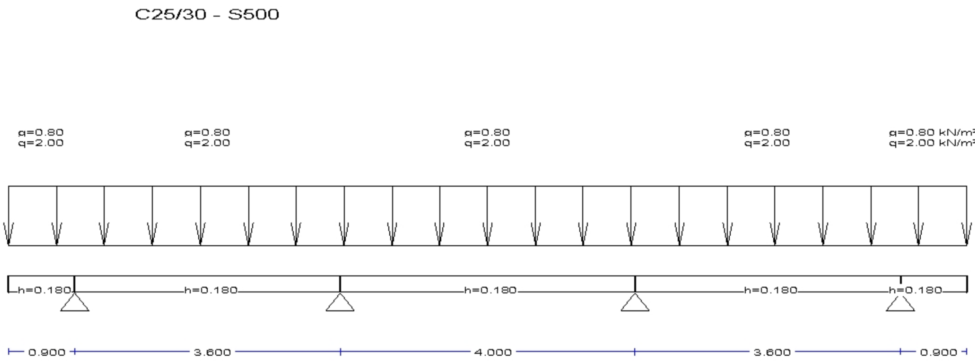
CONTENTS

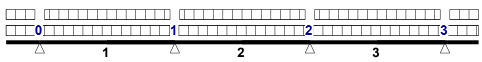
Design examples

1. SLAB-001

One-way continuous slab
(EC2 EN1992-1-1:2004, EC0 EN1990-1-1:2002)



Concrete-Steel class: C25/30-S500 (EC2 §3)
Concrete cover : Cnom=15 mm (EC2 §4.4.1)
Concrete weight : 25.0 kN/m^3
γ_c=1.50, γ_s=1.15 (EC2 Table 2.1N)

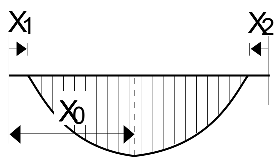


1.1. Dimensions and loads

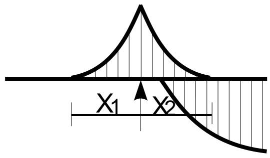
Continuous slab, number of spans=3, transverse length Ly=9.00 m
Partial safety factors for actions : γ_G=1.35, γ_Q=1.50 (EC0 Annex A1)
Combination of variable actions : ψ_1=0.60, ψ_2=0.30
Effective depth of cross section d=h-d1, d1=Cnomc+Ø/2=15+10/2=20mm

Spans (L), thickness (h), loads on spans (g=self weight +dead, q=live)
Cant-1, L= 0.900 m, h= 0.180 m, g=(4.50+ 0.80)x1.000= 5.30 kN/m^2, q= 2.00x1.000= 2.00 kN/m^2
Span-1, L= 3.600 m, h= 0.180 m, g=(4.50+ 0.80)x1.000= 5.30 kN/m^2, q= 2.00x1.000= 2.00 kN/m^2
Span-2, L= 4.000 m, h= 0.180 m, g=(4.50+ 0.80)x1.000= 5.30 kN/m^2, q= 2.00x1.000= 2.00 kN/m^2
Span-3, L= 3.600 m, h= 0.180 m, g=(4.50+ 0.80)x1.000= 5.30 kN/m^2, q= 2.00x1.000= 2.00 kN/m^2
Cant-2, L= 0.900 m, h= 0.180 m, g=(4.50+ 0.80)x1.000= 5.30 kN/m^2, q= 2.00x1.000= 2.00 kN/m^2

1.2. Shearing forces and bending moments



Maximum bending moments at spans for load combinations 1.35g+1.50q
Cant-1, Msd= -4.11 kNm/m, xo=0.900 m, x1=0.000m, x2=0.000m
Span-1, Msd= 9.83 kNm/m, xo=1.536 m, x1=0.144m, x2=0.672m
Span-2, Msd= 7.82 kNm/m, xo=2.000 m, x1=0.759m, x2=0.759m
Span-3, Msd= 9.83 kNm/m, xo=2.064 m, x1=0.672m, x2=0.144m
Cant-2, Msd= -4.11 kNm/m, xo=0.000 m, x1=0.000m, x2=0.000m



Maximum bending moments at supports for load combinations 1.35g+1.50q

Support-0, Msd= -4.11 kNm/m, x1=0.900 m, x2=0.296 m
Support-1, Msd= -16.37 kNm/m, x1=0.953 m, x2=1.033 m
Support-2, Msd= -16.37 kNm/m, x1=1.033 m, x2=0.953 m
Support-3, Msd= -4.11 kNm/m, x1=0.296 m, x2=0.900 m

Maximum shear forces for load combinations 1.35g+1.50q

Cant-1, Vsd,left= 0.00 kN/m, Vsd,right= -9.14 kN/m
Span-1, Vsd,left= 14.54 kN/m, Vsd,right= -22.02 kN/m
Span-2, Vsd,left= 19.53 kN/m, Vsd,right= -21.09 kN/m
Span-3, Vsd,left= 21.16 kN/m, Vsd,right= -15.40 kN/m
Cant-2, Vsd,left= 9.14 kN/m, Vsd,right= 0.00 kN/m

Maximum reactions due to dead and live loads (Rg and Rq)

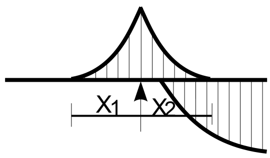
Support-0, Rg(x1.35)= 16.96 kN/m, Rq(x1.50)= 7.58 kN/m
Support-1, Rg(x1.35)= 29.55 kN/m, Rq(x1.50)= 13.56 kN/m
Support-2, Rg(x1.35)= 29.55 kN/m, Rq(x1.50)= 13.56 kN/m
Support-3, Rg(x1.35)= 16.96 kN/m, Rq(x1.50)= 7.58 kN/m

1.3. 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

Cant-1, Msd= -3.20 kNm/m, Vsd,left= 0.00 kN/m, Vsd,right= -9.14 kN/m
Span-1, Msd= 9.83 kNm/m, Vsd,left= 15.60 kN/m, Vsd,right= -20.96 kN/m
Span-2, Msd= 7.82 kNm/m, Vsd,left= 20.31 kN/m, Vsd,right= -20.31 kN/m
Span-3, Msd= 9.83 kNm/m, Vsd,left= 20.96 kN/m, Vsd,right= -15.60 kN/m
Cant-2, Msd= -3.20 kNm/m, Vsd,left= 9.14 kN/m, Vsd,right= 0.00 kN/m



Maximum bending moments at supports for load combinations 1.35g+1.50q

Support-0, Msd= -3.20 kNm/m, x1=0.900 m, x2=0.296 m
Support-1, Msd= -14.34 kNm/m, x1=0.953 m, x2=1.033 m
Support-2, Msd= -14.34 kNm/m, x1=1.033 m, x2=0.953 m
Support-3, Msd= -3.20 kNm/m, x1=0.296 m, x2=0.900 m

1.4. Ultimate limit state, design for bending (EC2 EN1992-1-1:2004, §6.1, §9.3.1)

Reinforcement of spans

Msd1= 9.83kNm/m, d=160mm, Kd=5.10 x/d=0.06 εc/εs=1.2/20.0 ks=2.35, **As= 1.44 cm^2/m**
Msd2= 7.82kNm/m, d=160mm, Kd=5.72 x/d=0.05 εc/εs=1.1/20.0 ks=2.34, **As= 1.14 cm^2/m**
Msd3= 9.83kNm/m, d=160mm, Kd=5.10 x/d=0.06 εc/εs=1.2/20.0 ks=2.35, **As= 1.44 cm^2/m**

Reinforcement over supports

Msd0= -3.20kNm/m, d=160mm, Kd=8.95 x/d=0.03 εc/εs=0.6/20.0 ks=2.32, **As= 0.46 cm^2/m**
Msd1=-14.34kNm/m, d=160mm, Kd=4.23 x/d=0.07 εc/εs=1.5/20.0 ks=2.36, **As= 2.11 cm^2/m**
Msd2=-14.34kNm/m, d=160mm, Kd=4.23 x/d=0.07 εc/εs=1.5/20.0 ks=2.36, **As= 2.11 cm^2/m**
Msd3= -3.20kNm/m, d=160mm, Kd=8.95 x/d=0.03 εc/εs=0.6/20.0 ks=2.32, **As= 0.46 cm^2/m**

1.5. Minimum reinforcement at spans (EC2 EN1992-1-1:2004, §9.3.1)

Span-1, As>=0.26bd·Fctm/fyk (As= 2.16 cm^2/m) Ø10/36.0(2.18 cm^2/m), second. Ø8/45.0(1.12 cm^2/m)
Span-2, As>=0.26bd·Fctm/fyk (As= 2.16 cm^2/m) Ø10/36.0(2.18 cm^2/m), second. Ø8/45.0(1.12 cm^2/m)
Span-3, As>=0.26bd·Fctm/fyk (As= 2.16 cm^2/m) Ø10/36.0(2.18 cm^2/m), second. Ø8/45.0(1.12 cm^2/m)

1.6. Serviceability limit state, deflection control

(EC2 EN1992-1-1:2004, §7.4.2)

(EC2 T.7.4N)

```
Cant-1, K=0.40, ρ=0.029%, L/d=  900/ 160= 5.62< 8.00
Span-1, K=1.50, ρ=0.090%, L/d= 3600/ 160=22.50<30.00
Span-2, K=1.50, ρ=0.072%, L/d= 4000/ 160=25.00<30.00
Span-3, K=1.50, ρ=0.090%, L/d= 3600/ 160=22.50<30.00
Cant-2, K=0.40, ρ=0.029%, L/d=  900/ 160= 5.62< 8.00
```

1.7. Reinforcement

Span reinforcement

Cant-1 **Ø10/36.0** (2.18 cm^2/m) principal at top, **Ø8/45.0** (1.12 cm^2/m) secondary
Span-1 **Ø10/36.0** (2.18 cm^2/m) principal at bottom, **Ø8/45.0** (1.12 cm^2/m) secondary
Span-2 **Ø10/36.0** (2.18 cm^2/m) principal at bottom, **Ø8/45.0** (1.12 cm^2/m) secondary
Span-3 **Ø10/36.0** (2.18 cm^2/m) principal at bottom, **Ø8/45.0** (1.12 cm^2/m) secondary
Cant-2 **Ø10/36.0** (2.18 cm^2/m) principal at top, **Ø8/45.0** (1.12 cm^2/m) secondary

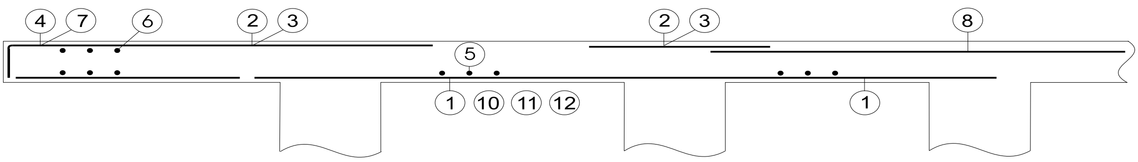
Reinforcement over supports

Support-0 **Ø10/36.0** (2.18 cm^2/m) reinforcement at top
Support-1 **Ø10/36.0** (2.18 cm^2/m) reinforcement at top
Support-2 **Ø10/36.0** (2.18 cm^2/m) reinforcement at top
Support-3 **Ø10/36.0** (2.18 cm^2/m) reinforcement at top

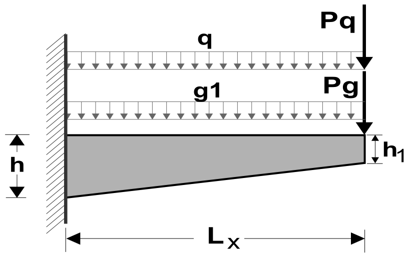
1.8. Reinforcing bar schedule

Num		type	reinforcing bar [mm]	items	Ø	g/m [kg/m]	length [m]	weight [kg]
1	(Span-1)	1	4020	25	10	0.617	4.020	62.01
2	(Span-2)	1	4420	25	10	0.617	4.420	68.18
3	(Span-3)	1	4020	25	10	0.617	4.020	62.01
4	(Cant-1)	4	150 2090	25	10	0.617	2.240	34.55
5	(Supp-1)	2	2570	25	10	0.617	2.570	39.64
6	(Supp-2)	2	2570	25	10	0.617	2.570	39.64
7	(Cant-2)	4	2090 160	25	10	0.617	2.250	34.71
8	(Cant-1)	5	9000	2	8	0.395	9.000	7.11
9	(Cant-1)	6	9000	2	8	0.395	9.000	7.11
10	(Span-1)	5	9000	8	8	0.395	9.000	28.44
11	(Span-2)	5	9000	9	8	0.395	9.000	32.00
12	(Span-3)	5	9000	8	8	0.395	9.000	28.44
13	(Cant-2)	5	9000	2	8	0.395	9.000	7.11
14	(Cant-2)	6	9000	2	8	0.395	9.000	7.11

Total weight [kg] 458.06



2. SLAB-002



One way cantilever slab

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

Concrete-Steel class: C25/30-S500 (EC2 §3)
Concrete cover : Cnom=15 mm (EC2 §4.4.1)
Concrete weight : 25.0 kN/m³
γc=1.50, γs=1.15 (EC2 Table 2.1N)

2.1. Dimensions and loads

Cantilever slab, free span Lx=1.200 m, transverse length Ly=4.800 m
Slab thickness, at support h=0.180 m, at free end h1=0.180 m
Slab loads: dead g=(4.50+0.80)=5.30 kN/m², live q=2.00 kN/m²
concentrated loads at free end, Pg=0.00 kN/m, Pq=0.00 kN/m
Partial safety factors for actions : γG=1.35, γQ=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+Ø/2=15+10/2=20mm, d=180-20=160mm

2.2. Ultimate limit state, design for bending

(EC2 EN1992-1-1:2004, §6.1, §9.3.1)

Support moment M=-0.5x(1.35x5.30+1.50x2.00)x1.20²=-7.31 kNm/m
Shear force V=(1.35x5.30+1.50x2.00)x1.20=12.19 kN/m
Reaction VgA=1.35x5.30x1.20=8.59 kN/m, VqA=1.50x2.00x1.20=3.60 kN/m

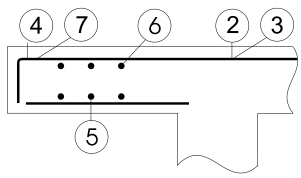
Reinforcement of slab (EC2 EN1992-1-1:2004, §6.1, §9.3.1)

Msd= -7.31kNm/m, d=160mm, Kd=5.92 x/d=0.05 $\varepsilon c/\varepsilon s$=1.0/20.0 ks=2.34, **As= 1.07cm²/m**
Minimum slab reinforcement, As>=0.26bd·Fctm/fyk (As= 2.16cm²/m) (EC2 §9.3.1)
minimum principal reinforcement Ø10/36.0(2.18cm²/m), secondary Ø8/45.0(1.12cm²/m)
span/eff. depth, K=0.40, ρ=0.067%, L/d= 1200/ 160= 7.50< 8.00 (EC2 T.7.4N)

Main reinforcement Ø10/36.0(2.18cm²/m) **(top), secondary Ø8/45.0**(1.12cm²/m) **(top- bottom**

2.3. Reinforcing bar schedule

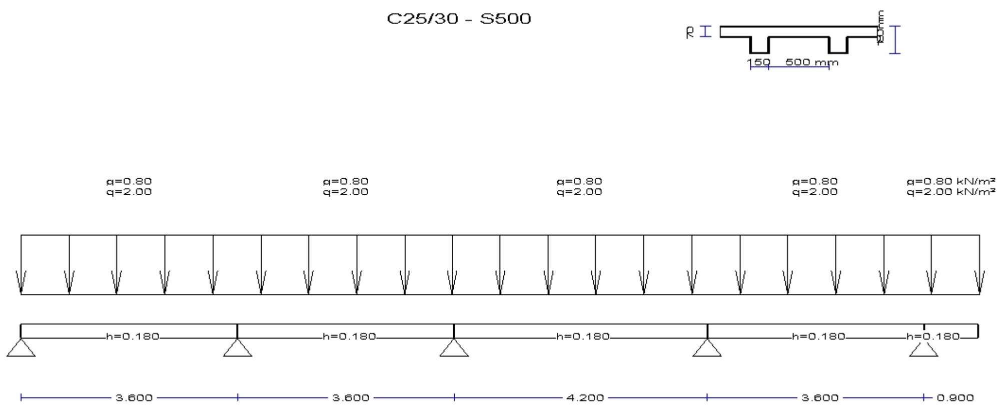
Num	type	reinforcing bar [mm]	items	Ø	g/m [kg/m]	length [m]	weight [kg]
15	4	150 2660	7	10	0.617	2.810	12.14
16	2	1780	7	10	0.617	1.780	7.69
17	5	4770	3	8	0.395	4.770	5.65
18	6	4770	3	8	0.395	4.770	5.65
		Total weight [kg]					**31.13**

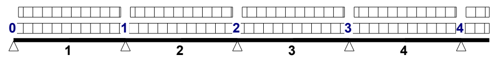


3. SLAB-003

One-way continuous ribbed slab

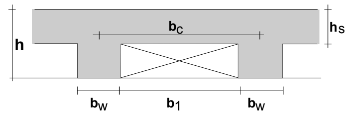
(EC2 EN1992-1-1:2004, EC0 EN1990-1-1:2002)





```
Concrete-Steel class: C25/30-S500          (EC2 §3)
Concrete cover      : Cnom=15 mm        (EC2 §4.4.1)
Concrete weight     : 25.0 kN/m³
γc=1.50, γs=1.15                     (EC2 Table 2.1N)
```

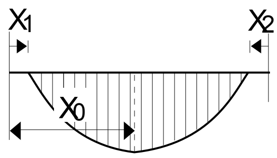
3.1. Dimensions and loads



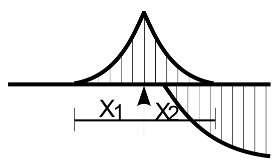

```
Continuous slab, number of spans=4, transverse length Ly=9.00 m
Partial safety factors for actions : γG=1.35, γQ=1.50          (EC0 Annex A1)
Combination of variable actions    : ψ1=0.60, ψ2=0.30
Thickness of top solid slab hs=0.070 m
Rib width bw=0.150m, clear distance between ribs b1=0.500m, rib spacing bc=0.650m
Effective depth of cross section d=h-d1, d1=Cnomc+Ø/2=15+10/2=20mm

Spans (L), thickness (h), loads on spans (g=self weight +dead, q=live)
Span-1, L= 3.600 m, h= 0.180 m, g=( 2.38+ 0.80)x1.000= 3.18 kN/m², q= 2.00x1.000= 2.00 kN/m²
Span-2, L= 3.600 m, h= 0.180 m, g=( 2.38+ 0.80)x1.000= 3.18 kN/m², q= 2.00x1.000= 2.00 kN/m²
Span-3, L= 4.200 m, h= 0.180 m, g=( 2.38+ 0.80)x1.000= 3.18 kN/m², q= 2.00x1.000= 2.00 kN/m²
Span-4, L= 3.600 m, h= 0.180 m, g=( 2.38+ 0.80)x1.000= 3.18 kN/m², q= 2.00x1.000= 2.00 kN/m²
Cant-2, L= 0.900 m, h= 0.180 m, g=( 2.38+ 0.80)x1.000= 3.18 kN/m², q= 2.00x1.000= 2.00 kN/m²
```

3.2. Shearing forces and bending moments




```
Maximum bending moments at spans for load combinations 1.35g+1.50q
Span-1, Msd=   8.55 kNm/m, xo=1.531 m, x1=0.000m, x2=0.537m
Span-2, Msd=   5.49 kNm/m, xo=1.858 m, x1=0.630m, x2=0.515m
Span-3, Msd=   7.71 kNm/m, xo=2.014 m, x1=0.560m, x2=0.733m
Span-4, Msd=   7.16 kNm/m, xo=2.078 m, x1=0.677m, x2=0.121m
Cant-2, Msd=  -2.95 kNm/m, xo=0.000 m, x1=0.000m, x2=0.000m
```




```
Maximum bending moments at supports for load combinations 1.35g+1.50q
Support-0, Msd=   0.00 kNm/m, x1=0.000 m, x2=0.000 m
Support-1, Msd= -10.27 kNm/m, x1=0.782 m, x2=0.951 m
Support-2, Msd=  -9.81 kNm/m, x1=0.932 m, x2=0.793 m
Support-3, Msd= -12.98 kNm/m, x1=0.993 m, x2=1.043 m
Support-4, Msd=  -2.95 kNm/m, x1=0.300 m, x2=0.900 m
```

Maximum shear forces for load combinations 1.35g+1.50q

Span-1,	Vsd,left= 10.27 kN/m,	Vsd,right= -15.98 kN/m
Span-2,	Vsd,left= 12.33 kN/m,	Vsd,right= -13.93 kN/m
Span-3,	Vsd,left= 13.94 kN/m,	Vsd,right= -16.70 kN/m
Span-4,	Vsd,left= 15.32 kN/m,	Vsd,right= -10.94 kN/m
Cant-2,	Vsd,left= 6.56 kN/m,	Vsd,right= 0.00 kN/m

Maximum reactions due to dead and live loads (Rg and Rq)

Support-0,	Rg(x1.35)= 6.13 kN/m,	Rq(x1.50)= 4.68 kN/m
Support-1,	Rg(x1.35)= 17.28 kN/m,	Rq(x1.50)= 12.96 kN/m
Support-2,	Rg(x1.35)= 15.81 kN/m,	Rq(x1.50)= 13.38 kN/m
Support-3,	Rg(x1.35)= 19.12 kN/m,	Rq(x1.50)= 13.83 kN/m
Support-4,	Rg(x1.35)= 9.91 kN/m,	Rq(x1.50)= 7.59 kN/m

3.3. 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)

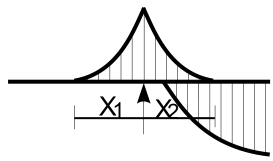
Check for minimum values, ($0.65ql^2/8$ or $0.65ql^2/12$) (EC2 §5.3.2.2.3N)

Maximum span bending moments and shear forces for load combinations 1.35g+1.50q

Span-1,	Msd= 8.55 kNm/m,	Vsd,left= 11.17 kN/m,	Vsd,right= -15.09 kN/m
Span-2,	Msd= 5.49 kNm/m,	Vsd,left= 13.55 kN/m,	Vsd,right= -12.71 kN/m
Span-3,	Msd= 7.71 kNm/m,	Vsd,left= 14.68 kN/m,	Vsd,right= -15.95 kN/m
Span-4,	Msd= 7.16 kNm/m,	Vsd,left= 15.15 kN/m,	Vsd,right= -11.10 kN/m
Cant-2,	Msd= -2.30 kNm/m,	Vsd,left= 6.56 kN/m,	Vsd,right= 0.00 kN/m

Maximum bending moments at supports for load combinations 1.35g+1.50q

Support-0,	Msd= 0.00 kNm/m,	x1=0.000 m,	x2=0.000 m
Support-1,	Msd= -8.92 kNm/m,	x1=0.782 m,	x2=0.951 m
Support-2,	Msd= -8.54 kNm/m,	x1=0.932 m,	x2=0.793 m
Support-3,	Msd= -11.46 kNm/m,	x1=0.993 m,	x2=1.043 m
Support-4,	Msd= -2.30 kNm/m,	x1=0.300 m,	x2=0.900 m



3.4. Ultimate limit state, design for bending (EC2 EN1992-1-1:2004, §6.1, §9.3.1)

Reinforcement of spans

Msd1= 8.55kNm/m, d=160mm, Kd=5.47 x/d=0.05 $\varepsilon c/\varepsilon s$=1.1/20.0 ks=2.34, **As= 1.25cm^2/m**
x=0.05x160= 8mm<= 70mm=hs, neutral axis within the depth of top flange

Msd2= 5.49kNm/m, d=160mm, Kd=6.83 x/d=0.04 $\varepsilon c/\varepsilon s$=0.9/20.0 ks=2.33, **As= 0.80cm^2/m**
x=0.04x160= 6mm<= 70mm=hs, neutral axis within the depth of top flange

Msd3= 7.71kNm/m, d=160mm, Kd=5.76 x/d=0.05 $\varepsilon c/\varepsilon s$=1.1/20.0 ks=2.34, **As= 1.13cm^2/m**
x=0.05x160= 8mm<= 70mm=hs, neutral axis within the depth of top flange

Msd4= 7.16kNm/m, d=160mm, Kd=5.98 x/d=0.05 $\varepsilon c/\varepsilon s$=1.0/20.0 ks=2.34, **As= 1.05cm^2/m**
x=0.05x160= 8mm<= 70mm=hs, neutral axis within the depth of top flange

Reinforcement over supports

Msd1= -8.92kNm/m, d=160mm, Kd=5.36 x/d=0.05 $\varepsilon c/\varepsilon s$=1.1/20.0 ks=2.34, **As= 1.31cm^2/m**

Msd2= -8.54kNm/m, d=160mm, Kd=5.47 x/d=0.05 $\varepsilon c/\varepsilon s$=1.1/20.0 ks=2.34, **As= 1.25cm^2/m**

Msd3=-11.46kNm/m, d=160mm, Kd=4.73 x/d=0.06 $\varepsilon c/\varepsilon s$=1.3/20.0 ks=2.35, **As= 1.68cm^2/m**

