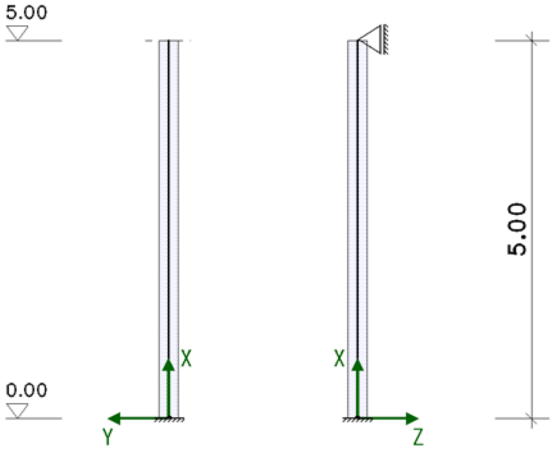
4H-STAST version: 4/2025-1a

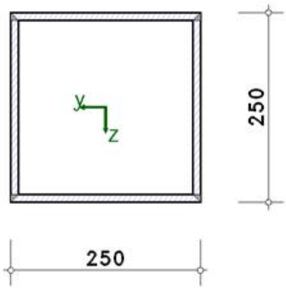
steel column

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 S355
dimensions of cross-section (user-defined, rectangular tube, welded, thermoformed):
profile depth h = 250.0 mm
flange width b = 250.0 mm
web thickness t_w = 10.0 mm
flange thickness t_f = 10.0 mm
butt weld (full penetration, DHV-weld)

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	----	----
foot	fix	fix	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

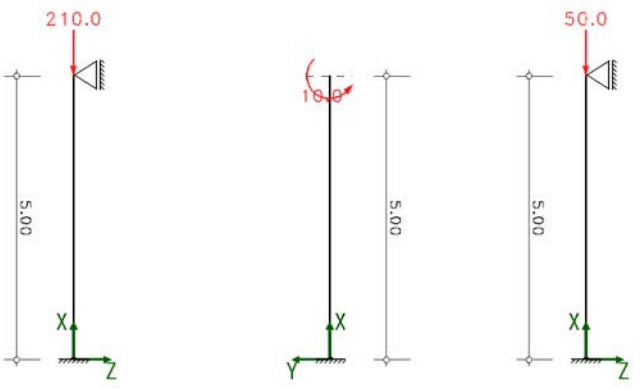
1: permanent loads	**permanent loads**
1: dead load	additive
4: live load	**transient live loads of storage rooms**
2: live load	additive
9: wind	**transient wind load**
3: Y-direction	alternative in group A
4: Z-direction	alternative in group A

1.2. table of load pictures

loadc.	loadpic.	introduct.	direction	value	entity
1	pointload	head	N	210.00	kN
2	pointload	head	N	50.00	kN
			M_z	10.00	kNm
3	line load full height		q_y	8.00	kN/m
4	line load full height		q_z	15.00	kN/m

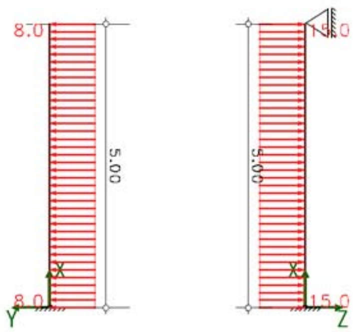
1.2.1. graphics of point loads

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



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.

1.4. load sums

Σ N = 275.70 kN Σ H_y = 40.00 kN Σ H_z = 75.00 kN

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: $\sigma_{x,Rd}$ = 322.7 N/mm², τ_{Rd} = 186.3 N/mm², $\sigma_{v,Rd}$ = 322.7 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	obliquity	2.00	10.00	0.0045	2.235	19.39	1.92
Z	curvat.	0.71	3.53	0.0071	0.000	19.39	15.32

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

2.1.2. direction of imperfection

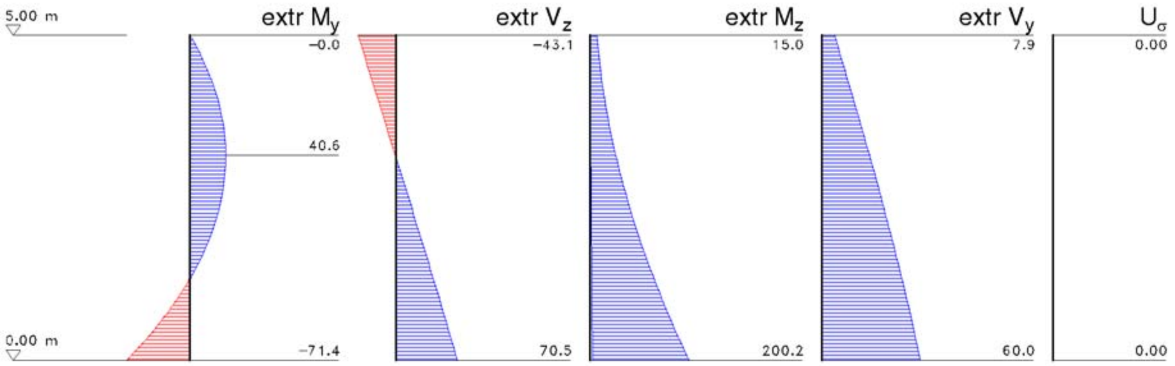
$N_{Ki,z}/N_{Ki,y}$ = 8.00 > 3 $\Rightarrow$ always in the direction of the lowest buckling safety (Y-axis)

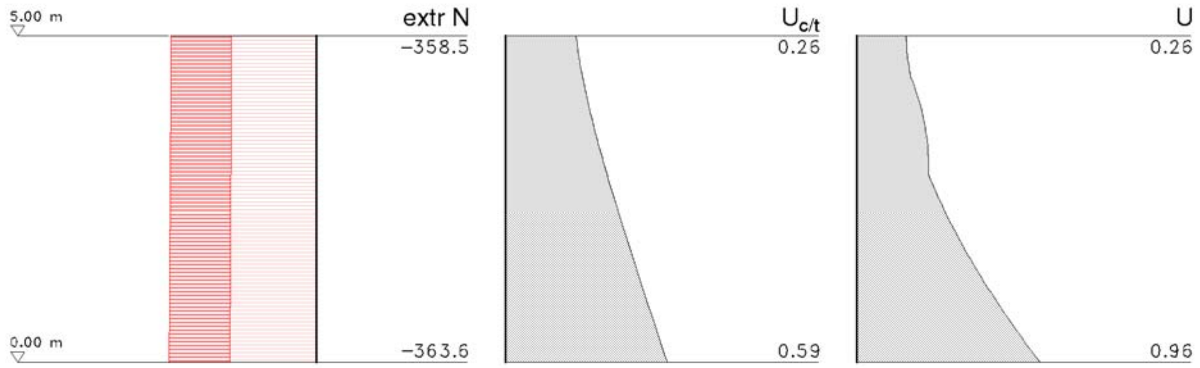
2.2. factorization of load case combinations

LS	design situat.	factorization	LS	design situat.	factorization
1	perman.	Lc1+I_y	11	perman.	Lc1+1.5· Lc2+0.6· 1.5· Lc4+I_y
2	perman.	1.35· Lc1+I_y	12	perman.	1.35· Lc1+1.5· Lc2+0.6· 1.5· Lc4+I_y
3	perman.	Lc1+1.5· Lc2+I_y	13	perman.	Lc1+1.5· Lc3+I_y
4	perman.	1.35· Lc1+1.5· Lc2+I_y	14	perman.	1.35· Lc1+1.5· Lc3+I_y
5	perman.	Lc1+0.6· 1.5· Lc3+I_y	15	perman.	Lc1+1.5· (Lc2+Lc3)+I_y
6	perman.	1.35· Lc1+0.6· 1.5· Lc3+I_y	16	perman.	1.35· Lc1+1.5· (Lc2+Lc3)+I_y
7	perman.	Lc1+1.5· Lc2+0.6· 1.5· Lc3+I_y	17	perman.	Lc1+1.5· Lc4+I_y
8	perman.	1.35· Lc1+1.5· Lc2+0.6· 1.5· Lc3+I_y	18	perman.	1.35· Lc1+1.5· Lc4+I_y
9	perman.	Lc1+0.6· 1.5· Lc4+I_y	19	perman.	Lc1+1.5· (Lc2+Lc4)+I_y
10	perman.	1.35· Lc1+0.6· 1.5· Lc4+I_y	20	perman.	1.35· Lc1+1.5· (Lc2+Lc4)+I_y

2.3. extreme results

the internal forces are related to the deformed system axis





x	N		M_y		V_z		M_z		V_y		U
	min	max	min	max	min	max	min	max	min	max	
m	kN	kN	kNm	kNm	kN	kN	kNm	kNm	kN	kN	-
5.00	-358.50	**-209.87**	-0.01	0.00	**-43.08**	0.00	**-0.00**	15.00	0.14	7.93	0.26
3.15	-360.38	-211.08	0.00	**40.56**	-0.50	0.00	2.00	52.50	0.12	29.14	0.38
1.30	-362.27	-212.40	0.00	1.85	0.00	42.14	3.94	127.67	0.07	48.39	0.65
0.00	**-363.59**	-213.77	**-71.43**	0.00	0.00	**70.53**	5.22	**200.21**	**0.00**	**60.00**	**0.96**

relevant utilisation:

from load spectrum 16 at the location x = 0.00 m, with the internal forces and moments:
N = -363.59 kN, M_y = 0.00 kNm, V_z = 0.00 kN, M_z = 200.21 kNm, V_y = 60.00 kN

$U_{c/t}$ = 0.59, U_σ = 0.00 ⇒ max U = 0.96 < 1.0 ⇒ **permissible utilisation is complied with**

