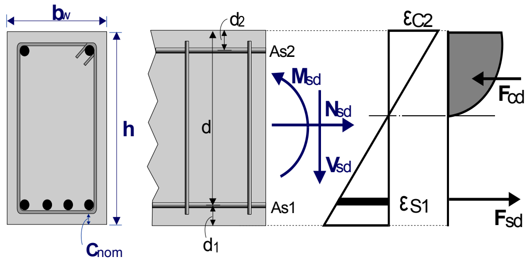
Example Beams

1. BEAM-001



Design of beam section for bending, shear and axial force
(EC2 EN1992-1-1:2004, EC0 EN1990-1-1:2002)

bxh=0.250x0.500 m, Msd=150.00 kNm,
Vsd= 40.00 kN, Nsd= 40.00 kN
Concrete-Steel class: C25/30-S500 (EC2 §3)
Concrete cover : Cnom=20 mm (EC2 §4.4.1)
γc=1.50, γs=1.15 (EC2 Table 2.1N)

1.1. Dimensions and loads

Beam width bw=0.250 m, beam height h=0.500 m
Bending moment Msd=150.00 kNm, Shear Vsd=40.00 kN, Axial force Nsd=40.00 kN (tension)
Effective depth of cross section d1=Cnomc+Øs+1.1Ø=20+8+1.1x10=39mm, d2=39mm, d=500-39=461mm

1.2. Ultimate limit state, 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)
Msd=150kNm, Nsd= 40kN, bw=250mm, d=461mm, Kd=1.94 x/d=0.26 εc/εs=3.5/ 9.9 ks=2.58, **As1= 8.84cm^2**
Minimum longitudinal tension reinf., As>=0.26bd·Fctm/fyk, (As,min= 1.56cm^2) (EC2 §9.2.1.1.1)
Maximum tension or compression reinf., As<=0.04Ac, (As,max=50.00cm^2) (EC2 §9.2.1.1.3)

Longitudinal reinforcement: 6Ø14 (9.24cm^2) **(bottom)**

1.3. Design against shear failure (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)^(0.333)+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
k=1+(200/d)^½ <=2, k=1.66, k1=0.15
ρ1=As1/(bw·d)=924/(250x461)=0.0080
σcp=Nsd/Ac=-1000x40.00/125000=-0.32N/mm^2
vmin=0.035·k^(1.50)·fck^½ = 0.37N/mm^2 (EC2 Eq.6.3N)
Vrd,c(min)=0.001x(0.37-0.15x0.32)x250x461=37.11kN
Vrdc=0.001x[0.120x1.66x(0.80x25.00)^(0.333)-0.15x0.32]x250x461=56.79kN
Vsd=40.00 kN <= Vrdc=56.79 kN, **Vsd<=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), Vsd/Vrdmax=0.11, θ=21.8° cotθ=2.50 tanθ=0.40
αcw=1.00 z=0.9d, fck=25.0<=60Mpa ν1=0.60, fcd=16.67Mpa
Vrdmax=0.001x1.00x250x0.9x461x0.60x16.67/2.90=357.7 kN

Minimum links for shear reinforcement (EC2 §9.2.2)
Minimum shear reinforcement ratio ρw,min (EC2 Eq.9.5N)
ρw,min=(0.08x(fck)^(0.5)/fyk, fck=25N/mm^2, fyk=500N/mm^2, ρw,min=0.0008
min Asw/s=10x0.0008x250xsin(90°)= 2.00cm^2/m

Maximum longitudinal spacing of links slmax=0.75d(1+cot90°)=345mm (EC2 §9.2.2.6, Eq.9.6N)
Maximum transverse spacing of link legs stmax=0.75d (<=600mm)=345mm (EC2 §9.2.2.8, Eq.9.8N)

Minimum shear reinforcement stirrups Ø8/34.5(Asw/s= 2.92cm^2/m)

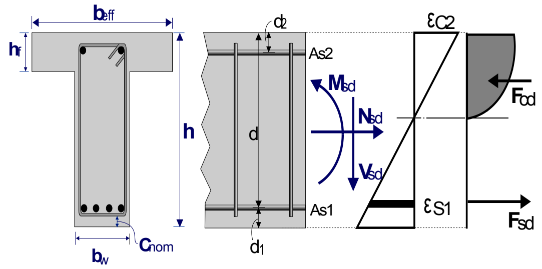
Shear reinforcement: stirrups Ø8/34.5 (Asw/s= 2.92cm^2/m)

1.4. Serviceability limit state, crack control (EC2 EN1992-1-1:2004, §7.3.2, §7.3.3)

Minimum reinforcement areas As,min=kc·k·fct,eff·Act/σs (EC2 Eq.7.1)
b=0.250m, beff=0.250m, h=0.500m, d=0.461m, N=-40.00kN, σc=(N/bh)=-0.32N/mm^2, Φ=14mm
max(h,b1)=500mm, fctm=2.60N/mm^2, hc,eff=2.50x(h-d)=98mm, k=0.86, kc=0.47 (EC2 Eq.7.2)
Min. reinf. without control of crack width, As,min=0.47x0.86x2.60x250x98/500=51mm^2=0.51cm^2
Crack control for crack width wk=0.3mm, using steel diameter Ø=14mm
Øs=Ø*s(fctm/2.9)[kc·hcr/2(h-d)], Øs=14mm, Ø*=10mm, (fctm=2.60, hcr=250mm) (EC2 Eq.7.6N)
Steel bar diameter Φ*=10mm, crack width wk=0.3mm, steel stress σs=320N/mm^2 (EC2 Table 7.2N)
Min. reinforcement for wk=0.3mm and Ø=14mm, As,min=0.47x0.86x2.60x250x98/320=80mm^2=0.80cm^2

2. BEAM-002

Design of beam section for bending, shear and axial force
(EC2 EN1992-1-1:2004, EC0 EN1990-1-1:2002)



bxh=0.200x0.600 m, Msd=100.00 kNm,
beff=0.300 m, hf=0.150 m
Vsd= 10.00 kN, Nsd= 10.00 kN
Concrete-Steel class: C25/30-S500 (EC2 §3)
Concrete cover : Cnom=20 mm (EC2 §4.4.1)
γc=1.50, γs=1.15 (EC2 Table 2.1N)

2.1. Dimensions and loads

Beam web width bw=0.200 m, beam height h=0.600 m
Effective flange width beff=0.300 m, slab thickness hf=0.150 m
Bending moment Msd=100.00 kNm, Shear Vsd=10.00 kN, Axial force Nsd=10.00 kN (tension)
Effective depth of cross section d1=Cnomc+Øs+1.1Ø=20+8+1.1x8=37mm, d2=37mm, d=600-37=563mm

2.2. Ultimate limit state, 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)
Msd=100kNm, Nsd= 10kN, beff=300mm, d=563mm, Kd=3.13 x/d=0.11 εc/εs=2.3/20.0 ks=2.40, **As1= 4.37cm^2**
x=0.11x563=62<hf=150mm neutral axis within the depth of top flange
Minimum longitudinal tension reinf., As>=0.26bd·Fctm/fyk, (As,min= 1.52cm^2) (EC2 §9.2.1.1.1)
Maximum tension or compression reinf., As<=0.04Ac, (As,max=48.00cm^2) (EC2 §9.2.1.1.3)

Longitudinal reinforcement:4Ø12 (4.52cm^2) **(bottom)**

2.3. Design against shear failure (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)^(0.333)+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
k=1+(200/d)^½ <=2, k=1.60, k1=0.15
ρ1=As1/(bw·d)=452/(200x563)=0.0040
σcp=Nsd/Ac=-1000x10.00/135000=-0.07N/mm^2
vmin=0.035·k^(1.50)·fck^½ = 0.35N/mm^2 (EC2 Eq.6.3N)
Vrd,c(min)=0.001x(0.35-0.15x0.07)x200x563=38.24kN
Vrdc=0.001x[0.120x1.60x(0.40x25.00)^(0.333)-0.15x0.07]x200x563=45.41kN
Vsd=10.00 kN <= Vrdc=45.41 kN, **Vsd<=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), Vsd/Vrdmax=0.03, θ=21.8° cotθ=2.50 tanθ=0.40
αcw=1.00 z=0.9d, fck=25.0<=60Mpa ν1=0.60, fcd=16.67Mpa
Vrdmax=0.001x1.00x200x0.9x563x0.60x16.67/2.90=349.6 kN

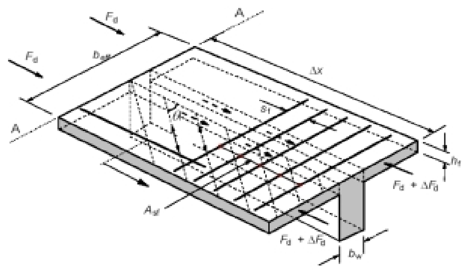
Minimum links for shear reinforcement (EC2 §9.2.2)
Minimum shear reinforcement ratio ρw,min (EC2 Eq.9.5N)
ρw,min=(0.08x(fck)^(0.5)/fyk, fck=25N/mm^2, fyk=500N/mm^2, ρw,min=0.0008
min Asw/s=10x0.0008x200xsin(90°)= 1.60cm^2/m

Maximum longitudinal spacing of links slmax=0.75d(1+cot90°)=420mm (EC2 §9.2.2.6, Eq.9.6N)
Maximum transverse spacing of link legs stmax=0.75d (<=600mm)=420mm (EC2 §9.2.2.8, Eq.9.8N)

Minimum shear reinforcement stirrups Ø8/42.0(Asw/s= 2.40cm^2/m)

Shear reinforcement: stirrups Ø8/42.0 (Asw/s= 2.40cm^2/m)

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

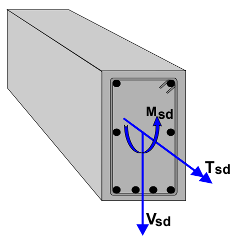


Fc=Fs=0.001x437x435=190 kN
ΔFd=Fc·(beff-bw)/(beff)=190.0x(300-200)/(300)=63kN
Beam span L=12.40 m, Δx=0.85x12.40/2=5.27 m (EC2 §5.3.2.1)
Vrdmax=ν·hf·fcd·sinθ·cosθ, fcd=16.67Mpa, θ=26.5°
ν=0.6(1-fck/250)=0.54 (EC2 Eq.6.6N)
Vrdmax=0.54x150x16.67xsin26.5°xcos26.5°=539kN/m (EC2 Eq.6.22)
ΔFd/Δx=63/5.27=12<=Vrdmax=539 kN/m, the check is verified
Transverse reinforcement per unit length Asf/sf (EC2 Eq.6.21)
Asf/sf=10x12/(435xcot26.5°)= 0.14cm^2/m
Transverse reinforcement Asf/sf=Ø8/42.0 (1.20cm^2/m)
ΔFd/Δx=12<=0.40·hf·Fctd=0.40x150x1.20=72 kN/m
In case of transverse flexural reinforcement from plate bending,
No extra reinforcement is needed (EC2 §6.2.4.6)

2.5. Serviceability limit state, crack control (EC2 EN1992-1-1:2004, §7.3.2, §7.3.3)

Minimum reinforcement areas As,min=kc·k·fct,eff·Act/σs (EC2 Eq.7.1)
b=0.200m, beff=0.300m, h=0.600m, d=0.563m, N=-10.00kN, σc=(N/bh)=-0.08N/mm^2, Φ=12mm
max(h,b1)=600mm, fctm=2.60N/mm^2, hc,eff=2.50x(h-d)=92mm, k=0.79, kc=0.42 (EC2 Eq.7.2)
Min. reinf. without control of crack width, As,min=0.42x0.79x2.60x200x92/500=32mm^2=0.32cm^2
Crack control for crack width wk=0.3mm, using steel diameter Ø=12mm
Øs=Ø*s(fctm/2.9)[kc·hcr/2(h-d)], Øs=12mm, Ø*=8mm, (fctm=2.60, hcr=300mm) (EC2 Eq.7.6N)
Steel bar diameter Φ*=8mm, crack width wk=0.3mm, steel stress σs=360N/mm^2 (EC2 Table 7.2N)
Min. reinforcement for wk=0.3mm and Ø=12mm, As,min=0.42x0.79x2.60x200x92/360=44mm^2=0.44cm^2

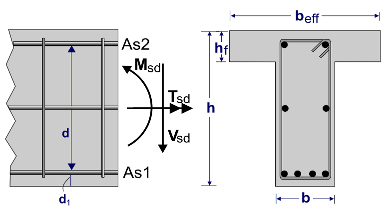
3. BEAM-003



Design of beam section for torsion, bending and shear
(EC2 EN1992-1-1:2004, EC0 EN1990-1-1:2002)

bxh=0.200x0.500 m, Tsd= 10.00 kNm,
beff=0.800 m, hf=0.180 m
Msd=100.00 kNm, Vsd= 10.00 kN
Concrete-Steel class: C25/30-S500 (EC2 §3)
Concrete cover : Cnom=20 mm (EC2 §4.4.1)
γc=1.50, γs=1.15 (EC2 Table 2.1N)

3.1. Dimensions and loads



Beam web width bw=0.200 m, beam height h=0.500 m
Effective flange width beff=0.800 m, slab thickness hf=0.180 m
Torsional moment Tsd= 10.00 kNm
Bending moment Msd=100.00 kNm
Shear force Vsd= 10.00 kN

Effective depth of cross section d1=Cnomc+Øs+0.5Ø=20+8+0.5x14=35mm, d2=35mm, d=500-35=465mm

3.2. Ultimate limit state, design for bending (EC2 EN1992-1-1:2004, §6.1, §9.2.1)

Reinforcement for bending (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)

Msd=100.00kNm, beff=800mm, d=465mm, Kd=4.16 x/d=0.07 $\varepsilon c/\varepsilon s$=1.6/20.0 ks=2.36, **As1= 5.08cm²**

x=0.07x465=33<hf=180mm neutral axis within the depth of top flange

Minimum longitudinal tension reinf., As>=0.26bd·Fctm/fyk, (As,min= 1.26cm²) (EC2 §9.2.1.1.1)

Maximum tension or compression reinf., As<=0.04Ac, (As,max=40.00cm²) (EC2 §9.2.1.1.3)

Reinforcement for bending:3Ø12+1Ø16 (5.40cm²) **(bottom)**

3.3. Design for shear and torsion (EC2 EN1992-1-1:2004, §6.3.2.4)

The design torsional resistance moment Trd,max is based on a truss model, with angle of inclined compression struts at θ=40.0° (1.0<cot40.0°=1.19<2.5) (EC2 Eq.6.7N)

Concrete strut capacity Vrdmax (EC2 §6.2.3 Eq.6.9)

Vrdmax=αcw·bw·z·ν1·fcd/(cotθ+tanθ), Vsd/Vrdmax=0.02, θ=40.0° cotθ=1.19 tanθ=0.84

αcw=1.00 z=0.9d, fck=25.0<=60Mpa ν1=0.60, fcd=16.67Mpa

Vrdmax=0.001x1.00x200x0.9x465x0.60x16.67/2.03=412.4 kN

Torsional resistance moment (EC2 §6.3.2.4)

Trd,max=2ν·αcw·fcd·Ak·tef·sinθ·cosθ, θ=40.0° (EC2 Eq.6.30)

αcw=1.00, ν=0.6(1-fck/250)=0.6(1-25.00/250)=0.540 (EC2 Eq.6.9 Eq.6.6N)

tef=A/u=0.500x0.200/(2x0.500+2x0.200)=0.071m = 71mm >=2xd1=2x35=70mm

Ak=(0.500-0.071)x(0.200-0.071)=0.055m² = 55102mm², uk=2x(0.129+0.429)=1.114m = 1114mm

Trd,max=2x0.540x1.00x0.001x16.67x55102x0.071x0.643x0.766=34.89kNm

Shear force and torsion (Tsd/Trd,Max)+(Vsd/Vrd,Max)<=1 (EC2 EN1992-1-1:2004, §6.3.2)

(10.00/34.89)+(10.00/412.40)=0.31<=1

Shear reinforcement of vertical stirrups (EC2 §6.2.3 Eq.6.8)

Vrds=(Asw/s)z·fywd·cotθ, Vrds=10.00kN, z=0.9d, fywd=0.8fyk=400.00N/mm², cotθ=1.19

Asw/s=Vrds/(z·fywd·cot40.00°)=(1.0E+006)x10.00/(0.9x465x400x1.19)=50mm²/m (Asw/s= 0.50cm²/m)

Required shear reinforcement: (Asw/s= 0.50cm²/m)

Required longitudinal reinforcement for torsion (EC2 Eq.6.28)

Asl/uk=Tsd·cot40.0°/(2Ak·fyd)=(1.0E+007)x10.00x1.192/(2x55102x435)= 2.49cm²/m

Required torsion links (EC2 Eq.6.26, Eq.6.27, Eq.6.8)

Asw/s =Tsd·tan40.0°/(2Ak·fyd)=(1.0E+007)x10.00x0.839/(2x55102x435)= 1.75cm²/m

Minimum links for shear reinforcement (EC2 EN1992-1-1:2004, §9.2.2)