Msd4= -2.30kNm/m, d=160mm, Kd=10.56 x/d=0.03 $\varepsilon c/\varepsilon s$=0.5/20.0 ks=2.32, **As= 0.33cm^2/m**

3.5. Minimum reinforcement at spans (EC2 EN1992-1-1:2004, §9.3.1)

Span-1, As>=0.26bd·Fctm/fyk (As= 0.50cm^2/m) minimum reinforcement 1Ø10/65.0(1.21cm^2/m)

Span-2, As>=0.26bd·Fctm/fyk (As= 0.50cm^2/m) minimum reinforcement 1Ø10/65.0(1.21cm^2/m)

Span-3, As>=0.26bd·Fctm/fyk (As= 0.50cm^2/m) minimum reinforcement 1Ø10/65.0(1.21cm^2/m)

Span-4, As>=0.26bd·Fctm/fyk (As= 0.50cm^2/m) minimum reinforcement 1Ø10/65.0(1.21cm^2/m)

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

```
Span-1, K=1.30, ρ=0.078%, L/d= 3600/ 160=22.50<26.00        (EC2 T.7.4N)
Span-2, K=1.50, ρ=0.050%, L/d= 3600/ 160=22.50<30.00
Span-3, K=1.50, ρ=0.070%, L/d= 4200/ 160=26.25<30.00
Span-4, K=1.50, ρ=0.065%, L/d= 3600/ 160=22.50<30.00
Cant-2, K=0.40, ρ=0.021%, L/d=  900/ 160= 5.62< 8.00
```

3.7. Reinforcement

Span reinforcement

```
Span-1 1Ø12/ 65.0 ( 1.74cm²/m) principal at bottom,  2Ø12/180.0 ( 1.26cm²/m) secondary
Span-2 1Ø12/ 65.0 ( 1.74cm²/m) principal at bottom,  2Ø12/180.0 ( 1.26cm²/m) secondary
Span-3 1Ø12/ 65.0 ( 1.74cm²/m) principal at bottom,  2Ø12/210.0 ( 1.08cm²/m) secondary
Span-4 1Ø12/ 65.0 ( 1.74cm²/m) principal at bottom,  2Ø12/180.0 ( 1.26cm²/m) secondary
Cant-2  Ø10/36.0  ( 2.18cm²/m) principal at top,       Ø8/45.0  ( 1.12cm²/m) secondary
```

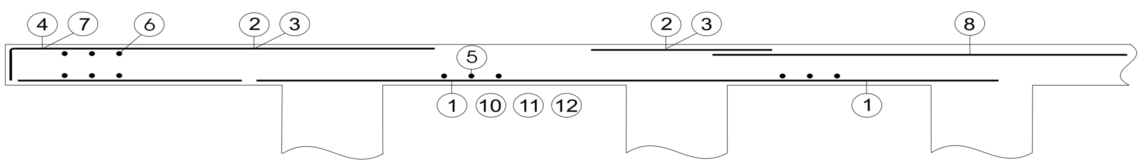
Reinforcement over supports

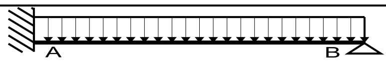
```
Support-0   Ø8/45.0  ( 1.12cm²/m) reinforcement at top
Support-1  Ø10/36.0  ( 2.18cm²/m) reinforcement at top
Support-2  Ø10/36.0  ( 2.18cm²/m) reinforcement at top
Support-3  Ø10/36.0  ( 2.18cm²/m) reinforcement at top
Support-4  Ø10/36.0  ( 2.18cm²/m) reinforcement at top
```

3.8. Reinforcing bar schedule

Num		type	reinforcing bar [mm]	items	Ø	g/m [kg/m]	length [m]	weight [kg]
19	(Span-1)	11	150 3930	14	12	0.888	4.080	50.72
20	(Span-2)	1	4100	14	12	0.888	4.100	50.97
21	(Span-3)	1	4700	14	12	0.888	4.700	58.43
22	(Span-4)	1	4100	14	12	0.888	4.100	50.97
23	(Supp-0)	4	100 1130	20	8	0.395	1.230	9.72
24	(Supp-1)	2	2380	25	10	0.617	2.380	36.71
25	(Supp-2)	2	2530	25	10	0.617	2.530	39.03
26	(Supp-3)	2	2620	25	10	0.617	2.620	40.41
27	(Cant-2)	4	2090 160	25	10	0.617	2.250	34.71
28	(Span-1)	5	9000	4	12	0.888	9.000	31.97
29	(Span-2)	5	9000	4	12	0.888	9.000	31.97
30	(Span-3)	5	9000	4	12	0.888	9.000	31.97
31	(Span-4)	5	9000	4	12	0.888	9.000	31.97
32	(Cant-2)	5	9000	4	8	0.395	9.000	14.22
33	(Cant-2)	6	9000	4	8	0.395	9.000	14.22

Total weight [kg] 527.99

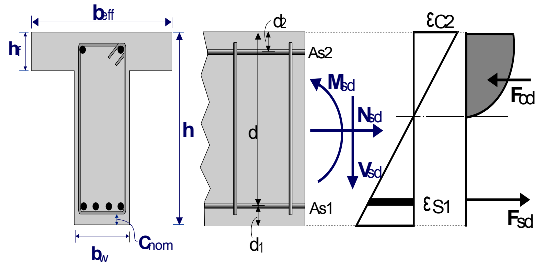




4. BEAM-001

One span beam in composite loading

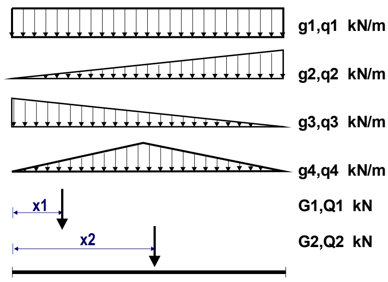
(EC2 EN1992-1-1:2004, EC0 EN1990-1-1:2002)



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

4.1. Dimensions and loads

Beam (T section symmetric) , span L=3.600 m
L=3.600m, bw=0.250m, h=0.500m hf=0.180m
Effective flange width beff=0.250+2x0.1x0.85x3.600=0.862m (EC2 §5.3.2.1)
Partial safety factors for actions : γG=1.35, γQ=1.50 (EC0 Annex A1)
Combination of variable actions : ψ1=0.60, ψ2=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= 2.00 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	

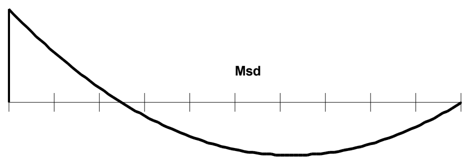
Cross section values (area A, moment of inertia Ixx, centroid yc)
Span-1 L= 3.600m, A=0.23516m²(2.35E+005mm²), Ixx=0.00440m4(4.40E+009mm4), yc=0.175m(175mm)

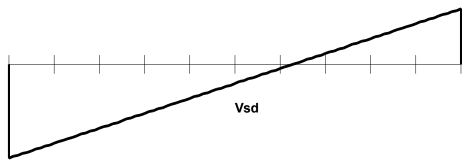
4.2. Design actions, shearing forces and bending moments

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

x/L=0.00,	x= 0.00m,	Msd= -92.10 kNm,	Vsd= 127.91 kN
x/L=0.10,	x= 0.36m,	Msd= -49.73 kNm,	Vsd= 107.45 kN
x/L=0.20,	x= 0.72m,	Msd= -14.74 kNm,	Vsd= 86.98 kN
x/L=0.30,	x= 1.08m,	Msd= 12.89 kNm,	Vsd= 66.51 kN
x/L=0.40,	x= 1.44m,	Msd= 33.15 kNm,	Vsd= 46.05 kN
x/L=0.50,	x= 1.80m,	Msd= 46.05 kNm,	Vsd= 25.58 kN
x/L=0.60,	x= 2.16m,	Msd= 51.57 kNm,	Vsd= 5.12 kN
x/L=0.70,	x= 2.52m,	Msd= 49.73 kNm,	Vsd= -15.35 kN
x/L=0.80,	x= 2.88m,	Msd= 40.52 kNm,	Vsd= -35.82 kN
x/L=0.90,	x= 3.24m,	Msd= 23.95 kNm,	Vsd= -56.28 kN
x/L=1.00,	x= 3.60m,	Msd= 0.00 kNm,	Vsd= -76.75 kN

*Zero moments values at x1=0.936 m and x2=3.600m





VsdA= 127.91 kN, VsdB= 76.75 kN, maxMsd= 92.10 kNm, maxVsd= 127.91 kN
Maximum moments: span Msd=51.80 kNm (x=2.232m), support MsdA=-92.10 kNm
Maximum moments at support face (bsup=0.200m) MsdA=-78.61 kNm
Maximum shear forces at distance d from support face
Span-A, b/2+d=0.565m, VsdA= 95.17kN, VsdB= 44.00kN

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+0.5Ø=20+8+0.5x14=35mm, d2=35mm, d=500-35=465mm
Reinforcement for bending (only tension reinforcement is needed)
Msd= 51.80kNm, beff=862mm, d=465mm, Kd=6.00 x/d=0.05 εc/εs=1.0/20.0 ks=2.34, **As1= 2.60cm²**
x=0.05x465=23<hf=180mm neutral axis within the depth of top flange
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 (3.14cm²) **(bottom)**

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

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=-78.61kNm, bw=250mm, d=465mm, Kd=2.62 x/d=0.14 εc/εs=3.2/20.0 ks=2.44, **As2= 4.12cm²**
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)**

4.5. 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)=314/(250x465)=0.0027
vmin=0.035·k^(1.50)·fck^½ = 0.37N/mm² (EC2 Eq.6.3N)
Vrd,c(min)=0.001x(0.37)x250x465=43.01kN
Vrdc=0.001x[0.120x1.66x(0.27x25.00)^(0.333)]x250x465=43.76kN
Vsd=95.17 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=95.2 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=95.17kN, z=0.9d, fywd=0.8fyk=400.00N/mm², cotθ=2.50
Asw/s=Vrds/(z·fywd·cot21.80°)=(1.0E+006)x95.17/(0.9x465x400x2.50)=227mm²/m (Asw/s= 2.27cm²/m)
Required shear reinforcement: (Asw/s= 2.27cm²/m)

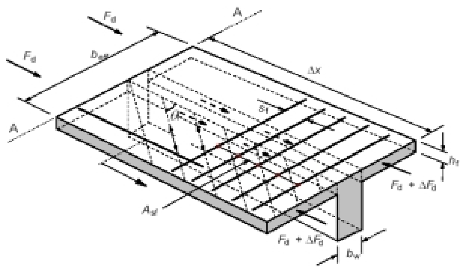
Minimum links for shear reinforcement (EC2 §9.2.2)
Minimum shear reinforcement ratio ρw,min (EC2 Eq.9.5N)
ρw,min=(0.08x(fck)^(0.5)/fyk, fck=25N/mm², fyk=500N/mm², ρw,min=0.0008
min Asw/s=10x0.0008x250xsin(90°)= 2.00cm²/m

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 Shear reinforcement: stirrups Ø8/34.5 (Asw/s= 2.92cm²/m)

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



Fc=Fs=0.001x260x435=113 kN
ΔFd=Fc·(beff-bw)/(2beff)=113.0x(862-250)/(2x862)=40kN
Beam span L=3.60 m, Δx=0.85x3.60/2=1.53 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=0.54x180x16.67xsin26.5°xcos26.5°=647kN/m (EC2 Eq.6.22)
ΔFd/Δx=40/1.53=26<=Vrdmax=647 kN/m, the check is verified
Transverse reinforcement per unit length Asf/sf (EC2 Eq.6.21)
Asf/sf=10x26/(435xcot26.5°)= 0.30cm^2/m
Transverse reinforcement Asf/sf=Ø8/34.5 (1.46cm^2/m)
ΔFd/Δx=26<=0.40·hf·Fctd=0.40x180x1.20=86 kN/m
In case of transverse flexural reinforcement from plate bending,
No extra reinforcement is needed (EC2 §6.2.4.6)

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

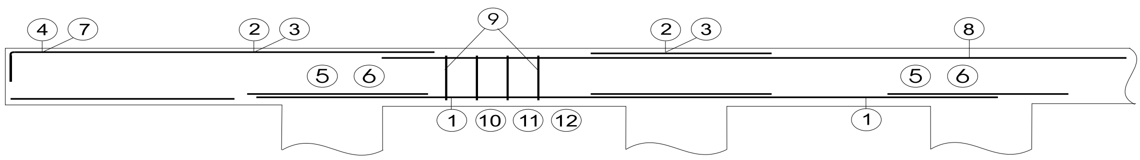
Minimum reinforcement areas As,min=kc·k·fct,eff·Act/σs (EC2 Eq.7.1)
b=0.250m, beff=0.862m, h=0.500m, d=0.465m, N=0.00kN, σc=(N/bh)=0.00N/mm^2, Φ=10mm
max(h,b1)=500mm, fctm=2.60N/mm^2, hc,eff=2.50x(h-d)=87mm, k=0.86, kc=0.40 (EC2 Eq.7.2)
Min. reinf. without control of crack width, As,min=0.40x0.86x2.60x250x87/500=39mm^2=0.39cm^2
Crack control for crack width wk=0.3mm, using steel diameter Ø=10mm
Øs=Ø*s(fctm/2.9)[kc·hcr/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 σs=360N/mm^2 (EC2 Table 7.2N)
Min. reinforcement for wk=0.3mm and Ø=10mm, As,min=0.40x0.86x2.60x250x87/360=54mm^2=0.54cm^2

4.8. 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=1.30, ρ=0.224%, L/d= 3600/ 465= 7.74<26.00

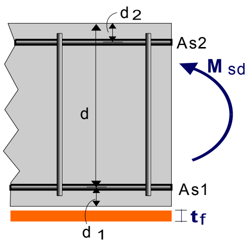
4.9. Reinforcing bar schedule

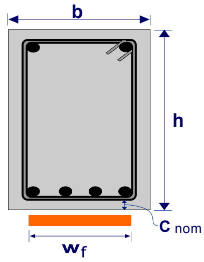
Num		type	reinforcing bar [mm]	items	Ø	g/m [kg/m]	length [m]	weight [kg]
34	(Span-1)	12	3890, 130	4	10	0.617	4.020	9.92
35	(Span-1)	8	3760	2	10	0.617	3.760	4.64
36	(Supp-A)	2	2450	4	10	0.617	2.450	6.05
37	(Supp-A)	2	2570	1	12	0.888	2.570	2.28
38	(Span-1)	9	80, 80, 200, 440, 200, 440	10	8	0.395	1.440	5.69
			Total weight [kg]					**28.58**



5. BEAM-002

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 (6.16cm^2), **As2=2Ø14** (3.08cm^2)
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)

5.1. Dimensions and loads

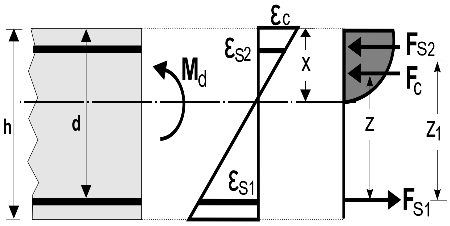
Beam cross section bw=0.250 m, h=0.500 m
Bottom reinforcement 4Ø14 (6.16cm^2)
Top reinforcement 2Ø14 (3.08cm^2)
Effective depth of cross section d1=Cnomc+Øs+0.5Ø=20+8+0.5x14=35mm, d=500-35=465mm

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 mm^2

5.2. Cross section moment capacity, without FRP strengthening

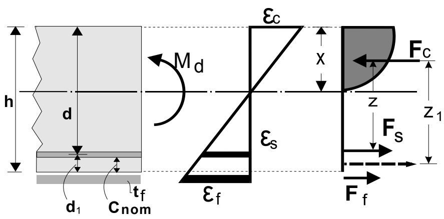
(EC2 EN1992-1-1:2004, §6.1)



(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

5.3. Cross section moment capacity, with FRP strengthening

(EC2 EN1992-1-1:2004, §6.1)



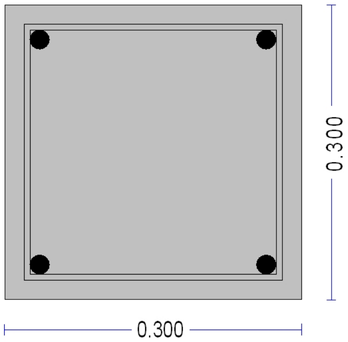
(iterations:5). From internal force equilibrium we have:
εc=3.50(o/oo), Fc=α·0.85fcd·b·x, α=0.810, x=132.0mm, x/d=0.28
Fc =-α·0.85fcd·b·x =0.001x0.810x0.85x16.67x 250x132.0= -378kN
εs1= 8.83(o/oo)>2.17=εy, Fs1= As1·fyd=0.001x 616x435.0= 268kN
εs2= 2.57(o/oo)>2.17=εy, Fs2=-As2·fyd=0.001x 308x435.0= -134kN
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

5.4. 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**

6. COLUMN-001



Strength of column (double eccentricity)

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

b =0.300 m, h =0.300 m
As=4Ø20 (12.56cm²)

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)

Dimensions and loads

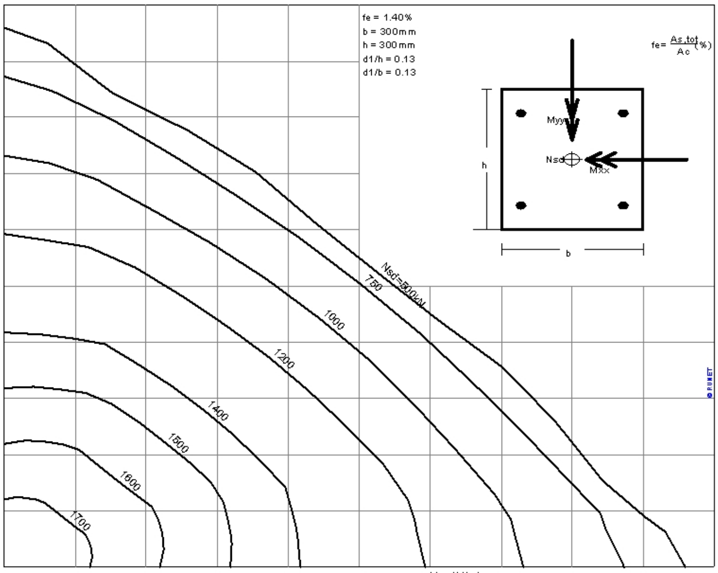
Column of rectangular cross section b=0.300 m, h=0.300 m
Reinforcement 4Ø20(12.56cm²) Astot/Ac=1.40%
Effective depth of cross section d=h-d1, d1=d2=Cnomc+Øs+Ø/2=20+8+20/2=38mm, dx=262mm, dy=262mm

6.1. Capacity of column cross-section (double eccentricity)

(EC2 EN1992-1-1:2004, §6.1)

Design chart for column capacity obtained from numerical integration using a grid of 10x10=100 cross-section subdivisions

b=0.30m, h=0.30m
d1/h=0.13, d1/b=0.13
Fe=4Ø20
Astot=(12.56cm²)
Astot/Ac=1.40%



```
Neutral axis slope θ=0.00°                          Neutral axis slope θ=7.50°
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.47)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.48)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.44)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.45)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.38)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.40)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.19)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.26)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-2.89)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.01)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-2.48)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-2.69)
 N= 1363 Mxx=  50 Myy=   0 (ec2/es1=-3.50/-0.44)   N= 1638 Mxx=  21 Myy=   3 (ec2/es1=-3.50/-1.07)
 N= 1285 Mxx=  58 Myy=   0 (ec2/es1=-3.50/-0.28)   N= 1602 Mxx=  25 Myy=   3 (ec2/es1=-3.50/-0.95)
 N= 1217 Mxx=  64 Myy=   0 (ec2/es1=-3.50/-0.10)   N= 1557 Mxx=  29 Myy=   4 (ec2/es1=-3.50/-0.81)
 N= 1134 Mxx=  71 Myy=   0 (ec2/es1=-3.50/ 0.10)   N= 1502 Mxx=  35 Myy=   5 (ec2/es1=-3.50/-0.65)
 N= 1061 Mxx=  76 Myy=   0 (ec2/es1=-3.50/ 0.32)   N= 1433 Mxx=  42 Myy=   5 (ec2/es1=-3.50/-0.47)
 N=  970 Mxx=  83 Myy=   0 (ec2/es1=-3.50/ 0.58)   N= 1346 Mxx=  51 Myy=   6 (ec2/es1=-3.50/-0.27)
 N=  890 Mxx=  88 Myy=   0 (ec2/es1=-3.50/ 0.87)   N= 1246 Mxx=  60 Myy=   7 (ec2/es1=-3.50/-0.04)
 N=  695 Mxx=  99 Myy=   0 (ec2/es1=-3.50/ 1.59)   N= 1029 Mxx=  77 Myy=   7 (ec2/es1=-3.50/ 0.54)
 N=  520 Mxx= 106 Myy=   0 (ec2/es1=-3.50/ 2.61)   N=  787 Mxx=  92 Myy=   8 (ec2/es1=-3.50/ 1.35)
 N=  417 Mxx= 102 Myy=   0 (ec2/es1=-3.50/ 4.14)   N=  508 Mxx= 103 Myy=   7 (ec2/es1=-3.50/ 2.56)
 N=  297 Mxx=  94 Myy=   0 (ec2/es1=-3.50/ 6.69)   N=  318 Mxx=  94 Myy=   9 (ec2/es1=-3.50/ 4.58)
 N=  187 Mxx=  84 Myy=   0 (ec2/es1=-3.50/ 8.73)   N=  212 Mxx=  85 Myy=  11 (ec2/es1=-3.50/ 6.20)
 N=   32 Mxx=  67 Myy=   0 (ec2/es1=-3.50/13.97)   N=    0 Mxx=  64 Myy=  14 (ec2/es1=-3.50/10.36)
 N= -546 Mxx=   0 Myy=   0 (ec2/es1=-3.50/57.63)   N= -538 Mxx=   1 Myy=   1 (ec2/es1=-3.50/45.00)
 N= -546 Mxx=   0 Myy=   0 (ec2/es1=-3.50/149.33)   N= -546 Mxx=   0 Myy=   0 (ec2/es1=-3.50/117.75)
 (Nsd [kN], Msd [kNm], ec2 es1 [o/oo])
```

```
Neutral axis slope θ=10.00°                         Neutral axis slope θ=15.00°
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.47)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.47)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.45)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.45)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.40)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.39)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.25)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.24)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.00)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-2.97)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-2.67)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-2.62)
 N= 1625 Mxx=  22 Myy=   4 (ec2/es1=-3.50/-1.00)   N= 1602 Mxx=  24 Myy=   6 (ec2/es1=-3.50/-0.85)
 N= 1587 Mxx=  26 Myy=   5 (ec2/es1=-3.50/-0.86)   N= 1559 Mxx=  29 Myy=   7 (ec2/es1=-3.50/-0.71)
 N= 1540 Mxx=  31 Myy=   5 (ec2/es1=-3.50/-0.72)   N= 1507 Mxx=  34 Myy=   9 (ec2/es1=-3.50/-0.56)
 N= 1482 Mxx=  37 Myy=   6 (ec2/es1=-3.50/-0.55)   N= 1442 Mxx=  41 Myy=  10 (ec2/es1=-3.50/-0.39)
 N= 1408 Mxx=  44 Myy=   7 (ec2/es1=-3.50/-0.37)   N= 1363 Mxx=  48 Myy=  12 (ec2/es1=-3.50/-0.19)
 N= 1319 Mxx=  53 Myy=   8 (ec2/es1=-3.50/-0.16)   N= 1270 Mxx=  57 Myy=  13 (ec2/es1=-3.50/ 0.03)
 N= 1217 Mxx=  62 Myy=   9 (ec2/es1=-3.50/ 0.08)   N= 1168 Mxx=  65 Myy=  13 (ec2/es1=-3.50/ 0.28)
 N=  999 Mxx=  79 Myy=   9 (ec2/es1=-3.50/ 0.67)   N=  937 Mxx=  80 Myy=  15 (ec2/es1=-3.50/ 0.91)
 N=  750 Mxx=  92 Myy=  10 (ec2/es1=-3.50/ 1.51)   N=  668 Mxx=  91 Myy=  18 (ec2/es1=-3.50/ 1.79)
 N=  470 Mxx= 100 Myy=  10 (ec2/es1=-3.50/ 2.76)   N=  397 Mxx=  94 Myy=  16 (ec2/es1=-3.50/ 3.12)
 N=  273 Mxx=  90 Myy=  12 (ec2/es1=-3.50/ 4.85)   N=  186 Mxx=  81 Myy=  19 (ec2/es1=-3.50/ 5.32)
 N=  161 Mxx=  80 Myy=  14 (ec2/es1=-3.50/ 6.52)   N=   61 Mxx=  69 Myy=  22 (ec2/es1=-3.50/ 7.09)
 N=  -62 Mxx=  56 Myy=  18 (ec2/es1=-3.50/10.81)   N= -178 Mxx=  43 Myy=  26 (ec2/es1=-3.50/11.63)
 N= -540 Mxx=   1 Myy=   1 (ec2/es1=-3.50/46.58)   N= -541 Mxx=   1 Myy=   1 (ec2/es1=-3.50/49.44)
 N= -546 Mxx=   0 Myy=   0 (ec2/es1=-3.50/121.69)   N= -546 Mxx=   0 Myy=   0 (ec2/es1=-3.50/128.86)
 (Nsd [kN], Msd [kNm], ec2 es1 [o/oo])
```

```
Neutral axis slope θ=22.50°                        Neutral axis slope θ=30.00°
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.47)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.47)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.44)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.44)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.39)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.38)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.22)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.20)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-2.94)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-2.91)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-2.56)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-2.52)
 N= 1573 Mxx=  27 Myy=  10 (ec2/es1=-3.50/-0.68)   N= 1551 Mxx=  28 Myy=  14 (ec2/es1=-3.50/-0.55)
 N= 1525 Mxx=  32 Myy=  12 (ec2/es1=-3.50/-0.53)   N= 1494 Mxx=  32 Myy=  17 (ec2/es1=-3.50/-0.39)
 N= 1466 Mxx=  37 Myy=  14 (ec2/es1=-3.50/-0.36)   N= 1426 Mxx=  38 Myy=  20 (ec2/es1=-3.50/-0.22)
 N= 1394 Mxx=  44 Myy=  16 (ec2/es1=-3.50/-0.18)   N= 1346 Mxx=  43 Myy=  23 (ec2/es1=-3.50/-0.03)
 N= 1306 Mxx=  51 Myy=  18 (ec2/es1=-3.50/ 0.03)   N= 1255 Mxx=  50 Myy=  26 (ec2/es1=-3.50/ 0.19)
 N= 1206 Mxx=  58 Myy=  20 (ec2/es1=-3.50/ 0.27)   N= 1152 Mxx=  56 Myy=  29 (ec2/es1=-3.50/ 0.44)
 N= 1094 Mxx=  65 Myy=  22 (ec2/es1=-3.50/ 0.53)   N= 1037 Mxx=  62 Myy=  31 (ec2/es1=-3.50/ 0.72)
 N=  846 Mxx=  77 Myy=  25 (ec2/es1=-3.50/ 1.21)   N=  776 Mxx=  72 Myy=  36 (ec2/es1=-3.50/ 1.42)
 N=  561 Mxx=  86 Myy=  29 (ec2/es1=-3.50/ 2.15)   N=  492 Mxx=  77 Myy=  39 (ec2/es1=-3.50/ 2.41)
 N=  301 Mxx=  83 Myy=  26 (ec2/es1=-3.50/ 3.56)   N=  225 Mxx=  72 Myy=  36 (ec2/es1=-3.50/ 3.88)
 N=   64 Mxx=  68 Myy=  29 (ec2/es1=-3.50/ 5.91)   N=  -29 Mxx=  57 Myy=  35 (ec2/es1=-3.50/ 6.34)
 N=  -67 Mxx=  54 Myy=  31 (ec2/es1=-3.50/ 7.80)   N= -158 Mxx=  44 Myy=  37 (ec2/es1=-3.50/ 8.31)
 N= -265 Mxx=  33 Myy=  30 (ec2/es1=-3.50/12.64)   N= -284 Mxx=  30 Myy=  29 (ec2/es1=-3.50/13.37)
 N= -543 Mxx=   0 Myy=   0 (ec2/es1=-3.50/52.98)   N= -545 Mxx=   0 Myy=   0 (ec2/es1=-3.50/55.55)
 N= -546 Mxx=   0 Myy=   0 (ec2/es1=-3.50/137.70)   N= -546 Mxx=   0 Myy=   0 (ec2/es1=-3.50/144.13)
 (Nsd [kN], Msd [kNm], ec2 es1 [o/oo])
```

