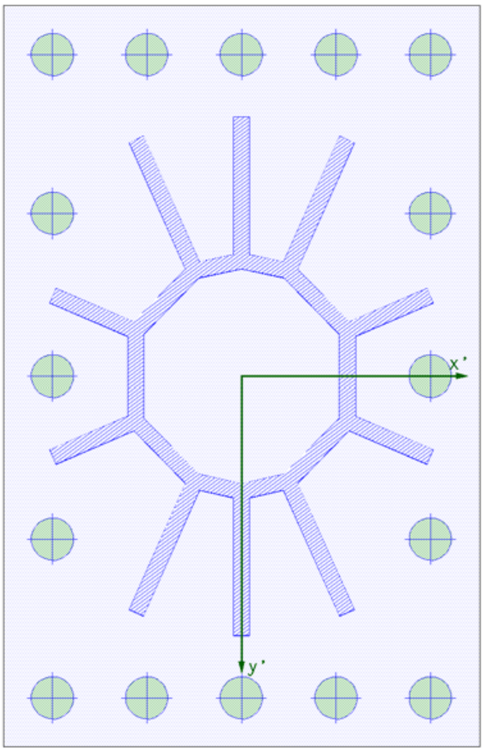
4H-FUND version: 12/2022-1a

steel column base with base plate on isolated foundation

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)

top view base plate
scale 1:10



column cross section

user defined profile: Kopie from cross-section 2, of quality S235

base plate

b_x = 630 mm b_y = 980 mm t = 25 mm, of quality S235

mortar joint

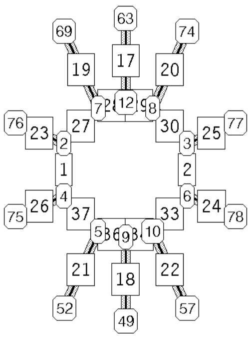
t_F = 20 mm

anchors

16 anchors, FK 4.6, M30, without shaft
with a length of 450 mm
positions on the base plate:

Nr	x'	y'	Nr	x'	y'
	mm	mm		mm	mm
1	-250	-425	9	250	0
2	-125	-425	10	-250	215
3	0	-425	11	250	215
4	125	-425	12	-250	425
5	250	-425	13	-125	425
6	-250	-215	14	0	425
7	250	-215	15	125	425
8	-250	0	16	250	425

description of column profile cross-section (Kopie from cross-section 2)



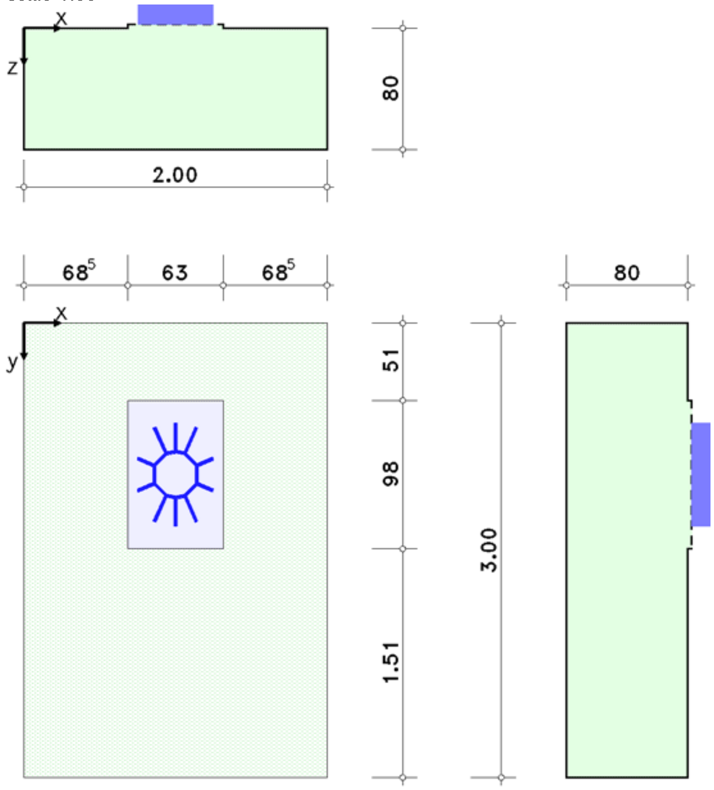
node coordinates

Nr.	x'	y'
-	mm	mm
49	0.0	341.8
52	-139.1	312.4
57	139.1	312.4
63	-0.0	-341.8
69	-139.1	-312.3
74	139.1	-312.4
75	-249.0	105.8
76	-249.0	-105.7
77	249.0	-105.8
78	249.0	105.7
3	139.9	-59.4
4	-139.9	59.4
2	-139.9	-59.4
6	139.9	59.4
5	-61.8	138.8
8	61.8	-138.8
7	-61.8	-138.8
10	61.8	138.8
9	0.0	151.9
12	-0.0	-151.9

line elements

Nr.	nodA	nodE	thickness
-	-	-	mm
17	63	12	20.0
18	9	49	20.0
19	7	69	20.0
20	8	74	20.0
21	5	52	20.0
22	10	57	20.0
23	2	76	20.0
24	6	78	20.0
25	3	77	20.0
26	4	75	20.0
27	2	7	20.0
28	7	12	20.0
29	12	8	20.0
30	8	3	20.0
33	6	10	20.0
34	10	9	20.0
36	9	5	20.0
37	5	4	20.0
1	2	4	20.0
2	3	6	20.0

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 = 2.40 m.
the ground water level (below top edge soil) is at t_W = 2.50 m.

