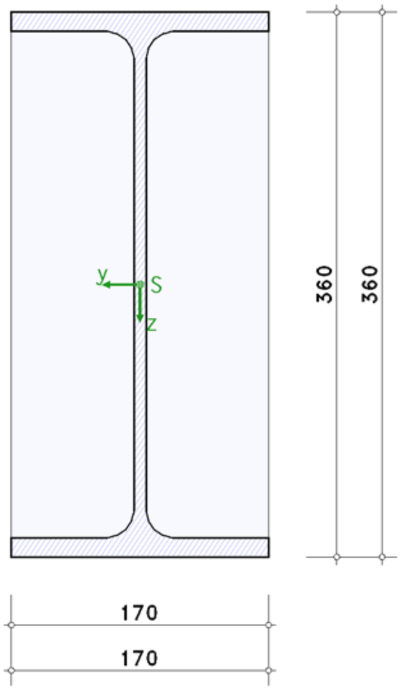
POS. 14: 8.2, COLUMN IPE360 - S235

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 with clamping depth f = 60.0 cm
standardized profile: IPE360, of quality S235

base plate
b = 170 mm h = 360 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	= 1.00
contributing width	b_m	= 170.0 mm
resulting pressure	p	= 24.08 kN/cm
red. plastic shear force	red $V_{pl,z}$	= 438.82 kN

required clamping depth

LK	D_o kN	D_u kN	$D_u/V_{pl,z}$ -	f_{erf} cm
1	501.82	438.82	1.00	**59.0**

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

maximum required clamping depth for bending around the y-axis $f_{erf,y}$ = 59.0 cm

2.2.2. set clamping depth

required	f_{erf}	= 59.0 cm	(from LK 1, Bieg. um y-Achse)
minimum value	$f_{min} = 1.5 \cdot 36.00$	= 54.0 < 59.0 cm	
maximum value	$f_{max} = 4.0 \cdot 36.00$	= 144.0 > 59.0 cm	
chosen	f_{gew}	= **60.0** > 59.0 cm	

2.3. resistance of cross section

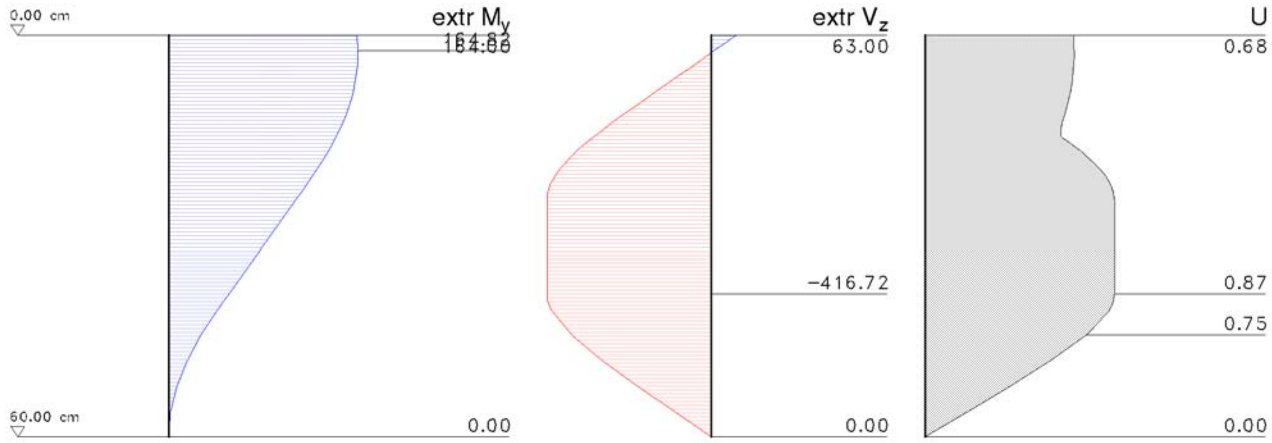
plastic stress analysis is carried out acc. to [2], Abs. 6.2.2 to 6.2.10.

2.3.1. supporting forces

LK	My/Vz			
	a_o	a_u	D_o	D_u
	cm	cm	kN	kN
1	24.6	21.4	480.39	417.39

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		extr V_z		U
	Min	Max	Min	Max	
cm	kNm	kNm	kNm	kNm	
0.00	164.00	164.00	63.00	**63.00**	0.68
2.35	164.82	**164.82**	6.52	6.52	0.69
3.52	164.73	164.73	-21.73	-21.73	0.69
24.63	110.52	110.52	**-416.72**	-416.72	**0.87**
60.00	**0.00**	0.00	0.00	0.00	0.00

maximum utilization U = 0.87 < 1.00

from load spectrum 1 at the location x = 24.63 cm

internal forces and moments: N = 98.00 kN, V_z/M_y = -416.72/110.52 kNm

utilization: U_σ = 0.87

2.4. weld between column and base plate

design with direction oriented method acc. to clause 4.5.3.2

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

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

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

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

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

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

minimum value of the flange weld thickness $a_{w,Fl,min}$ = 4 mm
minimum value der web weld thickness $a_{w,S,min}$ = 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	**4**	**3**	-16.66	-16.66	0.00	33.32	360.00	16.66	259.20	**0.09**

$a_{w,Fl}$ - 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 $a_{w,Fl,max}$ = 4 mm
maximum weld thickness of the web $a_{w,S,max}$ = 3 mm

maximum utilization U = 0.09 < 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 = 34.0 < 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 β_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	= 22.2 mm
loading area	A_{c0}	= 270.45 cm²
distribution area	A_{c1}	= 770.70 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}	= 15.94 N/mm²
load-bearing capacity under pressure	$F_{C,Rd}$	= 431.18 kN

2.5.4. utilization

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

maximum compressive force (LK 1) N_{Ed} = 98.00 < 431.18 kN

utilization U = 0.23 < 1.00

3. summary

all executed verifications and design calculations successful.

chosen clamping depth of the column cross-sect.	f_{gew}	= 60.0 cm
required clamping depth	f_{erf}	= 59.0 < 60.0 cm
load-bearing cap. column cross-section	μ_{max}	= 0.87
weld between column and base plate	μ_{max}	= 0.09
introd. of normal force	μ_{max}	= 0.23

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