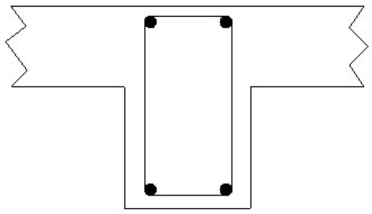
EC2 SIMPLE SUPPORTED RECTANGULAR BEAM DESIGN

Beam design

Design main reinforcement



Beam

Adjacent slab deep: $t := 160\ mm$

Beam dimension: $b := 230\ mm$

$h := 350\ mm$

$l := 6300\ mm$

Concrete class: C20/25 $f_{ck} := 20\ \frac{N}{mm^2}$ $\gamma_c := 1.5$

Steel class: B500 $f_{yk} := 500\ \frac{N}{mm^2}$ $\gamma_s := 1.15$

Length of supports: $a_1 := 300\ mm$ $a_2 := 300\ mm$

Effective length: $l_{eff} := l + min\left(\frac{a_1 + a_2}{2}, h\right) = \left(6.6 \cdot 10^3\right)\ mm$

Load and effects

Selfweight of beam: $SW_b := h \cdot b \cdot 25\ \frac{kN}{m^3} = 2.013\ \frac{kN}{m}$

Selfweight of covering: $SW_c := 1.5\ \frac{kN}{m}$

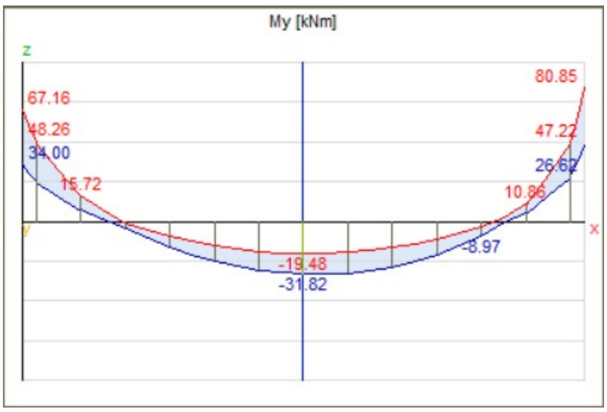
Permanent load: $g_k := 3.513\ \frac{kN}{m}$ $\gamma_g := 1.35$ $g_d := \gamma_g \cdot g_k = 4.743\ \frac{kN}{m}$

Live load: $q_k := 2\ \frac{kN}{m}$ $\gamma_q := 1.5$ $q_{d\triangle} := \gamma_q \cdot q_k = 3\ \frac{kN}{m}$

$\psi_2 := 0.3$

Load combination for ULS analysis: $p_{ed} := g_d + q_d = 7.743\ \frac{kN}{m}$

Load combination for SLS analysis: $p_{qp} := g_k + \psi_2 \cdot q_k = 4.113\ \frac{kN}{m}$



Design reinforcement at mid-span

Bending moment: $M_{ed} := 31.82\ kN \cdot m$

Concrete cover: $Cc := 25\ mm$ $\qquad \eta := 10\ mm$

Assume effective depth: $d := min(0.9 \cdot h, h - 50\ mm) = 300\ mm$

$d' := 50\ mm$

Design strength: $f_{cd} := \dfrac{f_{ck}}{\gamma_c} = 13.333\ \dfrac{N}{mm^2}$ $\qquad f_{yd} := \dfrac{f_{yk}}{\gamma_s} = 434.783\ \dfrac{N}{mm^2}$

Compatibility for plastic analysis

Compressed part: $\xi'_{co} := \dfrac{560}{700 - f_{yd}} = 2.111$

Tensield part: $\xi_{co} := \dfrac{560}{700 + f_{yd}} = 0.493$ $\qquad x_{co} := \xi_{co} \cdot d = 148.046\ mm$ $\qquad \alpha := 1$

Optimal bending moment: $M_u := x_{co} \cdot b \cdot \alpha \cdot f_{cd} \cdot \left(d - \dfrac{x_{co}}{2}\right) = 102.595\ kN \cdot m$

Because of Mu greater than Med, we don't need compression steel.