2. loading

2.1. Structure of action effects

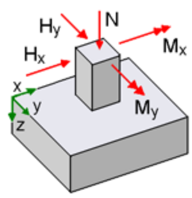
On the left-hand side, the action effects and load cases are shown in a tree structure. The right-hand side shows their characteristics of the superposition.

used symbols: action load case

1: permanent loads	**permanent loads**
1: dead load (1)	additive
2: live loads (2)	**variable live loads in assembly and salesrooms**
2: live loads (2/1)	additive
3: wind loads	**transient wind loads**
3: wind load (1)	alternative in group A
4: wind load (2)	alternative in group A

2.2. Characteristic column load

point of application in column centroid auf top edge foundation slab



loadc.	N_{St} kN	$H_{x,St}$ kN	$H_{y,St}$ kN	$M_{x,St}$ kNm	$M_{y,St}$ kNm
1	17.08	-1.26	-3.15	-26.96	10.49
2	1.18	-5.50	-13.20	-112.82	45.65
3	0.81	-1.19	-11.04	-86.77	9.14
4	0.81	-5.41	-0.98	-7.01	37.89

2.3. dead load

Das Gewicht der foundation slab wird with γ_E = 24.00 kN/m³ berücksichtigt.
the height of the earth load is h_A = 1.60 m.
the mean unit weight of the earth load is γ_A = 19.00 kN/m³.

resultant internal forces and moments in der floor joint: N_0 / $M_{0,x}$ / $M_{0,y}$ = 283.63 kN / 9.38 / 0.00 kNm.

Das dead load wird im load case 1 with berücksichtigt.

3. Verification of steel column base

3.1. partial safety factors for material

design situat.	γ_{M0}	γ_{M2}	γ_c	γ_μ
permanent	1.00	1.25	1.50	1.20

3.2. design values of steel verifications

3.2.1. factorization of load case combinations

LK	design situat.	factorization
1	permanent and transient	Lf1
2	permanent and transient	1.35· Lf1
3	permanent and transient	Lf1+1.5· Lf2
4	permanent and transient	1.35· Lf1+1.5· Lf2
5	permanent and transient	Lf1+0.6· 1.5· Lf3
6	permanent and transient	1.35· Lf1+0.6· 1.5· Lf3
7	permanent and transient	Lf1+1.5· Lf2+0.6· 1.5· Lf3
8	permanent and transient	1.35· Lf1+1.5· Lf2+0.6· 1.5· Lf3
9	permanent and transient	Lf1+0.6· 1.5· Lf4
10	permanent and transient	1.35· Lf1+0.6· 1.5· Lf4
11	permanent and transient	Lf1+1.5· Lf2+0.6· 1.5· Lf4
12	permanent and transient	1.35· Lf1+1.5· Lf2+0.6· 1.5· Lf4
13	permanent and transient	Lf1+0.7· 1.5· Lf2
14	permanent and transient	1.35· Lf1+0.7· 1.5· Lf2
15	permanent and transient	Lf1+1.5· Lf3
16	permanent and transient	1.35· Lf1+1.5· Lf3
17	permanent and transient	Lf1+0.7· 1.5· Lf2+1.5· Lf3
18	permanent and transient	1.35· Lf1+0.7· 1.5· Lf2+1.5· Lf3
19	permanent and transient	Lf1+1.5· Lf4
20	permanent and transient	1.35· Lf1+1.5· Lf4
21	permanent and transient	Lf1+0.7· 1.5· Lf2+1.5· Lf4
22	permanent and transient	1.35· Lf1+0.7· 1.5· Lf2+1.5· Lf4

3.2.2. column load

increasing factor for flex. mom.: $\Delta M_{St,TH.II.O} = M_{St} \cdot$ **20%**
(for the consideration of increase of moments from non-linear effects)

LK	$N_{St,d}$ kN	$H_{x,St,d}$ kN	$H_{y,St,d}$ kN	$M_{x,St,d}$ kNm	$M_{y,St,d}$ kNm
1	17.08	-1.26	-3.15	-32.35	12.59
2	23.06	-1.70	-4.25	-43.68	16.99
3	18.85	-9.51	-22.95	-235.43	94.76
4	24.83	-9.95	-24.05	-246.75	99.16
5	17.81	-2.33	-13.09	-126.06	22.46
6	23.79	-2.77	-14.19	-137.39	26.86
7	19.58	-10.58	-32.89	-329.14	104.63
8	25.56	-11.02	-33.99	-340.46	109.04
9	17.81	-6.13	-4.03	-39.92	53.51
10	23.79	-6.57	-5.13	-51.25	57.92
11	19.58	-14.38	-23.83	-243.00	135.68
12	25.56	-14.82	-24.93	-254.32	140.09
13	18.32	-7.03	-17.01	-174.51	70.11
14	24.30	-7.48	-18.11	-185.83	74.51
15	18.30	-3.05	-19.71	-188.54	29.04
16	24.27	-3.49	-20.81	-199.86	33.45
17	19.53	-8.82	-33.57	-330.69	86.56
18	25.51	-9.26	-34.67	-342.01	90.96
19	18.30	-9.38	-4.62	-44.97	80.79
20	24.27	-9.82	-5.72	-56.29	85.20
21	19.53	-15.15	-18.48	-187.12	138.31
22	25.51	-15.59	-19.58	-198.45	142.71

3.3. weld between column shaft and base plate

design with simplified method acc. to clause 4.5.3.3

$F_{w,Ed} = \sigma_{w,v} \cdot a_w$

$F_{w,Rd} = f_{vw,d} \cdot a_w$

$f_{vw,d} = (f_u/3^{0.5})/(\beta_w \cdot \gamma_{m2})$

$U = F_{w,Ed}/F_{w,Rd}$

connection designed with a **circumferential fillet weld.**
axial force transfer of 50 % by the weld.