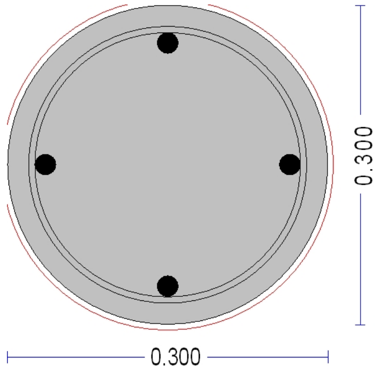
```
Neutral axis slope θ=37.50°                        Neutral axis slope θ=45.00°
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.47)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.47)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.44)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.44)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.38)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.38)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.20)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.19)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-2.89)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-2.89)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-2.49)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-2.48)
 N= 1529 Mxx=  26 Myy=  19 (ec2/es1=-3.50/-0.47)   N= 1522 Mxx=  23 Myy=  23 (ec2/es1=-3.50/-0.44)
 N= 1469 Mxx=  31 Myy=  23 (ec2/es1=-3.50/-0.31)   N= 1461 Mxx=  27 Myy=  27 (ec2/es1=-3.50/-0.28)
 N= 1399 Mxx=  35 Myy=  26 (ec2/es1=-3.50/-0.13)   N= 1389 Mxx=  31 Myy=  31 (ec2/es1=-3.50/-0.10)
 N= 1317 Mxx=  41 Myy=  30 (ec2/es1=-3.50/ 0.07)   N= 1307 Mxx=  36 Myy=  36 (ec2/es1=-3.50/ 0.10)
 N= 1224 Mxx=  46 Myy=  34 (ec2/es1=-3.50/ 0.29)   N= 1213 Mxx=  41 Myy=  41 (ec2/es1=-3.50/ 0.32)
 N= 1119 Mxx=  51 Myy=  37 (ec2/es1=-3.50/ 0.54)   N= 1107 Mxx=  45 Myy=  45 (ec2/es1=-3.50/ 0.58)
 N= 1002 Mxx=  56 Myy=  41 (ec2/es1=-3.50/ 0.83)   N=  989 Mxx=  49 Myy=  49 (ec2/es1=-3.50/ 0.87)
 N=  733 Mxx=  65 Myy=  46 (ec2/es1=-3.50/ 1.55)   N=  716 Mxx=  56 Myy=  56 (ec2/es1=-3.50/ 1.59)
 N=  445 Mxx=  68 Myy=  49 (ec2/es1=-3.50/ 2.56)   N=  425 Mxx=  58 Myy=  58 (ec2/es1=-3.50/ 2.61)
 N=  178 Mxx=  62 Myy=  45 (ec2/es1=-3.50/ 4.08)   N=  160 Mxx=  53 Myy=  53 (ec2/es1=-3.50/ 4.14)
 N=  -94 Mxx=  49 Myy=  41 (ec2/es1=-3.50/ 6.60)   N= -133 Mxx=  44 Myy=  44 (ec2/es1=-3.50/ 6.69)
 N= -190 Mxx=  40 Myy=  39 (ec2/es1=-3.50/ 8.62)   N= -199 Mxx=  39 Myy=  39 (ec2/es1=-3.50/ 8.73)
 N= -294 Mxx=  29 Myy=  28 (ec2/es1=-3.50/13.82)   N= -296 Mxx=  28 Myy=  28 (ec2/es1=-3.50/13.97)
 N= -546 Mxx=   0 Myy=   0 (ec2/es1=-3.50/57.11)   N= -546 Mxx=   0 Myy=   0 (ec2/es1=-3.50/57.63)
 N= -546 Mxx=   0 Myy=   0 (ec2/es1=-3.50/148.03)   N= -546 Mxx=   0 Myy=   0 (ec2/es1=-3.50/149.33)
 (Nsd [kN], Msd [kNm], ec2 es1 [o/oo])
```

```
Neutral axis slope θ=52.50°                             Neutral axis slope θ=60.00°
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.47)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.47)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.44)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.44)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.38)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.38)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.20)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.20)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-2.89)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-2.91)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-2.49)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-2.52)
 N= 1529 Mxx=  19 Myy=  26 (ec2/es1=-3.50/-0.47)   N= 1551 Mxx=  14 Myy=  28 (ec2/es1=-3.50/-0.55)
 N= 1469 Mxx=  23 Myy=  31 (ec2/es1=-3.50/-0.31)   N= 1494 Mxx=  17 Myy=  32 (ec2/es1=-3.50/-0.39)
 N= 1399 Mxx=  26 Myy=  35 (ec2/es1=-3.50/-0.13)   N= 1426 Mxx=  20 Myy=  38 (ec2/es1=-3.50/-0.22)
 N= 1317 Mxx=  30 Myy=  41 (ec2/es1=-3.50/ 0.07)   N= 1346 Mxx=  23 Myy=  43 (ec2/es1=-3.50/-0.03)
 N= 1224 Mxx=  34 Myy=  46 (ec2/es1=-3.50/ 0.29)   N= 1255 Mxx=  26 Myy=  50 (ec2/es1=-3.50/ 0.19)
 N= 1119 Mxx=  37 Myy=  51 (ec2/es1=-3.50/ 0.54)   N= 1152 Mxx=  29 Myy=  56 (ec2/es1=-3.50/ 0.44)
 N= 1002 Mxx=  41 Myy=  56 (ec2/es1=-3.50/ 0.83)   N= 1037 Mxx=  31 Myy=  62 (ec2/es1=-3.50/ 0.72)
 N=  733 Mxx=  46 Myy=  65 (ec2/es1=-3.50/ 1.55)   N=  776 Mxx=  36 Myy=  72 (ec2/es1=-3.50/ 1.42)
 N=  445 Mxx=  49 Myy=  68 (ec2/es1=-3.50/ 2.56)   N=  492 Mxx=  39 Myy=  77 (ec2/es1=-3.50/ 2.41)
 N=  178 Mxx=  45 Myy=  62 (ec2/es1=-3.50/ 4.08)   N=  225 Mxx=  36 Myy=  72 (ec2/es1=-3.50/ 3.88)
 N=  -94 Mxx=  41 Myy=  49 (ec2/es1=-3.50/ 6.60)   N=  -29 Mxx=  35 Myy=  57 (ec2/es1=-3.50/ 6.34)
 N= -190 Mxx=  39 Myy=  40 (ec2/es1=-3.50/ 8.62)   N= -158 Mxx=  37 Myy=  44 (ec2/es1=-3.50/ 8.31)
 N= -294 Mxx=  28 Myy=  29 (ec2/es1=-3.50/13.82)   N= -284 Mxx=  29 Myy=  30 (ec2/es1=-3.50/13.37)
 N= -546 Mxx=   0 Myy=   0 (ec2/es1=-3.50/57.11)   N= -545 Mxx=   0 Myy=   0 (ec2/es1=-3.50/55.55)
 N= -546 Mxx=   0 Myy=   0 (ec2/es1=-3.50/148.03)  N= -546 Mxx=   0 Myy=   0 (ec2/es1=-3.50/144.13)
 (Nsd [kN], Msd [kNm], ec2 es1 [o/oo])
```

```
Neutral axis slope θ=67.50°                             Neutral axis slope θ=75.00°
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.47)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.47)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.44)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.45)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.39)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.39)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.22)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.24)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-2.94)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-2.97)
 N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-2.56)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-2.62)
 N= 1573 Mxx=  10 Myy=  27 (ec2/es1=-3.50/-0.68)   N= 1602 Mxx=   6 Myy=  24 (ec2/es1=-3.50/-0.85)
 N= 1525 Mxx=  12 Myy=  32 (ec2/es1=-3.50/-0.53)   N= 1559 Mxx=   7 Myy=  29 (ec2/es1=-3.50/-0.71)
 N= 1466 Mxx=  14 Myy=  37 (ec2/es1=-3.50/-0.36)   N= 1507 Mxx=   9 Myy=  34 (ec2/es1=-3.50/-0.56)
 N= 1394 Mxx=  16 Myy=  44 (ec2/es1=-3.50/-0.18)   N= 1442 Mxx=  10 Myy=  41 (ec2/es1=-3.50/-0.39)
 N= 1306 Mxx=  18 Myy=  51 (ec2/es1=-3.50/ 0.03)   N= 1363 Mxx=  12 Myy=  48 (ec2/es1=-3.50/-0.19)
 N= 1206 Mxx=  20 Myy=  58 (ec2/es1=-3.50/ 0.27)   N= 1270 Mxx=  13 Myy=  57 (ec2/es1=-3.50/ 0.03)
 N= 1094 Mxx=  22 Myy=  65 (ec2/es1=-3.50/ 0.53)   N= 1168 Mxx=  13 Myy=  65 (ec2/es1=-3.50/ 0.28)
 N=  846 Mxx=  25 Myy=  77 (ec2/es1=-3.50/ 1.21)   N=  937 Mxx=  15 Myy=  80 (ec2/es1=-3.50/ 0.91)
 N=  561 Mxx=  29 Myy=  86 (ec2/es1=-3.50/ 2.15)   N=  668 Mxx=  18 Myy=  91 (ec2/es1=-3.50/ 1.79)
 N=  301 Mxx=  26 Myy=  83 (ec2/es1=-3.50/ 3.56)   N=  397 Mxx=  16 Myy=  94 (ec2/es1=-3.50/ 3.12)
 N=   64 Mxx=  29 Myy=  68 (ec2/es1=-3.50/ 5.91)   N=  186 Mxx=  19 Myy=  81 (ec2/es1=-3.50/ 5.32)
 N=  -67 Mxx=  31 Myy=  54 (ec2/es1=-3.50/ 7.80)   N=   61 Mxx=  22 Myy=  69 (ec2/es1=-3.50/ 7.09)
 N= -265 Mxx=  30 Myy=  33 (ec2/es1=-3.50/12.64)   N= -178 Mxx=  26 Myy=  43 (ec2/es1=-3.50/11.63)
 N= -543 Mxx=   0 Myy=   0 (ec2/es1=-3.50/52.98)   N= -541 Mxx=   1 Myy=   1 (ec2/es1=-3.50/49.44)
 N= -546 Mxx=   0 Myy=   0 (ec2/es1=-3.50/137.70)  N= -546 Mxx=   0 Myy=   0 (ec2/es1=-3.50/128.86)
 (Nsd [kN], Msd [kNm], ec2 es1 [o/oo])
```

```
Neutral axis slope θ=82.50°                          Neutral axis slope θ=90.00°
N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.48)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.47)
N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.45)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.44)
N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.40)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.38)
N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.26)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.19)
N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-3.01)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-2.89)
N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-2.69)   N= 1822 Mxx=   0 Myy=   0 (ec2/es1=-3.50/-2.48)
N= 1638 Mxx=   3 Myy=  21 (ec2/es1=-3.50/-1.07)   N= 1363 Mxx=   0 Myy=  50 (ec2/es1=-3.50/-0.44)
N= 1602 Mxx=   3 Myy=  25 (ec2/es1=-3.50/-0.95)   N= 1285 Mxx=   0 Myy=  58 (ec2/es1=-3.50/-0.28)
N= 1557 Mxx=   4 Myy=  29 (ec2/es1=-3.50/-0.81)   N= 1217 Mxx=   0 Myy=  64 (ec2/es1=-3.50/-0.10)
N= 1502 Mxx=   5 Myy=  35 (ec2/es1=-3.50/-0.65)   N= 1134 Mxx=   0 Myy=  71 (ec2/es1=-3.50/ 0.10)
N= 1433 Mxx=   5 Myy=  42 (ec2/es1=-3.50/-0.47)   N= 1061 Mxx=   0 Myy=  76 (ec2/es1=-3.50/ 0.32)
N= 1346 Mxx=   6 Myy=  51 (ec2/es1=-3.50/-0.27)   N=  970 Mxx=   0 Myy=  83 (ec2/es1=-3.50/ 0.58)
N= 1246 Mxx=   7 Myy=  60 (ec2/es1=-3.50/-0.04)   N=  890 Mxx=   0 Myy=  88 (ec2/es1=-3.50/ 0.87)
N= 1029 Mxx=   7 Myy=  77 (ec2/es1=-3.50/ 0.54)   N=  695 Mxx=   0 Myy=  99 (ec2/es1=-3.50/ 1.59)
N=  787 Mxx=   8 Myy=  92 (ec2/es1=-3.50/ 1.35)   N=  520 Mxx=   0 Myy= 106 (ec2/es1=-3.50/ 2.61)
N=  508 Mxx=   7 Myy= 103 (ec2/es1=-3.50/ 2.56)   N=  417 Mxx=   0 Myy= 102 (ec2/es1=-3.50/ 4.14)
N=  318 Mxx=   9 Myy=  94 (ec2/es1=-3.50/ 4.58)   N=  297 Mxx=   0 Myy=  94 (ec2/es1=-3.50/ 6.69)
N=  212 Mxx=  11 Myy=  85 (ec2/es1=-3.50/ 6.20)   N=  187 Mxx=   0 Myy=  84 (ec2/es1=-3.50/ 8.73)
N=    0 Mxx=  14 Myy=  64 (ec2/es1=-3.50/10.36)   N=   32 Mxx=   0 Myy=  67 (ec2/es1=-3.50/13.97)
N= -538 Mxx=   1 Myy=   1 (ec2/es1=-3.50/45.00)   N= -546 Mxx=   0 Myy=   0 (ec2/es1=-3.50/57.63)
N= -546 Mxx=   0 Myy=   0 (ec2/es1=-3.50/117.75)   N= -546 Mxx=   0 Myy=   0 (ec2/es1=-3.50/149.33)
 (Nsd [kN], Msd [kNm], ec2 es1 [o/oo])
```

7. COLUMN-002



Strength of Column with FRP jacket (double eccentricity)
(EC2 EN1992-1-1:2004, EC0 EN1990-1-1:2002)

D =0.300 m
As=4Ø20 (12.56cm²)
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)

Dimensions and loads
Circular column with diameter D=0.300 m
Reinforcement 4Ø20(12.56cm²) Astot/Ac=1.78%
Effective depth of cross section d=h-d1, d1=d2=Cnomc+Øs+Ø/2=20+8+20/2=38mm, d=262mm

Fibre Reinforced Polymer material (FRP)
Characteristic name : FRP+epoxy
Total thickness : 1.00 mm
Modulus of elasticity : 100 GPa
Tensile strength : 1000 MPa

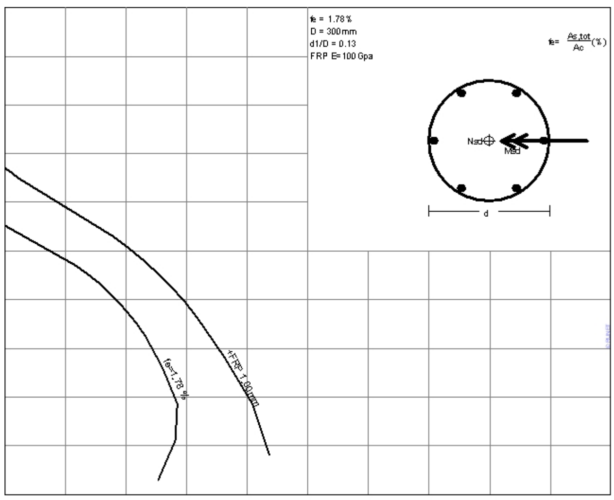
7.1. Increase of column shear strength
Vsf=a.ef.Ef.tf.b=2.24x0.002x100.0x1.000x300=134kN
(assumed effective design strain εf=0.002, shape coefficient a=2.24)

7.2. Capacity of column cross-section strengthened with FRP jacket (double eccentricity)
(EC2 EN1992-1-1:2004, §6.1)

Design chart for column capacity obtained from numerical integration of cross-section stresses
D=0.300m, d1/D=0.13, 4Ø20 Astot=(12.56cm²), Astot/Ac=1.78%
FRP:FRP+epoxy, t=1.00 mm, Ef=100 GPa

7.3. Maximum axial load Nsd, and maximum bending moment Msd

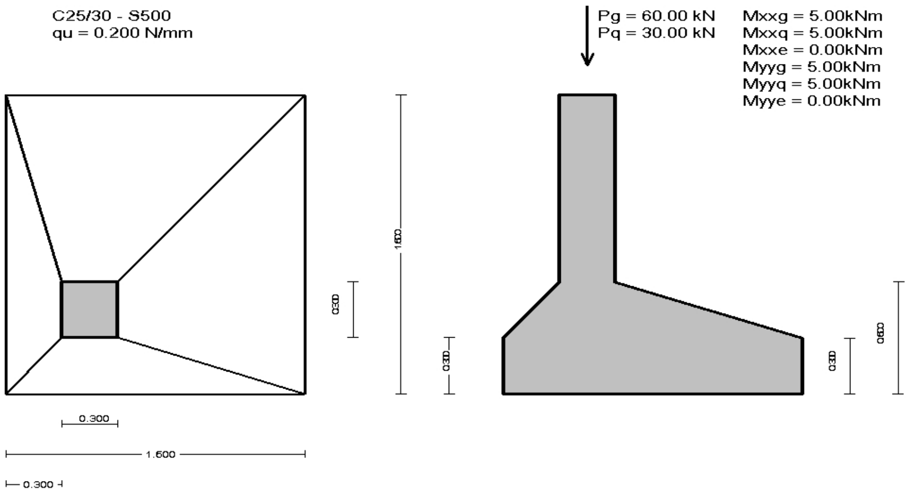
```
N= 1876kN, M=   0kNm, (ec2/es1=-3.50/-3.47)
N= 1875kN, M=   0kNm, (ec2/es1=-3.50/-3.45)
N= 1872kN, M=   0kNm, (ec2/es1=-3.50/-3.40)
N= 1862kN, M=   1kNm, (ec2/es1=-3.50/-3.24)
N= 1845kN, M=   2kNm, (ec2/es1=-3.50/-2.98)
N= 1823kN, M=   4kNm, (ec2/es1=-3.50/-2.63)
N= 1481kN, M=  36kNm, (ec2/es1=-3.50/-0.89)
N= 1421kN, M=  41kNm, (ec2/es1=-3.50/-0.75)
N= 1351kN, M=  45kNm, (ec2/es1=-3.50/-0.60)
N= 1275kN, M=  50kNm, (ec2/es1=-3.50/-0.43)
N= 1192kN, M=  55kNm, (ec2/es1=-3.50/-0.23)
N= 1102kN, M=  60kNm, (ec2/es1=-3.50/-0.02)
N= 1006kN, M=  64kNm, (ec2/es1=-3.50/ 0.23)
N=  787kN, M=  73kNm, (ec2/es1=-3.50/ 0.86)
N=  521kN, M=  82kNm, (ec2/es1=-3.50/ 1.73)
N=  229kN, M=  87kNm, (ec2/es1=-3.50/ 3.03)
```



8. FOOTING-001

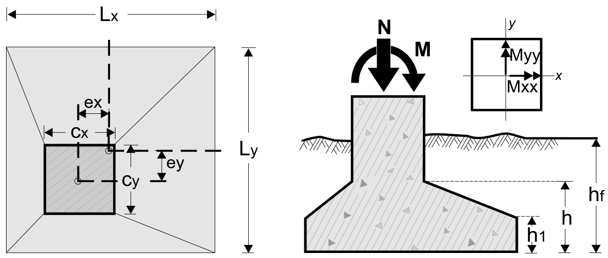
Asymmetric footing with eccentric load

(EC2 EN1992-1-1:2004, EC0 EN1990-1-1:2002, EC7 EN1997-1-1:2004)



Concrete-Steel class: C25/30-S500 (EC2 §3)
Concrete cover : Cnom=75 mm (EC2 §4.4.1)
Concrete weight : 25.0 kN/m³
γc=1.50, γs=1.15 (EC2 Table 2.1N)

8.1. Dimensions, materials, loads



Dimensions

Footing	Lx= 1.600 m	Ly= 1.600 m
Column	cx= 0.300 m	cy= 0.300 m
Eccentr.	ex=-0.350 m	ey=-0.350 m
Heights	h= 0.600 m	h1= 0.300 m
Depth of footing		hf= 1.200 m
Base area of footing		A= 2.56 m²
Volume of footing		V= 1.08 m³

Materials of footing

Concrete-Steel class: C25/30-S500 (EN1992-1-1, §3)
Concrete cover: Cnom=75 mm (EC2 §4.4.1)
Effective depth of cross section d=h-d1, d1=Cnomc+(3/2)Ø=75+3x14/2=96mm, d=600-96=504mm

Soil

Soil bearing pressure qu= 0.200 N/mm² (MPa)
Unit weight of soil γ=17.000 kN/m³

Loads

			permanent	variable	Seism-X	Seism-Y
Self weight	kN	[1.08x25.00]	27.00			
Soil weight	kN	[(2.56x 1.20- 1.08)x17.00]	33.86			
Vertical load	kN		60.00	30.00	10.00	10.00
Moment Mxx	kNm		5.00	5.00	0.00	0.00
Moment Myy	kNm		5.00	5.00	0.00	0.00

8.2. Eurocode parameters

Check of soil bearing capacity (EC7 EN1997-1-1:2004, §6)
Partial factors for actions and soil properties (EC7 Tables A.1-A.4, EC8-5 §3.1)
Equilibrium limit state (EQU), Structural limit state (STR), Geotechnical limit state (GEO)

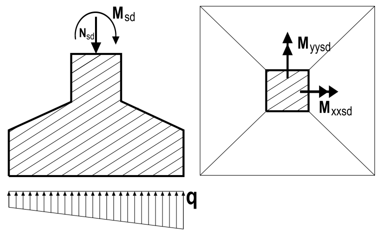
			(EQU)	(STR)	(GEO)	(SEISMIC)
Actions	Permanent Unfavourable	γ_{Gdst}:	1.10	1.35	1.00	1.00
	Permanent Favourable	γ_{Gstb}:	0.90	1.00	1.00	1.00
	Variable Unfavourable	γ_{Qdst}:	1.50	1.50	1.30	1.00
	Variable Favourable	γ_{Qstb}:	0.00	0.00	0.00	0.00
Soil	Angle of shearing resistance	γ_{φ}:	1.25	1.00	1.25	1.25
parameters	Effective cohesion	γ_{c}:	1.25	1.00	1.25	1.25
	Undrained shear strength	γ_{cu}:	1.40	1.00	1.40	1.40
	Unconfined strength	γ_{qu}:	1.40	1.00	1.40	1.40
	Weight density	γ_{w}:	1.00	1.00	1.00	1.00