Minimum shear reinforcement ratio ρw,min (EC2 Eq.9.5N)

ρw,min=(0.08x(fck)^(0.5)/fyk, fck=25N/mm², fyk=500N/mm², ρw,min=0.0008

min Asw/s=10x0.0008x200xsin(90°)= 1.60cm²/m

Maximum longitudinal spacing of links slmax=0.75d(1+cot90°)=345mm (EC2 §9.2.2.6, Eq.9.6N)

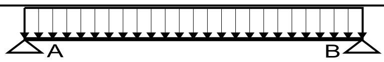
maximum space between torsion links sw=135mm (<=min(u/8=1114/8,200) (EC2 §9.2.3.3)

maximum spacing of longitudinal bars for torsion 130mm (<=350mm) (EC2 §9.2.3.4)

Maximum transverse spacing of link legs stmax=0.75d (<=600mm)=345mm (EC2 §9.2.2.8, Eq.9.8N)

Torsion-Shear reinforcement: closed links Ø8/13.5 (Asw/s= 7.45cm²/m)

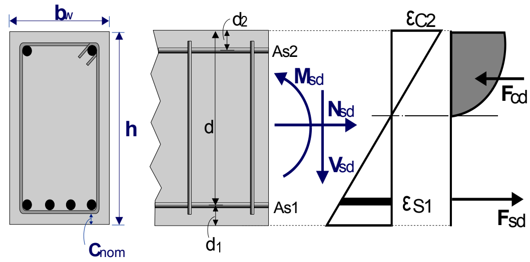
Longitudinal reinforcement for torsion: Ø10/13.0 (8Ø10) (6.04cm²/m)



4. BEAM-004

One span beam in composite loading

(EC2 EN1992-1-1:2004, EC0 EN1990-1-1:2002)



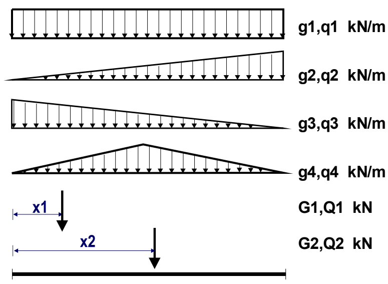
Concrete-Steel class: C25/30-S500 (EC2 §3)
Concrete cover : Cnom=20 mm (EC2 §4.4.1)
Concrete weight : 25.0 kN/m³
$\gamma c=1.50$, $\gamma s=1.15$ (EC2 Table 2.1N)

4.1. Dimensions and loads

Beam (rectangular section) , span L=6.000 m
L=6.000m, bw=0.250m, h=0.500m
Partial safety factors for actions : $\gamma G=1.35$, $\gamma Q=1.50$ (EC0 Annex A1)
Combination of variable actions : $\psi 1=0.60$, $\psi 2=0.30$
Effective depth of cross section d=h-d1, d1=Cnomc+Øs+0.5Ø=20+8+0.5x14=35mm

Beam loads

beam self weight	go= 3.13 kN/m		
uniform load	g1= 29.00 kN/m	q1= 10.00 kN/m	
triangular load	g2= 0.00 kN/m	q2= 0.00 kN/m	
triangular load	g3= 0.00 kN/m	q3= 0.00 kN/m	
triangular load	g4= 0.00 kN/m	q4= 0.00 kN/m	
concentrated load	G1= 0.00 kN	Q1= 0.00 kN	x1= 0.000 m
concentrated load	G2= 0.00 kN	Q2= 0.00 kN	x2= 0.000 m

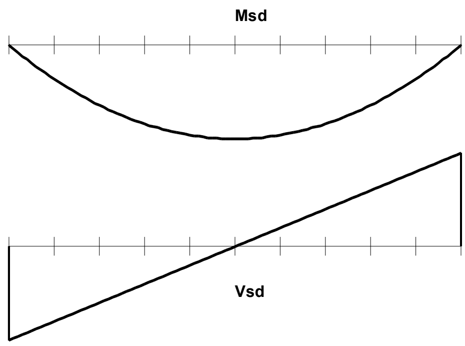


Cross section values (area A, moment of inertia Ixx, centroid yc)
Span-1 L= 6.000m, A=0.12500m²(1.25E+005mm²), Ixx=0.00260m4(2.60E+009mm4), yc=0.000m(0mm)

4.2. Design actions, shearing forces and bending moments

Bending moments and shears, load combination 1.35g+1.50q

x/L=0.00,	x= 0.00m,	Msd= 0.00 kNm,	Vsd= 175.11 kN
x/L=0.10,	x= 0.60m,	Msd= 94.56 kNm,	Vsd= 140.09 kN
x/L=0.20,	x= 1.20m,	Msd= 168.10 kNm,	Vsd= 105.06 kN
x/L=0.30,	x= 1.80m,	Msd= 220.63 kNm,	Vsd= 70.04 kN
x/L=0.40,	x= 2.40m,	Msd= 252.15 kNm,	Vsd= 35.02 kN
x/L=0.50,	x= 3.00m,	Msd= 262.66 kNm,	Vsd= 0.00 kN
x/L=0.60,	x= 3.60m,	Msd= 252.15 kNm,	Vsd= -35.02 kN
x/L=0.70,	x= 4.20m,	Msd= 220.63 kNm,	Vsd= -70.04 kN
x/L=0.80,	x= 4.80m,	Msd= 168.10 kNm,	Vsd= -105.06 kN
x/L=0.90,	x= 5.40m,	Msd= 94.56 kNm,	Vsd= -140.08 kN
x/L=1.00,	x= 6.00m,	Msd= 0.00 kNm,	Vsd= -175.11 kN



VsdA= 175.11 kN, VsdB= 175.11 kN, maxMsd= 262.66 kNm, maxVsd= 175.11 kN
Maximum span moment Msd=262.66 kNm (x=3.000m)
Maximum shear forces at distance d from support face
Span-A, b/2+d=0.557m, VsdA= 143.59kN, VsdB= 143.59kN

4.3. Span Ultimate limit state, design for bending (EC2 EN1992-1-1:2004, §6.1, §9.2.1)

Effective depth of cross section d1=Cnomc+Øs+1.1Ø=20+8+1.1x14=43mm, d2=43mm, d=500-43=457mm
Reinforcement for bending (tension and compression, reinforcement is needed)
Msd=262.66kNm, bw=250mm, d=457mm, Kd=1.41, ks1=2.78, ks2=0.42, **As1=16.00, As2= 2.44cm²**
Minimum longitudinal tension reinf., As>=0.26bd·Fctm/fyk, (As,min= 1.54cm²) (EC2 §9.2.1.1.1)
Maximum tension or compression reinf., As<=0.04Ac, (As,max=50.00cm²) (EC2 §9.2.1.1.3)
Reinforcement for bending: 4Ø18+2Ø20 (16.44cm²) **(bottom), 2Ø14** (3.08cm²) **(top)**

4.4. Span Design against shear failure (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)^(0.333)+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
k=1+(200/d)^½ <=2, k=1.66, k1=0.15
ρ1=As1/(bw·d)=1644/(250x457)=0.0144
vmin=0.035·k^(1.50)·fck^½ = 0.37N/mm² (EC2 Eq.6.3N)
Vrd,c(min)=0.001x(0.37)x250x457=42.24kN
Vrdc=0.001x[0.120x1.66x(1.44x25.00)^(0.333)]x250x457=75.08kN
Vsd=143.59 kN > Vrdc=75.08 kN, **Vsd>Vrdc shear reinforcement is needed**

Concrete strut capacity Vrdmax (EC2 §6.2.3 Eq.6.9)
Vrdmax=αcw·bw·z·ν1·fcd/(cotθ+tanθ), Vsd/Vrdmax=0.41, θ=21.8° cotθ=2.50 tanθ=0.40
αcw=1.00 z=0.9d, fck=25.0<=60Mpa ν1=0.60, fcd=16.67Mpa
Vrdmax=0.001x1.00x250x0.9x457x0.60x16.67/2.90=354.3 kN
Vsd=143.6 kN < 354.3 kN =Vrdmax, the check is verified

Shear reinforcement of vertical stirrups (EC2 §6.2.3 Eq.6.8)
Vrds=(Asw/s)z·fywd·cotθ, Vrds=143.59kN, z=0.9d, fywd=0.8fyk=400.00N/mm², cotθ=2.50
Asw/s=Vrds/(z·fywd·cot21.80°)=(1.0E+006)x143.59/(0.9x457x400x2.50)=349mm²/m (Asw/s= 3.49cm²/m)
Required shear reinforcement: (Asw/s= 3.49cm²/m)

Minimum links for shear reinforcement (EC2 §9.2.2)
Minimum shear reinforcement ratio ρw,min (EC2 Eq.9.5N)
ρw,min=(0.08x(fck)^(0.5)/fyk, fck=25N/mm², fyk=500N/mm², ρw,min=0.0008
min Asw/s=10x0.0008x250xsin(90°)= 2.00cm²/m

Maximum longitudinal spacing of links slmax=0.75d(1+cot90°)=340mm (EC2 §9.2.2.6, Eq.9.6N)
Maximum transverse spacing of link legs stmax=0.75d (<=600mm)=340mm (EC2 §9.2.2.8, Eq.9.8N)

Minimum shear reinforcement stirrups Ø8/34.0 (Asw/s= 2.96cm²/m)

Span Shear reinforcement: stirrups Ø8/28.5 (Asw/s= 3.53cm²/m)

4.5. Serviceability limit state, crack control (EC2 EN1992-1-1:2004, §7.3.2, §7.3.3)

Minimum reinforcement areas As,min=kc·k·fct,eff·Act/σs (EC2 Eq.7.1)
b=0.250m, beff=0.250m, h=0.500m, d=0.457m, N=0.00kN, σc=(N/bh)=0.00N/mm², Φ=18mm
max(h,b1)=500mm, fctm=2.60N/mm², hc,eff=2.50x(h-d)=108mm, k=0.86, kc=0.40 (EC2 Eq.7.2)
Min. reinf. without control of crack width, As,min=0.40x0.86x2.60x250x108/500=48mm²=0.48cm²
Crack control for crack width wk=0.3mm, using steel diameter Ø=18mm
Øs=Ø*s(fctm/2.9)[kc·hcr/2(h-d)], Øs=18mm, Ø*=17mm, (fctm=2.60, hcr=250mm) (EC2 Eq.7.6N)
Steel bar diameter Φ*=17mm, crack width wk=0.3mm, steel stress σs=236N/mm² (EC2 Table 7.2N)
Min. reinforcement for wk=0.3mm and Ø=18mm, As,min=0.40x0.86x2.60x250x108/236=102mm²=1.02cm²

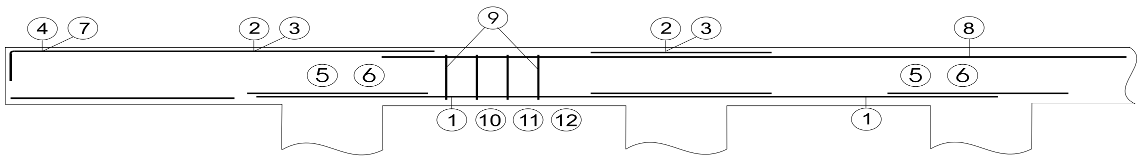
4.6. Serviceability limit state, deflection control (EC2 EN1992-1-1:2004, §7.4.2)

Span/effective depth, must be L/d<=limit of EC2 Table 7.4N
Span-, K=1.00, ρ=1.401%, L/d= 6000/ 457=13.14<14.59

4.7. Reinforcing bar schedule

Num		type	reinforcing bar [mm]	items	Ø	g/m [kg/m]	length [m]	weight [kg]

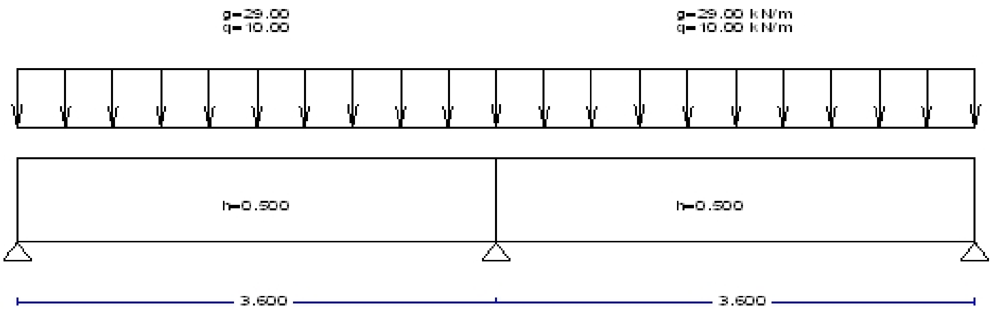
Total weight [kg] 0.00



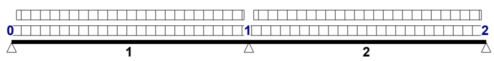
5. BEAM-005

Continuous beam with distributed loads
(EC2 EN1992-1-1:2004, EC0 EN1990-1-1:2002)

C25/30 - S500

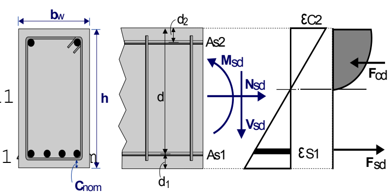


Concrete-Steel class: C25/30-S500 (EC2 §3)
Concrete cover : Cnom=20 mm (EC2 §4.4.1)
Concrete weight : 25.0 kN/m³
$\gamma c=1.50$, $\gamma s=1.15$ (EC2 Table 2.1N)



5.1. Dimensions and loads

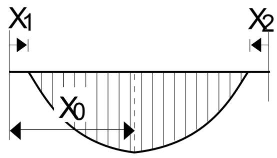
Continuous beam (rectangular section), number of spans=2
Partial safety factors for actions : $\gamma G=1.35$, $\gamma Q=1.50$ (EC0 Annex A1
Combination of variable actions : $\psi 1=0.60$, $\psi 2=0.30$
Effective depth of cross section d=h-d1, d1=Cnomc+Øs+0.5Ø=20+8+0.5x1



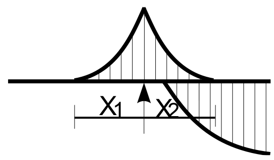
Spans, widths, thickness, load on spans (g=self weight +dead, q=live)
Span-1 L= 3.60m bw=0.250m beff=0.250m h=0.500m g=3.13+ 29.00= 32.13kN/m q= 10.00kN/m
Span-2 L= 3.60m bw=0.250m beff=0.250m h=0.500m g=3.13+ 29.00= 32.13kN/m q= 10.00kN/m

Cross section values (area A, moment of inertia Ixx, centroid yc)
Span-1 L= 3.600m, A=0.12500m^2(1.25E+005mm^2), Ixx=0.00260m4(2.60E+009mm4), yc=0.000m(0mm)
Span-2 L= 3.600m, A=0.12500m^2(1.25E+005mm^2), Ixx=0.00260m4(2.60E+009mm4), yc=0.000m(0mm)

5.2. Shearing forces and bending moments



Maximum bending moments at spans for load combinations 1.35g+1.50q
Span-1, Msd= 61.46 kNm, xo=1.451 m, x1=0.000m, x2=0.698m
Span-2, Msd= 61.46 kNm, xo=2.149 m, x1=0.698m, x2=0.000m



Maximum bending moments at supports for load combinations 1.35g+1.50q
Support-0, Msd= 0.00 kNm, x1=0.000 m, x2=0.000 m
Support-1, Msd= -94.56 kNm, x1=0.900 m, x2=0.900 m
Support-2, Msd= 0.00 kNm, x1=0.000 m, x2=0.000 m

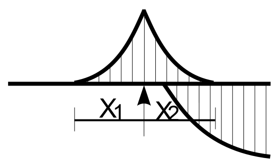
Maximum shear forces for load combinations 1.35g+1.50q
Span-1, Vsd,left= 78.80 kN, Vsd,right=-131.33 kN
Span-2, Vsd,left= 127.95 kN, Vsd,right= -82.17 kN

Maximum reactions due to dead and live loads (Rg and Rq)
Support-0, Rg(x1.35)= 58.55 kN, Rq(x1.50)= 23.63 kN
Support-1, Rg(x1.35)= 195.16 kN, Rq(x1.50)= 67.50 kN
Support-2, Rg(x1.35)= 58.55 kN, Rq(x1.50)= 23.63 kN

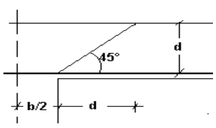
5.3. Design actions, shearing forces and bending moments

Design action values after moment redistribution by 0% (EC2 §5.5)
Reduction of support moments to moments at support faces (bsup=0.20 m) (EC2 §5.3.2.2.3)
Check for minimum values, (0.65ql^2/8 or 0.65ql^2/12) (EC2 §5.3.2.2.3N)

Maximum span bending moments and shear forces for load combinations 1.35g+1.50q
Span-1, Msd= 61.46 kNm, Vsd,left= 84.70 kN, Vsd,right=-125.42 kN
Span-2, Msd= 61.46 kNm, Vsd,left= 125.42 kN, Vsd,right= -84.70 kN