3. serviceability

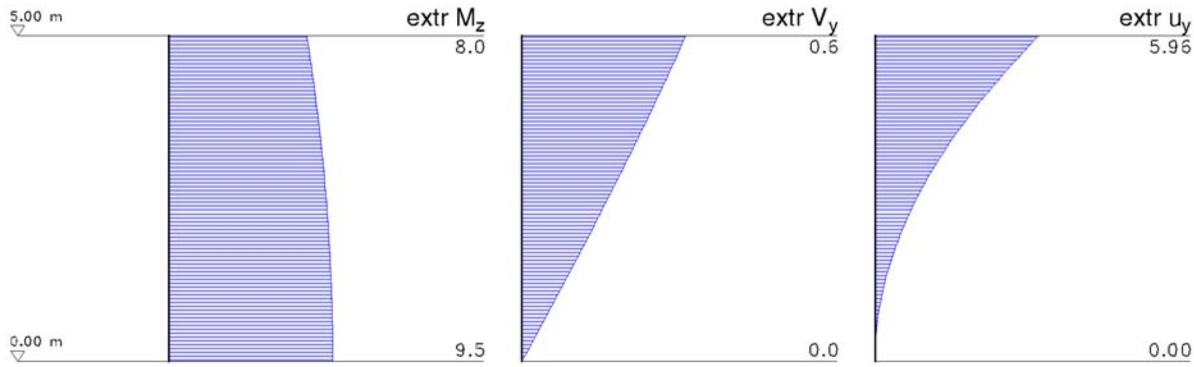
calculation acc. to theory II. Order for the quasi-permanent combination acc. to [3], eq, 6.16
verification of the permissible horizontal deformation acc. to [2], section 7.2.2

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 Y-direction



x	M_z min	M_z max	V_y min	V_y max	u_y min	u_y max
m	kNm	kNm	kN	kN	mm	mm
5.00	0.00	8.00	0.00	**0.58**	0.00	**5.96**
0.00	**0.00**	**9.50**	**0.00**	0.00	**0.00**	0.00

relevant deformation in Y-direction:

from load spectrum 2 at the location x = 5.00 m, with the internal forces and moments:
N = -250.00 kN, M_y = 0.00 kNm, V_z = 0.00 kN, M_z = 8.00 kNm, V_y = 0.58 kN

u_y = 5.96 < 8.00 mm ⇒ **permissible deformation is complied with**

4. support reaction

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

Lc	N_k kN	$H_{k,y}$ kN	$H_{k,z}$ kN	$M_{k,y}$ kN	$M_{k,z}$ kN
1	213.77	-0.00	-0.00	-0.00	-0.00
2	50.00	-0.00	-0.00	-0.00	10.00
3	0.00	40.00	-0.00	-0.00	100.00
4	0.00	-0.00	46.88	-46.88	-0.00
Σ	263.77	40.00	46.88	-46.88	110.00

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	213.77	0.00	-0.00	-0.00	5.22
2	288.59	-0.00	-0.00	-0.00	7.31
4	363.59	0.00	-0.00	-0.00	28.92
5	213.77	36.00	-0.00	-0.00	102.08
14	288.59	60.00	-0.00	-0.00	173.43
16	363.59	60.00	-0.00	-0.00	200.21
20	363.59	0.00	70.53	-71.43	28.92

5. summary
all verifications were carried out successfully.

maximum utilisation of the verifications:

design resist Th.II.O	**96 %**
deformation in Y-direct.	**74 %**

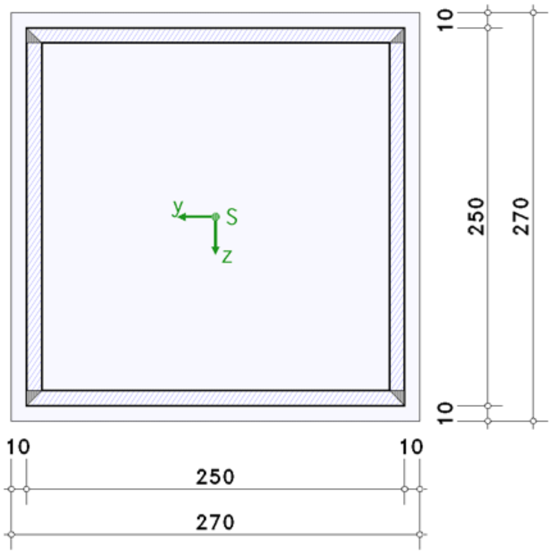
literature and standards:
[1] DIN EN 1993-1-1/NA: Nationaler Anhang - National festgelegte Parameter - Eurocode 3: Teil 1-1, Okt. 2022
[2] DIN EN 1993-1-1: Eurocode 3: Bem. und Konstr. von Stahlbauten - Teil 1-1: Allg. Bem.regeln u. Regeln für den Hochbau, Dez. 2010
[3] DIN EN 1990: Grundlagen der Tragwerksplanung,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

FUPLATTE AUF 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

cross-section, scale 1:5

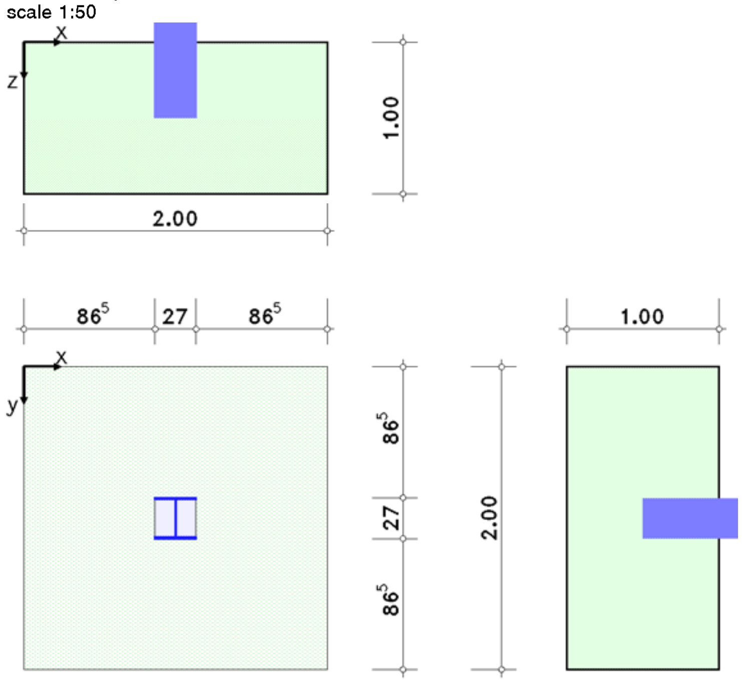


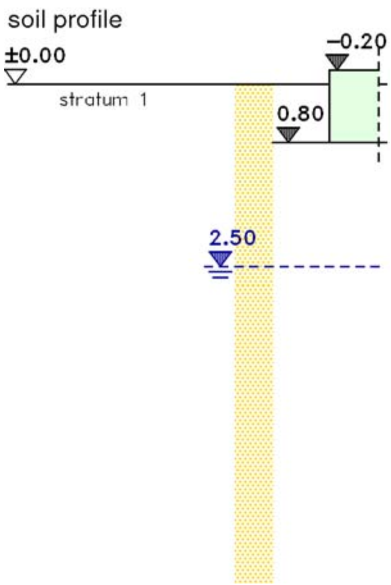
column cross section
non-standardised MSH-section, of quality S355
$h = 250.0$ mm $b = 250.0$ mm $t_w = 10.0$ mm $t_f = 10.0$ mm
fillet weld $a_w = 3.0$ mm

base plate
$b = 270$ mm $h = 270$ mm $t = 10$ mm, of quality S355

mortar joint under base plate
$h_f = 30$ mm

elevation, top view isolated foundation
scale 1:50





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.

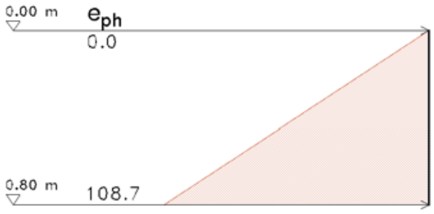
1.1. designation and characteristic values of soil strata

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

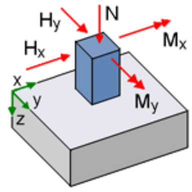
z m	$\Sigma(\gamma \cdot h)$ kN/m²	K_{pgh} -	e_{ph} kN/m²
0.00	0.00	7.152	0.00
0.80	15.20	7.152	108.71

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.	213.77	-0.00	0.00	5.22	-0.00
6	LS 2 (auto)	perman. a.v.	288.59	-0.00	-0.00	7.31	-0.00
7	LS 3 (auto)	perman. a.v.	288.77	-0.00	0.00	25.61	-0.00
8	LS 4 (auto)	perman. a.v.	363.59	-0.00	0.00	28.92	-0.00
9	LS 5 (auto)	perman. a.v.	213.77	-0.00	36.00	102.08	-0.00
10	LS 6 (auto)	perman. a.v.	288.59	-0.00	36.00	106.98	-0.00
11	LS 7 (auto)	perman. a.v.	288.77	-0.00	36.00	125.32	-0.00
12	LS 8 (auto)	perman. a.v.	363.59	-0.00	36.00	131.70	-0.00
13	LS 9 (auto)	perman. a.v.	213.77	42.26	0.00	5.22	-42.58
14	LS 10 (auto)	perman. a.v.	288.59	42.29	-0.00	7.31	-42.71
15	LS 11 (auto)	perman. a.v.	288.77	42.29	0.00	25.61	-42.72
16	LS 12 (auto)	perman. a.v.	363.59	42.32	0.00	28.92	-42.86
17	LS 13 (auto)	perman. a.v.	213.77	-0.00	60.00	166.65	-0.00
18	LS 14 (auto)	perman. a.v.	288.59	-0.00	60.00	173.43	-0.00
19	LS 15 (auto)	perman. a.v.	288.77	-0.00	60.00	191.79	-0.00
20	LS 16 (auto)	perman. a.v.	363.59	-0.00	60.00	200.21	-0.00
21	LS 17 (auto)	perman. a.v.	213.77	70.44	0.00	5.22	-70.96
22	LS 18 (auto)	perman. a.v.	288.59	70.49	-0.00	7.31	-71.19
23	LS 19 (auto)	perman. a.v.	288.77	70.49	0.00	25.61	-71.19
24	LS 20 (auto)	perman. a.v.	363.59	70.53	0.00	28.92	-71.43

