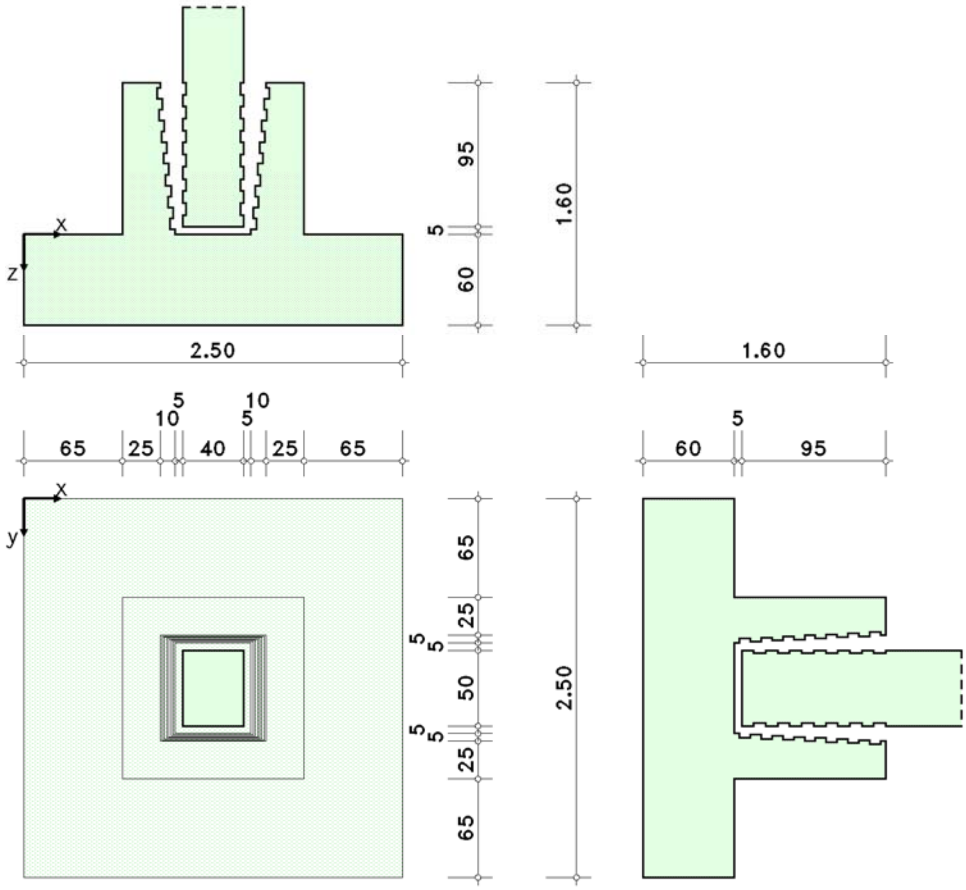
sleeve foundation

reinf. concr. design acc. to DIN EN 1992-1-1:2011-01 with NA-Deutschland(DIN EN 1992-1-1/NA:2013-04)

scale 1:50, profiling not to scale



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

column:
concrete strength class **C45/55**
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

indication: the thickness of the last stratum is assumed infinite.

stratum	d m	z m	γ kN/m³	γ' kN/m³	φ °	c_k kN/m²	E_m MN/m²	δ_p °
stratum 1	1.50	0.00	19.00	11.00	25.0	15.0	50.00	auto
stratum 2	99.00	1.50	22.00	13.00	37.0	---	80.00	auto

z - levelan top edge der stratum   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

2. loading

2.1. Structure of action effects

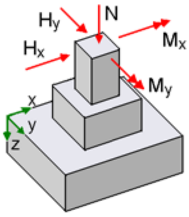
On the left-hand side, the action effects and load cases are shown in a tree structure. The right-hand side shows their characteristics of the superposition.

used symbols: action   load case

1: permanent loads	**permanent loads**
└ 1: dead load (1)	additive

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.

