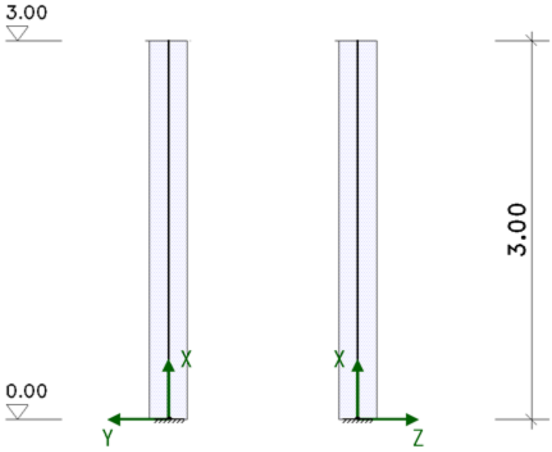
CANTILEVER COLUMN, HE300B

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

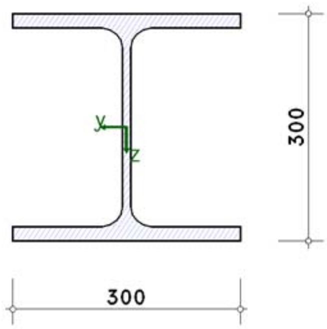
steel column

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 S235
section HE300B

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	----	----	----	----
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

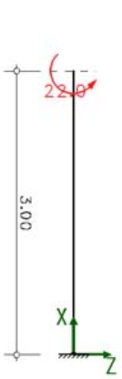
1: permanent loads	**permanent loads**
1: dead load	additive
4: live load	**transient live loads of storage rooms**
2: live load	additive
9: wind	**transient wind load**
3: Y-direction	alternative in group A
4: Z-direction	additive

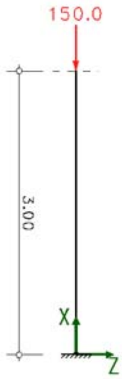
1.2. table of load pictures

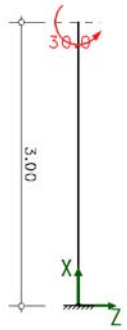
loadc.	loadpic.	introduct.	direction	value	entity
1	pointload	head	N	150.00	kN
			M_y	22.00	kNm
2	pointload	head	N	50.00	kN
			M_y	30.00	kNm
3	pointload	head	Q_y	15.00	kN
4	pointload	head	Q_z	30.00	kN

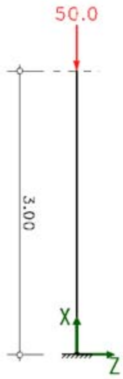
1.2.1. graphics of point loads

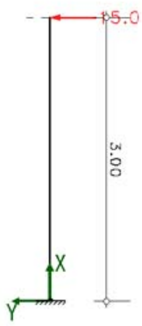
load case 1 (figure 1) load case 1 (figure 2) load case 2 (figure 3) load case 2 (figure 4) load case 3 (figure 5)



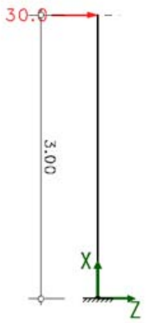








load case 4 (figure 6)



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 = 209.42 kN Σ H_y = 15.00 kN Σ H_z = 30.00 kN

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

cross sectional check: elastic 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	Φ	e	EI	N_{Ki}
		-	m	-	cm	MNm²	MN
Y	obliquity	2.00	6.00	0.0050	1.499	17.98	4.93
Z	obliquity	2.00	6.00	0.0050	1.499	52.84	14.50

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

$1 < N_{Ki,z}/N_{Ki,y} = 2.94 < 3 \Rightarrow$ interpolation between deformation direction and weak axis:

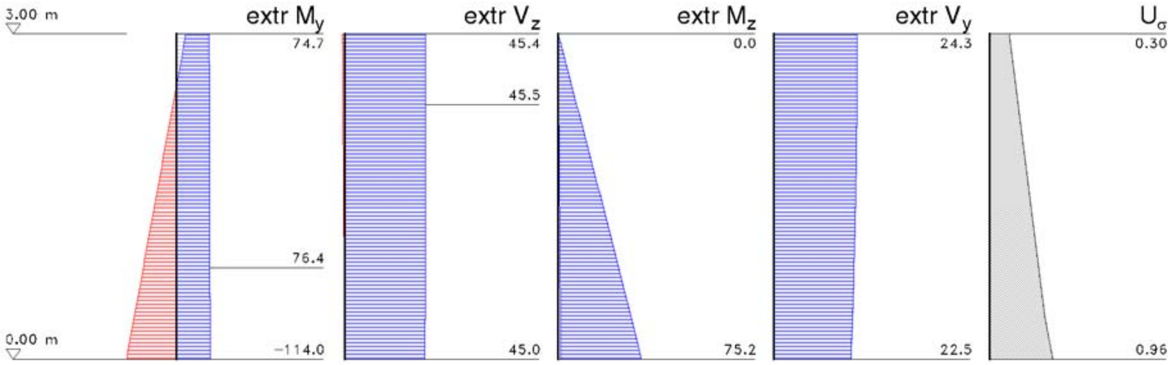
the decisive direction is interpolated between the direction of deformation due to the design load and the direction of least buckling safety.

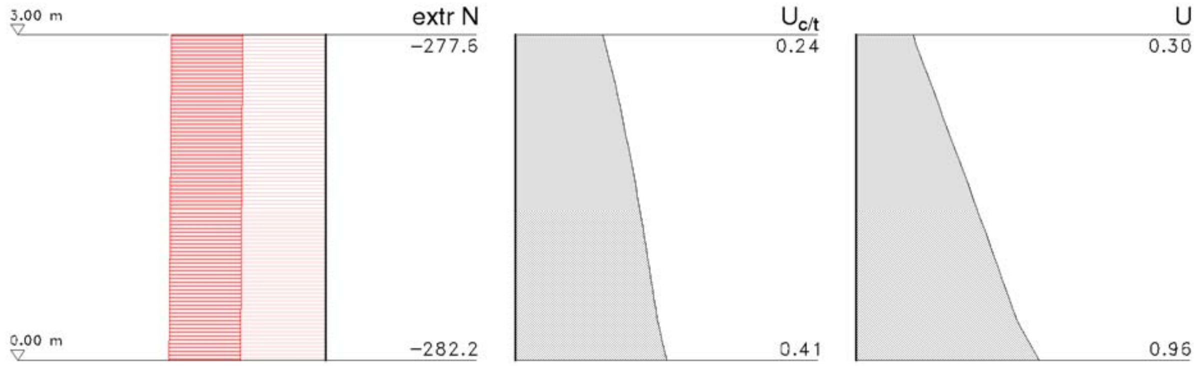
2.2. factorization of load case combinations

LS	design situat.	factorization
1	perman.	Lc1+0.99· I_y+0.14· I_z
2	perman.	1.35· Lc1+0.99· I_y+0.14· I_z
3	perman.	Lc1+1.5· Lc2+0.99· I_y+0.14· I_z
4	perman.	1.35· Lc1+1.5· Lc2+0.99· I_y+0.14· I_z
5	perman.	Lc1+0.6· 1.5· Lc3+I_y-0.008049· I_z
6	perman.	1.35· Lc1+0.6· 1.5· Lc3+I_y-0.0106· I_z
7	perman.	Lc1+1.5· Lc2+0.6· 1.5· Lc3+I_y-0.02082· I_z
8	perman.	1.35· Lc1+1.5· Lc2+0.6· 1.5· Lc3+I_y-0.02234· I_z
9	perman.	Lc1+0.6· 1.5· Lc4+I_y+0.04765· I_z
10	perman.	1.35· Lc1+0.6· 1.5· Lc4+I_y+0.04765· I_z
11	perman.	Lc1+1.5· Lc2+0.6· 1.5· Lc4+0.99· I_y+0.14· I_z
12	perman.	1.35· Lc1+1.5· Lc2+0.6· 1.5· Lc4+0.99· I_y+0.14· I_z
13	perman.	Lc1+0.6· 1.5· (Lc3+Lc4)+I_y+0.01141· I_z
14	perman.	1.35· Lc1+0.6· 1.5· (Lc3+Lc4)+I_y+0.008775· I_z
15	perman.	Lc1+1.5· Lc2+0.6· 1.5· (Lc3+Lc4)+I_y
16	perman.	1.35· Lc1+1.5· Lc2+0.6· 1.5· (Lc3+Lc4)+I_y-0.007481· I
17	perman.	Lc1+1.5· Lc3+I_y
18	perman.	1.35· Lc1+1.5· Lc3+I_y-0.006526· I_z
19	perman.	Lc1+1.5· (Lc2+Lc3)+I_y-0.01386· I_z
20	perman.	1.35· Lc1+1.5· (Lc2+Lc3)+I_y-0.01511· I_z
21	perman.	Lc1+1.5· Lc4+I_y+0.04765· I_z
22	perman.	1.35· Lc1+1.5· Lc4+I_y+0.04765· I_z
23	perman.	Lc1+1.5· (Lc2+Lc4)+I_y+0.04765· I_z
24	perman.	1.35· Lc1+1.5· (Lc2+Lc4)+I_y+0.04765· I_z
25	perman.	Lc1+1.5· (Lc3+Lc4)+I_y+0.01415· I_z
26	perman.	1.35· Lc1+1.5· (Lc3+Lc4)+I_y+0.01266· I_z
27	perman.	Lc1+1.5· (Lc2+Lc3+Lc4)+I_y+0.005058· I_z
28	perman.	1.35· Lc1+1.5· (Lc2+Lc3+Lc4)+I_y

2.3. extreme results

the internal forces are related to the deformed system axis





x	N min	N max	My min	My max	Vz min	Vz max	Mz min	Mz max	Vy min	Vy max	U
m	kN	kN	kNm	kNm	kN	kN	kNm	kNm	kN	kN	-
3.00	-277.55	**-149.75**	22.00	74.70	**-1.20**	45.44	**-0.00**	0.00	0.03	**24.26**	0.30
2.53	-278.28	-150.29	0.70	75.22	-1.01	45.48	0.36	12.02	0.03	24.22	0.40
2.44	-278.42	-150.41	-3.56	75.31	-0.98	45.48	0.43	14.42	0.03	24.21	0.42
2.34	-278.57	-150.52	-7.82	75.40	-0.94	**45.48**	0.51	16.82	0.03	24.18	0.44
0.00	**-282.24**	-153.51	**-113.99**	**76.51**	-0.00	45.00	2.31	**75.23**	**0.00**	22.50	**0.96**

relevant utilisation:

from load spectrum 26 at the location x = 0.00 m, with the internal forces and moments:
N = -207.24 kN, M_y = -106.40 kNm, V_z = 45.00 kN, M_z = 73.08 kNm, V_y = 22.50 kN

$U_{c/t}$ = 0.41, U_σ = 0.96 ⇒ max U = 0.96 < 1.0 ⇒ **permissible utilisation is complied with**

2.4. torsional-flexural buckling

verification of stability acc. to [2], section 6.3

note: fork support at the head and foot end of the column is assumed.

degree of arch tension: β_0 = 1.00

the cross-section is torsionally flexible.

evasive direction	β	buckl. curve	$N_{b,Rd}$	perman. a. variab. $N_{b,Rd}$	exceptional $N_{b,Rd}$
	-		kN	kN	kN
perpendic. to y-axis	2.00	b	0.00	0.00	0.00
perpendic. to z-axis	2.00	c	0.00	0.00	0.00

β - buckling coefficient $N_{b,Rd}$ - design value of the resistance to flexural buckling

