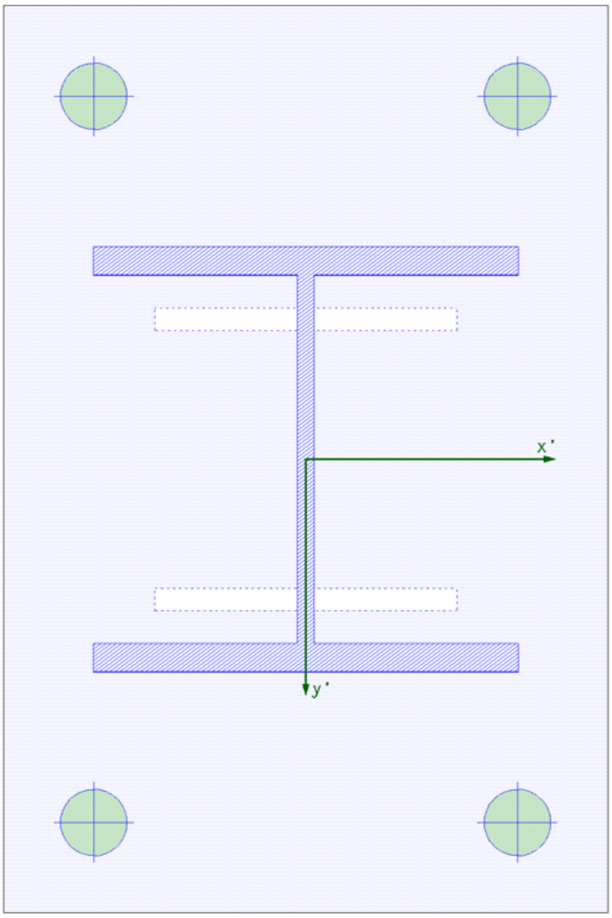
BASE PLATE HEB280

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)
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 1:5



column cross section
standardized profile: HE280B, of quality S275 N/NL

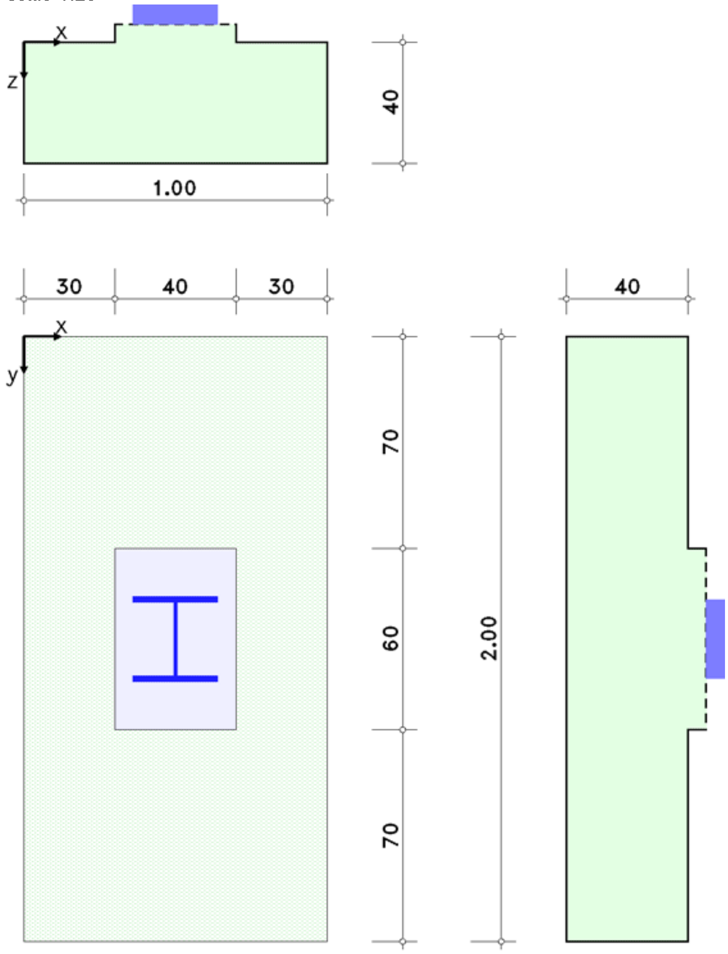
base plate
b_x = 400 mm b_y = 600 mm t = 60 mm, of quality S275 N/NL