Partial safety factors for actions : γ_G=1.35, γ_Q=1.50 (EC0 Annex A1)
Combination of accidental actions :(EC7) ψ_2 =0.30
Combination of accidental actions :(EC2) ψ_2 =0.30

Design of reinforced concrete (EC2 EN1992-1-1:2004)

Design for seismic loading (EC8 EN1998-5:2004)
Coefficients for seismic analysis (EC8-5 §3)
Soil parameters: γ_{φ}=1.25, γ_c=1.25, γ_{cu}=1.40, γ_{qu}=1.40, γ_w=1.00
(Effective footing area)/(footing area) with seismic load =0.50 (EC7 EN1997-1-1:2004, §6.5.4)

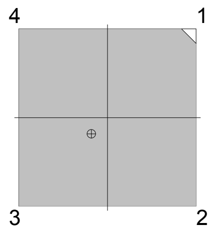
8.3. Check of soil bearing capacity (EC7 EN1997-1-1:2004, §6)

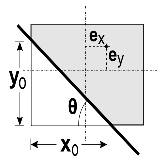


8.3.1. (EQU), 1.10xPermanent + 1.50xVariable (EC7 §2.4.7.2)

Design Loads
Nsd = 1.10x 120.86+1.50x 30.00 = 177.95 kN
Mxxsd= 1.10x 5.00+1.50x 5.00+(-0.35)x 111.00= -25.85 kNm
Myysd= 1.10x 5.00+1.50x 5.00+(-0.35)x 111.00= -25.85 kNm
(1.10x60.00+1.50x30.00=111.00)

Eccentricities, soil pressures, footing area
relative eccentricity ex/Lx=Myy/(N·Lx)=-0.091
relative eccentricity ey/Ly=Mxx/(N·Ly)=-0.091
soil pressure q1= 0.000 N/mm²
soil pressure q2= 0.070 N/mm²
soil pressure q3= 0.145 N/mm²
soil pressure q4= 0.070 N/mm²
zero pressure line xo=3.07m, yo=3.07m, θ=45°
effective footing area 99.64%



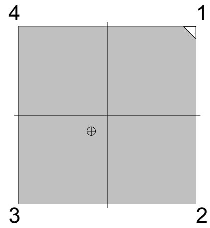


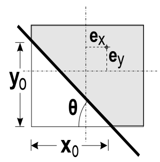
Check bearing resistance failure Vd<=Rd (EC7 EN1997-1-1:2004, §6.5.2)
relative load eccentricities ex/Lx=Myy/(N·Lx)=0.091, ey/Ly=Mxx/(N·Ly)=0.091
relative load eccentricities <=0.333 (EC7 §6.5.4)
effective design length of footing L'=1.600x(1-2x0.091)=1.309 m (EC7 Annex D)
effective design width of footing B'=1.600x(1-2x0.091)=1.309 m
effective design area of footing L'B'=1.309x1.309=1.71 m²
Design bearing resistance of footing Rd=1000x1.71x0.200/1.40=244.29 kN > Vd=177.95 kN
Effective footing area 99.64%>50.00% (EC7 §6.5.4)

8.3.2. (STR), 1.35xPermanent + 1.50xVariable (EC7 §2.4.7.3)

Design Loads
Nsd = 1.35x 120.86+1.50x 30.00 = 208.16 kN
Mxxsd= 1.35x 5.00+1.50x 5.00+(-0.35)x 126.00= -29.85 kNm
Myysd= 1.35x 5.00+1.50x 5.00+(-0.35)x 126.00= -29.85 kNm
(1.35x60.00+1.50x30.00=126.00)

Eccentricities, soil pressures, footing area
relative eccentricity ex/Lx=Myy/(N·Lx)=-0.090
relative eccentricity ey/Ly=Mxx/(N·Ly)=-0.090
soil pressure q1= 0.000 N/mm²
soil pressure q2= 0.081 N/mm²
soil pressure q3= 0.169 N/mm²
soil pressure q4= 0.081 N/mm²
zero pressure line xo=3.09m, yo=3.09m, θ=45°
effective footing area 99.72%



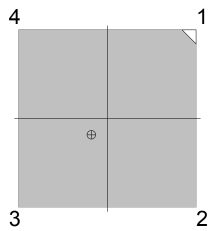


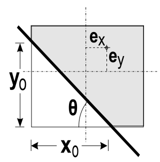
Check bearing resistance failure Vd<=Rd (EC7 EN1997-1-1:2004, §6.5.2)
relative load eccentricities ex/Lx=Myy/(N·Lx)=0.090, ey/Ly=Mxx/(N·Ly)=0.090
relative load eccentricities <=0.333 (EC7 §6.5.4)
effective design length of footing L'=1.600x(1-2x0.090)=1.312 m (EC7 Annex D)
effective design width of footing B'=1.600x(1-2x0.090)=1.312 m
effective design area of footing L'B'=1.312x1.312=1.72 m²
Design bearing resistance of footing Rd=1000x1.72x0.200/1.00=344.00 kN > Vd=208.16 kN
Effective footing area 99.72%>50.00% (EC7 §6.5.4)

8.3.3. (GEO), 1.00xPermanent + 1.30xVariable (EC7 §2.4.7.3)

Design Loads
Nsd = 1.00x 120.86+1.30x 30.00 = 159.86 kN
Mxxsd= 1.00x 5.00+1.30x 5.00+(-0.35)x 99.00= -23.15 kNm
Myysd= 1.00x 5.00+1.30x 5.00+(-0.35)x 99.00= -23.15 kNm
(1.00x60.00+1.30x30.00=99.00)

Eccentricities, soil pressures, footing area
relative eccentricity ex/Lx=Myy/(N·Lx)=-0.091
relative eccentricity ey/Ly=Mxx/(N·Ly)=-0.091
soil pressure q1= 0.000 N/mm²
soil pressure q2= 0.062 N/mm²
soil pressure q3= 0.130 N/mm²
soil pressure q4= 0.062 N/mm²
zero pressure line xo=3.07m, yo=3.07m, θ=45°
effective footing area 99.72%



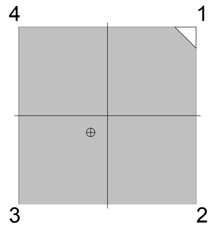


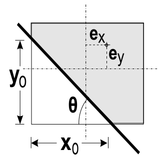
Check bearing resistance failure Vd<=Rd (EC7 EN1997-1-1:2004, §6.5.2)
relative load eccentricities ex/Lx=Myy/(N·Lx)=0.091, ey/Ly=Mxx/(N·Ly)=0.091
relative load eccentricities <=0.333 (EC7 §6.5.4)
effective design length of footing L'=1.600x(1-2x0.091)=1.309 m (EC7 Annex D)
effective design width of footing B'=1.600x(1-2x0.091)=1.309 m
effective design area of footing L'B'=1.309x1.309=1.71 m²
Design bearing resistance of footing Rd=1000x1.71x0.200/1.40=244.29 kN > Vd=159.86 kN
Effective footing area 99.72%>50.00% (EC7 §6.5.4)

8.3.4. Seismic load (x-x +) Permanent + 0.30xVariable + Seismic xx

Design Loads
Nsd = 1.00x 120.86+0.30x 30.00+1.00x(10.00) = 139.86 kN
Mxxsd= 1.00x 5.00+0.30x 5.00+1.00x(0.00)+(-0.35)x 79.00= -21.15 kNm
Myysd= 1.00x 5.00+0.30x 5.00+1.00x(0.00)+(-0.35)x 79.00= -21.15 kNm
(1.00x60.00+0.30x30.00+1.00x10.00=79.00)

Eccentricities, soil pressures, footing area
relative eccentricity ex/Lx=Myy/(N·Lx)=-0.095
relative eccentricity ey/Ly=Mxx/(N·Ly)=-0.095
soil pressure q1= 0.000 N/mm^2
soil pressure q2= 0.055 N/mm^2
soil pressure q3= 0.117 N/mm^2
soil pressure q4= 0.055 N/mm^2
zero pressure line xo=3.01m, yo=3.01m, θ=45°
effective footing area 99.22%



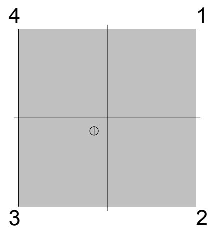


Check bearing resistance failure Vd<=Rd (EC7 EN1997-1-1:2004, §6.5.2)
relative load eccentricities ex/Lx=Myy/(N·Lx)=0.095, ey/Ly=Mxx/(N·Ly)=0.095
relative load eccentricities <=0.333 (EC7 §6.5.4)
effective design length of footing L'=1.600x(1-2x0.095)=1.296 m (EC7 Annex D)
effective design width of footing B'=1.600x(1-2x0.095)=1.296 m
effective design area of footing L'B'=1.296x1.296=1.68 m^2
Design bearing resistance of footing Rd=1000x1.68x0.200/1.40=240.00 kN > Vd=139.86 kN
Effective footing area 99.22%>50.00% (EC7 §6.5.4)

8.3.5. Seismic load (x-x -) Permanent + 0.30xVariable - Seismic xx

Design Loads
Nsd = 1.00x 120.86+0.30x 30.00+1.00x(-10.00) = 119.86 kN
Mxxsd= 1.00x 5.00+0.30x 5.00+1.00x(0.00)+(-0.35)x 59.00= -14.15 kNm
Myysd= 1.00x 5.00+0.30x 5.00+1.00x(0.00)+(-0.35)x 59.00= -14.15 kNm
(1.00x60.00+0.30x30.00-1.00x10.00=59.00)

Eccentricities, soil pressures, footing area
relative eccentricity ex/Lx=Myy/(N·Lx)=-0.074
relative eccentricity ey/Ly=Mxx/(N·Ly)=-0.074
soil pressure q1= 0.005 N/mm^2
soil pressure q2= 0.047 N/mm^2
soil pressure q3= 0.088 N/mm^2
soil pressure q4= 0.047 N/mm^2
effective footing area 100.00%

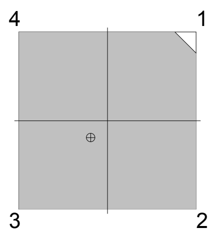


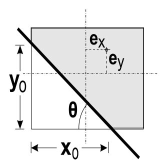
Check bearing resistance failure Vd<=Rd (EC7 EN1997-1-1:2004, §6.5.2)
relative load eccentricities ex/Lx=Myy/(N·Lx)=0.074, ey/Ly=Mxx/(N·Ly)=0.074
relative load eccentricities <=0.333 (EC7 §6.5.4)
effective design length of footing L'=1.600x(1-2x0.074)=1.363 m (EC7 Annex D)
effective design width of footing B'=1.600x(1-2x0.074)=1.363 m
effective design area of footing L'B'=1.363x1.363=1.86 m^2
Design bearing resistance of footing Rd=1000x1.86x0.200/1.40=265.71 kN > Vd=119.86 kN
Effective footing area 100.00%>50.00% (EC7 §6.5.4)

8.3.6. Seismic load (y-y +) Permanent + 0.30xVariable + Seismic yy

Design Loads
Nsd = 1.00x 120.86+0.30x 30.00+1.00x(10.00) = 139.86 kN
Mxxsd= 1.00x 5.00+0.30x 5.00+1.00x(0.00)+(-0.35)x 79.00= -21.15 kNm
Myysd= 1.00x 5.00+0.30x 5.00+1.00x(0.00)+(-0.35)x 79.00= -21.15 kNm
(1.00x60.00+0.30x30.00+1.00x10.00=79.00)

Eccentricities, soil pressures, footing area
relative eccentricity ex/Lx=Myy/(N·Lx)=-0.095
relative eccentricity ey/Ly=Mxx/(N·Ly)=-0.095
soil pressure q1= 0.000 N/mm^2
soil pressure q2= 0.055 N/mm^2
soil pressure q3= 0.117 N/mm^2
soil pressure q4= 0.055 N/mm^2
zero pressure line xo=3.01m, yo=3.01m, θ=45°
effective footing area 99.22%





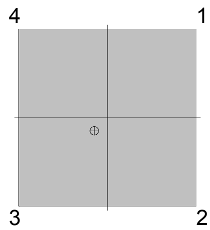
Check bearing resistance failure Vd<=Rd (EC7 EN1997-1-1:2004, §6.5.2)
relative load eccentricities ex/Lx=Myy/(N·Lx)=0.095, ey/Ly=Mxx/(N·Ly)=0.095
relative load eccentricities <=0.333 (EC7 §6.5.4)
effective design length of footing L'=1.600x(1-2x0.095)=1.296 m (EC7 Annex D)
effective design width of footing B'=1.600x(1-2x0.095)=1.296 m
effective design area of footing L'B'=1.296x1.296=1.68 m^2
Design bearing resistance of footing Rd=1000x1.68x0.200/1.40=240.00 kN > Vd=139.86 kN
Effective footing area 99.22%>50.00% (EC7 §6.5.4)

8.3.7. Seismic load (y-y -) Permanent + 0.30xVariable - Seismic yy

Design Loads

```
Nsd  = 1.00x  120.86+0.30x   30.00+1.00x(  -10.00)                =  119.86 kN
Mxxsd= 1.00x    5.00+0.30x    5.00+1.00x(    0.00)+(-0.35)x   59.00=  -14.15 kNm
Myysd= 1.00x    5.00+0.30x    5.00+1.00x(    0.00)+(-0.35)x   59.00=  -14.15 kNm
(1.00x60.00+0.30x30.00-1.00x10.00=59.00)
```

Eccentricities, soil pressures, footing area
relative eccentricity ex/Lx=Myy/(N·Lx)=-0.074
relative eccentricity ey/Ly=Mxx/(N·Ly)=-0.074
soil pressure q1= 0.005 N/mm^2
soil pressure q2= 0.047 N/mm^2
soil pressure q3= 0.088 N/mm^2
soil pressure q4= 0.047 N/mm^2
effective footing area 100.00%



Check bearing resistance failure Vd<=Rd (EC7 EN1997-1-1:2004, §6.5.2)
relative load eccentricities ex/Lx=Myy/(N·Lx)=0.074, ey/Ly=Mxx/(N·Ly)=0.074
relative load eccentricities <=0.333 (EC7 §6.5.4)
effective design length of footing L'=1.600x(1-2x0.074)=1.363 m (EC7 Annex D)
effective design width of footing B'=1.600x(1-2x0.074)=1.363 m
effective design area of footing L'B'=1.363x1.363=1.86 m^2
Design bearing resistance of footing Rd=1000x1.86x0.200/1.40=265.71 kN > Vd=119.86 kN
Effective footing area 100.00%>50.00% (EC7 §6.5.4)

8.4. Internal actions for reinforced concrete design

Moments M and shearing forces V, are computed at column faces.
Shearing forces V* are computed at distance d=0.504m from the column face.
They are computed, by numerical integration of the soil pressure under the footing.

8.4.1. Loading 1.35xPermanent + 1.50xVariable

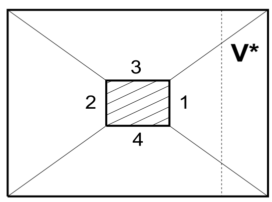
Design Loads

```
Nsd  = 1.35x  120.86+1.50x   30.00                 =  208.16 kN
Mxxsd= 1.35x    5.00+1.50x    5.00+(-0.35)x  126.00=  -29.85 kNm
Myysd= 1.35x    5.00+1.50x    5.00+(-0.35)x  126.00=  -29.85 kNm
(1.35x60.00+1.50x30.00=126.00)
```

Eccentricities, soil pressures, footing area
relative load eccentricities ex/Lx=Myy/(N·Lx)=-0.090, ey/Ly=Mxx/(N·Ly)=-0.090
soil pressures q1=0.000, q2=0.081, q3=0.169, q4=0.081 N/mm^2
zero pressure line xo=3.09m, yo=3.09m, θ=45°
pressure due to self weight+soil weight qg=0.001x1.35x(27.00+33.86)/2.56=0.032 N/mm^2
Shear at critical section + (self weight+soil weight) q·Acont+qg·A=172.81 kN

Internal actions (bending moments, shearing forces)

```
Myy(1)=   21.53 kNm, V(1)=   56.39 kN, V*(1)=   18.15 kN
Myy(2)=    6.30 kNm, V(2)=   41.17 kN, V*(2)=    0.00 kN
Mxx(3)=   21.53 kNm, V(3)=   56.39 kN, V*(3)=   18.15 kN
Mxx(4)=    6.30 kNm, V(4)=   41.17 kN, V*(4)=    0.00 kN
```



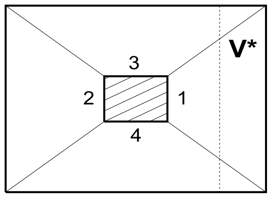
8.4.2. Seismic load (x-x +) Permanent + 0.30xVariable + Seismic xx

```
Design Loads
Nsd  = 1.00x  120.86+0.30x   30.00+1.00x(   10.00)                  =  139.86 kN
Mxxsd= 1.00x    5.00+0.30x    5.00+1.00x(    0.00)+(-0.35)x   79.00=  -21.15 kNm
Myysd= 1.00x    5.00+0.30x    5.00+1.00x(    0.00)+(-0.35)x   79.00=  -21.15 kNm
(1.00x60.00+0.30x30.00+1.00x10.00=79.00)

Eccentricities, soil pressures, footing area
relative load eccentricities ex/Lx=Myy/(N·Lx)=-0.095, ey/Ly=Mxx/(N·Ly)=-0.095
soil pressures q1=0.000, q2=0.055, q3=0.117, q4=0.055 N/mm²
zero pressure line xo=3.01m, yo=3.01m, θ=45°
pressure due to self weight+soil weight qg=0.001x1.35x(27.00+33.86)/2.56=0.032 N/mm²
Shear at critical section + (self weight+soil weight) q·Acont+qg·A=132.14 kN

Internal actions (bending moments, shearing forces)

Myy(1)=    8.64 kNm, V(1)=   24.65 kN, V*(1)=    6.80 kN
Myy(2)=    3.58 kNm, V(2)=   23.18 kN, V*(2)=    0.00 kN
Mxx(3)=    8.64 kNm, V(3)=   24.65 kN, V*(3)=    6.80 kN
Mxx(4)=    3.58 kNm, V(4)=   23.18 kN, V*(4)=    0.00 kN
```



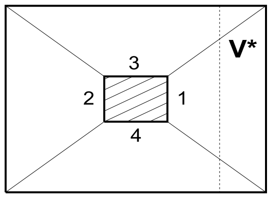
8.4.3. Seismic load (x-x -) Permanent + 0.30xVariable - Seismic xx

```
Design Loads
Nsd  = 1.00x  120.86+0.30x   30.00+1.00x(  -10.00)                  =  119.86 kN
Mxxsd= 1.00x    5.00+0.30x    5.00+1.00x(    0.00)+(-0.35)x   59.00=  -14.15 kNm
Myysd= 1.00x    5.00+0.30x    5.00+1.00x(    0.00)+(-0.35)x   59.00=  -14.15 kNm
(1.00x60.00+0.30x30.00-1.00x10.00=59.00)

Eccentricities, soil pressures, footing area
relative load eccentricities ex/Lx=Myy/(N·Lx)=-0.074, ey/Ly=Mxx/(N·Ly)=-0.074
soil pressures q1=0.005, q2=0.047, q3=0.088, q4=0.047 N/mm²
pressure due to self weight+soil weight qg=0.001x1.35x(27.00+33.86)/2.56=0.032 N/mm²
Shear at critical section + (self weight+soil weight) q·Acont+qg·A=115.06 kN

Internal actions (bending moments, shearing forces)

Myy(1)=    5.72 kNm, V(1)=   16.31 kN, V*(1)=    4.50 kN
Myy(2)=    2.36 kNm, V(2)=   15.33 kN, V*(2)=    0.00 kN
Mxx(3)=    5.72 kNm, V(3)=   16.31 kN, V*(3)=    4.50 kN
Mxx(4)=    2.36 kNm, V(4)=   15.33 kN, V*(4)=    0.00 kN
```

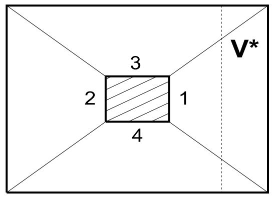


8.4.4. Seismic load (y-y +) Permanent + 0.30xVariable + Seismic yy

```
Design Loads
Nsd  = 1.00x  120.86+0.30x   30.00+1.00x(   10.00)                  =  139.86 kN
Mxxsd= 1.00x    5.00+0.30x    5.00+1.00x(    0.00)+(-0.35)x   79.00=  -21.15 kNm
Myysd= 1.00x    5.00+0.30x    5.00+1.00x(    0.00)+(-0.35)x   79.00=  -21.15 kNm
(1.00x60.00+0.30x30.00+1.00x10.00=79.00)

Eccentricities, soil pressures, footing area
relative load eccentricities ex/Lx=Myy/(N·Lx)=-0.095, ey/Ly=Mxx/(N·Ly)=-0.095
soil pressures q1=0.000, q2=0.055, q3=0.117, q4=0.055 N/mm²
zero pressure line xo=3.01m, yo=3.01m, θ=45°
pressure due to self weight+soil weight qg=0.001x1.35x(27.00+33.86)/2.56=0.032 N/mm²
Shear at critical section + (self weight+soil weight) q·Acont+qg·A=132.14 kN
```

Internal actions (bending moments, shearing forces)



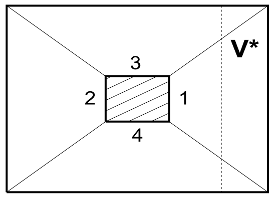
Myy(1)= 8.64 kNm, V(1)= 24.65 kN, V*(1)= 6.80 kN
Myy(2)= 3.58 kNm, V(2)= 23.18 kN, V*(2)= 0.00 kN
Mxx(3)= 8.64 kNm, V(3)= 24.65 kN, V*(3)= 6.80 kN
Mxx(4)= 3.58 kNm, V(4)= 23.18 kN, V*(4)= 0.00 kN

8.4.5. Seismic load (y-y -) Permanent + 0.30xVariable - Seismic yy

Design Loads
Nsd = 1.00x 120.86+0.30x 30.00+1.00x(-10.00) = 119.86 kN
Mxxsd= 1.00x 5.00+0.30x 5.00+1.00x(0.00)+(-0.35)x 59.00= -14.15 kNm
Myysd= 1.00x 5.00+0.30x 5.00+1.00x(0.00)+(-0.35)x 59.00= -14.15 kNm
(1.00x60.00+0.30x30.00-1.00x10.00=59.00)

Eccentricities, soil pressures, footing area
relative load eccentricities ex/Lx=Myy/(N·Lx)=-0.074, ey/Ly=Mxx/(N·Ly)=-0.074
soil pressures q1=0.005, q2=0.047, q3=0.088, q4=0.047 N/mm^2
pressure due to self weight+soil weight qg=0.001x1.35x(27.00+33.86)/2.56=0.032 N/mm^2
Shear at critical section + (self weight+soil weight) q·Acont+qg·A=115.06 kN

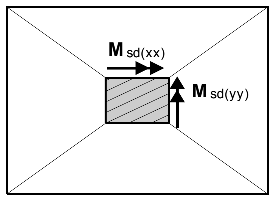
Internal actions (bending moments, shearing forces)



Myy(1)= 5.72 kNm, V(1)= 16.31 kN, V*(1)= 4.50 kN
Myy(2)= 2.36 kNm, V(2)= 15.33 kN, V*(2)= 0.00 kN
Mxx(3)= 5.72 kNm, V(3)= 16.31 kN, V*(3)= 4.50 kN
Mxx(4)= 2.36 kNm, V(4)= 15.33 kN, V*(4)= 0.00 kN

8.5. Design for bending (EC2 EN1992-1-1:2004, §6.1)

Maximum design moments
Msd(yy)= 21.53 kNm, b= 300 mm, d= 504 mm
Msd(xx)= 21.53 kNm, b= 300 mm, d= 504 mm



Msd=21.53kNm, b=300mm, d=504mm, Kd=5.95, x/d=0.05
ec/es=1.0/20.0, Ks=2.34, **As= 1.00cm^2**
Minimum reinforcement As>=0.26bd·Fctm/fyk (As= 4.09cm^2/m) (EC2 §9.3.1)
Minimum reinforcement Ø14/37.5(4.11cm^2/m)

Msd=21.53kNm, b=300mm, d=504mm, Kd=5.95, x/d=0.05
ec/es=1.0/20.0, Ks=2.34, **As= 1.00cm^2**
Minimum reinforcement As>=0.26bd·Fctm/fyk (As= 4.09cm^2/m) (EC2 §9.3.1)
Minimum reinforcement Ø14/37.5(4.11cm^2/m)