LS	QKL	N_{Ed}	N_{Ed} / $N_{b,Rd,y}$	N_{Ed} / $N_{b,Rd,z}$	$M_{Ed,y}$	$M_{b,Rd,y}$	$M_{Ed,y}$ / $M_{b,Rd,y}$	$M_{Ed,z}$	$M_{b,Rd,z}$	$M_{Ed,z}$ / $M_{b,Rd,z}$
		kN	-	-	kNm	kNm	-	kNm	kNm	-
1		-∞	999.99	999.99	0.00	0.00	999.99	-∞	0.00	999.99
2		0.00	999.99	999.99	0.00	0.00	999.99	1.42E188	0.00	999.99
3		-0.00	999.99	999.99	0.00	0.00	999.99	3.38E125	0.00	999.99
4		0.00	999.99	999.99	0.00	0.00	999.99	0.00	0.00	999.99
5	3	153.51	0.05	0.08	22.29	358.29	0.06	43.89	121.99	0.36
6	3	207.24	0.07	0.10	30.22	358.29	0.08	45.12	121.99	0.37
7	3	228.51	0.08	0.11	68.31	358.29	0.19	45.63	121.99	0.37
8	3	282.24	0.10	0.14	76.51	358.29	0.21	46.90	121.99	0.38
9	3	153.51	0.05	0.08	59.53	358.29	0.17	2.33	121.99	0.02
10	3	207.24	0.07	0.10	51.88	358.29	0.14	3.18	121.99	0.03
11	3	228.51	0.08	0.11	67.00	358.29	0.19	3.50	121.99	0.03
12	3	282.24	0.10	0.14	74.70	358.29	0.21	4.36	121.99	0.04
13	3	153.51	0.05	0.08	59.44	358.29	0.17	43.89	121.99	0.36
14	3	207.24	0.07	0.10	51.76	358.29	0.14	45.12	121.99	0.37
15	3	228.51	0.08	0.11	67.00	358.29	0.19	45.63	121.99	0.37
16	3	282.24	0.10	0.14	74.70	358.29	0.21	46.90	121.99	0.38
17	3	153.51	0.05	0.08	22.29	358.29	0.06	71.59	121.99	0.59
18	3	207.24	0.07	0.10	30.22	358.29	0.08	73.08	121.99	0.60
19	3	228.51	0.08	0.11	68.31	358.29	0.19	73.70	121.99	0.60
20	3	282.24	0.10	0.14	76.51	358.29	0.21	75.23	121.99	0.62
21	3	153.51	0.05	0.08	113.99	358.29	0.32	2.33	121.99	0.02
22	3	207.24	0.07	0.10	106.51	358.29	0.30	3.18	121.99	0.03
23	3	228.51	0.08	0.11	68.61	358.29	0.19	3.53	121.99	0.03
24	3	282.24	0.10	0.14	74.70	358.29	0.21	4.40	121.99	0.04
25	3	153.51	0.05	0.08	113.92	358.29	0.32	71.59	121.99	0.59
26	3	207.24	0.07	0.10	106.40	358.29	0.30	73.08	121.99	0.60
27	3	228.51	0.08	0.11	68.47	358.29	0.19	73.70	121.99	0.60
28	3	282.24	0.10	0.14	74.70	358.29	0.21	75.23	121.99	0.62

LS	eq.	C_{my}	k_{yy}	C_{mLT}	k_{zy}	C_{mz}	k_{zz}	k_{yz}	U
1	(6.61)	---	---	---	---	---	---	---	-1.00
	(6.62)	---	---	---	---	---	---	---	-1.00
2	(6.61)	---	---	---	---	---	---	---	-1.00
	(6.62)	---	---	---	---	---	---	---	-1.00
3	(6.61)	---	---	---	---	---	---	---	-1.00
	(6.62)	---	---	---	---	---	---	---	-1.00
4	(6.61)	---	---	---	---	---	---	---	-1.00
	(6.62)	---	---	---	---	---	---	---	-1.00
5	(6.61)	0.950	0.965	0.950	---	---	---	0.986	0.05
	(6.62)	---	---	0.950	0.995	0.950	0.986	---	0.08
6	(6.61)	0.950	0.971	0.950	---	---	---	0.999	0.07
	(6.62)	---	---	0.950	0.994	0.950	0.999	---	0.10
7	(6.61)	0.950	0.973	0.950	---	---	---	1.004	0.08
	(6.62)	---	---	0.950	0.993	0.950	1.004	---	0.11
8	(6.61)	0.950	0.978	0.950	---	---	---	1.017	0.10
	(6.62)	---	---	0.950	0.992	0.950	1.017	---	0.14
9	(6.61)	0.950	0.965	0.950	---	---	---	0.986	0.11
	(6.62)	---	---	0.950	0.995	0.950	0.986	---	0.14
10	(6.61)	0.950	0.971	0.950	---	---	---	0.999	0.15
	(6.62)	---	---	0.950	0.994	0.950	0.999	---	0.18
11	(6.61)	0.950	0.973	0.950	---	---	---	1.004	0.26
	(6.62)	---	---	0.950	0.993	0.950	1.004	---	0.30
12	(6.61)	0.950	0.978	0.950	---	---	---	1.017	0.30
	(6.62)	---	---	0.950	0.992	0.950	1.017	---	**0.35**
13	(6.61)	0.950	0.965	0.950	---	---	---	0.986	0.11
	(6.62)	---	---	0.950	0.995	0.950	0.986	---	0.14

LS	eq.	c_{my}	k_{yy}	c_{mLT}	k_{zy}	c_{mz}	k_{zz}	k_{yz}	U
14	(6.61)	0.950	0.971	0.950	---	---	---	0.999	0.15
	(6.62)	---	---	0.950	0.994	0.950	0.999	---	0.18
15	(6.61)	0.950	0.973	0.950	---	---	---	1.004	0.26
	(6.62)	---	---	0.950	0.993	0.950	1.004	---	0.30
16	(6.61)	0.950	0.978	0.950	---	---	---	1.017	0.30
	(6.62)	---	---	0.950	0.992	0.950	1.017	---	0.35
17	(6.61)	0.950	0.965	0.950	---	---	---	0.986	0.05
	(6.62)	---	---	0.950	0.995	0.950	0.986	---	0.08
18	(6.61)	0.950	0.971	0.950	---	---	---	0.999	0.07
	(6.62)	---	---	0.950	0.994	0.950	0.999	---	0.10
19	(6.61)	0.950	0.973	0.950	---	---	---	1.004	0.08
	(6.62)	---	---	0.950	0.993	0.950	1.004	---	0.11
20	(6.61)	0.950	0.978	0.950	---	---	---	1.017	0.10
	(6.62)	---	---	0.950	0.992	0.950	1.017	---	0.14
21	(6.61)	0.950	0.965	0.950	---	---	---	0.986	0.11
	(6.62)	---	---	0.950	0.995	0.950	0.986	---	0.14
22	(6.61)	0.950	0.971	0.950	---	---	---	0.999	0.15
	(6.62)	---	---	0.950	0.994	0.950	0.999	---	0.18
23	(6.61)	0.950	0.973	0.950	---	---	---	1.004	0.26
	(6.62)	---	---	0.950	0.993	0.950	1.004	---	0.30
24	(6.61)	0.950	0.978	0.950	---	---	---	1.017	0.30
	(6.62)	---	---	0.950	0.992	0.950	1.017	---	0.35
25	(6.61)	0.950	0.965	0.950	---	---	---	0.986	0.11
	(6.62)	---	---	0.950	0.995	0.950	0.986	---	0.14
26	(6.61)	0.950	0.971	0.950	---	---	---	0.999	0.15
	(6.62)	---	---	0.950	0.994	0.950	0.999	---	0.18
27	(6.61)	0.950	0.973	0.950	---	---	---	1.004	0.26
	(6.62)	---	---	0.950	0.993	0.950	1.004	---	0.30
28	(6.61)	0.950	0.978	0.950	---	---	---	1.017	0.30
	(6.62)	---	---	0.950	0.992	0.950	1.017	---	0.35

2.4.1. verification for relevant load combination 12

flexural buckling for normal force (incl. drill buckling)
I_p = 33730 cm^4, I_T = 186 cm^4, i_p^2 = 22638 mm, i_m^2 = 22638 mm^2
flexural buckling about the y-axis (evasion ⊥ y-axis):
i_y = 130.0 mm, β_y = 1.999, $L_{cr,y}$ = 5.997 m, λ_1 = 93.913
λ_y = 0.491, y-buckling curve b ⇒ α_y = 0.34, Φ_y = 0.670, χ_y = 0.888, **$N_{by,Rd}$ = 2826.80 kN**
the y-buckling curve b was selected as user-defined!
flexural buckling around the z-axis (evasion ⊥ z-axis):
i_z = 75.8 mm, β_z = 1.999, $L_{cr,z}$ = 5.997 m, λ_1 = 93.913
λ_z = 0.842, z-buckling curve c ⇒ α_z = 0.49, Φ_z = 1.012, χ_z = 0.636, **$N_{bz,Rd}$ = 2022.96 kN**
the z-buckling curve c was selected as user-defined!

Utilizations

Lk	N_d kN	U_y -	U_z -
1	282.24	**0.100**	**0.140**

torsional-flexural buckling for bending about the y-axis
c^2 = 109253 mm^2, buckling curve b ⇒ α_{LT} = 0.34, N_{cr} = 4933.15 kN

Utilizations

Lk	M_{cr} kNm	λ_{LT} -	f -	Φ_{LT} -	χ_{LT} m	$\chi_{LT,mod}$ m	M_{Ed} kNm	$M_{b,Rd}$ kNm	U -
1	1839.39	0.463	0.977	0.591	0.975	0.998	74.70	358.29	**0.208**

bending around the z-axis

Utilizations

Lk	M_{Ed} kNm	$M_{b,Rd}$ kNm	U -
1	-0.00	121.99	**0.000**

interaction

Lk	eq.	C_{my}	k_{yy}	C_{mLT}	k_{zy}	C_{mz}	k_{zz}	k_{yz}	U
		-	-	-	-	-	-	-	-
1	(6.61)	0.950	0.978	0.950	---	---	---	1.017	0.304
	(6.62)	---	---	0.950	0.992	0.950	1.017	---	**0.346**

max U = 0.35 < 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

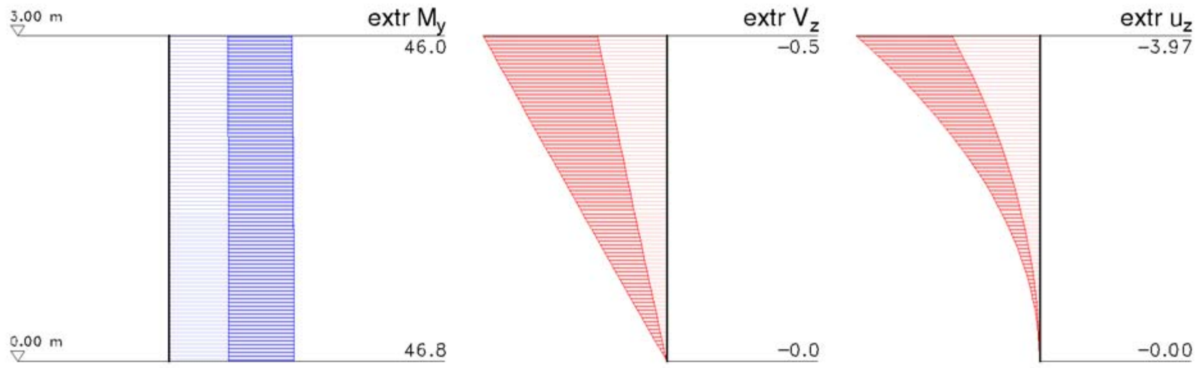
3.1. factorization of load case combinations

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
3.00	**22.00**	46.00	**-0.50**	-0.19	**-3.97**	-1.89
0.00	22.29	**46.76**	-0.00	**-0.00**	-0.00	**0.00**

relevant deformation in Z-direction:

from load spectrum 2 at the location x = 3.00 m, with the internal forces and moments:
N = -190.00 kN, M_y = 46.00 kNm, V_z = -0.50 kN, M_z = -0.00 kNm, V_y = -0.00 kN

u_z = -3.97 < 5.00 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	153.51	-0.00	0.00	22.00	-0.00
2	50.00	-0.00	0.00	30.00	-0.00
3	-0.00	15.00	-0.00	-0.00	45.00
4	0.00	-0.00	30.00	-90.00	-0.00
Σ	203.51	15.00	30.00	-38.00	45.00

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	153.51	0.00	-0.00	21.96	2.31
4	282.24	0.00	-0.00	75.90	4.36
8	282.24	13.50	-0.00	76.51	46.90
12	282.24	0.00	27.00	-6.41	4.36
17	153.51	22.50	0.00	22.29	71.59
21	153.51	0.00	45.00	-113.99	2.33
25	153.51	22.50	45.00	-113.92	71.59
28	282.24	22.50	45.00	-60.68	75.23

5. summary

all verifications were carried out successfully.

maximum utilisation of the verifications:	
design resist Th.II.0	96 %
tors-flex. buckl.	35 %
deformation in Z-direct.	79 %

literature and standards:

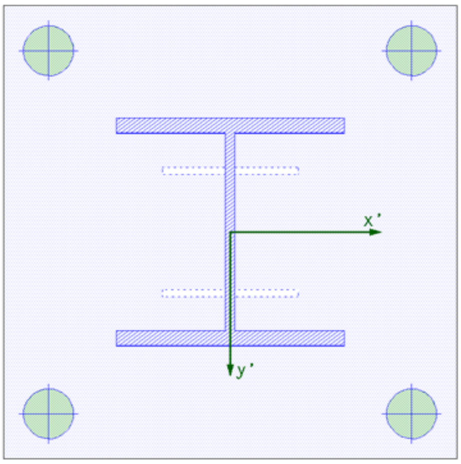
[1] DIN EN 1993-1-1/NA: Nationaler Anhang - National festgelegte Parameter - Eurocode 3: Teil 1-1, Okt. 2022
[2] DIN EN 1993-1-1: Eurocode 3: Bem. und Konstr. von Stahlbauten - Teil 1-1: Allg. Bem.regeln u. Regeln für den Hochbau, Dez. 2010
[3] DIN EN 1990: Grundlagen der Tragwerksplanung, Oktober 2021