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 $N_{0,deadl,k}$ = 100.00 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
permanent	1.10	1.25	1.50

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	5	Lc9	9	Lc13	13	Lc17	17	Lc21
2	Lc6	6	Lc10	10	Lc14	14	Lc18	18	Lc22
3	Lc7	7	Lc11	11	Lc15	15	Lc19	19	Lc23
4	Lc8	8	Lc12	12	Lc16	16	Lc20	20	Lc24

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	213.77	0.00	0.00	5.22	0.00
2	288.59	0.00	-0.00	7.31	0.00
3	288.77	0.00	0.00	25.61	0.00
4	363.59	0.00	0.00	28.92	0.00
5	213.77	0.00	36.00	102.08	0.00
6	288.59	0.00	36.00	106.98	0.00
7	288.77	0.00	36.00	125.32	0.00
8	363.59	0.00	36.00	131.70	0.00
9	213.77	42.26	0.00	5.22	-42.58
10	288.59	42.29	-0.00	7.31	-42.71
11	288.77	42.29	0.00	25.61	-42.72
12	363.59	42.32	0.00	28.92	-42.86
13	213.77	0.00	60.00	166.65	0.00
14	288.59	0.00	60.00	173.43	0.00
15	288.77	0.00	60.00	191.79	0.00
16	363.59	0.00	60.00	200.21	0.00
17	213.77	70.44	0.00	5.22	-70.96
18	288.59	70.49	-0.00	7.31	-71.19
19	288.77	70.49	0.00	25.61	-71.19
20	363.59	70.53	0.00	28.92	-71.43

3.3. clamping depth

determination of the required clamping depth acc. to [2]

3.3.1. required clamping depth for bending around the y-axis

coefficient of the contributing width	α_m	= 1.12
contributing width	b_m	= 121.3 mm
resulting pressure	p	= 20.63 kN/cm
perm. plastic shear force	$V_{pl,z}$	= 857.10 kN

required clamping depth

LS	D_o kN	D_u kN	$D_u/V_{pl,z}$ -	f_{min} cm
1	86.42	86.42	0.10	10.3
2	102.29	102.29	0.12	12.2
3	191.44	191.44	0.22	22.9
4	203.45	203.45	0.24	24.4
5	416.28	380.28	0.44	47.7
6	425.34	389.34	0.45	48.8
7	457.50	421.50	0.49	52.6
8	468.13	432.13	0.50	53.9
9	86.42	86.42	0.10	10.3
10	102.29	102.29	0.12	12.2
11	191.44	191.44	0.22	22.9
12	203.45	203.45	0.24	24.4
13	545.44	485.44	0.57	61.7
14	555.24	495.24	0.58	62.9
15	580.88	520.88	0.61	65.9
16	592.25	532.25	0.62	**67.3**
17	86.42	86.42	0.10	10.3
18	102.29	102.29	0.12	12.2
19	191.44	191.44	0.22	22.9
20	203.45	203.45	0.24	24.4

D_o/D_u - res. compressive force top/bottom f_{min} - req. clamping depth

maximum required clamping depth for bending around the y-axis $f_{min,y}$ = 67.3 cm

3.3.2. required clamping depth for bending around the z-axis

coefficient of the contributing width	α_m	= 1.12
contributing width	h_m	= 121.3 mm
resulting pressure	p	= 20.63 kN/cm
perm. plastic shear force	$V_{pl,y}$	= 931.63 kN

required clamping depth

LS	D_o kN	D_u kN	$D_u/V_{pl,y}$ -	f_{min} cm
1	0.00	0.00	0.00	0.0
2	0.00	0.00	0.00	0.0
3	0.00	0.00	0.00	0.0
4	0.00	0.00	0.00	0.0
5	0.00	0.00	0.00	0.0
6	0.00	0.00	0.00	0.0
7	0.00	0.00	0.00	0.0
8	0.00	0.00	0.00	0.0
9	287.40	245.14	0.26	31.9
10	287.83	245.54	0.26	31.9
11	287.84	245.54	0.26	31.9
12	288.27	245.95	0.26	32.0
13	0.00	0.00	0.00	0.0
14	0.00	0.00	0.00	0.0
15	0.00	0.00	0.00	0.0
16	0.00	0.00	0.00	0.0
17	386.90	316.46	0.34	42.1
18	387.45	316.97	0.34	42.2
19	387.46	316.98	0.34	42.2
20	388.03	317.49	0.34	**42.2**

D_o/D_u - res. compressive force top/bottom f_{min} - req. clamping depth

maximum required clamping depth for bending around the z-axis $f_{min,z}$ = 42.2 cm

3.3.3. set clamping depth

required	f_{min}	= 67.3 cm	(from LK 16, Bieg. about y-axis)
minimum value	f_{min} = 1.5·25.00	= 37.5 < 67.3 cm	
maximum value	f_{max} = 4.0·25.00	= 100.0 > 67.3 cm	
selected	f_{gew}	= **68.0** > 67.3 cm	

3.4. resistance of cross section

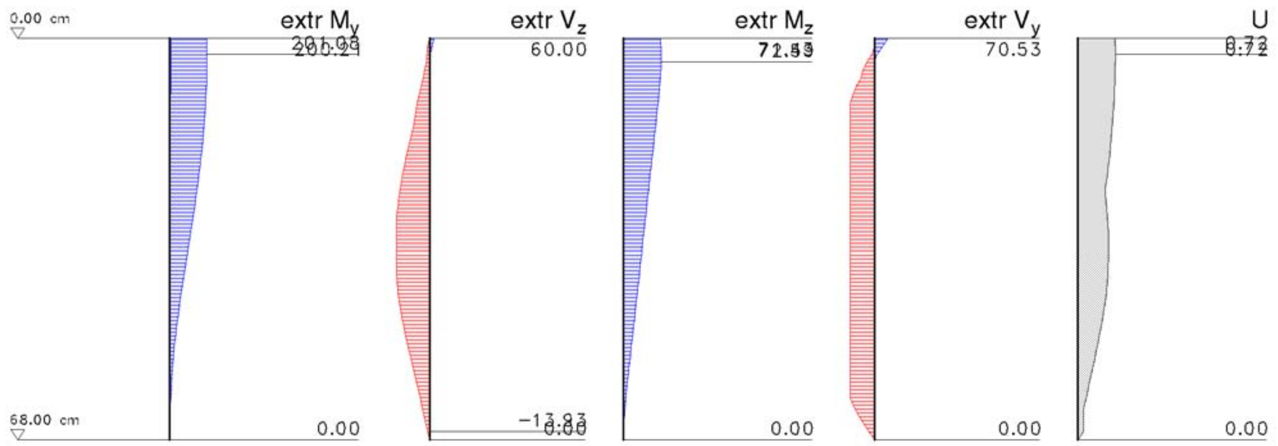
plastic stress analysis is carried out acc. to [3], sec. 6.2.2 to 6.2.10.

3.4.1. supporting forces

LS	M_y/V_z a_o cm	a_u cm	D_o kN	D_u kN	M_z/V_y a_o cm	a_u cm	D_o kN	D_u kN
1	0.5	0.5	7.72	7.72	---	---	---	---
2	0.6	0.6	10.84	10.84	---	---	---	---
3	2.3	2.3	38.76	38.76	---	---	---	---
4	2.6	2.6	43.94	43.94	---	---	---	---
5	12.9	10.7	214.72	178.72	---	---	---	---
6	13.5	11.3	224.95	188.95	---	---	---	---
7	15.9	13.7	265.37	229.37	---	---	---	---
8	16.8	14.6	280.35	244.35	---	---	---	---
9	0.5	0.5	7.72	7.72	6.7	4.1	111.14	68.88
10	0.6	0.6	10.84	10.84	6.7	4.1	111.40	69.11
11	2.3	2.3	38.76	38.76	6.7	4.1	111.41	69.12
12	2.6	2.6	43.94	43.94	6.7	4.2	111.68	69.36
13	24.8	21.2	413.55	353.55	---	---	---	---
14	26.2	22.6	437.24	377.24	---	---	---	---
15	30.8	27.2	515.30	455.30	---	---	---	---
16	33.8	30.2	564.34	504.34	---	---	---	---
17	0.5	0.5	7.72	7.72	11.6	7.4	194.23	123.79
18	0.6	0.6	10.84	10.84	11.7	7.4	194.73	124.24
19	2.3	2.3	38.76	38.76	11.7	7.4	194.74	124.25
20	2.6	2.6	43.94	43.94	11.7	7.5	195.24	124.70

a_o/a_u - pressure area top/bottom D_o/D_u - res. compressive force top/bottom

3.4.2. extremal internal forces and moments



extreme values of axial force: N_{min} / N_{max} = 213.77 / 363.59 kN

x cm	extr M_y min kNm	max kNm	extr V_z min kNm	max kNm	extr M_z min kNm	max kNm	extr V_y min kNm	max kNm	U
0.00	5.22	200.21	0.00	**60.00**	**0.00**	71.43	0.00	**70.53**	0.72
2.70	4.93	**201.08**	-41.18	4.26	0.00	72.58	-13.47	14.80	**0.72**
4.05	4.75	200.95	-47.61	-13.93	0.00	**72.59**	-39.95	0.00	0.72
12.16	3.62	192.26	-205.30	-13.93	0.00	65.82	**-124.94**	0.00	0.69
25.67	1.73	148.40	-443.42	-13.93	0.00	48.94	-124.94	0.00	0.53
33.78	0.61	109.28	**-503.69**	-13.93	0.00	38.81	-124.94	0.00	0.59
37.88	0.03	88.60	-503.58	-13.93	0.00	33.68	-124.94	0.00	0.59
39.25	-0.16	81.72	-501.21	-13.93	0.00	31.97	-124.94	0.00	0.58
66.63	**-3.97**	2.03	-28.98	-13.93	0.00	1.03	-29.86	0.00	0.12
68.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

maximum utilization U = 0.72 < 1.00

from load spectrum 16 at the location x = 2.70 cm

internal forces and moments: N = 363.59 kN, V_z/M_y = 4.26/201.08 kNm, V_y/M_z = 0.00/0.00 kNm utilization: U_σ = 0.72