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

LK	a_w mm	$\sigma_{w,max}$ kN/cm²	$\tau_{w,max}$ kN/cm²	$\sigma_{w,v,max}$ kN/cm²	$F_{w,Ed}$ kN/cm	$F_{w,Rd}$ kN/cm	U -
1	**5**	-1.72	0.00	1.72	0.86	10.39	0.08
2	5	-2.32	0.00	2.32	1.16	10.39	0.11
3	5	-12.86	0.01	12.86	6.43	10.39	0.62
4	5	-13.46	0.01	13.46	6.73	10.39	0.65
5	5	-5.79	0.01	5.79	2.89	10.39	0.28
6	5	-6.39	0.01	6.39	3.19	10.39	0.31
7	5	-16.92	0.02	16.92	8.46	10.39	0.81
8	5	-17.52	0.02	17.52	8.76	10.39	**0.84**
9	5	4.79	0.02	4.79	2.40	10.39	0.23
10	5	5.34	0.00	5.34	2.67	10.39	0.26
11	5	-14.72	0.01	14.72	7.36	10.39	0.71
12	5	-15.32	0.01	15.32	7.66	10.39	0.74
13	5	-9.52	0.01	9.52	4.76	10.39	0.46
14	5	-10.12	0.01	10.12	5.06	10.39	0.49
15	5	-8.50	0.01	8.50	4.25	10.39	0.41
16	5	-9.10	0.01	9.10	4.55	10.39	0.44
17	5	-16.29	0.02	16.29	8.15	10.39	0.78
18	5	-16.89	0.02	16.89	8.45	10.39	0.81
19	5	6.94	0.06	6.94	3.47	10.39	0.33
20	5	7.49	0.05	7.49	3.75	10.39	0.36
21	5	13.71	-0.14	13.71	6.86	10.39	0.66
22	5	14.26	-0.15	14.26	7.13	10.39	0.69

maximum weld thickness $a_{w,max}$ = 5 mm
maximum utilization U = 0.84 < 1.00

a_w - weld thickness $\sigma_{w,max}$ - max. normal stress along the weld $\tau_{w,max}$ - max. shear stress along the weld
$\sigma_{w,v,max}$ - max. equivalent stress along the weld $F_{w,Ed}$ - effective force in the weld per unit of length
$F_{w,Rd}$ - design resistance of the weld per unit of length U - utilization

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 21 elements in X-direction and 32 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 = 2618.00 kN/cm.

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

internal forces and moments

LK	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	46.5	13.8	-2.63	-3.51	0.07	-0.09	-0.22
2	46.5	13.8	-3.55	-4.74	0.10	-0.12	-0.30
3	46.5	13.8	-20.66	-27.46	0.53	-0.71	-1.75
4	46.5	13.8	-21.59	-28.69	0.56	-0.74	-1.82
5	46.5	13.8	-9.35	-13.01	0.24	-0.35	-0.80
6	46.5	13.8	-10.28	-14.25	0.27	-0.38	-0.88
7	46.5	13.8	-27.43	-37.05	0.72	-0.97	-2.32
8	46.5	13.8	-28.35	-38.27	0.75	-1.01	-2.40
9	61.5	38.3	-1.70	-7.62	0.39	0.67	0.24
10	61.5	38.3	-1.90	-8.52	0.44	0.74	0.25
11	46.5	13.8	-23.43	-30.33	0.57	-0.76	-1.97
12	46.5	13.8	-24.35	-31.56	0.60	-0.80	-2.05
13	46.5	13.8	-15.25	-20.27	0.39	-0.52	-1.29
14	46.5	13.8	-16.18	-21.50	0.42	-0.56	-1.37
15	46.5	13.8	-13.82	-19.32	0.34	-0.52	-1.18
16	46.5	13.8	-14.75	-20.56	0.37	-0.55	-1.26
17	46.5	13.8	-26.52	-36.23	0.70	-0.96	-2.25
18	46.5	13.8	-27.44	-37.46	0.73	-0.99	-2.33
19	61.5	38.3	-2.44	-10.97	0.56	0.97	0.37
20	61.5	38.3	-2.64	-11.88	0.61	1.05	0.38
21	46.5	13.8	-19.88	-25.08	0.46	-0.62	-1.67
22	46.5	13.8	-20.80	-26.31	0.48	-0.65	-1.74