mortar joint
t_F = 20 mm

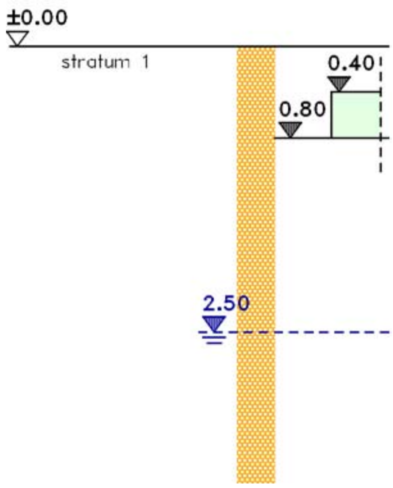
shear connector
standardized profile: HE200B, of quality S235

anchors
4 anchors, FK 5.8, M24, without shaft
with a length of 450 mm
edge distances a_x/a_y = 60/60 mm

elevation, top view isolated foundation
scale 1:25



soil profile



concrete strength class C30/37
steel class B500A

1. soil situation

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

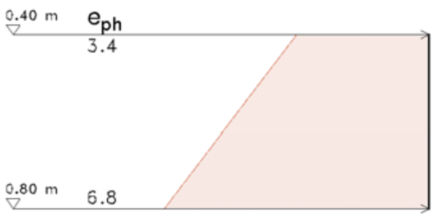
1.1. designation and characteristic values of soil strata

stratum	d m	z m	γ kN/m³	γ' kN/m³	φ °	c_k kN/m²	E_m MN/m²	δ_p °
stratum 1	99.00	0.00	20.00	11.00	35.0	---	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

1.2. char. passive earth pressure

Als passive earth pressure wird der earth pressure at rest angesetzt.



Σ(γ·h)   Summe soilgewicht in der betrachteten Tiefe
Σ(γ·h)cal   Summe soilgewicht in der betrachteten Tiefe zuzüglich Böschungseinfluß
K_{0gh} coeff. of earth pressure acc. to [1] clause 6.2.1, Gl.(7) (formulation acc. to Müller-Breslau)
e_{0h} horiz. Erddruckordinate

z m	Σ(γ·h) kN/m²	Σ(γ·h)cal kN/m²	K_{0gh} -	e_{0h} kN/m²
0.40	8.00	12.00	0.426	3.41
0.80	16.00	16.00	0.426	6.82

the resultant maximum passive earth pressure is E_{0hg} = 2.05 kN/m, at z_s = 0.62 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

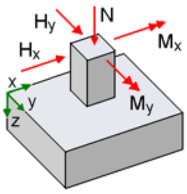
1: permanent loads	**permanent loads**
└ 1: dead load (1)	additive
2: live loads (2)	**variable live loads in assembly and salesrooms**
└ 2: live loads (2/1)	additive

2.2. Design calculation situation of load cases for external stability

loadc.	notation	BS-P	BS-T
1	dead load (1)	x	
2	live loads (2/1)	x	

2.3. 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	200.00	0.00	30.00	50.00	0.00
2	100.00	0.00	50.00	75.00	0.00

2.4. 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 = 0.40 m.
the mean unit weight of the earth load is γ_A = 20.00 kN/m³.

the resultant of dead load in the floor joint is $N_{0,deadl,k}$ = 34.08 kN.

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
permanent	1.00	1.25	1.50

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

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	200.00	0.00	30.00	60.00	0.00
2	270.00	0.00	40.50	81.00	0.00
3	350.00	0.00	105.00	195.00	0.00
4	420.00	0.00	115.50	216.00	0.00

3.3. weld between column shaft and base plate

design with direction oriented method acc. to clause 4.5.3.2

$\sigma_{V,w,Ed} = (\sigma_\perp^2 + 3 \cdot \tau_\perp^2 + 3 \cdot \tau_{II}^2)^{0.5}$,
$f_{1,w,Rd} = f_u / (\beta_w \cdot \gamma_{M2})$
$f_{2,w,Rd} = 0.9 \cdot f_u / \gamma_{M2}$
$U = \max\{ \sigma_{V,w,Ed}/f_{1,w,Rd}, \sigma_\perp / f_{2,w,Rd} \}$

connection designed with a **full-size double fillet weld.**
axial force transfer of 30 % by the weld.

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

LK	$a_{w,Fl}$ mm	$a_{w,S}$ mm	$\sigma_\perp$ kN/cm²	$\tau_\perp$ kN/cm²	τ_{II} kN/cm²	$\sigma_{V,w,Ed}$ kN/cm²	$f_{1,w,Rd}$ kN/cm²	$f_{2,w,Rd}$ kN/cm²	Maßgeb.	U -
1	**8**	**8**	-4.32	-4.32	0.00	8.65	36.71	28.08	flange	0.24
2	8	8	-5.84	-5.84	0.00	11.68	36.71	28.08	flange	0.32
3	8	8	-13.48	-13.48	0.00	26.97	36.71	28.08	flange	0.73
4	8	8	-15.00	-15.00	0.00	29.99	36.71	28.08	flange	**0.82**

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 = 0.82 < 1.00

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

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 22 elements in X-direction and 35 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 = 1647.33 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	19.1	14.6	12.10	28.26	0.78	0.61	-1.94
2	19.1	14.6	16.39	38.28	1.06	0.82	-2.63
3	20.9	14.6	46.78	111.69	-2.82	-2.25	-8.35
4	19.1	14.6	51.63	122.97	3.14	2.50	-9.11

stresses and utilizations

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

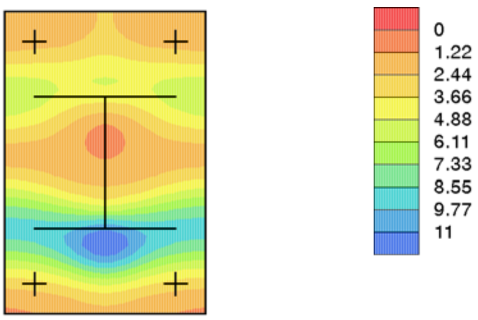
LK	x_{Fp} cm	y_{Fp} cm	$\sigma_{Pl,V}$ kN/cm²	σ_{Rd} kN/cm²	U -
1	19.1	14.6	2.80	25.50	0.11
2	19.1	14.6	3.79	25.50	0.15
3	20.9	14.6	11.09	25.50	0.44
4	19.1	14.6	12.21	25.50	**0.48**

maximum utilization U = 0.48 < 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 4 (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 ($A_{compression}$ > 120.0 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%

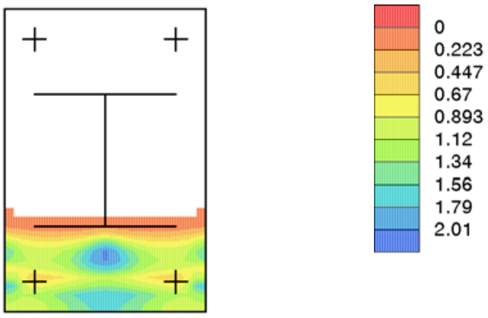
LK	lim $\sigma_{c,d}$ kN/cm²	$A_{compression}$ cm²	$F_{compression}$ kN	$A_{\sigma c>fjd}$ cm²	$\sigma_{c,max}$ kN/cm²	$\sigma_{c,m}$ kN/cm²	f_{jd} kN/cm²	U_{fjd} -	$A_{\sigma c>fjd}$ / $A_{compression}$ %
1	5.10	941.3	251.22	-	0.75	0.267	1.70	0.16	0.00
2	5.10	941.3	339.49	-	1.01	0.361	1.70	0.21	0.00
3	5.10	822.9	619.69	6.2	1.75	0.753	1.70	0.44	0.76
4	5.10	822.9	710.07	31.2	2.01	0.863	1.70	**0.51**	**3.79**

maximum utilization U_{fjd} = 0.51 < 1.00

maximum share of the compression area with $\sigma_c > f_{jd}$: $A_{\sigma c>fjd}/A_{compression}$ = 3.79 < 30.00 %
associated utilization $U_{A,compression}$ = 0.13 < 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 $\sigma_c > f_{cd}$ $\sigma_c > f_{jd}$

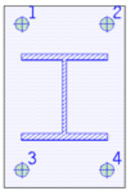
pressure distribution [kN/cm²]
LK 4 (max $\sigma_{c,m}$)



3.4.3. anchor tensile forces

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

numeration



stress area of M24: A_s = 3.53 cm²
No countersunk bolts used: k2 = 0.90

LK	$F_{t,Ed,1}$ kN	$F_{t,Ed,2}$ kN	$F_{t,Ed,3}$ kN	$F_{t,Ed,4}$ kN	$F_{t,Rd}$ kN	U_{max} -
1	25.61	25.61	---	---	127.08	0.20
2	34.75	34.75	---	---	127.08	0.27
3	134.84	134.84	---	---	127.08	1.06
4	145.03	145.03	---	---	127.08	**1.14**

maximum utilization U = **1.14 > 1.00** ⇒ **allowable tension force exceeded** **!!**

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 = 22.0 cm
length in concrete l_c = 20.0 cm

3.5.1. concrete compression

$\sigma_c = V_{Ed}/(l_c \cdot b)$
$U = \sigma_{c,max}/f_{cd}$

LK	$V_{Ed,flange}$ kN	$V_{Ed,web}$ kN	$\sigma_{c,flange}$ N/mm²	f_{cd} N/mm²	U -
1	30.00	0.00	2.50	17.00	0.15
2	40.50	0.00	2.89	17.00	0.17
3	105.00	0.00	2.92	17.00	**0.17**
4	115.50	0.00	2.89	17.00	0.17

maximum utilization U = 0.17 < 1.00

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

3.5.2. stresses in connection of base plate

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

LK	$M_{x,Ed}$ kNcm	$M_{y,Ed}$ kNcm	σ_{Ed} kN/cm²	τ_{Ed} kN/cm²	$\sigma_{v,Ed}$ kN/cm²	σ_{Rd} kN/cm²	U -
1	150.00	0.00	0.26	1.81	3.14	23.50	0.13
2	222.75	0.00	0.39	2.45	4.24	23.50	0.18
3	1155.00	0.00	2.03	6.35	10.99	23.50	0.47
4	1386.00	0.00	2.43	6.98	12.09	23.50	**0.51**

maximum utilization U = 0.51 < 1.00

$\sigma_{v,Ed}$ - equivalent 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
$\sigma_{V,w,Ed} = (\sigma_\perp^2 + 3 \cdot \tau_\perp^2 + 3 \cdot \tau_{II}^2)^{0.5}$,
$f_{1,w,Rd} = f_u/(\beta_w \cdot \gamma_{M2})$
$f_{2,w,Rd} = 0.9 \cdot f_u/\gamma_{M2}$
$U = \max\{ \sigma_{V,w,Ed}/f_{1,w,Rd},\ \sigma_\perp/f_{2,w,Rd} \}$

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

LK	$a_{w,Fl}$ mm	$a_{w,S}$ mm	$\sigma_\perp$ kN/cm²	$\tau_\perp$ kN/cm²	τ_{II} kN/cm²	$\sigma_{V,w,Ed}$ kN/cm²	$f_{1,w,Rd}$ kN/cm²	$f_{2,w,Rd}$ kN/cm²	Maßgeb.	U -
1	**8**	**8**	0.13	0.13	1.40	2.44	36.00	25.92	web	0.07
2	8	8	0.20	0.20	1.89	3.30	36.00	25.92	web	0.09
3	8	8	1.03	1.03	4.90	8.73	36.00	25.92	web	0.24
4	8	8	1.24	1.24	5.39	9.65	36.00	25.92	web	**0.27**

