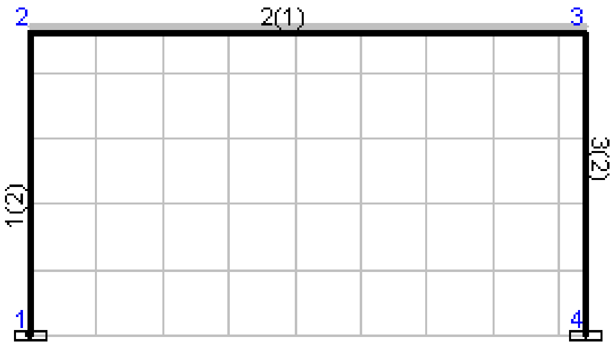
1-Finite element model (FEM)



Nodal points

Node	x [m]	y[m]
1	0.000	0.000
2	0.000	4.600
3	8.400	4.600
4	8.400	0.000

Supports

Node	kind	ux[mm]	uy[mm]	ur[rad]
1	fixed ux=uy=ur=0			
4	fixed ux=uy=ur=0			

Materials

Material : R.Concrete, E= 26.000 [GPa]
Weight density : ρ= 25.000 [kN/m³]
The element self weight is included in loads and masses

Element cross sections

Cr.sec.	b[cm]	h[cm]	b1[cm]	h1[cm]	Ac[cm²]	Ic[cm4]
1	25.000	70.000	120.000	15.000	3.17500E+003	1.33529E+006
2	30.000	60.000			1.80000E+003	5.40000E+005

Elements

Element	node 1	node 2	material	length(m)	angle(°)
1	1	2	2	4.600	90.000
2	2	3	1	8.400	0.000
3	3	4	2	4.600	270.000

Element distributed loads (γg=1.35, γq=1.50)

element	G[kN/m]	Q[kN/m]	γgG+γqQ[kN/m]	load kind	load direction
2	22.500	12.400	48.975	uniform	vertical

Element distributed loads due to self weight (γg=1.35, γq=1.50)

element	G[kN/m]	Q[kN/m]	γgG+γqQ[kN/m]	load kind	load direction
1	4.500	0.000	6.075	uniform	vertical
2	7.937	0.000	10.715	uniform	vertical
3	4.500	0.000	6.075	uniform	vertical

2-Results of static-linear-elastic analysis

Diagrams of internal forces M, V, N, and displacements d, of element 1

n	x/l	x[m]	M[kNm]	V[kN]	N[kN]	dx[mm]	dy[mm]	d[mm]
0	0.000	0.00	103.67	67.91	-278.64	0.000	0.000	0.000
1	0.100	0.46	72.43	67.91	-275.85	-0.070	-0.026	0.075
2	0.200	0.92	41.19	67.91	-273.05	-0.250	-0.052	0.255
3	0.300	1.38	9.95	67.91	-270.26	-0.491	-0.078	0.497
4	0.400	1.84	-21.29	67.91	-267.46	-0.748	-0.104	0.755
5	0.500	2.30	-52.53	67.91	-264.67	-0.972	-0.130	0.981
6	0.600	2.76	-83.77	67.91	-261.88	-1.118	-0.156	1.128
7	0.700	3.22	-115.01	67.91	-259.08	-1.137	-0.182	1.151
8	0.800	3.68	-146.25	67.91	-256.29	-0.982	-0.208	1.004
9	0.900	4.14	-177.49	67.91	-253.49	-0.608	-0.234	0.651
10	1.000	4.60	-208.73	67.91	-250.70	0.035	-0.260	0.263

Maximum values for element 1

```
maxM=      103.67 kNm, minM=     -208.73 kNm
maxV=       67.91 kN,  minV=       67.91 kN
maxN=     -250.70 kN,  minN=     -278.64 kN
maxd=       1.151 mm
```

Diagrams of internal forces M, V, N, and displacements d, of element 2

n	x/l	x[m]	M[kNm]	V[kN]	N[kN]	dx[mm]	dy[mm]	d[mm]
0	0.000	0.00	-208.73	-250.70	-67.91	0.035	-0.260	0.263
1	0.100	0.84	-19.20	-200.56	-67.91	0.028	-1.850	1.850
2	0.200	1.68	128.21	-150.42	-67.91	0.021	-3.486	3.486
3	0.300	2.52	233.51	-100.28	-67.91	0.014	-4.869	4.869
4	0.400	3.36	296.68	-50.14	-67.91	0.007	-5.784	5.784
5	0.500	4.20	317.74	0.00	-67.91	0.000	-6.103	6.103
6	0.600	5.04	296.68	50.14	-67.91	-0.007	-5.784	5.784
7	0.700	5.88	233.51	100.28	-67.91	-0.014	-4.869	4.869
8	0.800	6.72	128.21	150.42	-67.91	-0.021	-3.486	3.486
9	0.900	7.56	-19.20	200.56	-67.91	-0.028	-1.850	1.850
10	1.000	8.40	-208.73	250.70	-67.91	-0.034	-0.260	0.263