Maximum bending moments at supports for load combinations 1.35g+1.50q
Support-0, Msd= 0.00 kNm, x1=0.000 m, x2=0.000 m
Support-1, Msd= -82.01 kNm, x1=0.900 m, x2=0.900 m
Support-2, Msd= 0.00 kNm, x1=0.000 m, x2=0.000 m



Maximum shear forces at distance d from support face 1.35g+1.50q
Span-1, b/2+d=0.565m, 1.35g+1.50q=58.37kN/m, VsdA= 51.72kN, VsdB= 92.45kN
Span-2, b/2+d=0.565m, 1.35g+1.50q=58.37kN/m, VsdA= 92.45kN, VsdB= 51.72kN

5.4. Ultimate limit state, design for bending

(EC2 EN1992-1-1:2004, §6.1, §9.2.1)

Span-1

Effective depth of cross section d1=Cnomc+Øs+0.5Ø=20+8+0.5x14=35mm, d2=35mm, d=500-35=465mm
Reinforcement for bending (only tension reinforcement is needed)
Msd= 61.46kNm, bw=250mm, d=465mm, Kd=2.97 x/d=0.11 εc/εs=2.6/20.0 ks=2.41, **As1= 3.18cm^2**
Minimum longitudinal tension reinf., As>=0.26bd·Fctm/fyk, (As,min= 1.57cm^2) (EC2 §9.2.1.1.1)
Maximum tension or compression reinf., As<=0.04Ac, (As,max=50.00cm^2) (EC2 §9.2.1.1.3)
Reinforcement for bending: 4Ø10 (3.14cm^2) **(bottom)**

Span-2

Effective depth of cross section d1=Cnomc+Øs+0.5Ø=20+8+0.5x14=35mm, d2=35mm, d=500-35=465mm
Reinforcement for bending (only tension reinforcement is needed)
Msd= 61.46kNm, bw=250mm, d=465mm, Kd=2.97 x/d=0.11 εc/εs=2.6/20.0 ks=2.41, **As1= 3.18cm²**
Minimum longitudinal tension reinf., As>=0.26bd·Fctm/fyk, (As,min= 1.57cm²)
Maximum tension or compression reinf., As<=0.04Ac, (As,max=50.00cm²)
Reinforcement for bending: 4Ø10 (3.14cm²) **(bottom)**

Support-1

Effective depth of cross section d1=Cnomc+Øs+0.5Ø=20+8+0.5x14=35mm, d2=35mm, d=500-35=465mm
Reinforcement for bending (only tension reinforcement is needed)
Msd=-82.01kNm, bw=250mm, d=465mm, Kd=2.57 x/d=0.14 εc/εs=3.3/20.0 ks=2.44, **As2= 4.31cm²**
Minimum longitudinal tension reinf., As>=0.26bd·Fctm/fyk, (As,min= 1.57cm²) (EC2 §9.2.1.1.1)
Maximum tension or compression reinf., As<=0.04Ac, (As,max=50.00cm²) (EC2 §9.2.1.1.3)
Reinforcement for bending: 4Ø10+1Ø12 (4.27cm²) **(top)**

5.5. Design against shear failure (EC2 EN1992-1-1:2004, §6.2, §9.2.2)

Span-1 left

Shear capacity without shear reinforcement Vrdc (EC2 §6.2.2)
Vrdc=[Crdc·k·(100ρ1·fck)^(0.333)+k1·σcp]·bw·d, Vrdc>=(vmin+k1·σcp)·bw·d (EC2 Eq.6.2.a,b)
Crdc=0.18/γc=0.18/1.50=0.120, fck=25.00MPa, k=1+(200/d)^½ <=2, k=1.66, k1=0.15
Vrd,c(min)=0.001x(0.37)x250x465=43.01kN, vmin=0.035·k^(1.50)·fck^½ = 0.37N/mm² (EC2 Eq.6.3N)
ρ1=314/(250x465)=0.0027, Vrdc=0.001x[0.120x1.66x(0.27x25.00)^(0.333)]x250x465=43.76kN
Vsd=51.72 kN > Vrdc=43.76 kN, **Vsd>Vrdc shear reinforcement is needed**

Concrete strut capacity Vrdmax (EC2 §6.2.3 Eq.6.9)
Vrdmax=αcw·bw·z·ν1·fcd/(cotθ+tanθ), Vsd/Vrdmax=0.14, θ=21.8° cotθ=2.50 tanθ=0.40
αcw=1.00 z=0.9d, fck=25.0<=60Mpa ν1=0.60, fcd=16.67Mpa
Vrdmax=0.001x1.00x250x0.9x465x0.60x16.67/2.90=360.8 kN
Vsd=51.7 kN < 360.8 kN =Vrdmax, the check is verified

Shear reinforcement of vertical stirrups (EC2 §6.2.3 Eq.6.8)
Vrds=(Asw/s)z·fywd·cotθ, Vrds=51.72kN, z=0.9d, fywd=0.8fyk=400.00N/mm², cotθ=2.50
Asw/s=Vrds/(z·fywd·cot21.80°)=(1.0E+006)x51.72/(0.9x465x400x2.50)=124mm²/m (Asw/s= 1.24cm²/m)
Required shear reinforcement: (Asw/s= 1.24cm²/m)

Minimum links for shear reinforcement (EC2 §9.2.2)
Minimum shear reinforcement ratio ρw,min (EC2 Eq.9.5N)
ρw,min=(0.08x(fck)^(0.5)/fyk, fck=25N/mm², fyk=500N/mm², ρw,min=0.0008
min Asw/s=10x0.0008x250xsin(90°)= 2.00cm²/m
Maximum longitudinal spacing of links slmax=0.75d(1+cot90°)=345mm (EC2 §9.2.2.6, Eq.9.6N)
Maximum transverse spacing of link legs stmax=0.75d (<=600mm)=345mm (EC2 §9.2.2.8, Eq.9.8N)
Minimum shear reinforcement stirrups Ø8/34.5(Asw/s= 2.92cm²/m)

Span-1 left Shear reinforcement: stirrups Ø8/34.5 (Asw/s= 2.92cm²/m)

Span-1 right

Shear capacity without shear reinforcement Vrdc (EC2 §6.2.2)
Vrdc=[Crdc·k·(100ρ1·fck)^(0.333)+k1·σcp]·bw·d, Vrdc>=(vmin+k1·σcp)·bw·d (EC2 Eq.6.2.a,b)
Crdc=0.18/γc=0.18/1.50=0.120, fck=25.00MPa, k=1+(200/d)^½ <=2, k=1.66, k1=0.15
Vrd,c(min)=0.001x(0.37)x250x465=43.01kN, vmin=0.035·k^(1.50)·fck^½ = 0.37N/mm² (EC2 Eq.6.3N)
ρ1=314/(250x465)=0.0027, Vrdc=0.001x[0.120x1.66x(0.27x25.00)^(0.333)]x250x465=43.76kN
Vsd=92.45 kN > Vrdc=43.76 kN, **Vsd>Vrdc shear reinforcement is needed**

Concrete strut capacity Vrdmax (EC2 §6.2.3 Eq.6.9)
Vrdmax=αcw·bw·z·ν1·fcd/(cotθ+tanθ), Vsd/Vrdmax=0.26, θ=21.8° cotθ=2.50 tanθ=0.40
αcw=1.00 z=0.9d, fck=25.0<=60Mpa ν1=0.60, fcd=16.67Mpa
Vrdmax=0.001x1.00x250x0.9x465x0.60x16.67/2.90=360.8 kN
Vsd=92.4 kN < 360.8 kN =Vrdmax, the check is verified

Shear reinforcement of vertical stirrups (EC2 §6.2.3 Eq.6.8)
Vrds=(Asw/s)z·fywd·cotθ, Vrds=92.45kN, z=0.9d, fywd=0.8fyk=400.00N/mm², cotθ=2.50
Asw/s=Vrds/(z·fywd·cot21.80°)=(1.0E+006)x92.45/(0.9x465x400x2.50)=221mm²/m (Asw/s= 2.21cm²/m)
Required shear reinforcement: (Asw/s= 2.21cm²/m)

Minimum links for shear reinforcement (EC2 §9.2.2)
Minimum shear reinforcement ratio ρw,min (EC2 Eq.9.5N)
ρw,min=(0.08x(fck)^(0.5)/fyk, fck=25N/mm², fyk=500N/mm², ρw,min=0.0008
min Asw/s=10x0.0008x250xsin(90°)= 2.00cm²/m
Maximum longitudinal spacing of links slmax=0.75d(1+cot90°)=345mm (EC2 §9.2.2.6, Eq.9.6N)
Maximum transverse spacing of link legs stmax=0.75d (<=600mm)=345mm (EC2 §9.2.2.8, Eq.9.8N)
Minimum shear reinforcement stirrups Ø8/34.5(Asw/s= 2.92cm²/m)

Span-1 right Shear reinforcement: stirrups Ø8/34.5 (Asw/s= 2.92cm²/m)

Span-2 left

Shear capacity without shear reinforcement Vrdc (EC2 §6.2.2)
Vrdc=[Crdc·k·(100ρ1·fck)^(0.333)+k1·σcp]·bw·d, Vrdc>=(vmin+k1·σcp)·bw·d (EC2 Eq.6.2.a,b)
Crdc=0.18/γc=0.18/1.50=0.120, fck=25.00MPa, k=1+(200/d)^½ <=2, k=1.66, k1=0.15
Vrd,c(min)=0.001x(0.37)x250x465=43.01kN, vmin=0.035·k^(1.50)·fck^½ = 0.37N/mm² (EC2 Eq.6.3N)
ρ1=314/(250x465)=0.0027, Vrdc=0.001x[0.120x1.66x(0.27x25.00)^(0.333)]x250x465=43.76kN
Vsd=92.45 kN > Vrdc=43.76 kN, **Vsd>Vrdc shear reinforcement is needed**

Concrete strut capacity Vrdmax (EC2 §6.2.3 Eq.6.9)
Vrdmax=αcw·bw·z·ν1·fcd/(cotθ+tanθ), Vsd/Vrdmax=0.26, θ=21.8° cotθ=2.50 tanθ=0.40
αcw=1.00 z=0.9d, fck=25.0<=60Mpa ν1=0.60, fcd=16.67Mpa
Vrdmax=0.001x1.00x250x0.9x465x0.60x16.67/2.90=360.8 kN
Vsd=92.4 kN < 360.8 kN =Vrdmax, the check is verified

Shear reinforcement of vertical stirrups (EC2 §6.2.3 Eq.6.8)
Vrds=(Asw/s)z·fywd·cotθ, Vrds=92.45kN, z=0.9d, fywd=0.8fyk=400.00N/mm², cotθ=2.50
Asw/s=Vrds/(z·fywd·cot21.80°)=(1.0E+006)x92.45/(0.9x465x400x2.50)=221mm²/m (Asw/s= 2.21cm²/m)
Required shear reinforcement: (Asw/s= 2.21cm²/m)

Minimum links for shear reinforcement (EC2 §9.2.2)
Minimum shear reinforcement ratio ρw,min (EC2 Eq.9.5N)
ρw,min=(0.08x(fck)^(0.5)/fyk, fck=25N/mm², fyk=500N/mm², ρw,min=0.0008
min Asw/s=10x0.0008x250xsin(90°)= 2.00cm²/m
Maximum longitudinal spacing of links slmax=0.75d(1+cot90°)=345mm (EC2 §9.2.2.6, Eq.9.6N)
Maximum transverse spacing of link legs stmax=0.75d (<=600mm)=345mm (EC2 §9.2.2.8, Eq.9.8N)
Minimum shear reinforcement stirrups Ø8/34.5(Asw/s= 2.92cm²/m)

Span-2 left Shear reinforcement: stirrups Ø8/34.5 (Asw/s= 2.92cm²/m)

Span-2 right

Shear capacity without shear reinforcement Vrdc (EC2 §6.2.2)
Vrdc=[Crdc·k·(100ρ1·fck)^(0.333)+k1·σcp]·bw·d, Vrdc>=(vmin+k1·σcp)·bw·d (EC2 Eq.6.2.a,b)
Crdc=0.18/γc=0.18/1.50=0.120, fck=25.00MPa, k=1+(200/d)^½ <=2, k=1.66, k1=0.15
Vrd,c(min)=0.001x(0.37)x250x465=43.01kN, vmin=0.035·k^(1.50)·fck^½ = 0.37N/mm² (EC2 Eq.6.3N)
ρ1=314/(250x465)=0.0027, Vrdc=0.001x[0.120x1.66x(0.27x25.00)^(0.333)]x250x465=43.76kN
Vsd=51.72 kN > Vrdc=43.76 kN, **Vsd>Vrdc shear reinforcement is needed**

Concrete strut capacity Vrdmax (EC2 §6.2.3 Eq.6.9)
Vrdmax=αcw·bw·z·ν1·fcd/(cotθ+tanθ), Vsd/Vrdmax=0.14, θ=21.8° cotθ=2.50 tanθ=0.40
αcw=1.00 z=0.9d, fck=25.0<=60Mpa ν1=0.60, fcd=16.67Mpa
Vrdmax=0.001x1.00x250x0.9x465x0.60x16.67/2.90=360.8 kN
Vsd=51.7 kN < 360.8 kN =Vrdmax, the check is verified

Shear reinforcement of vertical stirrups (EC2 §6.2.3 Eq.6.8)
Vrds=(Asw/s)z·fywd·cotθ, Vrds=51.72kN, z=0.9d, fywd=0.8fyk=400.00N/mm², cotθ=2.50
Asw/s=Vrds/(z·fywd·cot21.80°)=(1.0E+006)x51.72/(0.9x465x400x2.50)=124mm²/m (Asw/s= 1.24cm²/m)
Required shear reinforcement: (Asw/s= 1.24cm²/m)

Minimum links for shear reinforcement (EC2 §9.2.2)
Minimum shear reinforcement ratio ρw,min (EC2 Eq.9.5N)
$\rho_{w,min}=(0.08x(f_{ck})^{(0.5)}/f_{yk}$, $f_{ck}=25N/mm^2$, $f_{yk}=500N/mm^2$, $\rho_{w,min}=0.0008$
min $A_{sw}/s=10x0.0008x250xsin(90°)= 2.00cm^2/m$
Maximum longitudinal spacing of links $s_{lmax}=0.75d(1+cot90°)=345mm$ (EC2 §9.2.2.6, Eq.9.6N)
Maximum transverse spacing of link legs $s_{tmax}=0.75d$ (<=600mm)=345mm (EC2 §9.2.2.8, Eq.9.8N)
Minimum shear reinforcement stirrups Ø8/34.5($A_{sw}/s= 2.92cm^2/m$)

Span-2 right Shear reinforcement: stirrups Ø8/34.5 ($A_{sw}/s= 2.92cm^2/m$)

5.6. Serviceability limit state, crack control (EC2 EN1992-1-1:2004, §7.3.2, §7.3.3)

Minimum reinforcement areas $A_{s,min}=k_c \cdot k \cdot f_{ct,eff} \cdot A_{ct}/\sigma_s$ (EC2 Eq.7.1)
b=0.250m, beff=0.250m, h=0.500m, d=0.465m, N=0.00kN, $\sigma_c=(N/bh)=0.00N/mm^2$, Φ=10mm
max(h,b1)=500mm, $f_{ctm}=2.60N/mm^2$, $h_{c,eff}=2.50x(h-d)=87mm$, k=0.86, kc=0.40 (EC2 Eq.7.2)
Min. reinf. without control of crack width, $A_{s,min}=0.40x0.86x2.60x250x87/500=39mm^2=0.39cm^2$
Crack control for crack width wk=0.3mm, using steel diameter Ø=10mm
$Ø_s=Ø^*s(f_{ctm}/2.9)[k_c \cdot h_{cr}/2(h-d)]$, Øs=10mm, Ø*=8mm, (fctm=2.60, hcr=250mm) (EC2 Eq.7.6N)
Steel bar diameter Φ*=8mm, crack width wk=0.3mm, steel stress $\sigma_s=360N/mm^2$ (EC2 Table 7.2N)
Min. reinforcement for wk=0.3mm and Ø=10mm, $A_{s,min}=0.40x0.86x2.60x250x87/360=54mm^2=0.54cm^2$

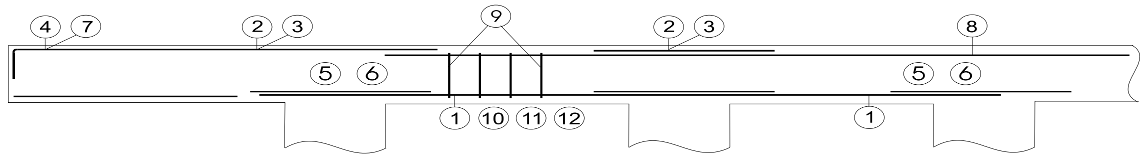
5.7. Serviceability limit state, deflection control (EC2 EN1992-1-1:2004, §7.4.2)

Span/effective depth, must be L/d<=limit of EC2 Table 7.4N
Span-1, K=1.30, ρ=0.274%, L/d= 3600/ 465= 7.74<26.00 (EC2 T.7.4N)
Span-2, K=1.30, ρ=0.274%, L/d= 3600/ 465= 7.74<26.00