BASE PLATE ON 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: HE300B, of quality S235

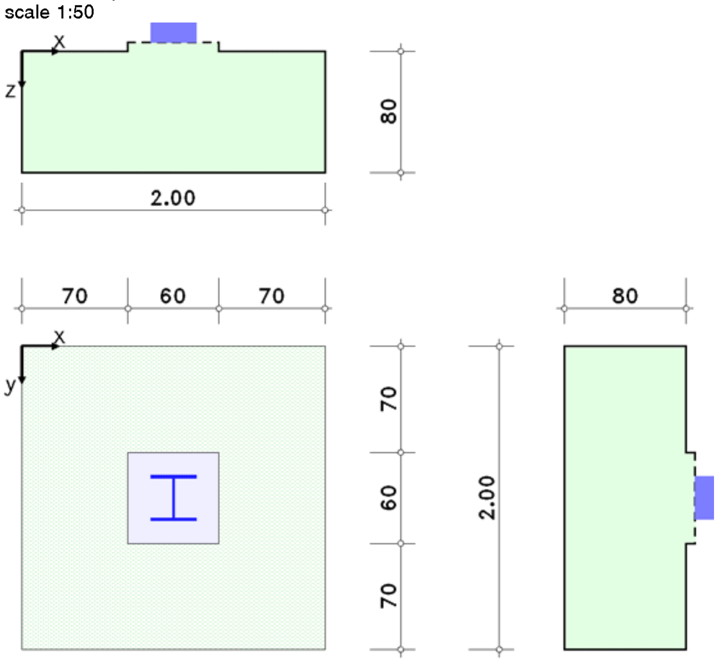
base plate
b_x = 600 mm b_y = 600 mm t = 60 mm, of quality S235

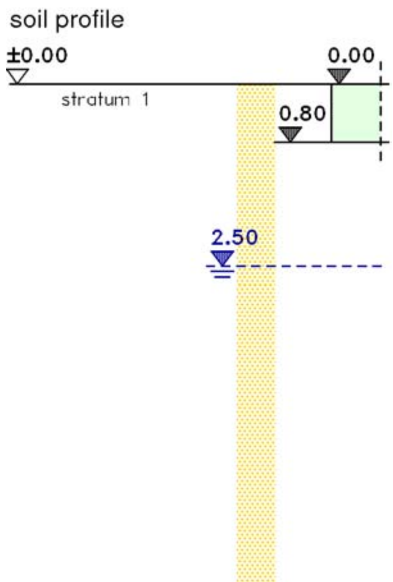
mortar joint
t_F = 20 mm

shear connector
standardized profile: HE180A, of quality S235
with a length of 100 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 1:50





concrete strength class **C25/30**
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.

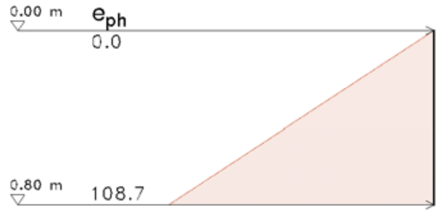
1.1. designation and characteristic values of soil strata

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 δ = 2/3·φ'_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

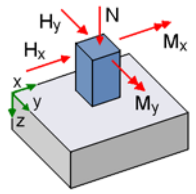
z m	$\Sigma(\gamma \cdot h)$ kN/m²	K_{pgh} -	e_{ph} kN/m²
0.00	0.00	7.152	0.00
0.80	15.20	7.152	108.71

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.	153.51	-0.00	0.00	2.31	21.96
6	LS 2 (auto)	perman. a.v.	207.24	-0.00	0.00	3.15	29.78
7	LS 3 (auto)	perman. a.v.	228.51	-0.00	0.00	3.50	67.82
8	LS 4 (auto)	perman. a.v.	282.24	-0.00	0.00	4.36	75.90
9	LS 5 (auto)	perman. a.v.	153.51	-0.00	13.50	43.89	22.29
10	LS 6 (auto)	perman. a.v.	207.24	0.00	13.50	45.12	30.22
11	LS 7 (auto)	perman. a.v.	228.51	-0.00	13.50	45.63	68.31
12	LS 8 (auto)	perman. a.v.	282.24	-0.00	13.50	46.90	76.51
13	LS 9 (auto)	perman. a.v.	153.51	27.00	0.00	2.33	-59.53
14	LS 10 (auto)	perman. a.v.	207.24	27.00	0.00	3.18	-51.88
15	LS 11 (auto)	perman. a.v.	228.51	27.00	0.00	3.50	-14.24
16	LS 12 (auto)	perman. a.v.	282.24	27.00	0.00	4.36	-6.41
17	LS 13 (auto)	perman. a.v.	153.51	27.00	13.50	43.89	-59.44
18	LS 14 (auto)	perman. a.v.	207.24	27.00	13.50	45.12	-51.76
19	LS 15 (auto)	perman. a.v.	228.51	27.00	13.50	45.63	-13.75
20	LS 16 (auto)	perman. a.v.	282.24	27.00	13.50	46.90	-5.80
21	LS 17 (auto)	perman. a.v.	153.51	0.00	22.50	71.59	22.29
22	LS 18 (auto)	perman. a.v.	207.24	0.00	22.50	73.08	30.22
23	LS 19 (auto)	perman. a.v.	228.51	-0.00	22.50	73.70	68.31
24	LS 20 (auto)	perman. a.v.	282.24	-0.00	22.50	75.23	76.51
25	LS 21 (auto)	perman. a.v.	153.51	45.00	0.00	2.33	-113.99
26	LS 22 (auto)	perman. a.v.	207.24	45.00	0.00	3.18	-106.51
27	LS 23 (auto)	perman. a.v.	228.51	45.00	0.00	3.53	-68.61
28	LS 24 (auto)	perman. a.v.	282.24	45.00	0.00	4.40	-60.87
29	LS 25 (auto)	perman. a.v.	153.51	45.00	22.50	71.59	-113.92
30	LS 26 (auto)	perman. a.v.	207.24	45.00	22.50	73.08	-106.40
31	LS 27 (auto)	perman. a.v.	228.51	45.00	22.50	73.70	-68.47
32	LS 28 (auto)	perman. a.v.	282.24	45.00	22.50	75.23	-60.68

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 $N_{0,deadl,k}$ = 80.00 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	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	153.51	-0.00	0.00	2.31	21.96
2	207.24	-0.00	0.00	3.15	29.78
3	228.51	-0.00	0.00	3.50	67.82
4	282.24	-0.00	0.00	4.36	75.90
5	153.51	-0.00	13.50	43.89	22.29
6	207.24	0.00	13.50	45.12	30.22
7	228.51	-0.00	13.50	45.63	68.31
8	282.24	-0.00	13.50	46.90	76.51
9	153.51	27.00	0.00	2.33	-59.53
10	207.24	27.00	0.00	3.18	-51.88
11	228.51	27.00	0.00	3.50	-14.24
12	282.24	27.00	0.00	4.36	-6.41
13	153.51	27.00	13.50	43.89	-59.44
14	207.24	27.00	13.50	45.12	-51.76

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
15	228.51	27.00	13.50	45.63	-13.75
16	282.24	27.00	13.50	46.90	-5.80
17	153.51	0.00	22.50	71.59	22.29
18	207.24	0.00	22.50	73.08	30.22
19	228.51	-0.00	22.50	73.70	68.31
20	282.24	-0.00	22.50	75.23	76.51
21	153.51	45.00	0.00	2.33	-113.99
22	207.24	45.00	0.00	3.18	-106.51
23	228.51	45.00	0.00	3.53	-68.61
24	282.24	45.00	0.00	4.40	-60.87
25	153.51	45.00	22.50	71.59	-113.92
26	207.24	45.00	22.50	73.08	-106.40
27	228.51	45.00	22.50	73.70	-68.47
28	282.24	45.00	22.50	75.23	-60.68

3.3. weld between column shaft and base plate

design with direction oriented method acc. to clause 4.5.3.2

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

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

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

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

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

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	$\tau_{\parallel}$ 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**	-4.29	-4.29	0.00	8.59	36.00	4.29	25.92	flange	0.24
2	8	8	-5.82	-5.82	0.00	11.63	36.00	5.82	25.92	flange	0.32
3	8	8	-11.59	-11.59	0.00	23.18	36.00	11.59	25.92	flange	0.64
4	8	8	-13.15	-13.15	0.00	26.30	36.00	13.15	25.92	flange	0.73
5	8	8	-6.73	-6.73	0.00	13.46	36.00	6.73	25.92	flange	0.37
6	8	8	-8.29	-8.29	0.00	16.59	36.00	8.29	25.92	flange	0.46
7	8	8	-14.08	-14.08	0.00	28.17	36.00	14.08	25.92	flange	0.78
8	8	8	-15.69	-15.69	0.00	31.37	36.00	15.69	25.92	flange	0.87
9	8	8	-9.85	-9.85	-0.32	19.70	36.00	9.85	25.92	flange	0.55
10	8	8	-9.09	-9.09	-0.32	18.18	36.00	9.09	25.92	flange	0.50
11	8	8	-3.67	-3.67	-0.32	7.35	36.00	3.67	25.92	flange	0.20
12	8	8	-2.88	-2.88	-0.32	5.78	36.00	2.88	25.92	flange	0.16
13	8	8	-12.22	-12.22	-0.32	24.46	36.00	12.22	25.92	flange	0.68
14	8	8	-11.48	-11.48	-0.32	22.96	36.00	11.48	25.92	flange	0.64
15	8	8	-6.01	-6.01	-0.32	12.04	36.00	6.01	25.92	flange	0.33
16	8	8	-5.23	-5.23	-0.32	10.48	36.00	5.23	25.92	flange	0.29
17	8	8	-8.32	-8.32	0.00	16.65	36.00	8.32	25.92	flange	0.46
18	8	8	-9.90	-9.90	0.00	19.80	36.00	9.90	25.92	flange	0.55
19	8	8	-15.70	-15.70	0.00	31.39	36.00	15.70	25.92	flange	0.87
20	8	8	-17.32	-17.32	0.00	34.63	36.00	17.32	25.92	flange	0.96
21	8	8	-17.90	-17.90	-0.53	35.82	36.00	17.90	25.92	flange	**0.99**
22	8	8	-17.16	-17.16	-0.53	34.34	36.00	17.16	25.92	flange	0.95
23	8	8	-11.71	-11.71	-0.53	23.43	36.00	11.71	25.92	flange	0.65
24	8	8	-10.93	-10.93	-0.53	21.88	36.00	10.93	25.92	flange	0.61
25	**10**	8	-17.59	-17.59	-0.42	35.18	36.00	17.59	25.92	flange	0.98

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 -
26	10	8	-17.04	-17.04	-0.42	34.09	36.00	17.04	25.92	flange	0.95
27	8	8	-15.72	-15.72	-0.53	31.45	36.00	15.72	25.92	flange	0.87
28	8	8	-14.98	-14.98	-0.53	29.97	36.00	14.98	25.92	flange	0.83