Maximum values for element 2

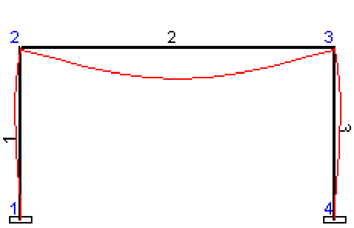
```
maxM=      317.74 kNm, minM=     -208.73 kNm
maxV=      250.70 kN,  minV=     -250.70 kN
maxN=      -67.91 kN,  minN=      -67.91 kN
maxd=       6.103 mm
```

Diagrams of internal forces M, V, N, and displacements d, of element 3

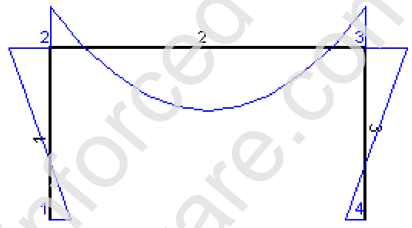
n	x/l	x[m]	M[kNm]	V[kN]	N[kN]	dx[mm]	dy[mm]	d[mm]
0	0.000	0.00	-208.73	-67.91	-250.70	-0.034	-0.260	0.263
1	0.100	0.46	-177.49	-67.91	-253.49	0.607	-0.234	0.651
2	0.200	0.92	-146.25	-67.91	-256.29	0.982	-0.208	1.004
3	0.300	1.38	-115.01	-67.91	-259.08	1.137	-0.182	1.151
4	0.400	1.84	-83.77	-67.91	-261.88	1.118	-0.156	1.129
5	0.500	2.30	-52.53	-67.91	-264.67	0.972	-0.130	0.980
6	0.600	2.76	-21.29	-67.91	-267.46	0.748	-0.104	0.755
7	0.700	3.22	9.95	-67.91	-270.26	0.491	-0.078	0.497
8	0.800	3.68	41.19	-67.91	-273.05	0.250	-0.052	0.255
9	0.900	4.14	72.43	-67.91	-275.85	0.071	-0.026	0.075
10	1.000	4.60	103.67	-67.91	-278.64	0.000	0.000	0.000

Maximum values for element 3

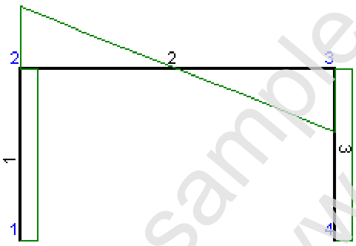
maxM=	103.67 kNm,	minM=	-208.73 kNm
maxV=	-67.91 kN,	minV=	-67.91 kN
maxN=	-250.70 kN,	minN=	-278.64 kN
maxd=	1.151 mm		



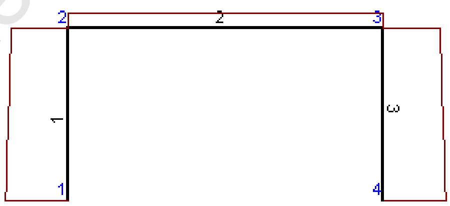
Displacement diagram
maxD=6.10 mm



Bending moment diagram
maxM=317.74 kNm, minM=-208.73 kNm



Shear force diagram
maxV=250.70 kN, minV=-250.70 kN



Axial force diagram
maxN=-67.91 kN, minN=-278.64 kN

3-Dimensioning of Concrete

Design codes

EN1990:2002, Eurocode 0 Basis of Structural Design
EN1991-1-1:2002, Eurocode 1-1 Actions on structures
EN1992-1-1:2004, Eurocode 2 Reinforced concrete
EN1997-1-1:2004, Eurocode 7 Geotechnical design
EN1998-1-1:2004, Eurocode 8 Design in earthquake environment
NA - National Annex:

Concrete-Steel class: C25/30-B500C (EC2 §3)
Concrete cover : Cnom=30 mm (EC2 §4.4.1)
$\gamma_c=1.50$, $\gamma_s=1.15$ (EC2 Table 2.1N)
$f_{cd}=\alpha_{cc}\cdot f_{ck}/\gamma_c=0.85\times25/1.50=14.17$ MPa (EC2 §3.1.6)
$f_{yd}=f_{yk}/\gamma_s=500/1.15=435$ MPa (EC2 §3.2.7)
Concrete weight : 25.0 kN/m^3

Reinforced concrete design, element 1, L= 4.600m, B= 300mm, H= 600mm

Med= 184.96 kNm, Ved= 67.91 kN, Ned= -278.64 kN
L=4.600m. Lcz=8.650m =1.88xL, Lcy=8.650m=1.88xL

Dimensions and loads

Column of rectangular cross section b=0.300 m, h=0.600 m, column length L=4.600 m
Loads, axial Ned=278.64 kN (compression), moments Medyy=184.96 kNm, Medzz=0.00 kNm
shear Ved=67.91 kN
Effective length direction z-z : Lcz= 1.88xL= 8.650m
Effective length direction y-y : Lcy= 1.88xL= 8.650m
Effective depth of cross section d=h-d1, d1=d2=Cnom+Øs+Ø/2=30+8+20/2=48mm, dx=252mm, dy=552mm