3.5. weld between column 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 **circumferential fillet weld.**
axial force transfer of 100 % by the weld.

minimum value of the weld thickness a_{min} = 3 mm

LS	a_w	$\sigma_\perp$	$\tau_\perp$	τ_{II}	$\sigma_{1,w,Ed}$	$f_{1,w,Rd}$	$\sigma_{2,w,Ed}$	$f_{2,w,Rd}$	U
	mm	N/mm²	N/mm²	N/mm²	N/mm²	N/mm²	N/mm²	N/mm²	-
1	**3**	-50.39	-50.39	0.00	100.77	435.56	50.39	352.80	0.23
2	3	-68.02	-68.02	0.00	136.04	435.56	68.02	352.80	0.31
3	3	-68.06	-68.06	0.00	136.13	435.56	68.06	352.80	0.31
4	3	-85.70	-85.70	0.00	171.40	435.56	85.70	352.80	**0.39**
5	3	-50.39	-50.39	0.00	100.77	435.56	50.39	352.80	0.23
6	3	-68.02	-68.02	0.00	136.04	435.56	68.02	352.80	0.31
7	3	-68.06	-68.06	0.00	136.13	435.56	68.06	352.80	0.31
8	3	-85.70	-85.70	0.00	171.40	435.56	85.70	352.80	0.39
9	3	-50.39	-50.39	0.00	100.77	435.56	50.39	352.80	0.23
10	3	-68.02	-68.02	0.00	136.04	435.56	68.02	352.80	0.31
11	3	-68.06	-68.06	0.00	136.13	435.56	68.06	352.80	0.31
12	3	-85.70	-85.70	0.00	171.40	435.56	85.70	352.80	0.39
13	3	-50.39	-50.39	0.00	100.77	435.56	50.39	352.80	0.23
14	3	-68.02	-68.02	0.00	136.04	435.56	68.02	352.80	0.31
15	3	-68.06	-68.06	0.00	136.13	435.56	68.06	352.80	0.31
16	3	-85.70	-85.70	0.00	171.40	435.56	85.70	352.80	0.39
17	3	-50.39	-50.39	0.00	100.77	435.56	50.39	352.80	0.23
18	3	-68.02	-68.02	0.00	136.04	435.56	68.02	352.80	0.31
19	3	-68.06	-68.06	0.00	136.13	435.56	68.06	352.80	0.31
20	3	-85.70	-85.70	0.00	171.40	435.56	85.70	352.80	0.39

a_w - 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 $a_{w,max}$ = 3 mm
maximum utilization U = 0.39 < 1.00

3.6. introduction of the normal force into the foundation
verification acc. to [5], section 6.2.5 and load-bearing capacity of the subareas acc. to [4], section6.7

3.6.1. requirements for the mortar under the base plate
0.2-fold of the smallest panel dimension = 54.0 > 30 mm mortar height
⇒ the characteristic strength of the mortar should be at least 20% of the foundation concrete.

3.6.2. load spreading
$c = t\cdot[f_y/3\cdot f_{jd}\cdot\gamma_{M0})]^{0.5} \le 0.5\cdot(h-2\cdot t)$

an undisturbed load propagation is assumed.

spreading width	c	= 18.7 mm
loading area	A_{c0}	= 276.64 cm²
distribution area	A_{c1}	= 3025.00 cm²

3.6.3. design resistance
$F_{C,Rdr} = f_{jd}\cdot A_{c0}$

$f_{jd} = \beta_j\cdot F_{Rdu}/A_{c0}$

$F_{Rdu} = A_{c0}\cdot f_{cd}\cdot(A_{c1}/A_{c0})^{0.5} \le 3.0\cdot f_{cd}\cdot A_{c0}$

joint coefficient	β_j	= 2/3
design value of the mortar strength	f_{jd}	= 34.00 N/mm²
load-bearing capacity under pressure	$F_{C,Rd}$	= 940.56 kN

3.6.4. utilization
$U = N_{Ed}/F_{C,Rd}$

maximum compressive force (LS 4) N_{Ed} = 363.59 < 940.56 kN

utilization U = 0.39 < 1.00

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	21	permanent and transient	Lc1* +Lc15
2	permanent and transient	1.35· Lc1* +Lc5	22	permanent and transient	1.35· Lc1* +Lc15
3	permanent and transient	Lc1* +Lc6	23	permanent and transient	Lc1* +Lc16
4	permanent and transient	1.35· Lc1* +Lc6	24	permanent and transient	1.35· Lc1* +Lc16
5	permanent and transient	Lc1* +Lc7	25	permanent and transient	Lc1* +Lc17
6	permanent and transient	1.35· Lc1* +Lc7	26	permanent and transient	1.35· Lc1* +Lc17
7	permanent and transient	Lc1* +Lc8	27	permanent and transient	Lc1* +Lc18
8	permanent and transient	1.35· Lc1* +Lc8	28	permanent and transient	1.35· Lc1* +Lc18
9	permanent and transient	Lc1* +Lc9	29	permanent and transient	Lc1* +Lc19
10	permanent and transient	1.35· Lc1* +Lc9	30	permanent and transient	1.35· Lc1* +Lc19
11	permanent and transient	Lc1* +Lc10	31	permanent and transient	Lc1* +Lc20
12	permanent and transient	1.35· Lc1* +Lc10	32	permanent and transient	1.35· Lc1* +Lc20
13	permanent and transient	Lc1* +Lc11	33	permanent and transient	Lc1* +Lc21
14	permanent and transient	1.35· Lc1* +Lc11	34	permanent and transient	1.35· Lc1* +Lc21
15	permanent and transient	Lc1* +Lc12	35	permanent and transient	Lc1* +Lc22
16	permanent and transient	1.35· Lc1* +Lc12	36	permanent and transient	1.35· Lc1* +Lc22
17	permanent and transient	Lc1* +Lc13	37	permanent and transient	Lc1* +Lc23
18	permanent and transient	1.35· Lc1* +Lc13	38	permanent and transient	1.35· Lc1* +Lc23
19	permanent and transient	Lc1* +Lc14	39	permanent and transient	Lc1* +Lc24
20	permanent and transient	1.35· Lc1* +Lc14	40	permanent and transient	1.35· Lc1* +Lc24

*) 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	213.77	0.00	0.00	5.22	0.00
2	213.77	0.00	0.00	5.22	0.00
3	288.59	0.00	-0.00	7.31	0.00
4	288.59	0.00	-0.00	7.31	0.00
5	288.77	0.00	0.00	25.61	0.00
6	288.77	0.00	0.00	25.61	0.00
7	363.59	0.00	0.00	28.92	0.00
8	363.59	0.00	0.00	28.92	0.00
9	213.77	0.00	36.00	102.08	0.00
10	213.77	0.00	36.00	102.08	0.00
11	288.59	0.00	36.00	106.98	0.00
12	288.59	0.00	36.00	106.98	0.00
13	288.77	0.00	36.00	125.32	0.00
14	288.77	0.00	36.00	125.32	0.00
15	363.59	0.00	36.00	131.70	0.00
16	363.59	0.00	36.00	131.70	0.00
17	213.77	42.26	0.00	5.22	-42.58
18	213.77	42.26	0.00	5.22	-42.58
19	288.59	42.29	-0.00	7.31	-42.71
20	288.59	42.29	-0.00	7.31	-42.71
21	288.77	42.29	0.00	25.61	-42.72
22	288.77	42.29	0.00	25.61	-42.72
23	363.59	42.32	0.00	28.92	-42.86
24	363.59	42.32	0.00	28.92	-42.86
25	213.77	0.00	60.00	166.65	0.00
26	213.77	0.00	60.00	166.65	0.00
27	288.59	0.00	60.00	173.43	0.00
28	288.59	0.00	60.00	173.43	0.00
29	288.77	0.00	60.00	191.79	0.00
30	288.77	0.00	60.00	191.79	0.00
31	363.59	0.00	60.00	200.21	0.00
32	363.59	0.00	60.00	200.21	0.00
33	213.77	70.44	0.00	5.22	-70.96
34	213.77	70.44	0.00	5.22	-70.96
35	288.59	70.49	-0.00	7.31	-71.19
36	288.59	70.49	-0.00	7.31	-71.19
37	288.77	70.49	0.00	25.61	-71.19
38	288.77	70.49	0.00	25.61	-71.19
39	363.59	70.53	0.00	28.92	-71.43
40	363.59	70.53	0.00	28.92	-71.43

4.3. design calculation for bending

4.3.1. longitudinal reinforcement in x-direction

reinforcement edge distance top/bottom h_{so}/h_{su} = 5.0/5.0 cm

moments in design calculation sections

LS	x = 80.0 cm kNm	x = 120.0 cm kNm
1	34.20	34.20
2	34.20	34.20
3	46.17	46.17
4	46.17	46.17
5	46.20	46.20
6	46.20	46.20
7	58.17	58.17
8	58.17	58.17
9	34.20	34.20
10	34.20	34.20
11	46.17	46.17
12	46.17	46.17
13	46.20	46.20
14	46.20	46.20
15	58.17	58.17
16	58.17	58.17
17	4.34	64.07
18	4.34	64.07
19	16.25	76.10
20	16.25	76.10
21	16.28	76.13
22	16.28	76.13
23	28.19	88.16
24	28.19	88.16
25	34.20	34.20
26	34.20	34.20
27	46.17	46.17
28	46.17	46.17
29	46.20	46.20
30	46.20	46.20
31	58.17	58.17
32	58.17	58.17
33	-12.52	86.15
34	**-14.32**	84.84
35	-3.42	96.23
36	-3.68	96.06
37	-3.09	96.48
38	-3.49	96.20
39	8.24	**108.17**
40	8.21	108.14

design calculation for LS 34: $\varepsilon_o/\varepsilon_u$ = 26.32/-0.16‰   min $A_{s,o}$ = **0.3 cm²**
design calculation for LS 39: $\varepsilon_o/\varepsilon_u$ = -0.44/26.34‰   min $A_{s,u}$ = **2.5 cm²**

4.3.2. longitudinal reinforcement in y-direction

reinforcement edge distance top/bottom h_{so}/h_{su} = 6.0/6.0 cm

moments in design calculation sections

LS	y = 80.0 cm kNm	y = 120.0 cm kNm	LS	y = 80.0 cm kNm	y = 120.0 cm kNm	LS	y = 80.0 cm kNm	y = 120.0 cm kNm
1	32.37	36.04	15	-0.61	117.37	29	-16.00	160.00
2	32.37	36.04	16	-0.85	117.21	30	-21.60	152.37
3	43.60	48.75	17	32.37	36.04	31	-15.86	**163.76**
4	43.60	48.75	18	32.37	36.04	32	-20.56	159.51
5	37.19	55.22	19	43.60	48.75	33	32.37	36.05
6	37.19	55.22	20	43.60	48.75	34	32.37	36.05
7	48.00	68.35	21	37.19	55.22	35	43.61	48.76
8	48.00	68.35	22	37.19	55.22	36	43.61	48.75
9	-11.91	84.57	23	48.00	68.35	37	37.33	55.35
10	-13.48	83.44	24	48.00	68.35	38	37.27	55.29
11	-3.85	96.71	25	-16.00	147.91	39	48.02	68.37
12	-4.15	96.51	26	**-21.60**	136.95	40	48.00	68.35
13	-8.80	104.22	27	-16.00	145.39			
14	-9.94	103.42	28	-21.35	139.83			

design calculation for LS 26: $\varepsilon_o/\varepsilon_u$ = 26.61/-0.19‰   min $A_{s,o}$ = **0.5 cm²**
design calculation for LS 31: $\varepsilon_o/\varepsilon_u$ = -0.56/26.63‰   min $A_{s,u}$ = **3.8 cm²**

4.3.3. selected reinforcement in x-direction

top   B500A, to distribute evenly
4 Ø 10 = 3.1 > 0.3 cm²

bottom (distribution acc. to [6])

width	cm	200.0
distrib.	%	100.0
min A_s	cm²	2.5

select.		6 Ø 10
exist A_s	cm²	4.7

total existing bottom reinforcement: ΣA_s = 4.7 > 2.5 cm²

4.3.4. selected reinforcement in y-direction

top   B500A, to distribute evenly
4 Ø 10 = 3.1 > 0.5 cm²

bottom (distribution acc. to [6])

width	cm	200.0
distrib.	%	100.0
min A_s	cm²	3.8

select.		6 Ø 10
exist A_s	cm²	4.7

total existing bottom reinforcement: ΣA_s = 4.7 > 3.8 cm²
$\varepsilon_o/\varepsilon_u$ - strains in extreme fibres (top/bottom)

4.4. punching shear calculation

4.4.1. action within the basic control perimeter

$v_{Ed,crit} = \beta \cdot V_{Ed,red} / (u_{crit} \cdot d)$
$V_{Ed,red} = V_{Ed} - \Delta V_{Ed}$
$\Delta V_{Ed} = A_{crit} (\sigma_{Ed,gd,m} - g_{Ed,slab})$
$\beta = 1 + \mathrm{sqrt}((k_x \cdot M_{Ed,x} / V_{Ed} \cdot u_{crit} / W_{crit,x})^2 + (k_y \cdot M_{Ed,y} / V_{Ed} \cdot u_{crit} / W_{crit,y})^2) \geq 1.10$
$W_{crit} = \int |e|\, dl$ mit dl:Differential des Umfangs
e :Abstand von dl zur Achse von M_{Ed}

coefficient for the calculation of shear stresses from moment action
(acc. to [4], table 6.1)
$c_1 = c_2 = 0.27 \Rightarrow k_x = k_y = 0.6$

calculated values of basic control perimeter

LS	a_{crit} cm	a/d -	u_{crit} m	A_{crit} m²	$W_{crit,x}$ m²	$W_{crit,y}$ m²	LS	a_{crit} cm	a/d -	u_{crit} m	A_{crit} m²	$W_{crit,x}$ m²	$W_{crit,y}$ m²
1	33.8	1.27	3.20	0.796	1.0350	1.0350	11	22.5	0.85	2.50	0.476	0.6250	0.6250
2	33.8	1.27	3.20	0.796	1.0350	1.0350	12	22.5	0.85	2.50	0.476	0.6250	0.6250
3	33.8	1.27	3.20	0.796	1.0350	1.0350	13	21.6	0.81	2.44	0.453	0.5958	0.5958
4	33.8	1.27	3.20	0.796	1.0350	1.0350	14	21.6	0.81	2.44	0.453	0.5958	0.5958
5	29.9	1.13	2.96	0.678	0.8837	0.8837	15	22.7	0.86	2.50	0.479	0.6292	0.6292
6	29.9	1.13	2.96	0.678	0.8837	0.8837	16	22.7	0.86	2.50	0.479	0.6292	0.6292
7	30.3	1.14	2.99	0.690	0.8988	0.8988	17	26.2	0.99	2.73	0.572	0.7489	0.7489
8	30.3	1.14	2.99	0.690	0.8988	0.8988	18	26.2	0.99	2.73	0.572	0.7489	0.7489
9	21.1	0.80	2.40	0.440	0.5794	0.5794	19	27.7	1.04	2.82	0.613	0.8005	0.8005
10	21.1	0.80	2.40	0.440	0.5794	0.5794	20	27.7	1.04	2.82	0.613	0.8005	0.8005

LS	a_{crit} cm	a/d -	u_{crit} m	A_{crit} m²	$W_{crit,x}$ m²	$W_{crit,y}$ m²
21	27.0	1.02	2.78	0.594	0.7768	0.7768
22	27.0	1.02	2.78	0.594	0.7768	0.7768
23	28.0	1.05	2.84	0.620	0.8101	0.8101
24	28.0	1.05	2.84	0.620	0.8101	0.8101
25	20.0	0.75	2.34	0.415	0.5472	0.5472
26	20.0	0.75	2.34	0.415	0.5472	0.5472
27	20.1	0.76	2.35	0.418	0.5512	0.5512
28	20.1	0.76	2.35	0.418	0.5512	0.5512
29	20.0	0.75	2.34	0.415	0.5472	0.5472
30	20.0	0.75	2.34	0.415	0.5472	0.5472
31	20.5	0.77	2.37	0.427	0.5632	0.5632
32	20.5	0.77	2.37	0.427	0.5632	0.5632
33	23.2	0.88	2.54	0.492	0.6463	0.6463
34	23.2	0.88	2.54	0.492	0.6463	0.6463
35	24.9	0.94	2.65	0.537	0.7034	0.7034
36	24.9	0.94	2.65	0.537	0.7034	0.7034
37	24.6	0.93	2.63	0.530	0.6944	0.6944
38	24.6	0.93	2.63	0.530	0.6944	0.6944
39	25.8	0.98	2.70	0.562	0.7351	0.7351
40	25.8	0.98	2.70	0.562	0.7351	0.7351

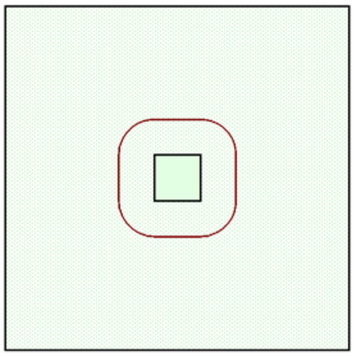
decisive shear stress within the basic control perimeter

LS	V_{Ed} kN	$\sigma_{Ed,gd,m}$ kN/m²	ΔV_{Ed} kN	$M_{Ed,x,Sp}$ kNm	$M_{Ed,y,Sp}$ kNm	β -	$v_{Ed,crit}$ N/mm²
1	213.77	53.48	42.60	5.22	0.00	1.10	0.222
2	213.77	53.48	42.60	5.22	0.00	1.10	0.222
3	288.59	72.20	57.50	7.31	0.00	1.10	0.299
4	288.59	72.20	57.50	7.31	0.00	1.10	0.299
5	288.77	72.24	48.98	25.61	0.00	1.18	0.360
6	288.77	72.24	48.98	25.61	0.00	1.18	0.360
7	363.59	90.96	62.75	28.92	0.00	1.16	0.440
8	363.59	90.96	62.75	28.92	0.00	1.16	0.440
9	213.77	53.49	23.53	102.08	0.00	2.19	0.654
10	213.77	53.49	23.53	102.08	0.00	2.19	0.654
11	288.59	72.21	34.34	106.98	0.00	1.89	0.726
12	288.59	72.21	34.34	106.98	0.00	1.89	0.726
13	288.77	72.25	32.71	125.32	0.00	2.07	0.819
14	288.77	72.25	32.71	125.32	0.00	2.07	0.819
15	363.59	90.97	43.56	131.70	0.00	1.86	0.899
16	363.59	90.97	43.56	131.70	0.00	1.86	0.899
17	213.77	53.49	30.62	5.22	42.58	1.44	0.364
18	213.77	53.49	30.62	5.22	42.58	1.44	0.364
19	288.59	72.20	44.25	7.31	42.71	1.32	0.431
20	288.59	72.20	44.25	7.31	42.71	1.32	0.431
21	288.77	72.25	42.94	25.61	42.72	1.37	0.457
22	288.77	72.25	42.94	25.61	42.72	1.37	0.457
23	363.59	90.96	56.43	28.92	42.86	1.30	0.531
24	363.59	90.96	56.43	28.92	42.86	1.30	0.531
25	213.77	39.52	16.39	166.65	0.00	3.00	0.955
26	213.77	39.52	16.39	166.65	0.00	3.00	0.955
27	288.59	68.16	28.48	173.43	0.00	2.53	1.061
28	288.59	68.16	28.48	173.43	0.00	2.53	1.061
29	288.77	62.56	25.94	191.79	0.00	2.70	1.147
30	288.77	62.56	25.94	191.79	0.00	2.70	1.147
31	363.59	87.48	37.37	200.21	0.00	2.39	1.242
32	363.59	87.48	37.37	200.21	0.00	2.39	1.242
33	213.77	53.49	26.33	5.22	70.96	1.78	0.497
34	213.77	53.49	26.33	5.22	70.96	1.78	0.497
35	288.59	72.20	38.76	7.31	71.19	1.56	0.556
36	288.59	72.20	38.76	7.31	71.19	1.56	0.556
37	288.77	72.25	38.28	25.61	71.19	1.60	0.574
38	288.77	72.25	38.28	25.61	71.19	1.60	0.574
39	363.59	90.96	51.09	28.92	71.43	1.47	0.640
40	363.59	90.96	51.09	28.92	71.43	1.47	0.640

ΔV_{Ed} - resultant of ground pressure $M_{Ed,x,Sp}/M_{Ed,y,Sp}$ - moments concerning centre of control perimeter
β - load increase factor from eccentric load $v_{Ed,crit}$ - decisive shear stress within the basic control perimeter

4.4.2. Punching shear resistance within the basic control perimeter

$v_{Rd,c} = C_{Rd,c} \cdot k \cdot (100 \cdot \rho_{l,tension} \cdot f_{ck})^{1/3} \cdot 2 \cdot d/a \geq v_{min} \cdot 2 \cdot d/a$ [N/mm²]
$C_{Rd,c} = 0.15/\gamma_C$
$k = 1 + sqrt(200/d) \leq 2.0$ with d [mm]
$\rho_{l,tension,max}$ = minimum from $(0.02,\ 0.5 \cdot f_{cd}/f_{yd})$
$\rho_{l,tension} = sqrt(\rho_{lx,tension} \cdot \rho_{ly,tension}) \leq \rho_{l,tension,max}$
$v_{min} = 0.0525/\gamma_C \cdot k^{3/2} \cdot f_{ck}^{1/2}$ for $d \leq 600$ mm



mean effective depth
$d_m = (27 + 26)/2 = 26.5$ cm

scale factor
$k = 1 + sqrt(200/265) = 1.87 < 2$

longitudinal reinf. ratio of the anchored tension reinf.
mean of the tension reinforcement to the distance 3d from the column
$a_{s,x,3d} = 4.4/1.86 = 2.36$ cm²/m
$a_{s,y,3d} = 4.4/1.86 = 2.36$ cm²/m
$\rho_{lx,tension} = 2.36/27 \cdot 10^{-2} = 0.00087$
$\rho_{ly,tension} = 2.36/26 \cdot 10^{-2} = 0.00091$
$\rho_{l,tension} = sqrt(0.00087 \cdot 0.00091) = 0.00089$

punching shear resistance without shear reinforcement
$C_{Rd,c} = 0.15/1.5 = 0.1$
$\rho_{l,tension,max}$ = minimum from $(0.02,\ 0.5 \cdot 17/434.78) = 0.0195 > 0.0009$
$v_{min} \cdot 2 \cdot d/a = 0.0525/1.5 \cdot 1.87^{3/2} \cdot 30^{0.5} \cdot 2 \cdot 26.5/20.5 = 1.264$ N/mm²
$v_{Rd,c} = 0.1 \cdot 1.87 \cdot (100 \cdot 0.00089 \cdot 30)^{1/3} \cdot 2 \cdot 26.5/20.5 = 0.669$ N/mm² < 1.264 N/mm² $\Rightarrow v_{Rd,c} = 1.264$ N/mm²

1.242 N/mm² < 1.264 N/mm² ⇒ no additional reinforcement required

4.4.3. Minimum longitudinal reinforcement to ensure shear resistance

acc. to [4] table NA.6.1.1

reduction value from ground pressure below load distribution for load spectrum 32
$\Delta V_{Ed} = 6.37$ kN

tensside	direction	η -	$m_{Ed,min}$ kNm/m	$a_{su,min}$ cm²/m	$a_{so,min}$ cm²/m
bottom	x	0.125	44.65	3.78	---
	y	0.125	44.65	3.78	---

η - moment coefficient $m_{Ed,min} = \eta \cdot V_{Ed}$ - minimum design moment

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

5.1. partial safety factors on load side

acc. to [7] table A 2.1

5.2. partial safety factors on res1istance side

acc. to [7] 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+1.5· Lc3
14	BS-P	1.1· Lc1+1.5· Lc3
15	BS-P	0.9· Lc1+1.5· (Lc2+Lc3)
16	BS-P	1.1· Lc1+1.5· (Lc2+Lc3)
17	BS-P	0.9· Lc1+1.5· Lc4
18	BS-P	1.1· Lc1+1.5· Lc4
19	BS-P	0.9· Lc1+1.5· (Lc2+Lc4)
20	BS-P	1.1· Lc1+1.5· (Lc2+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	192.39	0.00	0.00	0.00	0.00
2	235.14	0.00	0.00	0.00	0.00
3	267.39	0.00	-0.00	15.00	0.00
4	310.14	0.00	-0.00	15.00	0.00
5	192.39	0.00	36.00	90.00	0.00
6	235.14	0.00	36.00	90.00	0.00
7	267.39	0.00	36.00	105.00	0.00
8	310.14	0.00	36.00	105.00	0.00
9	192.39	42.19	0.00	0.00	-42.19
10	235.14	42.19	0.00	0.00	-42.19
11	267.39	42.19	-0.00	15.00	-42.19
12	310.14	42.19	-0.00	15.00	-42.19
13	192.39	0.00	60.00	150.00	0.00
14	235.14	0.00	60.00	150.00	0.00
15	267.39	0.00	60.00	165.00	0.00
16	310.14	0.00	60.00	165.00	0.00
17	192.39	70.31	0.00	0.00	-70.31
18	235.14	70.31	0.00	0.00	-70.31
19	267.39	70.31	-0.00	15.00	-70.31
20	310.14	70.31	-0.00	15.00	-70.31

5.4. verification against overturning (EQU)

LS	decisive edge	M_{dst} kNm	M_{stb} kNm	μ -
1	---	0.00	282.39	0.00
2	---	0.00	345.14	0.00
3	---	0.00	357.39	0.00
4	---	0.00	420.14	0.00
5	x (bottom)	126.00	282.39	0.45
6	x (bottom)	126.00	345.14	0.37
7	x (bottom)	126.00	342.39	0.37
8	x (bottom)	126.00	405.14	0.31
9	y (right)	84.38	282.39	0.30
10	y (right)	84.38	345.14	0.24
11	y (right)	84.38	357.39	0.24
12	y (right)	84.38	420.14	0.20
13	x (bottom)	210.00	282.39	**0.74**
14	x (bottom)	210.00	345.14	0.61
15	x (bottom)	210.00	342.39	0.61
16	x (bottom)	210.00	405.14	0.52
17	y (right)	140.63	282.39	0.50
18	y (right)	140.63	345.14	0.41
19	y (right)	140.63	357.39	0.39
20	y (right)	140.63	420.14	0.33

μ_{max} = 0.74 < 1.0 ⇒ **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+1.5· Lc3
14	BS-P	1.35· Lc1+1.5· Lc3
15	BS-P	Lc1+1.5· (Lc2+Lc3)
16	BS-P	1.35· Lc1+1.5· (Lc2+Lc3)
17	BS-P	Lc1+1.5· Lc4
18	BS-P	1.35· Lc1+1.5· Lc4
19	BS-P	Lc1+1.5· (Lc2+Lc4)
20	BS-P	1.35· Lc1+1.5· (Lc2+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	213.77	0.00	0.00	0.00	0.00
2	288.59	0.00	0.00	0.00	0.00
3	288.77	0.00	-0.00	15.00	0.00
4	363.59	0.00	-0.00	15.00	0.00
5	213.77	0.00	36.00	90.00	0.00
6	288.59	0.00	36.00	90.00	0.00
7	288.77	0.00	36.00	105.00	0.00
8	363.59	0.00	36.00	105.00	0.00
9	213.77	42.19	0.00	0.00	-42.19
10	288.59	42.19	0.00	0.00	-42.19
11	288.77	42.19	-0.00	15.00	-42.19
12	363.59	42.19	-0.00	15.00	-42.19
13	213.77	0.00	60.00	150.00	0.00
14	288.59	0.00	60.00	150.00	0.00
15	288.77	0.00	60.00	165.00	0.00
16	363.59	0.00	60.00	165.00	0.00
17	213.77	70.31	0.00	0.00	-70.31
18	288.59	70.31	0.00	0.00	-70.31
19	288.77	70.31	-0.00	15.00	-70.31
20	363.59	70.31	-0.00	15.00	-70.31

