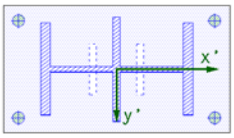
POS. 3: HE-500B+ => 4M36-5.6

4H-EC3FP version: 5/2013-1a

steel column base with base plate

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

top view base plate
scale 1:25



column cross section
user defined profile: Q 132, of quality S235
rotated by 90.0°

base plate
b_x = 750 mm b_y = 420 mm t = 35 mm, of quality S235

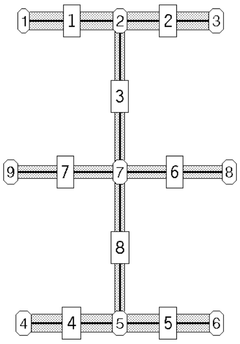
mortar joint
t_F = 30 mm

foundation/bedding
acc. to concrete C25/30

shear connector
standardized profile: HE160M, of quality S235
(rotated by 90°)

anchors
4 anchors, FK 5.6, M22
with a length of 600 mm
edge distances a_x/a_y = 50/50 mm

description of column profile cross-section (Q 132)



node coordinates

Nr.	x'	y'
-	mm	mm
1	236.0	-150.0
2	236.0	-0.0
3	236.0	150.0
4	-236.0	-150.0
5	-236.0	0.0
6	-236.0	150.0
7	0.0	0.0
8	0.0	170.0
9	-0.0	-170.0

line elements

Nr.	nodA	nodE	thickness
-	-	-	mm
1	1	2	28.0
2	3	2	28.0
3	2	7	14.5
4	4	5	28.0
5	6	5	28.0
6	8	7	20.0
7	7	9	20.0
8	7	5	14.5

1. loading

1.1. design values of column load

point of application in column centroid

LK	$N_{St,d}$	$H_{x,St,d}$	$H_{y,St,d}$	$M_{x,St,d}$	$M_{y,St,d}$	design situat.
	kN	kN	kN	kNm	kNm	
1	2557.00	38.30	4.20	0.00	0.00	perman.
2	-177.00	1.50	-157.60	0.00	0.00	perman.
3	1950.00	-70.60	-3.20	0.00	0.00	perman.
4	-174.00	-4.00	-172.10	0.00	0.00	perman.

2. verification

2.1. partial safety factors for material

design situat.	γ_{M0}	γ_{M2}	γ_c
perman.	1.10	1.10	1.50

2.2. weld between column shaft and base plate

design with simplified method acc. to clause 4.5.3.3

$F_{w,Ed} = \sigma_{w,v} \cdot a_w$
$F_{w,Rd} = f_{vw,d} \cdot a_w$
$f_{vw,d} = (f_u/3^{0.5})/(\beta_w \cdot \gamma_{m2})$
$U = F_{w,Ed}/F_{w,Rd}$

connection designed with a **circumferential fillet weld.**
axial force transfer of 100 % by the weld.

LK	a_w mm	$\sigma_{w,max}$ kN/cm²	$\tau_{w,max}$ kN/cm²	$\sigma_{w,v,max}$ kN/cm²	$F_{w,Ed}$ kN/cm	$F_{w,Rd}$ kN/cm	U -
1	**6**	19.75	-0.34	19.75	11.85	14.17	**0.84**
2	6	-3.55	2.16	3.56	2.13	14.17	0.15
3	6	15.18	-0.56	15.18	9.11	14.17	0.64
4	6	-3.74	2.38	3.75	2.25	14.17	0.16

maximum weld thickness $a_{w,max}$ = 6 mm

maximum utilization U = 0.84 < 1.00

a_w - weld thickness $\sigma_{w,max}$ - max. normal stress along the weld $\tau_{w,max}$ - max. shear stress along the weld
$\sigma_{w,v,max}$ - max. equivalent stress along the weld $F_{w,Ed}$ - effective force in the weld per unit of length
$F_{w,Rd}$ - design resistance of the weld per unit of length   U - utilization

2.3. 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 20 elements in X-direction and 24 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 = 1060.50 kN/cm.

2.3.1. 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	65.6	20.1	11.17	5.67	0.01	-4.22	0.02
2	9.4	6.1	-15.64	-5.61	-4.01	-6.34	-1.63
3	9.4	21.9	8.60	4.36	0.01	3.25	-0.02
4	65.6	6.1	-15.93	-5.72	4.09	6.47	-1.66

stresses and utilizations

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

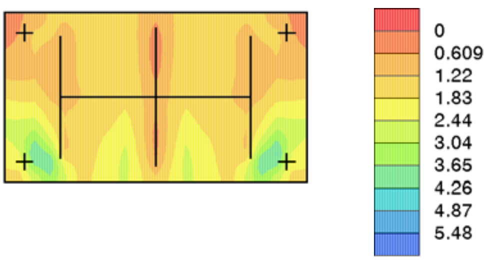
LK	x_{Fp} cm	y_{Fp} cm	$\sigma_{Pl,V}$ kN/cm²	σ_{Rd} kN/cm²	U -
1	65.6	20.1	3.79	21.36	0.18
2	9.4	6.1	5.98	21.36	0.28
3	9.4	21.9	2.92	21.36	0.14
4	65.6	6.1	6.09	21.36	**0.29**

maximum utilization U = 0.29 < 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 $\sigma_{Pl,V}$)



2.3.2. concrete compression under base plate

The permitted share of compression area with concrete compressions greater than the design value of concrete compressive strength (fcd) is 30%.

