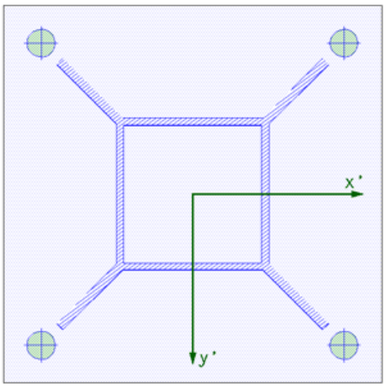
POS. 4: MSH200X200X8 + PLATES

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 1:10



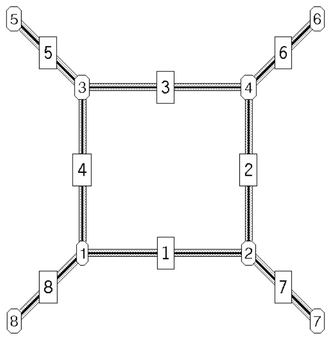
column cross section
user defined profile: MSH200X200X8+KN, of quality S355

base plate
b_x = 500 mm b_y = 500 mm   t = 20 mm, of quality S355

foundation/bedding
acc. to concrete C25/30

anchors
4 anchors, FK 4.8, M20, without shaft
with a length of 500 mm
edge distances a_x/a_y = 50/50 mm

description of column profile cross-section (MSH200X200X8+KN)



node coordinates

Nr.	x'	y'
-	mm	mm
1	-96.0	96.0
2	96.0	96.0
3	-96.0	-96.0
4	96.0	-96.0
5	-175.0	-175.0
6	175.0	-175.0
7	175.0	175.0
8	-175.0	175.0

line elements

Nr.	nodA	nodE	thickness
-	-	-	mm
1	1	2	8.0
2	2	4	8.0
3	4	3	8.0
4	3	1	8.0
5	3	5	10.0
6	4	6	10.0
7	2	7	10.0
8	1	8	10.0

1. loading

1.1. design values of column load

point of application in column centroid

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	design situat.
1	8.10	0.00	16.80	55.44	0.00	perman.

2. verification

2.1. partial safety factors for material

design situat.	γ_{M0}	γ_{M2}	γ_c	γ_μ
perman.	**1.10**	**1.10**	1.50	1.20

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	**4**	9.80	0.25	9.80	3.92	13.58	**0.29**

maximum weld thickness $a_{w,max}$ = 4 mm maximum utilization U = 0.29 < 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 43 elements in X-direction and 43 elements in Y-direction.
the concrete compression is limited to lim $\sigma_{c,d}$ = 4.04/γ_c kN/cm² .
The equivalent spring for the anchors is applied with c = E·A/l = 1029.00 kN/cm.

2.3.1. stresses in base plate (elast.-elast.)

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	43.6	43.6	0.04	0.30	-0.18	0.04	0.30

stresses and utilizations

$\sigma_V = (\sigma_x^2+\sigma_y^2-\sigma_x\cdot\sigma_y+3\cdot(\tau_{xy}^2+\tau_{xz}^2+\tau_{yz}^2))^{0.5}$
$\sigma_{Rd} = f_y/\gamma_{M0}$
$\tau_{Rd} = f_y/(3^{0.5}\cdot\gamma_{M0})$
U = max { σ_1/σ_{Rd}; τ_1/τ_{Rd}; σ_V/σ_{Rd} }

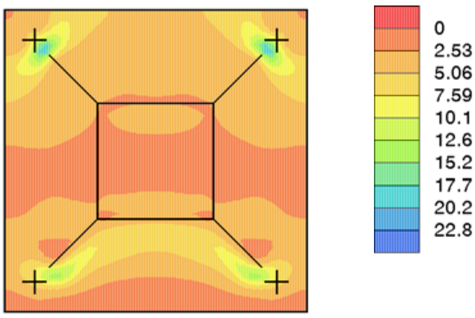
LK	x_{Fp} cm	y_{Fp} cm	σ_1 kN/cm²	τ_1 kN/cm²	σ_V kN/cm²	σ_{Rd} kN/cm²	τ_{Rd} kN/cm²	U -
1	43.6	43.6	24.64	14.60	25.29	32.27	18.63	**0.77**

maximum utilization U = 0.77 < 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
σ_1 - principal normal stress τ_1 - principal shear stress σ_V - equivalent stress σ_{Rd} - limit normal stress
U - utilization

stress distribution - σ_V [kN/cm²]

LK 1 (max σ_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%.

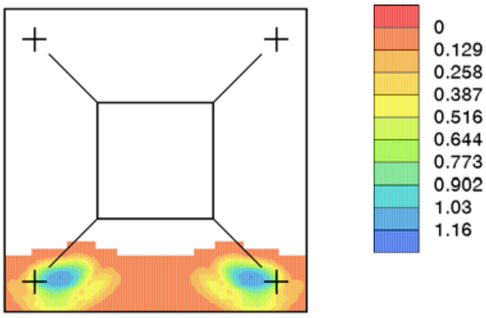
LK	lim $\sigma_{c,d}$ kN/cm²	$A_{compr.}$ cm²	$\sigma_{c,max}$ kN/cm²	$\sigma_{c,m}$ kN/cm²	f_{cd} kN/cm²	U -	$\sigma_c(A_D)>f_{cd}$ %
1	2.69	458.4	1.16	0.31	1.42	**0.22**	**0.00**

maximum utilization U = 0.22 < 1.00
maximum share of concrete compression with σ_c > f_{cd} = 0.00 < 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

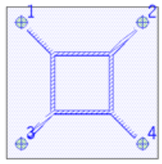
$F_{t,Rd} = k2 \cdot f_{ub} \cdot A_s / \gamma_{M2}$

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

stress area of M20: A_s = 2.45 cm²

No countersunk bolts used: k2 = 0.90

numeration



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	66.69	66.69	---	---	80.18	**0.83**

maximum utilization U = 0.83 < 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. slippage verification of base plate

$H_{res,d} = (H_{x,St,d}^2 + H_{y,St,d}^2)^{0.5}$

$N_{z,d} = A_{compression} \cdot \sigma_{c,m}$

$H_{Rd} = \mu_k / \gamma_\mu \cdot N_{z,d}$ with $\mu_k = 0.6$ acc. to [1]

LK	$H_{res,d}$ kN	$N_{z,d}$ kN	H_{Rd} kN	U -
1	16.80	141.48	70.74	**0.24**

maximum utilization U = 0.24 < 1.00

$H_{res,d}$ - design value of resultant sliding force $N_{z,d}$ - design value of compression force in sliding join
μ_k - characteristic value of slip factor H_{Rd} - design value of sliding force U - utilization

3. summary

all executed verifications and design calculations successful.

max. utilizations of the particular verifications	
weld between column and base plate	29%
stresses in base plate	77%
pressures under base plate	22%
anchor tension forces	83%
slippage of base plate	24%

[1] DIN V 4141-1: Lager im Bauwesen, Teil 1, Mai 2003