	15300.00	0.01
44	2.250	15300.00	0.01
45	2.250	15300.00	0.01
46	2.250	15300.00	0.01
47	2.250	15300.00	0.02
48	2.250	15300.00	0.02
49	2.250	15300.00	0.01
50	2.250	15300.00	0.01
51	2.250	15300.00	0.01
52	2.250	15300.00	0.01
53	2.250	15300.00	0.01
54	2.250	15300.00	0.01
55	2.250	15300.00	**0.02**
56	2.250	15300.00	0.02

$(N_{st}/F_{Rdu})_{max}$ = **0.02 < 1.0** ⇒ permissible partial area load complied with.

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

5.3.2. 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
1	138.44	3.96	0.00	-0.00	-0.00
2	169.20	4.84	0.00	-0.00	-0.00
3	213.44	12.96	0.00	0.00	-0.00
4	244.20	13.84	0.00	0.00	-0.00
5	138.44	3.96	22.50	0.00	-0.00
6	169.20	4.84	22.50	0.00	-0.00
7	213.44	12.96	22.50	0.00	-0.00
8	244.20	13.84	22.50	0.00	-0.00
9	138.44	30.96	0.00	-0.00	-0.00
10	169.20	31.84	0.00	-0.00	-0.00
11	213.44	39.96	0.00	0.00	-0.00
12	244.20	40.84	0.00	0.00	-0.00
13	138.44	30.96	22.50	0.00	-0.00
14	169.20	31.84	22.50	0.00	-0.00
15	213.44	39.96	22.50	0.00	-0.00
16	244.20	40.84	22.50	0.00	-0.00
17	138.44	3.96	37.50	0.00	-0.00
18	169.20	4.84	37.50	0.00	-0.00
19	213.44	12.96	37.50	0.00	-0.00
20	244.20	13.84	37.50	0.00	-0.00
21	138.44	48.96	0.00	-0.00	-0.00
22	169.20	49.84	0.00	-0.00	-0.00
23	213.44	57.96	0.00	0.00	-0.00
24	244.20	58.84	0.00	0.00	-0.00
25	138.44	48.96	37.50	0.00	-0.00
26	169.20	49.84	37.50	0.00	-0.00
27	213.44	57.96	37.50	0.00	-0.00
28	244.20	58.84	37.50	0.00	-0.00

5.4. verification against overturning (EQU)

LS	decisive edge	M_{dst} kNm	M_{stb} kNm	μ -
1	---	0.00	187.21	0.00
2	---	0.00	228.81	0.00
3	---	0.00	262.81	0.00
4	---	0.00	304.41	0.00
5	x (bottom)	20.25	183.64	0.11
6	x (bottom)	20.25	224.45	0.09
7	x (bottom)	20.25	251.14	0.08
8	x (bottom)	20.25	291.95	0.07
9	y (right)	24.30	180.08	0.13
10	y (right)	24.30	220.10	0.11
11	y (right)	24.30	239.48	0.10
12	y (right)	24.30	279.50	0.09
13	y (right)	24.30	180.08	0.13
14	y (right)	24.30	220.10	0.11
15	y (right)	24.30	239.48	0.10
16	y (right)	24.30	279.50	0.09
17	x (bottom)	33.75	183.64	0.18
18	x (bottom)	33.75	224.45	0.15
19	x (bottom)	33.75	251.14	0.13
20	x (bottom)	33.75	291.95	0.12
21	y (right)	40.50	180.08	**0.22**
22	y (right)	40.50	220.10	0.18
23	y (right)	40.50	239.48	0.17
24	y (right)	40.50	279.50	0.14
25	y (right)	40.50	180.08	0.22
26	y (right)	40.50	220.10	0.18
27	y (right)	40.50	239.48	0.17
28	y (right)	40.50	279.50	0.14

$\mu_{max} = 0.22 < 1.0$ $\Rightarrow$ **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
1	BS-P	Lc1
2	BS-P	1.35· Lc1
3	BS-P	Lc1+1.5· Lc2
4	BS-P	1.35· Lc1+1.5· Lc2
5	BS-P	Lc1+0.6· 1.5· Lc3
6	BS-P	1.35· Lc1+0.6· 1.5· Lc3
7	BS-P	Lc1+1.5· Lc2+0.6· 1.5· Lc3
8	BS-P	1.35· Lc1+1.5· Lc2+0.6· 1.5· Lc3
9	BS-P	Lc1+0.6· 1.5· Lc4
10	BS-P	1.35· Lc1+0.6· 1.5· Lc4
11	BS-P	Lc1+1.5· Lc2+0.6· 1.5· Lc4
12	BS-P	1.35· Lc1+1.5· Lc2+0.6· 1.5· Lc4
13	BS-P	Lc1+0.6· 1.5· (Lc3+Lc4)
14	BS-P	1.35· Lc1+0.6· 1.5· (Lc3+Lc4)
15	BS-P	Lc1+1.5· Lc2+0.6· 1.5· (Lc3+Lc4)
16	BS-P	1.35· Lc1+1.5· Lc2+0.6· 1.5· (Lc3+Lc4)
17	BS-P	Lc1+1.5· Lc3
18	BS-P	1.35· Lc1+1.5· Lc3
19	BS-P	Lc1+1.5· (Lc2+Lc3)
20	BS-P	1.35· Lc1+1.5· (Lc2+Lc3)
21	BS-P	Lc1+1.5· Lc4
22	BS-P	1.35· Lc1+1.5· Lc4
23	BS-P	Lc1+1.5· (Lc2+Lc4)
24	BS-P	1.35· Lc1+1.5· (Lc2+Lc4)
25	BS-P	Lc1+1.5· (Lc3+Lc4)
26	BS-P	1.35· Lc1+1.5· (Lc3+Lc4)
27	BS-P	Lc1+1.5· (Lc2+Lc3+Lc4)
28	BS-P	1.35· Lc1+1.5· (Lc2+Lc3+Lc4)

5.5.2. 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	153.82	4.40	0.00	-0.00	-0.00
2	207.66	5.94	0.00	-0.00	-0.00
3	228.82	13.40	0.00	0.00	-0.00
4	282.66	14.94	0.00	0.00	-0.00
5	153.82	4.40	22.50	0.00	-0.00
6	207.66	5.94	22.50	0.00	-0.00
7	228.82	13.40	22.50	0.00	-0.00
8	282.66	14.94	22.50	0.00	-0.00
9	153.82	31.40	0.00	-0.00	-0.00
10	207.66	32.94	0.00	-0.00	-0.00
11	228.82	40.40	0.00	0.00	-0.00
12	282.66	41.94	0.00	0.00	-0.00
13	153.82	31.40	22.50	0.00	-0.00
14	207.66	32.94	22.50	0.00	-0.00
15	228.82	40.40	22.50	0.00	-0.00
16	282.66	41.94	22.50	0.00	-0.00
17	153.82	4.40	37.50	0.00	-0.00
18	207.66	5.94	37.50	0.00	-0.00
19	228.82	13.40	37.50	0.00	-0.00
20	282.66	14.94	37.50	0.00	-0.00
21	153.82	49.40	0.00	-0.00	-0.00
22	207.66	50.94	0.00	-0.00	-0.00
23	228.82	58.40	0.00	0.00	-0.00
24	282.66	59.94	0.00	0.00	-0.00
25	153.82	49.40	37.50	0.00	-0.00
26	207.66	50.94	37.50	0.00	-0.00
27	228.82	58.40	37.50	0.00	-0.00
28	282.66	59.94	37.50	0.00	-0.00