stresses and utilizations

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

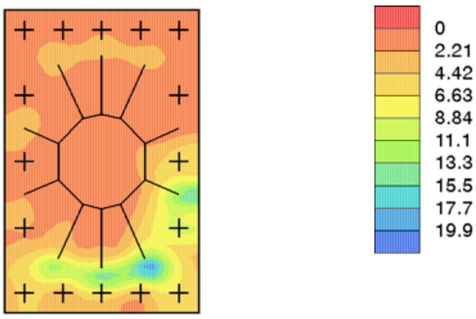
LK	x_{Fp} cm	y_{Fp} cm	$\sigma_{Pl,V}$ kN/cm²	σ_{Rd} kN/cm²	U -
1	46.5	13.8	2.03	23.50	0.09
2	46.5	13.8	2.74	23.50	0.12
3	46.5	13.8	15.92	23.50	0.68
4	46.5	13.8	16.63	23.50	0.71
5	46.5	13.8	7.47	23.50	0.32
6	46.5	13.8	8.18	23.50	0.35
7	46.5	13.8	21.40	23.50	0.91
8	46.5	13.8	22.11	23.50	**0.94**
9	61.5	38.3	4.48	23.50	0.19
10	61.5	38.3	5.01	23.50	0.21
11	46.5	13.8	17.70	23.50	0.75
12	46.5	13.8	18.41	23.50	0.78
13	46.5	13.8	11.75	23.50	0.50
14	46.5	13.8	12.46	23.50	0.53
15	46.5	13.8	11.08	23.50	0.47
16	46.5	13.8	11.79	23.50	0.50
17	46.5	13.8	20.87	23.50	0.89
18	46.5	13.8	21.58	23.50	0.92
19	61.5	38.3	6.46	23.50	0.27
20	61.5	38.3	6.99	23.50	0.30
21	46.5	13.8	14.73	23.50	0.63
22	46.5	13.8	15.44	23.50	0.66

maximum utilization U = 0.94 < 1.00

x_{Fp}/y_{Fp} - coordinates on the base plate m_{xx}/m_{yy} - flex. mom. 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]

LK 8 (max $\sigma_{Pl,V}$)



3.4.2. concrete compression under base plate

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

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

$U_{A,compression}$ = exis ($A_{\sigma c>fjd}/A_{compression}$) / perm ($A_{\sigma c>fjd}/A_{compression}$)

design value der concrete- resp. Mörtelfestigkeit unter bearing stress: $f_{jd} = 1.0 \cdot f_{cd}$

verification nur at surface of pressuren größer als 5% der slab nfläeche ($A_{compression}$ > 308.7 cm²)

control of heavily loaded compression areas:
the allowable relation between the area with concrete compressions greater than the design value ($A_{\sigma c>fjd}$) and the total compression area ($A_{compression}$) comes to: perm ($A_{\sigma c>fjd}/A_{compression}$) = 30%

LK	lim $\sigma_{c,d}$	$A_{compression}$	$F_{compression}$	$A_{\sigma c>fjd}$	$\sigma_{c,max}$	$\sigma_{c,m}$	f_{jd}	U_{fjd}	$A_{\sigma c>fjd}$ / $A_{compression}$
	kN/cm²	cm²	kN	cm²	kN/cm²	kN/cm²	kN/cm²	-	%
1	4.25	1736.4	70.18	-	0.39	0.040	1.42	0.03	0.00
2	4.25	1745.6	94.74	-	0.53	0.054	1.42	0.04	0.00
3	4.25	1662.9	463.08	73.5	2.71	0.278	1.42	0.20	4.42
4	4.25	1662.9	487.63	101.1	2.81	0.293	1.42	0.21	6.08
5	4.25	1727.3	225.62	-	1.16	0.131	1.42	0.09	0.00
6	4.25	1736.4	248.55	-	1.27	0.143	1.42	0.10	0.00
7	4.25	1681.3	605.66	156.2	3.27	0.360	1.42	0.25	9.29
8	4.25	1672.1	629.85	165.4	3.42	0.377	1.42	0.27	**9.89**
9	4.25	1782.4	145.21	-	1.10	0.081	1.42	0.06	0.00
10	4.25	1690.5	167.34	-	1.26	0.099	1.42	0.07	0.00
11	4.25	1681.3	531.60	137.8	3.11	0.316	1.42	0.22	8.20
12	4.25	1681.3	556.09	156.2	3.16	0.331	1.42	0.23	9.29
13	4.25	1653.8	345.19	45.9	2.01	0.209	1.42	0.15	2.78
14	4.25	1662.9	369.74	45.9	2.15	0.222	1.42	0.16	2.76
15	4.25	1745.6	332.69	45.9	1.69	0.191	1.42	0.13	2.63
16	4.25	1727.3	355.33	55.1	1.82	0.206	1.42	0.15	3.19
17	4.25	1653.8	588.66	147.0	3.15	0.356	1.42	0.25	8.89
18	4.25	1617.0	611.33	147.0	3.31	0.378	1.42	**0.27**	9.09
19	4.25	1699.7	202.25	9.2	1.58	0.119	1.42	0.08	0.54
20	4.25	1736.4	222.27	9.2	1.71	0.128	1.42	0.09	0.53
21	4.25	1690.5	462.51	73.5	3.02	0.274	1.42	0.19	4.35
22	4.25	1681.3	486.88	82.7	3.06	0.290	1.42	0.20	4.92

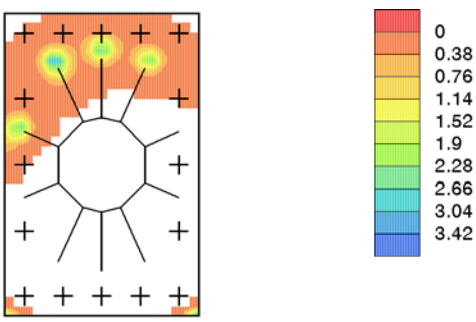
maximum utilization U_{fjd} = 0.27 < 1.00

maximum share of the compression area with $\sigma_c > f_{jd}$: $A_{\sigma c>fjd}/A_{compression}$ = 9.89 < 30.00 %
associated utilization $U_{A,compression}$ = 0.33 < 1.00

