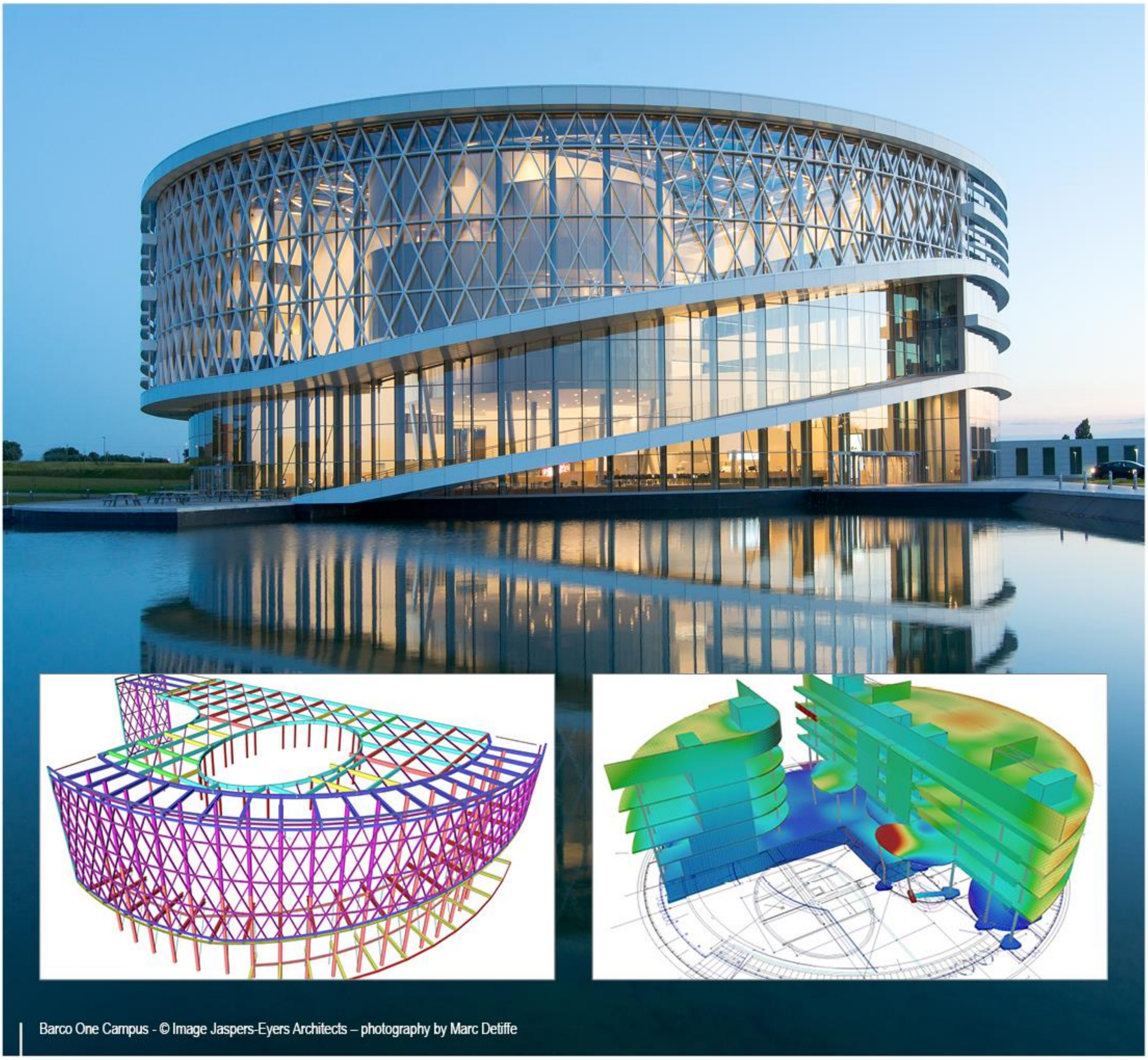
SCIA ENGINEER



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Advanced Concept Training

Reinforced concrete (EN 1992) – 1D members

Table of contents

Introduction

1D concrete design is available in the **Concept Edition** of SCIA Engineer.

Both beam and column design are part of the module esacd.01.01 (1D concrete design for EN1992). Practical reinforcement on 1D members is part of the module esacdt.01.

Both modules are part of the Concept Edition of SCIA Engineer.

All topics that will be treated in this training document about basic concrete calculation for 1D members are available in the modules described above.

This manual is handling the practical approach in SCIA Engineer in accordance with EN1992 for the concrete menu available since SCIA Engineer 17. For more theoretical background, reference is made to “Topic Training: New Concrete”.

For 2D members and advanced concrete calculations reference is made to the respective training documents “Advanced Concept Training: Reinforced concrete (EN1992) – 2D members” and “Advanced Concept Training: Reinforced concrete (EN1992) – Adv. modules”.

Materials

Verification by the partial factor method

Design values (art. 2.4.2)

Partial factors for materials (art. 2.4.2.4)

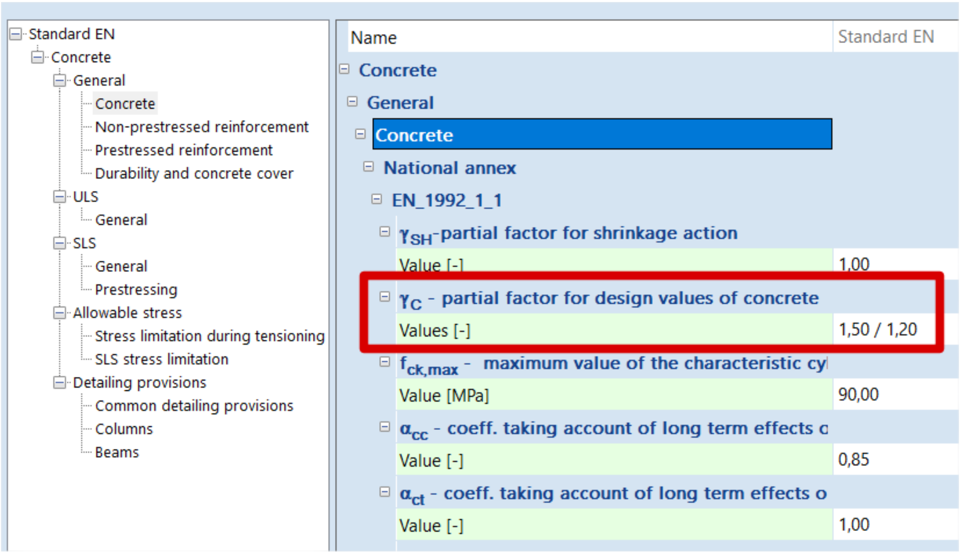
Partial factors for materials for ultimate limit states, γ_c and γ_s should be used.

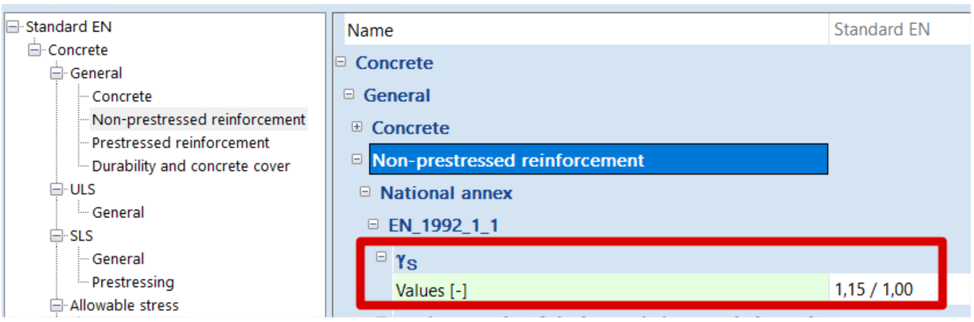
The recommended values of γ_c and γ_s for 'persistent & transient' and 'accidental, design situations are given in the following table. These are not valid for fire design for which reference should be made to EN 1992-1-2.

For fatigue verification the partial factors for persistent design situations given in this table are recommended for the values of $\gamma_{c,fat}$ and $\gamma_{s,fat}$.

Design situations	γ_C for concrete	γ_S for reinforcing steel	γ_S for prestressing steel
Persistent & Transient	1,5	1,15	1,15
Accidental	1,2	1,0	1,0

These values can also be found in the Concrete setup of the National Annex:





All factors related to the code are shown in green on the screen. By default, the values of the chosen code are taken.

The values for partial factors for materials for serviceability limit state verification should be taken as those given in the particular clauses of this Eurocode.

The recommended values of γ_c and γ_s in the serviceability limit state for situations not covered by particular clauses of this Eurocode is 1,0.

Lower values of γ_c and γ_s may be used if justified by measures reducing the uncertainty in the calculated resistance.

Concrete

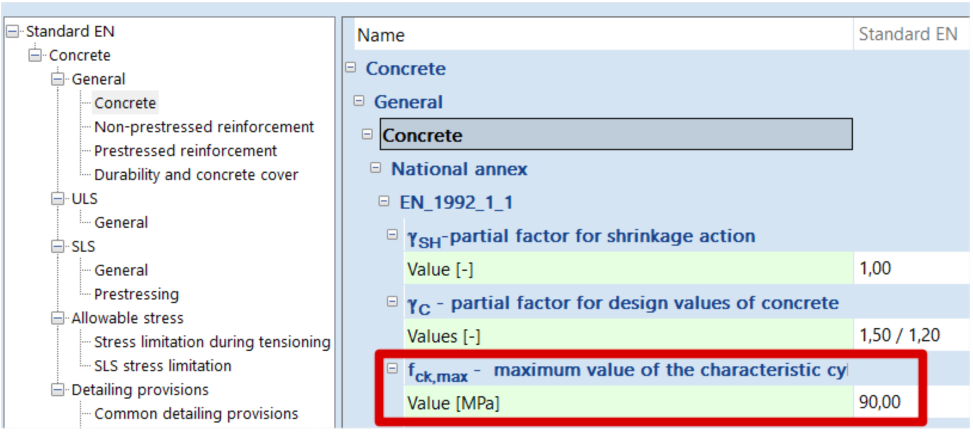
The following clauses give principles and rules for normal and high strength concrete.

Strength (art 3.1.2)

The compressive strength of concrete is denoted by concrete strength classes which relate to the characteristic (5%) cylinder strength f_{ck}, or the cube strength $f_{ck,cube}$.

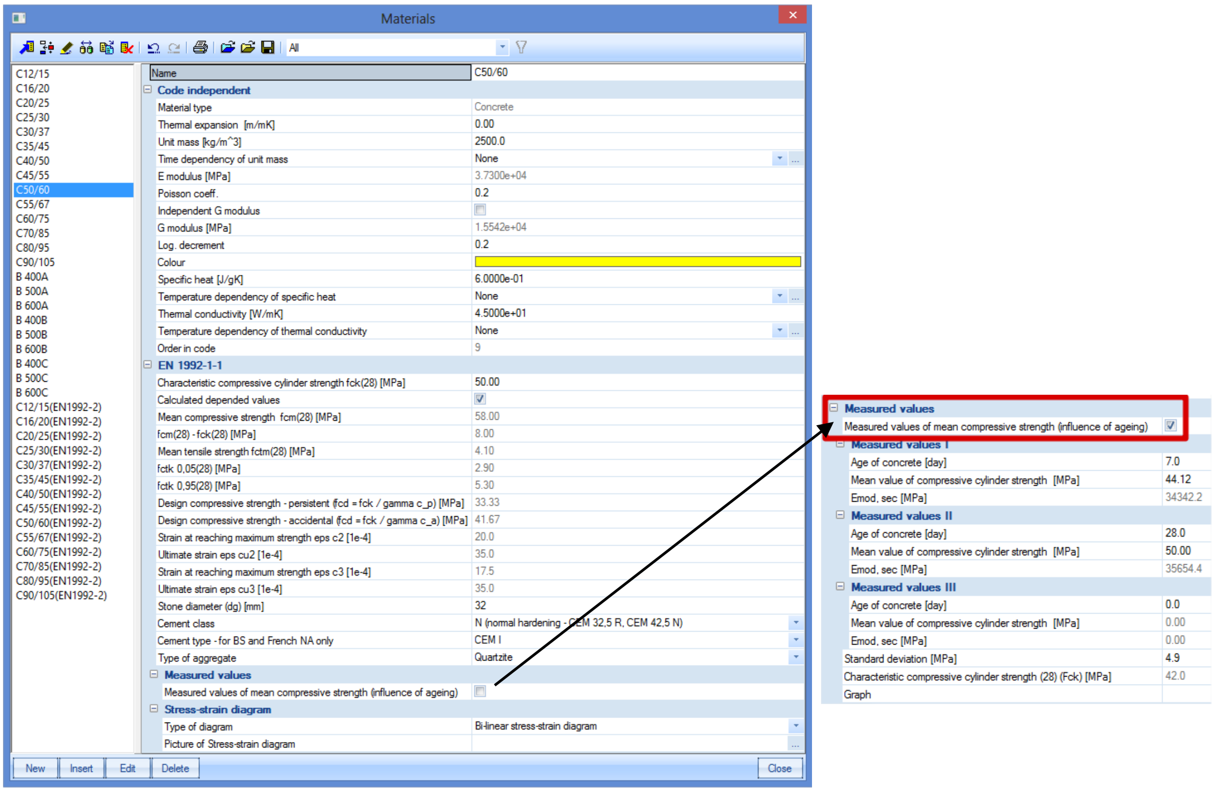
The strength classes in this code are based on the characteristic cylinder strength f_{ck} determined at 28 days with a maximum value of C_{max}.

The recommended value of C_{max} is C90/105.



In certain situations (e.g. prestressing) it may be appropriate to assess the compressive strength for concrete before or after 28 days, on the basis of test specimens stored under other conditions than prescribed in EN 12390.

All values can also be found in the material library of SCIA Engineer:



It may be required to specify the concrete compressive strength, $f_{ck}(t)$, at time t for a number of stages (e.g. demoulding, transfer of prestress), where:

$f_{ck}(t) = f_{cm}(t) - 8$ (MPa) for $3 < t < 28$ days
$f_{ck}(t) = f_{ck}$ for $t \geq 28$ days

The compressive strength of concrete at an age t depends on the type of cement, temperature and curing conditions. For a mean temperature of 20°C and curing in accordance with EN 12390 the compressive strength of concrete at various ages $f_{cm}(t)$ may be estimated from:

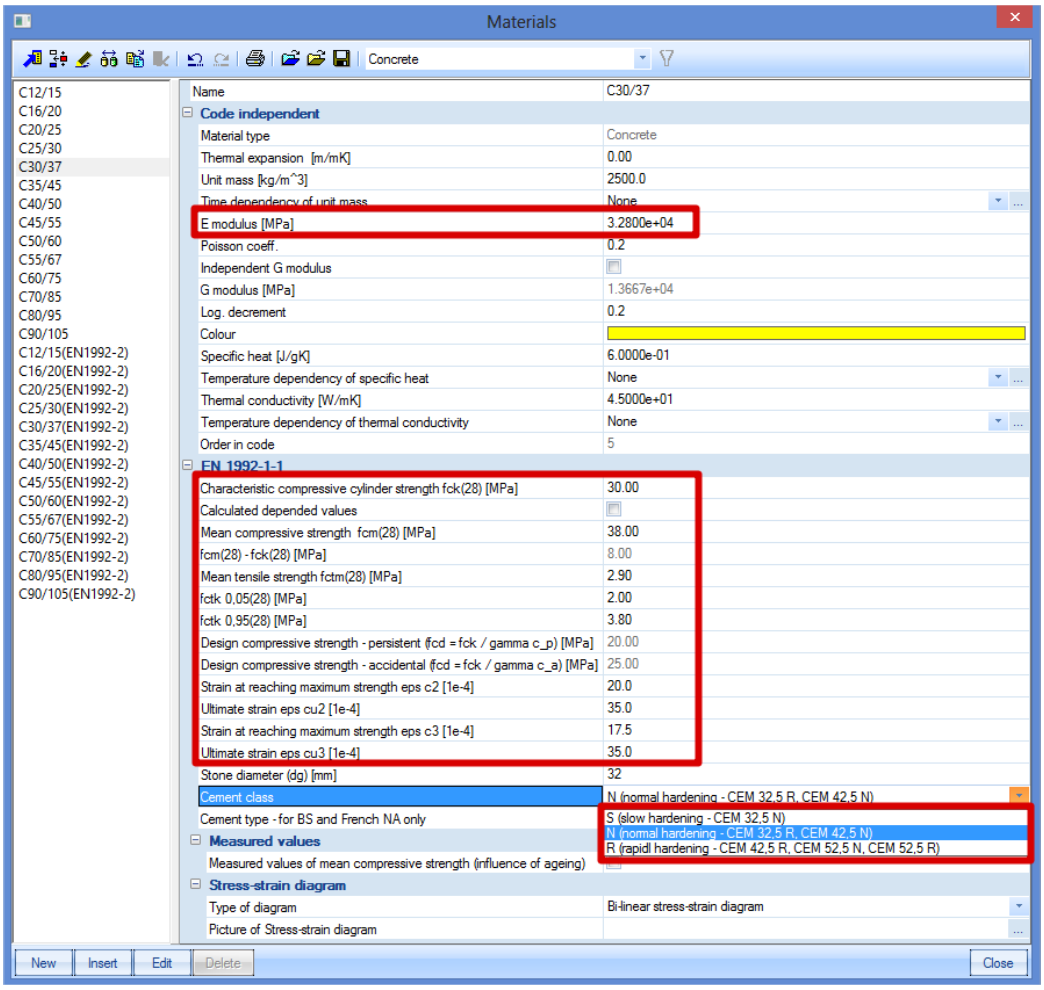
$$f_{cm}(t) = \beta_{cc}(t)\, f_{cm} \qquad (3.1)$$

$$\text{with } \beta_{cc}(t) = e^{\left\{s\left[1-\left(\frac{28}{t}\right)^{\frac{1}{2}}\right]\right\}} \qquad (3.2)$$

where:

$f_{cm}(t)$ is the mean concrete compressive strength at an age of t days
f_{cm} is the mean compressive strength at 28 days according to Table 3.1
$\beta_{cc}(t)$ is a coefficient which depends on the age of the concrete t
t is the age of the concrete in days
s is a coefficient which depends on the type of cement:
= 0,20 for cement of strength Classes CEM 42,5 R, CEM 52,5 N and CEM 52,5 R (Class R)
= 0,25 for cement of strength Classes CEM 32,5 R, CEM 42,5 N (Class N)
= 0,38 for cement of strength Classes CEM 32,5 N (Class S)

The type of cement can be chosen in the material library:



The tensile strength refers to the highest stress reached under concentric tensile loading.