$M_{ed} = x_c \cdot b \cdot \alpha \cdot f_{cd} \cdot \left(d - \dfrac{x_c}{2}\right)$ $\qquad x_c := d - \sqrt{d^2 - \dfrac{2 \cdot M_{ed}}{b \cdot f_{cd}}} = 36.85\ mm$

The requirment steel area

Mainbars: $\phi := 12\ mm$ $\qquad$ Stirrups: $\phi_{sw} := 6\ mm$

$A_{sreq} := \dfrac{x_c \cdot b \cdot \alpha \cdot f_{cd}}{f_{yd}} = 259.916\ mm^2$ $\qquad A_{rebars} := \dfrac{\pi \cdot \phi^2}{4} = 113.097\ mm^2$ $\qquad \dfrac{A_{sreq}}{A_{rebars}} = 2.298$

Provide number of rebars: $n := 3$

Provide area of reinforcement: $A_s := A_{rebars} \cdot n = 339.292\ mm^2$ (At bottom)

Provide depth: $d_{real} := h - Cc - \phi_{sw} - \dfrac{\phi}{2} - \eta = 303\ mm$

The real neutral axis: $x_{creal} := \dfrac{A_s \cdot f_{yd}}{b \cdot \alpha \cdot f_{cd}} = 48.104\ mm$

Plasticity check: $\frac{x_{creal}}{d_{real}} = 0.159$ Smaller than $\xi_{co} := 0.493$

Standard regulation:

$\rho_{sl} := \frac{A_s}{b \cdot d_{real}} = 0.005$ $f_{ctm} := 0.3 \cdot \sqrt[3]{f_{ck}^{2}} = 2.21$

$\rho_{s1min} := \max\left(0.13\%, 0.26 \cdot \frac{f_{ctm}}{f_{yk}}\right) = 0.001$

$\rho_{slmax} := 4\%$ $\rho_{slmin} < \rho_{s1} < \rho_{slmax}$

Space between bars

$\zeta := \max(\phi, 20\ mm) = 20\ mm$

$b_{min} := 2 \cdot (Cc + \phi_{sw}) + n \cdot \phi + (n-1) \cdot \zeta = 138\ mm$ $b > b_{min}$

$M_{rd} := x_{creal} \cdot b \cdot \alpha \cdot f_{cd} \cdot \left(d - \frac{x_{creal}}{2}\right) = 40.707\ kN \cdot m$ $M_{rd} > M_{ed}$

Performance: $\frac{M_{ed}}{M_{rd}} = 0.782$

In the moment disgram , we found that at the support where has a huge moment. In this case, we will also design the reinforcement at support. Because the compressed and tensiled zone will opposite to the zone location at midspan, the tensiled reinforcement will be set in the top.

Concrete class: C20/25 $f_{ck} := 20\ \frac{N}{mm^2}$ $\gamma_c := 1.5$

Steel class: B500 $f_{yk} := 500\ \frac{N}{mm^2}$ $\gamma_s := 1.15$

Design reinforcement at support

Bending moment: $M_{ed} := 80.80\ kN \cdot m$

Concrete cover: $Cc := 25\ mm$ $\eta := 10\ mm$

Assume effective depth: $d := min(0.9 \cdot h, h - 50\ mm) = 300\ mm$

$d' := 50\ mm$

Design strength: $f_{cd} := \frac{f_{ck}}{\gamma_c} = 13.333\ \frac{N}{mm^2}$ $f_{yd} := \frac{f_{yk}}{\gamma_s} = 434.783\ \frac{N}{mm^2}$

Compatibility for plastic analysis

Compressed part: $\xi'_{co} := \frac{560}{700 - f_{yd}} = 2.111$

Tensield part: $\xi_{co} := \frac{560}{700 + f_{yd}} = 0.493$ $\quad x_{co} := \xi_{co} \cdot d = 148.046\ mm$ $\quad \alpha := 1$

Optimal bending moment: $M_u := x_{co} \cdot b \cdot \alpha \cdot f_{cd} \cdot \left(d - \frac{x_{co}}{2}\right) = 102.595\ kN \cdot m$

Because of Mu greater than Med, we don't need compression steel.

$M_{ed} = x_c \cdot b \cdot \alpha \cdot f_{cd} \cdot \left(d - \frac{x_c}{2}\right)$ $\quad x_c := d - \sqrt{d^2 - \frac{2 \cdot M_{ed}}{b \cdot f_{cd}}} = 106.857\ mm$

The requirment steel area

Mainbars: $\phi := 12\ mm$ Stirrups: $\phi_{sw} := 6\ mm$

$A_{sreq} := \frac{x_c \cdot b \cdot \alpha \cdot f_{cd}}{f_{yd}} = 753.695\ mm^2$ $\quad A_{rebars} := \frac{\pi \cdot \phi^2}{4} = 113.097\ mm^2$ $\quad \frac{A_{sreq}}{A_{rebars}} = 6.664$