associated characteristic values

LS	$N_{St,k}$ kN	$H_{x,St,k}$ kN	$H_{y,St,k}$ kN	$M_{x,St,k}$ kNm	$M_{y,St,k}$ kNm
1	153.82	4.40	0.00	-0.00	-0.00
2	153.82	4.40	0.00	-0.00	-0.00
3	203.82	10.40	0.00	0.00	-0.00
4	203.82	10.40	0.00	0.00	-0.00
5	153.82	4.40	25.00	0.00	-0.00
6	153.82	4.40	25.00	0.00	-0.00
7	203.82	10.40	25.00	0.00	-0.00
8	203.82	10.40	25.00	0.00	-0.00
9	153.82	34.40	0.00	-0.00	-0.00
10	153.82	34.40	0.00	-0.00	-0.00
11	203.82	40.40	0.00	0.00	-0.00
12	203.82	40.40	0.00	0.00	-0.00
13	153.82	34.40	25.00	0.00	-0.00
14	153.82	34.40	25.00	0.00	-0.00
15	203.82	40.40	25.00	0.00	-0.00
16	203.82	40.40	25.00	0.00	-0.00
17	153.82	4.40	25.00	0.00	-0.00
18	153.82	4.40	25.00	0.00	-0.00
19	203.82	10.40	25.00	0.00	-0.00
20	203.82	10.40	25.00	0.00	-0.00
21	153.82	34.40	0.00	-0.00	-0.00
22	153.82	34.40	0.00	-0.00	-0.00
23	203.82	40.40	0.00	0.00	-0.00
24	203.82	40.40	0.00	0.00	-0.00
25	153.82	34.40	25.00	0.00	-0.00
26	153.82	34.40	25.00	0.00	-0.00
27	203.82	40.40	25.00	0.00	-0.00
28	203.82	40.40	25.00	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	226.72	0.00	-2.79	1.80	1.78	0.00	0.00
2	226.72	0.00	-2.79	1.80	1.78	0.00	0.00
3	276.72	0.00	-6.59	1.80	1.75	0.00	0.00
4	276.72	0.00	-6.59	1.80	1.75	0.00	0.00
5	226.72	15.83	-2.79	1.78	1.66	0.00	0.00
6	226.72	15.83	-2.79	1.78	1.66	0.00	0.00
7	276.72	15.83	-6.59	1.75	1.69	0.00	0.00
8	276.72	15.83	-6.59	1.75	1.69	0.00	0.00
9	226.72	0.00	-21.79	1.80	1.61	0.00	0.00
10	226.72	0.00	-21.79	1.80	1.61	0.00	0.00
11	276.72	0.00	-25.92	1.80	1.61	0.00	1.27
12	276.72	0.00	-25.92	1.80	1.61	0.00	1.27
13	226.72	15.83	-21.79	1.66	1.61	0.00	0.00
14	226.72	15.83	-21.79	1.66	1.61	0.00	0.00
15	276.72	15.83	-25.92	1.69	1.61	0.00	1.27
16	276.72	15.83	-25.92	1.69	1.61	0.00	1.27
17	226.72	15.83	-2.79	1.78	1.66	0.00	0.00
18	226.72	15.83	-2.79	1.78	1.66	0.00	0.00
19	276.72	15.83	-6.59	1.75	1.69	0.00	0.00
20	276.72	15.83	-6.59	1.75	1.69	0.00	0.00
21	226.72	0.00	-21.79	1.80	1.61	0.00	0.00
22	226.72	0.00	-21.79	1.80	1.61	0.00	0.00
23	276.72	0.00	-25.92	1.80	1.61	0.00	1.27
24	276.72	0.00	-25.92	1.80	1.61	0.00	1.27
25	226.72	15.83	-21.79	1.66	1.61	0.00	0.00
26	226.72	15.83	-21.79	1.66	1.61	0.00	0.00
27	276.72	15.83	-25.92	1.69	1.61	0.00	1.27
28	276.72	15.83	-25.92	1.69	1.61	0.00	1.27