$a_{w,Fl}$ - 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 $a_{w,Fl,max}$ = 10 mm
maximum weld thickness of the web $a_{w,S,max}$ = 8 mm
maximum utilization U = 0.99 < 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 45 elements in X-direction and 49 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·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	14.0	15.3	6.42	6.26	0.04	-0.12	0.23
2	14.0	15.3	8.70	8.49	0.05	-0.16	0.32
3	14.0	16.5	22.54	16.98	0.21	-1.11	-0.71
4	14.0	16.5	24.63	19.23	0.24	-1.11	-0.82
5	14.0	16.5	11.35	13.76	1.57	0.19	-0.29
6	14.0	16.5	13.55	15.27	1.29	0.10	-0.48
7	14.0	16.5	25.46	23.71	1.91	-0.57	-0.71
8	14.0	16.5	27.63	25.83	1.77	-0.60	-0.85
9	46.0	16.5	21.29	14.63	-0.18	1.25	-0.57
10	46.0	16.5	17.19	13.64	-0.16	0.74	-0.59
11	46.0	15.3	6.32	6.59	0.04	0.05	0.23
12	46.0	15.3	5.99	6.56	0.08	0.01	0.21
13	12.7	43.5	-21.67	-25.33	4.28	-0.39	-0.21
14	46.0	16.5	20.50	20.25	-1.71	0.28	-0.60
15	46.0	16.5	10.77	13.16	-1.02	-0.21	-0.42
16	46.0	16.5	10.22	12.74	-0.68	-0.22	-0.47
17	47.3	44.7	-20.44	-23.02	-4.03	0.53	0.93
18	47.3	44.7	-20.25	-22.74	-3.55	0.50	0.92
19	47.3	44.7	-29.12	-33.05	-5.56	0.68	1.45
20	47.3	43.5	-28.68	-33.25	-4.69	0.53	-0.41
21	46.0	16.5	42.29	25.63	-0.26	3.10	-0.86
22	46.0	16.5	39.01	25.10	-0.29	2.58	-0.91
23	46.0	16.5	23.92	17.70	-0.22	1.22	-0.73
24	46.0	16.5	19.74	16.57	-0.18	0.73	-0.74
25	12.7	44.7	-45.38	-52.20	11.53	-1.05	2.58
26	12.7	44.7	-40.67	-46.69	9.59	-0.94	2.24
27	12.7	44.7	-29.84	-33.90	5.78	-0.70	1.50
28	12.7	44.7	-25.33	-28.66	4.24	-0.59	1.20

stresses and utilizations

$\sigma_{Pl,V} = (\sigma_x^2+\sigma_y^2-\sigma_x\cdot\sigma_y+3\cdot(\tau_{xy}^2+\tau_{xz}^2+\tau_{yz}^2))^{0.5}$
$\sigma_{Rd} = f_y/\gamma_{M0}$
$U = \sigma_{Pl,V}/\sigma_{Rd}$

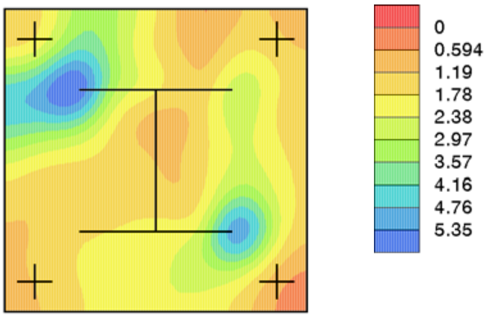
LS	x_{Fp} cm	y_{Fp} cm	$\sigma_{Pl,V}$ kN/cm²	σ_{Rd} kN/cm²	U -
1	14.0	15.3	0.71	21.50	0.03
2	14.0	15.3	0.96	21.50	0.04
3	14.0	16.5	2.29	21.50	0.11
4	14.0	16.5	2.52	21.50	0.12
5	14.0	16.5	1.45	21.50	0.07
6	14.0	16.5	1.63	21.50	0.08
7	14.0	16.5	2.77	21.50	0.13
8	14.0	16.5	3.01	21.50	0.14
9	46.0	16.5	2.13	21.50	0.10
10	46.0	16.5	1.77	21.50	0.08
11	46.0	15.3	0.72	21.50	0.03
12	46.0	15.3	0.70	21.50	0.03
13	12.7	43.5	2.76	21.50	0.13
14	46.0	16.5	2.30	21.50	0.11
15	46.0	16.5	1.37	21.50	0.06
16	46.0	16.5	1.31	21.50	0.06
17	47.3	44.7	2.57	21.50	0.12
18	47.3	44.7	2.51	21.50	0.12
19	47.3	44.7	3.66	21.50	0.17
20	47.3	43.5	3.59	21.50	0.17
21	46.0	16.5	4.20	21.50	0.20
22	46.0	16.5	3.89	21.50	0.18
23	46.0	16.5	2.42	21.50	0.11
24	46.0	16.5	2.06	21.50	0.10
25	12.7	44.7	5.95	21.50	**0.28**
26	12.7	44.7	5.27	21.50	0.25
27	12.7	44.7	3.76	21.50	0.18
28	12.7	44.7	3.15	21.50	0.15

maximum utilization U = 0.28 < 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 25 (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 cm²)

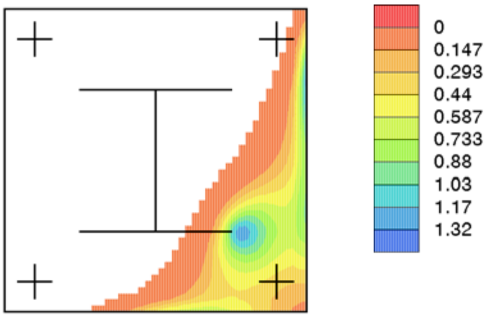
LS	lim $\sigma_{c,d}$ kN/cm²	A_{compr} cm²	F_{compr} kN	$\sigma_{c,max}$ kN/cm²	$\sigma_{c,m}$ kN/cm²	f_{jd} kN/cm²	U -
1	4.25	2619.2	154.75	0.28	0.059	1.42	0.04
2	4.25	2619.6	209.01	0.37	0.080	1.42	0.06
3	4.25	1477.6	277.19	0.69	0.188	1.42	0.13
4	4.25	1547.3	328.92	0.82	0.213	1.42	0.15
5	4.25	1490.2	193.51	0.49	0.130	1.42	0.09
6	4.25	1778.0	239.06	0.59	0.134	1.42	0.09
7	4.25	1433.1	294.71	0.94	0.206	1.42	0.15
8	4.25	1535.9	346.35	1.03	0.226	1.42	0.16
9	4.25	1271.0	214.02	0.51	0.168	1.42	0.12
10	4.25	1588.2	237.85	0.59	0.150	1.42	0.11
11	4.25	3576.3	228.51	0.29	0.064	1.42	0.05
12	4.25	3595.1	282.24	0.29	0.079	1.42	0.06
13	4.25	1192.7	231.71	0.79	0.194	1.42	0.14
14	4.25	1500.8	259.09	0.79	0.173	1.42	0.12
15	4.25	1886.1	249.78	0.50	0.132	1.42	0.09
16	4.25	2138.4	292.31	0.51	0.137	1.42	0.10
17	4.25	1231.8	245.80	0.57	0.200	1.42	0.14
18	4.25	1348.6	286.61	0.71	0.213	1.42	0.15
19	4.25	1252.7	326.53	1.07	0.261	1.42	0.18
20	4.25	1361.2	372.87	1.21	0.274	1.42	0.19
21	4.25	1026.9	318.82	0.78	0.310	1.42	0.22
22	4.25	1146.9	338.56	0.75	0.295	1.42	0.21
23	4.25	1429.4	283.23	0.70	0.198	1.42	0.14
24	4.25	1749.4	309.54	0.75	0.177	1.42	0.12
25	4.25	931.4	347.21	1.22	0.373	1.42	**0.26**
26	4.25	1058.0	367.54	1.32	0.347	1.42	0.25
27	4.25	1239.6	329.84	1.10	0.266	1.42	0.19
28	4.25	1439.6	359.84	1.08	0.250	1.42	0.18

maximum utilization U = 0.26 < 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 25 (max $\sigma_{c,m}$)



3.4.3. anchor tensile forces

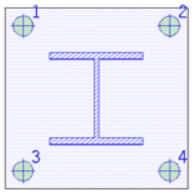
$F_{t,Rd} = k2 \cdot f_{ub} \cdot A_s / \gamma_{M2}$

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

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	---	1.19	---	0.05	470.36	0.00
2	---	1.67	---	0.10	470.36	0.00
3	---	26.56	---	22.12	470.36	0.06
4	---	25.99	---	20.70	470.36	0.06
5	6.71	33.30	---	---	470.36	0.07
6	0.76	31.06	---	---	470.36	0.07
7	---	59.86	---	6.34	470.36	0.13
8	---	58.38	---	5.73	470.36	0.12
9	31.88	---	28.63	---	470.36	0.07
10	17.19	---	13.42	---	470.36	0.04
11	---	---	---	---	470.36	0.00
12	---	---	---	---	470.36	0.00
13	67.28	1.33	9.59	---	470.36	0.14
14	49.92	---	1.92	---	470.36	0.11
15	18.66	2.62	---	---	470.36	0.04
16	8.54	1.53	---	---	470.36	0.02
17	31.20	61.08	---	---	470.36	0.13
18	20.54	58.84	---	---	470.36	0.13
19	7.06	87.35	---	3.61	470.36	0.19
20	2.94	84.84	---	2.85	470.36	0.18
21	84.46	---	80.85	---	470.36	0.18
22	68.00	---	63.32	---	470.36	0.14
23	29.65	---	25.07	---	470.36	0.06
24	16.01	---	11.29	---	470.36	0.03
25	148.65	7.71	37.34	---	470.36	**0.32**
26	130.12	4.98	25.20	---	470.36	0.28
27	89.93	6.74	4.65	---	470.36	0.19
28	73.43	4.17	---	---	470.36	0.16

maximum utilization U = 0.32 < 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. shear connector for transfer of horizontal force into the foundation

total length l = 10.0 cm

length in concrete l_c = 8.0 cm

3.5.1. concrete compression

$\sigma_c = V_{Ed} / (l_c \cdot b)$

$\sigma_{c,web,cal} = \sigma_{c,web} \cdot f_{\sigma,web}$

$U = \sigma_{c,max} / f_{cd}$

additional safety factor in case of concrete compressions by web $f_{\sigma,web}$ = 0.00

LK	$V_{Ed,flange}$ kN	$V_{Ed,web}$ kN	$\sigma_{c,flange}$ N/mm²	$\sigma_{c,web}$ N/mm²	$\sigma_{c,web,cal}$ N/mm²	f_{cd} N/mm²	U -
1	0.00	0.00	0.00	0.00	0.00	14.17	0.00
2	0.00	0.00	0.00	0.00	0.00	14.17	0.00
3	0.00	0.00	0.00	0.00	0.00	14.17	0.00
4	0.00	0.00	0.00	0.00	0.00	14.17	0.00
5	13.50	0.00	0.94	0.00	0.00	14.17	0.07
6	13.50	0.00	0.94	0.00	0.00	14.17	0.07
7	13.50	0.00	0.94	0.00	0.00	14.17	0.07
8	13.50	0.00	0.94	0.00	0.00	14.17	0.07
9	0.00	27.00	0.00	2.22	3.33	14.17	0.24
10	0.00	27.00	0.00	2.22	3.33	14.17	0.24
11	0.00	27.00	0.00	2.22	3.33	14.17	0.24
12	0.00	27.00	0.00	2.22	3.33	14.17	0.24
13	13.50	27.00	0.94	2.22	3.33	14.17	0.24
14	13.50	27.00	0.94	2.22	3.33	14.17	0.24
15	13.50	27.00	0.94	2.22	3.33	14.17	0.24
16	13.50	27.00	0.94	2.22	3.33	14.17	0.24
17	22.50	0.00	1.56	0.00	0.00	14.17	0.11
18	22.50	0.00	1.56	0.00	0.00	14.17	0.11
19	22.50	0.00	1.56	0.00	0.00	14.17	0.11
20	22.50	0.00	1.56	0.00	0.00	14.17	0.11
21	0.00	45.00	0.00	3.70	5.55	14.17	**0.39**
22	0.00	45.00	0.00	3.70	5.55	14.17	0.39
23	0.00	45.00	0.00	3.70	5.55	14.17	0.39
24	0.00	45.00	0.00	3.70	5.55	14.17	0.39
25	22.50	45.00	1.56	3.70	5.55	14.17	0.39
26	22.50	45.00	1.56	3.70	5.55	14.17	0.39
27	22.50	45.00	1.56	3.70	5.55	14.17	0.39
28	22.50	45.00	1.56	3.70	5.55	14.17	0.39

maximum utilization U = 0.39 < 1.00

$\sigma_{c,flange}$ - concrete compression by flange $\sigma_{c,web}$ - concrete compression by web   U - utilization

3.5.2. stresses in connection of base plate

$\sigma_{v,Ed} = (\sigma_{Ed}^2 + 3 \cdot \tau_{Ed}^2)^{0.5}$
$\sigma_{Rd} = f_y/\gamma_{M0}$
$u = \sigma_{v,Ed}/\sigma_{Rd}$

