RC MEMBER DESIGN

In accordance with EN1992-1-1:2004 incorporating Corrigenda January 2008 and the UK national annex

Tedds calculation version 3.2.00

Concrete details - Table 3.1. Strength and deformation characteristics for concrete

Concrete strength class	C30/37
Aggregate type	Quartzite
Aggregate adjustment factor - cl.3.1.3(2)	AAF = **1.0**
Characteristic compressive cylinder strength	f_{ck} = **30** N/mm^2
Mean value of compressive cylinder strength	$f_{cm} = f_{ck} + 8$ N/mm^2 = **38** N/mm^2
Mean value of axial tensile strength	$f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck}/ 1$ N/mm$^2)^{2/3}$ = **2.9** N/mm^2
Secant modulus of elasticity of concrete	$E_{cm} = 22$ kN/mm$^2\times[f_{cm}/10$ N/mm$^2]^{0.3} \times$ AAF = **32837** N/mm^2
Ultimate strain - Table 3.1	ε_{cu2} = **0.0035**
Shortening strain - Table 3.1	ε_{cu3} = **0.0035**
Effective compression zone height factor	λ = **0.80**
Effective strength factor	η = **1.00**
Coefficient k_1	k_1 = **0.40**
Coefficient k_2	$k_2 = 1.0 \times (0.6 + 0.0014 / \varepsilon_{cu2})$ = **1.00**
Coefficient k_3	k_3 = **0.40**
Coefficient k_4	$k_4 = 1.0 \times (0.6 + 0.0014 / \varepsilon_{cu2})$ = **1.00**
Partial factor for concrete -Table 2.1N	γ_C = **1.50**
Compressive strength coefficient - cl.3.1.6(1)	α_{cc} = **0.85**
Design compressive concrete strength - exp.3.15	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **17.0** N/mm^2
Compressive strength coefficient - cl.3.1.6(1)	α_{ccw} = **1.00**
Design compressive concrete strength - exp.3.15	$f_{cwd} = \alpha_{ccw} \times f_{ck} / \gamma_C$ = **20.0** N/mm^2
Maximum aggregate size	h_{agg} = **20** mm
Monolithic simple support moment factor	β_1 = **0.25**

Reinforcement details

Characteristic yield strength of reinforcement	f_{yk} = **500** N/mm^2
Partial factor for reinforcing steel - Table 2.1N	γ_S = **1.15**
Design yield strength of reinforcement	$f_{yd} = f_{yk} / \gamma_S$ = **435** N/mm^2

Nominal cover to reinforcement

Nominal cover to top reinforcement	c_{nom_t} = **30** mm
Nominal cover to bottom reinforcement	c_{nom_b} = **30** mm
Nominal cover to side reinforcement	c_{nom_s} = **30** mm

Fire resistance

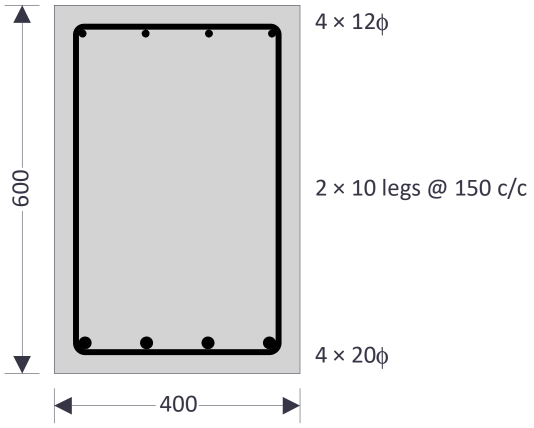
Standard fire resistance period	R = **120** min
Number of sides exposed to fire	3
Minimum width of beam - EN1992-1-2 Table 5.5	b_{min} = **200** mm

Section 1 - Multiple layers

Rectangular section details

Section width	b = **400** mm
Section depth	h = **600** mm

PASS - Minimum dimensions for fire resistance met



Positive moment - section 6.1

Design bending moment — $M = M_{pos\_s1} = \mathbf{200.0}$ kNm

Effective depth of tension reinforcement — $d = \mathbf{550}$ mm

Redistribution ratio — $\delta = \min(\delta_{pos\_s1}, 1) = \mathbf{1.000}$

$K = M / (b \times d^2 \times f_{ck}) = \mathbf{0.055}$

$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2))$

$= \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d) = \mathbf{522}$ mm

Depth of neutral axis — $x = 2 \times (d - z) / \lambda = \mathbf{70}$ mm

Area of tension reinforcement required — $A_{s,req} = M / (f_{yd} \times z) = \mathbf{882}$ mm^2

Tension reinforcement provided — $4 \times 20\phi$

Area of tension reinforcement provided — $A_{s,prov} = \mathbf{1257}$ mm^2

Minimum area of reinforcement - exp.9.1N — $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d = \mathbf{331}$ mm^2

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{s,max} = 0.04 \times b \times h = \mathbf{9600}$ mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3