Provide number of rebars: $n := 7$

Provide area of reinforcement: $A_s := A_{rebars} \cdot n = 791.681\ mm^2$ (At the top)

Provide depth: $d_{real} := h - Cc - \phi_{sw} - \frac{\phi}{2} - \eta = 303\ mm$

The real neutral axis: $x_{creal} := \frac{A_s \cdot f_{yd}}{b \cdot \alpha \cdot f_{cd}} = 112.242\ mm$

Plasticity check: $\frac{x_{creal}}{d_{real}} = 0.37$ Smaller than $\xi_{co} := 0.493$

Standard regulation:

$\rho_{sl} := \frac{A_s}{b \cdot d_{real}} = 0.011$ $\quad f_{ctm} := 0.3 \cdot \sqrt[3]{f_{ck}^{\ 2}} = 2.21$

$\rho_{s1min} := \max\left(0.13\%, 0.26 \cdot \frac{f_{ctm}}{f_{yk}}\right) = 0.001$

$\rho_{slmax} := 4\%$ $\quad \rho_{slmin} < \rho_{s1} < \rho_{slmax}$

Space between bars

$\zeta := \max(\phi, 20\ mm) = 20\ mm$

$b_{min} := 2 \cdot (Cc + \phi_{sw}) + n \cdot \phi + (n-1) \cdot \zeta = 266\ mm$ $\qquad b < b_{min}$

We should arrange the steel rebars in 2 rows. Number at first row is 4,at second row is 3

$M_{rd} := x_{creal} \cdot b \cdot \alpha \cdot f_{cd} \cdot \left(d - \dfrac{x_{creal}}{2}\right) = 83.945\ kN \cdot m$ $\qquad M_{rd} > M_{ed}$

Performance: $\dfrac{M_{ed}}{M_{rd}} = 0.963$

Shear design

$V_{ed} := 50.09\ kN$

Then we will figure out the VRdc $\qquad d := 303$

Size factor: $K := min\left(1 + \sqrt[1]{\dfrac{200}{d}}, 2.0\right) = 1.66$

Smallest width: $b_w := 230\ mm$

Reinforcement ratio: $\rho_\rho := min\left(\dfrac{A_s}{b_w \cdot d}, 0.02\right) = 0.011$

$\nu_{min} := 0.035 \cdot K^{\frac{3}{2}} \cdot f_{ck}^{\frac{1}{2}} = 0.31$

$V_{Rdc} := \max\left(\dfrac{0.18}{\gamma_c} \cdot K \cdot (100 \cdot \rho_\rho \cdot f_{ck})^{\frac{1}{3}} \cdot b_w \cdot d, \nu_{min}\right) = 3.684 \cdot 10^4 \qquad N$

$V_{Rdc} := 27.77\ kN$

$V_{Rdc} < V_{ed}$ $\qquad$ Shear reinforcement is necessary

Then we will figure out the VRdcmax $\qquad \alpha_{cw} := 1.0$

Distance between Fc and Fs $\qquad Z := 0.9 \cdot d = 272.7\ mm$

Performance factor of the concrete $\qquad \nu := 0.6 \cdot \left(1 - \dfrac{f_{ck}}{250}\right) = 0.552$

$V_{Rdmax} := \alpha_{cw} \cdot b_w \cdot Z \cdot \nu \cdot f_{cd} \cdot 0.5\ mm = 230.813\ kN$

$V_{Rdmax} > V_{ed}$ $\qquad$ The geometry is suitable for shear reinforcement

Design of shear links $\qquad$ Assume $\phi_{sw} := 6\ mm$

$A_{sw} := 2 \cdot \dfrac{\pi \cdot \phi_{sw}^2}{4} = 56.549\ mm^2$

Design the space of shear links

$$S_{req} := \frac{A_{sw} \cdot f_{yd}}{V_{ed}} \cdot Z \cdot 1.0 = 133.853\ mm$$

$$S_{max1} := 0.75 \cdot d = 227.25\ mm \qquad b_w := 230\ mm - 2 \cdot 25\ mm = 180\ mm$$