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	213.77	0.00	0.00	0.00	0.00
2	213.77	0.00	0.00	0.00	0.00
3	263.77	0.00	-0.00	10.00	0.00
4	263.77	0.00	-0.00	10.00	0.00
5	213.77	0.00	40.00	100.00	0.00
6	213.77	0.00	40.00	100.00	0.00
7	263.77	0.00	40.00	110.00	0.00
8	263.77	0.00	40.00	110.00	0.00
9	213.77	46.88	0.00	0.00	-46.88
10	213.77	46.88	0.00	0.00	-46.88
11	263.77	46.88	-0.00	10.00	-46.88
12	263.77	46.88	-0.00	10.00	-46.88
13	213.77	0.00	40.00	100.00	0.00
14	213.77	0.00	40.00	100.00	0.00
15	263.77	0.00	40.00	110.00	0.00
16	263.77	0.00	40.00	110.00	0.00
17	213.77	46.88	0.00	0.00	-46.88
18	213.77	46.88	0.00	0.00	-46.88
19	263.77	46.88	-0.00	10.00	-46.88
20	263.77	46.88	-0.00	10.00	-46.88

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	313.77	0.00	0.00	2.00	2.00	0.00	0.00
2	313.77	0.00	0.00	2.00	2.00	0.00	0.00
3	363.77	10.00	0.00	2.00	1.95	0.00	0.00
4	363.77	10.00	0.00	2.00	1.95	0.00	0.00
5	313.77	129.33	0.00	2.00	1.18	0.00	0.00
6	313.77	129.33	0.00	2.00	1.18	0.00	0.00
7	363.77	139.33	0.00	2.00	1.23	0.00	0.00
8	363.77	139.33	0.00	2.00	1.23	0.00	0.00
9	313.77	0.00	-82.15	2.00	1.48	0.00	3.39

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
10	313.77	0.00	-82.15	2.00	1.48	0.00	3.39
11	363.77	10.00	-82.15	1.95	1.55	0.00	3.39
12	363.77	10.00	-82.15	1.95	1.55	0.00	3.39
13	313.77	129.33	0.00	2.00	1.18	0.00	0.00
14	313.77	129.33	0.00	2.00	1.18	0.00	0.00
15	363.77	139.33	0.00	2.00	1.23	0.00	0.00
16	363.77	139.33	0.00	2.00	1.23	0.00	0.00
17	313.77	0.00	-82.15	2.00	1.48	0.00	3.39
18	313.77	0.00	-82.15	2.00	1.48	0.00	3.39
19	363.77	10.00	-82.15	1.95	1.55	0.00	3.39
20	363.77	10.00	-82.15	1.95	1.55	0.00	3.39

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	---	3.46	14.93	32.50	---
2	19.00	32.50	---	3.46	14.93	32.50	---
3	19.00	32.50	---	3.37	15.04	32.50	---
4	19.00	32.50	---	3.37	15.04	32.50	---
5	19.00	32.50	---	2.04	17.68	32.50	---
6	19.00	32.50	---	2.04	17.68	32.50	---
7	19.00	32.50	---	2.14	17.36	32.50	---
8	19.00	32.50	---	2.14	17.36	32.50	---
9	19.00	32.50	---	2.51	16.42	32.50	---
10	19.00	32.50	---	2.51	16.42	32.50	---
11	19.00	32.50	---	2.64	16.15	32.50	---
12	19.00	32.50	---	2.64	16.15	32.50	---
13	19.00	32.50	---	2.04	17.68	32.50	---
14	19.00	32.50	---	2.04	17.68	32.50	---
15	19.00	32.50	---	2.14	17.36	32.50	---
16	19.00	32.50	---	2.14	17.36	32.50	---
17	19.00	32.50	---	2.51	16.42	32.50	---
18	19.00	32.50	---	2.51	16.42	32.50	---
19	19.00	32.50	---	2.64	16.15	32.50	---
20	19.00	32.50	---	2.64	16.15	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 [8]
shape factors ν_b,ν_d,ν_c acc. to [8], tab.2
load inclination factors i_b,i_d,i_c acc. to [8], 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	---	0.700	1.537	---	1.000	1.000	---
2	15.03	24.58	---	0.700	1.537	---	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	---	0.824	1.316	---	1.000	1.000	---
6	15.03	24.58	---	0.824	1.316	---	1.000	1.000	---
7	15.03	24.58	---	0.815	1.331	---	1.000	1.000	---
8	15.03	24.58	---	0.815	1.331	---	1.000	1.000	---
9	15.03	24.58	---	0.779	1.397	---	0.972	0.983	---
10	15.03	24.58	---	0.779	1.397	---	0.972	0.983	---
11	15.03	24.58	---	0.761	1.428	---	0.976	0.986	---
12	15.03	24.58	---	0.761	1.428	---	0.976	0.986	---
13	15.03	24.58	---	0.824	1.316	---	1.000	1.000	---
14	15.03	24.58	---	0.824	1.316	---	1.000	1.000	---
15	15.03	24.58	---	0.815	1.331	---	1.000	1.000	---
16	15.03	24.58	---	0.815	1.331	---	1.000	1.000	---
17	15.03	24.58	---	0.779	1.397	---	0.972	0.983	---
18	15.03	24.58	---	0.779	1.397	---	0.972	0.983	---
19	15.03	24.58	---	0.761	1.428	---	0.976	0.986	---
20	15.03	24.58	---	0.761	1.428	---	0.976	0.986	---

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	3553.68	1.40	2538.34	313.77	0.12
2	3553.68	1.40	2538.34	423.59	0.17
3	3423.87	1.40	2445.62	388.77	0.16
4	3423.87	1.40	2445.62	498.59	0.20
5	1760.82	1.40	1257.73	313.77	0.25
6	1760.82	1.40	1257.73	423.59	0.34
7	1875.29	1.40	1339.49	388.77	0.29
8	1875.29	1.40	1339.49	498.59	**0.37**
9	2328.84	1.40	1663.46	313.77	0.19
10	2328.84	1.40	1663.46	423.59	0.25
11	2424.36	1.40	1731.69	388.77	0.22
12	2424.36	1.40	1731.69	498.59	0.29
13	1760.82	1.40	1257.73	313.77	0.25
14	1760.82	1.40	1257.73	423.59	0.34
15	1875.29	1.40	1339.49	388.77	0.29
16	1875.29	1.40	1339.49	498.59	0.37
17	2328.84	1.40	1663.46	313.77	0.19
18	2328.84	1.40	1663.46	423.59	0.25
19	2424.36	1.40	1731.69	388.77	0.22
20	2424.36	1.40	1731.69	498.59	0.29

$\mu_{max} = 0.37 < 1.0$ ⇒ **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	313.77	---	1.10	1.40	---	---	0.00	---
2	313.77	---	1.10	1.40	---	---	0.00	---
3	363.77	---	1.10	1.40	---	---	0.00	---
4	363.77	---	1.10	1.40	---	---	0.00	---
5	313.77	199.89	1.10	1.40	181.72	25.71	36.00	0.17
6	313.77	199.89	1.10	1.40	181.72	25.71	36.00	0.17
7	363.77	231.75	1.10	1.40	210.68	25.71	36.00	0.15
8	363.77	231.75	1.10	1.40	210.68	25.71	36.00	0.15
9	313.77	199.89	1.10	1.40	181.72	30.13	42.19	0.20
10	313.77	199.89	1.10	1.40	181.72	30.13	42.19	0.20
11	363.77	231.75	1.10	1.40	210.68	30.13	42.19	0.18
12	363.77	231.75	1.10	1.40	210.68	30.13	42.19	0.18
13	313.77	199.89	1.10	1.40	181.72	42.86	60.00	0.27
14	313.77	199.89	1.10	1.40	181.72	42.86	60.00	0.27
15	363.77	231.75	1.10	1.40	210.68	42.86	60.00	0.24
16	363.77	231.75	1.10	1.40	210.68	42.86	60.00	0.24
17	313.77	199.89	1.10	1.40	181.72	50.22	70.31	**0.30**
18	313.77	199.89	1.10	1.40	181.72	50.22	70.31	0.30
19	363.77	231.75	1.10	1.40	210.68	50.22	70.31	0.27
20	363.77	231.75	1.10	1.40	210.68	50.22	70.31	0.27

$\mu_{max} = 0.30 < 1.0$ ⇒ **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	213.77	0.00	0.00	0.00	0.00

6.2. limitation of gaping joint under permanent load

no eccentric loading ⇒ verification is not necessary.