Maximum crack width — $w_k = \mathbf{0.3}$ mm

Design value modulus of elasticity reinf – 3.2.7(4) — $E_s = \mathbf{200000}$ N/mm^2

Mean value of concrete tensile strength — $f_{ct,eff} = f_{ctm} = \mathbf{2.9}$ N/mm^2

Stress distribution coefficient — $k_c = \mathbf{0.4}$

Non-uniform self-equilibrating stress coefficient — $k = \min(\max(1 + (300\text{ mm} - \min(h, b)) \times 0.35 / 500\text{ mm}, 0.65), 1) =$ $\mathbf{0.93}$

Actual tension bar spacing — $s_{bar} = (b - (2 \times (c_{nom\_s} + \phi_{s1\_v}) + \phi_{s1\_b\_L1} \times N_{s1\_b\_L1})) / (N_{s1\_b\_L1} - 1) +$ $\phi_{s1\_b\_L1} = \mathbf{100}$ mm

Maximum stress permitted - Table 7.3N — $\sigma_s = \mathbf{320}$ N/mm^2

Steel to concrete modulus of elast. ratio — $\alpha_{cr} = E_s / E_{cm} = \mathbf{6.09}$

Distance of the Elastic NA from bottom of beam — $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1)) =$ $\mathbf{294}$ mm

Area of concrete in the tensile zone — $A_{ct} = b \times y = \mathbf{117404}$ mm^2

Minimum area of reinforcement required - exp.7.1 — $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s = \mathbf{395}$ mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent moment — $M_{QP} = M_{pos_QP_s1} = \mathbf{140.0}$kNm

Permanent load ratio $R_{PL} = M_{QP} / M = \mathbf{0.70}$

Service stress in reinforcement $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL} = \mathbf{214}$ N/mm^2

Maximum bar spacing - Tables 7.3N $s_{bar,max} = \mathbf{233.1}$ mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Minimum bar spacing (Section 8.2)

Top bar spacing $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{s1\_v}) + \phi_{s1\_t\_L1} \times N_{s1\_t\_L1})) / (N_{s1\_t\_L1} - 1) = \mathbf{90.7}$ mm

Minimum allowable top bar spacing $s_{top,min} = \max(\phi_{s1\_t\_L1} \times k_{s1}, h_{agg} + k_{s2}, 20mm) = \mathbf{25.0}$ mm

PASS - Actual bar spacing exceeds minimum allowable

Bottom bar spacing $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{s1\_v}) + \phi_{s1\_b\_L1} \times N_{s1\_b\_L1})) / (N_{s1\_b\_L1} - 1) = \mathbf{80.0}$ mm

Minimum allowable bottom bar spacing $s_{bot,min} = \max(\phi_{s1\_b\_L1} \times k_{s1}, h_{agg} + k_{s2}, 20mm) = \mathbf{25.0}$ mm

PASS - Actual bar spacing exceeds minimum allowable

Section in shear (section 6.2)

Angle of comp. shear strut for maximum shear $\theta_{max} = 45$ deg

Strength reduction factor - cl.6.2.3(3) $v_1 = 0.6 \times (1 - f_{ck} / 250 \text{ N/mm}^2) = \mathbf{0.528}$

Compression chord coefficient - cl.6.2.3(3) $\alpha_{cw} = \mathbf{1.00}$

Minimum area of shear reinforcement - exp.9.5N $A_{sv,min} = 0.08 \text{ N/mm}^2 \times b \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / f_{yk} = \mathbf{351}$ mm^2/m

Design shear force at support $V_{Ed,max} = V_{Ed,max\_s1} = \mathbf{300}$ kN

Min lever arm in shear zone $z = \mathbf{522}$ mm

Maximum design shear resistance - exp.6.9 $V_{Rd,max} = \alpha_{cw} \times b \times z \times v_1 \times f_{cwd} / (\cot(\theta_{max}) + \tan(\theta_{max})) = \mathbf{1102}$ kN

PASS - Design shear force at support is less than maximum design shear resistance

Design shear force $V_{Ed} = \mathbf{300}$ kN

Design shear stress $v_{Ed} = V_{Ed} / (b \times z) = \mathbf{1.437}$ N/mm^2

Angle of concrete compression strut - cl.6.2.3 $\theta = \min(\max(0.5 \times \text{Asin}(\min(2 \times v_{Ed} / (\alpha_{cw} \times f_{cwd} \times v_1),1)), 21.8 \text{ deg}), 45\text{deg}) = \mathbf{21.8}$ deg

Area of shear reinforcement required - exp.6.8 $A_{sv,des} = v_{Ed} \times b / (f_{yd} \times \cot(\theta)) = \mathbf{529}$ mm^2/m

Area of shear reinforcement required $A_{sv,req} = \max(A_{sv,min}, A_{sv,des}) = \mathbf{529}$ mm^2/m

Shear reinforcement provided 2×10 legs @ 150 c/c

Area of shear reinforcement provided $A_{sv,prov} = \mathbf{1047}$ mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N $s_{vl,max} = 0.75 \times d = \mathbf{412}$ mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum