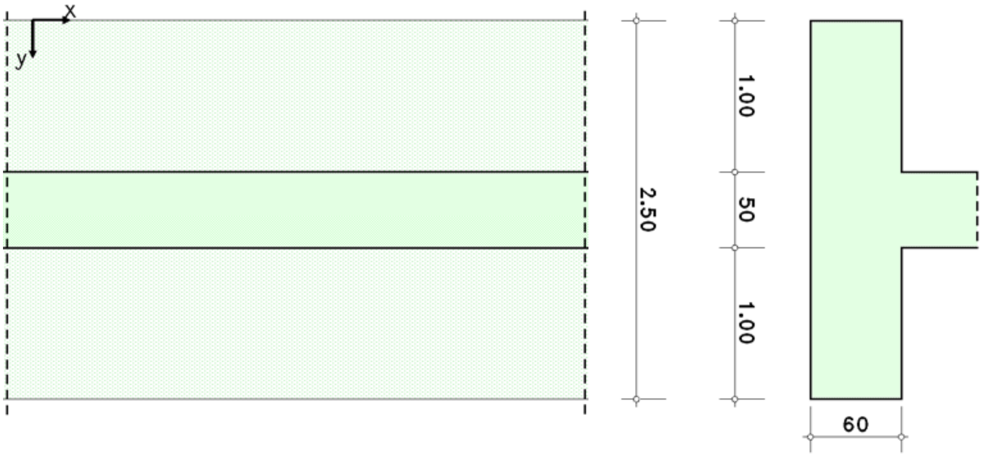
4H-FUND version: 12/2022-1a

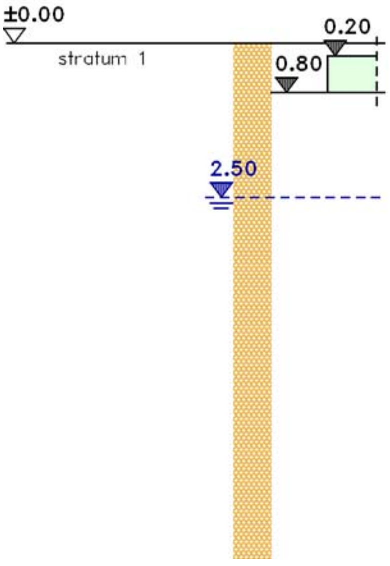
strip foundation

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

scale 1:50



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.

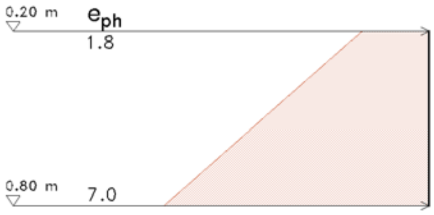
1.1. designation and characteristic values of soil strata

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	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

1.2. char. passive earth pressure

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



$\Sigma(\gamma \cdot h)$ Summe soilgewicht in der betrachteten Tiefe
$\Sigma(\gamma \cdot h)_{cal}$ 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	$\Sigma(\gamma \cdot h)$ kN/m²	$\Sigma(\gamma \cdot h)_{cal}$ kN/m²	K_{0gh} -	e_{0h} kN/m²
0.20	4.40	7.70	0.398	1.75
0.80	17.60	17.60	0.398	7.01

the resultant maximum passive earth pressure is E_{0hg} = 2.63 kN/m, at z_S = 0.56 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

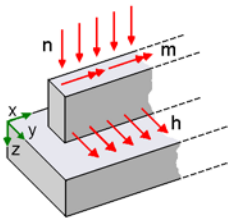
2.2. Design calculation situation of load cases for external stability

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

2.3. characteristic wall load

point of application in centroid of wall auf top edge foundation slab

loadc.	nw kN/m	hw kN/m	mw kNm/m
1	650.00	100.00	150.00
2	175.00	30.00	50.00



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 = 0.20 m.
the mean unit weight of the earth load is γ_A = 22.00 kN/m³.

the resultant of dead load in the floor joint is $n_{0,Eigen,k}$ = 46.30 kN/m.

Das dead load wird im load case 1 with berücksichtigt.

3. design calculation of foundation slab

3.1. partial safety factors for material

design situat.	γ_C	γ_S
permanent and transient	1.50	1.15

3.2. design values of reinforced concrete design

Die Mobilisierung des passive earth pressurees wird vernachlässigt.