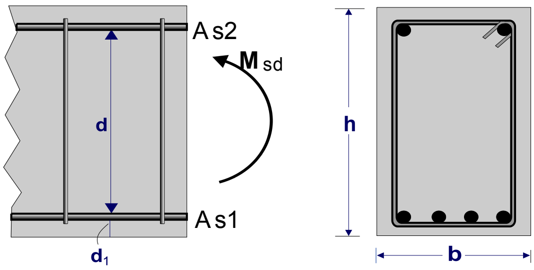
5.8. Reinforcing bar schedule

Num		type	reinforcing bar [mm]	items	Ø	g/m [kg/m]	length [m]	weight [kg]

Total weight [kg] **0.00**



6. BEAM-006



Moment capacity of beam section
(EC2 EN1992-1-1:2004, EC0 EN1990-1-1:2002)

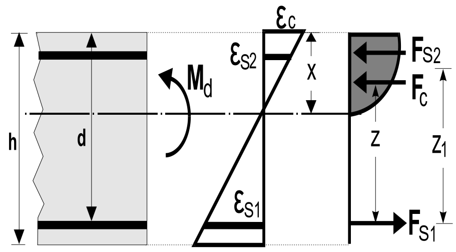
bw=0.250 m, h =0.500 m
As1=4Ø14 (6.16cm²) , **As2=2Ø14** (3.08cm²)
Concrete-Steel class: C25/30-S500 (EC2 §3)
Concrete cover : Cnom=20 mm (EC2 §4.4.1)
γc=1.50, γs=1.15 (EC2 Table 2.1N)

6.1. Dimensions and loads

Beam cross section bw=0.250 m, h=0.500 m
Bottom reinforcement 4Ø14 (6.16cm²)
Top reinforcement 2Ø14 (3.08cm²)
Effective depth of cross section d1=Cnomc+Øs+0.5Ø=20+8+0.5x14=35mm, d=500-35=465mm

6.2. Cross section moment capacity

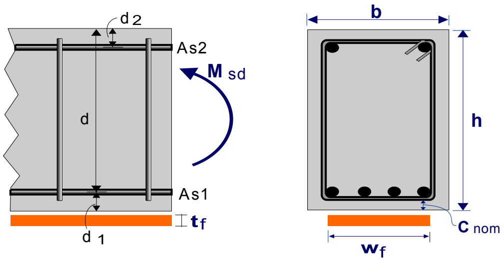
(EC2 EN1992-1-1:2004, §6.1)



(iterations:12). From internal force equilibrium we have:
εc=3.50(o/oo), Fc=α·0.85fcd·b·x, α=0.810, x=60.7mm, x/d=0.13
Fc =-α·0.85fcd·b·x =0.001x0.810x0.85x16.67x 250x 60.7= -174kN
εs1=20.00(o/oo)>2.17=εy, Fs1= As1·fyd=0.001x 616x435.0= 268kN
εs2= 1.48(o/oo)<2.17=εy, Fs2=-As2·Es·εs2= 308x200x1.48= -91kN
z=d-Ka·x, Ka=0.416, z=465-0.416x60.75=436mm
z1=(zFc+(d-d2)Fs2)/(Fc+Fs2)=(436x174+430x91)/(174+91)=434mm
Moment capacity of cross section Md=z1·Fs=0.434x268= 116.31kNm

Ultimate moment capacity of beam cross section Md= 116.31 kNm

7. BEAM-007



Moment capacity of beam section with FRP strengthening
(EC2 EN1992-1-1:2004, EC0 EN1990-1-1:2002)

bw=0.250 m, h =0.500 m
As1=4Ø14 (6.16cm²) , **As2=2Ø14** (3.08cm²)
FRP+epoxy, t(FRP)= 1.00 mm
Concrete-Steel class: C25/30-S500 (EC2 §3)
Concrete cover : Cnom=20 mm (EC2 §4.4.1)
γc=1.50, γs=1.15 (EC2 Table 2.1N)

7.1. Dimensions and loads

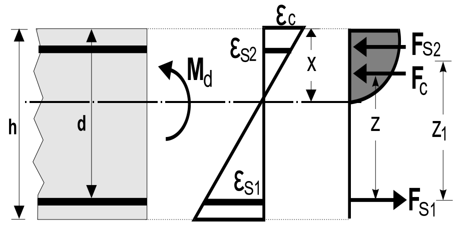
Beam cross section bw=0.250 m, h=0.500 m
Bottom reinforcement 4Ø14 (6.16cm²)
Top reinforcement 2Ø14 (3.08cm²)
Effective depth of cross section d1=Cnomc+Øs+0.5Ø=20+8+0.5x14=35mm, d=500-35=465mm

Fibre Reinforced Polymer material (FRP)

Characteristic name : FRP+epoxy
Total thickness : 1.00 mm
Modulus of elasticity : 100 GPa
Tensile strength : 1000 MPa
Cross section area : 250x1.00= 250 mm²

7.2. Cross section moment capacity, without FRP strengthening

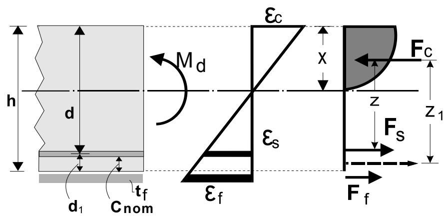
(EC2 EN1992-1-1:2004, §6.1)



(iterations:12). From internal force equilibrium we have:
εc=3.50(o/oo), Fc=α·0.85fcd·b·x, α=0.810, x=60.7mm, x/d=0.13
Fc =-α·0.85fcd·b·x =0.001x0.810x0.85x16.67x 250x 60.7= -174kN
εs1=20.00(o/oo)>2.17=εy, Fs1= As1·fyd=0.001x 616x435.0= 268kN
εs2= 1.48(o/oo)<2.17=εy, Fs2=-As2·Es·εs2= 308x200x1.48= -91kN
z=d-Ka·x, Ka=0.416, z=465-0.416x60.75=436mm
z1=(zFc+(d-d2)Fs2)/(Fc+Fs2)=(436x174+430x91)/(174+91)=434mm
Moment capacity of cross section Md=z1·Fs=0.434x268= 116.31kNm

7.3. Cross section moment capacity, with FRP strengthening

(EC2 EN1992-1-1:2004, §6.1)



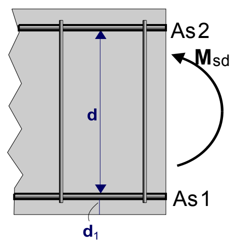
(iterations:5). From internal force equilibrium we have:
εc=3.50(o/oo), Fc=α·0.85fcd·b·x, α=0.810, x=132.0mm, x/d=0.28
Fc =-α·0.85fcd·b·x =0.001x0.810x0.85x16.67x 250x132.0= -378kN
εs1= 8.83(o/oo)>2.17=εy, Fs1= As1·fyd=0.001x 616x435.0= 268kN
εs2= 2.57(o/oo)>2.17=εy, Fs2=-As2·fyd=0.001x 308x435.0= -134kN
εf+εfo=9.78(o/oo), εf=9.78 , σf=Ef·εf=100x9.78=978MPa
σf= 978MPa<1000(tensile strength) Ff=Af·σf= 250x 978= 244kN
z=d-Ka·x, Ka=0.416, z=465-0.416x131.95=410mm
z1=((d-z-d2)·Fs2+(d1+tf/2)·Ff)/(Fs+Ff)+z)=(20x134+36x244)/512+410=432mm
Moment capacity of cross section Md=z1·(Fs+Ff)=0.432x(268+244)=221.18kNm

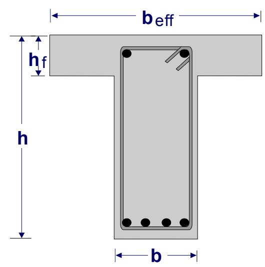
Ultimate moment capacity of beam cross section Md= 221.18 kNm

7.4. Increase of beam shear strength

FRP strengthening jacket on the vertical beam faces of thickness 1.000 mm
(assumed effective design strain εf=0.002, shape coefficient a=2.00)
Vsf=a·εf·Ef·tf·h·cotθ=2.00x0.002x100.0x1.000x500x1=200kN, **Vsf=200kN**

8. BEAM-008





Moment capacity of T beam section
(EC2 EN1992-1-1:2004, EC0 EN1990-1-1:2002)