LS	$M_{x,Ed}$ kNcm	$M_{y,Ed}$ kNcm	σ_{Ed} kN/cm²	τ_{Ed} kN/cm²	$\sigma_{v,Ed}$ kN/cm²	σ_{Rd} kN/cm²	U -
1	-0.00	-0.00	0.00	0.00	0.00	23.50	0.00
2	-0.00	-0.00	0.00	0.00	0.00	23.50	0.00
3	-0.00	-0.00	0.00	0.00	0.00	23.50	0.00
4	-0.00	-0.00	0.00	0.00	0.00	23.50	0.00
5	81.00	-0.00	0.28	1.39	2.41	23.50	0.10
6	81.00	0.00	0.28	1.39	2.41	23.50	0.10
7	81.00	-0.00	0.28	1.39	2.41	23.50	0.10
8	81.00	-0.00	0.28	1.39	2.41	23.50	0.10
9	0.00	162.00	1.58	-1.18	2.05	23.50	0.09
10	0.00	162.00	1.58	-1.18	2.05	23.50	0.09
11	0.00	162.00	1.58	-1.18	2.05	23.50	0.09
12	0.00	162.00	1.58	-1.18	2.05	23.50	0.09
13	81.00	162.00	1.85	1.39	2.41	23.50	0.10
14	81.00	162.00	1.85	1.39	2.41	23.50	0.10
15	81.00	162.00	1.85	1.39	2.41	23.50	0.10
16	81.00	162.00	1.85	1.39	2.41	23.50	0.10
17	135.00	0.00	0.46	2.32	4.02	23.50	0.17
18	135.00	0.00	0.46	2.32	4.02	23.50	0.17
19	135.00	-0.00	0.46	2.32	4.02	23.50	0.17
20	135.00	-0.00	0.46	2.32	4.02	23.50	0.17
21	0.00	270.00	2.63	-1.97	3.41	23.50	0.15
22	0.00	270.00	2.63	-1.97	3.41	23.50	0.15
23	0.00	270.00	2.63	-1.97	3.41	23.50	0.15
24	0.00	270.00	2.63	-1.97	3.41	23.50	0.15
25	135.00	270.00	3.09	2.32	4.02	23.50	**0.17**
26	135.00	270.00	3.09	2.32	4.02	23.50	0.17
27	135.00	270.00	3.09	2.32	4.02	23.50	0.17
28	135.00	270.00	3.09	2.32	4.02	23.50	0.17

maximum utilization U = 0.17 < 1.00

$\sigma_{v,Ed}$ - equivalent tensile stress σ_{Rd} - limit normal stress τ_{Rd} - limit shear stress   U - utilization

3.5.3. weld between base plate and shear connector

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**	0.00	0.00	0.00	0.00	0.00	0.00	---	-	0.00
2	8	8	0.00	0.00	0.00	0.00	0.00	0.00	---	-	0.00
3	8	8	0.00	0.00	0.00	0.00	0.00	0.00	---	-	0.00
4	8	8	0.00	0.00	0.00	0.00	0.00	0.00	---	-	0.00
5	8	8	0.09	0.09	0.69	1.21	36.00	0.09	25.92	web	0.03
6	8	8	0.09	0.09	0.69	1.21	36.00	0.09	25.92	web	0.03
7	8	8	0.09	0.09	0.69	1.21	36.00	0.09	25.92	web	0.03
8	8	8	0.09	0.09	0.69	1.21	36.00	0.09	25.92	web	0.03
9	8	8	0.66	0.66	-0.52	1.61	36.00	0.66	25.92	flange	0.04
10	8	8	0.66	0.66	-0.52	1.61	36.00	0.66	25.92	flange	0.04
11	8	8	0.66	0.66	-0.52	1.61	36.00	0.66	25.92	flange	0.04
12	8	8	0.66	0.66	-0.52	1.61	36.00	0.66	25.92	flange	0.04
13	8	8	0.80	0.80	-0.52	1.83	36.00	0.80	25.92	flange	0.05
14	8	8	0.80	0.80	-0.52	1.83	36.00	0.80	25.92	flange	0.05
15	8	8	0.80	0.80	-0.52	1.83	36.00	0.80	25.92	flange	0.05
16	8	8	0.80	0.80	-0.52	1.83	36.00	0.80	25.92	flange	0.05
17	8	8	0.16	0.16	1.15	2.02	36.00	0.16	25.92	web	0.06
18	8	8	0.16	0.16	1.15	2.02	36.00	0.16	25.92	web	0.06
19	8	8	0.16	0.16	1.15	2.02	36.00	0.16	25.92	web	0.06
20	8	8	0.16	0.16	1.15	2.02	36.00	0.16	25.92	web	0.06
21	8	8	1.11	1.11	-0.87	2.68	36.00	1.11	25.92	flange	0.07
22	8	8	1.11	1.11	-0.87	2.68	36.00	1.11	25.92	flange	0.07
23	8	8	1.11	1.11	-0.87	2.68	36.00	1.11	25.92	flange	0.07
24	8	8	1.11	1.11	-0.87	2.68	36.00	1.11	25.92	flange	0.07
25	8	8	1.33	1.33	-0.87	3.06	36.00	1.33	25.92	flange	0.08
26	8	8	1.33	1.33	-0.87	3.06	36.00	1.33	25.92	flange	0.08
27	8	8	1.33	1.33	-0.87	3.06	36.00	1.33	25.92	flange	**0.08**
28	8	8	1.33	1.33	-0.87	3.06	36.00	1.33	25.92	flange	0.08

$a_{w,Fl}$ - 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 = 0.08 < 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
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

LS	design situat.	factorization
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	153.51	-0.00	0.00	2.31	21.96
2	153.51	-0.00	0.00	2.31	21.96
3	207.24	-0.00	0.00	3.15	29.78
4	207.24	-0.00	0.00	3.15	29.78
5	228.51	-0.00	0.00	3.50	67.82
6	228.51	-0.00	0.00	3.50	67.82
7	282.24	-0.00	0.00	4.36	75.90
8	282.24	-0.00	0.00	4.36	75.90
9	153.51	-0.00	13.50	43.89	22.29
10	153.51	-0.00	13.50	43.89	22.29
11	207.24	0.00	13.50	45.12	30.22
12	207.24	0.00	13.50	45.12	30.22
13	228.51	-0.00	13.50	45.63	68.31
14	228.51	-0.00	13.50	45.63	68.31
15	282.24	-0.00	13.50	46.90	76.51
16	282.24	-0.00	13.50	46.90	76.51
17	153.51	27.00	0.00	2.33	-59.53
18	153.51	27.00	0.00	2.33	-59.53
19	207.24	27.00	0.00	3.18	-51.88
20	207.24	27.00	0.00	3.18	-51.88
21	228.51	27.00	0.00	3.50	-14.24
22	228.51	27.00	0.00	3.50	-14.24
23	282.24	27.00	0.00	4.36	-6.41
24	282.24	27.00	0.00	4.36	-6.41
25	153.51	27.00	13.50	43.89	-59.44
26	153.51	27.00	13.50	43.89	-59.44
27	207.24	27.00	13.50	45.12	-51.76
28	207.24	27.00	13.50	45.12	-51.76
29	228.51	27.00	13.50	45.63	-13.75
30	228.51	27.00	13.50	45.63	-13.75
31	282.24	27.00	13.50	46.90	-5.80
32	282.24	27.00	13.50	46.90	-5.80
33	153.51	0.00	22.50	71.59	22.29
34	153.51	0.00	22.50	71.59	22.29
35	207.24	0.00	22.50	73.08	30.22
36	207.24	0.00	22.50	73.08	30.22
37	228.51	-0.00	22.50	73.70	68.31
38	228.51	-0.00	22.50	73.70	68.31
39	282.24	-0.00	22.50	75.23	76.51
40	282.24	-0.00	22.50	75.23	76.51
41	153.51	45.00	0.00	2.33	-113.99
42	153.51	45.00	0.00	2.33	-113.99
43	207.24	45.00	0.00	3.18	-106.51
44	207.24	45.00	0.00	3.18	-106.51
45	228.51	45.00	0.00	3.53	-68.61
46	228.51	45.00	0.00	3.53	-68.61
47	282.24	45.00	0.00	4.40	-60.87
48	282.24	45.00	0.00	4.40	-60.87
49	153.51	45.00	22.50	71.59	-113.92
50	153.51	45.00	22.50	71.59	-113.92
51	207.24	45.00	22.50	73.08	-106.40
52	207.24	45.00	22.50	73.08	-106.40
53	228.51	45.00	22.50	73.70	-68.47
54	228.51	45.00	22.50	73.70	-68.47
55	282.24	45.00	22.50	75.23	-60.68
56	282.24	45.00	22.50	75.23	-60.68

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 = 70.0 cm kNm	x = 130.0 cm kNm
1	24.99	12.62
2	24.99	12.62
3	33.78	17.00
4	33.78	17.00
5	47.10	8.88
6	47.10	8.88
7	55.96	13.19
8	55.96	13.19
9	25.08	12.53
10	25.08	12.53
11	33.90	16.87
12	33.90	16.87
13	47.29	8.83
14	47.25	8.76
15	56.14	13.04
16	56.13	13.02
17	-4.02	41.68
18	-4.05	41.66
19	4.68	46.09
20	4.68	46.09
21	17.90	38.09
22	17.90	38.09
23	26.68	42.46
24	26.68	42.46
25	-2.60	42.63
26	-3.18	42.18
27	5.01	46.23
28	4.83	46.12
29	18.03	37.95
30	18.03	37.95
31	26.85	42.29
32	26.85	42.29
33	25.24	12.69
34	25.17	12.64
35	33.99	17.00
36	33.93	16.91
37	47.76	9.47
38	47.55	9.21
39	56.43	13.48
40	56.29	13.27
41	-9.80	73.65
42	**-13.23**	68.67
43	-9.40	68.94
44	-11.64	67.41
45	-1.47	57.48
46	-1.48	57.47
47	7.28	61.87
48	7.28	61.87
49	-9.80	**75.63**
50	-12.93	71.25
51	-7.82	71.77
52	-9.28	70.13
53	0.89	59.28
54	0.33	58.72
55	8.38	62.53
56	8.04	62.27

design calculation for LS 42: $\varepsilon_o/\varepsilon_u$ = 26.68/-0.21‰ **min $A_{s,o}$ = 0.4 cm²**

design calculation for LS 49: $\varepsilon_o/\varepsilon_u$ = -0.52/26.70‰ **min $A_{s,u}$ = 2.2 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 = 70.0 cm kNm	y = 130.0 cm kNm	LS	y = 70.0 cm kNm	y = 130.0 cm kNm	LS	y = 70.0 cm kNm	y = 130.0 cm kNm
1	18.15	19.46	20	24.49	26.28	39	8.82	61.16
2	18.15	19.46	21	27.01	28.98	40	8.59	61.01
3	24.50	26.27	22	27.01	28.98	41	18.15	19.46
4	24.50	26.27	23	33.35	35.80	42	18.15	19.46
5	27.01	28.98	24	33.35	35.80	43	24.49	26.28
6	27.01	28.98	25	4.25	34.94	44	24.49	26.28
7	33.35	35.80	26	3.98	34.65	45	27.00	28.99
8	33.35	35.80	27	9.89	41.30	46	27.00	28.99
9	3.40	34.21	28	9.74	41.20	47	33.34	35.81
10	3.40	34.21	29	12.09	43.89	48	33.34	35.81
11	9.63	41.14	30	12.09	43.89	49	-3.38	47.10
12	9.63	41.14	31	18.32	50.83	50	-3.80	46.69
13	12.18	43.94	32	18.32	50.83	51	2.17	53.49
14	12.11	43.90	33	-5.63	44.53	52	1.91	53.22
15	18.34	50.84	34	**-6.15**	44.21	53	4.07	55.46
16	18.32	50.83	35	-0.06	51.17	54	3.65	55.00
17	18.15	19.46	36	-0.22	51.08	55	9.32	**61.54**
18	18.15	19.46	37	3.10	54.44	56	8.99	61.29
19	24.49	26.28	38	2.72	54.17			

design calculation for LS 34: $\varepsilon_o/\varepsilon_u$ = 27.04/-0.14‰ **min $A_{s,o}$ = 0.2 cm²**

design calculation for LS 55: $\varepsilon_o/\varepsilon_u$ = -0.47/27.07‰ **min $A_{s,u}$ = 1.8 cm²**

4.3.3. selected reinforcement in x-direction

top B500A, to distribute evenly
4 Ø 10 = 3.1 > 0.4 cm²

bottom (distribution acc. to [2])

width	cm	200.0	select.		3 Ø 10
distrib.	%	100.0	exist A_s	cm²	2.4
min A_s	cm²	2.2			

total existing bottom reinforcement: ΣA_s = 2.4 > 2.2 cm²

4.3.4. selected reinforcement in y-direction

top B500A, to distribute evenly
4 Ø 10 = 3.1 > 0.2 cm²

bottom (distribution acc. to [2])

width	cm	200.0	select.		3 Ø 10
distrib.	%	100.0	exist A_s	cm²	2.4
min A_s	cm²	1.8			

total existing bottom reinforcement: ΣA_s = 2.4 > 1.8 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 [3], table 6.1)