Design for compression with small eccentricity (ULS) (EC2 §6.1, §9.2.1)

Ned=278.64kN, Med,yy=184.96kNm, Med,zz=0.00kNm

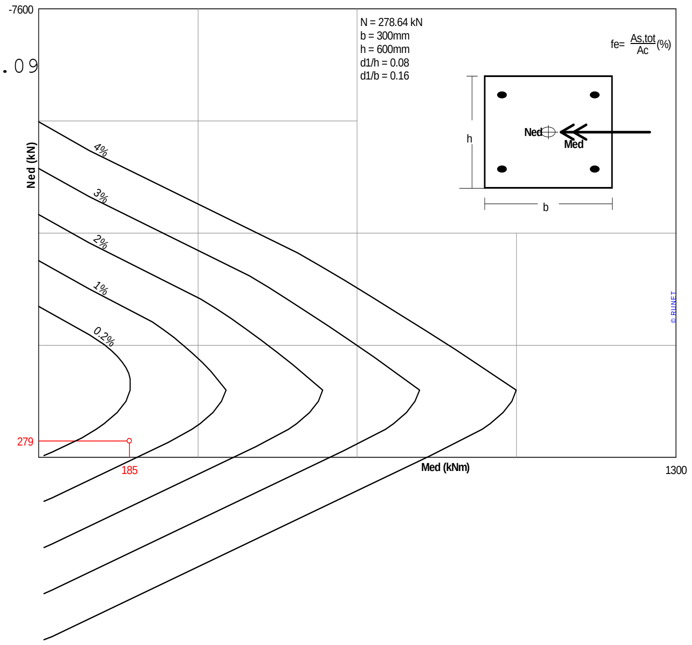
Approximate design from Tables (d1/h=0.10)

Kordina K, Bemessungshilfsmittel zu EC 2 Teil 1 Planung von Stahlbeton ..., Berlin, Beuth, 1992

$My/(bh^2 fcd)=0.10$, $Mz/(hb^2 fcd)=0.00$, $N/(bh\cdot fcd)=-0.09$
$As\cdot fyk/(bh\cdot fck)=0.18$, As= 1242mm^2, As/Ac=0.69%

Design using numerical integration

Design chart for single bending and axial force obtained from numerical integration of the concrete and steel forces over the cross-section
Ned=278.64kN (compression), Med=184.96kNm
C25/30-B500C
b=300mm, h=600mm
d=552mm, d1= 48mm, d2= 48mm, d1/h=0.080
e=Med/Ned=184.96/278.64=0.664m=664mm
zs=h/2-d1=600/2-48=252mm, e=664mm>sz=252mm
As1=As2=675mm^2, (As1+As2)/Ac=0.75%
$\varepsilon c2/\varepsilon s1=-3.50/-10.50$



As1= 675mm^2, As2= 675mm^2
As,tot=1350mm^2

Minimum longitudinal reinforcement, As>=0.0020Ac, Øs>=8, As,min=4Ø12(452mm^2) (EC2 §9.5.2.2)
Maximum longitudinal reinforcement, As<=0.04Ac, (As,max=7200mm^2) (EC2 §9.5.2.3)
Transverse reinforcement, links with minimum Øs at maximum spacing Scl,t (EC2 §9.5.3)
  at column heights from 0.60m to H-0.60m: Links Øs>=6, Scl,t<=300mm
  at regions 0 to 0.60m and H-0.60m to H : Links Øs>=6, Scl,t<=180mm
Basic required anchorage length Lbd=570mm =0.570m (EC2 Eq.8.3)

Longitudinal reinforcement: 8Ø20 (2512mm^2)

Transverse reinforcement: Links Ø 8/300 [h:0.60m~H-0.60m] , **Ø 8/180** [h:0~0.60m, H-0.60m~H]

Ultimate limit state (ULS), Design for shear (EC2 EN1992-1-1:2004, §6.2, §9.2.2)

Shear capacity without shear reinforcement Vrdc (EC2 §6.2.2)
Vrdc=[Crdc·k·(100ρ1·fck)$^{0.33}$+k1·σcp]·bw·d (EC2 Eq.6.2.a)
Vrdc>=(vmin+k1·σcp)·bw·d (EC2 Eq.6.2.b)
Crdc=0.18/γc=0.18/1.50=0.120, fck=25.00MPa ,bw=300mm, d=552mm
k=1+√(200/d) <=2, k=1.60, k1=0.15
ρ1=As1/(bw·d)=1250/(300x552)=0.0075
σcp=Ned/Ac=1000x278.64/180000=1.55N/mm^2
vmin=0.035·k$^{1.50}$·√fck = 0.35N/mm^2 (EC2 Eq.6.3N)
Vrd,c(min)=0.001x(0.35+0.15x1.55)x300x552=96.46kN
Vrdc=0.001x[0.120x1.60x(0.75x25.00)$^{0.33}$+0.15x1.55]x300x552=122.97kN
Ved=67.91 kN <= Vrdc=122.97 kN, **Ved<=Vrdc shear reinforcement is not needed**