The characteristic strengths for f_{ck} and the corresponding mechanical characteristics necessary for design, are given in Table 3.1:

Table 3.1 Strength and deformation characteristics for concrete

<table>
<tr><th></th><th colspan="14">Strength classes for concrete</th><th>Analytical relation / Explanation</th></tr>
<tr><td>f_{ck} (MPa)</td><td>12</td><td>16</td><td>20</td><td>25</td><td>30</td><td>35</td><td>40</td><td>45</td><td>50</td><td>55</td><td>60</td><td>70</td><td>80</td><td>90</td><td></td></tr>
<tr><td>$f_{ck,cube}$ (MPa)</td><td>15</td><td>20</td><td>25</td><td>30</td><td>37</td><td>45</td><td>50</td><td>55</td><td>60</td><td>67</td><td>75</td><td>85</td><td>95</td><td>105</td><td></td></tr>
<tr><td>f_{cm} (MPa)</td><td>20</td><td>24</td><td>28</td><td>33</td><td>38</td><td>43</td><td>48</td><td>53</td><td>58</td><td>63</td><td>68</td><td>78</td><td>88</td><td>98</td><td>$f_{cm} = f_{ck}+8$(MPa)</td></tr>
<tr><td>f_{ctm} (MPa)</td><td>1,6</td><td>1,9</td><td>2,2</td><td>2,6</td><td>2,9</td><td>3,2</td><td>3,5</td><td>3,8</td><td>4,1</td><td>4,2</td><td>4,4</td><td>4,6</td><td>4,8</td><td>5,0</td><td>$f_{ctm}=0,30\times f_{ck}^{(2/3)} \leq$ C50/60
$f_{ctm}=2,12 \cdot \ln(1+(f_{cm}/10))$ > C50/60</td></tr>
<tr><td>$f_{ctk,0,05}$ (MPa)</td><td>1,1</td><td>1,3</td><td>1,5</td><td>1,8</td><td>2,0</td><td>2,2</td><td>2,5</td><td>2,7</td><td>2,9</td><td>3,0</td><td>3,1</td><td>3,2</td><td>3,4</td><td>3,5</td><td>$f_{ctk;0,05} = 0,7\times f_{ctm}$
5% fractile</td></tr>
<tr><td>$f_{ctk,0,95}$ (MPa)</td><td>2,0</td><td>2,5</td><td>2,9</td><td>3,3</td><td>3,8</td><td>4,2</td><td>4,6</td><td>4,9</td><td>5,3</td><td>5,5</td><td>5,7</td><td>6,0</td><td>6,3</td><td>6,6</td><td>$f_{ctk;0,95} = 1,3\times f_{ctm}$
95% fractile</td></tr>
<tr><td>E_{cm} (GPa)</td><td>27</td><td>29</td><td>30</td><td>31</td><td>33</td><td>34</td><td>35</td><td>36</td><td>37</td><td>38</td><td>39</td><td>41</td><td>42</td><td>44</td><td>$E_{cm} = 22[(f_{cm})/10]^{0,3}$
(f_{cm} in MPa)</td></tr>
<tr><td>ε_{c1} (‰)</td><td>1,8</td><td>1,9</td><td>2,0</td><td>2,1</td><td>2,2</td><td>2,25</td><td>2,3</td><td>2,4</td><td>2,45</td><td>2,5</td><td>2,6</td><td>2,7</td><td>2,8</td><td>2,8</td><td>see Figure 3.2
ε_{c1} (‰) $= 0,7 f_{cm}^{0,31} < 2,8$</td></tr>
<tr><td>ε_{cu1} (‰)</td><td colspan="9">3,5</td><td>3,2</td><td>3,0</td><td>2,8</td><td>2,8</td><td>2,8</td><td>see Figure 3.2
for $f_{ck} \geq 50$ Mpa
ε_{cu1}(‰)$=2,8+27[(98-f_{cm})/100]^4$</td></tr>
<tr><td>ε_{c2} (‰)</td><td colspan="9">2,0</td><td>2,2</td><td>2,3</td><td>2,4</td><td>2, 5</td><td>2,6</td><td>see Figure 3.3
for $f_{ck} \geq 50$ Mpa
ε_{c2}(‰)$=2,0+0,085(f_{ck}-50)^{0,53}$</td></tr>
<tr><td>ε_{cu2} (‰)</td><td colspan="9">3,5</td><td>3,1</td><td>2,9</td><td>2,7</td><td>2,6</td><td>2,6</td><td>see Figure 3.3
for $f_{ck} \geq 50$ Mpa
ε_{cu2}(‰)$=2,6+35[(90-f_{ck})/100]^4$</td></tr>
<tr><td>n</td><td colspan="9">2,0</td><td>1,75</td><td>1,6</td><td>1,45</td><td>1,4</td><td>1,4</td><td>for $f_{ck} \geq 50$ Mpa
$n=1,4+23,4[(90-f_{ck})/100]^4$</td></tr>
<tr><td>ε_{c3} (‰)</td><td colspan="9">1,75</td><td>1,8</td><td>1,9</td><td>2,0</td><td>2,2</td><td>2,3</td><td>see Figure 3.4
for $f_{ck} \geq 50$ Mpa
ε_{c3}(‰)$=1,75+0,55[(f_{ck}-50)/40]$</td></tr>
<tr><td>ε_{cu3} (‰)</td><td colspan="9">3,5</td><td>3,1</td><td>2,9</td><td>2,7</td><td>2,6</td><td>2,6</td><td>see Figure 3.4
for $f_{ck} \geq 50$ Mpa
ε_{cu3}(‰)$=2,6+35[(90-f_{ck})/100]^4$</td></tr>
</table>

Design compressive and tensile strengths (art 3.1.6)

1) The value of the design compressive strength is defined as

$$f_{cd} = \alpha_{cc}\, f_{ck} / \gamma_C \qquad (3.15)$$

where:

γ_C is the partial safety factor for concrete.
α_{cc} is the coefficient taking account of long term effects on the compressive strength and of unfavourable effects resulting from the way the load is applied.

The value of α_{cc} should lie between 0,8 and 1,0. The recommended value is 1,0.[1]

2) The value of the design tensile strength, f_{ctd}, is defined as

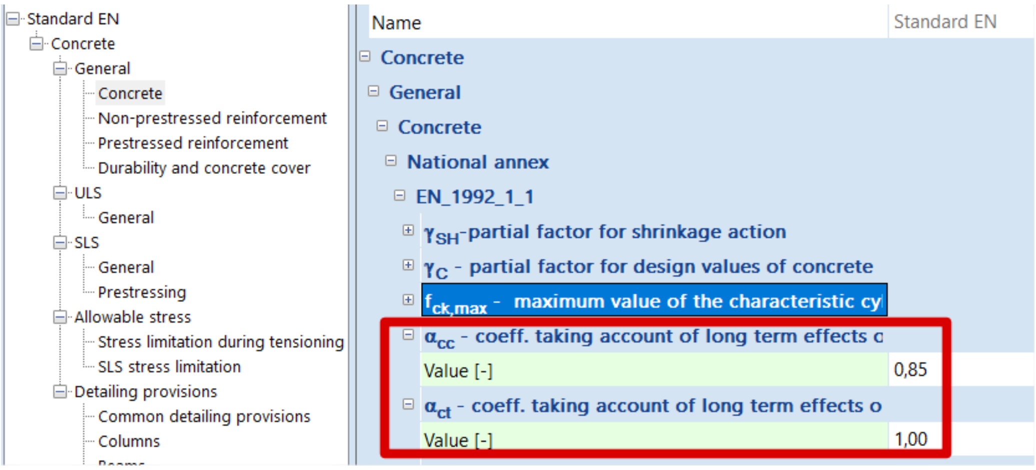
$$f_{ctd} = \alpha_{ct}\, f_{ctk,0,05} / \gamma_C \qquad (3.16)$$

where:

γ_C is the partial safety factor for concrete.
α_{ct} is a coefficient taking account of long term effects on the tensile strength and of unfavourable effects, resulting from the way the load is applied.

The recommended value of α_{ct} is 1,0.

The values of the coefficients taking account of long term effects can be found in the Concrete setup of the National Annex:



If the concrete strength is determined at an age t > 28 days the values α_{cc} and α_{ct} should be reduced by a factor k_t.

The recommended value of k_t is 0,85.

[1] Remark: *the Belgian National Annex recommends the use of the value 0,85.*

Elastic deformation (art 3.1.3)

The elastic deformations of concrete largely depend on its composition (especially the aggregates). The values given in this Standard should be regarded as indicative for general applications. However, they should be specifically assessed if the structure is likely to be sensitive to deviations from these general values.

The modulus of elasticity of a concrete is controlled by the moduli of elasticity of its components. Approximate values for the modulus of elasticity E_{cm}, secant value between $\sigma_c = 0$ and $0,4f_{cm}$, for concretes with quartzite aggregates, are given in Table 3.1.

For limestone and sandstone aggregates the value should be reduced by 10% and 30% respectively. For basalt aggregates the value should be increased by 20%.

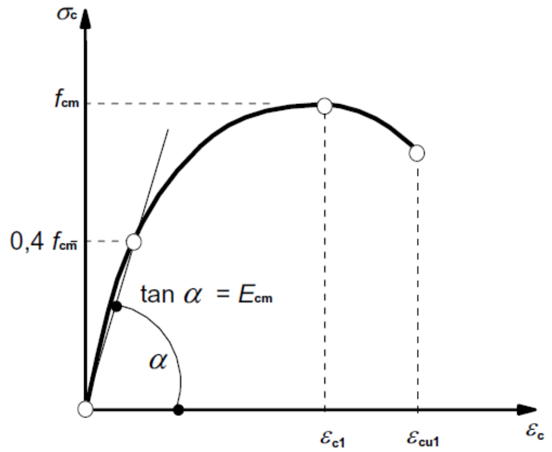


Figure 3.2: Schematic representation of the stress-strain relation for structural analysis (the use $0,4f_{cm}$ for the definition of E_{cm} is approximate).

Variation of the modulus of elasticity with time can be estimated by:

$$E_{cm}(t) = (f_{cm}(t) / f_{cm})^{0,3} E_{cm} \qquad (3.5)$$

where $E_{cm}(t)$ and $f_{cm}(t)$ are the values at an age of t days and E_{cm} and f_{cm} are the values determined at an age of 28 days. The relation between $f_{cm}(t)$ and f_{cm} follows from Expression (3.1).

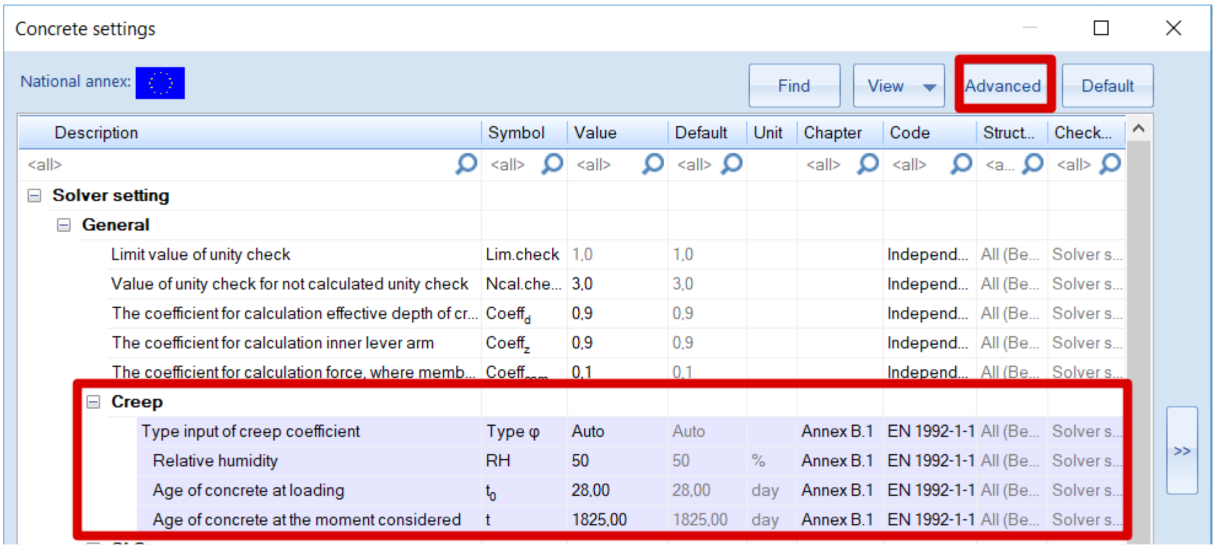
Poisson's ratio may be taken equal to 0,2 for uncracked concrete and 0 for cracked concrete.

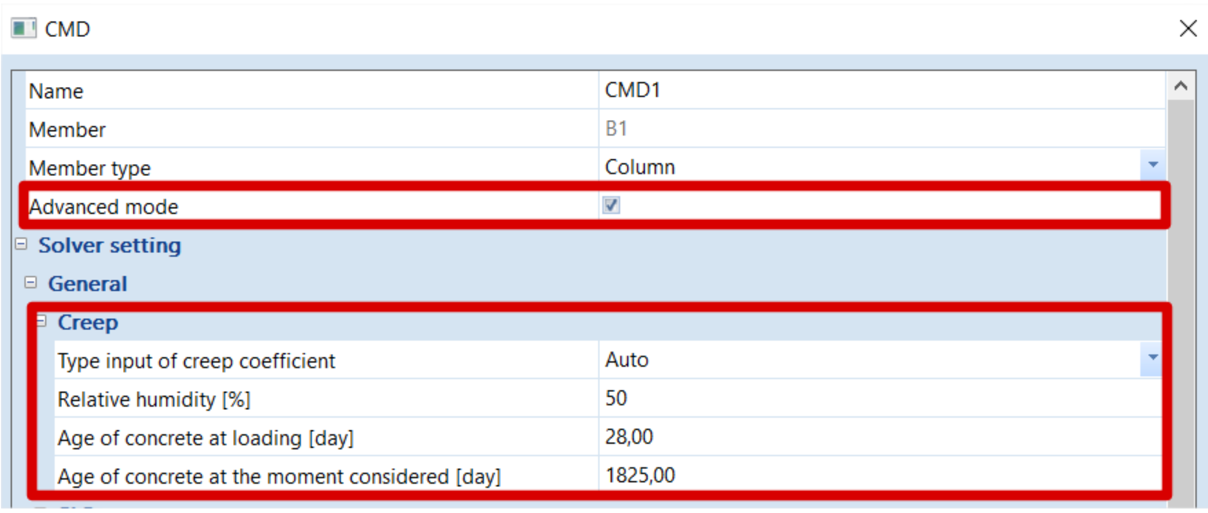
Creep and shrinkage (art 3.1.4)

Creep and shrinkage of the concrete depend on the ambient humidity, the dimensions of the element and the composition of the concrete. Creep is also influenced by the maturity of the concrete when the load is first applied and depends on the duration and magnitude of the loading.

The value of the creep coefficient can be set in the concrete settings by using ‘Advanced level’ or in the 1D member data (advanced mode is ON) if it is defined. If the type input of the creep coefficient is **“Auto”**, the creep coefficient can be calculated automatically by inputting the age of concrete and the relative humidity (see annex B.1. in EN 1992-1-1).

If the type input of the creep coefficient is **“User value”**, the creep coefficient can be inputted directly by the user.