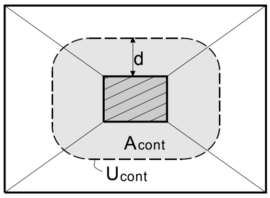
Reinforcement of footing
Steel reinforcement in x-x direction: Ø14/37.5(4.11cm^2/m), **5Ø14**(7.70cm^2)
Steel reinforcement in y-y direction: Ø14/37.5(4.11cm^2/m), **5Ø14**(7.70cm^2)

8.6. Design for shear (EC2 EN1992-1-1:2004, §6.2)

The design for shear is covered by the design in punching shear
because the critical rupture surface is considered at angle 45°

8.7. Design for punching shear (EC2 EN1992-1-1:2004, §6.4)

Footing cantilevers in x-x, L1=1.000>d=0.504m, L2=0.300<d=0.504m
Footing cantilevers in y-y, L1=1.000>d=0.504m, L2=0.300<d=0.504m

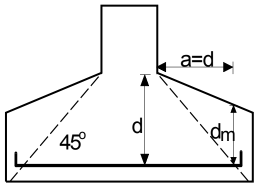


Control perimeter, at 1.0d=0.504m<2.0d (EC2 §6.4.2.2)
we consider rupture surface at angle 45°
Ucont=(0.300+0.000+0.000+0.300)+3.14x(0.252+0.252)=2.183m
Base area within the control perimeter
Acont=0.300x0.300+0.300x0.504+0.300x0.504+3.14x0.252x0.252=0.59m^2
Minimum effective height of footing at control section dm= 204mm

Applied shear force at control perimeter Ved=Nsd-σo·Acont, ved=Vedxβ/Ucont
ved=(208.16-172.81)x1.50/2.18=24.29 kN/m, β=1.50 (EC2 §6.4.3 Fig.6.21N)

Tension reinforcement at control section Asxx= 4.11cm^2/m, Asyy= 4.11cm^2/m
As1^2=(Asxx)(Asyy)=4.11x4.11, As1=4.11 cm^2

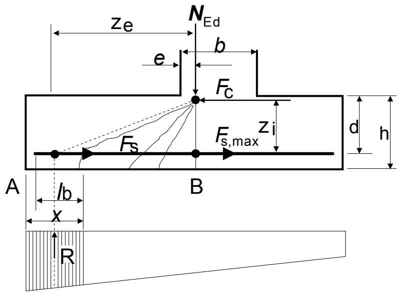
Punching shear capacity without shear reinforcement Vrdc (EC2 §6.4.4)



Vrdc=[Crdc·k·(100ρ1·fck)^(0.333)·(2d/a)]·bw·d (EC2 Eq.6.50)
Vrdc>=[vmin·2d/a]·bw·d, d=dm=204mm, a=504mm
Crdc=0.18/γc=0.18/1.50=0.120, fck=25.00MPa
k=1+(200/d)^½ <=2, k=1.99
ρ1=As1/(bw·d)=411/(1000x204)=0.0020
vmin=0.035·k^(1.50)·fck^½ = 0.49N/mm^2 (EC2 Eq.6.3N)
Vrd,c(min)=0.001x(0.49x2x204/504)x1000x204=80.92kN/m
Vrdc=0.001x[0.120x1.99x(0.20x25.00)^(0.333)x2x204/504]x1000x204=67.43, Vrdc=Vrcd(min)=80.92kN/m
Vsd=24.29 kN/m <= Vrdc=80.92 kN/m, shear and punching shear OK

8.8. Reinforcement anchorage

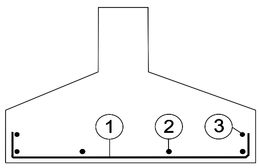
(EC2 EN1992-1-1:2004, §9.8.2.2, §8.4)

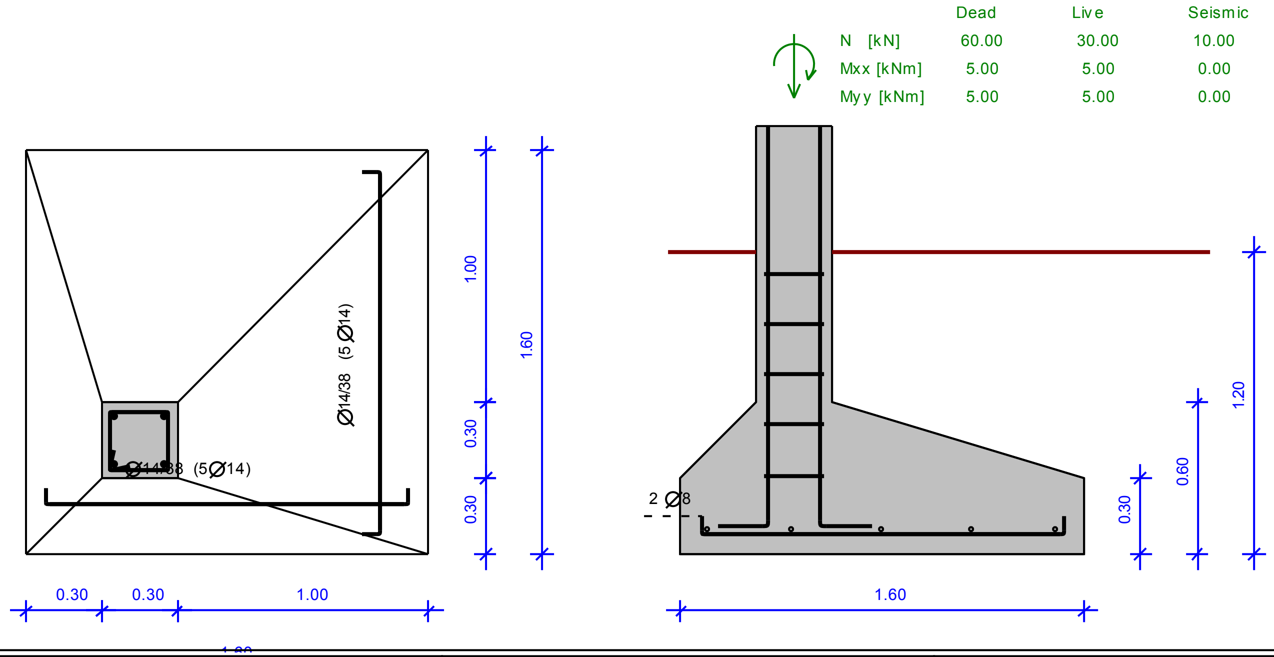


x=h/2=0.150m, R=1000x0.169x0.150x1.600=40.51 kN
e=0.15b=0.045m ze=0.620 m, zi=0.900d=0.454m
Fs=R·ze/zi=40.51x0.620/0.454=55.37 kN
σsd=Fs/As=1000x55.37/770=72 MPa
Basic required anchorage length (EC2 Eq.8.3)
lb,rqd=(Φ/4)(σsd/fbd)=(14/4)x(72/2.70)=93mm
fbd=2.25x1.00x(fctk0.05/γc)=2.70 MPa (EC2 §8.4.2)
Design anchorage length (EC2 §8.4.4, T.8.2)
lbd=0.70x93=65mm, Cnom=75mm>3Ø=42mm
Minimum anchorage length lb,min=max(0.30lbrqd,10Ø,100mm)=140mm
Necessary anchorage length of longitudinal reinforcement Lbd=140mm =0.140m
lbd=140mm>(x-Cnom)=75.00. Necessary bends 70mm at bar ends for anchorage

8.9. Reinforcing bar schedule

Num	type	reinforcing bar [mm]	items	Ø	g/m [kg/m]	length [m]	weight [kg]
39	1	70 1450 70	5	14	1.210	1.590	9.62
40	2	70 1450 70	5	14	1.210	1.590	9.62
41	3	1450	2	8	0.395	1.450	1.15
42	3	1450	2	8	0.395	1.450	1.15
		Total weight [kg]					**21.54**





General information

Asymmetric footing with eccentric load

Concrete and steel class: C25/30 - S500

Concrete cover: Cnom=75 mm

Soil bearing pressure: 0.20 [N/mm²]

Design codes

Eurocode 0 EN1991-1-1, Basis of structural design

Eurocode 1 EN1991-1-1, Actions on structures

Eurocode 2 EN1992-1-1, Design of concrete structures

Eurocode 7 EN1997-1-1, Geotechnical design

Eurocode 8 EN1998-5, Earthquaqe design

Loads

	Dead	Live	Seismic
N [kN]	60.00	30.00	10.00
Mxx [kNm]	5.00	5.00	0.00
Myy [kNm]	5.00	5.00	0.00

Reinforcing bar schedule

#		reinforcing bar [mm]	items	Ø [mm]	g/m [kg/m]	length [m]	weight [kg]
1	F1	70 1450 70	5	14	1.210	1.590	9.62
2	F2	70 1450 70	5	14	1.210	1.590	9.62
3	F3	1450	2	8	0.395	1.450	1.15
4	F3	1450	2	8	0.395	1.450	1.15
		Total weight [kg]					**21.54**

Concrete volume of footing V= 1.08 [m³]

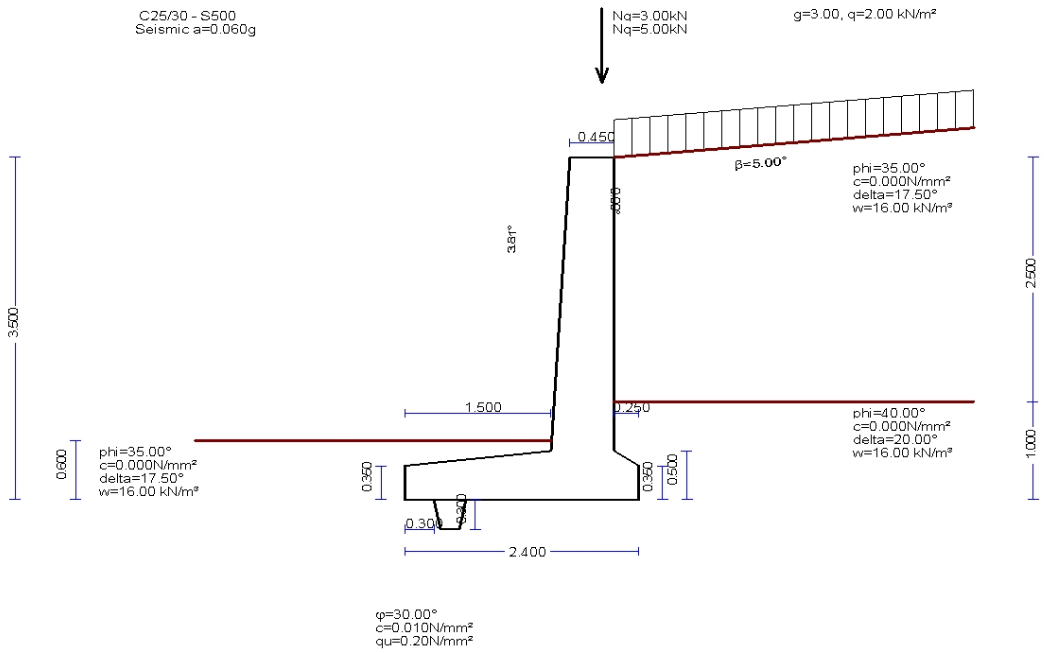
Reinforcement weight G=21.54 [kg]

Project: Design examples 12/03/2007	
FOOTING-001	
Scale : 1:30	**Date:** 13/03/2007
Designer:	**Draw.No.:**
Filename: Design examples	**Sign:**
RUNET Norway as	
BETONexpress www.runet-software.com	

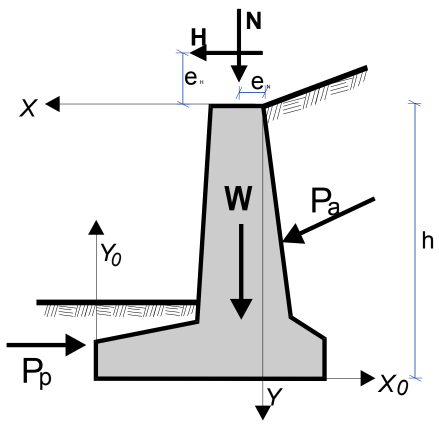
9. C. WALL-001

Cantilever concrete wall

(EC2 EN1992-1-1:2004, EC0 EN1990-1-1:2002, EC7 EN1997-1-1:2004, EC8 EN1998-5:2004)



9.1. Wall properties-Parameters-Code requirements



Dimensions

Height of wall	h=	3.500 m
Transverse length of wall	L=	10.000 m
Steam thickness at top	B1=	0.450 m
Steam thickness at bottom	B2=	0.650 m
Width of wall base	B=	2.400 m
Width of wall toe		1.500 m
Width of wall heel		0.250 m
Height of wall steam		3.000 m
Thickness of wall footing		0.500 m
Front thickness of wall toe		0.350 m
Back thickness of wall heel		0.350 m
Slope (batter) at frontface		3.814° (1:15.0)
Slope (batter) at backface		0.000° (0:1)
Height of wall base key		0.300 m

Loads on wall top

Vertical permanent load	Ng=	3.00 kN/m
Vertical variable load	Nq=	5.00 kN/m
Eccentricity of vertical load	eN=	0.13 kN/m
Horizontal permanent load	Hg=	0.00 kN/m
Horizontal variable load	Hq=	0.00 kN/m
Eccentricity of horizontal load	eH=	0.00 kN/m

Weight of wall

Unit weight of wall material γg=25.000 kN/m³
Cross section area of wall A= 2.719 m²
Self weight per meter of wall W= 2.719x25.000= 67.97 kN/m
Center of gravity of wall at x=0.521 m, y=2.253 m (xo=1.629 m, yo=1.247 m)

Wall materials

Steam : Concrete-Steel class: C25/30-S500 (EN1992-1-1, §3)
: Concrete cover: Cnom=25 mm (EN1992-1-1, §4.4.1)
Footing : Concrete-Steel class: C25/30-S500
: Concrete cover: Cnom=75 mm

Weight of backfill

Weight of backfill per meter Ws=12.34 kN/m
Center of gravity of backfill x=-0.125 m, y=1.532 m

9.2. Partial factors for actions and soil properties (EC7 Tables A.1-A.4, EC8-5 §3.1)

Equilibrium limit state (EQU), Structural limit state (STR), Geotechnical limit state (GEO)

			(EQU)	(STR)	(GEO)	(SEISMIC)
Actions	Permanent Unfavourable	γGdst:	1.10	1.35	1.00	1.00
	Permanent Favourable	γGstb:	0.90	1.00	1.00	1.00
	Variable Unfavourable	γQdst:	1.50	1.50	1.30	1.00
	Variable Favourable	γQstb:	0.00	0.00	0.00	0.00
Soil	Angle of shearing resistance	γφ:	1.25	1.00	1.25	1.25
parameters	Effective cohesion	γc:	1.25	1.00	1.25	1.25
	Undrained shear strength	γcu:	1.40	1.00	1.40	1.40
	Unconfined strength	γqu:	1.40	1.00	1.40	1.40
	Weight density	γw:	1.00	1.00	1.00	1.00

9.3. Properties of foundation soil

Bearing capacity of foundation soil qu=0.20 N/mm²
Friction angle between wall footing and soil φ=30.00°, friction coefficient tan(φ)=0.577
Cohesion between wall footing and soil c=0.010 N/mm²

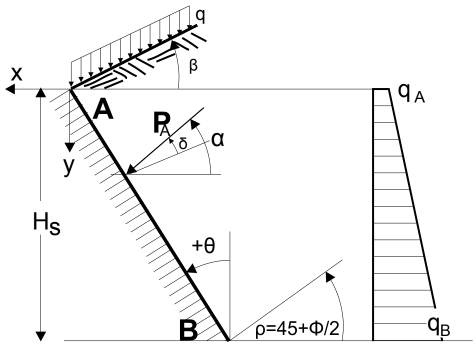
9.4. Seismic coefficients (EC8 EN1998-5:2004, §7.3.2)

Design ground acceleration ratio gh=axg, a=0.06 (EC8-5 §7.3.2)
Reduction factor for seismic coefficient r=1.50 (EC8-5 Table 7.1)
Coefficient for horizontal seismic force kh=0.06/1.500=0.040 (EC8-5 Eq.7.1)
Coefficient for vertical seismic force kv=0.50x0.040=0.020 (EC8-5 Eq.7.2)

Forces due to seismic load (except from earth pressure)

Horizontal seismic force due to self weight Fwx= 67.97x0.040= 2.72 kN/m
Vertical seismic force due to self weight Fwy= 67.97x0.020= 1.36 kN/m
Horizontal seismic force of top loading Ng Fgx= 3.00x0.040= 0.12 kN/m
Vertical seismic force of top loading Ng Fgy= 3.00x0.020= 0.06 kN/m
Horizontal seismic force of top loading Nq Fqx= 5.00x0.040= 0.20 kN/m
Vertical seismic force of top loading Nq Fqy= 5.00x0.020= 0.10 kN/m
Horizontal seismic force of backfill Fwsx= 12.34x0.040= 0.49 kN/m
Vertical seismic force of backfill Fwsy= 12.34x0.020= 0.25 kN/m

9.5. Computation of active earth pressure (Coulomb theory)



9.5.1. Wall part from y=0.000 m to y=2.500 m, Hs=2.500 m

Top point A x= 0.000 m y= 0.000 m
Bottom point B x= 0.000 m y= 2.500 m

Soil properties

Soil type : Thin gravel

Unit weight of soil	γ =16.00 kN/m³
Unit weight of soil (saturated)	γs=20.00 kN/m³
Unit weight of water	γw=10.00 kN/m³
Angle of shearing resistance of ground	φ=35.00°
Cohesion of ground	c=0.000 N/mm²
Slope angle of ground surface	β= 5.00°
Inclination angle of the wall backface	θ= 0.00°
Angle of shear resist. between ground-wall	δ=17.50°

Loads on soil surface

Permanent uniform load	g= 3.00 kN/m²
Variable uniform load	q= 2.00 kN/m²

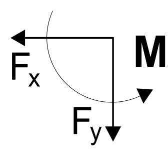
Earth pressure according to Coulomb theory

$$K_A = \frac{\cos^2(\varphi-\theta)}{\cos^2\theta\,\cos(\theta+\delta)\left[1+\sqrt{\frac{\sin(\theta+\delta)\sin(\theta-\beta)}{\cos(\theta+\delta)\cos(\theta-\beta)}}\right]^2}$$

	EQU	STR	GEO
Angle of rupture plane ρ=45°+φ/2 =	59.00	62.50	59.00°
Coefficient of active earth pressure Ka=	0.348	0.260	0.348

Earth pressure q(y)=qA+γ·y·Ka

Permanent actions



		EQU	STR	GEO	
Earth pressure at the top (y=yA)	qA=	1.04	0.78	1.04	kN/m²
Earth pressure at the bottom (y=yA+ 2.50m)	qB=	14.96	11.18	14.96	kN/m²
Earth force Pa=½(qA+qB)H	Pa=	20.00	14.95	20.00	kN/m
Angle of earth force	α =	14.00	17.50	14.00	°
Earth force in x direction	Pax=	19.07	14.26	19.07	kN/m
Earth force in y direction	Pay=	6.01	4.50	6.01	kN/m
Moment of earth force at top point (x=0,y=0)	M =	-30.74	-22.99	-30.74	kNm/m

Point of application of earth force x= 0.000 m, y= 1.612 m

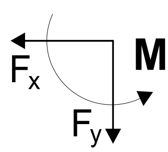
Variable actions

		EQU	STR	GEO	
Earth pressure at the top (y=yA)	qA=	0.70	0.52	0.70	kN/m²
Earth pressure at the bottom (y=yA+ 2.50m)	qB=	0.70	0.52	0.70	kN/m²
Earth force Pa=½(qA+qB)H	Pa=	1.75	1.30	1.75	kN/m
Angle of earth force	α =	14.00	17.50	14.00	°
Earth force in x direction	Pax=	1.67	1.24	1.67	kN/m
Earth force in y direction	Pay=	0.53	0.39	0.53	kN/m
Moment of earth force at top point (x=0,y=0)	M =	-2.09	-1.55	-2.09	kNm/m

Point of application of earth force x= 0.000 m, y= 1.250 m

Total forces and moments

Forces and moments at bottom point B (x=0.000 m, y=2.500 m)



Permanent actions

		EQU	STR	GEO	
Total horizontal earth force	Fsx=	19.07	14.26	19.07	kN/m
Total vertical earth force	Fsy=	6.01	4.50	6.01	kN/m
Total moment of earth force	Ms =	16.93	12.66	16.93	kNm/m

Variable actions

	EQU	STR	GEO	
Total horizontal earth force Fsx=	1.67	1.24	1.67	kN/m
Total vertical earth force Fsy=	0.53	0.39	0.53	kN/m
Total moment of earth force Ms =	2.09	1.55	2.09	kNm/m

Seismic loading (EC8 EN1998-5:2004, §7.3.2, Annex E)

Horizontal seismic coefficient kh=0.06/1.500=0.040 (EC8-5 Eq.7.1, T.7.1)
Vertical seismic coefficient kv=0.50x0.040=0.020 (EC8-5 Eq.7.2)
Soil above the water table (EC8-5 Annex E.5)
tan(ω)=kh/(1-kv)=0.040/(1-0.020)=0.041, ω=2.34°

Method Mononobe-Okabe (EC8-5 Annex E.4)
for active earth force during seismic loading
Coefficient of active earth pressure, Ke*= 0.372
Additional earth pressure due to seismic load
over STR load case ξ=(Ke*/Ke-1)=(0.372/0.260-1)=0.431

$$K_E=\frac{\cos^2(\varphi-\omega-\theta)}{\cos\omega\,\cos^2\theta\,\cos(\delta+\theta+\omega)\left[1+\sqrt{\frac{\sin(\varphi+\delta)\sin(\varphi-\omega-\beta)}{\cos(\theta+\omega+\delta)\,\cos(\theta-\beta)}}\right]^2}$$

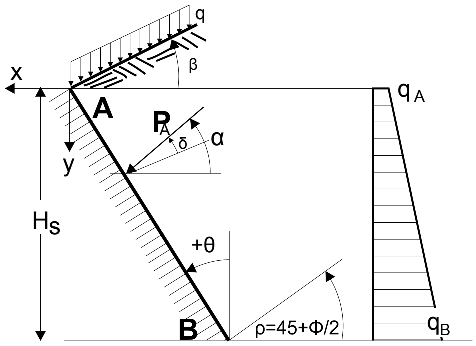
Earth force due to seismic load (Permanent actions) Fx=1.431x14.26=20.41 kN/m
Earth force due to seismic load (Variable actions) Fx=1.431x 1.24= 1.77 kN/m

9.5.2. Wall part from y=2.500 m to y=3.500 m, Hs=1.000 m

Top point A x= 0.000 m y= 2.500 m
Bottom point B x= 0.000 m y= 3.500 m

Soil properties

Soil type : Mean gravel
Unit weight of soil γ =16.00 kN/m³
Unit weight of soil (saturated) γs=20.00 kN/m³
Unit weight of water γw=10.00 kN/m³
Angle of shearing resistance of ground φ=40.00°
Cohesion of ground c=0.000 N/mm²
Slope angle of ground surface β= 0.00°
Inclination angle of the wall backface θ= 0.00°
Angle of shear resist. between ground-wall δ=20.00°



Loads on soil surface

Permanent uniform load g= 43.00 kN/m²
Variable uniform load q= 2.00 kN/m²

Earth pressure according to Coulomb theory

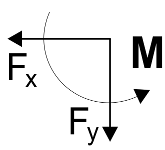
	EQU	STR	GEO
Angle of rupture plane ρ=45°+φ/2 =	61.00	65.00	61.00°
Coefficient of active earth pressure Ka=	0.278	0.199	0.278

Earth pressure q(y)=qA+γ·y·Ka

$$K_A=\frac{\cos^2(\varphi-\theta)}{\cos^2\theta\,\cos(\theta+\delta)\left[1+\sqrt{\frac{\sin(\theta+\delta)\sin(\theta-\beta)}{\cos(\theta+\delta)\,\cos(\theta-\beta)}}\right]^2}$$

Permanent actions

		EQU	STR	GEO	
Earth pressure at the top (y=yA)	qA=	11.95	8.56	11.95	kN/m²
Earth pressure at the bottom (y=yA+ 1.00m)	qB=	16.40	11.74	16.40	kN/m²
Earth force Pa=½(qA+qB)H	Pa=	14.17	10.15	14.17	kN/m
Angle of earth force	α =	16.00	20.00	16.00	°
Earth force in x direction	Pax=	13.32	9.54	13.32	kN/m
Earth force in y direction	Pay=	4.85	3.47	4.85	kN/m
Moment of earth force at top point (x=0,y=0)	M =	-40.31	-28.87	-40.31	kNm/m



Point of application of earth force x= 0.000 m, y= 3.026 m

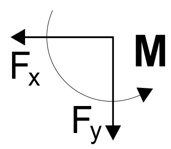
Variable actions

		EQU	STR	GEO	
Earth pressure at the top (y=yA)	qA=	0.56	0.40	0.56	kN/m²
Earth pressure at the bottom (y=yA+ 1.00m)	qB=	0.56	0.40	0.56	kN/m²
Earth force Pa=½(qA+qB)H	Pa=	0.56	0.40	0.56	kN/m
Angle of earth force	α =	16.00	20.00	16.00	°
Earth force in x direction	Pax=	0.53	0.38	0.53	kN/m
Earth force in y direction	Pay=	0.19	0.14	0.19	kN/m
Moment of earth force at top point (x=0,y=0)	M =	-1.59	-1.14	-1.59	kNm/m