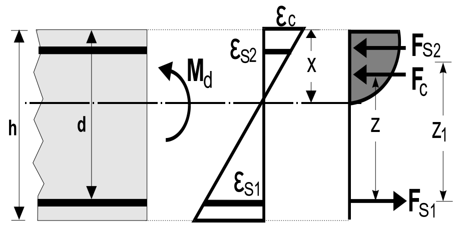
bw=0.200 m, h =0.600 m
beff=1.250 m, hf=0.180 m
As1=4Ø14 (6.16cm²), **As2=2Ø14** (3.08cm²)
Concrete-Steel class: C25/30-S500 (EC2 §3)
Concrete cover : Cnom=25 mm (EC2 §4.4.1)
γc=1.50, γs=1.15 (EC2 Table 2.1N)

8.1. Dimensions and loads

Beam cross section bw=0.200 m, h=0.600 m, beff=1.250 m, hf=0.180 m
Bottom reinforcement 4Ø14 (6.16cm²)
Top reinforcement 2Ø14 (3.08cm²)
Effective depth of cross section d1=Cnomc+Øs+0.5Ø=25+8+0.5x14=40mm, d=600-40=560mm

8.2. Cross section moment capacity (EC2 EN1992-1-1:2004, §6.1)

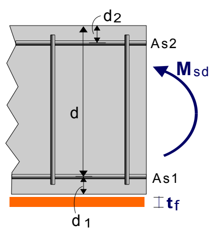
T beam cross section, neutral axis within the depth of top flange x=27.8<=h=180.0mm
(iterations:3). From internal force equilibrium we have:
εc=1.43(o/oo), Fc=α·0.85fcd·b·x, α=0.545, x=27.8mm, x/d=0.05
Fc =-α·0.85fcd·b·x =0.001x0.545x0.85x16.67x1250x 27.8= -268kN
εs1=20.00(o/oo)>2.17=εy, Fs1= As1·fyd=0.001x 616x435.0= 268kN
εs2= 0.00(o/oo)<2.17=εy, Fs2=-As2·Es·εs2= 308x200x0.00= 0kN
z=d-Ka·x, Ka=0.359, z=560-0.359x27.78=547mm
z1=(zFc+(d-d2)Fs2)/(Fc+Fs2)=(547x268+520x0)/(268+0)=547mm
Moment capacity of cross section Md=z1·Fs=0.547x268= 146.60kNm

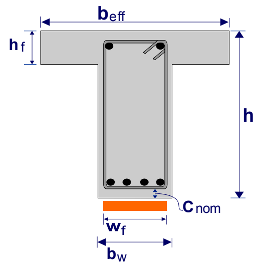


Ultimate moment capacity of beam cross section Md= 146.60 kNm

9. BEAM-009

Moment capacity of T beam section with FRP strengthening
(EC2 EN1992-1-1:2004, EC0 EN1990-1-1:2002)





bw=0.250 m, h =0.700 m
beff=1.100 m, hf=0.200 m
As1=4Ø14 (6.16cm²) , **As2=2Ø14** (3.08cm²)
FRP+epoxy, t(FRP)= 1.00 mm
Concrete-Steel class: C25/30-S500 (EC2 §3)
Concrete cover : Cnom=20 mm (EC2 §4.4.1)
γc=1.50, γs=1.15 (EC2 Table 2.1N)

9.1. Dimensions and loads

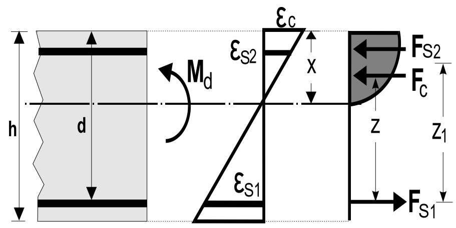
Beam cross section bw=0.250 m, h=0.700 m, beff=1.100 m, hf=0.200 m
Bottom reinforcement 4Ø14 (6.16cm²)
Top reinforcement 2Ø14 (3.08cm²)
Effective depth of cross section d1=Cnomc+Øs+0.5Ø=20+8+0.5x14=35mm, d=700-35=665mm

Fibre Reinforced Polymer material (FRP)

Characteristic name : FRP+epoxy
Total thickness : 1.00 mm
Modulus of elasticity : 100 GPa
Tensile strength : 1000 MPa
Cross section area : 250x1.00= 250 mm²

9.2. Cross section moment capacity, without FRP strengthening (EC2 EN1992-1-1:2004, §6.1)

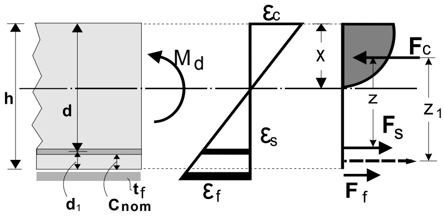
T beam cross section, neutral axis within the depth of top flange x=31.6<=h=200.0mm
(iterations:3). From internal force equilibrium we have:
εc=1.43(o/oo), Fc=α·0.85fcd·b·x, α=0.545, x=31.6mm, x/d=0.05
Fc =-α·0.85fcd·b·x =0.001x0.545x0.85x16.67x1100x 31.6= -268kN
εs1=20.00(o/oo)>2.17=εy, Fs1= As1·fyd=0.001x 616x435.0= 268kN
εs2= 0.00(o/oo)<2.17=εy, Fs2=-As2·Es·εs2= 308x200x0.00= 0kN
z=d-Ka·x, Ka=0.359, z=665-0.359x31.57=649mm
z1=(zFc+(d-d2)Fs2)/(Fc+Fs2)=(649x268+630x0)/(268+0)=649mm
Moment capacity of cross section Md=z1·Fs=0.649x268= 173.93kNm



9.3. Cross section moment capacity, with FRP strengthening

(EC2 EN1992-1-1:2004, §6.1)

T beam cross section, neutral axis within the depth of top flange x=39.2<=h=200.0mm
(iterations:12). From internal force equilibrium we have:
εc=3.50(o/oo), Fc=α·0.85fcd·b·x, α=0.810, x=39.2mm, x/d=0.06
Fc =-α·0.85fcd·b·x =0.001x0.810x0.85x16.67x1100x 39.2= -495kN
εs1=20.00(o/oo)>2.17=εy, Fs1= As1·fyd=0.001x 616x435.0= 268kN
εs2= 0.38(o/oo)<2.17=εy, Fs2=-As2·Es·εs2= 308x200x0.38= -23kN
εf+εfo=10.00(o/oo), εf=10.00 , σf=Ef·εf=100x10.00=1000MPa
σf=1000MPa<1000(tensile strength) Ff=Af·σf= 250x1000= 250kN
z=d-Ka·x, Ka=0.416, z=665-0.416x39.21=624mm
z1=((d-z-d2)·Fs2+(d1+tf/2)·Ff)/(Fs+Ff)+z)=(6x23+36x250)/518+624=641mm
Moment capacity of cross section Md=z1·(Fs+Ff)=0.641x(268+250)=332.04kNm



Ultimate moment capacity of beam cross section Md= 332.04 kNm

9.4. Increase of beam shear strength

FRP strengthening jacket on the vertical beam faces of thickness 1.000 mm
(assumed effective design strain εf=0.002, shape coefficient a=2.00)
Vsf=a·εf·Ef·tf·h·cotθ=2.00x0.002x100.0x1.000x500x1=200kN, **Vsf=200kN**

Reinforcing bar schedule

Num	Structure object	type	reinforcing bar [mm]	items	∅	g/m [kg/m]	length [m]	weight [kg]
1	BEAM-005(Span-1)	B11	130 3890	4	10	0.617	4.020	9.92
2	BEAM-005(Span-1)	B8	3760	2	10	0.617	3.760	4.64
3	BEAM-005(Span-2)	B12	3890 130	4	10	0.617	4.020	9.92
4	BEAM-005(Span-2)	B8	3760	2	10	0.617	3.760	4.64
5	BEAM-005(Supp-1)	B2	2380	4	10	0.617	2.380	5.87
6	BEAM-005(Supp-1)	B2	2500	1	12	0.888	2.500	2.22
7	BEAM-005(Span-1)	B9	80 200 80 440 200 440	10	8	0.395	1.440	5.69
8	BEAM-005(Span-2)	B9	80 200 80 440 200 440	10	8	0.395	1.440	5.69
9	BEAM-004(Span-1)	B10	290 6160 290	4	18	2.000	6.740	53.92
10	BEAM-004(Span-1)	B10	330 6160 330	2	20	2.470	6.820	33.69
11	BEAM-004(Span-1)	B8	6160	2	14	1.210	6.160	14.91
12	BEAM-004(Span-1)	B9	80 200 80 420 200 420	21	8	0.395	1.400	11.61
13	BEAM-004(Cant2)	B9	80 200 80 440 200 440	0	8	0.395	1.440	0.00
			Total weight [kg]					**162.72**