$c_1 = c_2 = 0.6 \Rightarrow k_x = k_y = 0.6$

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	30.2	0.41	4.30	1.370	1.8350	1.8350	9	28.3	0.38	4.18	1.291	1.7339	1.7339
2	30.2	0.41	4.30	1.370	1.8350	1.8350	10	28.3	0.38	4.18	1.291	1.7339	1.7339
3	30.2	0.41	4.30	1.370	1.8350	1.8350	11	28.7	0.38	4.20	1.307	1.7539	1.7539
4	30.2	0.41	4.30	1.370	1.8350	1.8350	12	28.7	0.38	4.20	1.307	1.7539	1.7539
5	28.3	0.38	4.18	1.291	1.7339	1.7339	13	27.9	0.38	4.16	1.276	1.7141	1.7141
6	28.3	0.38	4.18	1.291	1.7339	1.7339	14	27.9	0.38	4.16	1.276	1.7141	1.7141
7	28.7	0.38	4.20	1.307	1.7539	1.7539	15	28.3	0.38	4.18	1.291	1.7339	1.7339
8	28.7	0.38	4.20	1.307	1.7539	1.7539	16	28.3	0.38	4.18	1.291	1.7339	1.7339

LS	a_{crit} cm	a/d -	u_{crit} m	A_{crit} m²	$W_{crit,x}$ m²	$W_{crit,y}$ m²
17	27.6	0.37	4.13	1.260	1.6943	1.6943
18	27.6	0.37	4.13	1.260	1.6943	1.6943
19	28.7	0.38	4.20	1.307	1.7539	1.7539
20	28.7	0.38	4.20	1.307	1.7539	1.7539
21	31.7	0.43	4.39	1.435	1.9178	1.9178
22	31.7	0.43	4.39	1.435	1.9178	1.9178
23	31.7	0.43	4.39	1.435	1.9178	1.9178
24	31.7	0.43	4.39	1.435	1.9178	1.9178
25	27.2	0.37	4.11	1.245	1.6746	1.6746
26	27.2	0.37	4.11	1.245	1.6746	1.6746
27	27.9	0.38	4.16	1.276	1.7141	1.7141
28	27.9	0.38	4.16	1.276	1.7141	1.7141
29	29.1	0.39	4.23	1.323	1.7740	1.7740
30	29.1	0.39	4.23	1.323	1.7740	1.7740
31	30.2	0.41	4.30	1.370	1.8350	1.8350
32	30.2	0.41	4.30	1.370	1.8350	1.8350
33	27.2	0.37	4.11	1.245	1.6746	1.6746
34	27.2	0.37	4.11	1.245	1.6746	1.6746
35	27.6	0.37	4.13	1.260	1.6943	1.6943
36	27.6	0.37	4.13	1.260	1.6943	1.6943
37	27.6	0.37	4.13	1.260	1.6943	1.6943
38	27.6	0.37	4.13	1.260	1.6943	1.6943
39	27.9	0.38	4.16	1.276	1.7141	1.7141
40	27.9	0.38	4.16	1.276	1.7141	1.7141
41	27.6	0.37	4.13	1.260	1.6943	1.6943
42	27.6	0.37	4.13	1.260	1.6943	1.6943
43	26.8	0.36	4.09	1.230	1.6551	1.6551
44	26.8	0.36	4.09	1.230	1.6551	1.6551
45	28.3	0.38	4.18	1.291	1.7339	1.7339
46	28.3	0.38	4.18	1.291	1.7339	1.7339
47	29.1	0.39	4.23	1.323	1.7740	1.7740
48	29.1	0.39	4.23	1.323	1.7740	1.7740
49	28.3	0.38	4.18	1.291	1.7339	1.7339
50	28.3	0.38	4.18	1.291	1.7339	1.7339
51	27.6	0.37	4.13	1.260	1.6943	1.6943
52	27.6	0.37	4.13	1.260	1.6943	1.6943
53	27.6	0.37	4.13	1.260	1.6943	1.6943
54	27.6	0.37	4.13	1.260	1.6943	1.6943
55	27.9	0.38	4.16	1.276	1.7141	1.7141
56	27.9	0.38	4.16	1.276	1.7141	1.7141

decisive shear stress within the basic control perimeter

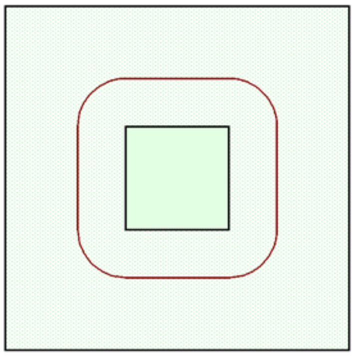
LS	V_{Ed} kN	$\sigma_{Ed,gd,m}$ kN/m²	ΔV_{Ed} kN	$M_{Ed,x,Sp}$ kNm	$M_{Ed,y,Sp}$ kNm	β -	$v_{Ed,crit}$ N/mm²
1	153.51	38.42	52.64	2.31	-21.96	1.20	0.038
2	153.51	38.42	52.64	2.31	-21.96	1.20	0.038
3	207.24	51.86	71.05	3.15	-29.78	1.20	0.051
4	207.24	51.86	71.05	3.15	-29.78	1.20	0.051
5	228.51	57.18	73.83	3.50	-67.82	1.43	0.071
6	228.51	57.18	73.83	3.50	-67.82	1.43	0.071
7	282.24	70.62	92.29	4.36	-75.90	1.39	0.084
8	282.24	70.62	92.29	4.36	-75.90	1.39	0.084
9	153.51	38.42	49.61	43.89	-22.29	1.46	0.049
10	153.51	38.42	49.61	43.89	-22.29	1.46	0.049
11	207.24	51.86	67.77	45.12	-30.22	1.38	0.061
12	207.24	51.86	67.77	45.12	-30.22	1.38	0.061
13	228.51	57.17	72.93	45.63	-68.31	1.52	0.077
14	228.51	57.17	72.93	45.63	-68.31	1.52	0.077
15	282.24	70.62	91.19	46.90	-76.51	1.46	0.090
16	282.24	70.62	91.19	46.90	-76.51	1.46	0.090
17	153.51	38.42	48.42	2.33	59.53	1.57	0.054
18	153.51	38.42	48.42	2.33	59.53	1.57	0.054
19	207.24	51.86	67.77	3.18	51.88	1.36	0.061
20	207.24	51.86	67.77	3.18	51.88	1.36	0.061
21	228.51	57.18	82.04	3.50	14.24	1.10	0.049
22	228.51	57.18	82.04	3.50	14.24	1.10	0.049
23	282.24	70.62	101.33	4.36	6.41	1.10	0.061
24	282.24	70.62	101.33	4.36	6.41	1.10	0.061
25	153.51	38.17	47.52	43.89	59.44	1.71	0.059
26	153.51	38.17	47.52	43.89	59.44	1.71	0.059
27	207.24	51.86	66.15	45.12	51.76	1.48	0.068
28	207.24	51.86	66.15	45.12	51.76	1.48	0.068
29	228.51	57.18	75.62	45.63	13.75	1.30	0.063
30	228.51	57.18	75.62	45.63	13.75	1.30	0.063
31	282.24	70.62	96.76	46.90	5.80	1.24	0.072
32	282.24	70.62	96.76	46.90	5.80	1.24	0.072
33	153.51	38.33	47.71	71.59	-22.29	1.72	0.059
34	153.51	38.33	47.71	71.59	-22.29	1.72	0.059
35	207.24	51.85	65.35	73.08	-30.22	1.56	0.072
36	207.24	51.85	65.35	73.08	-30.22	1.56	0.072
37	228.51	56.71	71.47	73.70	-68.31	1.64	0.084
38	228.51	56.71	71.47	73.70	-68.31	1.64	0.084
39	282.24	70.44	89.86	75.23	-76.51	1.55	0.097
40	282.24	70.44	89.86	75.23	-76.51	1.55	0.097
41	153.51	33.11	41.72	2.33	113.99	2.09	0.076
42	153.51	33.11	41.72	2.33	113.99	2.09	0.076
43	207.24	51.59	63.44	3.18	106.51	1.76	0.083
44	207.24	51.59	63.44	3.18	106.51	1.76	0.083
45	228.51	57.18	73.83	3.53	68.61	1.43	0.071
46	228.51	57.18	73.83	3.53	68.61	1.43	0.071
47	282.24	70.62	93.40	4.40	60.87	1.31	0.079

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
48	282.24	70.62	93.40	4.40	60.87	1.31	0.079
49	153.51	25.70	33.18	71.59	113.92	2.27	0.088
50	153.51	25.70	33.18	71.59	113.92	2.27	0.088
51	207.24	46.99	59.22	73.08	106.40	1.91	0.092
52	207.24	46.99	59.22	73.08	106.40	1.91	0.092
53	228.51	56.70	71.46	73.70	68.47	1.64	0.084
54	228.51	56.70	71.46	73.70	68.47	1.64	0.084
55	282.24	70.59	90.06	75.23	60.68	1.50	0.093
56	282.24	70.59	90.06	75.23	60.68	1.50	0.093

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]
$C_{Rd,c} = 0.15/\gamma_c$
$k = 1 + \mathrm{sqrt}(200/d) \leq 2.0$ with d [mm]
$\rho_{l,tension,max}$ = minimum from $(0.02,\ 0.5 \cdot f_{cd}/f_{yd})$
$\rho_{l,tension} = \mathrm{sqrt}(\rho_{lx,tension} \cdot \rho_{ly,tension}) \leq \rho_{l,tension,max}$
$v_{min} = 0.0525/\gamma_c \cdot k^{3/2} \cdot f_{ck}^{1/2}$ for $d \leq 600$ mm
$v_{min} = 0.0375/\gamma_c \cdot k^{3/2} \cdot f_{ck}^{1/2}$ for $d \geq 800$ mm



mean effective depth
$d_m = (75 + 74)/2 = 74.5$ cm

scale factor
$k = 1 + \mathrm{sqrt}(200/745) = 1.52 < 2$

longitudinal reinf. ratio of the anchored tension reinf.
mean of the tension reinforcement to the distance 3d from the column
$a_{s,x,3d} = 2.4/2 = 1.18$ cm²/m
$a_{s,y,3d} = 2.4/2 = 1.18$ cm²/m
$\rho_{lx,tension} = 1.18/75 \cdot 10^{-2} = 0.00016$
$\rho_{ly,tension} = 1.18/74 \cdot 10^{-2} = 0.00016$
$\rho_{l,tension} = \mathrm{sqrt}(0.00016 \cdot 0.00016) = 0.00016$

punching shear resistance without shear reinforcement
$C_{Rd,c} = 0.15/1.5 = 0.1$
$\rho_{l,tension,max}$ = minimum from $(0.02,\ 0.5 \cdot 14.17/434.78) = 0.0163 > 0.0002$
$v_{min} \cdot 2 \cdot d/a = 0.0416/1.5 \cdot 1.52^{3/2} \cdot 25^{0.5} \cdot 2 \cdot 74.5/27.9 = 1.384$ N/mm²
$v_{Rd,c} = 0.1 \cdot 1.52 \cdot (100 \cdot 0.00016 \cdot 25)^{1/3} \cdot 2 \cdot 74.5/27.9 = 0.595$ N/mm² < 1.384 N/mm² $\Rightarrow v_{Rd,c} = 1.384$ N/mm²

0.097 N/mm² < 1.384 N/mm² ⇒ no additional reinforcement required

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	138.16	0.00	-0.00	-0.00	19.80
2	168.86	0.00	-0.00	-0.00	24.20
3	213.16	0.00	-0.00	-0.00	64.80
4	243.86	0.00	-0.00	-0.00	69.20
5	138.16	0.00	13.50	40.50	19.80
6	168.86	0.00	13.50	40.50	24.20
7	213.16	0.00	13.50	40.50	64.80
8	243.86	0.00	13.50	40.50	69.20
9	138.16	27.00	-0.00	-0.00	-61.20
10	168.86	27.00	-0.00	-0.00	-56.80
11	213.16	27.00	-0.00	-0.00	-16.20
12	243.86	27.00	-0.00	-0.00	-11.80

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
13	138.16	27.00	13.50	40.50	-61.20
14	168.86	27.00	13.50	40.50	-56.80
15	213.16	27.00	13.50	40.50	-16.20
16	243.86	27.00	13.50	40.50	-11.80
17	138.16	0.00	22.50	67.50	19.80
18	168.86	0.00	22.50	67.50	24.20
19	213.16	0.00	22.50	67.50	64.80
20	243.86	0.00	22.50	67.50	69.20
21	138.16	45.00	-0.00	-0.00	-115.20
22	168.86	45.00	-0.00	-0.00	-110.80
23	213.16	45.00	-0.00	-0.00	-70.20
24	243.86	45.00	-0.00	-0.00	-65.80
25	138.16	45.00	22.50	67.50	-115.20
26	168.86	45.00	22.50	67.50	-110.80
27	213.16	45.00	22.50	67.50	-70.20
28	243.86	45.00	22.50	67.50	-65.80