(1) The creep coefficient $\varphi(t,t_0)$ may be calculated from:

$$\varphi(t,t_0) = \varphi_0 \cdot \beta_c(t,t_0) \tag{B.1}$$

where:

φ_0 is the notional creep coefficient and may be estimated from:

$$\varphi_0 = \varphi_{RH} \cdot \beta(f_{cm}) \cdot \beta(t_0) \tag{B.2}$$

φ_{RH} is a factor to allow for the effect of relative humidity on the notional creep coefficient:

$$\varphi_{RH} = 1 + \frac{1 - RH/100}{0{,}1 \cdot \sqrt[3]{h_0}} \quad \text{for } f_{cm} \leq 35 \text{ MPa} \tag{B.3a}$$

$$\varphi_{RH} = \left[1 + \frac{1 - RH/100}{0{,}1 \cdot \sqrt[3]{h_0}} \cdot \alpha_1\right] \cdot \alpha_2 \quad \text{for } f_{cm} > 35 \text{ MPa} \tag{B.3b}$$

RH is the relative humidity of the ambient environment in %

$\beta(f_{cm})$ is a factor to allow for the effect of concrete strength on the notional creep coefficient:

$$\beta(f_{cm}) = \frac{16{,}8}{\sqrt{f_{cm}}} \tag{B.4}$$

f_{cm} is the mean compressive strength of concrete in MPa at the age of 28 days

$\beta(t_0)$ is a factor to allow for the effect of concrete age at loading on the notional creep coefficient:

$$\beta(t_0) = \frac{1}{(0{,}1 + t_0^{0{,}20})} \tag{B.5}$$

h_0 is the notional size of the member in mm where:

$$h_0 = \frac{2A_c}{u} \tag{B.6}$$

A_c is the cross-sectional area

u is the perimeter of the member in contact with the atmosphere

$\beta_c(t,t_0)$ is a coefficient to describe the development of creep with time after loading, and may be estimated using the following Expression:

$$\beta_c(t,t_0) = \left[\frac{(t - t_0)}{(\beta_H + t - t_0)}\right]^{0{,}3} \tag{B.7}$$

t is the age of concrete in days at the moment considered

t_0 is the age of concrete at loading in days

$t - t_0$ is the non-adjusted duration of loading in days

β_H is a coefficient depending on the relative humidity (RH in %) and the notional member size (h_0 in mm). It may be estimated from:

$$\beta_H = 1{,}5\,[1 + (0{,}012\,RH)^{18}]\,h_0 + 250 \leq 1500 \quad \text{for } f_{cm} \leq 35 \tag{B.8a}$$

$$\beta_H = 1{,}5\,[1 + (0{,}012\,RH)^{18}]\,h_0 + 250\,\alpha_3 \leq 1500\,\alpha_3 \quad \text{for } f_{cm} \geq 35 \tag{B.8b}$$

$\alpha_{1/2/3}$ are coefficients to consider the influence of the concrete strength:

$$\alpha_1 = \left[\frac{35}{f_{cm}}\right]^{0{,}7} \quad \alpha_2 = \left[\frac{35}{f_{cm}}\right]^{0{,}2} \quad \alpha_3 = \left[\frac{35}{f_{cm}}\right]^{0{,}5} \tag{B.8c}$$

Where great accuracy is not required, a value found from a figure (Figure 3.1) may be considered as the creep coefficient, provided that the concrete is not subjected to a compressive stress greater than 0,45 fck (t_0) at an age t_0, the age of concrete at the time of loading.

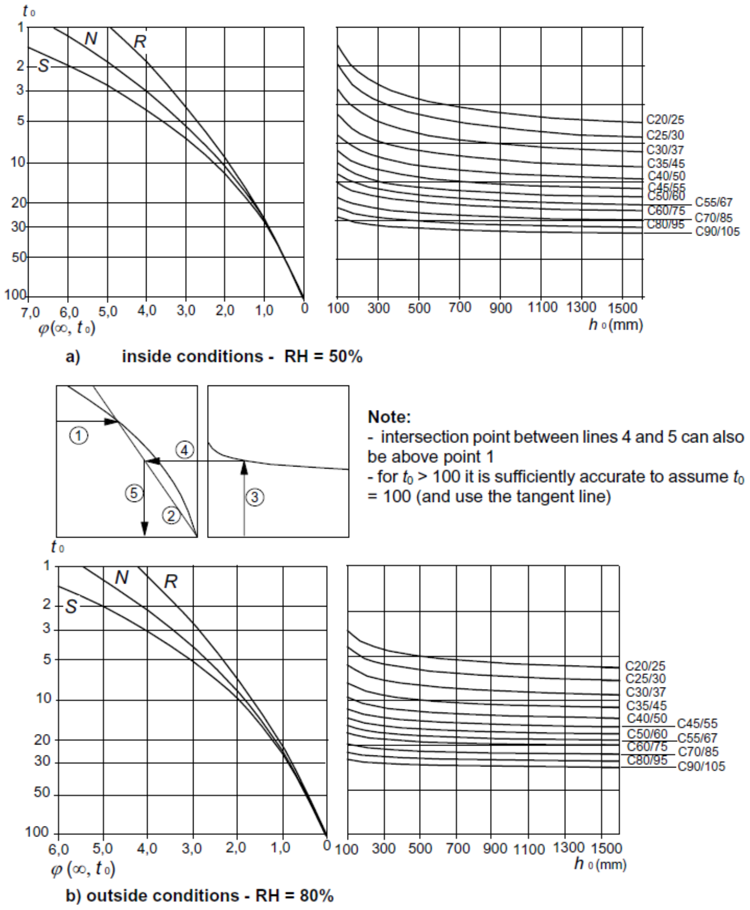


Figure 3.1: Method for determining the creep coefficient $\varphi(\infty, t_0)$ for concrete under normal environmental conditions

Stress-strain relations for the design of cross-sections (art 3.1.7)

For the design of cross-sections, the following stress-strain relationship may be used:

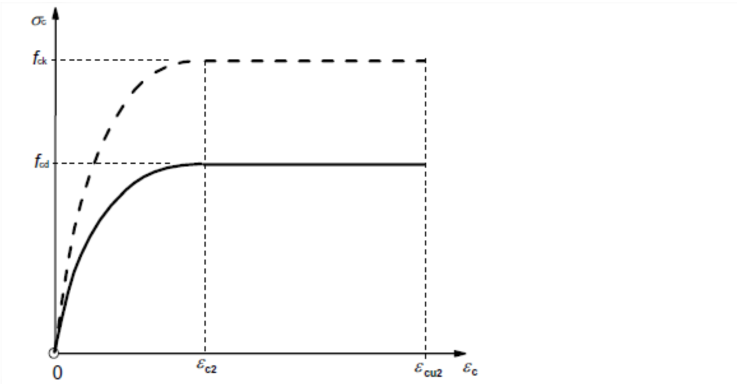


Figure 3.3: Parabola-rectangle diagram for concrete under compression.

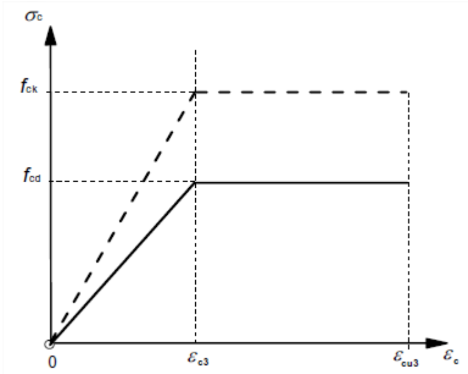


Figure 3.4: Bi-linear stress-strain relation.

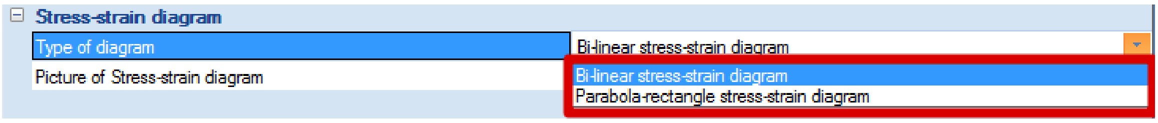
ε_{c2} is the strain at reaching the maximum strength in the parabola-rectangle diagram

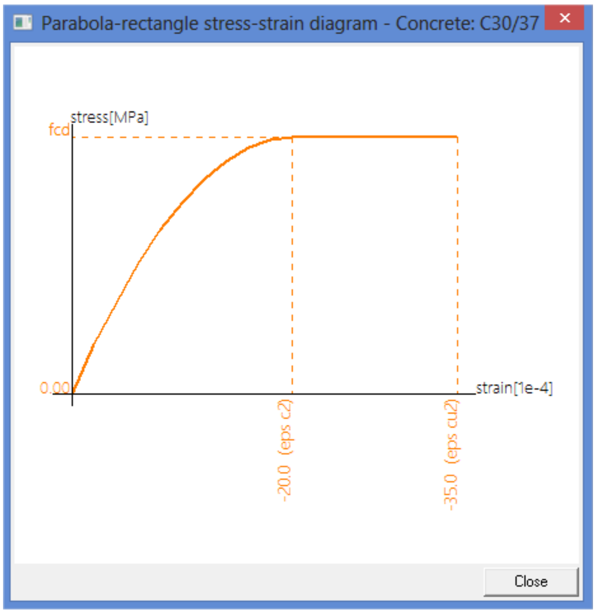
ε_{cu2} is the ultimate strain in the parabola-rectangle diagram

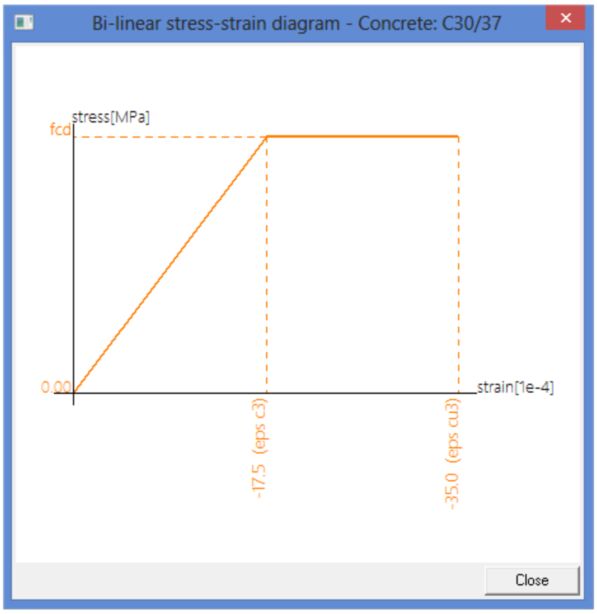
ε_{c3} is the strain at reaching the maximum strength in the bi-linear diagram

ε_{cu3} is the ultimate strain in the bi-linear diagram

The user can choose in the material library which one of the diagrams should be used for the calculation:







Reinforcing steel

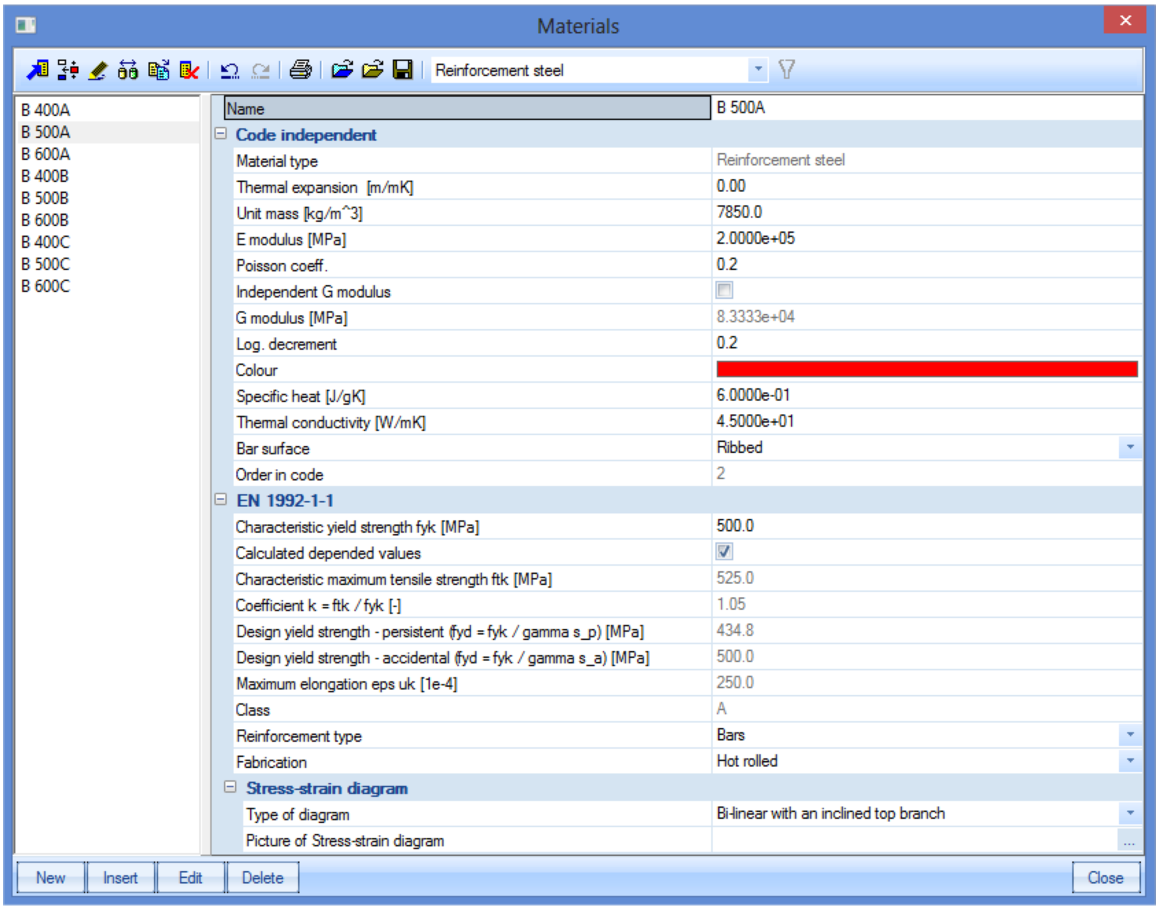
The following clauses give principles and rules for reinforcement which is in the form of bars, de-coiled rods, welded fabric and lattice girders. They do not apply to specially coated bars.

Properties (art 3.2.2)

The behaviour of reinforcing steel is specified by the following properties:

- yield strength (f_{yk} or $f_{0,2k}$)
- maximum actual yield strength ($f_{y,max}$)
- tensile strength (f_t)
- ductility (ε_{uk} and f_t/f_{yk})
- bendability
- bond characteristics (f_R)
- section sizes and tolerances
- fatigue strength
- weldability
- shear and weld strength for welded fabric and lattice girders

The steel properties can be found in the material library:



The mean value of density may be assumed to be 7850 kg/m^3.
The design value of the modulus of elasticity E_s may be assumed to be 200 GPa.

This Eurocode applies to ribbed and weldable reinforcement, including fabric.

The application rules for design and detailing in this Eurocode are valid for a specified yield strength range, f_{yk} = 400 to 600 MPa.

Table C.1 gives the properties of reinforcement suitable for use with this Eurocode:

Table C.1: Properties of reinforcement

Product form		Bars and de-coiled rods			Wire Fabrics			Requirement or quantile value (%)
Class		A	B	C	A	B	C	-
Characteristic yield strength f_{yk} or $f_{0,2k}$ (MPa)		400 to 600						5,0
Minimum value of $k = (f_t/f_y)_k$		≥1,05	≥1,08	≥1,15 <1,35	≥1,05	≥1,08	≥1,15 <1,35	10,0
Characteristic strain at maximum force, ε_{uk} (%)		≥2,5	≥5,0	≥7,5	≥2,5	≥5,0	≥7,5	10,0
Bendability		Bend/Rebend test			-			
Shear strength		-			0,3 A f_{yk} (A is area of wire)			Minimum
Maximum deviation from nominal mass (individual bar or wire) (%)	Nominal bar size (mm) ≤ 8 > 8	± 6,0 ± 4,5						5,0

Design assumptions (art 3.2.7)

For normal design, either of the following assumptions may be made:

B1) an inclined top branch with a strain limit of ε_{ud} and a maximum stress of kf_{yk} / γ_s at ε_{uk}, where $k = (f_t/f_y)_k$.

B2) a horizontal top branch without the need to check the strain limit.

The recommended value of ε_{ud} is 0,9 ε_{uk}. The value of $(f_t/f_y)_k$ is given in Table C.1.

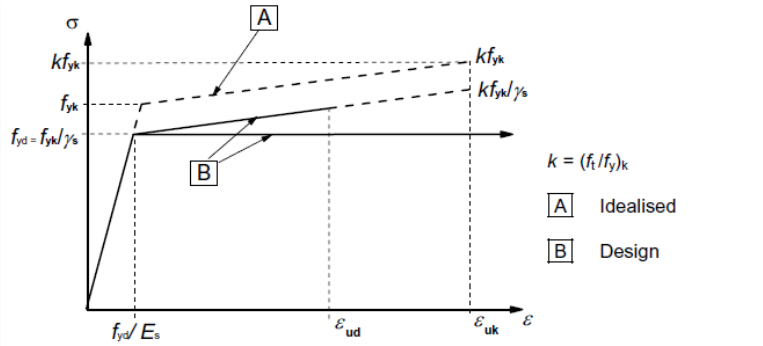
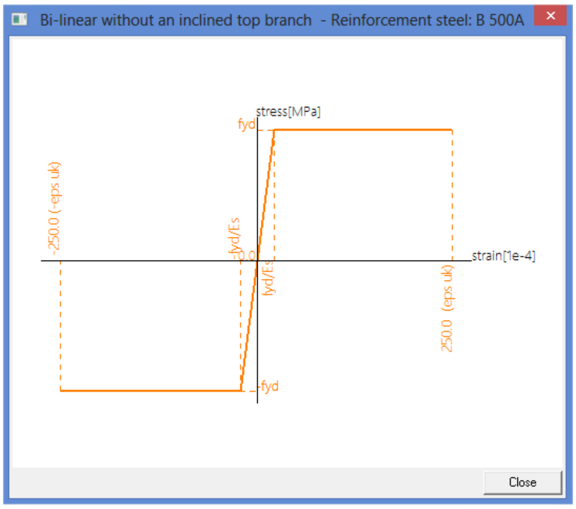
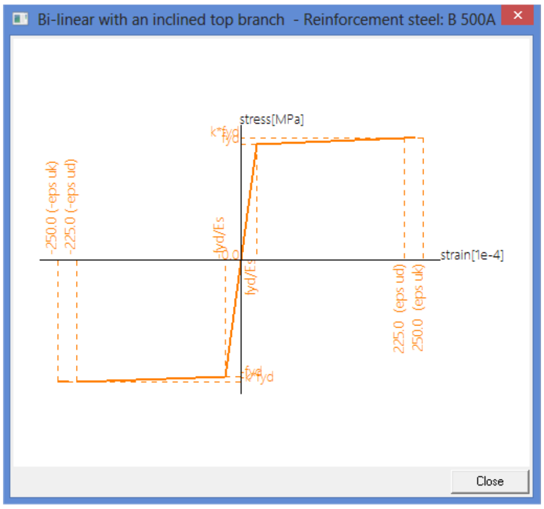


Figure 3.8: Idealised and design stress-strain diagrams for reinforcing steel (for tension and compression)

In the material library the user can choose between the two assumptions:





Durability and cover to reinforcement

Environmental conditions (art 4.2)

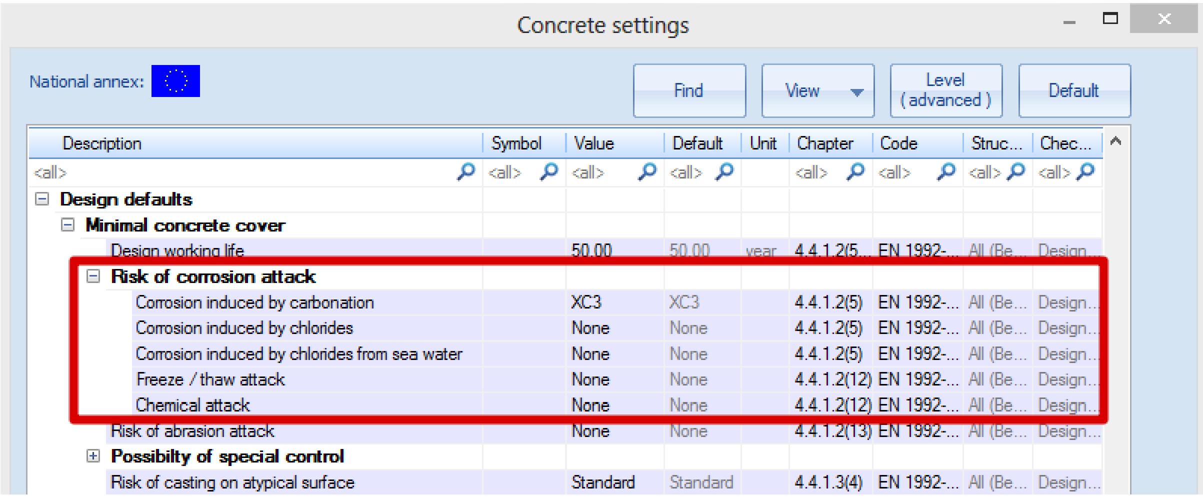
Exposure conditions are chemical and physical conditions to which the structure is exposed in addition to the mechanical actions.

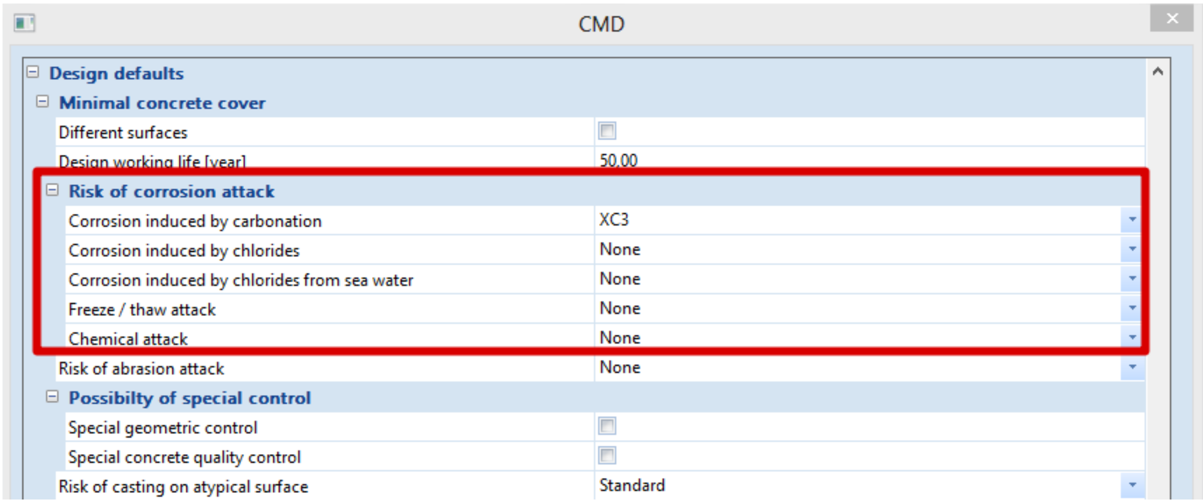
Environmental conditions are classified according to Table 4.1:

Table 4.1: Exposure classes related to environmental conditions in accordance with EN 206-1

Class designation	Description of the environment	Informative examples where exposure classes may occur
1 No risk of corrosion or attack		
X0	For concrete without reinforcement or embedded metal: all exposures except where there is freeze/thaw, abrasion or chemical attack For concrete with reinforcement or embedded metal: very dry	Concrete inside buildings with very low air humidity
2 Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2
3 Corrosion induced by chlorides		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD2	Wet, rarely dry	Swimming pools Concrete components exposed to industrial waters containing chlorides
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides Pavements Car park slabs
4 Corrosion induced by chlorides from sea water		
XS1	Exposed to airborne salt but not in direct contact with sea water	Structures near to or on the coast
XS2	Permanently submerged	Parts of marine structures
XS3	Tidal, splash and spray zones	Parts of marine structures
5. Freeze/Thaw Attack		
XF1	Moderate water saturation, without de-icing agent	Vertical concrete surfaces exposed to rain and freezing
XF2	Moderate water saturation, with de-icing agent	Vertical concrete surfaces of road structures exposed to freezing and airborne de-icing agents
XF3	High water saturation, without de-icing agents	Horizontal concrete surfaces exposed to rain and freezing
XF4	High water saturation with de-icing agents or sea water	Road and bridge decks exposed to de-icing agents Concrete surfaces exposed to direct spray containing de-icing agents and freezing Splash zone of marine structures exposed to freezing
6. Chemical attack		
XA1	Slightly aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water
XA2	Moderately aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water
XA3	Highly aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water

In the Concrete settings the user can choose the desired exposure class. All items with a blue background colour can be overwritten in the 1D member data.





Methods of verification (art 4.4)

Concrete cover (art 4.4.1)

General (art 4.4.1.1)

The concrete cover is the distance between the surface of the reinforcement closest to the nearest concrete surface (including links and stirrups and surface reinforcement where relevant) and the nearest concrete surface.

The nominal cover shall be specified on the drawings. It is defined as a minimum cover, c_{min}, plus an allowance in design for deviation, Δc_{dev}:

$$c_{nom} = c_{min} + \Delta c_{dev} \qquad (4.1)$$

Minimum cover, c_{min} (art 4.4.1.2)

Minimum concrete cover, c_{min}, shall be provided in order to ensure:

- the safe transmission of bond forces
- the protection of the steel against corrosion (durability)
- an adequate fire resistance

The greater value for c_{min} satisfying the requirements for both bond and environmental conditions shall be used:

$$c_{min} = max \{c_{min,b}; c_{min,dur} + \Delta c_{dur,\gamma} - \Delta c_{dur,st} - \Delta c_{dur,add}; 10 \text{ mm}\} \quad (4.2)$$

where:

$c_{min,b}$ minimum cover due to bond requirement
$c_{min,dur}$ minimum cover due to environmental conditions
$\Delta c_{dur,\gamma}$ additive safety element
$\Delta c_{dur,st}$ reduction of minimum cover for use of stainless steel
$\Delta c_{dur,add}$ reduction of minimum cover for use of additional protection

The recommended value of $\Delta c_{dur,\gamma}$, $\Delta c_{dur,st}$ and $\Delta c_{dur,add}$, without further specification, is 0 mm.

- In order to transmit bond forces safely and to ensure adequate compaction of the concrete, the minimum cover should not be less than $c_{min,b}$ given in table 4.2.

Table 4.2: Minimum cover, $c_{min,b}$, requirements with regard to bond

Bond Requirement	
Arrangement of bars	Minimum cover $c_{min,b}$*
Separated	Diameter of bar
Bundled	Equivalent diameter (ϕ_n)(see 8.9.1)
*: If the nominal maximum aggregate size is greater than 32 mm, $c_{min,b}$ should be increased by 5 mm.	

- The minimum cover values for reinforcement and prestressing tendons in normal weight concrete taking account of the exposure classes and the structural classes is given by $c_{min,dur}$.

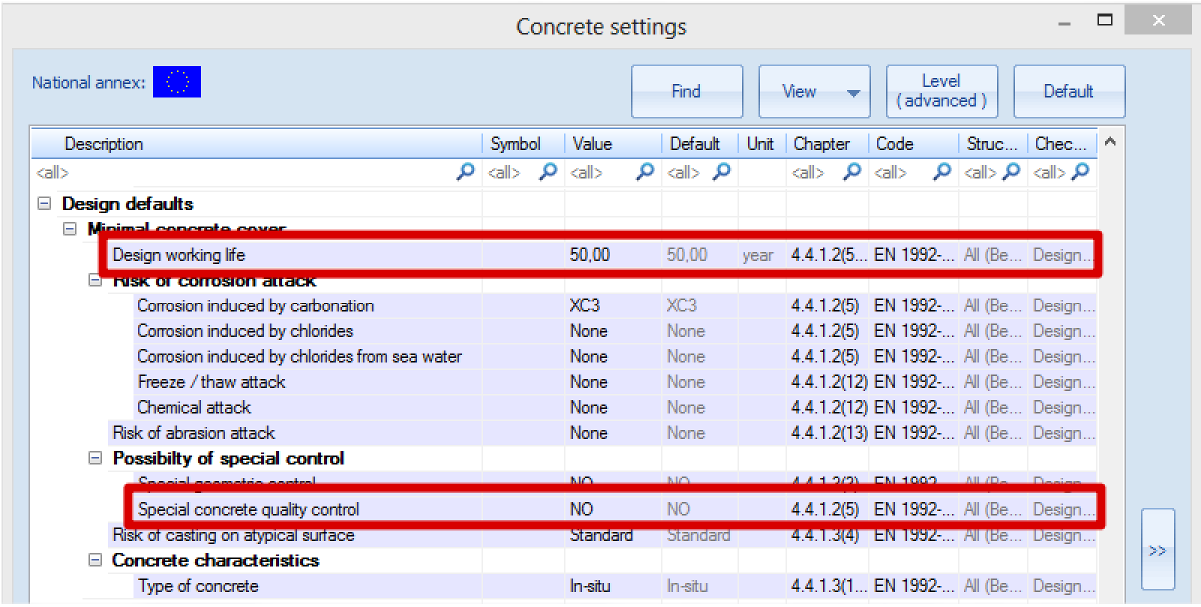
The recommended Structural Class (design working life of 50 years) is S4 for the indicative concrete strengths (given in Annex E of EN 1992-1-1). The recommended minimum Structural Class is S1.

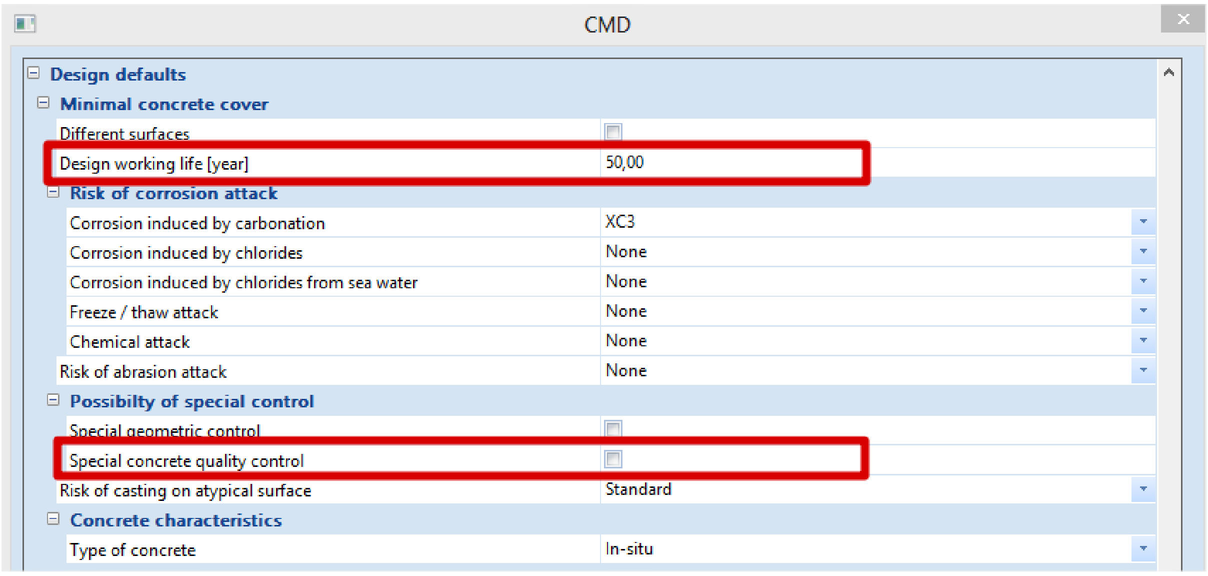
The recommended modifications to the structural class is given in Table 4.3N:

Table 4.3N: Recommended structural classification

Structural Class							
Criterion	Exposure Class according to Table 4.1						
	X0	XC1	XC2 / XC3	XC4	XD1	XD2 / XS1	XD3 / XS2 / XS3
Design Working Life of 100 years	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2
Strength Class [1) 2)]	≥ C30/37 reduce class by 1	≥ C30/37 reduce class by 1	≥ C35/45 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C45/55 reduce class by 1
Member with slab geometry (position of reinforcement not affected by construction process)	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1
Special Quality Control of the concrete production ensured	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1

The design working life and the special quality control can be defined in the concrete settings or in the 1D member data:





The recommended values of $c_{min,dur}$ are given in Table 4.4N (reinforcing steel):

Table 4.4N: Values of minimum cover, $c_{min,dur}$, requirements with regard to durability for reinforcement steel in accordance with EN 10080.

Environmental Requirement for $c_{min,dur}$ (mm)							
Structural Class	Exposure Class according to Table 4.1						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

- The concrete cover should be increased by the additive safety element $\Delta c_{dur,\gamma}$.

Where stainless steel is used or where other special measures have been taken, the minimum cover may be reduced by $\Delta c_{dur,st}$. For such situations the effects on all relevant material properties should be considered, including bond.

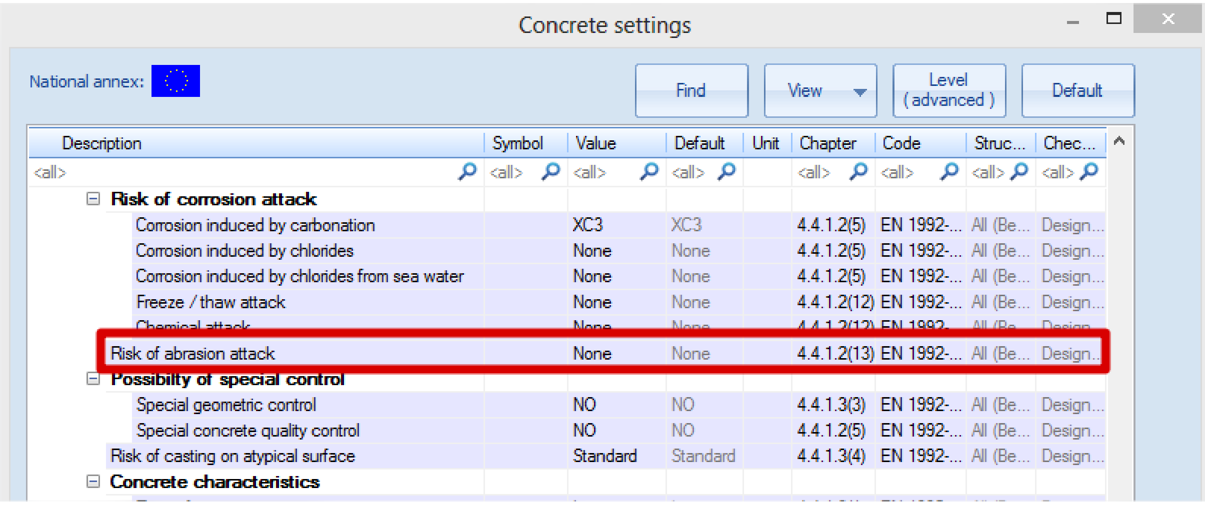
For concrete with additional protection (e.g. coating) the minimum cover may be reduced by $\Delta c_{dur,add}$.