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 = 0.27 < 1.00

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

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

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

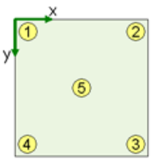
4.2.2. column load

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	200.00	0.00	30.00	60.00	0.00
2	270.00	0.00	40.50	81.00	0.00
3	350.00	0.00	105.00	195.00	0.00
4	420.00	0.00	115.50	216.00	0.00

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	σ_1 kN/m²	σ_2 kN/m²	σ_3 kN/m²	σ_4 kN/m²	σ_5 kN/m²
1	9.04	9.04	225.04	225.04	117.04
2	12.20	12.20	303.80	303.80	158.00
3	0.00	0.00	668.65	668.65	86.62
4	0.00	0.00	710.35	**710.35**	168.94

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 = 30.0 cm kNm	x = 70.0 cm kNm
1	8.91	8.91
2	12.03	12.03
3	15.66	15.66
4	**18.78**	18.78

Design calculation for LK 4: $\varepsilon_o/\varepsilon_u$ = -0.50/28.64‰ **min $A_{s,u}$ = 1.2 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 = 70.0 cm kNm	y = 130.0 cm kNm
1	3.98	44.55
2	5.37	60.14
3	-4.41	126.14
4	**-5.95**	**137.13**

Design calculation for LK 4: $\varepsilon_o/\varepsilon_u$ = 29.48/-0.41‰ **min $A_{s,o}$ = 0.4 cm²**

Design calculation for LK 4: $\varepsilon_o/\varepsilon_u$ = -2.67/29.88‰ **min $A_{s,u}$ = 9.2 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

$v_{Ed,crit} = \beta \cdot V_{Ed,red} / (u_{crit} \cdot d)$

$V_{Ed,red} = V_{Ed} - \Delta V_{Ed}$

$\Delta V_{Ed} = A_{crit} (\sigma_{Ed,gd,m} - g_{Ed,slab})$

$\beta = 1 + k \cdot M_{Ed} / V_{Ed} \cdot u_{crit} / W_{crit} \geq 1.10$

$W_{crit} = \int |e|\, dl$ 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)

$c_1/c_2 = 0.6/0.4 = 1.5 \Rightarrow k_x = 0.65$

calculated values of basic control perimeter

LK	a_{crit} cm	a/d -	u_{crit} m	A_{crit} m²	$W_{crit,x}$ m²
1	19.1	0.56	3.20	0.738	1.0808
2	19.1	0.56	3.20	0.738	1.0808
3	19.8	0.57	3.25	0.760	1.1100
4	19.3	0.56	3.21	0.744	1.0880

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	β -	$v_{Ed,crit}$ N/mm²
1	200.00	100.09	73.88	60.00	1.58	0.180
2	270.00	135.11	99.73	81.00	1.58	0.243
3	350.00	151.86	115.47	195.00	2.06	0.431
4	420.00	193.94	144.22	216.00	1.99	0.494

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]

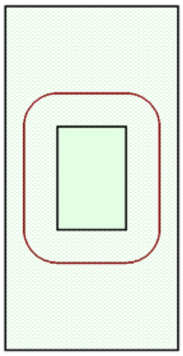
$C_{Rd,c} = 0.15/\gamma_C$

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



mean effective depth

$d_m = (35 + 34)/2 = 34.5$ cm

scale factor

$k = 1 + \mathrm{sqrt}(200/345) = 1.76 < 2$

longitudinal reinf. ratio of the anchored tension reinf.

mean of the tension reinforcement to the distance 3d from the column

$a_{s,x,3d} = 1.2/2 = 0.59$ cm²/m

$a_{s,y,3d} = 9.2/1 = 9.19$ cm²/m

$\rho_{lx,tension} = 0.59/35 \cdot 10^{-2} = 0.00017$

$\rho_{ly,tension} = 9.19/34 \cdot 10^{-2} = 0.0027$

$\rho_{l,tension} = \mathrm{sqrt}(0.00017 \cdot 0.0027) = 0.00068$

punching shear resistance without shear reinforcement

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

$\rho_{l,tension,max}$ = minimum from $(0.02,\ 0.5 \cdot 17/434.78) = 0.0195 > 0.0007$