$A_{compression}$ - area with concrete compressions $F_{compression}$ - Res. compressionkraft auf den concrete $\sigma_{c,max}$ - maximale concrete compression
$\sigma_{c,m}$ - mean concrete compression U_{fjd} - utilization withtl. bearing stress
$U_{A,compression}$ - utilization of the allowable compr. area with $\sigma_c > f_{cd}$ $\sigma_c > f_{jd}$

pressure distribution [kN/cm²]

LK 18 (max $\sigma_{c,m}$)

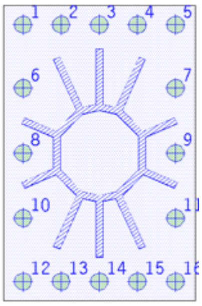


3.4.3. anchor tensile forces

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

stress area of M30: A_s = 5.61 cm^2
No countersunk bolts used: k2 = 0.90

numeration



LK	$F_{t,Ed,1}$ kN	$F_{t,Ed,2}$ kN	$F_{t,Ed,3}$ kN	$F_{t,Ed,4}$ kN	$F_{t,Ed,5}$ kN	$F_{t,Ed,6}$ kN	$F_{t,Rd}$ kN	U_{max} -
1	---	---	---	---	---	---	161.57	0.00
2	---	---	---	---	---	---	161.57	0.00
3	---	---	---	---	---	---	161.57	0.00
4	---	---	---	---	---	---	161.57	0.00
5	---	---	---	---	---	---	161.57	0.00
6	---	---	---	---	---	---	161.57	0.00
7	---	---	---	---	---	---	161.57	0.00
8	---	---	---	---	---	---	161.57	0.00
9	---	---	---	---	---	---	161.57	0.00
10	---	---	---	---	---	---	161.57	0.00
11	---	---	---	---	---	---	161.57	0.00
12	---	---	---	---	---	---	161.57	0.00
13	---	---	---	---	---	---	161.57	0.00
14	---	---	---	---	---	---	161.57	0.00
15	---	---	---	---	---	---	161.57	0.00
16	---	---	---	---	---	---	161.57	0.00
17	---	---	---	---	---	---	161.57	0.00
18	---	---	---	---	---	---	161.57	0.00
19	---	---	---	0.72	0.60	---	161.57	**0.00**
20	---	---	---	---	---	---	161.57	0.00
21	---	---	---	---	---	---	161.57	0.00
22	---	---	---	---	---	---	161.57	0.00

LK	$F_{t,Ed,7}$ kN	$F_{t,Ed,8}$ kN	$F_{t,Ed,9}$ kN	$F_{t,Ed,10}$ kN	$F_{t,Ed,11}$ kN	$F_{t,Ed,12}$ kN	$F_{t,Rd}$ kN	U_{max} -
1	0.47	---	8.78	2.05	11.96	1.96	161.57	0.07
2	0.63	---	11.85	2.77	16.14	2.65	161.57	0.10
3	10.59	---	80.32	18.37	97.75	16.13	161.57	0.60
4	10.75	---	83.39	19.08	101.93	16.82	161.57	0.63
5	---	3.53	21.68	17.47	38.38	11.11	161.57	0.24
6	---	2.81	24.44	17.61	42.59	11.81	161.57	0.26
7	5.56	---	91.27	30.09	124.39	25.29	161.57	0.77
8	5.63	---	94.21	30.83	128.51	25.96	161.57	**0.80**
9	15.41	---	41.69	---	32.86	---	161.57	0.26
10	15.03	---	44.58	---	36.97	---	161.57	0.28
11	24.42	---	112.71	13.88	118.40	13.20	161.57	0.73
12	24.56	---	115.76	14.58	122.57	13.89	161.57	0.76
13	7.55	---	58.85	13.47	71.99	11.88	161.57	0.45
14	7.71	---	61.92	14.18	76.18	12.57	161.57	0.47
15	---	7.39	30.90	28.93	55.99	17.21	161.57	0.35
16	---	6.55	33.64	29.01	60.16	17.90	161.57	0.37
17	---	2.46	77.07	35.79	116.40	27.20	161.57	0.72
18	---	1.72	79.96	35.81	120.50	27.87	161.57	0.75
19	27.14	---	63.82	---	46.84	---	161.57	0.39
20	26.39	---	66.78	---	51.00	---	161.57	0.41
21	31.03	---	113.29	7.13	106.65	7.22	161.57	0.70
22	31.13	---	116.32	7.80	110.82	7.90	161.57	0.72

LK	$F_{t,Ed,13}$ kN	$F_{t,Ed,14}$ kN	$F_{t,Ed,15}$ kN	$F_{t,Ed,16}$ kN	$F_{t,Rd}$ kN	U_{max} -
1	5.44	9.61	8.67	4.16	161.57	0.06
2	7.34	12.98	11.70	5.62	161.57	0.08
3	44.03	76.20	68.20	32.64	161.57	0.47
4	45.94	79.57	71.23	34.10	161.57	0.49
5	27.12	40.60	32.82	15.11	161.57	0.25
6	29.05	43.99	35.87	16.58	161.57	0.27
7	65.92	107.38	92.49	43.68	161.57	0.66
8	67.78	110.72	95.51	45.14	161.57	0.69