Concrete strut capacity Vrdmax (EC2 §6.2.3 Eq.6.9)
Vrdmax=αcw·bw·z·ν1·fcd/(cotθ+tanθ), Ved/max(Vrdmax)=0.12, θ=45.0° cotθ=1.00 tanθ=1.00
αcw=1.00 z=0.9d, fck=25.0<=60Mpa ν1=0.6[1-fck/250]=0.6[1-25/250]=0.540, fcd=14.17Mpa
Vrdmax=0.001x1.00x300x0.9x552x0.540x14.17/2.00=570.2 kN

Design for second order effects (EC2 EN1992-1-1:2004, §5.8.3))

Final creep coefficient φ(∞,to)=2.50 (EC2 §3.1.4, Annex B)
Effective creep coefficient φef=φ(∞,to)·(MoEqp/MoEd)=2.50x0.50=1.25 (EC2 §5.8.4)
Modulus of elasticity of concrete Ecd=Ecm/γce=1000x31/1.20=25.42GPa=25417MPa (EC2 Eq.5.20)
Modulus of elasticity of steel Es=200GPa=200000MPa
Reinforcement ratio ρ=As/(b·d)=2512/(300x600)=0.014

Slenderness criterion for isolated members (EC2 EN1992-1-1:2004, §5.8.3.1)

λ,lim=20·A·B·C/√n (Eq.5.13N)
ω=As·fyd/(Ac·fcd)=2512x435/(300x600x14.17)=0.43
n=Ned/(Ac·fcd)=278643/(300x600x14.17)=0.109
A=1/(1+0.2·φef)=1/(1+0.2x1.25)=0.80
B=√(1+2.0·ω)=√(1+2.0·0.43)=1.36
C=1.70-rm=0.70, (rm=M01/M02=1.0)
λ,lim=20x0.80x1.36x0.70/√0.109=46.17

Slenderness and effective length, direction z-z (EC2 EN1992-1-1:2004, §5.8.3.2)

Unbraced members β=Lo/L=max(√[1+10·k1·k2/(k1+k2)],[1+k1/(1+k1)][1+k2/(1+k2)]) (Eq.5.16)
Effective length Lo=β·L=1.88x4.600=8.650 m
Slenderness ratio λ=Lo/i, i=0.289x600mm, λ=8650/173=49.88 (Eq.5.14)
λ=49.88>λ,lim=46.17, **second order effects must be considered**

Slenderness and effective length, direction y-y (EC2 EN1992-1-1:2004, §5.8.3.2)

Unbraced members β=Lo/L=max(√[1+10·k1·k2/(k1+k2)],[1+k1/(1+k1)][1+k2/(1+k2)]) (Eq.5.16)
Effective length Lo=β·L=1.88x4.600=8.650 m
Slenderness ratio λ=Lo/i, i=0.289x300mm, λ=8650/87=99.77 (Eq.5.14)
λ=99.77>λ,lim=46.17, **second order effects must be considered**

Nominal Stiffness (EC2 EN1992-1-1:2004, §5.8.7.2)

$EI=Kc\cdot Ecd\cdot Ic + Ks\cdot Es\cdot Is$ (EC2 Eq.5.21)
$\rho=As/Ac=0.014$, $Ecd=25417MPa$, $Es=200000MPa$
$n=Ned/(Ac\cdot fcd)=278643/(300x600x14.17)=0.109$
$Ks=1$, $Kc=k1\cdot k2/(1+\varphi ef)$, $\varphi ef=1.25$ (EC2 Eq.5.22)
$k1=\sqrt{(fck/20)}MPa=\sqrt{(25/20)}=1.12MPa$ (EC2 Eq.5.23)

direction z-z
$k2=n\cdot\lambda/170<=0.20$, $n=0.109$, $\lambda=49.88$, $k2=0.032$ (EC2 Eq.5.24)
$Kc=1.118x0.032/(1+1.25)=0.016$
$EI=0.016x25417x300x600^3/12+1.0x200000x1256x(552/2)^2=21.32\cdot 10^{12}Nmm^2=21322kNm^2$

direction y-y
$k2=n\cdot\lambda/170<=0.20$, $n=0.109$, $\lambda=99.77$, $k2=0.064$
$Kc=1.118x0.064/(1+1.25)=0.032$
$EI=0.032x25417x600x300^3/12+1.0x200000x1256x(252/2)^2=5.08\cdot 10^{12}Nmm^2=5081kNm^2$

Moment magnification factor (EC2 EN1992-1-1:2004, §5.8.7.3)