LK	lim $\sigma_{c,d}$ kN/cm²	$A_{compr.}$ cm²	$\sigma_{c,max}$ kN/cm²	$\sigma_{c,m}$ kN/cm²	f_{cd} kN/cm²	U -	$\sigma_c(A_D)>f_{cd}$ %
1	4.25	3150.0	1.52	0.81	1.42	**0.57**	**13.54**
2	4.25	-	-	-	1.42	0.00	0.00
3	4.25	3150.0	1.17	0.62	1.42	0.44	0.00
4	4.25	-	-	-	1.42	0.00	0.00

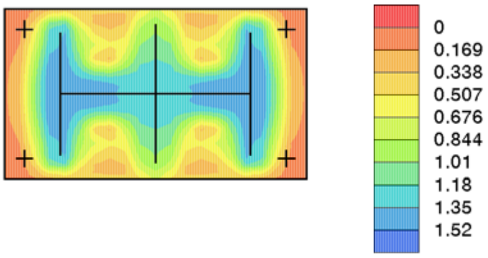
maximum utilization U = 0.57 < 1.00

maximum share of concrete compression with $\sigma_c > f_{cd}$ = 13.54 < 30.00

$A_{compr.}$ - area with concr. compr. $\sigma_{c,max}$ - maxmimum concr. compr. $\sigma_{c,m}$ - mean concr. compr. U - utilization

pressure distribution [kN/cm²]

LK 1 (max $\sigma_{c,m}$)

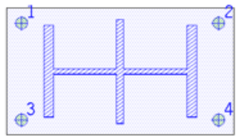


2.3.3. anchor tensile forces

$F_{t,Rd} = k2 \cdot f_{ub} \cdot A_s / \gamma M2$

$U = F_{t,Ed,max}/F,Rd$

numeration



stress area of M22: A_s = 3.03 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	---	---	---	---	123.95	0.00
2	24.68	24.50	64.00	63.82	123.95	0.52
3	---	---	---	---	123.95	0.00
4	21.79	22.28	64.72	65.21	123.95	**0.53**

maximum utilization U = 0.53 < 1.00

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

2.4. shear connector for transfer of horizontal force into the foundation

total length l = 13.0 cm
length in concrete l_c = 10.0 cm

2.4.1. concrete compression

$\sigma_c = V_{Ed}/(l_c \cdot b)$

$\sigma_{c,web,cal} = \sigma_{c,web} \cdot f_{\sigma,web}$

$U = \sigma_{c,max}/f_{cd}$

additional safety factor in case of concrete compressions by web $f_{\sigma,web}$ = 1.1

LK	$V_{Ed,flange}$ kN	$V_{Ed,web}$ kN	$\sigma_{c,flange}$ N/mm²	$\sigma_{c,web}$ N/mm²	$\sigma_{c,web,cal}$ N/mm²	f_{cd} N/mm²	U -
1	38.30	4.20	4.61	0.63	0.69	14.17	0.33
2	1.50	157.60	0.09	11.76	12.94	14.17	0.91
3	70.60	3.20	8.51	0.48	0.53	14.17	0.60
4	4.00	172.10	0.24	12.84	14.13	14.17	**1.00**

maximum utilization U = 1.00 = 1.00

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

2.4.2. 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	23.10	-210.65	-0.48	-1.78	3.08	21.36	0.14
2	-1260.80	-12.00	-5.97	-3.10	5.97	21.36	0.28
3	-17.60	388.30	0.77	3.27	5.67	21.36	0.27
4	-1376.80	32.00	6.55	-3.32	6.55	21.36	**0.31**

maximum utilization U = 0.31 < 1.00

$\sigma_{v,Ed}$ - equivalent stress σ_{Rd} - limit normal stress τ_{Rd} - limit shear stress   U - utilization

2.4.3. weld between base plate and shear connector

design with direction oriented method acc. to clause 4.5.3.2
$\sigma_{V,w,Ed} = (\sigma_\perp^2+3\cdot\tau_\perp^2+3\cdot\tau_{II}^2)^{0.5}$,
$f_{1,w,Rd} = f_u/(\beta_w\cdot\gamma_{M2})$
$f_{2,w,Rd} = 0.9\cdot f_u/\gamma_{M2}$
$U = \max\{\sigma_{V,w,Ed}/f_{1,w,Rd},\ \sigma_\perp^2/f_{2,w,Rd}\}$

connection designed with a **double fillet weld.**
axial force transfer of 100 % by the weld.

2.4.3.1. web weld

minimum value of the weld thickness a_{min} = 6 mm

LK	a_w 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	U -
1	**6**	0.00	0.00	-3.07	5.32	40.91	---	0.13
2	6	0.00	0.00	-0.12	0.21	40.91	---	0.01
3	6	0.00	0.00	5.66	9.80	40.91	---	**0.24**
4	6	0.00	0.00	0.32	0.56	40.91	---	0.01

maximum weld thickness $a_{w,max}$ = 6 mm
maximum utilization U = 0.24 < 1.00

2.4.3.2. flange weld

minimum value of the weld thickness a_{min} = 6 mm

LK	a_w 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	U -
1	**6**	0.74	0.74	-0.12	1.49	40.91	29.45	0.04
2	6	8.20	8.20	4.56	18.20	40.91	29.45	0.44
3	6	-1.20	-1.20	0.09	2.41	40.91	29.45	0.06
4	6	-9.01	-9.01	4.98	19.97	40.91	29.45	**0.49**

maximum weld thickness $a_{w,max}$ = 6 mm
maximum utilization U = 0.49 < 1.00

a_w - weld thickness $\sigma_\perp^2$ - normal stresses perpendicular to weld $\tau_\perp^2$ - shear stresses perpendicular to weld
τ_{II}^2 - shear stresses parallel to weld   U - utilization

3. summary

all executed verifications and design calculations successful.

max. utilizations of the particular verifications	
weld between column and base plate	84%
stresses in base plate	29%
pressures under base plate	57%
anchor tension forces	53%
shear connector	100%