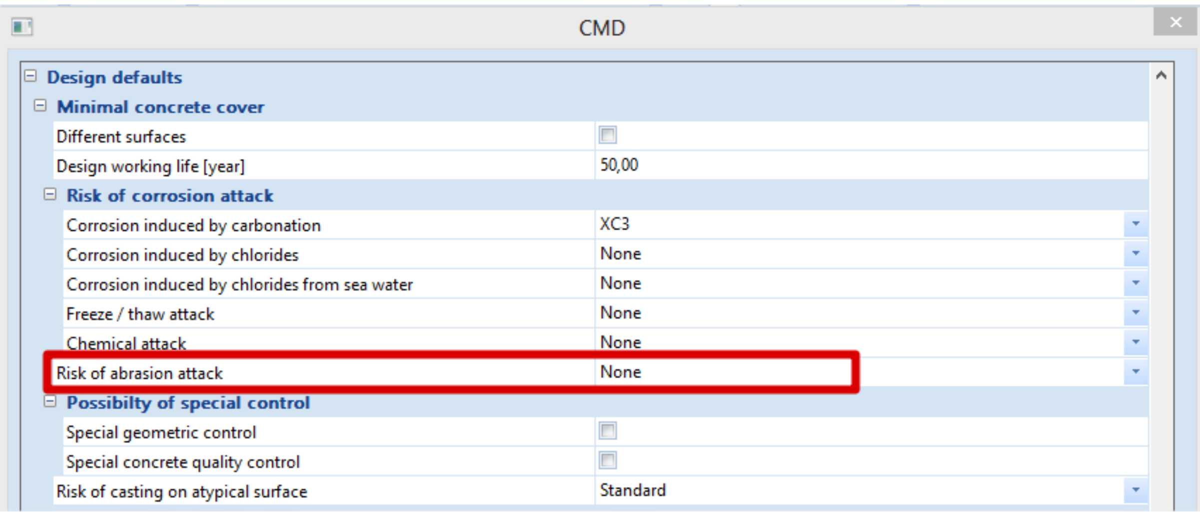
For concrete abrasion special attention should be given on the aggregate. Optionally concrete abrasion may be allowed for by increasing the concrete cover (sacrificial layer). In that case, the minimum cover c_{min} should be increased by k_1 for Abrasion Class XM1, by k_2 for XM2 and by k_3 for XM3.

Abrasion Class XM1 means a moderate abrasion like for members of industrial sites frequented by vehicles with air tyres. Abrasion Class XM2 means a heavy abrasion like for members of industrial sites frequented by fork lifts with air or solid rubber tyres. Abrasion Class XM3 means an extreme abrasion like for members industrial sites frequented by fork lifts with elastomer or steel tyres or track vehicles.

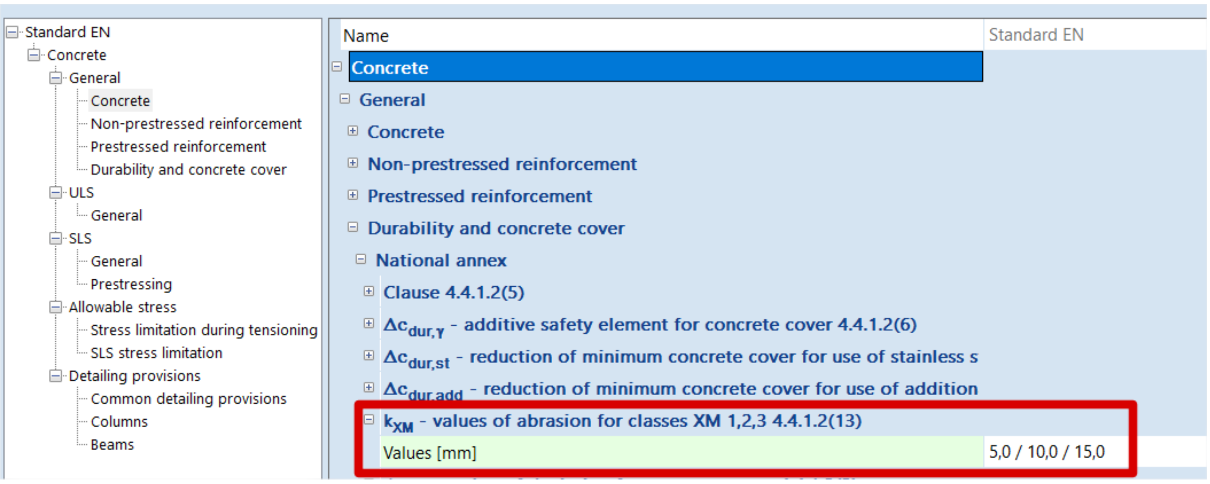
The recommended values of k_1, k_2 and k_3 are respectively: 5 mm, 10 mm and 15 mm.

The abrasion class can be inputted in the concrete settings or the 1D member data:





The values of k_1, k_2 and k_3 are available in the National Annex:



Allowance in design for deviation (art 4.4.1.3)

To calculate the nominal cover, c_{nom}, an addition to the minimum cover shall be made in design to allow for the deviation (Δc_{dev}). The required minimum cover shall be increased by the absolute value of the accepted negative deviation.

The recommended value of Δc_{dev} is 10 mm.

In certain situations, the accepted deviation and hence allowance, Δc_{dev}, may be reduced.

The recommended values are:

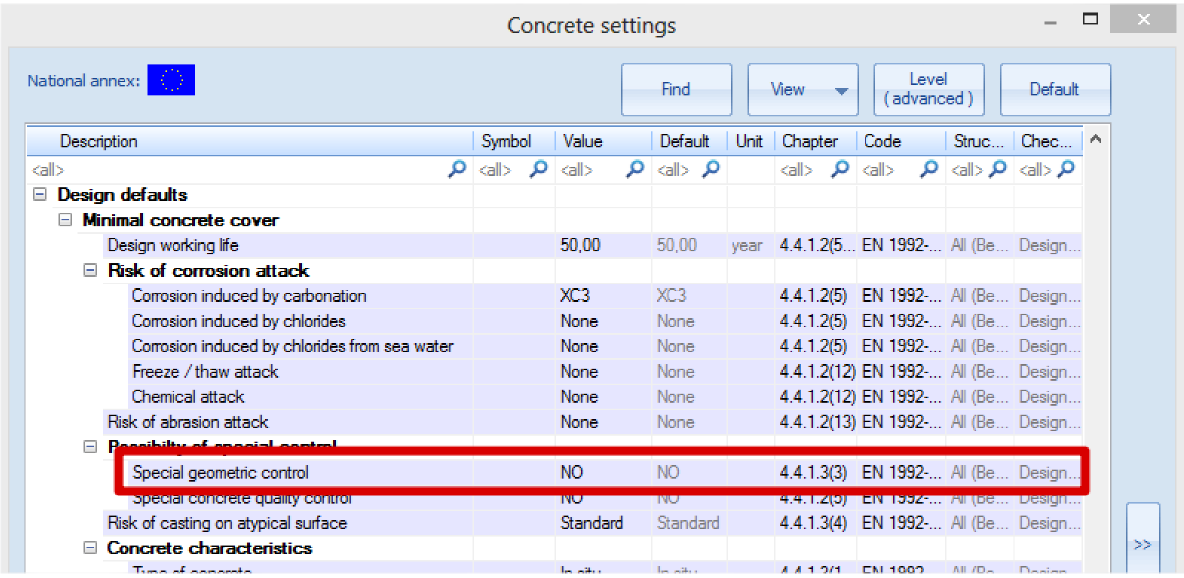
- *where fabrication is subjected to a quality assurance system, in which the monitoring includes measurements of the concrete cover, the allowance in design for deviation Δc_{dev} may be reduced:*

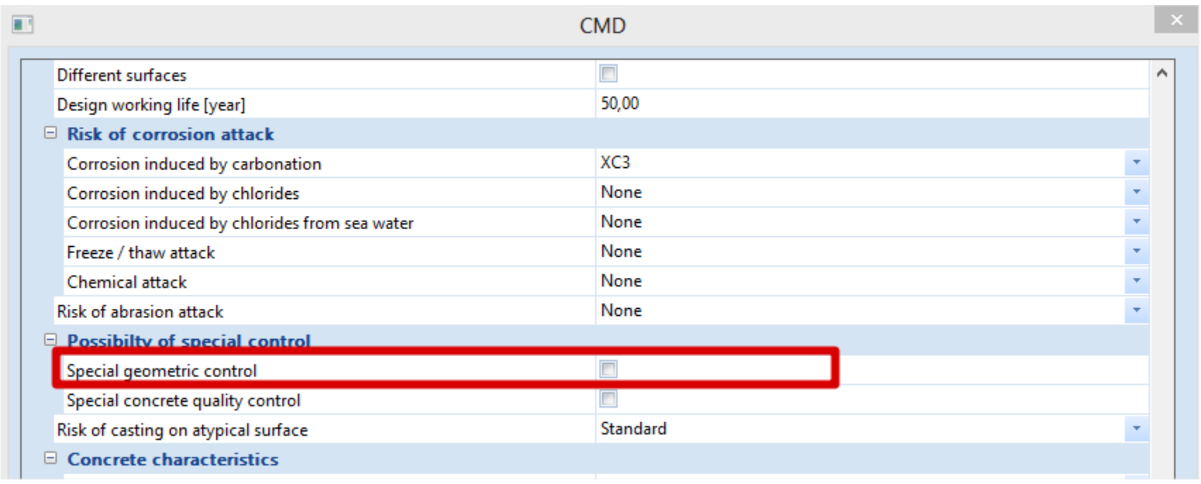
  $10 \text{ mm} \geq \Delta c_{dev} \geq 5 \text{ mm}$

- *where it can be assured that a very accurate measurement device is used for monitoring and non conforming members are rejected (e.g. precast elements), the allowance in design for deviation Δc_{dev} may be reduced:*

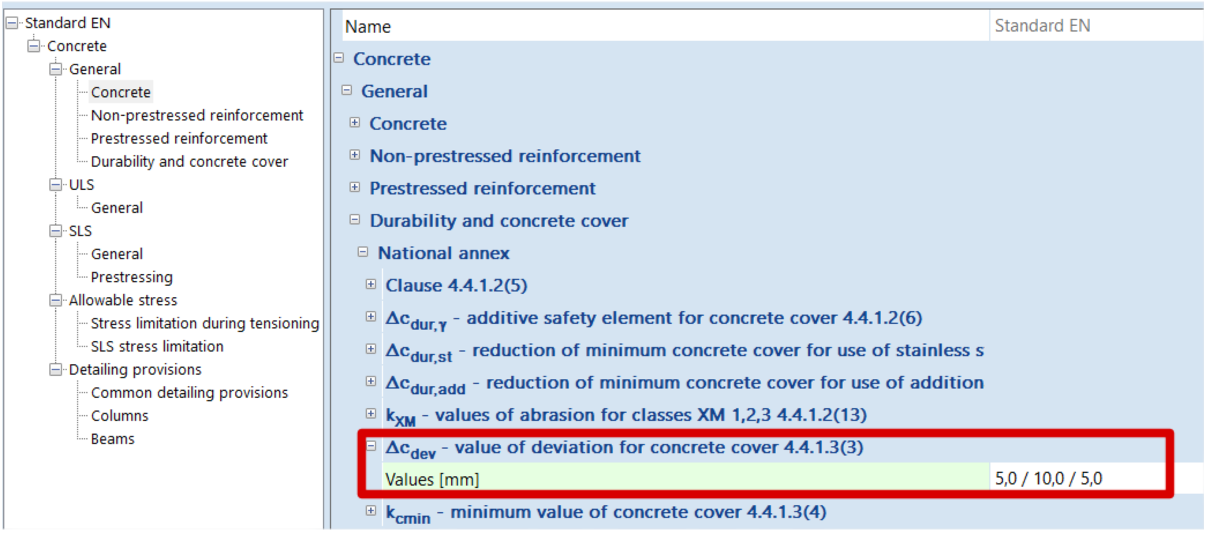
  $10 \text{ mm} \geq \Delta c_{dev} \geq 0 \text{ mm}$

The special geometric control can be checked in the concrete settings or the 1D member data:





The values of Δc_{dev} can be found in the National Annex:



Analysis models

Eurocode

Structural models for overall analysis (art 5.3.1)

The elements of a structure are classified, by consideration of their nature and function, as beams, columns, slabs, walls, plates, arches, shells etc. Rules are provided for the analysis of the commoner of these elements and of structures consisting of combinations of these elements.

For buildings the following provisions are applicable:

1) A beam is a member for which the span is not less than 3 times the overall section depth. Otherwise it should be considered as a deep beam.

2) A slab is a member for which the minimum panel dimension is not less than 5 times the overall slab thickness.

3) A slab subjected to dominantly uniformly distributed loads may be considered to be one way spanning if either:
   - it possesses two free (unsupported) and sensibly parallel edges.
   - it is the central part of a sensibly rectangular slab supported on four edges with a ratio of the longer to shorter span greater than 2.

4) Ribbed or waffle slabs need not be treated as discrete elements for the purposes of analysis, provided that the flange or structural topping and transverse ribs have sufficient torsional stiffness.

   This may be assumed provided that:
   - the rib spacing does not exceed 1500 mm
   - the depth of the rib below the flange does not exceed 4 times its width.
   - the depth of the flange is at least 1/10 of the clear distance between ribs or 50 mm, whichever is the greater.
   - transverse ribs are provided at a clear spacing not exceeding 10 times the overall depth of the slab.

The minimum flange thickness of 50 mm may be reduced to 40 mm where permanent blocks are incorporated between the ribs.

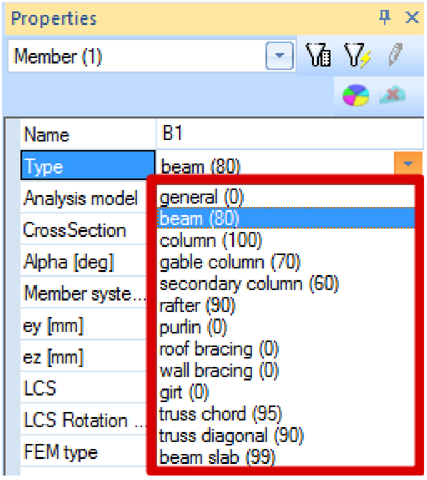
5) A column is a member for which the section depth does not exceed 4 times its width and the height is at least 3 times the section depth. Otherwise it should be considered as a wall.

SCIA Engineer

Assignment of analysis model

In SCIA Engineer several types of analysis models are available. It is up to the user to decide which model should be used for which element.

For 1D members, there is the choice between Beam, Beam slab and Column calculation.
Each element has a property ‘Type’ assigned to it, to determine which type of calculation will be used:

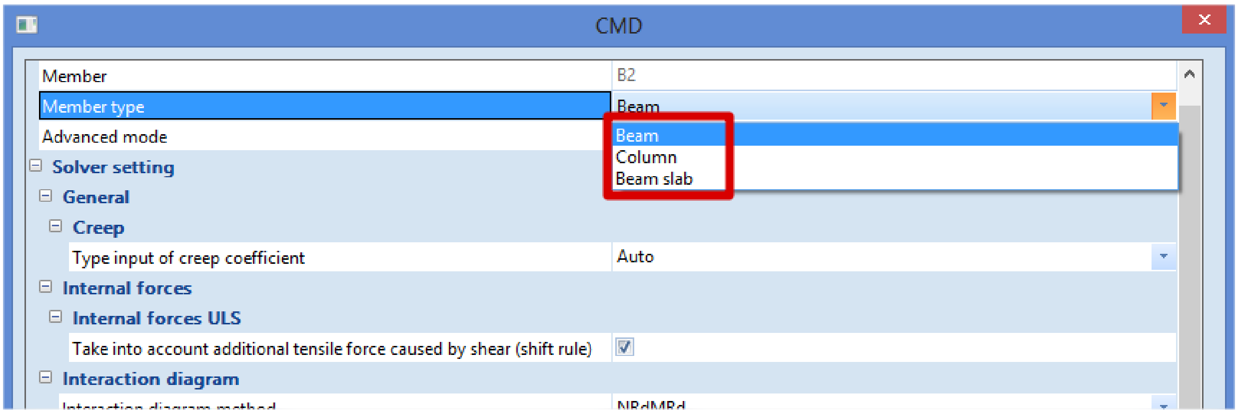


The Beam calculation is used for the Types ‘General’, ‘Beam, ‘Rafter’, ‘Purlin’, ‘Roof bracing’, ‘Wall bracing’, ‘Girt’, ‘Truss chord’ and ‘Truss diagonal’.