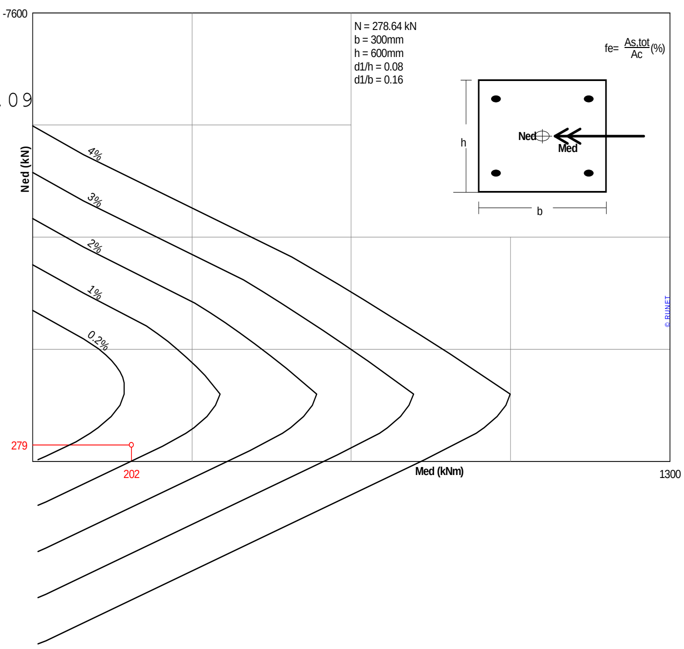
$Med=Moed[1+\beta/((Nb/Ned-1))]$, $Nb=\pi^2\cdot EI/Lo^2$ (EC2 Eq.5.28)
direction z-z
$\beta=\pi^2/co$, $co=12.0$, $\beta=0.82$ (EC2 Eq.5.29)
$Nb=3.14^2x21322/8.650^2=2812.48$ kN
$Med/Moed=1+0.82/(2812.48/278.64-1)=1.09$, **Med,yy=201.68kNm**
direction y-y
$\beta=\pi^2/co$, $co=12.0$, $\beta=0.82$ (EC2 Eq.5.29)
$Nb=3.14^2x5081/8.650^2=670.25$ kN
$Med/Moed=1+0.82/(670.25/278.64-1)=1.59$, **Med,zz=0.00kNm**

Design for compression with small eccentricity (ULS) (EC2 §6.1, §9.2.1)
(with second order effects)

Ned=278.64kN, Med,yy=201.68kNm, Med,zz=0.00kNm
Approximate design from Tables (d1/h=0.10)
Kordina K, Bemessungshilfsmittel zu EC 2 Teil 1
Planung von Stahlbeton ..., Berlin, Beuth, 1992
$My/(bh^2fcd)=0.11$, $Mz/(hb^2fcd)=0.00$, $N/(bh\cdot fcd)=-0.09$
$As\cdot fyk/(bh\cdot fck)=0.20$, $As=1380mm^2$, $As/Ac=0.77\%$

Design using numerical integration
Design chart for single bending and axial force
obtained from numerical integration of the
concrete and steel forces over the cross-section
Ned=278.64kN (compression), Med=201.68kNm
C25/30-B500C
b=300mm, h=600mm
d=552mm, d1= 48mm, d2= 48mm, d1/h=0.080
$e=Med/Ned=201.68/278.64=0.724m=724mm$
$zs=h/2-d1=600/2-48=252mm$, $e=724mm>sz=252mm$
$As1=As2=675mm^2$, $(As1+As2)/Ac=0.75\%$
$\varepsilon c2/\varepsilon s1=-3.50/-10.50$

$As1= 675mm^2$, $As2= 675mm^2$
$As,tot=1350mm^2$



Minimum longitudinal reinforcement, As>=0.0020Ac, Øs>=8, As,min=4Ø12(452mm^2) (EC2 §9.5.2.2)
Maximum longitudinal reinforcement, As<=0.04Ac, (As,max=7200mm^2) (EC2 §9.5.2.3)
Transverse reinforcement, links with minimum Øs at maximum spacing Scl,t (EC2 §9.5.3)
at column heights from 0.60m to H-0.60m: Links Øs>=6, Scl,t<=300mm
at regions 0 to 0.60m and H-0.60m to H : Links Øs>=6, Scl,t<=180mm
Basic required anchorage length Lbd=570mm =0.570m (EC2 Eq.8.3)

Longitudinal reinforcement: 8Ø20(2512mm^2)

Transverse reinforcement: Links Ø 8/300[h:0.60m~H-0.60m] **, Ø 8/180**[h:0~0.60m, H-0.60m~H]

Reinforced concrete design, element 2, [Span], L= 8.400m, B= 250mm, H= 700mm

Med = 317.74 kNm, Ved = 0.00 kN, Ned = -67.91 kN (x=4.20m)

Dimensions and loads

Beam web width bw=0.250 m, beam height h=0.700 m
Effective flange width beff=1.200 m, slab thickness hf=0.150 m
Effective depth of cross section d1=Cnom+Øs+1.1Ø=30+8+1.1x16=56mm, d2=56mm, d=700-56=644mm

