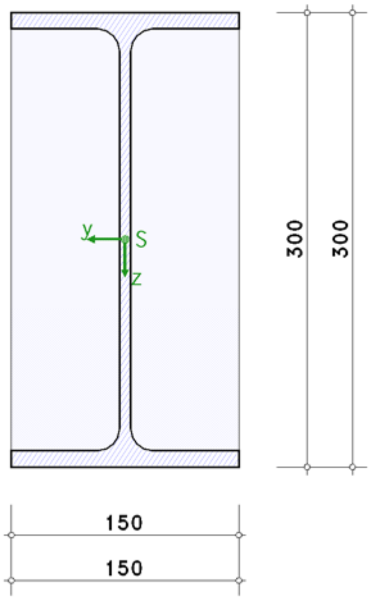
POS. 15: 8.2, COLUMN IPE300 - S355

4H-EC3KF version: 4/2024-1d

clamped steel support foot

steel code verifications acc. to DIN EN 1993-1:2010-12 with NA-Germany

cross-section, scale 1:5



column cross section
standardized profile: IPE300, of quality S355

base plate
b = 150 mm h = 300 mm t = 10 mm, of quality S355

mortar joint under base plate
h_f = 40 mm

foundation
concrete quality C25/30
height = 100.0 cm
splitting tensile reinforcement is provided

1. loading

1.1. design values of column load

point of application in column centroid

LK	notation	design situat.	$N_{St,d}$ kN	$M_{y,St,Ed}$ kNm	$H_{z,St,Ed}$ kN	$M_{z,St,Ed}$ kNm	$H_{y,St,Ed}$ kN
1	new design load c.	perman. a.v.	98.00	164.00	63.00	0.00	0.00

2. verification

2.1. partial safety factors for material

design situat.	γ_{M0}	γ_{M2}	γ_c
perman.	1.00	1.25	1.50

2.2. clamping depth

determination of the required clamping depth acc. to [1]

2.2.1. required clamping depth for bending around the y-axis

coefficient of the contributing width α_m = 1.23
contributing width b_m = 150.0 mm
resulting pressure p = 21.25 kN/cm
red. plastic shear force red $V_{pl,z}$ = 485.69 kN

required clamping depth

LK	D_o kN	D_u kN	$D_u/V_{pl,z}$ -	f_{erf} cm
1	548.69	485.69	1.00	**60.5**

D_o/D_u - res. compressive force top/bottom f_{erf} - req. clamping depth

maximum required clamping depth for bending around the y-axis $f_{erf,y}$ = 60.5 cm

2.2.2. set clamping depth

required f_{erf} = 60.5 cm (from LK 1, Bieg. um y-Achse)

minimum value $f_{min} = 1.5 \cdot 30.00$ = 45.0 < 60.5 cm
maximum value $f_{max} = 4.0 \cdot 30.00$ = 120.0 > 60.5 cm

chosen f_{gew} = **61.0** > 60.5 cm

2.3. resistance of cross section

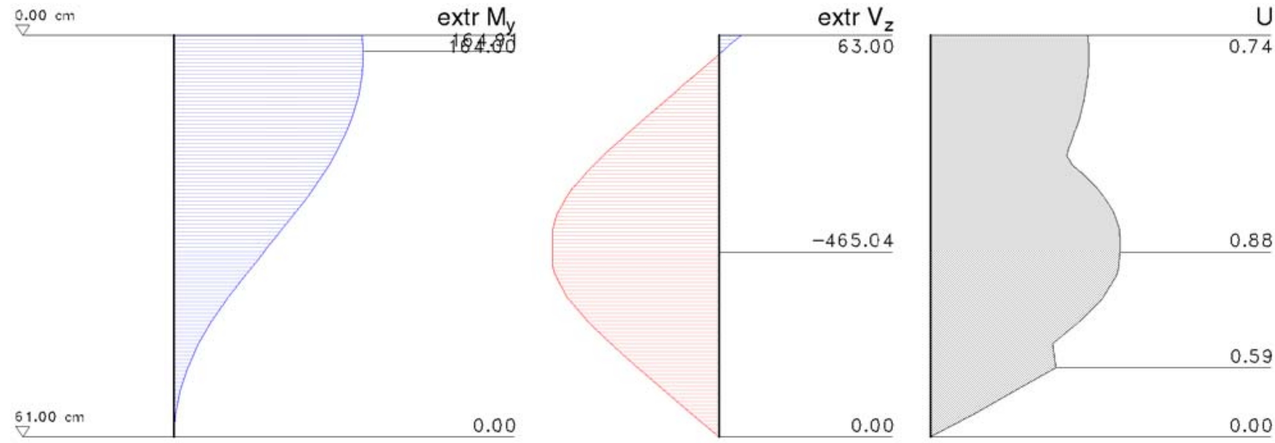
plastic stress analysis is carried out acc. to [2], Abs. 6.2.2 to 6.2.10.