The Beam slab calculation is used only for the Type ‘Beam slab’. For this type, by default no shear reinforcement is added (unless necessary in case of a slab thickness of 200 mm or more, as defined in the Concrete Setup for slabs). As diameter for the longitudinal reinforcement, the default diameter for 2D structures – and not for beams! – is taken from the Concrete Setup.

The Column calculation is used for the Types ‘Column’, ‘Gable column’ and ‘Secondary column’.

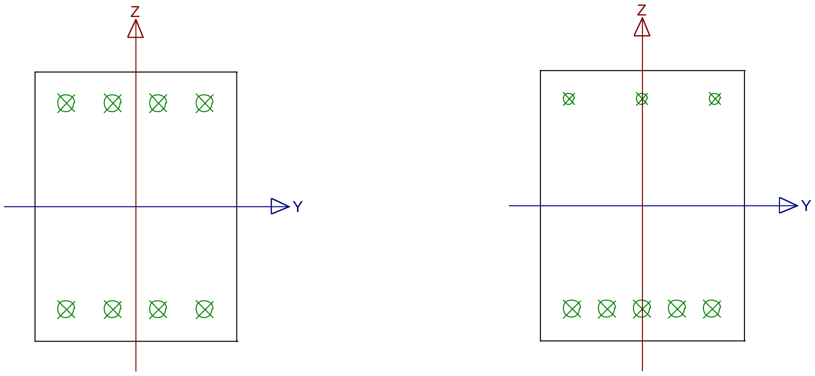
Be careful when 1D member data are added to an element, via Concrete menu > Setting per member > 1D member data. Also there, the user has the choice for the 3 different analysis models, by means of the option “Member type”:



These 1D member data *overwrite* both the element properties and the default settings in the Concrete settings.

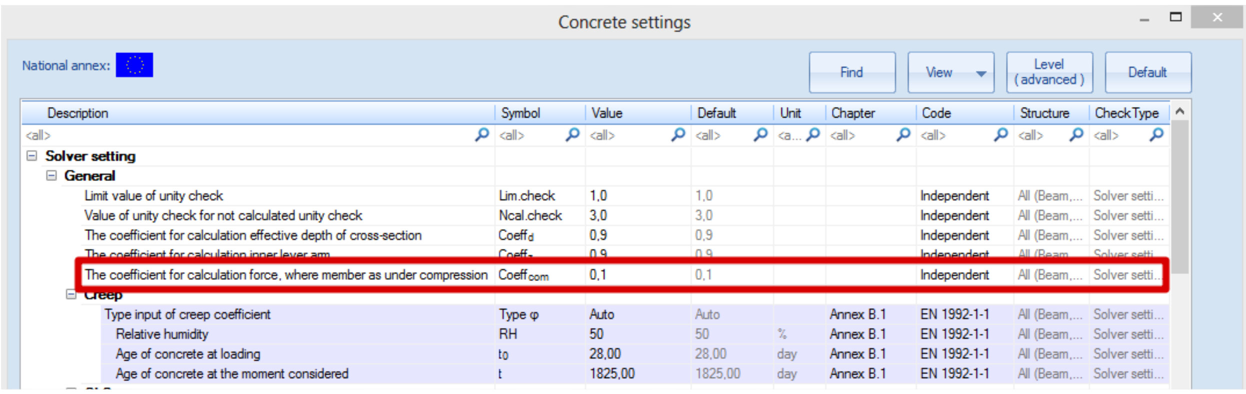
Difference between Beam and Column analysis model

The most important difference between Beam and Column calculation is the difference in reinforcement area per direction. A beam has an upper reinforcement area that differs from the lower reinforcement area. A column always has the same reinforcement configuration for the parallel sides, per direction.



These configurations are obvious, and caused by the difference in dominant internal forces per calculation type. For a beam calculation the bending moment is dominant, while for a column calculation the axial compression force + bending moments (if present).

So in fact, when the axial pressure on a beam is too high, the user should choose to calculate the element as a column. In the concrete settings an option is available to consider if the member is in compression or not. If the member is compressed, the second order effect is taken into account. Go to Concrete > Concrete settings (structure) > Solver setting > General:

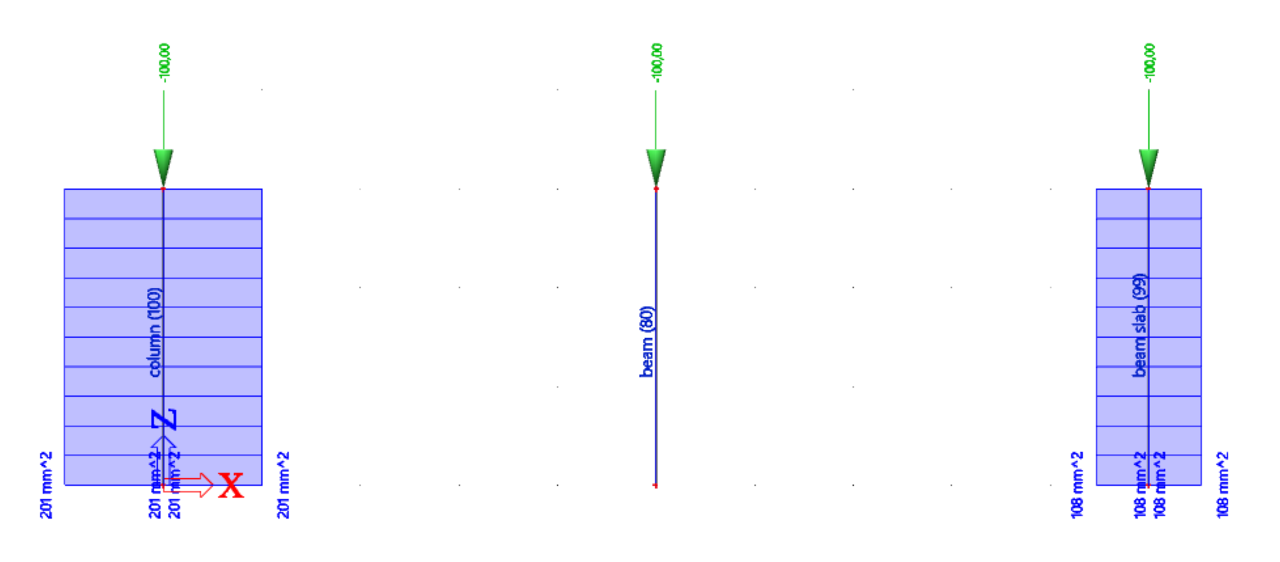


This option 'The coefficient for calculation force, where member as under compression' will check how important the contribution of the axial compression force is:

- If the axial compression load $N_{Ed} < 0{,}1 \cdot A_c \cdot f_{cd}$, the member is not considered to be in compression, which means the type 'Beam' is the right choice.

- If the axial compression load $N_{Ed} > 0{,}1 \cdot A_c \cdot f_{cd}$, the member is considered to be in compression, which means the beam has to be modelled as type 'Column' and the second order effect will be taken into account.

Example



Overall Design (ULS)

Linear calculation
Load case: LC2
Coordinate system: Member
Extreme 1D: Member
Selection: All

Longitudinal required reinforcement

Name	dx [m]	Case	Member	A_{sz} req+ [mm²] A_{sz} req bar+ [mm²]	A_{sz} req- [mm²] A_{sz} req bar- [mm²]	A_{sy} req+ [mm²] A_{sy} req bar+ [mm²]	A_{sy} req- [mm²] A_{sy} req bar- [mm²]	A_{sz} req [mm²] A_{sz} req bar [mm²]	A_{sy} req [mm²] A_{sy} req bar [mm²]	A_s req [mm²] A_s req bar [mm²]	ReinfReq
B1	0,000	LC2	Column	**201** **201**	**201** **201**	**201** **201**	**201** **201**	**402** **402**	**402** **402**	**804** **804**	**[z]4ϕ16*, [y]4ϕ16***
B2	0,000	LC2	Beam	**0** **0**	**0** **0**	**0** **0**	**0** **0**	**0** **0**	**0** **0**	**0** **0**	
B3	0,000	LC2	Beam slab	**108** **201**	**108** **201**	**108** **201**	**108** **201**	**215** **402**	**215** **402**	**430** **804**	**[z+]2ϕ16*, [z-]2ϕ16*, [y+]2ϕ16*, [y-]2ϕ16***

Shear reinforcement

Name	dx [m]	Case	Member	A_{swm} req [mm²/m]	A_{swm} prov [mm²/m]	ShearReinf
B1	0,000	LC2	Column	**0**	**0**	
B2	0,000	LC2	Beam	**0**	**0**	
B3	0,000	LC2	Beam slab	**0**	**0**	**Not required**

Under internal forces, a warning will be displayed in the detailed output whether it is necessary to calculate an element as column, to take into account the compression forces. If needed, the type has to be changed manually to column in the member properties or via 1D member data.

Compression member

Limit axial force to consider member as compression:

$$N_{com} = -Coeff_{com} \cdot \left(f_{cd} \cdot A_c\right) = -0.1 \cdot \left(6.4 \cdot 10^6 \cdot 0.09\right) = -57.6 \text{ kN}$$

Check condition:

$N_{Ed} < N_{com} = -100 \text{ kN} < -58 \text{ kN}$ ... compression member

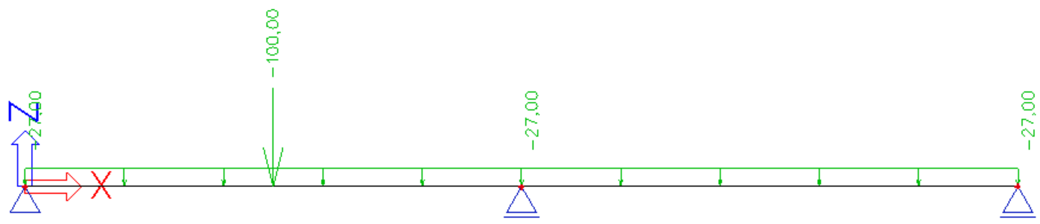
> Warning: First and second order eccentricities should be taken to account, member should be evaluated as column (significant compressive normal force). Change type of member to Column.

Beam design

Description of used example

The example that will be used to explain reinforcement calculation in a beam is called 'beam.esa'.

The beam reinforcement calculation is explained by means of the following two span beam:



The length of the total beam is 10 m and it has a dimension of 500x300mm.

The inputted loads are:

1) BG1: self weight
2) BG2: permanent load
    - Line load: -27 kN/m
    - Point load: -100 kN at position x = 0.25
3) BG3: variable load
    - Line load: -15 kN/m
    - Point load: -150 kN at position x = 0

Recalculated internal forces

Reinforcement calculation in SCIA Engineer is based on recalculated internal forces. The pure internal forces calculated by the mechanical FEM calculation are transformed according to code regulation into 'recalculated internal forces' to design the reinforcement.

These recalculated internal forces can be viewed in the Concrete menu of SCIA Engineer.

Shifting of bending moments (art 9.2.1.3)

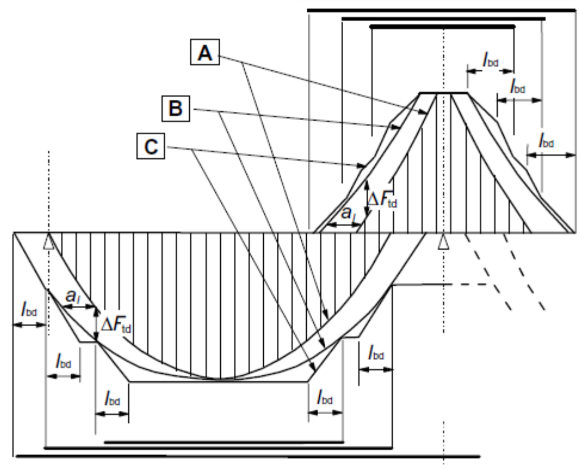
Sufficient reinforcement should be provided at all sections to resist the envelope of the acting tensile force, including the effect of inclined cracks in webs and flanges.

Additional tensile forces caused by shear and torsion are taken into account in SCIA Engineer by using the simplified calculation based on shifting of bending moments according to clause 9.2.1.3(2). Shifting of bending moments is calculated only for beams and beams as slab.

For members with shear reinforcement the additional tensile force, ΔF_{td}, should be calculated. For members without shear reinforcement, ΔF_{td} may be estimated by shifting the moment curve a distance a_l = d (for beams as slab). This "shift rule" may also be used as an alternative for members with shear reinforcement, where:

$$a_l = z\ (\cot \theta - \cot \alpha)/2 \qquad \text{(for beams)} \qquad (9.2)$$

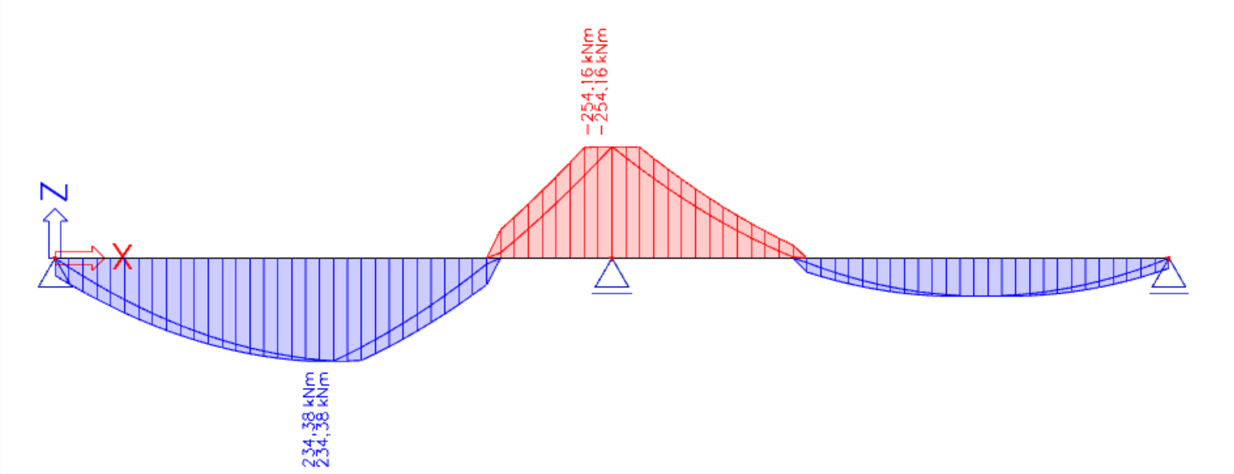
The additional tensile force is illustrated in Figure 9.2:



A - Envelope of $M_{Ed}/z + N_{Ed}$ B - acting tensile force F_s C - resisting tensile force F_{Rs}

Figure 9.2: Illustration of the curtailment of longitudinal reinforcement, taking into account the effect of inclined cracks and the resistance of reinforcement within anchorage lengths

In SCIA Engineer the user can review the recalculated internal forces. In the Concrete menu it is possible to view the internal forces and recalculated internal forces. In the figure below the difference is clearly visible:



The shifted moment line is taken into account for recalculated internal forces and by this also for the calculation of longitudinal reinforcement, if activated in the concrete settings (for the global structure) or in the 1D member data (individually per member):

Internal forces ULS						
Take into account additional tensile force caused by shear (shi...		☑	☑		9.2.1.3(2)	EN 1992-1-1
Use minimum value of eccentricity		☑	☑		6.1(4)	EN 1992-1-1
Use geometric imperfection		☑	☑		5.2(5)	EN 1992-1-1
Use second order effect		☑	☑		5.8.8	EN 1992-1-1
Estimation ratio of longitudinal reinforcement for recalculation in...	μ_s	2,00	2,00	%	5.8.3.1	EN 1992-1-1
Shear force reduction above supports		☐	☐		6.2.1(8)	EN 1992-1-1
Moment reduction above supports		☐	☐		5.3.2.2 (4)	EN 1992-1-1

Internal forces ULS	
Take into account additional tensile force caused by shear (shift rule)	☑
Shear force reduction above supports	☐
Moment reduction above supports	☐

Reduction of bending moment (art 5.3.2.2 (3) & 5.3.2.2 (4))

Another typical case of recalculated internal forces is the moment capping at supports.

Where a beam or slab is monolithic with its supports, the critical design moment at the support should be taken as that at the face of the support. The design moment and reaction transferred to the supporting element (e.g. column, wall, etc.) should be generally taken as the greater of the elastic or redistributed values.