Point of application of earth force x= 0.000 m, y= 3.000 m

Total forces and moments

Forces and moments at bottom point B (x=0.000 m, y=3.500 m)



Permanent actions

		EQU	STR	GEO	
Total horizontal earth force	Fsx=	32.39	23.80	32.39	kN/m
Total vertical earth force	Fsy=	10.86	7.97	10.86	kN/m
Total moment of earth force	Ms =	42.31	31.44	42.31	kNm/m

Variable actions

		EQU	STR	GEO	
Total horizontal earth force	Fsx=	2.20	1.62	2.20	kN/m
Total vertical earth force	Fsy=	0.72	0.53	0.72	kN/m
Total moment of earth force	Ms =	4.02	2.98	4.02	kNm/m

Seismic loading

(EC8 EN1998-5:2004, §7.3.2, Annex E)

Horizontal seismic coefficient kh=0.06/1.500=0.040 (EC8-5 Eq.7.1, T.7.1)
Vertical seismic coefficient kv=0.50x0.040=0.020 (EC8-5 Eq.7.2)
Soil above the water table (EC8-5 Annex E.5)
tan(ω)=kh/(1-kv)=0.040/(1-0.020)=0.041, ω=2.34°

Method Mononobe-Okabe (EC8-5 Annex E.4)
for active earth force during seismic loading
Coefficient of active earth pressure, Ke*= 0.298
Additional earth pressure due to seismic load
over STR load case ξ=(Ke*/Ke-1)=(0.298/0.199-1)=0.497

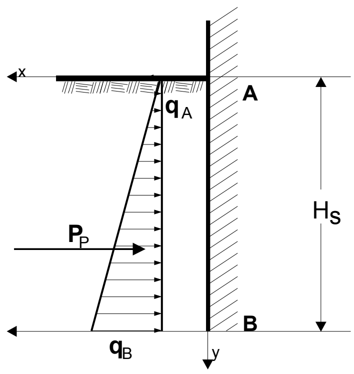
$$K_E=\frac{\cos^2(\varphi-\omega-\theta)}{\cos\omega\,\cos^2\theta\,\cos(\delta+\theta+\omega)\left[1+\sqrt{\frac{\sin(\varphi+\delta)\sin(\varphi-\omega-\beta)}{\cos(\theta+\omega+\delta)\cos(\theta-\beta)}}\right]^2}$$

Earth force due to seismic load (Permanent actions) Fx=1.497x 9.54=14.28 kN/m
Earth force due to seismic load (Variable actions) Fx=1.497x 0.38= 0.57 kN/m

9.6. Computation of passive earth pressure (Rankine theory)

9.6.1. Wall part from y=2.900 m to y=3.800 m, Hs=0.900 m

Top point A x= 2.150 m y= 2.900 m
Bottom point B x= 2.150 m y= 3.800 m



Soil properties

Soil type : Thin gravel

Unit weight of soil	γ =16.00 kN/m³
Unit weight of soil (saturated)	γs=20.00 kN/m³
Unit weight of water	γw=10.00 kN/m³
Angle of shearing resistance of ground	φ=35.00°
Cohesion of ground	c=0.000 N/mm²
Slope angle of ground surface	β= 0.00°
Earth pressure on vertical surface	θ= 0.00°
Angle of shear resist. between ground-wall	δ= 0.00°

Earth pressure according to Coulomb theory

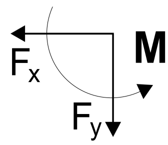
		EQU	STR	GEO
Angle of rupture plane ρ=45°-φ/2	=	31.00	27.50	31.00°
Coefficient of passive earth pressure	Kp=	2.770	3.690	2.770

Earth pressure q(y)=qA+γ·y·Kp

$$K_p = \frac{\cos^2(\varphi+\theta)}{\cos^2\theta\cos(\theta-\delta)\left[1-\sqrt{\frac{\sin(\theta+\delta)\sin(\theta+\beta)}{\cos(\theta-\delta)\cos(\theta-\beta)}}\right]^2}$$

Permanent actions

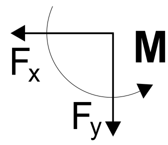
		EQU	STR	GEO	
Earth pressure at the top (y=yA)	qA=	0.00	0.00	0.00	kN/m²
Earth pressure at the bottom (y=yA+ 0.90m)	qB=	-39.89	-53.14	-39.89	kN/m²
Earth force Pa=½(qA+qB)H	Pp=	17.95	23.91	17.95	kN/m
Angle of earth force	α =	0.00	0.00	0.00	°
Earth force in x direction	Ppx=	-17.95	-23.91	-17.95	kN/m
Earth force in y direction	Ppy=	0.00	0.00	0.00	kN/m
Moment of earth force at top point (x=0,y=0)	M =	62.83	83.68	62.83	kNm/m



Point of application of earth force x= 2.150 m, y= 3.500 m

Total forces and moments

Forces and moments at bottom point B (x=2.150 m, y=3.800 m)



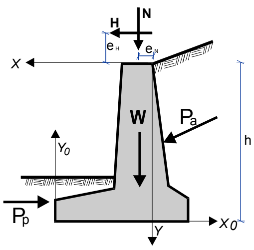
Permanent actions

		EQU	STR	GEO	
Total horizontal earth force	Fsx=	-17.95	-23.91	-17.95	kN/m
Total vertical earth force	Fsy=	0.00	0.00	0.00	kN/m
Total moment of earth force	Ms =	-5.38	-7.17	-5.38	kNm/m

9.7. Checks of wall stability (EQU)

9.7.1. Forces (driving and resisting) on the wall (EQU)

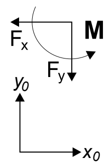
Action		y1 - y2	Fx [kN/m]	Fy [kN/m]	x [m]	y [m]
Active earth pressure	Pa	0.00- 2.50	19.07	6.01	0.000	1.612
Backfill surcharge (live)	Pq	0.00- 2.50	1.67	0.53	0.000	1.250
Active earth pressure	Pa	2.50- 3.50	13.32	4.85	0.000	3.026
Backfill surcharge (live)	Pq	2.50- 3.50	0.53	0.19	0.000	3.000
Passive earth pressure	Pp	2.90- 3.80	-17.95	0.00	2.150	3.500
Wall weight	W		0.00	67.97	0.521	2.253
Backfill weight	Ws		0.00	12.34	-0.125	1.532
Vert. load on top (dead)	Ng		0.00	3.00	0.125	0.000
Vert. load on top (live)	Nq		0.00	5.00	0.125	0.000



9.7.2. Check of soil bearing capacity (EQU)

(EC7 EN1997-1-1:2004, §6.5.2)

Check for 0.90x(self weight+top vertical dead load)+0.00x(top vertical live load)



Action	(γ)	y1 - y2	Fx [kN/m]	Fy [kN/m]	xo [m]	yo [m]	M [kNm/m]
Active earth pressure	Pax1.10	0.00- 2.50	20.98	6.61	2.150	1.888	25.39
Backfill surcharge (live)	Pqx1.50	0.00- 2.50	2.50	0.79	2.150	2.250	3.93
Active earth pressure	Pax1.10	2.50- 3.50	14.65	5.34	2.150	0.474	-4.53
Backfill surcharge (live)	Pqx1.50	2.50- 3.50	0.79	0.28	2.150	0.500	-0.22
Wall weight	W x0.90		0.00	61.17	1.629	1.247	-99.66
Backfill weight	Wsx0.90		0.00	11.11	2.275	1.968	-25.26
Vert. load on top (dead)	Ngx0.90		0.00	2.70	2.025	3.500	-5.47
			Sum=	88.00			-105.82

```
Sum of vertical forces           =   88.00 kN/m
Sum of moments at front toe      = -105.82 kNm/m
Sum of moments at middle of base =   -0.22 kNm/m
Eccentricity  ec=-0.22/88.00=-0.003m, ec<=2.400/6=0.400m
Soil pressure q1=0.036 N/mm²  q2=0.037 N/mm²
Effective footing L=2.400-2x0.003= 2.395 m                                  (EC7 Annex D)
Soil bearing capacity Rd=L·qu/γM=2.395x(1000x0.20)/1.40= 342.14 kN/m
Bearing resistance check Vd=88.00 < Rd=342.14 kN/m,  Check is verified      (EC7 Eq.2.2, Eq.6.1)
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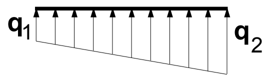
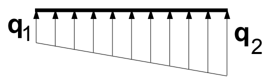


Check for 1.10x(self weight+top vertical dead load)+1.50x(top vertical live load)

Action	(γ)	y1 - y2	Fx [kN/m]	Fy [kN/m]	xo [m]	yo [m]	M [kNm/m]
Active earth pressure	Pax1.10	0.00- 2.50	20.98	6.61	2.150	1.888	25.39
Backfill surcharge (live)	Pqx1.50	0.00- 2.50	2.50	0.79	2.150	2.250	3.93
Active earth pressure	Pax1.10	2.50- 3.50	14.65	5.34	2.150	0.474	-4.53
Backfill surcharge (live)	Pqx1.50	2.50- 3.50	0.79	0.28	2.150	0.500	-0.22
Wall weight	W x1.10		0.00	74.77	1.629	1.247	-121.80
Backfill weight	Wsx1.10		0.00	13.57	2.275	1.968	-30.88
Vert. load on top (dead)	Ngx1.10		0.00	3.30	2.025	3.500	-6.69
Vert. load on top (live)	Nqx1.50		0.00	7.50	2.025	3.500	-15.20
			Sum=	112.16			-150.00

```
Sum of vertical forces           =  112.16 kN/m
Sum of moments at front toe      = -150.00 kNm/m
Sum of moments at middle of base =  -15.41 kNm/m
Eccentricity  ec=-15.41/112.16=-0.137m, ec<=2.400/6=0.400m
Soil pressure q1=0.031 N/mm²  q2=0.063 N/mm²
Effective footing L=2.400-2x0.137= 2.125 m                                  (EC7 Annex D)
Soil bearing capacity Rd=L·qu/γM=2.125x(1000x0.20)/1.40= 303.57 kN/m
Bearing resistance check Vd=112.16 < Rd=303.57 kN/m,  Check is verified     (EC7 Eq.2.2, Eq.6.1)
```

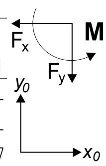


9.7.3. Failure check due to overturning (EQU)

(EC7 EN1997-1-1:2004, §9.7.4)

Overturning with respect to the toe (xo=0,yo=0) (x=2.150,y=3.500 m)

Action	(γ)	y1 - y2	Fx [kN/m]	Fy [kN/m]	xo [m]	yo [m]	Mo+ [kNm/m]	Mo- [kNm/m]
Active earth pressure	Pax1.10	0.00- 2.50	20.98	6.61	2.150	1.888	39.60	14.21
Backfill surcharge (live)	Pqx1.50	0.00- 2.50	2.50	0.79	2.150	2.250	5.64	1.71
Active earth pressure	Pax1.10	2.50- 3.50	14.65	5.34	2.150	0.474	6.94	11.47
Backfill surcharge (live)	Pqx1.50	2.50- 3.50	0.79	0.28	2.150	0.500	0.39	0.62
Wall weight	W x0.90		0.00	61.17	1.629	1.247	0.00	99.66
Backfill weight	Wsx0.90		0.00	11.11	2.275	1.968	0.00	25.26
Vert. load on top (dead)	Ngx0.90		0.00	2.70	2.025	3.500	0.00	5.47
					Sum=		52.57	158.40

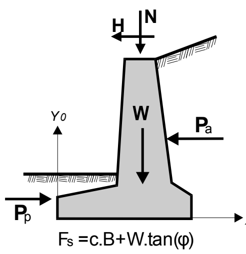


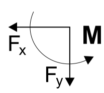

```
Sum of overturning moments           =   52.57 kNm/m
Sum of moments resisting overturning =  158.40 kNm/m
Overturning check Msd=52.57 < Mrd=158.40 kNm/m,  Check is verified
```

9.7.4. Failure check against sliding (EQU)

(EC7 EN1997-1-1:2004, §9.7.3, §6.5.3)

Action	(γ)	y1 - y2	Fx+ [kN/m]	Fx- [kN/m]	Fy [kN/m]
Active earth pressure	Pax1.10	0.00- 2.50	20.98	0.00	6.61
Backfill surcharge (live)	Pqx1.50	0.00- 2.50	2.50	0.00	0.79
Active earth pressure	Pax1.10	2.50- 3.50	14.65	0.00	5.34
Backfill surcharge (live)	Pqx1.50	2.50- 3.50	0.79	0.00	0.28
Passive earth pressure	Ppx0.90	2.90- 3.80	0.00	16.15	0.00
Wall weight	W x0.90		0.00	0.00	61.17
Backfill weight	Wsx0.90		0.00	0.00	11.11
Vert. load on top (dead)	Ngx0.90		0.00	0.00	2.70
		Sum=	38.92	16.15	88.00



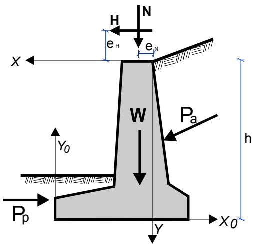


Soil friction Rd=Vd·tanφ/γM= 88.00xtan(30.00°)/1.25= 40.65 kN/m
Soil cohesion Rd=A·cu/γM = 1000x2.400x0.010/1.25= 19.20 kN/m
(resisting forces from effective cohesion are neglected) (EC7 §6.5.3. 10)
Sum of driving forces = 38.92 kN/m
Sum of resisting forces (16.15+40.65) = 56.80 kN/m
Sliding resistance check Hd=38.92 < Rd=56.80 kN/m, Check is verified

9.8. Checks of wall stability (STR)

9.8.1. Forces (driving and resisting) on the wall (STR)

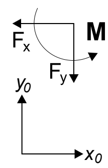
Action		y1 - y2	Fx [kN/m]	Fy [kN/m]	x [m]	y [m]
Active earth pressure	Pa	0.00- 2.50	14.26	4.50	0.000	1.612
Backfill surcharge (live)	Pq	0.00- 2.50	1.24	0.39	0.000	1.250
Active earth pressure	Pa	2.50- 3.50	9.54	3.47	0.000	3.026
Backfill surcharge (live)	Pq	2.50- 3.50	0.38	0.14	0.000	3.000
Passive earth pressure	Pp	2.90- 3.80	-23.91	0.00	2.150	3.500
Wall weight	W		0.00	67.97	0.521	2.253
Backfill weight	Ws		0.00	12.34	-0.125	1.532
Vert. load on top (dead)	Ng		0.00	3.00	0.125	0.000
Vert. load on top (live)	Nq		0.00	5.00	0.125	0.000



9.8.2. Check of soil bearing capacity (STR)

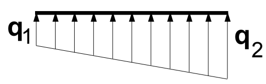
(EC7 EN1997-1-1:2004, §6.5.2)

Check for 1.00x(self weight+top vertical dead load)+0.00x(top vertical live load)



Action	(γ)	y1 - y2	Fx [kN/m]	Fy [kN/m]	xo [m]	yo [m]	M [kNm/m]
Active earth pressure	Pax1.35	0.00- 2.50	19.25	6.08	2.150	1.888	23.27
Backfill surcharge (live)	Pqx1.50	0.00- 2.50	1.86	0.58	2.150	2.250	2.93
Active earth pressure	Pax1.35	2.50- 3.50	12.88	4.68	2.150	0.474	-3.97
Backfill surcharge (live)	Pqx1.50	2.50- 3.50	0.57	0.21	2.150	0.500	-0.16
Wall weight	W x1.00		0.00	67.97	1.629	1.247	-110.73
Backfill weight	Wsx1.00		0.00	12.34	2.275	1.968	-28.07
Vert. load on top (dead)	Ngx1.00		0.00	3.00	2.025	3.500	-6.08
			Sum=	94.86			-122.81

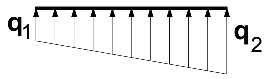
Sum of vertical forces = 94.86 kN/m
Sum of moments at front toe = -122.81 kNm/m
Sum of moments at middle of base = -8.98 kNm/m
Eccentricity ec=-8.98/94.86=-0.095m, ec<=2.400/6=0.400m
Soil pressure q1=0.030 N/mm² q2=0.049 N/mm²
Effective footing L=2.400-2x0.095= 2.211 m (EC7 Annex D)
Soil bearing capacity Rd=L·qu/γM=2.211x(1000x0.20)/1.00= 442.20 kN/m
Bearing resistance check Vd=94.86 < Rd=442.20 kN/m, Check is verified (EC7 Eq.2.2, Eq.6.1)



Check for 1.35x(self weight+top vertical dead load)+1.50x(top vertical live load)

Action	(γ)	y1 - y2	Fx [kN/m]	Fy [kN/m]	xo [m]	yo [m]	M [kNm/m]
Active earth pressure	Pax1.35	0.00- 2.50	19.25	6.08	2.150	1.888	23.27
Backfill surcharge (live)	Pqx1.50	0.00- 2.50	1.86	0.58	2.150	2.250	2.93
Active earth pressure	Pax1.35	2.50- 3.50	12.88	4.68	2.150	0.474	-3.97
Backfill surcharge (live)	Pqx1.50	2.50- 3.50	0.57	0.21	2.150	0.500	-0.16
Wall weight	W x1.35		0.00	91.76	1.629	1.247	-149.49
Backfill weight	Wsx1.35		0.00	16.66	2.275	1.968	-37.89
Vert. load on top (dead)	Ngx1.35		0.00	4.05	2.025	3.500	-8.21
Vert. load on top (live)	Nqx1.50		0.00	7.50	2.025	3.500	-15.20
			Sum=	131.52			-188.72

Sum of vertical forces = 131.52 kN/m
Sum of moments at front toe = -188.72 kNm/m
Sum of moments at middle of base = -30.90 kNm/m
Eccentricity ec=-30.90/131.52=-0.235m, ec<=2.400/6=0.400m
Soil pressure q1=0.023 N/mm² q2=0.087 N/mm²
Effective footing L=2.400-2x0.235= 1.930 m (EC7 Annex D)
Soil bearing capacity Rd=L·qu/γM=1.930x(1000x0.20)/1.00= 386.00 kN/m
Bearing resistance check Vd=131.52 < Rd=386.00 kN/m, Check is verified (EC7 Eq.2.2, Eq.6.1)

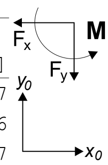


9.8.3. Failure check due to overturning (STR)

(EC7 EN1997-1-1:2004, §9.7.4)

Overturning with respect to the toe (xo=0,yo=0) (x=2.150,y=3.500 m)

Action	(γ)	y1 - y2	Fx [kN/m]	Fy [kN/m]	xo [m]	yo [m]	Mo+ [kNm/m]	Mo- [kNm/m]
Active earth pressure	Pax1.35	0.00- 2.50	19.25	6.08	2.150	1.888	36.34	13.07
Backfill surcharge (live)	Pqx1.50	0.00- 2.50	1.86	0.58	2.150	2.250	4.18	1.26
Active earth pressure	Pax1.35	2.50- 3.50	12.88	4.68	2.150	0.474	6.10	10.07
Backfill surcharge (live)	Pqx1.50	2.50- 3.50	0.57	0.21	2.150	0.500	0.28	0.45
Wall weight	W x1.00		0.00	67.97	1.629	1.247	0.00	110.73
Backfill weight	Wsx1.00		0.00	12.34	2.275	1.968	0.00	28.07
Vert. load on top (dead)	Ngx1.00		0.00	3.00	2.025	3.500	0.00	6.08
						Sum=	46.90	169.73

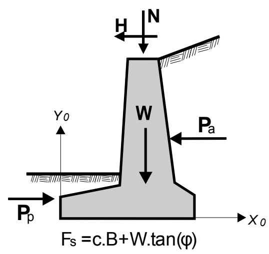


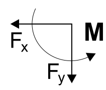
Sum of overturning moments = 46.90 kNm/m
Sum of moments resisting overturning = 169.73 kNm/m
Overturning check Msd=46.90 < Mrd=169.73 kNm/m, Check is verified

9.8.4. Failure check against sliding (STR)

(EC7 EN1997-1-1:2004, §9.7.3, §6.5.3)

Action	(γ)	y1 - y2	Fx+ [kN/m]	Fx- [kN/m]	Fy [kN/m]
Active earth pressure	Pax1.35	0.00- 2.50	19.25	0.00	6.08
Backfill surcharge (live)	Pqx1.50	0.00- 2.50	1.86	0.00	0.58
Active earth pressure	Pax1.35	2.50- 3.50	12.88	0.00	4.68
Backfill surcharge (live)	Pqx1.50	2.50- 3.50	0.57	0.00	0.21
Passive earth pressure	Ppx1.00	2.90- 3.80	0.00	23.91	0.00
Wall weight	W x1.00		0.00	0.00	67.97
Backfill weight	Wsx1.00		0.00	0.00	12.34
Vert. load on top (dead)	Ngx1.00		0.00	0.00	3.00
		Sum=	34.56	23.91	94.86



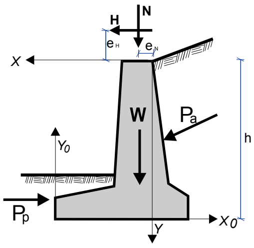


Soil friction Rd=Vd·tanφ/γM= 94.86xtan(30.00°)/1.00= 54.77 kN/m
Soil cohesion Rd=A·cu/γM = 1000x2.400x0.010/1.00= 24.00 kN/m
(resisting forces from effective cohesion are neglected) (EC7 §6.5.3. 10)
Sum of driving forces = 34.56 kN/m
Sum of resisting forces (23.91+54.77) = 78.68 kN/m
Sliding resistance check Hd=34.56 < Rd=78.68 kN/m, Check is verified

9.9. Checks of wall stability (GEO)

9.9.1. Forces (driving and resisting) on the wall (GEO)

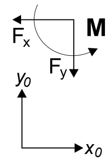
Action		y1 - y2	Fx [kN/m]	Fy [kN/m]	x [m]	y [m]
Active earth pressure	Pa	0.00- 2.50	19.07	6.01	0.000	1.612
Backfill surcharge (live)	Pq	0.00- 2.50	1.67	0.53	0.000	1.250
Active earth pressure	Pa	2.50- 3.50	13.32	4.85	0.000	3.026
Backfill surcharge (live)	Pq	2.50- 3.50	0.53	0.19	0.000	3.000
Passive earth pressure	Pp	2.90- 3.80	-17.95	0.00	2.150	3.500
Wall weight	W		0.00	67.97	0.521	2.253
Backfill weight	Ws		0.00	12.34	-0.125	1.532
Vert. load on top (dead)	Ng		0.00	3.00	0.125	0.000
Vert. load on top (live)	Nq		0.00	5.00	0.125	0.000



9.9.2. Check of soil bearing capacity (GEO)

(EC7 EN1997-1-1:2004, §6.5.2)

Check for 1.00x(self weight+top vertical dead load)+0.00x(top vertical live load)



Action	(γ)	y1 - y2	Fx [kN/m]	Fy [kN/m]	xo [m]	yo [m]	M [kNm/m]
Active earth pressure	Pax1.00	0.00- 2.50	19.07	6.01	2.150	1.888	23.08
Backfill surcharge (live)	Pqx1.30	0.00- 2.50	2.17	0.69	2.150	2.250	3.41
Active earth pressure	Pax1.00	2.50- 3.50	13.32	4.85	2.150	0.474	-4.12
Backfill surcharge (live)	Pqx1.30	2.50- 3.50	0.69	0.25	2.150	0.500	-0.19
Wall weight	W x1.00		0.00	67.97	1.629	1.247	-110.73
Backfill weight	Wsx1.00		0.00	12.34	2.275	1.968	-28.07
Vert. load on top (dead)	Ngx1.00		0.00	3.00	2.025	3.500	-6.08
			Sum=	95.11			-122.70

```
Sum of vertical forces          =   95.11 kN/m
Sum of moments at front toe     = -122.70 kNm/m
Sum of moments at middle of base =   -8.57 kNm/m
Eccentricity  ec=-8.57/95.11=-0.090m, ec<=2.400/6=0.400m
Soil pressure q1=0.031 N/mm²  q2=0.049 N/mm²
Effective footing L=2.400-2x0.090= 2.220 m                          (EC7 Annex D)
Soil bearing capacity Rd=L·qu/γM=2.220x(1000x0.20)/1.40= 317.14 kN/m
Bearing resistance check Vd=95.11 < Rd=317.14 kN/m,  Check is verified    (EC7 Eq.2.2, Eq.6.1)
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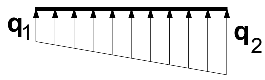

Check for 1.00x(self weight+top vertical dead load)+1.30x(top vertical live load)