5.4. verification against overturning (EQU)

LS	decisive edge	M_{dst} kNm	M_{stb} kNm	μ -
1	---	0.00	190.36	0.00
2	---	0.00	232.66	0.00
3	---	0.00	220.36	0.00
4	---	0.00	262.66	0.00
5	x (bottom)	51.30	210.16	0.24
6	x (bottom)	51.30	256.86	0.20
7	x (bottom)	51.30	285.16	0.18
8	x (bottom)	51.30	331.86	0.15
9	y (right)	102.60	229.96	0.45
10	y (right)	102.60	281.06	0.37
11	y (right)	102.60	349.96	0.29
12	y (right)	102.60	401.06	0.26
13	y (right)	102.60	229.96	0.45
14	y (right)	102.60	281.06	0.37
15	y (right)	102.60	349.96	0.29
16	y (right)	102.60	401.06	0.26
17	x (bottom)	85.50	210.16	0.41
18	x (bottom)	85.50	256.86	0.33
19	x (bottom)	85.50	285.16	0.30
20	x (bottom)	85.50	331.86	0.26
21	y (right)	171.00	229.96	**0.74**
22	y (right)	171.00	281.06	0.61
23	y (right)	171.00	349.96	0.49
24	y (right)	171.00	401.06	0.43
25	y (right)	171.00	229.96	0.74
26	y (right)	171.00	281.06	0.61
27	y (right)	171.00	349.96	0.49
28	y (right)	171.00	401.06	0.43

$\mu_{max} = 0.74 < 1.0$ ⇒ **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+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	153.51	0.00	-0.00	-0.00	22.00
2	207.24	0.00	-0.00	-0.00	29.70
3	228.51	0.00	-0.00	-0.00	67.00
4	282.24	0.00	-0.00	-0.00	74.70
5	153.51	0.00	13.50	40.50	22.00
6	207.24	0.00	13.50	40.50	29.70
7	228.51	0.00	13.50	40.50	67.00
8	282.24	0.00	13.50	40.50	74.70
9	153.51	27.00	-0.00	-0.00	-59.00
10	207.24	27.00	-0.00	-0.00	-51.30
11	228.51	27.00	-0.00	-0.00	-14.00
12	282.24	27.00	-0.00	-0.00	-6.30
13	153.51	27.00	13.50	40.50	-59.00
14	207.24	27.00	13.50	40.50	-51.30
15	228.51	27.00	13.50	40.50	-14.00
16	282.24	27.00	13.50	40.50	-6.30
17	153.51	0.00	22.50	67.50	22.00
18	207.24	0.00	22.50	67.50	29.70
19	228.51	0.00	22.50	67.50	67.00
20	282.24	0.00	22.50	67.50	74.70
21	153.51	45.00	-0.00	-0.00	-113.00
22	207.24	45.00	-0.00	-0.00	-105.30
23	228.51	45.00	-0.00	-0.00	-68.00
24	282.24	45.00	-0.00	-0.00	-60.30
25	153.51	45.00	22.50	67.50	-113.00
26	207.24	45.00	22.50	67.50	-105.30
27	228.51	45.00	22.50	67.50	-68.00
28	282.24	45.00	22.50	67.50	-60.30

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	153.51	0.00	-0.00	-0.00	22.00
2	153.51	0.00	-0.00	-0.00	22.00
3	203.51	0.00	-0.00	-0.00	52.00
4	203.51	0.00	-0.00	-0.00	52.00
5	153.51	0.00	15.00	45.00	22.00
6	153.51	0.00	15.00	45.00	22.00
7	203.51	0.00	15.00	45.00	52.00
8	203.51	0.00	15.00	45.00	52.00
9	153.51	30.00	-0.00	-0.00	-68.00
10	153.51	30.00	-0.00	-0.00	-68.00
11	203.51	30.00	-0.00	-0.00	-38.00
12	203.51	30.00	-0.00	-0.00	-38.00
13	153.51	30.00	15.00	45.00	-68.00
14	153.51	30.00	15.00	45.00	-68.00
15	203.51	30.00	15.00	45.00	-38.00
16	203.51	30.00	15.00	45.00	-38.00
17	153.51	0.00	15.00	45.00	22.00
18	153.51	0.00	15.00	45.00	22.00
19	203.51	0.00	15.00	45.00	52.00
20	203.51	0.00	15.00	45.00	52.00
21	153.51	30.00	-0.00	-0.00	-68.00
22	153.51	30.00	-0.00	-0.00	-68.00
23	203.51	30.00	-0.00	-0.00	-38.00
24	203.51	30.00	-0.00	-0.00	-38.00
25	153.51	30.00	15.00	45.00	-68.00
26	153.51	30.00	15.00	45.00	-68.00
27	203.51	30.00	15.00	45.00	-38.00
28	203.51	30.00	15.00	45.00	-38.00

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	233.51	-0.00	22.00	2.00	1.81	0.00	0.00
2	233.51	-0.00	22.00	2.00	1.81	0.00	0.00
3	283.51	-0.00	52.00	2.00	1.63	0.00	0.00
4	283.51	-0.00	52.00	2.00	1.63	0.00	0.00
5	233.51	53.00	22.00	1.81	1.55	0.00	0.00
6	233.51	53.00	22.00	1.81	1.55	0.00	0.00
7	283.51	53.00	52.00	1.63	1.63	0.00	0.00
8	283.51	53.00	52.00	1.63	1.63	0.00	0.00
9	233.51	-0.00	-84.00	2.00	1.28	0.00	0.00
10	233.51	-0.00	-84.00	2.00	1.28	0.00	0.00
11	283.51	-0.00	-54.00	2.00	1.62	0.00	0.00
12	283.51	-0.00	-54.00	2.00	1.62	0.00	0.00
13	233.51	53.00	-84.00	1.55	1.28	0.00	0.00
14	233.51	53.00	-84.00	1.55	1.28	0.00	0.00
15	283.51	53.00	-54.00	1.63	1.62	0.00	0.00
16	283.51	53.00	-54.00	1.63	1.62	0.00	0.00
17	233.51	53.00	22.00	1.81	1.55	0.00	0.00
18	233.51	53.00	22.00	1.81	1.55	0.00	0.00
19	283.51	53.00	52.00	1.63	1.63	0.00	0.00
20	283.51	53.00	52.00	1.63	1.63	0.00	0.00
21	233.51	-0.00	-84.00	2.00	1.28	0.00	0.00
22	233.51	-0.00	-84.00	2.00	1.28	0.00	0.00
23	283.51	-0.00	-54.00	2.00	1.62	0.00	0.00
24	283.51	-0.00	-54.00	2.00	1.62	0.00	0.00
25	233.51	53.00	-84.00	1.55	1.28	0.00	0.00
26	233.51	53.00	-84.00	1.55	1.28	0.00	0.00
27	283.51	53.00	-54.00	1.63	1.62	0.00	0.00
28	283.51	53.00	-54.00	1.63	1.62	0.00	0.00

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	---	3.14	15.33	32.50	---
2	19.00	32.50	---	3.14	15.33	32.50	---
3	19.00	32.50	---	2.83	15.81	32.50	---
4	19.00	32.50	---	2.83	15.81	32.50	---
5	19.00	32.50	---	2.68	16.08	32.50	---
6	19.00	32.50	---	2.68	16.08	32.50	---
7	19.00	32.50	---	2.82	15.83	32.50	---
8	19.00	32.50	---	2.82	15.83	32.50	---
9	19.00	32.50	---	2.22	17.13	32.50	---
10	19.00	32.50	---	2.22	17.13	32.50	---
11	19.00	32.50	---	2.80	15.85	32.50	---
12	19.00	32.50	---	2.80	15.85	32.50	---
13	19.00	32.50	---	2.22	17.13	32.50	---
14	19.00	32.50	---	2.22	17.13	32.50	---
15	19.00	32.50	---	2.80	15.85	32.50	---
16	19.00	32.50	---	2.80	15.85	32.50	---

LS	γ_1 kN/m³	φ_1 °	c_1 kN/m²	d_s m	γ_2 kN/m³	φ_2 °	c_2 kN/m²
17	19.00	32.50	---	2.68	16.08	32.50	---
18	19.00	32.50	---	2.68	16.08	32.50	---
19	19.00	32.50	---	2.82	15.83	32.50	---
20	19.00	32.50	---	2.82	15.83	32.50	---
21	19.00	32.50	---	2.22	17.13	32.50	---
22	19.00	32.50	---	2.22	17.13	32.50	---
23	19.00	32.50	---	2.80	15.85	32.50	---
24	19.00	32.50	---	2.80	15.85	32.50	---
25	19.00	32.50	---	2.22	17.13	32.50	---
26	19.00	32.50	---	2.22	17.13	32.50	---
27	19.00	32.50	---	2.80	15.85	32.50	---
28	19.00	32.50	---	2.80	15.85	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	---	0.728	1.487	---	1.000	1.000	---
2	15.03	24.58	---	0.728	1.487	---	1.000	1.000	---
3	15.03	24.58	---	0.755	1.439	---	1.000	1.000	---
4	15.03	24.58	---	0.755	1.439	---	1.000	1.000	---
5	15.03	24.58	---	0.744	1.459	---	1.000	1.000	---
6	15.03	24.58	---	0.744	1.459	---	1.000	1.000	---
7	15.03	24.58	---	0.701	1.535	---	1.000	1.000	---
8	15.03	24.58	---	0.701	1.535	---	1.000	1.000	---
9	15.03	24.58	---	0.808	1.344	---	1.000	1.000	---
10	15.03	24.58	---	0.808	1.344	---	1.000	1.000	---
11	15.03	24.58	---	0.757	1.435	---	1.000	1.000	---
12	15.03	24.58	---	0.757	1.435	---	1.000	1.000	---
13	15.03	24.58	---	0.752	1.445	---	1.000	1.000	---
14	15.03	24.58	---	0.752	1.445	---	1.000	1.000	---
15	15.03	24.58	---	0.701	1.535	---	1.000	1.000	---
16	15.03	24.58	---	0.701	1.535	---	1.000	1.000	---
17	15.03	24.58	---	0.744	1.459	---	1.000	1.000	---
18	15.03	24.58	---	0.744	1.459	---	1.000	1.000	---
19	15.03	24.58	---	0.701	1.535	---	1.000	1.000	---
20	15.03	24.58	---	0.701	1.535	---	1.000	1.000	---
21	15.03	24.58	---	0.808	1.344	---	1.000	1.000	---
22	15.03	24.58	---	0.808	1.344	---	1.000	1.000	---
23	15.03	24.58	---	0.757	1.435	---	1.000	1.000	---
24	15.03	24.58	---	0.757	1.435	---	1.000	1.000	---
25	15.03	24.58	---	0.752	1.445	---	1.000	1.000	---
26	15.03	24.58	---	0.752	1.445	---	1.000	1.000	---
27	15.03	24.58	---	0.701	1.535	---	1.000	1.000	---
28	15.03	24.58	---	0.701	1.535	---	1.000	1.000	---

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	3114.09	1.40	2224.35	233.51	0.10
2	3114.09	1.40	2224.35	315.24	0.14
3	2712.68	1.40	1937.63	308.51	0.16
4	2712.68	1.40	1937.63	390.24	0.20
5	2304.76	1.40	1646.26	233.51	0.14
6	2304.76	1.40	1646.26	315.24	0.19
7	2243.55	1.40	1602.53	308.51	0.19
8	2243.55	1.40	1602.53	390.24	0.24
9	1968.26	1.40	1405.90	233.51	0.17
10	1968.26	1.40	1405.90	315.24	0.22
11	2681.60	1.40	1915.43	308.51	0.16
12	2681.60	1.40	1915.43	390.24	0.20
13	1559.45	1.40	1113.89	233.51	0.21
14	1559.45	1.40	1113.89	315.24	**0.28**

LS	$R_{n,k}$ kN	$\gamma_{R,v}$ -	$R_{n,d}$ kN	N_d kN	μ -
15	2222.01	1.40	1587.15	308.51	0.19
16	2222.01	1.40	1587.15	390.24	0.25
17	2304.76	1.40	1646.26	233.51	0.14
18	2304.76	1.40	1646.26	315.24	0.19
19	2243.55	1.40	1602.53	308.51	0.19
20	2243.55	1.40	1602.53	390.24	0.24
21	1968.26	1.40	1405.90	233.51	0.17
22	1968.26	1.40	1405.90	315.24	0.22
23	2681.60	1.40	1915.43	308.51	0.16
24	2681.60	1.40	1915.43	390.24	0.20
25	1559.45	1.40	1113.89	233.51	0.21
26	1559.45	1.40	1113.89	315.24	0.28
27	2222.01	1.40	1587.15	308.51	0.19
28	2222.01	1.40	1587.15	390.24	0.25