Regardless of the method of analysis used, where a beam or slab is continuous over a support which may be considered to provide no restraint to rotation (e.g. over walls), the design support moment, calculated on the basis of a span equal to the centre-to-centre distance between supports, may be reduced by an amount ΔM_{Ed} as follows:

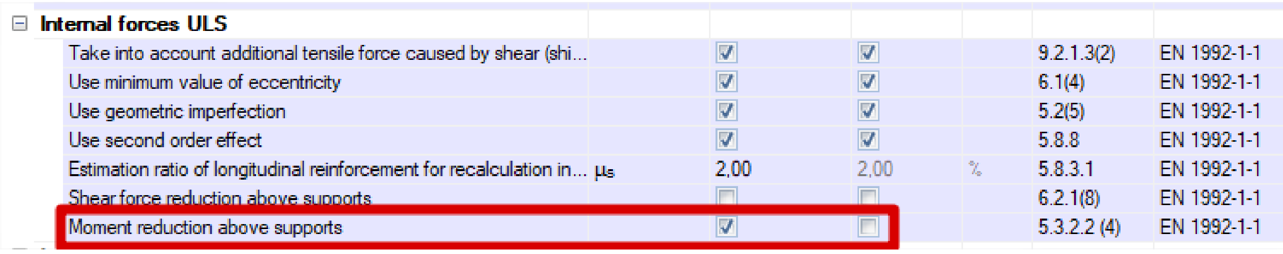
$$\Delta M_{Ed} = F_{Ed,sup}\, t / 8 \qquad (5.9)$$

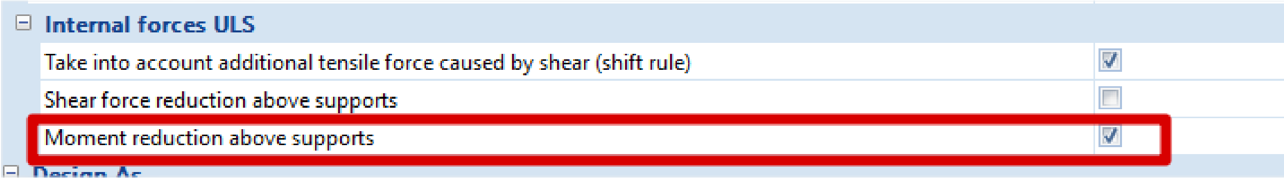
where:

$F_{Ed,sup}$ is the design support reaction

t is the width of the support

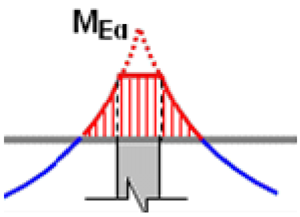
In SCIA Engineer this reduction of bending moment is only taken into account if it is activated in the concrete settings (for the global structure) or in the 1D member data (individually per member):



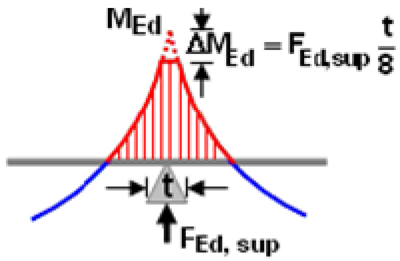


The way in which the moment reduction is performed, is based on the type of support. If a standard support is defined, the reduction will be done following formula 5.9. If a column is defined the, reduction at the face of the column is used.

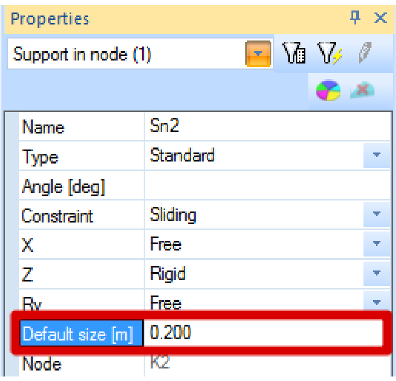
At the face of the column (5.3.2.2 (3))



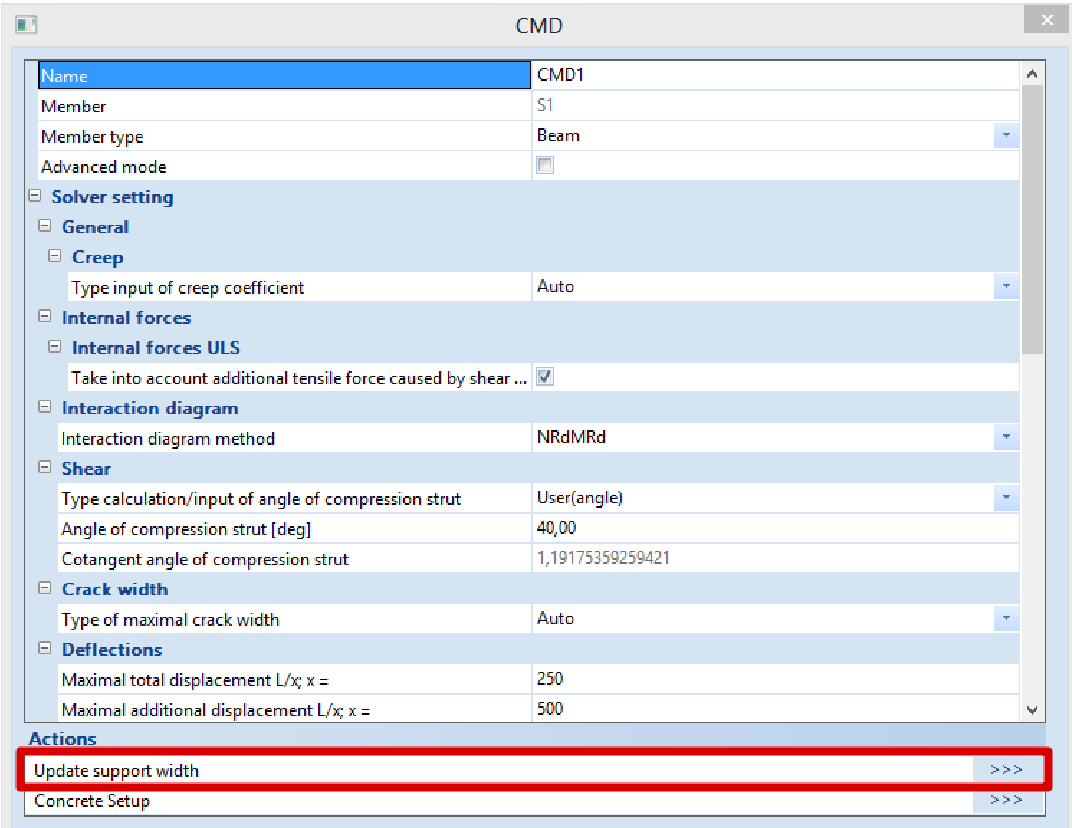
Using formula 5.9 (5.3.2.2 (4))

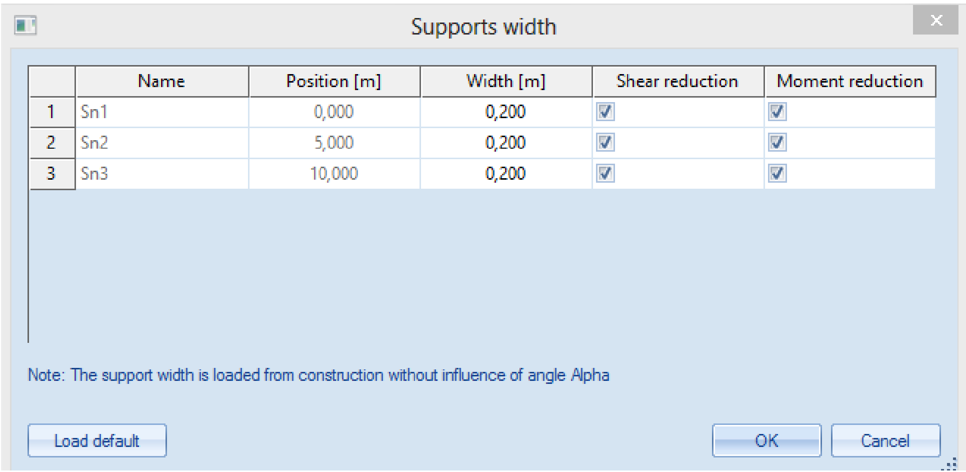


In SCIA Engineer the width t used for the moment reduction at supports can be set in the properties of that support:



In the bottom of the 1D member data there is an action button “Update support width”. This button collects all linked members or supports of the selected member and reads their support widths.

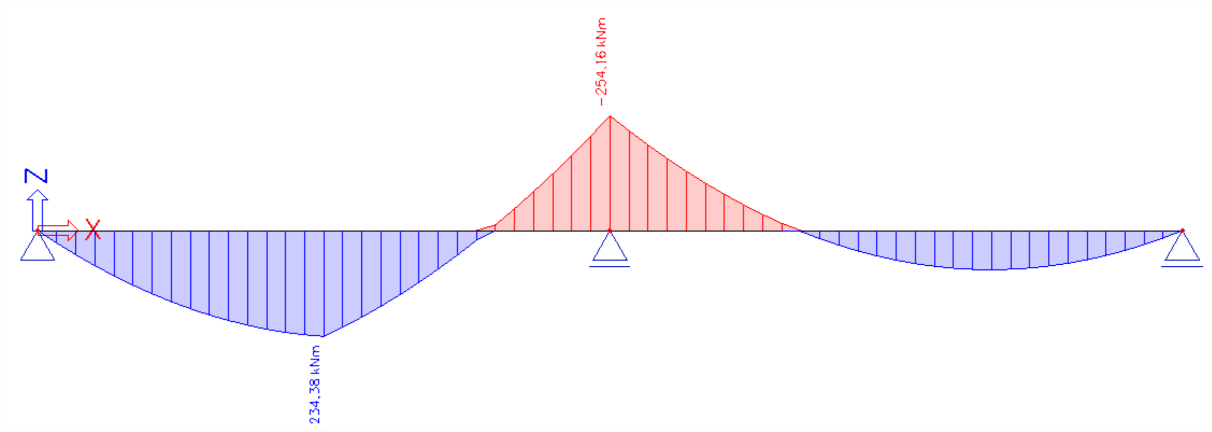




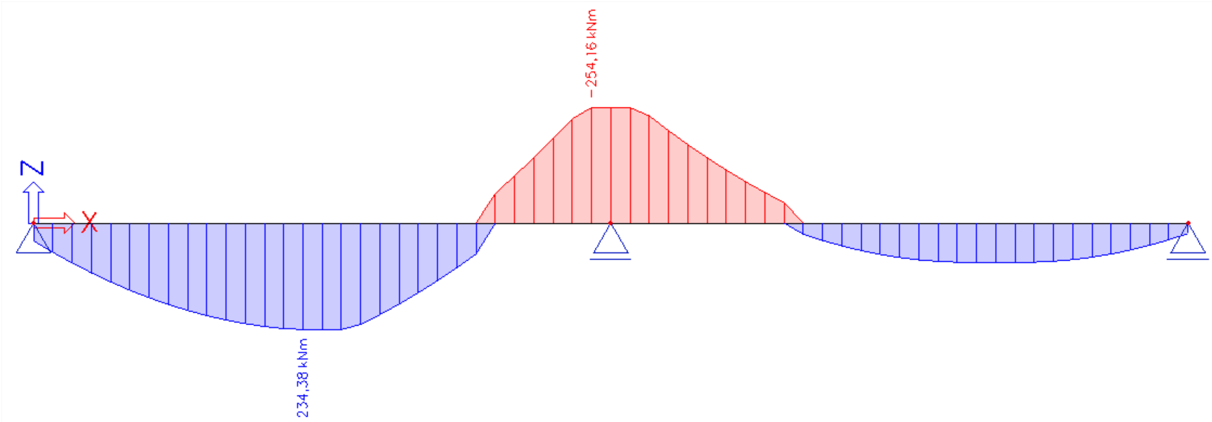
The reduction of moment by moment capping at supports is illustrated for our example below:

- $t = 0.2$ m
- $F_{Ed,sup} = 477.5$ kN
- $\Delta M_{Ed} = 477.5*0.2 / 8 = 11.94$ kNm

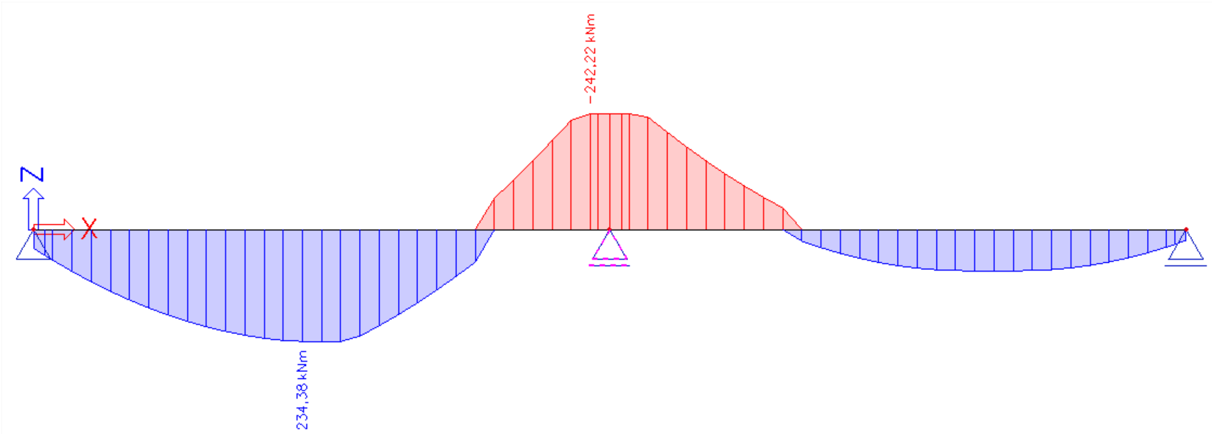
The original moment My at the support was 254.16 kNm.



The recalculated moment clearly shows the shifting of the moment line.



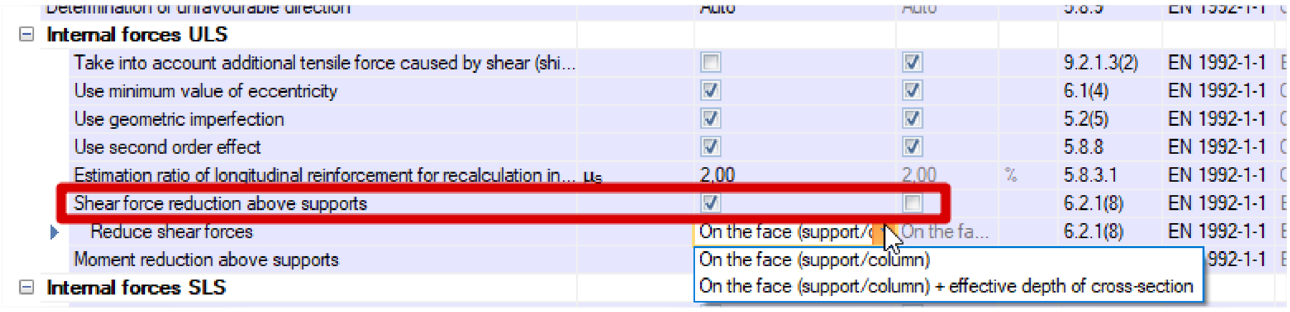
With moment capping at support taken into account the recalculated moment is 242.22 kNm.



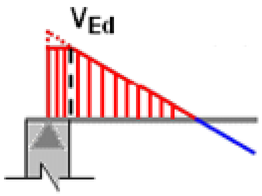
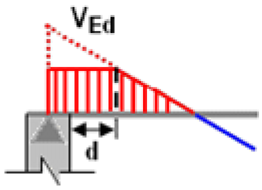
Reduction of shear forces (art 6.2.1 (8))

For members subject to predominantly uniformly distributed loading, the design shear force does not need to be checked at a distance less than d from the face of the support. Any shear reinforcement required should continue to the support. In addition it should be verified that the shear at the support does not exceed $V_{Rd,max}$.

In SCIA Engineer this reduction of shear forces is only taken into account if it is activated in the concrete settings (for the global structure) or in the 1D member data (individually per member):



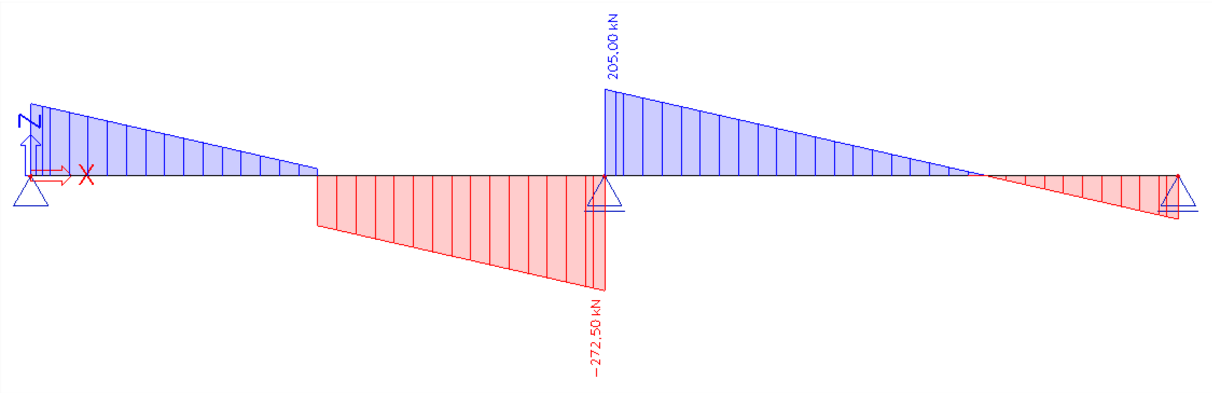
It is possible to choose the type of reduction of shear forces at the face of the support or at a distance d from the face of the support:

At the face of the column	**At the face of the column + effective depth of the cross-section (based on 6.2.1 (8))**
	

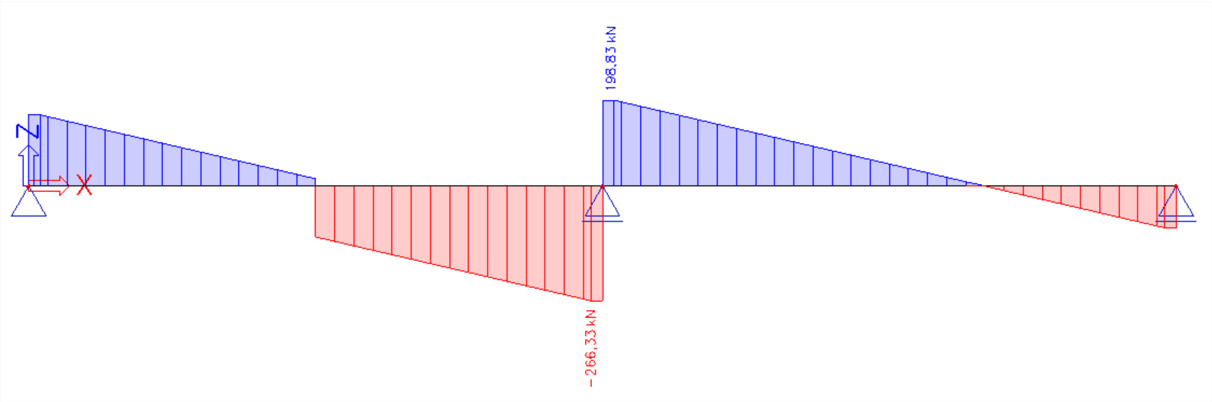
Also for the reduction of shear forces, the support width t is taken into account, which is taken from the properties of the support or the 1D member data.