Ultimate limit state (ULS)
Bending moment Med=317.74 kNm, Shear Ved=0.00 kN, Axial force Ned=-67.91 kN (compression)
Serviceability limit state (SLS)
Bending moment Med=158.87 kNm, Shear Ved=0.00 kN, Axial force Ned=-33.96 kN (compression)

Ultimate limit state (ULS), design for bending with axial force (EC2 §6.1, §9.2.1)

Reinforcement for bending with axial force (only tension reinforcement is needed)
Dimensioning for bending: Allgower,G.-Avak,R. Bemessungstafeln nach Eurocode 2
fur Rechteck und Plattenbalkenquerschnitte, In: Beton - und Stahlbetonbau 87 (1992)
Med=318kNm Nsd=-68kN beff=1200mm d=644mm Kd=1.215 x/d=0.08 εc2/εs1=-1.7/20.0 ks=2368, **As1=1085mm^2**
x=0.08x644=52<hf=150mm neutral axis within the depth of top flange
Minimum longitudinal tension reinf., As>=0.26bd·fctm/fyk, (As,min= 218mm^2) (EC2 §9.2.1.1.1)
Maximum tension or compression reinf., As<=0.04Ac, (As,max=7000mm^2) (EC2 §9.2.1.1.3)

Longitudinal reinforcement: 6Ø16(1206mm^2) **(bottom)**

Ultimate moment capacity of cross section (EC2 EN1992-1-1:2004, §6.1)

bw=250mm, h=700mm, bf=1200mm, hf=150mm, d=644mm, As1=1206mm^2, As2=0mm^2
εc2=-1.70o/oo, εs1=19.91o/oo, As1/b·d=0.00156(0.156%)
x/d=εc2/(εc2+εs1)=1.70/(1.70+19.91)=0.079, x=50.7mm
ar=0.609, ka=0.366, Fc=ar·b·x·fcd=Fs1=525.09kN, As1=Fs1/fyd=1207mm^2
z=d-ka·x=([1-ka·εc2/(εc2+εs1)]d, z/d=1.0-0.366x0.079=0.971, z=625.8mm,
Kd^2=1/(0.609·0.079·0.971·14.17)=1.516 mm^2/N, Kd=1.231
Bending capacity Mr=b·d^2/Kd^2=[10^{-6}]x1200x644^2/1.516=329.00kNm
x=50.7 mm<= Hf= 150mm neutral axis in flange
Bending capacity (εc2/εs1=1.70/19.91) Mr =329.00kNm

Shear between web and flanges (EC2 EN1992-1-1:2004, §6.2.4)

Fc=Fs=0.001x1206x435=525 kN
ΔFd=Fc·(beff-bw)/(2beff)=525.0x(1200-250)/(2x1200)=208kN
Beam span L=8.40 m, Δx=0.70x8.40/4=1.47 m (EC2 §5.3.2.1)
Vrdmax=ν·hf·fcd·sinθ·cosθ, fcd=14.17Mpa, θ=26.5°
ν=0.6(1-fck/250)=0.54 (EC2 Eq.6.6N)
Vrdmax=0.54x150x14.17xsin26.5°xcos26.5°=458kN/m (EC2 Eq.6.22)
ΔFd/Δx=208/1.47=141<=Vrdmax=458 kN/m, the check is verified
Transverse reinforcement per unit length Asf/sf (EC2 Eq.6.21)
Asf/sf=1000x141/(435xcot26.5°)= 162mm²/m
Transverse reinforcement Asf/sf=Ø 8/310 (162mm²/m)
ΔFd/Δx=141>0.40·hf·Fctd=0.40x150x1.02=61 kN/m
In case of transverse flexural reinforcement from plate bending, the steel area should be the greater or half the above, plus the required for transverse bending (EC2 §6.2.4.5)

Serviceability limit state (SLS) (EC2 EN1992-1-1:2004, §7)

Med(SLS)=158.87 kNm, Ned(SLS)=-33.96 kN
Final creep coefficient φ(∞,to)=2.50 (EC2 §3.1.4, Annex B)
Total shrinkage strain εcs=-0.30o/oo
γc=1.00, γs=1.00 (EC2 §2.4.2.4.2)
Modulus of elasticity of concrete Ecm=31GPa, Ec,eff=31/(1+2.50)=8.71GPa=8710MPa (EC2 Eq.7.20)
Modulus of elasticity of steel Es=200GPa=200000MPa
Modular ratio αe=Es/Ec=200/30.50=6.56, effective αe=Es/Ec,eff=200/8.71=22.96
Tension reinforcement: 6Ø16
Reinforcement ratio ρ=As1/(b·d)=1206/(1200x644)=0.002