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. wall load

increasing factor for flex. mom.: $\Delta M_{St,TH.II.O} = M_{St} \cdot 50\%$
(for the consideration of increase of moments from non-linear effects)

LK	$n_{w,d}$ kN/m	$h_{w,d}$ kN/m	$m_{w,d}$ kNm/m
1	650.00	100.00	225.00
2	877.50	135.00	303.75
3	912.50	145.00	337.50
4	1140.00	180.00	416.25

3.3. base pressure

determination of base pressures assuming linear soil stresses and elimination of tension
minimum and maximum stresses: σ_{min}, σ_{max}, stress in plate centroid: σ_{SP}

LK	σ_{min} kN/m²	σ_{max} kN/m²	σ_{SP} kN/m²
1	4.92	552.12	278.52
2	6.64	745.36	376.00
3	0.00	791.82	383.12
4	0.00	**984.81**	480.73

3.4. Design calculation for bending

3.4.1. longitudinal reinforcement in y-direction (perpendicular to the wall)

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

moments in design calculation sections

LK	y = 100.0 cm kNm/m	y = 150.0 cm kNm/m
1	29.24	229.88
2	39.47	310.34
3	32.98	331.71
4	43.10	**412.10**

Design calculation for LK 4: $\varepsilon_o/\varepsilon_u$ = -3.11/28.12‰ **min $a_{s,u}$ = 17.51 cm²/m**

3.4.2. longitudinal reinforcement in x-direction (parallel to the wall)

acc. to [2], clause 9.2.1.1(2) a minimum transverse reinforcement of 20% is required in uniaxial spanned slabs.

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

min $a_{s,u,x} = 0.2 \cdot \min a_{s,u,y}$ = 3.50 cm²/m

3.4.3. selected reinforcement in x-direction

bottom **B500A,** laid parallel to the wall
Ø 12 / 32.0 cm = 3.53 = 3.50 cm²/m

3.4.4. selected reinforcement in y-direction

bottom **B500A,** laid perpendicular to the wall
Ø 12 / 6.0 cm = 18.85 > 17.51 cm²/m

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

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

4.1. partial safety factors auf der actionsseite

acc. to [3] table A 2.1

4.2. partial safety factors auf der resistance side

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

4.3. design values overturning (EQU)

Die Mobilisierung des passive earth pressurees wird vernachlässigt.

4.3.1. 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

4.3.2. wall load

LK	$n_{w,d}$ kN/m	$h_{w,d}$ kN/m	$m_{w,d}$ kNm/m
1	585.00	90.00	135.00
2	715.00	110.00	165.00
3	847.50	135.00	210.00
4	977.50	155.00	240.00

4.4. verification against overturning (EQU)

the degree of utilization is $\mu = m_{dst}/m_{stb}$

no destabilising loading ⇒ verification is not necessary.

4.5. design values base failure (GEO-2)

the assumed mobilised passive earth pressure is $e_{phg,mob} = 0.50 \cdot e_{phg}$.

4.5.1. 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

4.5.2. wall load

LK	$n_{w,d}$ kN/m	$h_{w,d}$ kN/m	$m_{w,d}$ kNm/m
1	650.00	100.00	150.00
2	877.50	135.00	202.50
3	912.50	145.00	225.00
4	1140.00	180.00	277.50

associated characteristic values

LK	$n_{w,k}$ kN/m	$h_{w,k}$ kN/m	$m_{w,k}$ kNm/m
1	650.00	100.00	150.00
2	650.00	100.00	150.00
3	825.00	130.00	200.00
4	825.00	130.00	200.00

4.6. verification of safety against base failure

4.6.1. loading and substituting dimensions

LK	$n_{0,k}$ kN/m	$m_{0,k}$ kNm/m	b' m	$h_{,k}$ kN/m
1	696.30	209.68	1.90	98.69
2	696.30	209.68	1.90	98.69
3	871.30	277.68	1.86	128.69
4	871.30	277.68	1.86	128.69

4.6.2. 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	22.00	37.00	---	3.02	18.06	37.00	---
2	22.00	37.00	---	3.02	18.06	37.00	---
3	22.00	37.00	---	2.93	18.22	37.00	---
4	22.00	37.00	---	2.93	18.22	37.00	---

4.6.3. 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	31.59	42.92	---	1.000	1.000	---	0.632	0.737	---
2	31.59	42.92	---	1.000	1.000	---	0.632	0.737	---
3	31.59	42.92	---	1.000	1.000	---	0.619	0.726	---
4	31.59	42.92	---	1.000	1.000	---	0.619	0.726	---

4.6.4. ultimate load and allowable load

characteristic design bearing capacity $R_{n,k} = b' \cdot (\gamma_2 \cdot b' \cdot N_{b0} \cdot \nu_b \cdot i_b + \gamma_1 \cdot t \cdot N_{d0} \cdot \nu_d \cdot i_d + c_2 \cdot N_{c0} \cdot \nu_c \cdot i_c)$

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/m	$\gamma_{R,v}$ -	$R_{n,d}$ kN/m	N_d kN/m	μ -
1	2355.23	1.40	1682.31	696.30	0.41
2	2355.23	1.40	1682.31	940.01	0.56
3	2258.47	1.40	1613.19	958.80	0.59
4	2258.47	1.40	1613.19	1202.51	**0.75**

μ_{max} = 0.75 < 1.0 ⇒ **design bearing capacity sufficient**

4.7. 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.

4.8. 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 $\mu = (R_{t,d}+e_{p,d})/h_d$

angle of base friction (for raue base area) δ_S = 37.0 > 35 ⇒ δ_S = 35°

LK	$n_{0,k}$ kN/m	$R_{t,k}$ kN	$\gamma_{R,h}$ -	$\gamma_{R,e}$ -	$R_{t,d}$ kN/m	$e_{p,d}$ kN/m	h_d kN/m	μ -
1	696.30	487.55	1.10	1.40	443.23	1.88	100.00	0.22
2	696.30	487.55	1.10	1.40	443.23	1.88	135.00	0.30
3	871.30	610.09	1.10	1.40	554.63	1.88	145.00	0.26
4	871.30	610.09	1.10	1.40	554.63	1.88	180.00	**0.32**

μ_{max} = 0.32 < 1.0 ⇒ **slip resistance sufficient**

5. External stability - verification of serviceability

5.1. design values limitation of gaping joint under permanent load

Die Mobilisierung des passive earth pressurees wird vernachlässigt.

5.1.1. factorization of load case combinations

LK	factorization
1	Lf1

5.1.2. wall load

LK	$n_{w,d}$ kN/m	$h_{w,d}$ kN/m	$m_{w,d}$ kNm/m
1	650.00	100.00	150.00

5.1.3. base pressure

determination of base pressures assuming linear soil stresses and elimination of tension
minimum and maximum stresses: σ_{min}, σ_{max}, stress in plate centroid: σ_{SP}

LK	σ_{min} kN/m²	σ_{max} kN/m²	σ_{SP} kN/m²
1	76.92	**480.12**	278.52

5.2. limitation of gaping joint under permanent load

internal forces and moments in centroid of foundation base: $n_{0,k}$ = 696.30 kN/m
$m_{0,k}$ = 210.00 kNm/m

resultant eccentricity: e = 0.30 m

e/b = 0.12 < 1/6

⇒ **the resultant is located in the 1. core area**
sc. no emerge of a gaping joint due to permanent load.

5.3. design values limitation of gaping joint under total load

Die Mobilisierung des passive earth pressurees wird vernachlässigt.

5.3.1. factorization of load case combinations

LK	factorization
1	Lf1
2	Lf1+Lf2

5.3.2. wall load

increasing factor for flex. mom.: $\Delta M_{St,TH.II.O} = M_{St} \cdot$ **50%**
(for the consideration of increase of moments from non-linear effects)

LK	$n_{w,d}$ kN/m	$h_{w,d}$ kN/m	$m_{w,d}$ kNm/m
1	650.00	100.00	150.00
2	825.00	130.00	200.00

5.3.3. base pressure

determination of base pressures assuming linear soil stresses and elimination of tension
minimum and maximum stresses: σ_{min}, σ_{max}, stress in plate centroid: σ_{SP}

LK	σ_{min} kN/m²	σ_{max} kN/m²	σ_{SP} kN/m²
1	76.92	480.12	278.52
2	81.64	**615.40**	348.52

5.4. limitation of gaping joint under total load

LK	$n_{0,k}$ kN/m	$m_{0,k}$ kNm/m	e m	e/b -
1	696.30	210.00	0.30	0.121
2	871.30	278.00	0.32	0.128

$(e/b)_{max} = 0.128 < 1/3$

⇒ the decisive resultant is located in the 2. core area, sc. no gaping joint beyond centroid.

5.5. 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/m	$R_{t,k}$ kN	$\gamma_{R,h}$ -	$R_{t,d}$ kN/m	h_d kN/m	μ -
1	696.30	487.55	1.10	443.23	100.00	0.23
2	696.30	487.55	1.10	443.23	135.00	0.30
3	871.30	610.09	1.10	554.63	145.00	0.26
4	871.30	610.09	1.10	554.63	180.00	**0.32**

$\mu_{max} = 0.32 < 1.0$ **⇒ verification against displacement in base area successful**

5.6. design values settlement

Die Mobilisierung des passive earth pressurees wird vernachlässigt.

5.6.1. factorization of load case combinations

LK	factorization
1	Lf1
2	Lf1+Lf2

5.6.2. wall load

LK	$n_{w,d}$ kN/m	$h_{w,d}$ kN/m	$m_{w,d}$ kNm/m
1	650.00	100.00	150.00
2	825.00	130.00	200.00

5.7. settlements

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

allowable maximum settlement perm $s_{max} = 5.0$ cm
allowable obliquity perm $\alpha = 0.5$ °

5.7.1. 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 $\sigma_a = 17.60$ kN/m²

LK	$n_{0,k}$ kN/m	$m_{0,k}$ kNm/m	σ_0 kN/m²	σ_0' kN/m²	d_s m
1	696.30	210.00	278.52	260.92	11.30
2	871.30	278.00	348.52	330.92	12.91

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

coefficient f for settlement below significant point acc. to [6], vol. 2, tab. 4
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 $s_{r,i} = b/2 \cdot m_0/(E_{m,i} \cdot b^2) \cdot 12/\pi$

LK 1:
$\sigma_0' = 260.92$ kN/m²
$m_0 = 210.00$ kNm/m

level m	z m	f -	s_m cm	s_r cm
2.50	1.70	0.508	0.41	0.20
12.10	11.30	1.531	0.83	0.20

LK 2:
$\sigma_0' = 330.92$ kN/m^2
$m_0 = 278.00$ kNm/m

level	z	f	s_m	s_r
m	m	-	cm	cm
2.50	1.70	0.508	0.53	0.27
13.71	12.91	1.614	1.14	0.27

5.7.3. resultant settlements and obliquity per LK

$s_1 = \Sigma(s_{m,i} - s_{r,i})$ $s_2 = \Sigma(s_{m,i} + s_{r,i})$ $s_3 = \Sigma s_{m,i}$
tan $\alpha = 2 \cdot \Sigma s_{y,i}/b_y$

LK	s_1	s_2	s_3	s_{max}	α
	cm	cm	cm	cm	°
1	0.8	1.6	1.2	1.6	0.2
2	1.1	2.2	1.7	**2.2**	**0.2**

max $s_{max} = 2.2 < 5.0$ cm max $|\alpha| = 0.2° < 0.5°$

⇒ allowable settlement and obliquity kept

n_0 - normal force in foundation joint m_0 - moment load in centroid of foundation joint b' - substituting width due eccentric load
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

6. 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 [7]
$k_s = E_s / (f \cdot (b_x \cdot b_x)^{0.5})$
with Formfaktor f abhängig vom aspect ratio: 1:1 -> f = 0.45, 1:2 -> f 0.42, 1:4 -> f = 0.35

assumption for Korrekturfaktor $\kappa = 1$
stiffenerziffer $E_s = 1 \cdot 80000.00 = 80000.00$ kN/m^2

Formfaktor f = 0.25

beddingsmodul $k_s = 202385.77$ kN/m^3

Trägheitsmoment I_X / I_X = 0.21 / 1.30 kN/m^3

Drehfeder about the x-axis **$c_{v,x}$ = 42163.70 kNm**
Drehfeder about the y-axis **$c_{v,y}$ = 263523.14 kNm**

7. summary

all executed verifications and design calculations successful.

longitudinal reinforcement x-direction
min $A_{s,x}$
= 3.5 cm^2
longitudinal reinforcement y-direction
min $A_{s,y}$
= 17.5 cm^2

overturning
μ_{max}
= 0.00
base failure
μ_{max}
= 0.75
slip
μ_{max}
= 0.32

Klaffende clearance unter perman.er load
μ_{exis}
= 0.72
Klaffende clearance unter total load
μ_{max}
= 0.38
displacement in der base area
μ_{max}
= 0.32

settlement
s_{max}
= 2.2 cm
obliquity
$\alpha_{max,x}$
= 0.2°

Drehfeder about the x-axis
$c_{v,X}$
= 42163.70 kNm

Literatur and Normen:

[1] DIN 4085: soil, calculation des Erddrucks, August 2017
[2] DIN EN 1992-1-1: Eurocode 2: Bemessung und Konstruktion von Stahlbeton- und Spannbetontragwerken, Teil 1-1, Januar 2011
[3] DIN 1054: soil - safetysnachweise im Erd- and Grundbau - Ergänzende Regeln zu DIN EN 1997-1, April 2021
[4] DIN 4017: Baugrund, Berechnung des Grundbruchwiderstandes von Flächengründungen, März 2006
[5] DIN 4019: soil - settlementsberechnungen, Januar 2014
[6] Kany, M.: Berechnung von Flächengründungen, Verlag von Wilhelm Ernst & Sohn, 2.Aufl. 1974
[7] Rausch, E.: Maschinenfundamente, concretekalender 1973, part 2, Ernst & Sohn