$v_{min} \cdot 2 \cdot d/a = 0.0525/1.5 \cdot 1.76^{3/2} \cdot 30^{0.5} \cdot 2 \cdot 34.5/19.3 = 1.6$ N/mm²

$v_{Rd,c} = 0.1 \cdot 1.76 \cdot (100 \cdot 0.00068 \cdot 30)^{1/3} \cdot 2 \cdot 34.5/19.3 = 0.796$ N/mm² < 1.6 N/mm² $\Rightarrow v_{Rd,c} = 1.6$ N/mm²

0.494 N/mm² < 1.6 N/mm² ⇒ no additional reinforcement required

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

5.1. partial safety factors auf der actionsseite

acc. to [3] table A 2.1

5.2. partial safety factors auf der resistance side

acc. to [3] tables A 2.2 and A 2.3

5.3. design values overturning (EQU)

Die Mobilisierung des passive earth pressurees wird vernachlässigt.

5.3.1. factorization of load case combinations

LK	design situat.	factorization
1	BS-P	0.9· Lf1
2	BS-P	1.1· Lf1
3	BS-P	0.9· Lf1+1.5· Lf2
4	BS-P	1.1· Lf1+1.5· Lf2

5.3.2. column load

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	180.00	0.00	27.00	45.00	0.00
2	220.00	0.00	33.00	55.00	0.00
3	330.00	0.00	102.00	157.50	0.00
4	370.00	0.00	108.00	167.50	0.00

5.4. verification against overturning (EQU)

no destabilising loading ⇒ verification is not necessary.

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

LK	design situat.	factorization
1	BS-P	Lf1
2	BS-P	1.35· Lf1
3	BS-P	Lf1+1.5· Lf2
4	BS-P	1.35· Lf1+1.5· Lf2

5.5.2. column load

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	200.00	0.00	30.00	50.00	0.00
2	270.00	0.00	40.50	67.50	0.00
3	350.00	0.00	105.00	162.50	0.00
4	420.00	0.00	115.50	180.00	0.00

associated characteristic values

LK	$N_{St,k}$ kN	$H_{x,St,k}$ kN	$H_{y,St,k}$ kN	$M_{x,St,k}$ kNm	$M_{y,St,k}$ kNm
1	200.00	0.00	30.00	50.00	0.00
2	200.00	0.00	30.00	50.00	0.00
3	300.00	0.00	80.00	125.00	0.00
4	300.00	0.00	80.00	125.00	0.00

5.6. verification of safety against base failure

5.6.1. loading and substituting dimensions

LK	$N_{0,k}$ kN	$M_{0,x,k}$ kNm	$M_{0,y,k}$ kNm	a' m	b' m	$H_{a',k}$ kN	$H_{b',k}$ kN
1	234.08	61.82	0.00	1.47	1.00	28.98	0.00
2	234.08	61.82	0.00	1.47	1.00	28.98	0.00
3	334.08	156.82	0.00	1.06	1.00	78.98	0.00
4	334.08	156.82	0.00	1.06	1.00	78.98	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

LK	γ_1 kN/m³	φ_1 °	c_1 kN/m²	d_s m	γ_2 kN/m³	φ_2 °	c_2 kN/m²
1	20.00	35.00	---	1.90	19.04	35.00	---
2	20.00	35.00	---	1.90	19.04	35.00	---
3	20.00	35.00	---	1.90	19.04	35.00	---
4	20.00	35.00	---	1.90	19.04	35.00	---

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 [4]
shape factors ν_b,ν_d,ν_c acc. to [4], tab.2
load inclination factors i_b,i_d,i_c acc. to [4], tab.3