State I (uncracked section) (SLS)
Bending stiffness of uncracked section, EI=(200/22.96)x(0.001x32.700)=284841 kNm²
S=As.zs1=(0.001)²x1206x0.171=(0.001)x0.206 m³ (EC2 Eq.7.21)
Curvature due to moment 1/rM=158.870/284841=(0.001)x0.558 (1/m)
Curvature due to shrinkage 1/rcs=(0.001x0.30)x22.960x(0.206/32.700)=(0.001)x0.043(1/m)
Total curvature 1/r=(0.001)x0.558+(0.001)x0.043=(0.001)x0.601(1/m)
Cracking moment, Mcr=fctm·(I/y2)=2.6x(32.700/0.227)=375.24 kNm

State II (fully cracked section) (SLS)
ρ=As/(b·d)=0.002, n=αe=22.96, n·ρ=0.046, ξ=0.675, α=0.261, x=α·d=0.168m
Bending stiffness of fully cracked section, EI=ξ·Es·As·d²=0.675x200x1206x0.644²=67625 kNm²
S=As.zs1=(0.001)²x1206x0.476=(0.001)x0.575 m³ (EC2 Eq.7.21)
Curvature due to moment 1/rM=158.870/67625=(0.001)x2.349 (1/m)
Curvature due to shrinkage 1/rcs=(0.001x0.30)x22.960x(0.575/7.764)=(0.001)x0.121(1/m)
Total curvature 1/r=(0.001)x2.349+(0.001)x0.121=(0.001)x2.470(1/m)
Med=158.87 kNm, , Ned=-33.96kN, εc/εs=0.42/1.10, x=177mm, σs=219 N/mm²

Checking deflections by calculation (SLS) (EN1992-1-1, §7.4.3)
Med=158.87<0.70xMcr=0.70x375.24=262.67 kNm, ζ=0.00 (Eq.7.19)
Final curvature (1/r)=0.00x(0.001x2.470)+(1-0.00)x(0.001x0.601)=(0.001)x0.601(1/m) (Eq.7.18)

Minimum reinforcement areas (SLS) (EC2 EN1992-1-1:2004, §7.3.2)

Minimum reinforcement areas As,min=kc·k·fct,eff·Act/σs (EC2 Eq.7.1)
b=0.250m, beff=1.200m, h=0.700m, d=0.644m, x=0.177m, Ø=16mm
Ned=-33.96kN, σc=(Ned/bh)=0.2N/mm², σs=219N/mm²
Act=(h-x)·b=(700-177)x250=130648 mm²
max(h,b1)=1mm, fctm=2.60N/mm², Ac,eff=130648mm², k=0.72, kc=0.35, k1=0.67
Minimum reinforcement, As,min=0.35x0.72x2.60x130648/219=390mm²

Calculation of crack width (SLS) (EC2 EN1992-1-1:2004, §7.3.3)

wk=sr,max·(εsm-εcm) (EC2 Eq.7.8)
εsm-εcm=[σs-kt·(fct,eff/ρeff)(1+αe·ρeff)]/Es >=0.6 σs/Es (EC2 Eq.7.9)
σs=219N/mm^2, short term loading: αe=6.56, kt=0.6, long term loading: αe=22.96, kt=0.4
Aceff=2.5(h-d)b=2.5x(700-644)x250=34750 mm^2 (§7.3.2.3)
ρeff=As/Ac,eff=1206/34750=0.035
εsm-εcm=[219-0.4x(2.6/0.035)(1+22.96x0.035)]/200=0.83 o/oo>=0.6x219/200=0.66 o/oo
sr,max=k3·(Cnom+Øs)+k1·k2·k4·Ø/ρeff (EC2 Eq.7.11)
Ø=16mm, k1=0.8, k2=(e1+e2)/2e1=0.5, k3=3.4, k4=0.425
sr,max=3.4x31.00+0.8x0.5x0.425x16/0.035=183.77 mm
wk=sr,max·(εsm-εcm)=183.77x0.001x0.83=0.15 mm

Reinforced concrete design, element 2, [Left end], L= 8.400m, B= 250mm, H= 700mm

MedA=-136.20kNm (x=t/2=0.30m), VedA=198.77kN (x=t/2+d=0.97m), VedAmax=250.70kN, NedA=-67.91kN

Dimensions and loads

Beam web width bw=0.250 m, beam height h=0.700 m
Effective flange width beff=1.200 m, slab thickness hf=0.150 m
Effective depth of cross section d1=Cnom+Øs+0.5Ø=56+8+0.5x16=72mm, d2=72mm, d=700-72=628mm

Ultimate limit state (ULS)
Bending moment Med=-136.20 kNm, Shear Ved=198.77 kN, Axial force Ned=-67.91 kN (compression)

Ultimate limit state (ULS), design for bending with axial force (EC2 §6.1, §9.2.1)