2: live loads (2)	**variable live loads in assembly and salesrooms**
2: live loads (2/1)	additive
3: new action	**earthquake load**
3: Erbeben	alternative in group B



2.2. Characteristic column load

point of application in column centroid auf top edge Köcher

loadc.	N_{St} kN	$H_{x,St}$ kN	$H_{y,St}$ kN	$M_{x,St}$ kNm	$M_{y,St}$ kNm
1	2500.00	-150.00	0.00	0.00	350.00
2	1500.00	-50.00	0.00	0.00	150.00
3	0.00	0.00	500.00	200.00	0.00

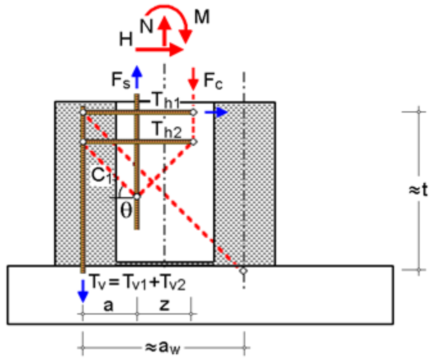
2.3. dead load

Das Gewicht der foundation slab wird with γ_E = 25.00 kN/m³ berücksichtigt.
the height of the earth load is h_A = 0.20 m.
the mean unit weight of the earth load is γ_A = 19.00 kN/m³.
the resultant of dead load in the floor joint is $N_{0,deadl,k}$ = 148.03 kN.
Das dead load wird im load case 1 with berücksichtigt.

3. transmission of force from the column to the foundation

sleeve design with a strut and tie model for distinctive moment action acc. to [1]:
back hung of shear force $T_{h1} = V_{Ed}$
vertical tensile force from curvature of T_{h1} $T_{v1} = T_{h1} \cdot t/a_w$
maximum column tensile force F_s = min $A_{s,St,tension} \cdot f_{yd,St}$
vertical tensile force from column tensile force F_s and reinf.offset $T_{v2} = F_s \cdot z/(a+z)$
horizontal component of the compression strut C_1 $T_{h2} = T_{v2}/\tan\theta$
horizontal reinforcem. min $A_{s,h} = (T_{h1}+T_{h2})/(f_{yd,K})$
vertical reinforcem. min $A_{s,v} = (T_{v1}+T_{v2})/(f_{yd,K})$

requirements: profiled lateral surfaces of column base
and interior surfaces of the sleeve acc. to figure 6.9 aus [2].



3.1. design values sleeve design

3.1.1. 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+0.7· 1.5· Lf2
6	permanent and transient	1.35· Lf1+0.7· 1.5· Lf2
7	earthqu.	Lf1+Lf3
8	earthqu.	Lf1+Lf3+0.6· Lf2

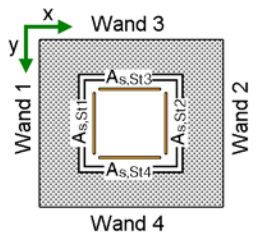
3.1.2. column load

in column centroid, top edge of sleeve

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	2500.00	-150.00	0.00	0.00	420.00
2	3375.00	-202.50	0.00	0.00	567.00
3	4750.00	-225.00	0.00	0.00	690.00
4	5625.00	-277.50	0.00	0.00	837.00
5	4075.00	-202.50	0.00	0.00	609.00
6	4950.00	-255.00	0.00	0.00	756.00
7	2500.00	-150.00	500.00	240.00	420.00
8	3400.00	-180.00	500.00	240.00	528.00

3.2. input values



edge distance column longit. reinf. $d_{1,St}$ = 5.0 cm
upper clearance thickness t_{Fx}/t_{Fy} = 15.0/10.0 cm
edge distance vert. sleeve reinf. $d_{1,K}$ = 4.0 cm
distance der vert. reinforcementsschenkel im Köcher s_K = 15.0 cm
existing column longitudinal reinforcement
° face 1 and 2: $A_{s,St,1} = A_{s,St,2}$ = 25.1 cm²
° face 3 and 4: $A_{s,St,3} = A_{s,St,4}$ = 25.1 cm²
max. diameter of existing column longitudinal reinforcement d_s = Ø 10
assumption for lower concrete cover in column base $c_{nom,St,u}$ = 3.0 cm
assumption for upper concrete cover in the slab $c_{nom,F,o}$ = 3.5 cm
assumption for angle of compression strut in the clearance Θ = 45 °

3.3. Design calculation in x/z-plane

minimum column reinforcement

LK	$A_{s,St}$ (per side) cm²	$A_{s,St,comp.}$ cm²	$A_{s,St,tens.}$ cm²
1	14.5	14.5	14.5
2	32.1	32.1	32.1
3	54.2	54.2	54.2
4	74.9	74.9	74.9

LK	$A_{s,St}$ (per side) cm²	$A_{s,St,comp.}$ cm²	$A_{s,St,tens.}$ cm²
5	41.3	41.3	41.3
6	61.7	61.7	61.7
7	14.5	14.5	14.5
8	29.0	29.0	29.0

$a_w = h_{St,x} + 2 \cdot t_{F,x} + d_{w,x} = 0.95$ m
$a = d_{1,St} + t_{F,x} + d_{1,K} + s_K = 31.5$ cm
$z = \max\{ 0.9 \cdot (h_{St,x} - d_1); (h_{St,x} - 2 \cdot d_1) \} = 31.5$ cm

minimum sleeve reinforcement

LK	H_x kN	T_{h1} kN	T_{v1} kN	F_s kN	T_{v2} kN	T_{h2} kN	min $A_{s,h,K}$ cm²	min $A_{s,v,K}$ cm²
1	-150.00	150.0	150.0	628.8	314.4	314.4	10.7	10.7
2	-202.50	202.5	202.5	1397.3	698.7	698.7	20.7	20.7
3	-225.00	225.0	225.0	2357.1	1178.5	1178.5	32.3	32.3
4	-277.50	277.5	277.5	3257.0	1628.5	1628.5	**43.8**	**43.8**
5	-202.50	202.5	202.5	1796.3	898.1	898.1	25.3	25.3
6	-255.00	255.0	255.0	2680.6	1340.3	1340.3	36.7	36.7
7	-150.00	150.0	150.0	628.8	314.4	314.4	10.7	10.7
8	-180.00	180.0	180.0	1260.5	630.3	630.3	18.6	18.6

3.4. Design calculation in y/z-plane

minimum column reinforcement

LK	$A_{s,St}$ (per side) cm²	$A_{s,St,comp.}$ cm²	$A_{s,St,tens.}$ cm²
1	-	-	-
2	-	-	-
3	-	-	-
4	6.6	6.6	-

LK	$A_{s,St}$ (per side) cm²	$A_{s,St,comp.}$ cm²	$A_{s,St,tens.}$ cm²
5	-	-	-
6	-	-	-
7	-	-	-
8	-	-	-

$a_w = h_{St,y} + 2 \cdot t_{F,y} + d_{w,y} = 0.95$ m
$a = d_{1,St} + t_{F,y} + d_{1,K} + s_K = 31.5$ cm
$z = \max\{ 0.9 \cdot (h_{St,y} - d_1); (h_{St,y} - 2 \cdot d_1) \} = 40.5$ cm

LK	H_x kN	T_{h1} kN	T_{v1} kN	F_s kN	T_{v2} kN	T_{h2} kN	min $A_{s,h,K}$ cm²	min $A_{s,v,K}$ cm²
1	0.00	0.0	0.0	0.0	0.0	0.0	-	-
2	0.00	0.0	0.0	0.0	0.0	0.0	-	-
3	0.00	0.0	0.0	0.0	0.0	0.0	-	-
4	0.00	0.0	0.0	0.0	0.0	0.0	-	-
5	0.00	0.0	0.0	0.0	0.0	0.0	-	-
6	0.00	0.0	0.0	0.0	0.0	0.0	-	-
7	500.00	500.0	500.0	0.0	0.0	0.0	**11.5**	**11.5**
8	500.00	500.0	500.0	0.0	0.0	0.0	11.5	11.5

3.5. resultant horizontal sleeve reinforcement

minimum reinforcement per sleeve face min $A_{s,h,wall} = 43.8 / 2 =$ **21.9 cm²**

3.6. selection of sleeve reinforcement

selected: **B500A,** vertical for face 1 and 2 jeweils
20 double shear stirrups Ø 12 = 45.2 > 43.8 cm²
B500A, vertical for face 3 and 4 jeweils
6 double shear stirrups Ø 12 = 13.6 > 11.5 cm²
B500A, horizontal per sleeve cheek
10 double shear stirrups Ø 12 = 22.6 > 21.9 cm²

3.7. minimum anchoring depth

min $t = 1.5 \cdot h_{St} = 75.0$ cm < 95.0 cm

3.8. anchorage of column longitudinal reinforcement

existing anchorage length exis $l_b = t - c_{nom,St,u} = 92.0$ cm

basic anchorage length

design value of steel stress (B500A, $\gamma_s = 1.15$) $f_{yd} = 434.78$ N/mm²
design value of bond stress f_{bd}:
° concrete parameters (C45/55) $f_{ctk;0.05} = 2.66$ N/mm², $\gamma_c = 1.50$
° good bond conditions
$f_{bd} = 2.25 \cdot f_{ctk;0.05} / \gamma_c = 3.99$ N/mm²

basic anchorage length $l_b = (d_s/4)\cdot(f_{yd}/f_{bd}) = 27.2$ cm

anchorage as compression reinforcement
straight bar ends: $\alpha_a = 1.0$
unfavourable ratio of min $A_{s,compr.}$/exis $A_s = 2.98$
minimum value of anchorage length $l_{b,min} = \max\{0.6\cdot l_b,\ 10\cdot d_s\} = 16.3$ cm
minimum anchorage length $l_{b,net} = \max\{\alpha_a\cdot l_b\cdot A_{s,min}/A_{s,exis},\ l_{b,min}\} =$ **81.3** < 92.0 cm

anchorage as bending-tension reinforcement
straight bar ends: $\alpha_a = 1.0$
unfavourable ratio of min $A_{s,tens.}$/exis $A_s = 2.98$
minimum value of anchorage length $l_{b,min} = \max\{0.3\cdot\alpha_a\cdot l_b,\ 10\cdot d_s\} = 10.0$ cm
minimum anchorage length $l_{b,net} = \max\{\alpha_a\cdot l_b\cdot A_{s,min}/A_{s,exis},\ l_{b,min}\} =$ **81.3** < 92.0 cm

lap length of bending-tension reinforcement with vertical upstanding stirrup leg
existing lap length exis $l_b = t - c_{nom,St,u} - c_{nom,F,o} = 88.5$ cm
clear bar distance $a_n = a - (d_{s,St} + d_{s,K})/2. = 30.4$ cm
reduced anchorage length $l_{b,net} = \max\{(T_{v2}/F_s)\cdot l_{b,net},\ l_{b,min}\} = 40.7$ cm
coefficient for the lap lengths of a tensile joint with > 30: $\alpha_l = 1.4$
minimum value of lap length $l_{s,min} = \max\{0.3\cdot\alpha_a\cdot\alpha_l\cdot l_b,\ 15\cdot d_s,\ 200\text{ mm}\} = 20.0$ cm
minimum lap length $l_s = \max\{l_{b,net}\cdot\alpha_l,\ l_{s,min}\} + (a_n - 4\cdot d_s) =$ **83.3** < 88.5 cm

3.9. anchorage of vertical stirrups

basic anchorage length
design value of steel stress (B500A, $\gamma_s = 1.15$) $f_{yd} = 434.78$ N/mm²
design value of bond stress f_{bd}:
° concrete parameters (C30/37) $f_{ctk;0.05} = 2.03$ N/mm², $\gamma_c = 1.50$
° good bond conditions
$f_{bd} = 2.25\cdot f_{ctk;0.05}/\gamma_c = 3.04$ N/mm²
basic anchorage length $l_b = (d_s/4)\cdot(f_{yd}/f_{bd}) = 42.8$ cm

minimum anchorage length
unfavourable ratio of $A_{s,min}/A_{s,exis} = 0.97$
bar end formed as loop, acc. to [3], table 26 (with $d_{br} > 15\ d_s$) $\alpha_a = 0.5$
minimum value of anchorage length $l_{b,min} = \max\{0.3\cdot\alpha_a\cdot l_b,\ 10\cdot d_s\} = 12.0$ cm
minimum anchorage length $l_{b,net} = \max\{\alpha_a\cdot l_b\cdot A_{s,min}/A_{s,exis},\ l_{b,min}\} =$ **20.8** cm

lap length with column longitudinal reinforcement
coefficient for the lap lengths of a tensile joint with > 30: $\alpha_l = 1.4$
minimum value of lap length $l_{s,min} = \max\{0.3\cdot\alpha_a\cdot\alpha_l\cdot l_b,\ 15\cdot d_s,\ 200\text{ mm}\} = 20.0$ cm
minimum lap length $l_s = \max\{l_{b,net}\cdot\alpha_l,\ l_{s,min}\} + (a_n - 4\cdot d_s) =$ **54.7** < 88.5 cm
t_{Fx}/t_{Fy} - upper clearance thickness in the particular axis direction t - column anchoring depth

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
earthquake	1.50	1.15

4.2. design values of reinforced concrete design

Die Mobilisierung des passive earth pressurees wird vernachlässigt.

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+0.7· 1.5· Lf2
6	permanent and transient	1.35· Lf1+0.7· 1.5· Lf2
7	earthqu.	Lf1+Lf3
8	earthqu.	Lf1+Lf3+0.6· Lf2

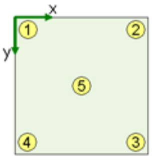
4.2.2. column load

in centroid of column, on top edge of slab
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	2536.00	-150.00	0.00	0.00	570.00
2	3423.60	-202.50	0.00	0.00	769.50
3	4786.00	-225.00	0.00	0.00	915.00
4	5673.60	-277.50	0.00	0.00	1114.50
5	4111.00	-202.50	0.00	0.00	811.50
6	4998.60	-255.00	0.00	0.00	1011.00
7	2536.00	-150.00	500.00	740.00	570.00
8	3436.00	-180.00	500.00	740.00	708.00

4.3. base pressure

determination of base pressures assuming linear soil stresses and elimination of tension stress in the corner points: σ_1 to σ_4, stress in centroid: σ_5



LK	σ1 kN/m²	σ2 kN/m²	σ3 kN/m²	σ4 kN/m²	σ5 kN/m²
1	677.12	170.24	170.24	677.12	423.68
2	914.12	229.83	229.83	914.12	571.97
3	1186.88	380.48	380.48	1186.88	783.68
4	**1423.88**	440.07	440.07	1423.88	931.97
5	1033.96	317.41	317.41	1033.96	675.68
6	1270.95	377.00	377.00	1270.95	823.97
7	268.38	0.00	564.17	1104.33	416.27
8	480.42	0.00	652.63	1285.43	566.53

4.4. Design calculation for bending

4.4.1. 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 = 65.0 cm kNm	x = 185.0 cm kNm	LK	x = 65.0 cm kNm	x = 185.0 cm kNm
1	324.48	103.18	5	503.33	190.50
2	438.04	139.30	6	616.90	226.62
3	579.98	227.92	7	327.83	108.34
4	**693.55**	264.04	8	427.40	154.75

Design calculation for LK 4: $\varepsilon_o/\varepsilon_u$ = -2.18/27.47‰ **min $A_{s,u}$ = 28.5 cm²**

4.4.2. 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 = 65.0 cm kNm	y = 185.0 cm kNm	LK	y = 65.0 cm kNm	y = 185.0 cm kNm
1	213.83	213.83	5	346.92	346.92
2	288.67	288.67	6	421.76	421.76
3	403.95	403.95	7	48.14	392.40
4	**478.79**	478.79	8	117.39	464.99

Design calculation for LK 4: $\varepsilon_o/\varepsilon_u$ = -1.72/27.97‰ **min $A_{s,u}$ = 19.9 cm²**