Minimun shear reinforcement ρ_w

$$\rho_w := \frac{A_{sw}}{S_{max1} \cdot b_w} = 0.001 \qquad f_{ck} := 20 \qquad f_{yk} := 500$$

$$\rho_{wmin} := 0.08 \cdot \frac{\left(\sqrt{f_{ck}}\right)}{f_{yk}} = 7.155 \cdot 10^{-4} \qquad \alpha_c := 1.0$$

$$\rho_{wmax} := 0.5 \cdot \frac{\alpha_c \cdot \nu \cdot f_{cd}}{(1 - \cos(0.5\ \pi)) \cdot f_{yd}} = 0.008$$

$$S_{min} := \frac{A_{sw}}{\rho_{wmax} \cdot b_w} = 37.117\ mm \qquad S_{max2} := \frac{A_{sw}}{\rho_{wmin} \cdot b_w} = 439.051\ mm$$

$$S_{max} := min\left(S_{max1}, S_{max2}\right) = 227.25\ mm$$

Stirrups space in the middle span

$$S_{provide} := 220\ mm \qquad S_{provide} > S_{min}$$

Stirrups space at the support

$$S_{provide} := 10 \cdot \left(\left(\frac{S_{req}}{10}\right) - 0.5\ mm\right) = 128.853\ mm$$

$$S_{provide} := 120\ mm$$

SLS analysis at middle span

$$M_{qp} := 21.21\ kN \cdot m$$

Uncrack cross section properties SS1

$$h := 350\ mm \qquad b := 230\ mm \qquad d := 303\ mm \qquad A_s := 339.292\ mm^2$$

E_s effective modolus for steel which is 200 $\qquad E_s := 200\ \frac{kN}{mm^2}$

E_c effective modolus for concrete C20/25 which is 30 $\qquad E_c := 30\ \frac{kN}{mm^2}$

$$\alpha_e := \frac{E_s}{E_{ceff}} = 22.54 \qquad f_{ctm} := 2.2\ \frac{N}{mm^2}$$

Homogenious concrete area

$$A_{i1} := h \cdot b + \left(\alpha_e - 1\right) \cdot A_s = \left(8.781 \cdot 10^4\right)\ mm^2$$

Neutral Axis

$$x_{i1} := \frac{h \cdot b \cdot \frac{h}{2} + (\alpha_e - 1) \cdot (A_s \cdot d)}{A_{i1}} = 185.653 \ mm$$

Inertia

$$I_{i1} := \frac{b \cdot x_{i1}^{3}}{3} + \frac{b \cdot (h - x_{i1})^{3}}{3} + (\alpha_e - 1) \cdot \left(A_s \cdot (d - x_{i1})^{2}\right) = \left(9.315 \cdot 10^{8}\right) \ mm^{4}$$

Crack moment

$$M_{cr} := \frac{f_{ctm} \cdot I_{i1}}{h - x_{i1}} = 12.47 \ kN \cdot m \qquad \frac{M_{cr}}{M_{qp}} = 0.588$$

Cracked section

Geometrical properties for craked analysis SS2

Neutral Axis

$$x_{i2} := x_{i2} = \frac{\frac{1}{2} \cdot x_{i2}^{2} \cdot b + \alpha_e \cdot A_s \cdot d}{x_{i2} \cdot b + \alpha_e \cdot A_s} \xrightarrow{solve, x_{i2}} \begin{bmatrix} 112.54135350539928277 \cdot mm \\ -179.04164888152074089 \cdot mm \end{bmatrix} = \begin{bmatrix} 112.541 \\ -179.042 \end{bmatrix} mm$$

$$x_{i2} := 112.541 \ mm$$

Inertia

$$I_{i2} := \frac{b \cdot x_{i2}^{3}}{3} + \alpha_e \cdot A_s \cdot (d - x_{i2})^{2} = \left(3.867 \cdot 10^{8}\right) \ mm^{4}$$

Significant stress value

Concrete stress $\sigma_c := \frac{M_{qp}}{I_{i2}} \cdot x_{i2} = 6.173 \ \frac{N}{mm^{2}}$

Steel stress $\sigma_s := \alpha_e \cdot \frac{M_{qp}}{I_{i2}} \cdot (d - x_{i2}) = 235.465 \ \frac{N}{mm^{2}}$

Check for the deformation
According to uncracked analysis:

$$k_1 := \frac{M_{qp}}{E_{ceff} \cdot I_{i1}} = \left(2.566 \cdot 10^{-6}\right) \frac{1}{mm} \qquad e_1 := \frac{5}{384} \cdot \frac{p_{qp} \cdot l_{eff}^{4}}{E_{ceff} \cdot I_{i1}} = 12.294 \ mm$$