$\mu_{max} = 0.28 < 1.0$ $\Rightarrow$ **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	233.51	---	1.10	1.40	---	---	0.00	---
2	233.51	---	1.10	1.40	---	---	0.00	---
3	283.51	---	1.10	1.40	---	---	0.00	---
4	283.51	---	1.10	1.40	---	---	0.00	---
5	233.51	148.76	1.10	1.40	135.24	9.64	13.50	0.09
6	233.51	148.76	1.10	1.40	135.24	9.64	13.50	0.09
7	283.51	180.62	1.10	1.40	164.20	9.64	13.50	0.08
8	283.51	180.62	1.10	1.40	164.20	9.64	13.50	0.08
9	233.51	148.76	1.10	1.40	135.24	19.29	27.00	0.17
10	233.51	148.76	1.10	1.40	135.24	19.29	27.00	0.17
11	283.51	180.62	1.10	1.40	164.20	19.29	27.00	0.15
12	283.51	180.62	1.10	1.40	164.20	19.29	27.00	0.15
13	233.51	148.76	1.10	1.40	135.24	21.56	30.19	0.19
14	233.51	148.76	1.10	1.40	135.24	21.56	30.19	0.19
15	283.51	180.62	1.10	1.40	164.20	21.56	30.19	0.16
16	283.51	180.62	1.10	1.40	164.20	21.56	30.19	0.16
17	233.51	148.76	1.10	1.40	135.24	16.07	22.50	0.15
18	233.51	148.76	1.10	1.40	135.24	16.07	22.50	0.15
19	283.51	180.62	1.10	1.40	164.20	16.07	22.50	0.12
20	283.51	180.62	1.10	1.40	164.20	16.07	22.50	0.12
21	233.51	148.76	1.10	1.40	135.24	32.14	45.00	0.27
22	233.51	148.76	1.10	1.40	135.24	32.14	45.00	0.27
23	283.51	180.62	1.10	1.40	164.20	32.14	45.00	0.23
24	283.51	180.62	1.10	1.40	164.20	32.14	45.00	0.23
25	233.51	148.76	1.10	1.40	135.24	35.94	50.31	0.29
26	233.51	148.76	1.10	1.40	135.24	35.94	50.31	**0.29**
27	283.51	180.62	1.10	1.40	164.20	35.94	50.31	0.25
28	283.51	180.62	1.10	1.40	164.20	35.94	50.31	0.25

$\mu_{max} = 0.29 < 1.0$ **⇒ 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	153.51	0.00	-0.00	-0.00	22.00

6.2. limitation of gaping joint under permanent load

int. f. and mom. in centroid of foundation base: $N_{0,k} = 233.51$ kN
$M_{0,x,k} = -0.00$ kNm
$M_{0,y,k} = 22.00$ kNm

resultant eccentricity: $e_x = 0.09$ m
$e_y = -0.00$ m

$e_x/b_x + e_y/b_y = 0.05 < 1/6$

⇒ 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	153.51	0.00	-0.00	-0.00	22.00
2	203.51	0.00	-0.00	-0.00	52.00
3	153.51	0.00	15.00	45.00	22.00
4	203.51	0.00	15.00	45.00	52.00

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	153.51	30.00	-0.00	-0.00	-68.00
6	203.51	30.00	-0.00	-0.00	-38.00
7	153.51	30.00	15.00	45.00	-68.00
8	203.51	30.00	15.00	45.00	-38.00

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	233.51	-0.00	22.00	0.09	-0.00	0.002
2	283.51	-0.00	52.00	0.18	-0.00	0.008
3	233.51	57.00	22.00	0.09	0.24	0.017
4	283.51	57.00	52.00	0.18	0.20	0.019
5	233.51	-0.00	-92.00	-0.39	-0.00	0.039
6	283.51	-0.00	-62.00	-0.22	-0.00	0.012
7	233.51	57.00	-92.00	-0.39	0.24	0.054
8	283.51	57.00	-62.00	-0.22	0.20	0.022

$((e_x/b_x)^2 + (e_y/b_y)^2)_{max} = 0.054 < 1/9$

⇒ **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	233.51	---	1.10	---	0.00	---
2	233.51	---	1.10	---	0.00	---
3	283.51	---	1.10	---	0.00	---
4	283.51	---	1.10	---	0.00	---
5	233.51	148.76	1.10	135.24	13.50	0.10
6	233.51	148.76	1.10	135.24	13.50	0.10
7	283.51	180.62	1.10	164.20	13.50	0.08
8	283.51	180.62	1.10	164.20	13.50	0.08
9	233.51	148.76	1.10	135.24	27.00	0.20
10	233.51	148.76	1.10	135.24	27.00	0.20
11	283.51	180.62	1.10	164.20	27.00	0.16
12	283.51	180.62	1.10	164.20	27.00	0.16
13	233.51	148.76	1.10	135.24	30.19	0.22
14	233.51	148.76	1.10	135.24	30.19	0.22
15	283.51	180.62	1.10	164.20	30.19	0.18
16	283.51	180.62	1.10	164.20	30.19	0.18
17	233.51	148.76	1.10	135.24	22.50	0.17
18	233.51	148.76	1.10	135.24	22.50	0.17
19	283.51	180.62	1.10	164.20	22.50	0.14
20	283.51	180.62	1.10	164.20	22.50	0.14
21	233.51	148.76	1.10	135.24	45.00	0.33
22	233.51	148.76	1.10	135.24	45.00	0.33
23	283.51	180.62	1.10	164.20	45.00	0.27
24	283.51	180.62	1.10	164.20	45.00	0.27
25	233.51	148.76	1.10	135.24	50.31	0.37
26	233.51	148.76	1.10	135.24	50.31	**0.37**
27	283.51	180.62	1.10	164.20	50.31	0.31
28	283.51	180.62	1.10	164.20	50.31	0.31

$\mu_{max} = 0.37 < 1.0$ ⇒ **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	153.51	0.00	-0.00	-0.00	22.00
2	203.51	0.00	-0.00	-0.00	52.00
3	153.51	0.00	15.00	45.00	22.00
4	203.51	0.00	15.00	45.00	52.00
5	153.51	30.00	-0.00	-0.00	-68.00
6	203.51	30.00	-0.00	-0.00	-38.00
7	153.51	30.00	15.00	45.00	-68.00
8	203.51	30.00	15.00	45.00	-38.00

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 $\sigma_B(z) = 0.2 \cdot \sigma_{\ddot{u}}(z)$ 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	233.51	-0.00	22.00	58.38	43.18	1.99
2	283.51	-0.00	52.00	70.88	55.68	2.35
3	233.51	57.00	22.00	58.38	43.18	1.99
4	283.51	57.00	52.00	70.88	55.68	2.35
5	233.51	-0.00	-92.00	58.38	43.18	1.99
6	283.51	-0.00	-62.00	70.88	55.68	2.35
7	233.51	57.00	-92.00	58.38	43.18	1.99
8	283.51	57.00	-62.00	70.88	55.68	2.35

6.7.2. determination of settlement values and settlement parts per soil stratum
coefficient f for settlement below significant point acc. to [7], vol. 2, tab. 4
coefficients f_x/f_y for obliquity of a rigid foundation acc. to [8], fig. 19
settlement parts from central load $s_{m,i} = \sigma_0' \cdot b_y \cdot (f_i - f_{i-1})/E_{m,i}$
settlement parts from $M_{0,y}$ $s_{x,i} = b_x/2 \cdot M_{0,y}/(E_{m,i} \cdot b_y \cdot b_x^2) \cdot (f_{x,i} - f_{x,i-1})$
settlement parts from $M_{0,x}$ $s_{y,i} = b_y/2 \cdot M_{0,x}/(E_{m,i} \cdot b_x \cdot b_y^2) \cdot (f_{y,i} - f_{y,i-1})$

LS 1:
σ_0' = 43.18 kN/m²
$M_{0,x}$ = -0.00 kNm
$M_{0,y}$ = 22.00 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	0.39	0.09	-0.00
2.79	1.99	0.487	3.613	3.613	0.03	0.01	-0.00

LS 2:
σ_0' = 55.68 kN/m²
$M_{0,x}$ = -0.00 kNm
$M_{0,y}$ = 52.00 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	0.50	0.22	-0.00
3.15	2.35	0.526	3.667	3.667	0.08	0.02	-0.00

LS 3:
σ_0' = 43.18 kN/m²
$M_{0,x}$ = 57.00 kNm
$M_{0,y}$ = 22.00 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	0.39	0.09	0.24
2.79	1.99	0.487	3.613	3.613	0.03	0.01	0.01

LS 4:
σ_0' = 55.68 kN/m²
$M_{0,x}$ = 57.00 kNm
$M_{0,y}$ = 52.00 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	0.50	0.22	0.24
3.15	2.35	0.526	3.667	3.667	0.08	0.02	0.02

LS 5:
σ_0' = 43.18 kN/m²
$M_{0,x}$ = -0.00 kNm
$M_{0,y}$ = -92.00 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	0.39	-0.39	-0.00
2.79	1.99	0.487	3.613	3.613	0.03	-0.02	-0.00

LS 6:
σ_0' = 55.68 kN/m²
$M_{0,x}$ = -0.00 kNm
$M_{0,y}$ = -62.00 kNm

level	z	f	f_x	f_y	s_m	s_x	s_y
m	m	-	-	-	cm	cm	cm
2.50	1.70	0.450	3.422	3.422	0.50	-0.27	-0.00
3.15	2.35	0.526	3.667	3.667	0.08	-0.02	-0.00

LS 7:
σ_0' = 43.18 kN/m²
$M_{0,x}$ = 57.00 kNm
$M_{0,y}$ = -92.00 kNm

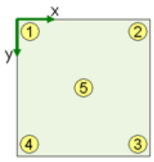
level	z	f	f_x	f_y	s_m	s_x	s_y
m	m	-	-	-	cm	cm	cm
2.50	1.70	0.450	3.422	3.422	0.39	-0.39	0.24
2.79	1.99	0.487	3.613	3.613	0.03	-0.02	0.01

LS 8:
σ_0' = 55.68 kN/m²
$M_{0,x}$ = 57.00 kNm
$M_{0,y}$ = -62.00 kNm

level	z	f	f_x	f_y	s_m	s_x	s_y
m	m	-	-	-	cm	cm	cm
2.50	1.70	0.450	3.422	3.422	0.50	-0.27	0.24
3.15	2.35	0.526	3.667	3.667	0.08	-0.02	0.02

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	s2	s3	s4	s5	Smax	α_x	α_y
	cm	cm	cm	cm	cm	cm	°	°
1	0.5	0.3	0.3	0.5	0.4	0.5	-0.0	0.1
2	0.8	0.3	0.3	0.8	0.6	0.8	-0.0	0.1
3	0.3	0.1	0.6	0.8	0.4	0.8	0.1	0.1
4	0.6	0.1	0.6	1.1	0.6	1.1	**0.1**	0.1
5	0.0	0.8	0.8	0.0	0.4	0.8	-0.0	**-0.2**
6	0.3	0.9	0.9	0.3	0.6	0.9	-0.0	-0.2
7	-0.3	0.6	1.1	0.3	0.4	1.1	0.1	-0.2
8	0.0	0.6	1.1	0.6	0.6	**1.1**	0.1	-0.2

max s_{max} = 1.1 < 5.0 cm   max $|\alpha_x|$ = 0.1° < 0.5°   max $|\alpha_y|$ = 0.2° < 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 κ = 1
constrained modulus E_s = 1·10000.00 = 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.

longit. reinf. x-direction (top)	exist $A_{so,x}$	=	3.1 cm²
longit. reinf. x-direct. (bottom)	exist $A_{su,x}$	=	2.4 cm²
longit. reinf. y-direction (top)	exist $A_{so,y}$	=	3.1 cm²
longit. reinf. y-direct. (bottom)	exist $A_{su,y}$	=	2.4 cm²
overturning	μ_{max}	=	0.74
base failure	μ_{max}	=	0.28
slip	μ_{max}	=	0.29
gaping joint under permant load	μ_{exist}	=	0.28
gaping joint under total load	μ_{max}	=	0.48
displacement in the base area	μ_{max}	=	0.37
settlement	s_{max}	=	1.1 < 5.0 cm
obliquity (about x-axis)	$\alpha_{max,x}$	=	0.1 < 0.5 °
obliquity (about y-axis)	$\alpha_{max,y}$	=	0.2 < 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] DAfStb Heft 240: Hilfsmittel zur Berech. der Schnittgr. und Formänderungen von Stahlbetontragwerken, Beuth, 3.Aufl., 1991
[3] DIN EN 1992-1-1: Eurocode 2: Bemessung und Konstruktion von Stahlbeton- und Spannbetontragwerken, Teil 1-1, Januar 2011
[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