LK	N_{b0} -	N_{d0} -	N_{c0} -	ν_b -	ν_d -	ν_c -	i_b -	i_d -	i_c -
1	22.61	33.30	---	0.796	1.390	---	0.728	0.831	---
2	22.61	33.30	---	0.796	1.390	---	0.728	0.831	---
3	22.61	33.30	---	0.717	1.541	---	0.512	0.670	---
4	22.61	33.30	---	0.717	1.541	---	0.512	0.670	---

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 $\mu = N_d/R_{n,d}$

LK	$R_{n,k}$ kN	$\gamma_{R,v}$ -	$R_{n,d}$ kN	N_d kN	μ -
1	1272.19	1.40	908.71	234.08	0.26
2	1272.19	1.40	908.71	316.01	0.35
3	751.09	1.40	536.49	384.08	0.72
4	751.09	1.40	536.49	466.01	**0.87**

$\mu_{max} = 0.87 < 1.0$ ⇒ **design bearing capacity sufficient**

5.7. design values slippage (GEO-2)

the assumed mobilised passive earth pressure is $e_{phg,mob} = 1.00 \cdot e_{phg}$.

design values of applied loads see base failure.

5.8. verification of safety against sliding

slip resistance in case of consolidated soil $R_{t,k} = N_{0,k} \cdot \tan(\delta_s)$
design value of slip resistance $R_{t,d} = R_{t,k}/\gamma_{R,h}$
design value of mobilised passive earth pressure $E_{p,d} = E_{p,k,mob}/\gamma_{R,e}$
the degree of utilization is $\mu = (R_{t,d}+E_{p,d})/H_{Res,d}$

angle of base friction (for raue base area) $\delta_s = 35.0°$

LK	$N_{0,k}$ 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	234.08	163.90	1.10	1.40	149.00	1.46	30.00	0.20
2	234.08	163.90	1.10	1.40	149.00	1.46	40.50	0.27
3	334.08	233.93	1.10	1.40	212.66	1.46	105.00	0.49
4	334.08	233.93	1.10	1.40	212.66	1.46	115.50	**0.54**

$\mu_{max} = 0.54 < 1.0$ ⇒ **slip resistance sufficient**

6. External stability - verification of serviceability

6.1. design values limitation of gaping joint under permanent load

Die Mobilisierung des passive earth pressurees wird vernachlässigt.

6.1.1. factorization of load case combinations

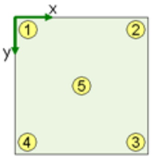
LK	factorization
1	Lf1

6.1.2. column load

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	200.00	0.00	30.00	50.00	0.00

6.1.3. 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	24.04	24.04	210.04	**210.04**	117.04

6.2. limitation of gaping joint under permanent load

internal forces and moments in centroid of foundation base: $N_{0,k}$ = 234.08 kN

$M_{0,x,k}$ = 62.00 kNm

$M_{0,y,k}$ = 0.00 kNm

resultant eccentricity: e_x = 0.00 m

e_y = 0.26 m

$e_x/b_x + e_y/b_y = 0.13 < 1/6$

⇒ 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

Die Mobilisierung des passive earth pressurees wird vernachlässigt.

6.3.1. factorization of load case combinations

LK	factorization
1	Lf1
2	Lf1+Lf2

6.3.2. column load

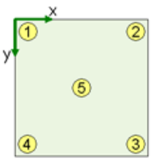
increasing factor for flex. mom.: $\Delta M_{St,TH.II.0} = M_{St} \cdot$ **-200%**

(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	200.00	0.00	30.00	50.00	0.00
2	300.00	0.00	80.00	125.00	0.00

6.3.3. 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	24.04	24.04	210.04	210.04	117.04
2	0.00	0.00	420.18	**420.18**	155.94

6.4. limitation of gaping joint under total load

LK	$N_{0,k}$ 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	234.08	62.00	0.00	0.00	0.26	0.018
2	334.08	157.00	0.00	0.00	0.47	0.055

$((e_x/b_x)^2 + (e_y/b_y)^2)_{max} = 0.055 < 1/9$

⇒ 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.).