Dimensioning for bending: Allgower,G.-Avak,R. Bemessungstafeln nach Eurocode 2
fur Rechteck und Plattenbalkenquerschnitte, In: Beton - und Stahlbetonbau 87 (1992)
Reinforcement for bending with axial force (only tension reinforcement is needed)
Med=-136kNm Nsd=-68kN bw=250mm d=654mm Kd=0.826 x/d=0.14 εc2/εs1=-3.2/20.0 ks=2437, **As2= 428mm^2**
Minimum longitudinal tension reinf., As>=0.26bd·fctm/fyk, (As,min= 221mm^2) (EC2 §9.2.1.1.1)
Maximum tension or compression reinf., As<=0.04Ac, (As,max=7000mm^2) (EC2 §9.2.1.1.3)

Longitudinal reinforcement: 4Ø16 (804mm^2) **(top)**

Ultimate moment capacity of cross section (EC2 EN1992-1-1:2004, §6.1)

b=250mm, h=700mm, d=654mm, As1=804mm^2, As2=0mm^2
εc2=-3.50o/oo, εs1=15.25o/oo, As1/b·d=0.00492(0.492%)
x/d=εc2/(εc2+εs1)=3.50/(3.50+15.25)=0.187, x=122.1mm
ar=0.810, ka=0.416, Fc=ar·b·x·fcd=Fs1=350.09kN, As1=Fs1/fyd=805mm^2
z=d-ka·x=([1-ka·εc2/(εc2+εs1)]d, z/d=1.0-0.416x0.187=0.922, z=603.2mm,
Kd^2=1/(0.810·0.187·0.922·14.17)=0.506 mm^2/N, Kd=0.712
Bending capacity Mr=b·d^2/Kd^2=[10^{-6}]x250x654^2/0.506=212.00kNm

Ultimate limit state (ULS), Design for shear (EC2 EN1992-1-1:2004, §6.2, §9.2.2)

Shear capacity without shear reinforcement Vrdc (EC2 §6.2.2)
Vrdc=[Crdc·k·(100ρ1·fck)$^{0.33}$+k1·σcp]·bw·d (EC2 Eq.6.2.a)
Vrdc>=(vmin+k1·σcp)·bw·d (EC2 Eq.6.2.b)
Crdc=0.18/γc=0.18/1.50=0.120, fck=25.00MPa ,bw=250mm, d=654mm
k=1+$\sqrt{}$(200/d) <=2, k=1.55, k1=0.15
ρ1=As1/(bw·d)=1206/(250x654)=0.0074
σcp=Ned/Ac=1000x67.91/317500=0.21N/mm^2
vmin=0.035·k$^{1.50}$·$\sqrt{}$fck = 0.34N/mm^2 (EC2 Eq.6.3N)
Vrd,c(min)=0.001x(0.34+0.15x0.21)x250x654=60.74kN
Vrdc=0.001x[0.120x1.55x(0.74x25.00)$^{0.33}$+0.15x0.21]x250x654=85.58kN
Ved=198.77 kN > Vrdc=85.58 kN, **Ved>Vrdc shear reinforcement is needed**

Concrete strut capacity Vrdmax (EC2 §6.2.3 Eq.6.9)
$Vrdmax=\alpha cw\cdot bw\cdot z\cdot \nu 1\cdot fcd/(\cot\theta+\tan\theta)$, Ved/max(Vrdmax)=0.45, θ=21.8° $\cot\theta$=2.50 $\tan\theta$=0.40
αcw=1.00 z=0.9d, fck=25.0<=60Mpa $\nu 1$=0.6[1-fck/250]=0.6[1-25/250]=0.540, fcd=14.17Mpa
Vrdmax=0.001x1.00x250x0.9x654x0.540x14.17/2.90=388.3 kN
Ved=250.7 kN < 388.3 kN =Vrdmax, the check is verified

Shear reinforcement of vertical linkss (EC2 §6.2.3 Eq.6.8)
$Vrds=(Asw/s)z\cdot fywd\cdot \cot\theta$, Vrds=198.77kN, z=0.9d, fywd=0.8fyk=400.00N/mm², $\cot\theta$=2.50
Asw/s=Vrds/(z·fywd·cot21.80°)=(1.0E+006)x198.77/(0.9x654x400x2.50)=338mm²/m (Asw/s= 338mm²/m)
Required shear reinforcement: (Asw/s= 338mm²/m)

Minimum links for shear reinforcement (EC2 §9.2.2)
Minimum shear reinforcement ratio $\rho w,min$ (EC2 Eq.9.5N)
$\rho w,min=(0.08x(fck)^{0.5}/fyk$, fck=25N/mm², fyk=500N/mm², $\rho w,min$=0.0008
min Asw/s=1000x0.0008x250xsin(90°)= 200mm²/m

Maximum longitudinal spacing of links slmax=0.75d (<=600mm)=490mm (EC2 §9.2.2.6, Eq.9.6N)
Maximum transverse spacing of link legs stmax=0.75d (<=600mm)=490mm (EC2 §9.2.2.8, Eq.9.8N)

Minimum shear reinforcement links Ø 8/490 (Asw/s= 205mm²/m)

Shear reinforcement: links Ø 8/295 (Asw/s= 341mm²/m)