4.4.3. selected reinforcement in x-direction

bottom (distribution acc. to [4])

width	cm	31.3	31.3	31.3	31.3	31.3	31.3	31.3	31.3
distrib.	%	8.0	10.0	14.0	18.0	18.0	14.0	10.0	8.0
min A_s	cm²	2.3	2.8	4.0	5.1	5.1	4.0	2.8	2.3
select.		**3 Ø 12**	**3 Ø 12**	**4 Ø 12**	**5 Ø 12**	**5 Ø 12**	**4 Ø 12**	**3 Ø 12**	**3 Ø 12**
exis A_s	cm²	3.4	3.4	4.5	5.7	5.7	4.5	3.4	3.4

total existing bottom reinforcement: ΣA_s = 33.9 > 28.5 cm²

4.4.4. selected reinforcement in y-direction

bottom (distribution acc. to [4])

width	cm	31.3	31.3	31.3	31.3	31.3	31.3	31.3	31.3
distrib.	%	7.6	10.0	14.0	18.4	18.4	14.0	10.0	7.6
min A_s	cm²	1.5	2.0	2.8	3.7	3.7	2.8	2.0	1.5
select.		**2 Ø 12**	**2 Ø 12**	**3 Ø 12**	**4 Ø 12**	**4 Ø 12**	**3 Ø 12**	**2 Ø 12**	**2 Ø 12**
exis A_s	cm²	2.3	2.3	3.4	4.5	4.5	3.4	2.3	2.3

total existing bottom reinforcement: ΣA_s = 24.9 > 19.9 cm²

$\varepsilon_o/\varepsilon_u$ - strains in extreme fibres (top/bottom)

4.5. punching shear calculation

4.5.1. 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 [2], table 6.1)

$c_1 = c_2 = 1.2 \Rightarrow k_x = k_y = 0.6$

calculated values of basic control perimeter

LK	a_{crit} cm	a/d -	u_{crit} m	A_{crit} m^2	$W_{crit,x}$ m^2	$W_{crit,y}$ m^2
1	30.2	0.56	6.70	3.179	4.3922	4.3922
2	30.2	0.56	6.70	3.179	4.3922	4.3922
3	30.5	0.56	6.72	3.198	4.4156	4.4156
4	30.5	0.56	6.72	3.198	4.4156	4.4156
5	30.5	0.56	6.72	3.198	4.4156	4.4156
6	30.5	0.56	6.72	3.198	4.4156	4.4156
7	29.7	0.55	6.67	3.143	4.3455	4.3455
8	30.0	0.55	6.68	3.161	4.3688	4.3688

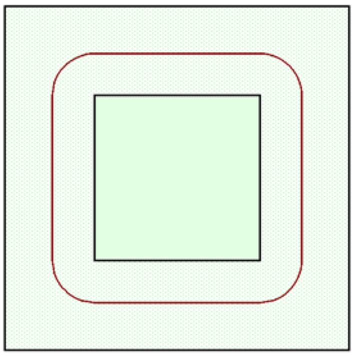
decisive shear stress within the basic control perimeter

LK	V_{Ed} kN	$\sigma_{Ed,gd,m}$ kN/m^2	ΔV_{Ed} kN	$M_{Ed,x,Sp}$ kNm	$M_{Ed,y,Sp}$ kNm	β -	$v_{Ed,crit}$ N/mm^2
1	2536.00	406.04	1290.93	-0.00	-570.00	1.21	0.411
2	3423.60	548.15	1742.75	-0.00	-769.50	1.21	0.555
3	4786.00	766.28	2450.26	0.00	-915.00	1.17	0.749
4	5673.60	908.39	2904.67	0.00	-1114.50	1.18	0.892
5	4111.00	658.21	2104.69	0.00	-811.50	1.18	0.647
6	4998.60	800.32	2559.10	0.00	-1011.00	1.18	0.789
7	2536.00	405.68	1275.01	740.00	-570.00	1.34	0.465
8	3436.00	550.13	1739.00	740.00	-708.00	1.27	0.593

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

4.5.2. 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]
$C_{Rd,c} = 0.15/\gamma_C$
$k = 1 + \text{sqrt}(200/d) \leq 2.0$ with d [mm]
$\rho_{l,tension,max}$ = minimum from $(0.02, 0.5 \cdot f_{cd}/f_{yd})$
$\rho_{l,tension} = \text{sqrt}(\rho_{lx,tension} \cdot \rho_{ly,tension}) \leq \rho_{l,tension,max}$
$v_{min} = 0.0525/\gamma_C \cdot k^{3/2} \cdot f_{ck}^{1/2}$ for d ≤ 600 mm