LK	$N_{0,k}$ kN	$R_{t,k}$ kN	$\gamma_{R,h}$ -	$R_{t,d}$ kN	$H_{Res,d}$ kN	μ -
1	234.08	163.90	1.10	149.00	30.00	0.20
2	234.08	163.90	1.10	149.00	40.50	0.27
3	334.08	233.93	1.10	212.66	105.00	0.49
4	334.08	233.93	1.10	212.66	115.50	**0.54**

$\mu_{max} = 0.54 < 1.0$ ⇒ **verification against displacement in base area successful**

6.6. design values settlement

Die Mobilisierung des passive earth pressurees wird vernachlässigt.

6.6.1. factorization of load case combinations

LK	factorization
1	Lf1
2	Lf1+Lf2

6.6.2. column load

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	200.00	0.00	30.00	50.00	0.00
2	300.00	0.00	80.00	125.00	0.00

6.7. settlements

determination of settlement by use of closed formulas acc. to [5]

allowable maximum settlement perm $s_{max} = 5.0$ cm
allowable obliquity about the x-axis perm $\alpha_x = 0.5$ °
allowable obliquity about the y-axis perm $\alpha_y = 0.5$ °

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 $\sigma_a = 16.00$ kN/m²

LK	$N_{0,k}$ kN	$M_{0,x,k}$ kNm	$M_{0,y,k}$ kNm	σ_0 kN/m²	σ_0' kN/m²	d_s m
1	234.08	62.00	0.00	117.04	101.04	2.41
2	334.08	157.00	0.00	167.04	151.04	2.95

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

coefficient f for settlement below significant point acc. to [6], vol. 2, tab. 4
coefficients f_x/f_y for obliquity of a rigid foundation acc. to [7], fig. 19
settlement parts from centrical load $s_{m,i} = \sigma_0'\cdot b_y\cdot(f_i - f_{i-1})/E_{m,i}$
settlement parts from $M_{0,y}$ $s_{x,i} = b_x/2 \cdot M_{0,y}/(E_{m,i}\cdot b_y\cdot b_x^2)\cdot(f_{x,i} - f_{x,i-1})$
settlement parts from $M_{0,x}$ $s_{y,i} = b_y/2 \cdot M_{0,x}/(E_{m,i}\cdot b_x\cdot b_y^2)\cdot(f_{y,i} - f_{y,i-1})$

LK 1:
$\sigma_0' = 101.04$ kN/m²
$M_{0,x} = 62.00$ kNm
$M_{0,y} = 0.00$ kNm

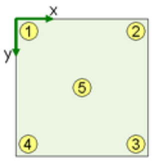
level m	z m	f -	f_x -	f_y -	s_m cm	s_x cm	s_y cm
2.50	1.70	0.366	4.296	2.930	0.09	0.00	0.06
3.21	2.41	0.418	4.408	3.075	0.01	0.00	0.00

LK 2:
$\sigma_0' = 151.04$ kN/m²
$M_{0,x} = 157.00$ kNm
$M_{0,y} = 0.00$ kNm

level m	z m	f -	f_x -	f_y -	s_m cm	s_x cm	s_y cm
2.50	1.70	0.366	4.296	2.930	0.14	0.00	0.14
3.75	2.95	0.444	4.409	3.076	0.03	0.00	0.01

6.7.3. resultant settlements and obliquity per LK

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



LK	s_1 cm	s_2 cm	s_3 cm	s_4 cm	s_5 cm	s_{max} cm	α_x °	α_y °
1	0.0	0.0	0.2	0.2	0.1	0.2	0.0	**0.0**
2	0.0	0.0	0.3	0.3	0.2	**0.3**	**0.1**	0.0

max $s_{max} = 0.3 < 5.0$ cm max $|\alpha_x| = 0.1° < 0.5°$ max $|\alpha_y| = 0.0° < 0.5°$

⇒ allowable settlement and obliquity kept

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

$\sigma_ü$ - 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. summary

° the allowable tension force of one or more anchors is exceeded.

desing calculation not successful!

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