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	4	Lc1+Lc2+Lc3
2	Lc1+Lc2	5	Lc1+Lc4
3	Lc1+Lc3	6	Lc1+Lc2+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	213.77	0.00	0.00	0.00	0.00	4	263.77	0.00	40.00	110.00	0.00
2	263.77	0.00	-0.00	10.00	0.00	5	213.77	46.88	0.00	0.00	-46.88
3	213.77	0.00	40.00	100.00	0.00	6	263.77	46.88	-0.00	10.00	-46.88

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	313.77	0.00	0.00	0.00	0.00	0.000
2	363.77	10.00	0.00	0.00	0.03	0.000
3	313.77	140.00	0.00	0.00	0.45	0.050
4	363.77	150.00	0.00	0.00	0.41	0.043
5	313.77	0.00	-93.75	-0.30	0.00	0.022
6	363.77	10.00	-93.75	-0.26	0.03	0.017

$((e_x/b_x)^2 + (e_y/b_y)^2)_{max} = 0.050 < 1/9$

**⇒ 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	313.77	---	1.10	---	0.00	---
2	313.77	---	1.10	---	0.00	---
3	363.77	---	1.10	---	0.00	---
4	363.77	---	1.10	---	0.00	---
5	313.77	199.89	1.10	181.72	36.00	0.20
6	313.77	199.89	1.10	181.72	36.00	0.20
7	363.77	231.75	1.10	210.68	36.00	0.17
8	363.77	231.75	1.10	210.68	36.00	0.17
9	313.77	199.89	1.10	181.72	42.19	0.23
10	313.77	199.89	1.10	181.72	42.19	0.23
11	363.77	231.75	1.10	210.68	42.19	0.20
12	363.77	231.75	1.10	210.68	42.19	0.20
13	313.77	199.89	1.10	181.72	60.00	0.33
14	313.77	199.89	1.10	181.72	60.00	0.33
15	363.77	231.75	1.10	210.68	60.00	0.28
16	363.77	231.75	1.10	210.68	60.00	0.28
17	313.77	199.89	1.10	181.72	70.31	**0.39**
18	313.77	199.89	1.10	181.72	70.31	0.39
19	363.77	231.75	1.10	210.68	70.31	0.33
20	363.77	231.75	1.10	210.68	70.31	0.33

$\mu_{max} = 0.39 < 1.0$ **⇒ 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	4	Lc1+Lc2+Lc3
2	Lc1+Lc2	5	Lc1+Lc4
3	Lc1+Lc3	6	Lc1+Lc2+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	213.77	0.00	0.00	0.00	0.00
2	263.77	0.00	-0.00	10.00	0.00
3	213.77	0.00	40.00	100.00	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
4	263.77	0.00	40.00	110.00	0.00
5	213.77	46.88	0.00	0.00	-46.88
6	263.77	46.88	-0.00	10.00	-46.88

6.7. settlements

determination of settlement by use of closed formulas acc. to [9]

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 $\sigma_B(z) = 0.2\cdot\sigma_{\ddot{u}}(z)$ 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	313.77	0.00	0.00	78.44	63.24	2.53
2	363.77	10.00	0.00	90.94	75.74	2.80
3	313.77	140.00	0.00	78.44	63.24	2.53
4	363.77	150.00	0.00	90.94	75.74	2.80
5	313.77	0.00	-93.75	78.44	63.24	2.53
6	363.77	10.00	-93.75	90.94	75.74	2.80

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

coefficient f for settlement below significant point acc. to [10], vol. 2, tab. 4
coefficients f_x/f_y for obliquity of a rigid foundation acc. to [11], 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' = 63.24 kN/m²
$M_{0,x}$ = 0.00 kNm
$M_{0,y}$ = 0.00 kNm

level m	z m	f -	f_x -	f_y -	s_m cm	s_x cm	s_y cm
2.50	1.70	0.450	3.422	3.422	0.57	0.00	0.00
3.33	2.53	0.543	3.693	3.693	0.12	0.00	0.00

LS 2:
σ_0' = 75.74 kN/m²
$M_{0,x}$ = 10.00 kNm
$M_{0,y}$ = 0.00 kNm

level m	z m	f -	f_x -	f_y -	s_m cm	s_x cm	s_y cm
2.50	1.70	0.450	3.422	3.422	0.68	0.00	0.04
3.60	2.80	0.566	3.731	3.731	0.17	0.00	0.00

LS 3:
σ_0' = 63.24 kN/m²
$M_{0,x}$ = 140.00 kNm
$M_{0,y}$ = 0.00 kNm

level m	z m	f -	f_x -	f_y -	s_m cm	s_x cm	s_y cm
2.50	1.70	0.450	3.422	3.422	0.57	0.00	0.60
3.33	2.53	0.543	3.693	3.693	0.12	0.00	0.05

LS 4:
σ_0' = 75.74 kN/m²
$M_{0,x}$ = 150.00 kNm
$M_{0,y}$ = 0.00 kNm

level m	z m	f -	f_x -	f_y -	s_m cm	s_x cm	s_y cm
2.50	1.70	0.450	3.422	3.422	0.68	0.00	0.64
3.60	2.80	0.566	3.731	3.731	0.17	0.00	0.06

LS 5:
σ_0' = 63.24 kN/m²
$M_{0,x}$ = 0.00 kNm
$M_{0,y}$ = -93.75 kNm

level m	z m	f -	f_x -	f_y -	s_m cm	s_x cm	s_y cm
2.50	1.70	0.450	3.422	3.422	0.57	-0.40	0.00
3.33	2.53	0.543	3.693	3.693	0.12	-0.03	0.00

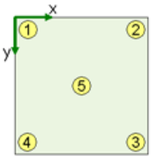
LS 6:
σ_0' = 75.74 kN/m²
$M_{0,x}$ = 10.00 kNm
$M_{0,y}$ = -93.75 kNm

level m	z m	f -	f_x -	f_y -	s_m cm	s_x cm	s_y cm
2.50	1.70	0.450	3.422	3.422	0.68	-0.40	0.04
3.60	2.80	0.566	3.731	3.731	0.17	-0.04	0.00

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	s1 cm	s2 cm	s3 cm	s4 cm	s5 cm	Smax cm	αx °	αy °
1	0.7	0.7	0.7	0.7	0.7	0.7	0.0	0.0
2	0.8	0.8	0.9	0.9	0.9	0.9	0.0	0.0
3	0.0	0.0	1.3	1.3	0.7	1.3	0.4	0.0
4	0.2	0.2	1.6	1.6	0.9	**1.6**	**0.4**	0.0
5	0.3	1.1	1.1	0.3	0.7	1.1	0.0	-0.2
6	0.4	1.2	1.3	0.5	0.9	1.3	0.0	**-0.3**

max s_{max} = 1.6 < 5.0 cm max $|\alpha_x|$ = 0.4° < 0.5° max $|\alpha_y|$ = 0.3° < 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 [12]
$k_s = E_s / (f \cdot (b_x \cdot b_x)^{0.5})$
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 κ = 1
constrained modulus $E_s = 1 \cdot 10000.00 = 10000.00$ kN/m²
shape factor f = 0.45
foundation modulus k_s = 11111.11 kN/m³
moment of inertia I_x / I_y = 1.33 / 1.33 kN/m³
torsional spring about the x-axis **$c_{v,x}$ = 14814.81 kNm**
torsional spring about the y-axis **$c_{v,y}$ = 14814.81 kNm**

8. Summary of foundation design

all executed verifications and design calculations successful.

required clamping depth of the column cross section	f_{min}	= 67.3 cm
chosen clamping depth	f_{gew}	= 68.0 > 67.3 cm
load-bearing cap. column cross-section	μ_{max}	= 0.72
weld between column and base plate	μ_{max}	= 0.39
introd. of normal force	μ_{max}	= 0.39
longit. reinf. x-direction (top)	exist $A_{so,x}$	= 3.1 cm²
longit. reinf. x-direct. (bottom)	exist $A_{su,x}$	= 4.7 cm²
longit. reinf. y-direction (top)	exist $A_{so,y}$	= 3.1 cm²
longit. reinf. y-direct. (bottom)	exist $A_{su,y}$	= 4.7 cm²
overturning	μ_{max}	= 0.74
base failure	μ_{max}	= 0.37
slip	μ_{max}	= 0.30
gaping joint under permant load	μ_{exist}	= 0.00
gaping joint under total load	μ_{max}	= 0.45
displacement in the base area	μ_{max}	= 0.39
settlement	s_{max}	= 1.6 < 5.0 cm
obliquity (about x-axis)	$\alpha_{max,x}$	= 0.4 < 0.5 °
obliquity (about y-axis)	$\alpha_{max,y}$	= 0.3 < 0.5 °
torsional spring (about the x-axis)	$c_{v,x}$	= 14815 kNm
torsional spring (about the y-axis)	$c_{v,y}$	= 14815 kNm

literature and standards:

[1] DIN 4085: Baugrund, Berechnung des Erddrucks, August 2017
[2] Kindmann, Kraus, Laumann, Vette: Verallgem. Berech.methode für in Beton eingesp. Stahlprofile, Stahlbau 92, Heft 1, Ernst & Sohn, 2023
[3] 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
[4] DIN EN 1992-1-1: Eurocode 2: Bemessung und Konstruktion von Stahlbeton- und Spannbetontragwerken, Teil 1-1, Januar 2011
[5] DIN EN 1993-1-8: Eurocode 3: Bemessung und Konstruktion von Stahlbauten - Teil 1-8: Bemessung von Anschlüssen, Dez. 2010
[6] DAfStb Heft 240: Hilfsmittel zur Berech. der Schnittgr. und Formänderungen von Stahlbetontragwerken, Beuth, 3.Aufl., 1991
[7] DIN 1054: Baugrund - Sicherheitsnachweise im Erd- und Grundbau - Ergänzende Regeln zu DIN EN 1997-1, April 2021
[8] DIN 4017: Baugrund, Berechnung des Grundbruchwiderstandes von Flächengründungen, März 2006
[9] DIN 4019: Baugrund - Setzungsberechnungen, Januar 2014
[10] Kany, M.: Berechnung von Flächengründungen, Verlag von Wilhelm Ernst & Sohn, 2.Aufl. 1974
[11] Sherif, G.; König, G.: Platten und Balken auf nachgiebigem Baugrund, Springer, 1975
[12] Rausch, E.: Maschinenfundamente, Betonkalender 1973, Teil 2, Ernst & Sohn