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/m³	φ_1 °	c_1 kN/m²	d_s m	γ_2 kN/m³	φ_2 °	c_2 kN/m²
1	19.00	32.50	---	3.08	15.42	32.50	---
2	19.00	32.50	---	3.08	15.42	32.50	---
3	19.00	32.50	---	3.04	15.48	32.50	---
4	19.00	32.50	---	3.04	15.48	32.50	---
5	19.00	32.50	---	2.88	15.73	32.50	---
6	19.00	32.50	---	2.88	15.73	32.50	---
7	19.00	32.50	---	2.92	15.66	32.50	---
8	19.00	32.50	---	2.92	15.66	32.50	---
9	19.00	32.50	---	2.79	15.88	32.50	---
10	19.00	32.50	---	2.79	15.88	32.50	---
11	19.00	32.50	---	2.77	15.91	32.50	---
12	19.00	32.50	---	2.77	15.91	32.50	---
13	19.00	32.50	---	2.79	15.88	32.50	---
14	19.00	32.50	---	2.79	15.88	32.50	---
15	19.00	32.50	---	2.77	15.91	32.50	---
16	19.00	32.50	---	2.77	15.91	32.50	---
17	19.00	32.50	---	2.88	15.73	32.50	---
18	19.00	32.50	---	2.88	15.73	32.50	---
19	19.00	32.50	---	2.92	15.66	32.50	---
20	19.00	32.50	---	2.92	15.66	32.50	---
21	19.00	32.50	---	2.79	15.88	32.50	---
22	19.00	32.50	---	2.79	15.88	32.50	---
23	19.00	32.50	---	2.77	15.91	32.50	---
24	19.00	32.50	---	2.77	15.91	32.50	---
25	19.00	32.50	---	2.79	15.88	32.50	---
26	19.00	32.50	---	2.79	15.88	32.50	---
27	19.00	32.50	---	2.77	15.91	32.50	---
28	19.00	32.50	---	2.77	15.91	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 ν_b,ν_d,ν_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} -	ν_b -	ν_d -	ν_c -	i_b -	i_d -	i_c -
1	15.03	24.58	---	0.704	1.530	---	1.000	1.000	---
2	15.03	24.58	---	0.704	1.530	---	1.000	1.000	---
3	15.03	24.58	---	0.708	1.523	---	1.000	1.000	---
4	15.03	24.58	---	0.708	1.523	---	1.000	1.000	---
5	15.03	24.58	---	0.719	1.502	---	1.000	1.000	---
6	15.03	24.58	---	0.719	1.502	---	1.000	1.000	---
7	15.03	24.58	---	0.711	1.517	---	1.000	1.000	---
8	15.03	24.58	---	0.711	1.517	---	1.000	1.000	---
9	15.03	24.58	---	0.732	1.480	---	1.000	1.000	---
10	15.03	24.58	---	0.732	1.480	---	1.000	1.000	---
11	15.03	24.58	---	0.731	1.481	---	0.988	0.993	---
12	15.03	24.58	---	0.731	1.481	---	0.988	0.993	---
13	15.03	24.58	---	0.709	1.520	---	1.000	1.000	---
14	15.03	24.58	---	0.709	1.520	---	1.000	1.000	---
15	15.03	24.58	---	0.713	1.514	---	0.989	0.993	---
16	15.03	24.58	---	0.713	1.514	---	0.989	0.993	---
17	15.03	24.58	---	0.719	1.502	---	1.000	1.000	---
18	15.03	24.58	---	0.719	1.502	---	1.000	1.000	---
19	15.03	24.58	---	0.711	1.517	---	1.000	1.000	---
20	15.03	24.58	---	0.711	1.517	---	1.000	1.000	---
21	15.03	24.58	---	0.732	1.480	---	1.000	1.000	---
22	15.03	24.58	---	0.732	1.480	---	1.000	1.000	---
23	15.03	24.58	---	0.731	1.481	---	0.988	0.993	---
24	15.03	24.58	---	0.731	1.481	---	0.988	0.993	---
25	15.03	24.58	---	0.709	1.520	---	1.000	1.000	---
26	15.03	24.58	---	0.709	1.520	---	1.000	1.000	---
27	15.03	24.58	---	0.713	1.514	---	0.989	0.993	---
28	15.03	24.58	---	0.713	1.514	---	0.989	0.993	---

5.6.4. ultimate load and allowable load

characteristic design bearing capacity $R_{n,k} = a' \cdot b' \cdot (\gamma_2 \cdot b' \cdot N_{b0} \cdot \nu_b \cdot i_b + \gamma_1 \cdot t \cdot N_{d0} \cdot \nu_d \cdot i_d + c_2 \cdot N_{c0} \cdot \nu_c \cdot i_c)$
design value of resistance $R_{n,d} = R_{n,k}/\gamma_{Gr}$
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 -
1	2752.76	1.40	1966.26	226.72	0.12
2	2752.76	1.40	1966.26	306.07	0.16
3	2705.45	1.40	1932.46	301.72	0.16
4	2705.45	1.40	1932.46	381.07	0.20
5	2487.15	1.40	1776.53	226.72	0.13
6	2487.15	1.40	1776.53	306.07	0.17
7	2507.52	1.40	1791.09	301.72	0.17
8	2507.52	1.40	1791.09	381.07	0.21
9	2413.35	1.40	1723.82	226.72	0.13
10	2413.35	1.40	1723.82	306.07	0.18
11	2404.28	1.40	1717.34	301.72	0.18
12	2404.28	1.40	1717.34	381.07	0.22
13	2243.27	1.40	1602.33	226.72	0.14
14	2243.27	1.40	1602.33	306.07	0.19
15	2265.66	1.40	1618.33	301.72	0.19
16	2265.66	1.40	1618.33	381.07	**0.24**
17	2487.15	1.40	1776.53	226.72	0.13
18	2487.15	1.40	1776.53	306.07	0.17
19	2507.52	1.40	1791.09	301.72	0.17
20	2507.52	1.40	1791.09	381.07	0.21
21	2413.35	1.40	1723.82	226.72	0.13
22	2413.35	1.40	1723.82	306.07	0.18
23	2404.28	1.40	1717.34	301.72	0.18
24	2404.28	1.40	1717.34	381.07	0.22
25	2243.27	1.40	1602.33	226.72	0.14
26	2243.27	1.40	1602.33	306.07	0.19
27	2265.66	1.40	1618.33	301.72	0.19
28	2265.66	1.40	1618.33	381.07	0.24

$\mu_{max} = 0.24 < 1.0$ $\Rightarrow$ **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) $\delta_s = 32.5°$

LS	$N_{0,k}$ kN	$R_{t,k}$ kN	$\gamma_{R,h}$ -	$\gamma_{R,e}$ -	$R_{t,d}$ kN	$E_{p,d}$ kN	$H_{Res,d}$ kN	μ -
1	226.72	144.44	1.10	1.40	131.31	3.14	4.40	0.03
2	226.72	144.44	1.10	1.40	131.31	4.24	5.94	0.04
3	276.72	176.29	1.10	1.40	160.26	9.57	13.40	0.08
4	276.72	176.29	1.10	1.40	160.26	10.67	14.94	0.09
5	226.72	144.44	1.10	1.40	131.31	16.38	22.93	0.16
6	226.72	144.44	1.10	1.40	131.31	16.62	23.27	0.16
7	276.72	176.29	1.10	1.40	160.26	18.71	26.19	0.15
8	276.72	176.29	1.10	1.40	160.26	19.29	27.01	0.15
9	226.72	144.44	1.10	1.40	131.31	22.43	31.40	0.20
10	226.72	144.44	1.10	1.40	131.31	23.53	32.94	0.21
11	276.72	176.29	1.10	1.40	160.26	28.86	40.40	0.21
12	276.72	176.29	1.10	1.40	160.26	29.96	41.94	0.22
13	226.72	144.44	1.10	1.40	131.31	27.59	38.63	0.24
14	226.72	144.44	1.10	1.40	131.31	28.49	39.89	0.25
15	276.72	176.29	1.10	1.40	160.26	33.03	46.24	0.24
16	276.72	176.29	1.10	1.40	160.26	34.00	47.59	0.25
17	226.72	144.44	1.10	1.40	131.31	26.97	37.76	0.24
18	226.72	144.44	1.10	1.40	131.31	27.12	37.97	0.24
19	276.72	176.29	1.10	1.40	160.26	28.44	39.82	0.21
20	276.72	176.29	1.10	1.40	160.26	28.83	40.37	0.21
21	226.72	144.44	1.10	1.40	131.31	35.29	49.40	0.30
22	226.72	144.44	1.10	1.40	131.31	36.39	50.94	0.30
23	276.72	176.29	1.10	1.40	160.26	41.71	58.40	0.29
24	276.72	176.29	1.10	1.40	160.26	42.81	59.94	0.30
25	226.72	144.44	1.10	1.40	131.31	44.30	62.02	0.35
26	226.72	144.44	1.10	1.40	131.31	45.18	63.25	**0.36**
27	276.72	176.29	1.10	1.40	160.26	49.57	69.40	0.33
28	276.72	176.29	1.10	1.40	160.26	50.50	70.70	0.34

$\mu_{max} = 0.36 < 1.0$ $\Rightarrow$ **slip resistance sufficient**

6. external stability - verification of serviceability

6.1. design values limitation of gaping 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	153.82	4.40	0.00	-0.00	-0.00

6.2. limitation of gaping joint under permanent load

int. f. and mom. in centroid of foundation base: $N_{0,k}$ = 226.72 kN
$M_{0,x,k}$ = 0.00 kNm
$M_{0,y,k}$ = -3.96 kNm

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

$e_x/b_x + e_y/b_y = 0.01 < 1/6$

⇒ the resultant is located in the 1. core area
sc. no emerge of a gaping joint due to permanent load.

6.3. design values limitation of gaping 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
1	153.82	4.40	0.00	-0.00	-0.00
2	203.82	10.40	0.00	0.00	-0.00
3	153.82	4.40	25.00	0.00	-0.00
4	203.82	10.40	25.00	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
5	153.82	34.40	0.00	-0.00	-0.00
6	203.82	40.40	0.00	0.00	-0.00
7	153.82	34.40	25.00	0.00	-0.00
8	203.82	40.40	25.00	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	226.72	0.00	-3.96	-0.02	0.00	0.000
2	276.72	0.00	-9.36	-0.03	0.00	0.000
3	226.72	22.50	-3.96	-0.02	0.10	0.003
4	276.72	22.50	-9.36	-0.03	0.08	0.002
5	226.72	0.00	-30.96	-0.14	0.00	0.006
6	276.72	0.00	-36.36	-0.13	0.00	0.005
7	226.72	22.50	-30.96	-0.14	0.10	0.009
8	276.72	22.50	-36.36	-0.13	0.08	0.007

$((e_x/b_x)^2 + (e_y/b_y)^2)_{max} = 0.009 < 1/9$

⇒ 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	$\gamma_{R,h}$ -	$R_{t,d}$ kN	$H_{Res,d}$ kN	μ -
1	226.72	144.44	1.10	131.31	4.40	0.03
2	226.72	144.44	1.10	131.31	5.94	0.05
3	276.72	176.29	1.10	160.26	13.40	0.08
4	276.72	176.29	1.10	160.26	14.94	0.09
5	226.72	144.44	1.10	131.31	22.93	0.17
6	226.72	144.44	1.10	131.31	23.27	0.18
7	276.72	176.29	1.10	160.26	26.19	0.16
8	276.72	176.29	1.10	160.26	27.01	0.17

LS	$N_{0,k}$ kN	$R_{t,k}$ kN	$\gamma_{R,h}$ -	$R_{t,d}$ kN	$H_{Res,d}$ kN	μ -
9	226.72	144.44	1.10	131.31	31.40	0.24
10	226.72	144.44	1.10	131.31	32.94	0.25
11	276.72	176.29	1.10	160.26	40.40	0.25
12	276.72	176.29	1.10	160.26	41.94	0.26
13	226.72	144.44	1.10	131.31	38.63	0.29
14	226.72	144.44	1.10	131.31	39.89	0.30
15	276.72	176.29	1.10	160.26	46.24	0.29
16	276.72	176.29	1.10	160.26	47.59	0.30
17	226.72	144.44	1.10	131.31	37.76	0.29
18	226.72	144.44	1.10	131.31	37.97	0.29
19	276.72	176.29	1.10	160.26	39.82	0.25
20	276.72	176.29	1.10	160.26	40.37	0.25
21	226.72	144.44	1.10	131.31	49.40	0.38
22	226.72	144.44	1.10	131.31	50.94	0.39
23	276.72	176.29	1.10	160.26	58.40	0.36
24	276.72	176.29	1.10	160.26	59.94	0.37
25	226.72	144.44	1.10	131.31	62.02	0.47
26	226.72	144.44	1.10	131.31	63.25	**0.48**
27	276.72	176.29	1.10	160.26	69.40	0.43
28	276.72	176.29	1.10	160.26	70.70	0.44

μ_{max} = 0.48 < 1.0 ⇒ **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
1	153.82	4.40	0.00	-0.00	-0.00
2	203.82	10.40	0.00	0.00	-0.00
3	153.82	4.40	25.00	0.00	-0.00
4	203.82	10.40	25.00	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
5	153.82	34.40	0.00	-0.00	-0.00
6	203.82	40.40	0.00	0.00	-0.00
7	153.82	34.40	25.00	0.00	-0.00
8	203.82	40.40	25.00	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 α_x = 0.5 °
allowable obliquity about the y-axis perm α_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 $2 \cdot \sigma_a > \sigma_0$ then $\sigma_0' = \sigma_0$
the limiting depth d_s results from $d_s = z$, if $\sigma_B(z) = 0.2 \cdot \sigma_ü(z)$ below significant point.

unloading from excavation due to foundation depth σ_a = 15.20 kN/m²

LS	$N_{0,k}$ kN	$M_{0,x,k}$ kNm	$M_{0,y,k}$ kNm	σ_0 kN/m²	σ_0' kN/m²	d_s m
1	226.72	0.00	-3.96	69.97	54.78	2.16
2	276.72	0.00	-9.36	85.41	70.21	2.49
3	226.72	22.50	-3.96	69.97	54.78	2.16
4	276.72	22.50	-9.36	85.41	70.21	2.49
5	226.72	0.00	-30.96	69.97	54.78	2.16
6	276.72	0.00	-36.36	85.41	70.21	2.49
7	226.72	22.50	-30.96	69.97	54.78	2.16
8	276.72	22.50	-36.36	85.41	70.21	2.49

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 centrical 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:
σ_0' = 54.77 kN/m^2
$M_{0,x}$ = 0.00 kNm
$M_{0,y}$ = -3.96 kNm

level	z	f	f_x	f_y	s_m	s_x	s_y
m	m	-	-	-	cm	cm	cm
2.50	1.70	0.475	3.545	3.545	0.47	-0.02	0.00
2.96	2.16	0.530	3.673	3.673	0.05	-0.00	0.00

LS 2:
σ_0' = 70.21 kN/m^2
$M_{0,x}$ = 0.00 kNm
$M_{0,y}$ = -9.36 kNm

level	z	f	f_x	f_y	s_m	s_x	s_y
m	m	-	-	-	cm	cm	cm
2.50	1.70	0.475	3.545	3.545	0.60	-0.05	0.00
3.29	2.49	0.563	3.726	3.726	0.11	-0.00	0.00

LS 3:
σ_0' = 54.77 kN/m^2
$M_{0,x}$ = 22.50 kNm
$M_{0,y}$ = -3.96 kNm

level	z	f	f_x	f_y	s_m	s_x	s_y
m	m	-	-	-	cm	cm	cm
2.50	1.70	0.475	3.545	3.545	0.47	-0.02	0.12
2.96	2.16	0.530	3.673	3.673	0.05	-0.00	0.00

LS 4:
σ_0' = 70.21 kN/m^2
$M_{0,x}$ = 22.50 kNm
$M_{0,y}$ = -9.36 kNm

level	z	f	f_x	f_y	s_m	s_x	s_y
m	m	-	-	-	cm	cm	cm
2.50	1.70	0.475	3.545	3.545	0.60	-0.05	0.12
3.29	2.49	0.563	3.726	3.726	0.11	-0.00	0.01

LS 5:
σ_0' = 54.77 kN/m^2
$M_{0,x}$ = 0.00 kNm
$M_{0,y}$ = -30.96 kNm

level	z	f	f_x	f_y	s_m	s_x	s_y
m	m	-	-	-	cm	cm	cm
2.50	1.70	0.475	3.545	3.545	0.47	-0.17	0.00
2.96	2.16	0.530	3.673	3.673	0.05	-0.01	0.00

LS 6:
σ_0' = 70.21 kN/m^2
$M_{0,x}$ = 0.00 kNm
$M_{0,y}$ = -36.36 kNm

level	z	f	f_x	f_y	s_m	s_x	s_y
m	m	-	-	-	cm	cm	cm
2.50	1.70	0.475	3.545	3.545	0.60	-0.20	0.00
3.29	2.49	0.563	3.726	3.726	0.11	-0.01	0.00

LS 7:
σ_0' = 54.77 kN/m^2
$M_{0,x}$ = 22.50 kNm
$M_{0,y}$ = -30.96 kNm

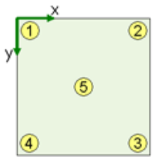
level	z	f	f_x	f_y	s_m	s_x	s_y
m	m	-	-	-	cm	cm	cm
2.50	1.70	0.475	3.545	3.545	0.47	-0.17	0.12
2.96	2.16	0.530	3.673	3.673	0.05	-0.01	0.00

LS 8:
σ_0' = 70.21 kN/m^2
$M_{0,x}$ = 22.50 kNm
$M_{0,y}$ = -36.36 kNm

level	z	f	f_x	f_y	s_m	s_x	s_y
m	m	-	-	-	cm	cm	cm
2.50	1.70	0.475	3.545	3.545	0.60	-0.20	0.12
3.29	2.49	0.563	3.726	3.726	0.11	-0.01	0.01