2.3.1. supporting forces

LK	M_y/V_z a_o cm	a_u cm	D_o kN	D_u kN
1	30.7	27.1	528.65	465.65

a_o/a_u - pressure area top/bottom D_o/D_u - res. compressive force top/bottom

2.3.2. extreme internal forces and moments



extreme values of axial force: N_{Min} / N_{Max} = 98.00 / 98.00 kN

x	extr M_y Min	Max	extr V_z Min	Max	U
cm	kNm	kNm	kNm	kNm	
0.00	164.00	164.00	63.00	**63.00**	0.74
2.46	164.91	**164.91**	10.79	10.79	0.74
3.69	164.88	164.88	-15.32	-15.32	0.74
18.43	139.58	139.58	-325.15	-325.15	0.63
30.71	88.63	88.63	**-465.04**	-465.04	**0.88**
46.94	21.20	21.20	-299.84	-299.84	0.57
50.45	11.96	11.96	-226.07	-226.07	0.59*
61.00	**0.00**	0.00	0.00	0.00	0.00*

* verification carried out elastically

maximum utilization U = 0.88 < 1.00

from load spectrum 1 at the location x = 30.71 cm

internal forces and moments: N = 98.00 kN, V_z/M_y = -465.04/88.63 kNm

utilization: U_σ = 0.88

2.4. weld between column and base plate

design with direction oriented method acc. to clause 4.5.3.2

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

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

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

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

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

connection designed with a **full-size double fillet weld** (no end craters).
axial force transfer of 100 % by the weld.

minimum value of the weld thickness a_{min} = 3 mm

LK	$a_{w,Fl}$	$a_{w,S}$	$\sigma_\perp$	$\tau_\perp$	τ_{II}	$\sigma_{1,w,Ed}$	$f_{1,w,Rd}$	$\sigma_{2,w,Ed}$	$f_{2,w,Rd}$	U
	mm	mm	N/mm²	N/mm²	N/mm²	N/mm²	N/mm²	N/mm²	N/mm²	-
1	3	3	-22.58	-22.58	0.00	45.16	435.56	22.58	352.80	**0.10**

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

maximum weld thickness flange $a_{w,Fl,max}$ = 3 mm
maximum weld thickness of the web $a_{w,S,max}$ = 3 mm
maximum utilization U = 0.10 < 1.00

2.5. introduction of the normal force into the foundation

verification acc. to [4], parag. 6.2.5 and load-bearing capacity of the subareas acc. to [3], parag.6.7

2.5.1. requirements for the mortar under the base plate

0.2-fold of the smallest panel dimension = 30.0 < 40 mm mortar height

⇒ the characteristic strength of the mortar should be greater than 20% of that of the foundation concrete. alternatively, the connection coefficient is to be set at $\beta_j < 2/3$.

2.5.2. load spreading

$c = t \cdot [f_y / 3 \cdot f_{jd} \cdot \gamma_{M0})]^{0.5} \leq 0.5 \cdot (h - 2 \cdot t)$

an undisturbed load propagation is assumed.

spreading width	c	= 27.2 mm
loading area	A_{c0}	= 251.84 cm²
distribution area	A_{c1}	= 717.99 cm²

2.5.3. design resistance

$F_{C,Rdr} = f_{jd} \cdot A_{c0}$

$f_{jd} = \beta_j \cdot F_{Rdu} / A_{c0}$

$F_{Rdu} = A_{c0} \cdot f_{cd} \cdot (A_{c1} / A_{c0})^{0.5} \leq 3.0 \cdot f_{cd} \cdot A_{c0}$

joint coefficient	β_j	= 2/3
design value of the mortar strength	f_{jd}	= 15.95 N/mm²
load-bearing capacity under pressure	$F_{C,Rd}$	= 401.60 kN

2.5.4. utilization

$U = N_{Ed} / F_{C,Rd}$

maximum compressive force (LK 1) N_{Ed} = 98.00 < 401.60 kN

utilization U = 0.24 < 1.00

3. summary

all executed verifications and design calculations successful.

required clamping depth of the column cross section	f_{erf}	= 60.5 cm
chosen clamping depth	f_{gew}	= 61.0 > 60.5 cm
load-bearing cap. column cross-section	μ_{max}	= 0.88
weld between column and base plate	μ_{max}	= 0.10
introd. of normal force	μ_{max}	= 0.24

literature and standards:

[1] Kindmann, Kraus, Laumann, Vette: Verallgem. Berech.methode für in Beton eingesp. Stahlprofile, Stahlbau 92, Heft 1, Ernst & Sohn, 2023

[2] DIN EN 1993-1-1: Eurocode 3: Bem. und Konstr. von Stahlbauten - Teil 1-1: Allg. Bem.regeln u. Regeln für den Hochbau, Dez. 2010

[3] DIN EN 1992-1-1: Eurocode 2: Bemessung und Konstruktion von Stahlbeton- und Spannbetontragwerken, Teil 1-1, Januar 2011

[4] DIN EN 1993-1-8: Eurocode 3: Bemessung und Konstruktion von Stahlbauten - Teil 1-8: Bemessung von Anschlüssen, Dez. 2010