According to cracked analysis:

$$k_2 := \frac{M_{qp}}{E_{ceff} \cdot I_{i2}} = \left(6.182 \cdot 10^{-6}\right) \frac{1}{mm} \qquad e_2 := \frac{5}{384} \cdot \frac{p_{qp} \cdot l_{eff}^{4}}{E_{ceff} \cdot I_{i2}} = 29.616 \ mm$$

Combination factor:

$$\zeta' := 1 - 0.5 \cdot \left(\frac{M_{cr}}{M_{qp}}\right)^2 = 0.827$$

Calculated deformations:

$$k_{EC} := \zeta' \cdot k_2 + (1 - \zeta') \cdot k_1 = (5.557 \cdot 10^{-6}) \frac{1}{mm}$$

$$e_{EC} := \zeta' \cdot e_2 + (1 - \zeta') \cdot e_1 = 26.622\ mm$$

Verification:

$$e_{max} := \frac{l_{eff}}{200} = 33\ mm \qquad e_{max} > e_{EC} \qquad Ok$$

Crack width analysis:

Steel strain: $\varepsilon_s := \frac{\sigma_s}{E_s} = 0.001$

Concrete strain: $\varepsilon_c := \frac{f_{ctm}}{E_{ceff}} = 2.479 \cdot 10^{-4}$

Tension stiffening of concrete:
Effective concrete area for tension stiffening

$$h_{c.eff} := min\left(2.5 \cdot (h - d), \frac{(h - x_{i2})}{3}, \frac{h}{2}\right) = 79.153\ mm$$

$$A_{ceff} := h_{c.eff} \cdot b = (1.821 \cdot 10^4)\ mm^2 \qquad \varepsilon_{sc} := \frac{f_{ctm} \cdot A_{ceff}}{E_s \cdot A_s} = 5.902 \cdot 10^{-4}$$

$$\rho_{\rho.eff} := \frac{A_s}{A_{ceff}} = 0.019$$

Durability of loads:

Long term load: $k_t := 0.4$

Difference of strain: $\triangle\varepsilon := \varepsilon_s - k_t \cdot \varepsilon_{sc} - k_t \cdot \varepsilon_c = 8.421 \cdot 10^{-4}$

Maximal distance between two neighbouring cracks:

where:

$c := 25\ mm$ Conncrete cover
$k_1 := 0.8$ For ribbed bars
$k_2 := 0.5$ For flexure
$\phi := 12\ mm$ Bar diameter

$$S_{r.max} := 3.4\ c + \frac{0.425\ k_1 \cdot k_2 \cdot \phi}{\rho_{\rho.eff}} = 194.459\ mm$$

Crack width:

$w_k := S_{r.max} \cdot \triangle\varepsilon = 0.164\ mm$

$w_{k.max} := 0.3\ mm$ $\qquad w_{k.max} > w_k \qquad Ok$

SLS analysis at support

$M_{qp} := 49.67\ kN \cdot m$

Uncrack cross section properties SS1

$h := 350\ mm \qquad b := 230\ mm \qquad d := 303\ mm \qquad A_s := 791.681\ mm^2$

E_s effective modolus for steel which is 200 $\qquad E_s := 200\ \dfrac{kN}{mm^2}$

E_c effective modolus for concrete C20/25 which is 30 $\qquad E_c := 30\ \dfrac{kN}{mm^2}$

$\alpha_e := \dfrac{E_s}{E_{ceff}} = 22.54 \qquad f_{ctm} := 2.2\ \dfrac{N}{mm^2}$

Homogenious concrete area

$$A_{i1} := h \cdot b + (\alpha_e - 1) \cdot A_s = (9.755 \cdot 10^4)\ mm^2$$

Neutral Axis

$$x_{i1} := \frac{h \cdot b \cdot \frac{h}{2} + (\alpha_e - 1) \cdot (A_s \cdot d)}{A_{i1}} = 197.375\ mm$$

Inertia

$$I_{i1} := \frac{b \cdot x_{i1}^{\ 3}}{3} + \frac{b \cdot (h - x_{i1})^3}{3} + (\alpha_e - 1) \cdot \left(A_s \cdot (d - x_{i1})^2\right) = (1.052 \cdot 10^9)\ mm^4$$

Crack moment

$$M_{cr} := \frac{f_{ctm} \cdot I_{i1}}{h - x_{i1}} = 15.169\ kN \cdot m \qquad \frac{M_{cr}}{M_{qp}} = 0.305$$