Action	(γ)	y1 - y2	Fx [kN/m]	Fy [kN/m]	xo [m]	yo [m]	M [kNm/m]
Active earth pressure	Pax1.00	0.00- 2.50	19.07	6.01	2.150	1.888	23.08
Backfill surcharge (live)	Pqx1.30	0.00- 2.50	2.17	0.69	2.150	2.250	3.41
Active earth pressure	Pax1.00	2.50- 3.50	13.32	4.85	2.150	0.474	-4.12
Backfill surcharge (live)	Pqx1.30	2.50- 3.50	0.69	0.25	2.150	0.500	-0.19
Wall weight	W x1.00		0.00	67.97	1.629	1.247	-110.73
Backfill weight	Wsx1.00		0.00	12.34	2.275	1.968	-28.07
Vert. load on top (dead)	Ngx1.00		0.00	3.00	2.025	3.500	-6.08
Vert. load on top (live)	Nqx1.30		0.00	6.50	2.025	3.500	-13.17
			Sum=	101.61			-135.87

```
Sum of vertical forces          =  101.61 kN/m
Sum of moments at front toe     = -135.87 kNm/m
Sum of moments at middle of base =  -13.94 kNm/m
Eccentricity  ec=-13.94/101.61=-0.137m, ec<=2.400/6=0.400m
Soil pressure q1=0.028 N/mm²  q2=0.057 N/mm²
Effective footing L=2.400-2x0.137= 2.126 m                          (EC7 Annex D)
Soil bearing capacity Rd=L·qu/γM=2.126x(1000x0.20)/1.40= 303.71 kN/m
Bearing resistance check Vd=101.61 < Rd=303.71 kN/m,  Check is verified    (EC7 Eq.2.2, Eq.6.1)
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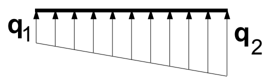
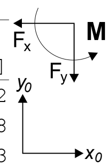


9.9.3. Failure check due to overturning (GEO)

(EC7 EN1997-1-1:2004, §9.7.4)

Overturning with respect to the toe (xo=0,yo=0) (x=2.150,y=3.500 m)



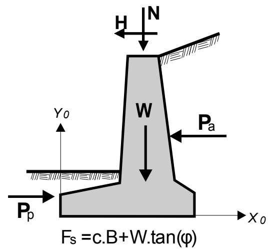
Action	(γ)	y1 - y2	Fx [kN/m]	Fy [kN/m]	xo [m]	yo [m]	Mo+ [kNm/m]	Mo- [kNm/m]
Active earth pressure	Pax1.00	0.00- 2.50	19.07	6.01	2.150	1.888	36.00	12.92
Backfill surcharge (live)	Pqx1.30	0.00- 2.50	2.17	0.69	2.150	2.250	4.89	1.48
Active earth pressure	Pax1.00	2.50- 3.50	13.32	4.85	2.150	0.474	6.31	10.43
Backfill surcharge (live)	Pqx1.30	2.50- 3.50	0.69	0.25	2.150	0.500	0.34	0.53
Wall weight	W x1.00		0.00	67.97	1.629	1.247	0.00	110.73
Backfill weight	Wsx1.00		0.00	12.34	2.275	1.968	0.00	28.07
Vert. load on top (dead)	Ngx1.00		0.00	3.00	2.025	3.500	0.00	6.08
						Sum=	47.54	170.24

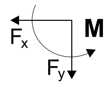
Sum of overturning moments = 47.54 kNm/m
Sum of moments resisting overturning = 170.24 kNm/m
Overturning check Msd=47.54 < Mrd=170.24 kNm/m, Check is verified

9.9.4. Failure check against sliding (GEO)

(EC7 EN1997-1-1:2004, §9.7.3, §6.5.3)

Action	(γ)	y1 - y2	Fx+ [kN/m]	Fx- [kN/m]	Fy [kN/m]
Active earth pressure	Pax1.00	0.00- 2.50	19.07	0.00	6.01
Backfill surcharge (live)	Pqx1.30	0.00- 2.50	2.17	0.00	0.69
Active earth pressure	Pax1.00	2.50- 3.50	13.32	0.00	4.85
Backfill surcharge (live)	Pqx1.30	2.50- 3.50	0.69	0.00	0.25
Passive earth pressure	Ppx1.00	2.90- 3.80	0.00	17.95	0.00
Wall weight	W x1.00		0.00	0.00	67.97
Backfill weight	Wsx1.00		0.00	0.00	12.34
Vert. load on top (dead)	Ngx1.00		0.00	0.00	3.00
		Sum=	35.25	17.95	95.11





Soil friction Rd=Vd·tanφ/γM= 95.11xtan(30.00°)/1.25= 43.93 kN/m
Soil cohesion Rd=A·cu/γM = 1000x2.400x0.010/1.25= 19.20 kN/m
(resisting forces from effective cohesion are neglected) (EC7 §6.5.3. 10)
Sum of driving forces = 35.25 kN/m
Sum of resisting forces (17.95+43.93) = 61.88 kN/m
Sliding resistance check Hd=35.25 < Rd=61.88 kN/m, Check is verified

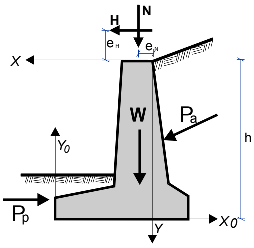
9.10. Seismic design

(EC8 EN1998-5:2004)

Checks of wall stability (with seismic loading)

9.10.1. Forces (driving and resisting) on the wall

Action		y1 - y2	Fx [kN/m]	Fy [kN/m]	x [m]	y [m]
Active earth pressure	Pa	0.00- 2.50	14.26	4.50	0.000	1.612
Backfill surcharge (live)	Pq	0.00- 2.50	1.24	0.39	0.000	1.250
Active earth pressure	Pa	2.50- 3.50	9.54	3.47	0.000	3.026
Backfill surcharge (live)	Pq	2.50- 3.50	0.38	0.14	0.000	3.000
Passive earth pressure	Pp	2.90- 3.80	-23.91	0.00	2.150	3.500
Wall weight	W		0.00	67.97	0.521	2.253
Backfill weight	Ws		0.00	12.34	-0.125	1.532
Vert. load on top (dead)	Ng		0.00	3.00	0.125	0.000
Vert. load on top (live)	Nq		0.00	5.00	0.125	0.000



9.10.2. Additional forces due to seismic load

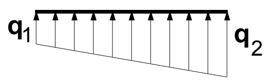
Action		y1 - y2	Fx [kN/m]	Fy [kN/m]	x [m]	y [m]
Active earth pressure	Pa	0.00- 2.50	6.15		0.000	1.612
Backfill surcharge (live)	Pq	0.00- 2.50	0.53		0.000	1.250
Active earth pressure	Pa	2.50- 3.50	4.74		0.000	3.026
Backfill surcharge (live)	Pq	2.50- 3.50	0.19		0.000	3.000
Wall weight	W		2.72	-1.36	0.521	2.253
Backfill weight	Ws		0.49	-0.25	-0.125	1.532
Vert. load on top (dead)	Ng		0.12	-0.06	0.125	0.000
Vert. load on top (live)	Nq		0.20	-0.10	0.125	0.000

9.10.3. Check of soil bearing capacity (with seismic loading)

(EC7 §6.5.2)

Action	(γ)	y1 - y2	Fx [kN/m]	Fy [kN/m]	xo [m]	yo [m]	M [kNm/m]
Active earth pressure	Pax1.00	0.00- 2.50	20.41	4.50	2.150	1.888	28.84
Backfill surcharge (live)	Pqx1.00	0.00- 2.50	1.77	0.39	2.150	2.250	3.15
Active earth pressure	Pax1.00	2.50- 3.50	14.28	3.47	2.150	0.474	-0.69
Backfill surcharge (live)	Pqx1.00	2.50- 3.50	0.57	0.14	2.150	0.500	-0.02
Wall weight	W x1.00		2.72	66.61	1.629	1.247	-105.12
Backfill weight	Wsx1.00		0.49	12.09	2.275	1.968	-26.54
Vert. load on top (dead)	Ngx1.00		0.12	2.94	2.025	3.500	-5.54
Vert. load on top (live)	Nqx1.00		0.20	4.90	2.025	3.500	-9.23
			Sum=	95.04			-115.15

Sum of vertical forces = 95.04 kN/m
Sum of moments at front toe = -115.15 kNm/m
Sum of moments at middle of base = -1.10 kNm/m
Eccentricity ec=-1.10/95.04=-0.012m, ec<=2.400/6=0.400m
Soil pressure q1=0.038 N/mm² q2=0.041 N/mm²



Effective footing L=2.400-2x0.012= 2.377 m (EC7 Annex D)
Soil bearing capacity Rd=L·qu/γM=2.377x(1000x0.20)/1.00= 475.40 kN/m
Bearing resistance check Vd=95.04 < Rd=475.40 kN/m, Check is verified (EC7 Eq.2.2, Eq.6.1)

9.10.4. Failure check due to overturning (with seismic loading)

(EC7 §9.7.4)

Overturning with respect to the toe (xo=0,yo=0) (x=2.150,y=3.500 m)

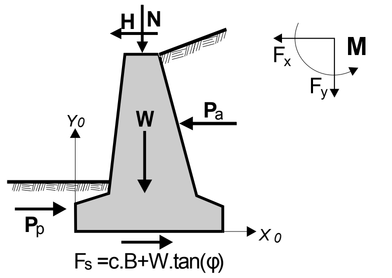
Action	(γ)	y1 - y2	Fx [kN/m]	Fy [kN/m]	xo [m]	yo [m]	Mo+ [kNm/m]	Mo- [kNm/m]
Active earth pressure	Pax1.00	0.00- 2.50	20.41	4.50	2.150	1.888	38.52	9.68
Backfill surcharge (live)	Pqx1.00	0.00- 2.50	1.77	0.39	2.150	2.250	3.99	0.84
Active earth pressure	Pax1.00	2.50- 3.50	14.28	3.47	2.150	0.474	6.77	7.46
Backfill surcharge (live)	Pqx1.00	2.50- 3.50	0.57	0.14	2.150	0.500	0.28	0.30
Wall weight	W x1.00		2.72	66.61	1.629	1.247	5.61	110.73*
Backfill weight	Wsx1.00		0.49	12.09	2.275	1.968	1.53	28.07*
Vert. load on top (dead)	Ngx1.00		0.12	2.94	2.025	3.500	0.54	6.08*
Vert. load on top (live)	Nqx1.00		0.20	4.90	2.025	3.500	0.90	10.13*
					Sum=		58.14	173.29

(*moments of negative seismic vertical loads, are added to the overturning moments)

Sum of overturning moments = 58.14 kNm/m
Sum of moments resisting overturning = 173.29 kNm/m
Overturning check Msd=58.14 < Mrd=173.29 kNm/m, Check is verified

9.10.5. Failure check against sliding (with seismic loading)

(EC7 §9.7.3, §6.5.3)



Action	(γ)	y1 - y2	Fx+ [kN/m]	Fx- [kN/m]	Fy [kN/m]
Active earth pressure	Pax1.00	0.00- 2.50	20.41	0.00	4.50
Backfill surcharge (live)	Pqx1.00	0.00- 2.50	1.77	0.00	0.39
Active earth pressure	Pax1.00	2.50- 3.50	14.28	0.00	3.47
Backfill surcharge (live)	Pqx1.00	2.50- 3.50	0.57	0.00	0.14
Passive earth pressure	Ppx1.00	2.90- 3.80	0.00	23.91	0.00
Wall weight	W x1.00		2.72	0.00	66.61
Backfill weight	Wsx1.00		0.49	0.00	12.09
Vert. load on top (dead)	Ngx1.00		0.12	0.00	2.94
Vert. load on top (live)	Nqx1.00		0.20	0.00	4.90
		Sum=	40.56	23.91	95.04

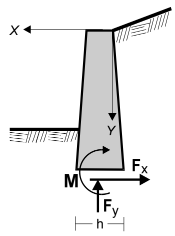
Soil friction Rd=Vd·tanφ/γM= 95.04xtan(30.00°)/1.00= 54.87 kN/m
Soil cohesion Rd=A·cu/γM = 1000x2.400x0.010/1.00= 24.00 kN/m
(resisting forces from effective cohesion are neglected) (EC7 §6.5.3. 10)
Sum of driving forces = 40.56 kN/m
Sum of resisting forces (23.91+54.87) = 78.78 kN/m
Sliding resistance check Hd=40.56 < Rd=78.78 kN/m, Check is verified

9.11. Design of wall steam

(EC2 EN1992-1-1:2004)

9.11.1. Loading 1.35x(permanent unfavourable)+1.00x(permanent favourable)+1.50x(variable unfav.)

Forces (at cross section centroid) at wall steam



y [m]	h [m]	Fx [kN/m]	Fy [kN/m]	M [kNm/m]
0.50	0.483	1.55	9.32	-1.06
1.00	0.517	4.44	16.48	-0.19
1.50	0.550	8.64	24.47	2.21
2.00	0.583	14.20	33.31	6.75
2.50	0.617	21.11	42.99	14.12
3.00	0.650	27.34	53.17	24.55

9.11.2. Design of wall steam in bending

(EC2 §9.6, §6.1)

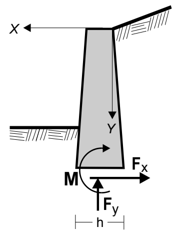
Concrete-Steel class: C25/30-S500, Concrete cover: Cnom=25 mm (§3, §4.4.1.1)
Vertical reinforcement minimum: 0.0020Ac, maximum: 0.0400Ac (EC2 §9.6.2)

y [m]	Msd [kN/m]	Nsd [kN]	d [mm]	Kd	x/d	ec/es	Ks	As [cm²/m]	min reinf. [cm²/m]
0.50	-1.06	-9.32	453	26.05	0.01	0.2/20.0	2.31	0.00	(4.83)
1.00	-0.19	-16.48	487	24.46	0.01	0.2/20.0	2.31	0.00	(5.17)
1.50	2.21	-24.47	520	18.16	0.01	0.3/20.0	2.31	0.00	(5.50)
2.00	6.75	-33.31	553	14.07	0.02	0.4/20.0	2.31	0.00	(5.83)
2.50	14.12	-42.99	587	11.49	0.02	0.5/20.0	2.32	0.04	(6.17)
3.00	24.55	-53.17	620	9.77	0.03	0.6/20.0	2.32	0.28	(6.50)

9.11.3. Loading 1.00x(permanent unfav.)+1.00x(permanent favour.)+1.00x(variable)+1.00x(seismic)

Forces (at cross section centroid) at wall steam (with seismic loading)

y [m]	h [m]	Fx [kN/m]	Fy [kN/m]	M [kNm/m]
0.50	0.483	2.21	9.32	-0.46
1.00	0.517	5.48	16.48	0.94
1.50	0.550	10.16	24.47	4.12
2.00	0.583	16.30	33.31	9.76
2.50	0.617	23.88	42.99	18.61
3.00	0.650	30.54	53.17	30.01



9.11.4. Design of wall steam in bending (with seismic loading)

(EC2 §9.6, §6.1)

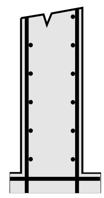
Concrete-Steel class: C25/30-S500, Concrete cover: Cnom=25 mm (§3, §4.4.1.1)
Vertical reinforcement minimum: 0.0020Ac, maximum: 0.0400Ac (EC2 §9.6.2)

y [m]	Msd [kN/m]	Nsd [kN]	d [mm]	Kd	x/d	ec/es	Ks	As [cm²/m]	min reinf. [cm²/m]
0.50	-0.46	-9.32	453	29.06	0.01	0.2/20.0	2.31	0.00	(4.83)
1.00	0.94	-16.48	487	22.44	0.01	0.2/20.0	2.31	0.00	(5.17)
1.50	4.12	-24.47	520	16.35	0.02	0.3/20.0	2.31	0.00	(5.50)
2.00	9.76	-33.31	553	12.87	0.02	0.4/20.0	2.32	0.01	(5.83)
2.50	18.61	-42.99	587	10.61	0.03	0.5/20.0	2.32	0.22	(6.17)
3.00	30.01	-53.17	620	9.17	0.03	0.6/20.0	2.32	0.49	(6.50)

9.11.5. Reinforcement of wall steam

Reinforcement at back steam face **Ø12/17.0** (6.65cm²/m)
Secondary transverse reinforcement **Ø8/50.0** (1.01cm²/m)

Reinforcement at front steam face **Ø12/17.0** (6.65cm²/m)
Secondary transverse reinforcement **Ø8/50.0** (1.01cm²/m)



9.11.6. Anchorage of wall steam reinforcement

(EC2 §8.4)

Basic required anchorage length lb,rqd=(Φ/4)(σsd/fbd)=(12/4)x(32/1.89)=51mm (EC2 Eq.8.3)
σsd=435.00x49/665=32MPa fbd=2.25x0.70x(fctk0.05/γc)=1.89 MPa (EC2 §8.4.2)
Design anchorage length lbd=1.00x51=51mm, Cnom=25mm<3Ø=36mm (EC2 §8.4.4, T.8.2)
Minimum anchorage length lb,min=max(0.30lbrqd,10Ø,100mm)=120mm
Necessary bend 120mm at lower bar end for anchorage

9.11.7. Shear check of wall steam

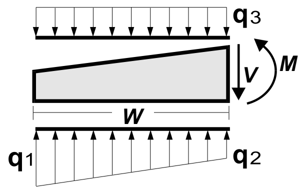
(EC2 EN1992-1-1:2004, §6.2.2)

Concrete-Steel class: C25/30-S500, Concrete cover: Cnom=25 mm (§3, §4.4.1.1)
The earth pressure load variation is linear, so the variation of shear force is parabolic. The variation of steam cross section is linear.
The most unfavourable place for shear check is the base of the steam.

```
Vsd=27.34 kN/m, Vsd (+seismic)=30.54 kN/m, Nsd=-53.17 kN/m
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=1.57, k1=0.15
ρ1=As1/(bw·d)=665/(1000x620)=0.0011
σcp=Nsd/Ac=1000x53.17/650000=0.08N/mm²
vmin=0.035·k^(1.50)·fck^½ = 0.34N/mm²                          (EC2 Eq.6.3N)
Vrd,c(min)=0.001x(0.34+0.15x0.08)x1000x620=218.24kN/m
Vrdc=0.001x[0.120x1.57x(0.11x25.00)^(0.333)+0.15x0.08]x1000x620=171.09, Vrdc=Vrcd(min)=218.24kN/m
Vsd=30.54 kN/m <= Vrdc=218.24 kN/m, shear OK
```

9.12. Design of wall footing and reinforcement (EC2 EN1992-1-1:2004)

9.12.1. Design of front toe x=2.150 m to x=0.650 m




```
Sum of vertical forces          =  131.52 kN/m
Sum of moments at middle of base =  -30.90 kNm/m
q1= 0.023 N/mm², q2= 0.063 N/mm², w= 1.500 m
pressure from self weight q3= 0.013 N/mm²
M= 25.90 kNm/m, V= 44.59 kN/m
V at distance d=425mm from the face of the steam = 25.75 kN/m
Msd= 25.90k Nm/m, Vsd= 25.75 kN/m
```

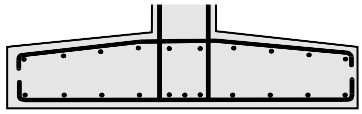
9.12.2. Design of front toe x=2.150 m to x=0.650 m (with seismic loading)

```
Sum of vertical forces          =   95.04 kN/m
Sum of moments at middle of base =   -1.10 kNm/m
q1= 0.038 N/mm², q2= 0.040 N/mm², w= 1.500 m
pressure from self weight q3= 0.013 N/mm²
M= 29.17 kNm/m, V= 39.26 kN/m
V at distance d=425mm from the face of the steam = 27.91 kN/m
Msd= 29.17k Nm/m, Vsd= 27.91 kN/m
```

9.12.3. Design of wall footing in bending (EC2 EN1992-1-1:2004, §6.1)

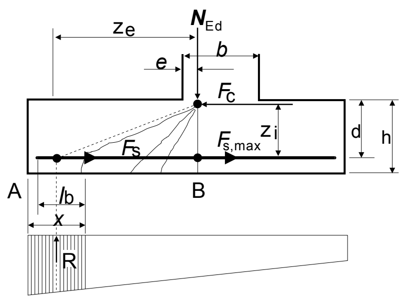
```
Concrete-Steel class: C25/30-S500, Concrete cover: Cnom=75 mm          (§3, §4.4.1.1)
Msd= 29.17kNm/m, d=419mm, Kd=7.76 x/d=0.04 εc/εs=0.8/20.0 ks=2.33,     As= 1.62cm²
Minimum reinforcement As>=0.26bd·Fctm/fyk (As= 5.66cm²/m)              (EC2 §9.3.1)
Minimum reinforcement Ø12/19.5( 5.79cm²/m)
```

9.12.4. Reinforcement of wall footing



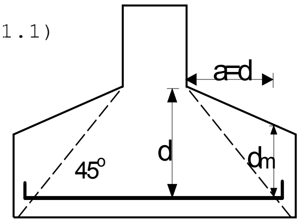
Footing reinforcement at bottom Ø12/19.5 (5.79cm²/m)
Secondary tranverse reinforcement Ø12/40.0 (2.82cm²/m)

9.12.5. Anchorage of footing reinforcement (EC2 EN1992-1-1:2004, §9.8.2.2, §8.4)