LK	$F_{t,Ed,13}$ kN	$F_{t,Ed,14}$ kN	$F_{t,Ed,15}$ kN	$F_{t,Ed,16}$ kN	$F_{t,Rd}$ kN	U_{max} -
9	2.29	11.85	15.27	8.04	161.57	0.09
10	4.06	15.14	18.27	9.49	161.57	0.11
11	40.08	78.12	74.71	36.51	161.57	0.48
12	41.99	81.48	77.74	37.96	161.57	0.50
13	32.45	56.22	50.34	24.11	161.57	0.35
14	34.36	59.59	53.38	25.56	161.57	0.37
15	41.52	61.21	48.88	22.37	161.57	0.38
16	43.44	64.59	51.93	23.84	161.57	0.40
17	68.84	108.11	90.78	42.49	161.57	0.67
18	70.74	111.46	93.80	43.96	161.57	**0.69**
19	0.97	13.58	19.67	10.60	161.57	0.12
20	2.29	16.76	22.70	12.07	161.57	0.14
21	26.22	59.59	61.27	30.57	161.57	0.38
22	28.12	62.95	64.30	32.03	161.57	0.40

maximum utilization U = 0.80 < 1.00

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

3.5. slippage verification of base plate

$H_{res,d} = (H_{x,St,d}^2+H_{y,St,d}^2)^{0.5}$
$N_{Z,d} = A_{compression} \cdot \sigma_{c,m}$
$H_{Rd} = \mu_k/\gamma_\mu \cdot N_{Z,d}$ with $\mu_k = 0.6$ acc. to [1]

LK	$H_{res,d}$ kN	$N_{z,d}$ kN	H_{Rd} kN	U -
1	3.39	70.18	35.09	0.10
2	4.58	94.74	47.37	0.10
3	24.84	463.08	231.54	0.11
4	26.03	487.63	243.81	0.11
5	13.29	225.62	112.81	0.12
6	14.46	248.55	124.27	0.12
7	34.55	605.66	302.83	0.11
8	35.73	629.85	314.92	0.11
9	7.34	145.21	72.61	0.10
10	8.34	167.34	83.67	0.10
11	27.83	531.60	265.80	0.10
12	29.01	556.09	278.04	0.10
13	18.41	345.19	172.59	0.11
14	19.59	369.74	184.87	0.11
15	19.94	332.69	166.35	**0.12**
16	21.10	355.33	177.66	0.12
17	34.71	588.66	294.33	0.12
18	35.89	611.33	305.66	0.12
19	10.45	202.25	101.13	0.10
20	11.36	222.27	111.13	0.10
21	23.90	462.51	231.25	0.10
22	25.03	486.88	243.44	0.10

maximum utilization U = 0.12 < 1.00

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

4. design calculation of foundation slab

4.1. partial safety factors for material

design situat.	γ_c	γ_s
permanent and transient	1.50	1.15

4.2. design values of reinforced concrete design

4.2.1. factorization of load case combinations

LK	design situat.	factorization
1	permanent and transient	Lf1
2	permanent and transient	1.35· Lf1
3	permanent and transient	Lf1+1.5· Lf2
4	permanent and transient	1.35· Lf1+1.5· Lf2
5	permanent and transient	Lf1+0.6· 1.5· Lf3
6	permanent and transient	1.35· Lf1+0.6· 1.5· Lf3
7	permanent and transient	Lf1+1.5· Lf2+0.6· 1.5· Lf3
8	permanent and transient	1.35· Lf1+1.5· Lf2+0.6· 1.5· Lf3
9	permanent and transient	Lf1+0.6· 1.5· Lf4
10	permanent and transient	1.35· Lf1+0.6· 1.5· Lf4
11	permanent and transient	Lf1+1.5· Lf2+0.6· 1.5· Lf4
12	permanent and transient	1.35· Lf1+1.5· Lf2+0.6· 1.5· Lf4
13	permanent and transient	Lf1+0.7· 1.5· Lf2
14	permanent and transient	1.35· Lf1+0.7· 1.5· Lf2
15	permanent and transient	Lf1+1.5· Lf3
16	permanent and transient	1.35· Lf1+1.5· Lf3
17	permanent and transient	Lf1+0.7· 1.5· Lf2+1.5· Lf3
18	permanent and transient	1.35· Lf1+0.7· 1.5· Lf2+1.5· Lf3
19	permanent and transient	Lf1+1.5· Lf4
20	permanent and transient	1.35· Lf1+1.5· Lf4
21	permanent and transient	Lf1+0.7· 1.5· Lf2+1.5· Lf4
22	permanent and transient	1.35· Lf1+0.7· 1.5· Lf2+1.5· Lf4

4.2.2. column load

increasing factor for flex. mom.: $\Delta M_{St,TH.II.O} = M_{St} \cdot$ **20%**
(for the consideration of increase of moments from non-linear effects)

LK	$N_{St,d}$ kN	$H_{x,St,d}$ kN	$H_{y,St,d}$ kN	$M_{x,St,d}$ kNm	$M_{y,St,d}$ kNm
1	17.08	-1.26	-3.15	-32.35	12.59
2	23.06	-1.70	-4.25	-43.68	16.99
3	18.85	-9.51	-22.95	-235.43	94.76
4	24.83	-9.95	-24.05	-246.75	99.16
5	17.81	-2.33	-13.09	-126.06	22.46
6	23.79	-2.77	-14.19	-137.39	26.86
7	19.58	-10.58	-32.89	-329.14	104.63
8	25.56	-11.02	-33.99	-340.46	109.04

LK	$N_{St,d}$ kN	$H_{x,St,d}$ kN	$H_{y,St,d}$ kN	$M_{x,St,d}$ kNm	$M_{y,St,d}$ kNm
9	17.81	-6.13	-4.03	-39.92	53.51
10	23.79	-6.57	-5.13	-51.25	57.92
11	19.58	-14.38	-23.83	-243.00	135.68
12	25.56	-14.82	-24.93	-254.32	140.09
13	18.32	-7.03	-17.01	-174.51	70.11
14	24.30	-7.48	-18.11	-185.83	74.51
15	18.30	-3.05	-19.71	-188.54	29.04
16	24.27	-3.49	-20.81	-199.86	33.45
17	19.53	-8.82	-33.57	-330.69	86.56
18	25.51	-9.26	-34.67	-342.01	90.96
19	18.30	-9.38	-4.62	-44.97	80.79
20	24.27	-9.82	-5.72	-56.29	85.20
21	19.53	-15.15	-18.48	-187.12	138.31
22	25.51	-15.59	-19.58	-198.45	142.71

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

LK	x = 68.5 cm kNm	x = 131.5 cm kNm
1	3.49	-3.89
2	4.72	-5.25
3	30.72	-24.88
4	31.25	-26.91
5	6.51	-6.68
6	7.72	-8.08
7	34.45	-26.98
8	34.80	-29.21
9	15.75	-15.98
10	16.97	-17.34
11	47.12	-32.82
12	46.10	-36.39
13	21.96	-19.14
14	22.43	-21.09
15	8.74	-8.35
16	9.90	-9.76
17	27.90	-22.95
18	28.62	-24.81
19	23.97	-23.93
20	25.14	-25.39
21	**48.40**	-33.26
22	46.36	**-37.44**

Design calculation for LK 22: $\varepsilon_o/\varepsilon_u$ = 26.69/-0.29‰ **min $A_{s,o}$ = 1.1 cm²**

Design calculation for LK 21: $\varepsilon_o/\varepsilon_u$ = -0.33/26.69‰ **min $A_{s,u}$ = 1.4 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

LK	y = 51.0 cm kNm	y = 149.0 cm kNm
1	2.54	-27.02
2	3.43	-36.48
3	25.27	-117.42
4	22.74	-137.06
5	10.43	-78.20
6	11.31	-87.73
7	55.94	-124.11
8	38.09	-163.44
9	3.24	-31.11
10	4.13	-40.57
11	27.58	-118.53
12	24.93	-137.53
13	15.80	-98.38
14	15.77	-111.58
15	16.67	-107.66
16	16.73	-121.03
17	**56.62**	-124.11
18	37.88	**-164.22**
19	3.72	-33.74
20	4.59	-43.29
21	18.87	-100.76
22	18.49	-113.34

Design calculation for LK 18: $\varepsilon_o/\varepsilon_u$ = 27.09/-0.80‰ **min $A_{s,o}$ = 4.9 cm²**

Design calculation for LK 17: $\varepsilon_o/\varepsilon_u$ = -0.45/27.06‰ **min $A_{s,u}$ = 1.7 cm²**

4.3.3. selected reinforcement in x-direction

top **B500A,** gleichmäßig zu verteilen
2 Ø 10 = 1.6 > 1.1 cm²

bottom (distribution acc. to [2])

width	cm	50.0	50.0	50.0	50.0	50.0	50.0
distrib.	%	17.1	20.3	20.3	17.1	13.6	11.6
min A_s	cm²	0.2	0.3	0.3	0.2	0.2	0.2
select.		**1 Ø 10**	**1 Ø 10**	**1 Ø 10**	**1 Ø 10**	**1 Ø 10**	**1 Ø 10**
exis A_s	cm²	0.8	0.8	0.8	0.8	0.8	0.8

total existing bottom reinforcement: ΣA_s = 4.7 > 1.4 cm²

4.3.4. selected reinforcement in y-direction

top **B500A,** gleichmäßig zu verteilen
7 Ø 10 = 5.5 > 4.9 cm²

bottom (distribution acc. to [2])

width	cm	200.0
distrib.	%	100.0
min A_s	cm²	1.7

select.		**3 Ø 10**
exis A_s	cm²	2.4

total existing bottom reinforcement: ΣA_s = 2.4 > 1.7 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 of perimeter
e : distance of dl to axis of M_{Ed}

coefficient for the calculation of shear stresses from moment action

(acc. to [3], table 6.1)

$c_1/c_2 = 0.98/0.63 = 1.56 \Rightarrow k_x = 0.66$

$c_1/c_2 = 0.63/0.98 = 0.64 \Rightarrow k_y = 0.49$

calculated values of basic control perimeter

LK	a_{crit} cm	a/d -	u_{crit} m	A_{crit} m²	$W_{crit,x}$ m²	$W_{crit,y}$ m²
1	15.6	0.21	4.20	1.198	1.8743	1.5300
2	15.6	0.21	4.20	1.198	1.8743	1.5300
3	2.2	0.03	3.36	0.691	1.1966	0.9059
4	4.8	0.06	3.52	0.781	1.3171	1.0160
5	4.8	0.06	3.52	0.781	1.3171	1.0160
6	6.7	0.09	3.64	0.847	1.4065	1.0980
7	12.3	0.17	3.99	1.061	1.6914	1.3605
8	15.3	0.20	4.18	1.182	1.8535	1.5108
9	13.8	0.19	4.09	1.121	1.7716	1.4348
10	14.2	0.19	4.11	1.136	1.7919	1.4536
11	7.8	0.10	3.71	0.889	1.4615	1.1485
12	10.1	0.14	3.85	0.973	1.5744	1.2525
13	2.6	0.04	3.38	0.703	1.2135	0.9213
14	4.5	0.06	3.50	0.768	1.2995	0.9999
15	1.9	0.03	3.34	0.678	1.1798	0.8906
16	3.7	0.05	3.45	0.742	1.2648	0.9681
17	11.2	0.15	3.92	1.016	1.6324	1.3060
18	13.4	0.18	4.06	1.106	1.7514	1.4160
19	13.0	0.17	4.04	1.091	1.7313	1.3974
20	13.8	0.19	4.09	1.121	1.7716	1.4348
21	6.7	0.09	3.64	0.847	1.4065	1.0980
22	8.9	0.12	3.78	0.930	1.5175	1.2000

decisive shear stress within the basic control perimeter

LK	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	17.08	7.43	8.90	-32.35	-12.59	3.96	0.010
2	23.06	10.01	12.00	-43.68	-16.99	3.96	0.014
3	18.85	24.22	16.73	-235.43	-94.76	25.76	0.022
4	24.83	24.84	19.39	-246.75	-99.16	19.72	0.041
5	17.81	17.98	14.04	-126.06	-22.46	13.60	0.020
6	23.79	20.58	17.44	-137.39	-26.86	10.97	0.026
7	19.58	3.98	4.22	-329.14	-104.63	28.14	0.145
8	25.56	3.19	3.77	-340.46	-109.04	21.53	0.151
9	17.81	8.43	9.45	-39.92	-53.51	6.41	0.018
10	23.79	11.02	12.52	-51.25	-57.92	5.69	0.021
11	19.58	14.98	13.31	-243.00	-135.68	24.43	0.055
12	25.56	14.81	14.41	-254.32	-140.09	18.99	0.074
13	18.32	22.82	16.05	-174.51	-70.11	19.74	0.018
14	24.30	25.05	19.23	-185.83	-74.51	15.51	0.030
15	18.30	24.84	16.86	-188.54	-29.04	20.33	0.012
16	24.27	27.26	20.22	-199.86	-33.45	15.94	0.025
17	19.53	6.59	6.70	-330.69	-86.56	28.46	0.125
18	25.51	5.08	5.62	-342.01	-90.96	22.00	0.145
19	18.30	9.10	9.92	-44.97	-80.79	8.33	0.023
20	24.27	11.68	13.09	-56.29	-85.20	7.05	0.026
21	19.53	16.89	14.32	-187.12	-138.31	20.96	0.040
22	25.51	17.64	16.41	-198.45	-142.71	16.39	0.053

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]

$C_{Rd,c} = 0.15/\gamma_c$

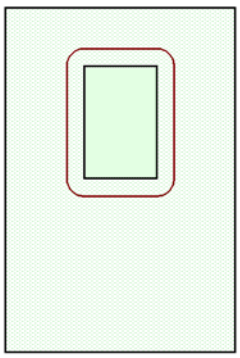
$k = 1 + \mathrm{sqrt}(200/d) \leq 2.0$ with d [mm]

$\rho_{l,tension,max}$ = 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

$a_{s,x,3d} = 4.7/3 = 1.58$ cm²/m

$a_{s,y,3d} = 2.4/2 = 1.18$ cm²/m

$\rho_{lx,tension} = 1.58/75 \cdot 10^{-2} = 0.00021$

$\rho_{ly,tension} = 1.18/74 \cdot 10^{-2} = 0.00016$

$\rho_{l,tension}$ = sqrt(0.00021·0.00016) = 0.00018

punching shear resistance without shear reinforcement

$C_{Rd,c}$ = 0.15/1.5 = 0.1

$\rho_{l,tension,max}$ = minimum from (0.02, 0.5·14.17/434.78) = 0.0163 > 0.0002

v_{min}·2·d/a = 0.0416/1.5·1.52$^{3/2}$·25$^{0.5}$·2·74.5/15.3 = 2.532 N/mm^2

$v_{Rd,c}$ = 0.1·1.52·(100·0.00018·25)$^{1/3}$·2·74.5/15.3 = 1.142 N/mm^2 < 2.532 N/mm^2 ⇒ $v_{Rd,c}$ = 2.532 N/mm^2

0.151 N/mm^2 < 2.532 N/mm^2 ⇒ no additional reinforcement required

5. summary

all executed verifications and design calculations successful.

longitudinal reinforcement x-direction (top) min $A_{s,x}$ = 1.1 cm^2	longitudinal reinforcement y-direction (top) min $A_{s,y}$ = 4.9 cm^2
longitudinal reinforcement x-direction (bottom) min $A_{s,x}$ = 1.4 cm^2	longitudinal reinforcement y-direction (bottom) min $A_{s,y}$ = 1.7 cm^2

Literatur and Normen:

[1] DIN V 4141-1: Lager im Bauwesen, Teil 1, Mai 2003
[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