Cracked section

Geometrical properties for craked analysis SS2

Neutral Axis

$$x_{i2.} := x_{i2.} = \frac{\frac{1}{2} \cdot x_{i2.}^{\ 2} \cdot b + \alpha_e \cdot A_s \cdot d}{x_{i2.} \cdot b + \alpha_e \cdot A_s} \xrightarrow{solve,\, x_{i2.}} \begin{bmatrix} 152.70945286319035251 \cdot mm \\ -307.8767434083939296 \cdot mm \end{bmatrix} = \begin{bmatrix} 152.709 \\ -307.877 \end{bmatrix} mm$$

$x_{i2} := 152.709\ mm$

Inertia

$$I_{i2} := \frac{b \cdot x_{i2}^{\ 3}}{3} + \alpha_e \cdot A_s \cdot \left(d - x_{i2}\right)^2 = \left(6.761 \cdot 10^8\right)\ mm^4$$

Significant stress value

Concrete stress $\quad \sigma_c := \frac{M_{qp}}{I_{i2}} \cdot x_{i2} = 11.219\ \frac{N}{mm^2}$

Steel stress $\quad \sigma_s := \alpha_e \cdot \frac{M_{qp}}{I_{i2}} \cdot \left(d - x_{i2}\right) = 248.873\ \frac{N}{mm^2}$

Check for the deformation
According to uncracked analysis:

$$k_1 := \frac{M_{qp}}{E_{ceff} \cdot I_{i1}} = \left(5.319 \cdot 10^{-6}\right)\ \frac{1}{mm} \qquad e_1 := \frac{5}{384} \cdot \frac{p_{qp} \cdot l_{eff}^{\ 4}}{E_{ceff} \cdot I_{i1}} = 10.883\ mm$$

According to cracked analysis:

$$k_2 := \frac{M_{qp}}{E_{ceff} \cdot I_{i2}} = \left(8.28 \cdot 10^{-6}\right)\ \frac{1}{mm} \qquad e_2 := \frac{5}{384} \cdot \frac{p_{qp} \cdot l_{eff}^{\ 4}}{E_{ceff} \cdot I_{i2}} = 16.939\ mm$$

Combination factor:

$$\zeta' := 1 - 0.5 \cdot \left(\frac{M_{cr}}{M_{qp}}\right)^2 = 0.953$$

Calculated deformations:

$$k_{EC} := \zeta' \cdot k_2 + \left(1 - \zeta'\right) \cdot k_1 = \left(8.142 \cdot 10^{-6}\right)\ \frac{1}{mm}$$

$$e_{EC} := \zeta' \cdot e_2 + \left(1 - \zeta'\right) \cdot e_1 = 16.657\ mm$$

Verification:

$$e_{max} := \frac{l_{eff}}{200} = 33\ mm \qquad e_{max} > e_{EC} \qquad Ok$$

Crack width analysis:

Steel strain: $\quad \varepsilon_s := \frac{\sigma_s}{E_s} = 0.001$

Concrete strain: $\quad \varepsilon_c := \frac{f_{ctm}}{E_{ceff}} = 2.479 \cdot 10^{-4}$

Tension stiffening of concrete:
Effective concrete area for tension stiffening

$$h_{c.eff} := min\left(2.5 \cdot (h-d), \frac{(h-x_{i2})}{3}, \frac{h}{2}\right) = 65.764\ mm$$

$$A_{ceff} := h_{c.eff} \cdot b = \left(1.513 \cdot 10^4\right)\ mm^2 \qquad \varepsilon_{sc} := \frac{f_{ctm} \cdot A_{ceff}}{E_s \cdot A_s} = 2.102 \cdot 10^{-4}$$

$$\rho_{\rho.eff} := \frac{A_s}{A_{ceff}} = 0.052$$

Durability of loads:

Long term load: $k_t := 0.4$

Difference of strain: $\triangle\varepsilon := \varepsilon_s - k_t \cdot \varepsilon_{sc} - k_t \cdot \varepsilon_c = 0.001$

Maximal distance between two neighbouring cracks:

where:

$c := 25\ mm$	Conncrete cover
$k_1 := 0.8$	For ribbed bars
$k_2 := 0.5$	For flexure
$\phi := 12\ mm$	Bar diameter

$$S_{r.max} := 3.4\ c + \frac{0.425\ k_1 \cdot k_2 \cdot \phi}{\rho_{\rho.eff}} = 123.976\ mm$$

Crack width:

$$w_k := S_{r.max} \cdot \triangle\varepsilon = 0.132\ mm$$

$w_{k.max} := 0.3\ mm \qquad w_{k.max} > w_k \qquad Ok$