mean effective depth
$d_m = (55 + 54)/2 = 54.5$ cm

scale factor
$k = 1 + \text{sqrt}(200/545) = 1.61 < 2$

longitudinal reinf. ratio of the anchored tension reinf.
mean of the tension reinforcement to the distance 3d from the column
$a_{s,x,3d} = 33.9/2.5 = 13.56$ cm²/m
$a_{s,y,3d} = 24.9/2.5 = 9.94$ cm²/m
$\rho_{lx,tension} = 13.56/55 \cdot 10^{-2} = 0.00247$
$\rho_{ly,tension} = 9.94/54 \cdot 10^{-2} = 0.00184$
$\rho_{l,tension} = \text{sqrt}(0.00247 \cdot 0.00184) = 0.00213$

4.5.2.1. Permanent and transient design situation (LK 4)

$C_{Rd,c} = 0.15/1.5 = 0.1$
$\rho_{l,tension,max}$ = minimum from $(0.02, 0.5 \cdot 17/434.78) = 0.0195 > 0.0021$
$v_{min} \cdot 2 \cdot d/a = 0.0525/1.5 \cdot 1.61^{3/2} \cdot 30^{0.5} \cdot 2 \cdot 54.5/30.5 = 1.393$ N/mm²
$v_{Rd,c} = 0.1 \cdot 1.61 \cdot (100 \cdot 0.00213 \cdot 30)^{1/3} \cdot 2 \cdot 54.5/30.5 = 1.064$ N/mm² < 1.393 N/mm² $\Rightarrow v_{Rd,c} = 1.393$ N/mm²

0.892 N/mm² < 1.393 N/mm² ⇒ no additional reinforcement required

4.5.2.2. Design calculation situation earthquake (LK 8)

$C_{Rd,c} = 0.15/1.5 = 0.1$
$\rho_{l,tension,max}$ = minimum from $(0.02, 0.5 \cdot 17/434.78) = 0.0195 > 0.0021$
$v_{min} \cdot 2 \cdot d/a = 0.0525/1.5 \cdot 1.61^{3/2} \cdot 30^{0.5} \cdot 2 \cdot 54.5/30 = 1.418$ N/mm²
$v_{Rd,c} = 0.1 \cdot 1.61 \cdot (100 \cdot 0.00213 \cdot 30)^{1/3} \cdot 2 \cdot 54.5/30 = 1.084$ N/mm² < 1.418 N/mm² $\Rightarrow v_{Rd,c} = 1.418$ N/mm²

0.593 N/mm² < 1.418 N/mm² ⇒ no additional reinforcement required

5. Drehfeder des Systems foundation-soil

determination der Drehfederkonstante with Hilfe des beddingsmodules.
$c_{v,X} = k_S \cdot I_X$
$c_{v,Y} = k_S \cdot I_Y$
Abschätzung des beddingsmodules acc. to [5]
$k_S = E_S / (f \cdot (b_x \cdot b_x)^{0.5})$
with Formfaktor f abhängig vom aspect ratio: 1:1 -> f = 0.45, 1:2 -> f 0.42, 1:4 -> f = 0.35

Abschätzung fuer limiting depth with $d_S = 2 \cdot b_{min} = 2 \cdot 2.50 = 5.00$m
Gewogenes Mittel fuer Zusamendrückungsmodul to zur limiting depth $E_{m,cal} = 75800.00$ kN/m²
assumption for Korrekturfaktor $\kappa = 1$
stiffenerziffer $E_S = 1 \cdot 75800.00 = 75800.00$ kN/m²

Formfaktor f = 1.00

beddingsmodul $k_S = 30320.00$ kN/m³

Trägheitsmoment I_X / I_X = 3.26 / 3.26 kN/m³

Drehfeder about the x-axis **$c_{v,x}$ = 98697.92 kNm**
Drehfeder about the y-axis **$c_{v,y}$ = 98697.92 kNm**

6. summary

all executed verifications and design calculations successful.

longitudinal reinforcement x-direction	Drehfeder about the x-axis
min $A_{s,x}$	$c_{v,X}$
= 28.5 cm^2	= 98697.92 kNm
longitudinal reinforcement y-direction	Drehfeder about the y-axis
min $A_{s,y}$	$c_{v,Y}$
= 19.9 cm^2	= 98697.92 kNm

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