6.7.3. resultant settlements and obliquity per LK

$s_1 = \Sigma(s_{m,i} + s_{x,i} - s_{y,i})$ $s_2 = \Sigma(s_{m,i} - s_{x,i} - s_{y,i})$ $s_3 = \Sigma(s_{m,i} - s_{x,i} + s_{y,i})$ $s_4 = \Sigma(s_{m,i} + s_{x,i} + s_{y,i})$ $s_5 = \Sigma s_{m,i}$
$\tan \alpha_x = 2 \cdot \Sigma s_{y,i}/b_y$ $\tan \alpha_y = 2 \cdot \Sigma s_{x,i}/b_x$



LS	s_1	s_2	s_3	s_4	s_5	s_{max}	α_x	α_y
	cm	cm	cm	cm	cm	cm	°	°
1	0.5	0.5	0.5	0.5	0.5	0.5	0.0	-0.0
2	0.7	0.8	0.8	0.7	0.7	0.8	0.0	-0.0
3	0.4	0.4	0.7	0.6	0.5	0.7	0.1	-0.0
4	0.5	0.6	0.9	0.8	0.7	0.9	**0.1**	-0.0
5	0.3	0.7	0.7	0.3	0.5	0.7	0.0	-0.1
6	0.5	0.9	0.9	0.5	0.7	0.9	0.0	**-0.1**
7	0.2	0.6	0.8	0.5	0.5	0.8	0.1	-0.1
8	0.4	0.8	1.0	0.6	0.7	**1.0**	0.1	-0.1

max s_{max} = 1.0 < 5.0 cm   max $|\alpha_x|$ = 0.1° < 0.5°   max $|\alpha_y|$ = 0.1° < 0.5°

⇒ allowable settlement and obliquity kept

M_{dst} - destabilising moment M_{stb} - stabilising moment N_0 - normal force in foundation joint
M_0 - 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 σ_0 - mean normal soil stress
σ_B - soil stress from structural load $\sigma_{\ddot{u}}$ - 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. torison 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]

$k_S = E_S / (f \cdot (b_X \cdot b_X)^{0.5})$

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 $\kappa = 1$

constrained modulus $E_s = 1 \cdot 10000.00 = 10000.00$ kN/m²

shape factor f = 0.45

foundation modulus $k_s = 12345.68$ kN/m³

moment of inertia I_x / I_y = 0.87 / 0.87 kN/m³

torsional spring about the x-axis **$c_{v,x}$ = 10800.00 kNm**

torsional spring about the y-axis **$c_{v,y}$ = 10800.00 kNm**

8. Summary of foundation design

all executed verifications and design calculations successful.

overturning	μ_{max}	= 0.22
base failure	μ_{max}	= 0.24
slip	μ_{max}	= 0.36
gaping joint under permant load	μ_{exist}	= 0.06
gaping joint under total load	μ_{max}	= 0.08
displacement in the base area	μ_{max}	= 0.48
settlement	s_{max}	= 1.0 < 5.0 cm
obliquity (about x-axis)	$\alpha_{max,x}$	= 0.1 < 0.5 °
obliquity (about y-axis)	$\alpha_{max,y}$	= 0.1 < 0.5 °
torsional spring (about the x-axis)	$c_{v,x}$	= 10800 kNm
torsional spring (about the y-axis)	$c_{v,y}$	= 10800 kNm

literature and standards:

[1] DIN 4085: Baugrund, Berechnung des Erddrucks, August 2017
[2] DIN V 4141-1: Lager im Bauwesen, Teil 1, Mai 2003
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[4] DIN 1054: Baugrund - Sicherheitsnachweise im Erd- und Grundbau - Ergänzende Regeln zu DIN EN 1997-1, April 2021
[5] DIN 4017: Baugrund, Berechnung des Grundbruchwiderstandes von Flächengründungen, März 2006
[6] DIN 4019: Baugrund - Setzungsberechnungen, Januar 2014
[7] Kany, M.: Berechnung von Flächengründungen, Verlag von Wilhelm Ernst & Sohn, 2.Aufl. 1974
[8] Sherif, G.; König, G.: Platten und Balken auf nachgiebigem Baugrund, Springer, 1975
[9] Rausch, E.: Maschinenfundamente, Betonkalender 1973, Teil 2, Ernst & Sohn