```
x=h/2=0.175m, R=1000x0.063x0.175=11.02 kN/m
e=0.15b=0.097m ze=1.510 m, zi=0.900d=0.377m
Fs=R·ze/zi=11.02x1.510/0.377=44.13 kN/m
σsd=Fs/As=1000x44.13/579=76 MPa
Basic required anchorage length       (EC2 Eq.8.3)
lb,rqd=(Φ/4)(σsd/fbd)=(12/4)x(76/2.70)=84mm
fbd=2.25x1.00x(fctk0.05/γc)=2.70 MPa       (EC2 §8.4.2)
Design anchorage length        (EC2 §8.4.4, T.8.2)
lbd=0.70x84=59mm, Cnom=75mm>3Ø=36mm
Minimum anchorage length lb,min=max(0.30lbrqd,10Ø,100mm)=120mm
Necessary anchorage length of longitudinal reinforcement Lbd=120mm =0.120m
lbd=120mm>(x-Cnom)=100.00. Necessary bends 60mm at bar ends for anchorage
```

9.12.6. Design of wall footing for shear and punching shear (EC2 EN1992-1-1:2004, §6.2.2)



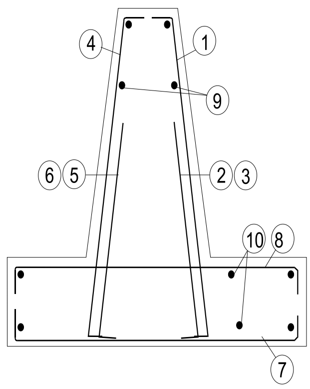
Concrete-Steel class: C25/30-S500, Concrete cover: Cnom=75 mm (§3, §4.4.1.1)
Punching shear capacity without shear reinforcement Vrdc (EC2 §6.4.4)
Vrdc=[Crdc·k·(100ρ1·fck)^(0.333)·(2d/a)]·bw·d (EC2 Eq.6.50)
Vrdc>=[vmin·2d/a]·bw·d, d=dm=377mm, a=419mm
Crdc=0.18/γc=0.18/1.50=0.120, fck=25.00MPa
k=1+(200/d)^½ <=2, k=1.73
ρ1=As1/(bw·d)=579/(1000x377)=0.0015
vmin=0.035·k^(1.50)·fck^½ = 0.40N/mm^2 (EC2 Eq.6.3N)
Vrd,c(min)=0.001x(0.40x2x377/419)x1000x377=271.51kN/m
Vrdc=0.001x[0.120x1.73x(0.15x25.00)^(0.333)x2x377/419]x1000x377=218.93, Vrdc=Vrcd(min)=271.51kN/m
Vsd=27.91 kN/m <= Vrdc=271.51 kN/m, shear and punching shear OK

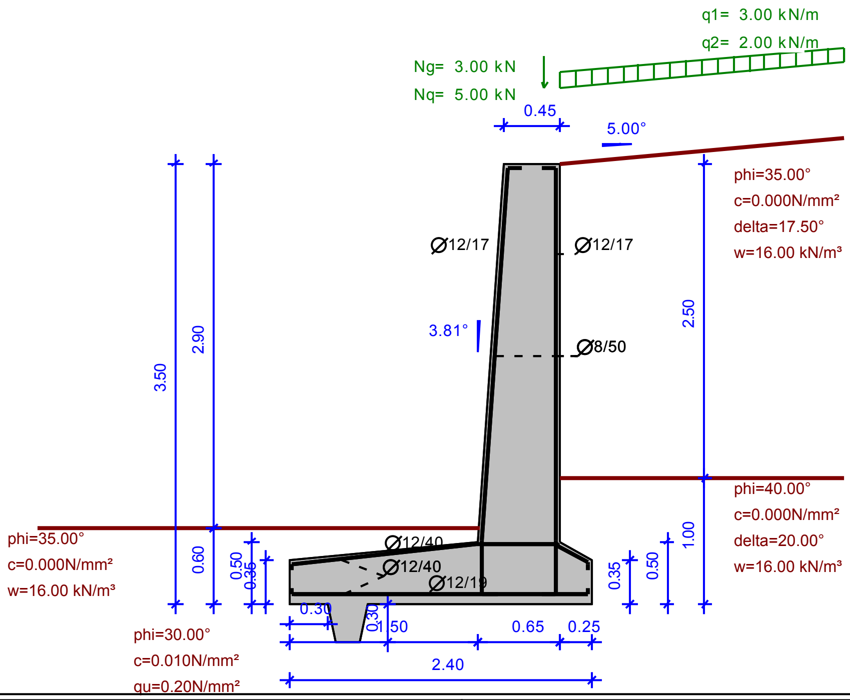
9.13. Material estimate

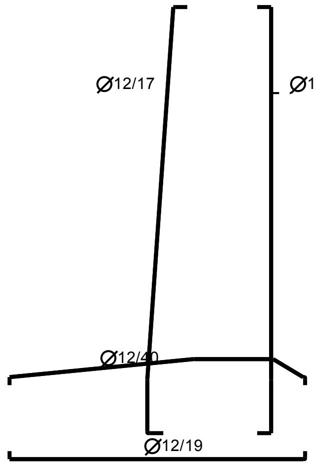
Concrete per meter of wall length		2.719 m^3/m	
Reinforcing steel per meter of wall		69.941 kg/m	
Total concrete of wall	10.000x	2.719=	27.187 m^3
Total reinforcing steel of wall	10.000x	69.941=	699.410 kg

9.14. Reinforcing bar schedule

Num	type	reinforcing bar [mm]	items	∅	g/m [kg/m]	length [m]	weight [kg]
43	1	120 3370 120	59	12	0.888	3.610	189.14
44	4	120 3370 120	59	12	0.888	3.610	189.14
45	9	10000	14	8	0.395	10.000	55.30
46	7	60 2240 60	51	12	0.888	2.360	106.88
47	8	60 2240 60	25	12	0.888	2.360	52.39
48	10	10000	12	12	0.888	10.000	106.56
		Total weight [kg]					**699.41**







General information

Wall type : Cantilever wall

Concrete and steel class

Steam :C25/30-S500

Footing :C25/30-S500

Design codes

Eurocode 0 EN1991-1-1, Basis of structural design

Eurocode 1 EN1991-1-1, Actions on structures

Eurocode 2 EN1992-1-1, Design of concrete structures

Eurocode 7 EN1997-1-1, Geotechnical design

Eurocode 8 EN1998-5, Earthquaqe design

Loads

Vertical : dead Ng=3.00kN, live Nq=5.00kN

Horizontal: dead Hg=0.00kN, live Hq=0.00kN

Surcharge : dead g=3.00kN/m², live q=2.00kN/m²

Seismic coefficients

Design ground acceleration ratio a =0.060

Coefficient for horizontal seismic force kh=0.040

Coefficient for vertical seismic force kv=0.020

Reinforcing bar schedule

#		reinforcing bar [mm]	items	Ø [mm]	g/m [kg/m]	length [m]	weight [kg]
1	W1	120 3370 120	59	12	0.888	3.610	189.14
2	W4	120 3370 120	59	12	0.888	3.610	189.14
3	W9	10000	14	8	0.395	10.000	55.30
4	W7	60 2240 60	51	12	0.888	2.360	106.88
5	W8	60 2240 60	25	12	0.888	2.360	52.39
6	W10	10000	12	12	0.888	10.000	106.56
		Total weight [kg]					**699.41**

Soil properties back-1

phi=35.00°

c=0.000N/mm²

delta=17.50°

w=16.00 kN/m³

Soil properties back-2

phi=40.00°

c=0.000N/mm²

delta=20.00°

w=16.00 kN/m³

Soil properties front

phi=35.00°

c=0.000N/mm²

w=16.00 kN/m³

Foundation soil properties

phi=30.00°

c=0.010N/mm²

qu=0.20N/mm²

Concrete volume V= 27.19 [m³]

Reinforcement weight G=699.41 [kg]

Project: Design examples 12/03/2007

C. WALL-001

Scale : 1:60 **Date:** 13/03/2007

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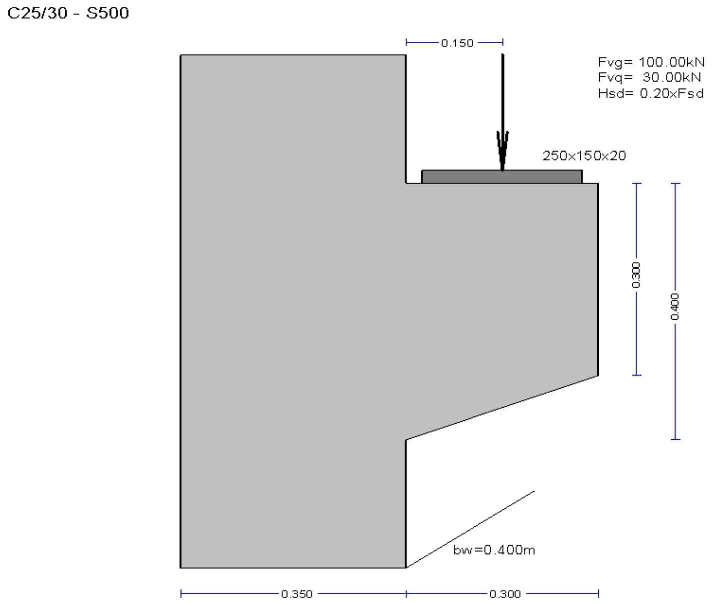
10. CORBEL-001

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

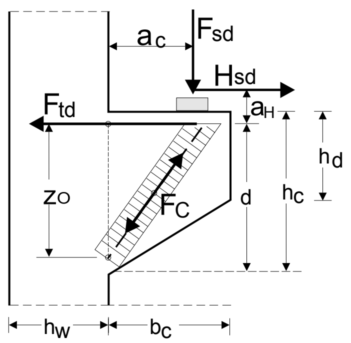


Concrete-Steel class: C25/30-S500 (EC2 §3)
Concrete cover : Cnom=20 mm (EC2 §4.4.1)
γc=1.50, γs=1.15 (EC2 Table 2.1N)
Partial safety factors for actions : γG=1.35, γQ=1.50 (EC0 Annex A1)

10.1. Dimensions and loads

Dimensions hw=0.350m, bw=0.400m, bc=0.300m
hc=0.400m, hd=0.300m, ac=0.150m
Bearing plate bxhxt=250x150x20 mm
Dead load Fvg=100.00kN, Live load Fvq=30.00kN
Horizontal force Hsd=0.200xFsd

10.2. Design model (EC2 §5.6.4,§6.5,J.3)

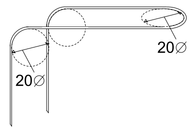


ac<0.40hc (0.150<0.40x0.400=0.160 m)
conditions of short corbel
dimensioning with hc=2.50xac=2.50x0.150=0.375 m
Design using a strut-and-tie model with strut the compressive stress field and tie the reinforcement
d =h-Cnomc-2Ø=375-20-2x14=327mm, d= 0.327m
aH=Cnomc+2Ø+t= 20+2x14+20= 68mm, aH= 0.068m
d/ac=0.327/0.150, θ=65.4°, tanθ=2.18
Fsd=1.35x100.00+1.50x 30.00 = 180.00 kN
Hsd=0.20x 180.00 = 36.00 kN

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

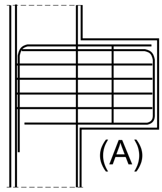
Vrdmax=αcw·bw·z·ν1·fcd/(cotθ+tanθ),
Vsd/Vrdmax=0.44, θ=21.8° cotθ=2.50 tanθ=0.40
αcw=1.00 z=0.9d, fck=25.0<=60Mpa ν1=0.60, fcd=16.67Mpa
Vrdmax=0.001x1.00x400x0.9x327x0.60x16.67/2.90=406.0 kN
Vsd=180.0 kN < 406.0 kN =Vrdmax, the check is verified

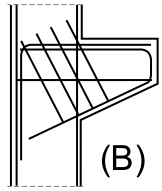
10.4. Force in tie



zo=d(1-0.4Vsd/Vrdmax)=0.327x(1-0.4x180.0/406.0)=0.269m
Force in tie Ftd=Fsd·ac/zo+Hsd·(aH+zo)/zo
Ftd=180.0x0.150/0.269+36.0x(0.068+0.269)/0.269=145.47kN
As,req=Ftd/fyd=10x145.47/435= **3.34cm²**
Main tension reinforcement (looped) 1Ø16 (4.02cm²)

10.5. Links (EC2 Annex J.3)





ac/hc=0.150/0.375=0.400<0.50
Horizontal closed links Figure A
or inclined links, Figure B
Total area Asw=0.25x 3.34cm²= **0.83cm²**
Use closed links 2Ø8 (2.01cm²)

10.6. Check pressure under bearing plate (EC2 §6.5.4.b)

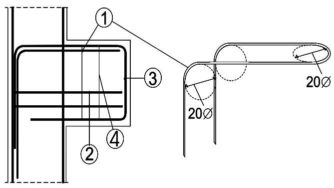
Mean concrete compressive stress σc=Fsd/Ac <= σrdmax =0.60ν.fcd ν=1-fck/250 (Eq.6.56, 6.57N)
σc=1000x180.0/(250x150)=4.80<0.54x16.67=9.00N/mm² the check is verified

10.7. Reinforcement anchorage (EC2 §8.4)

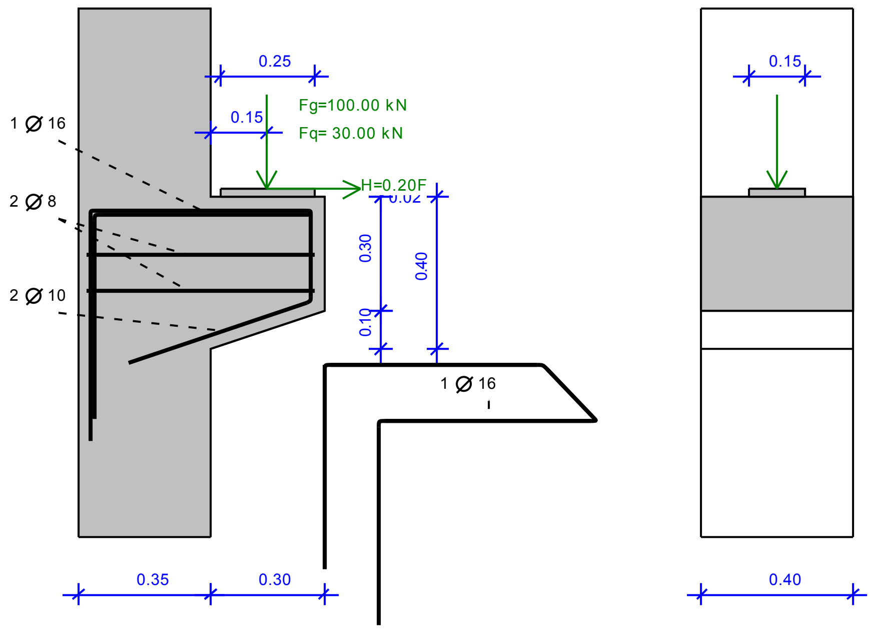
Minimum mandrel diameter of main reinforcement 4x16=64 mm (EC2 Table 8.1N)
Required corbel width breq=1.50x64+2x16+2x20=168mm

Basic required anchorage length lb,rqd=(Φ/4)(σsd/fbd)=(16/4)x(362/2.70)=536mm (EC2 Eq.8.3)
σsd=1000x145.47/402=362MPa fbd=2.25x1.00x(fctk0.05/γc)=2.70 MPa (EC2 §8.4.2)
Design anchorage length lbd=1.00x536=536mm, Cnom=20mm<3Ø=48mm (EC2 §8.4.4, T.8.2)
Minimum anchorage length lb,min=max(0.30lbrqd,10Ø,100mm)=161mm
Necessary anchorage length of main reinforcement Lbd=540mm =0.540m

10.8. Reinforcing bar schedule



Num	type	reinforcing bar [mm]	items	Ø	g/m [kg/m]	length [m]	weight [kg]
49	1	540 580 60 / 540 580	1	16	1.580	2.300	3.63
50	2	80 600 350 / 80 350 600	2	8	0.395	2.060	1.63
51	3	610 580 240 / 510	2	10	0.617	1.940	2.39
		Total weight [kg]					**7.65**



General information

Corbel/Bracket

Concrete and steel class: C25/30 - S500

Concrete cover: Cnom=20 mm

Design codes

Eurocode 0 EN1991-1-1, Basis of structural design

Eurocode 1 EN1991-1-1, Actions on structures

Eurocode 2 EN1992-1-1, Design of concrete structures

Loads

dead Fvg=100.00kN/m

live Fvg= 30.00kN/m

Horizontal force Hsd=0.200xFsd

Reinforcing bar schedule

#		reinforcing bar [mm]	items	Ø [mm]	g/m [kg/m]	length [m]	weight [kg]
1	Q1	580 540 60 540 580	1	16	1.580	2.300	3.63
2	Q2	80 600 80 350 350 600	2	8	0.395	2.060	1.63
3	Q3	580 610 240 510	2	10	0.617	1.940	2.39

Total weight [kg] **7.65**

Concrete volume V= 0.04 [m³]

Reinforcement weight G=7.65 [kg]

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CORBEL-001

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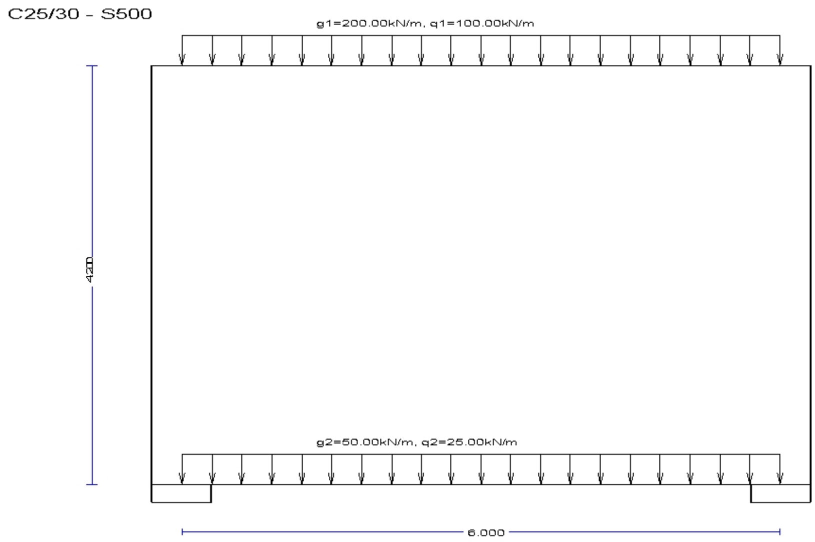
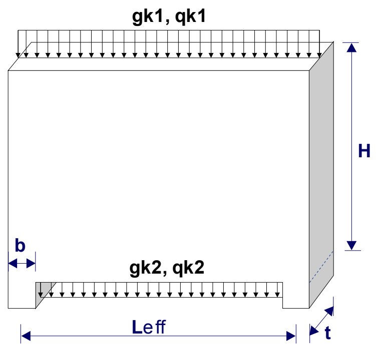
11. D. BEAM-001

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



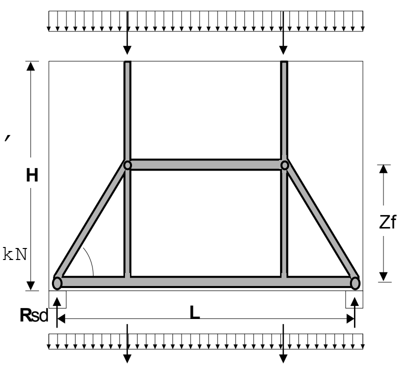
Concrete-Steel class: C25/30-S500 (EC2 §3)
Concrete cover : Cnom=20 mm (EC2 §4.4.1)
Concrete weight : 25.0 kN/m^3
γ_c=1.50, γ_s=1.15 (EC2 Table 2.1N)
Partial safety factors for actions : γ_G=1.35, γ_Q=1.50 (EC0 Annex A1)

11.1. Dimensions and loads



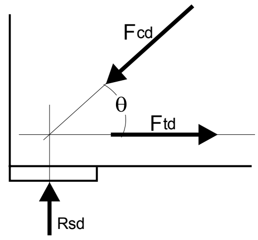
Beam span Leff=6.000m, beam height H=4.200m
Web thickness t=0.300m, bearing width b=0.600m
Load on the top: dead gk1=200.00kN/m, live qk1=100.00kN/m
Load at bottom : dead gk2= 50.00kN/m, live qk2= 25.00kN/m
Beam self weight gk3=0.30x4.200x25.00=31.50 kN/m
Design loads
Load on the top dead gd1=1.35x200.00 =270.00kN/m
live qd1=1.50x100.00 =150.00kN/m
Load at bottom dead gd2=1.35x(50.00+31.50)=110.03kN/m
live qd2=1.50x 25.00 = 37.50kN/m

11.2. Design model (EC2 §5.6.4,§6.5)



The design method is based on elasto-plastic material behaviour.
Design using a strut-and-tie model, with strut the compressive stress field of the concrete and tie the steel reinforcement.
References, Schlaich,J Schafer,K, Konstruieren im Stahlbetonnbau, Betonkalender 82,1993 Teil 2,313-458, Berlin, Ernst-Son,1993
The lever arm Zf of the internal forces is:
0.5<=H/L=4.200/6.000=0.70<=1.0, Zf=0.30x4.200(3-0.700)=2.898 m
Reactions Rsd,A=Rsd,B=(270.00+150.00+110.03+37.50)x6.00/2=1702.6kN
Angle of inclined strut tanθ=Zf/(L/4)=2.90/1.50=1.93, θ=62.61°
Strut compression Fcd=Rsd,A/sinθ=1702.6/0.89=1913.0kN
Tie tension Ftd=Rsd,A/tanθ=1702.6/1.93=882.2kN

11.3. Check of concrete compressive stresses (EC2 §6.5.4.b)



$\sigma c <= \sigma rdmax = 0.85\nu \cdot fcd = 0.765x16.67 = 12.75 N/mm^2$ ($\nu = 1 - fck/250$)
Compression at support $\sigma rd1 = 1000x1702.6/(600x300) = 9.46 N/mm^2$
$\sigma rd1 = 9.46 < \sigma rdmax = 12.75\ N/mm^2$ the check is verified
Concrete compressive stress at the diagonal strut
strut width $b1 = (600+0.10x4200xcot62.6°)xsin62.6° = 726mm$
Strut compression $\sigma rd2 = 1000x1913.0/(726x300) = 8.78 N/mm^2$
$\sigma rd2 = 8.78 < \sigma rdmax = 12.75\ N/mm^2$ the check is verified

11.4. Tension in tie

$As,req = Ftd/fyd = 1000x882.2/435.0 = 2028\ mm^2 = 20.28\ cm^2$
Reinforcement at lower beam face 4Ø20+2Ø24 ($21.60 cm^2$)
Reinforcement at upper beam face 4Ø18 ($10.16 cm^2$)

11.5. Surface reinforcement

Horizontal transverse reinforcement
Transverse reinforcement must be placed at the lower beam part until a height Zf=2.90m,
to take the deviation forces Ftd,2=0.25Fcd=0.25x1913.0=478.2 kN
At each face $As,req = 0.5Ftd,2/fyd = 0.50x1000x478.2/435.0 = 550mm^2 = 550/2.90 = 190mm^2/m = 1.90cm^2/m$

Suspension reinforcement
The applied loads at the lower face of the beam are suspended by vertical
reinforcement at each beam face $As,req = 0.5x1000x(67.50+37.50)/435.00 = 121mm^2/m = 1.21cm^2/m$

Minimum required reinforcement (EC2 §9.7)
Orthogonal reinforcement mesh near each beam face, with minimum As=0.001xAc, $As > 150\ mm^2/m$
$As,min = 0.0010x300x1000.00 = 300mm^2/m$, $As,min = 300\ mm^2/m = 3.00\ cm^2/m$

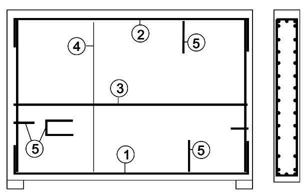
Orthogonal reinforcement mesh near each face Ø12/30.0 ($3.77cm^2$)
Along the free faces are placed open links U Ø12/30.0 ($3.77cm^2$)

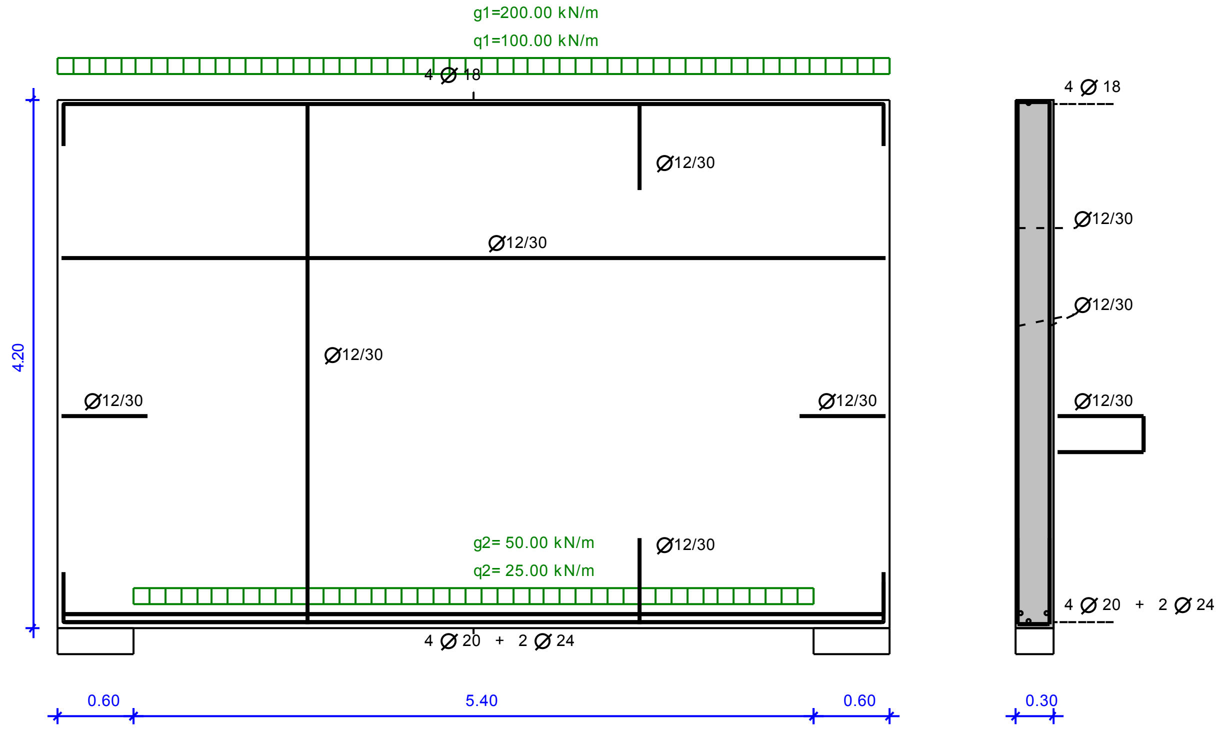
11.6. Reinforcement anchorage (EC2 §8.4)

Basic required anchorage length $lb,rqd = (\Phi/4)(\sigma sd/fbd) = (24/4)x(408/2.70) = 908mm$ (EC2 Eq.8.3)
$\sigma sd = 1000x882.20/2160 = 408MPa$ $fbd = 2.25x1.00x(fctk0.05/\gamma c) = 2.70\ MPa$ (EC2 §8.4.2)
Design anchorage length lbd=1.00x908=908mm, Cnom=20mm<3Ø=72mm (EC2 §8.4.4, T.8.2)
Minimum anchorage length lb,min=max(0.30lbrqd,10Ø,100mm)=272mm
Necessary anchorage length of longitudinal reinforcement Lbd=910mm =0.910m
lbd=910mm>(0.600-Cnom)=580.00. Necessary bends 330mm at bar ends for anchorage

11.7. Reinforcing bar schedule

Num	type	reinforcing bar [mm]	items	Ø	g/m [kg/m]	length [m]	weight [kg]
52	1	330 6560 330	4	20	2.470	7.220	71.33
53	1	330 6560 330	2	24	3.550	7.220	51.26
54	2	330 6560 330	4	18	2.000	7.220	57.76
55	3	6560	28	12	0.888	6.560	163.11
56	4	4160	40	12	0.888	4.160	147.76
57	5	690 690 280	68	12	0.888	1.660	100.24
		Total weight [kg]					**591.46**





General information

Deep beam

Concrete and steel class: C25/30 - S500

Concrete cover: Cnom=20 mm

Design codes

Eurocode 0 EN1991-1-1, Basis of structural design

Eurocode 1 EN1991-1-1, Actions on structures

Eurocode 2 EN1992-1-1, Design of concrete structures

Loads

Load on the top

dead g1=200.00kN/m

live q1=100.00kN/m

Load at bottom

dead g2= 50.00kN/m

live q2= 25.00kN/m

Reinforcing bar schedule

#		reinforcing bar [mm]	items	Ø [mm]	g/m [kg/m]	length [m]	weight [kg]
1	D1	330 6560 330	4	20	2.470	7.220	71.33
2	D1	330 6560 330	2	24	3.550	7.220	51.26
3	D2	330 6560 330	4	18	2.000	7.220	57.76
4	D3	6560	28	12	0.888	6.560	163.11
5	D4	4160	40	12	0.888	4.160	147.76
6	D5	690 690 280	68	12	0.888	1.660	100.24
		Total weight [kg]					**591.46**

Concrete volume V= 7.56 [m³]

Reinforcement weight G=591.46 [kg]

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D. BEAM-001

Scale : 1:60 **Date:** 13/03/2007

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