The reduction of shear forces at supports is illustrated for our example below with t = 0,2 m:

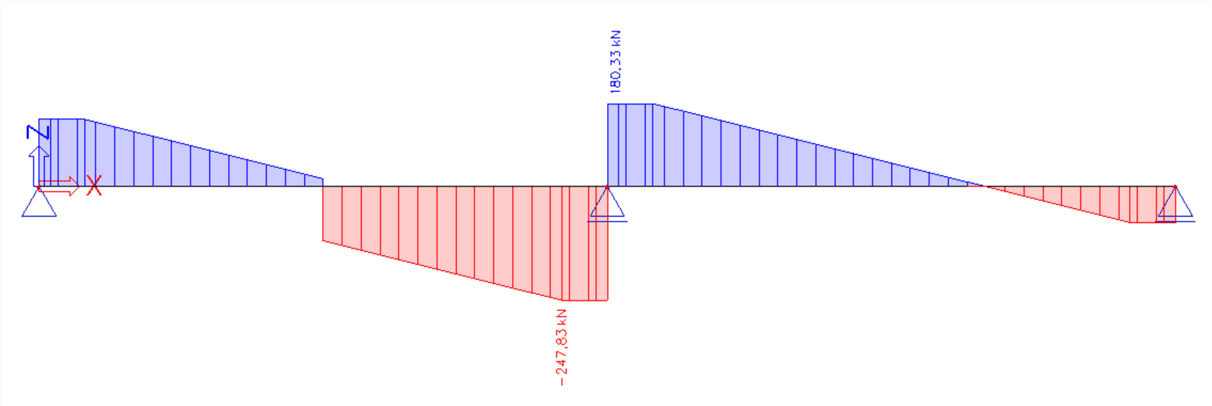
The first image displays the original V_z :



The second image shows the reduction at the face of the support:



The last image shows the reduction at the effective depth from the face:

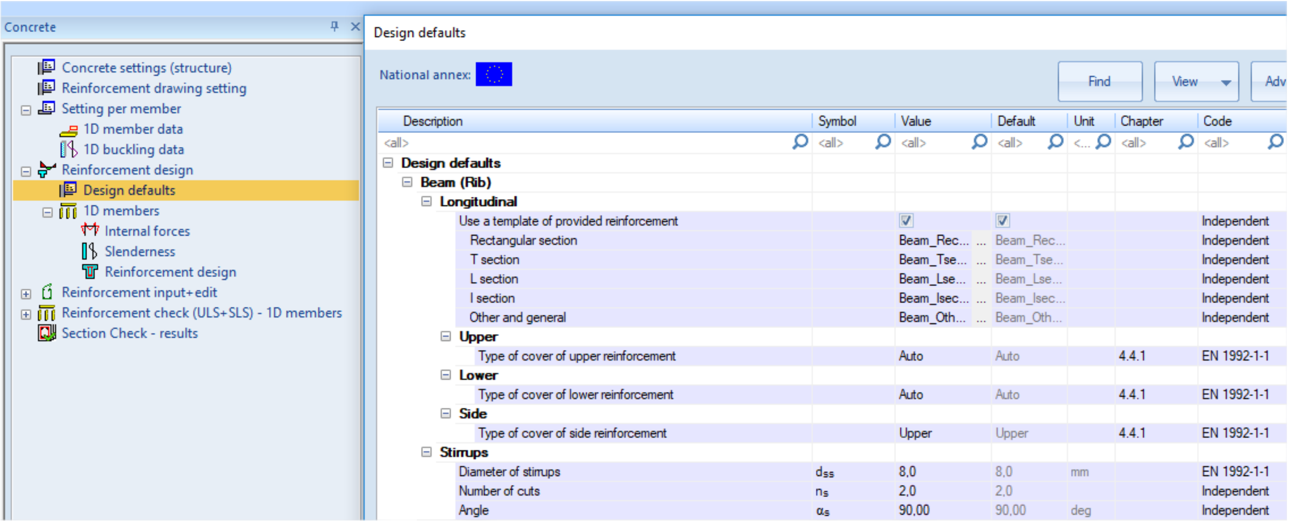


Theoretical reinforcement

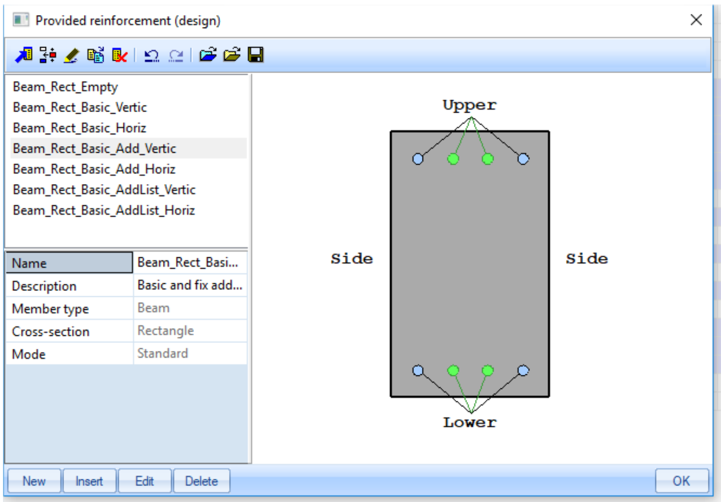
Configuration

The theoretical reinforcement is calculated out of the recalculated internal forces. It gives the amount of reinforcement needed to resist the internal forces induced by ULS loads. Since there are several workflows possible to design concrete beam elements, the theoretical reinforcement design is not mandatory to perform. Experienced users can directly jump to practical reinforcement to perform the checks on, but this theoretical approach gives a good idea of how this practical reinforcement should look like.

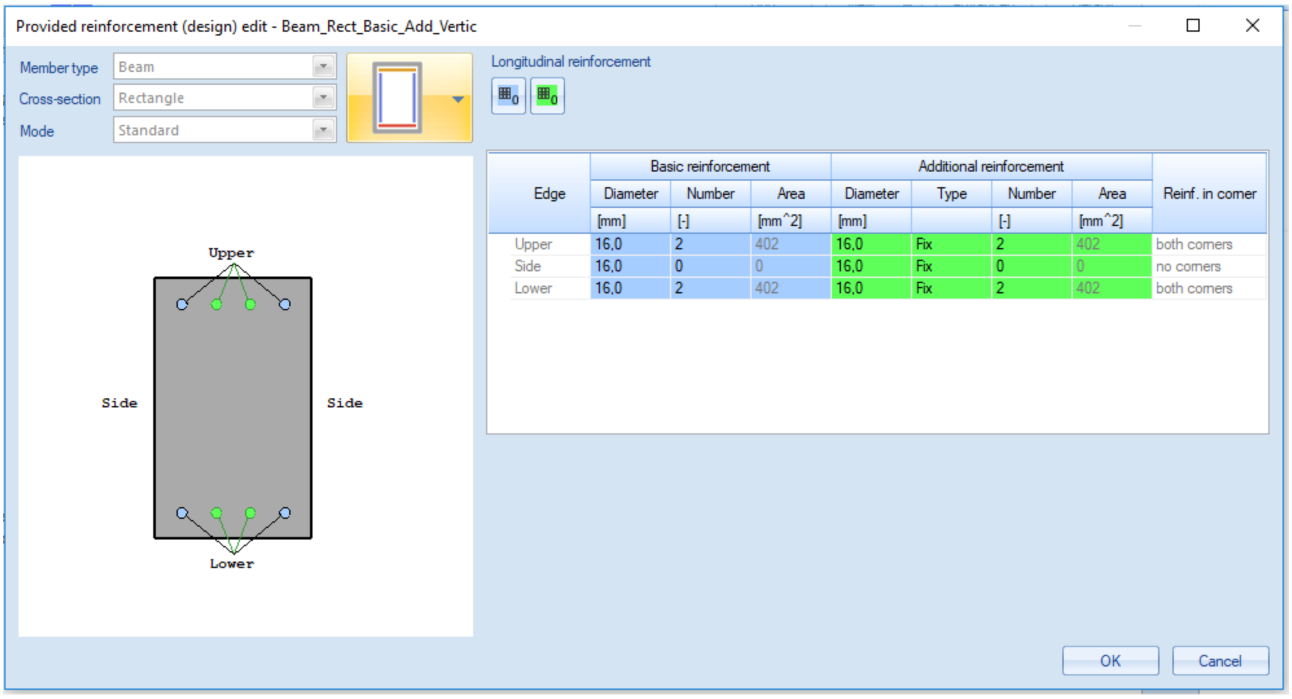
The configuration of this theoretical reinforcement can be found in the Design defaults under Reinforcement design. Templates of longitudinal reinforcement for different shapes of beam are available. The concrete cover can be set for upper, lower and side faces. For the stirrups, diameter, number of cuts and angle can be adapted. Note that for the practical reinforcement and checks, only vertical stirrups (α_s = 90°) are supported.



Several default templates for longitudinal reinforcement are available for the different section types. These can be adapted or new ones can be made.



This template exists of basic and additional reinforcement for the upper, lower and side faces. The exact use of these templates will be elaborated under next paragraph on longitudinal reinforcement. However, the purpose is to compare these templates with the required reinforcement, to model the longitudinal reinforcement that should be introduced later on. The basic reinforcement is present along the whole length of the beam; the additional reinforcement is present only at the zones where basic reinforcement is not sufficient to withstand (recalculated) internal forces. A choice can be made between fixed additional bars (diameter and number) or a list with different numbers of bars with a fixed diameter. SCIA Engineer uses the least amount of necessary additional bars, or places the maximum if this template is still not sufficient to resist the (recalculated) internal forces.



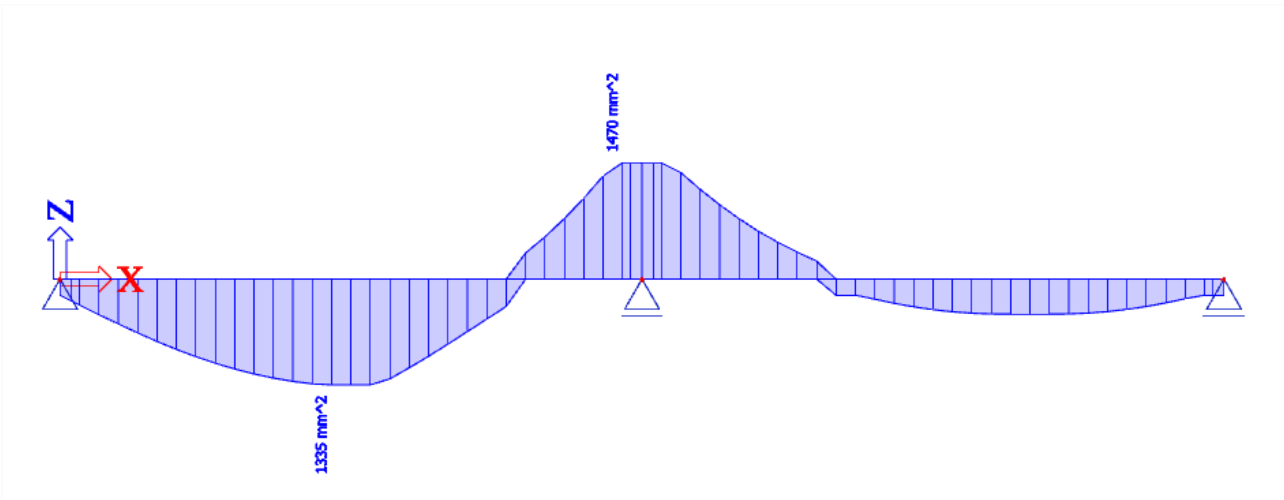
Calculation of longitudinal reinforcement As

The longitudinal reinforcement calculation is based on $M_{y,recalc}$ represented in the previous chapter.

The only thing left to be set in the concrete setup is the material quality and default diameter:

- Material quality is set to B 500A. This can be changed in the project data or concrete 1D member data.
- The default diameter is set to 16mm. This parameter is taken from the additional reinforcement diameter of the reinforcement template under Design defaults, or from 1D member data.

The following results are obtained with these settings:



In the following image you can see the brief output in the preview:

Longitudinal required reinforcement

Name	dx [m]	Case	Member	A_{sz_req+} [mm²] $A_{sz\_req\_bar+}$ [mm²]	A_{sz_req-} [mm²] $A_{sz\_req\_bar-}$ [mm²]	A_{sy_req+} [mm²] $A_{sy\_req\_bar+}$ [mm²]	A_{sy_req-} [mm²] $A_{sy\_req\_bar-}$ [mm²]	A_{sz_req} [mm²] $A_{sz\_req\_bar}$ [mm²]	A_{sy_req} [mm²] $A_{sy\_req\_bar}$ [mm²]	A_{s_req} [mm²] $A_{s\_req\_bar}$ [mm²]	ReinfReq
S1	2,333-	ULS	Beam	0 0	**1335** **1407**	**0** **0**	**0** **0**	1335 1407	**0** **0**	1335 1407	**[z-]7ϕ16**
S1	4,833-	ULS	Beam	**1470** **1608**	0 0	0 0	0 0	**1470** **1608**	0 0	**1470** **1608**	[z+]8ϕ16

You can also ask a standard or a more detailed output where you can find more information about certain parameters used in the calculation, for example:

d: lever arm of reinforcement.

$d = h - cover - \Phi_{stirrup} - \phi_{longitudinal\ beam}/2 = 500 - 35 - 8 - \mathbf{16}/2 = 449$ mm

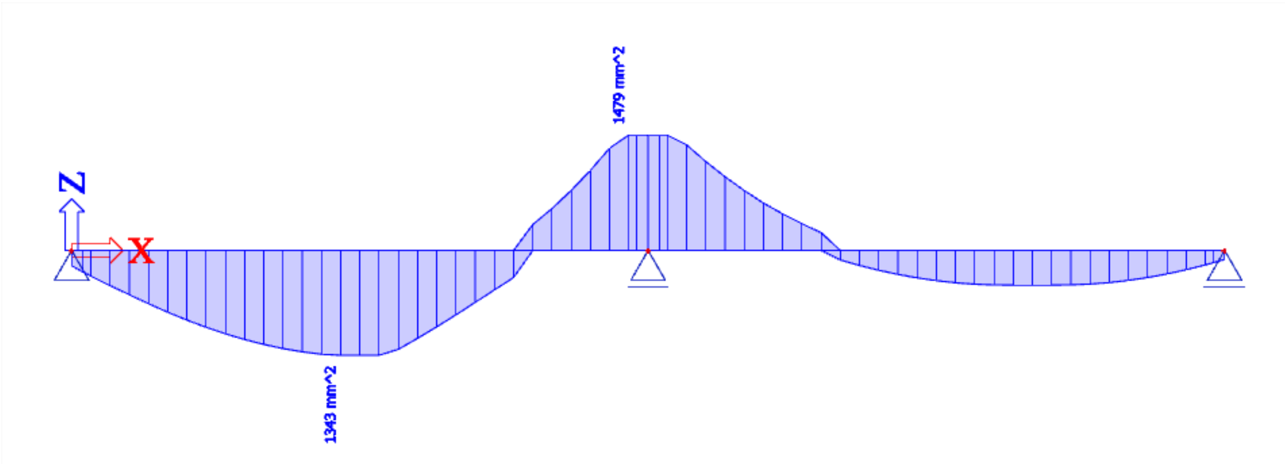
(the cover is defined by the environmental class and is 35 mm for XC3)

The only internal force working on this beam is M_{yd}. N_d and T_d are zero.

$A_{sy\_req} = 0$ because there is no torsion on this beam.

Note that the detailing provisions are deactivated. Otherwise no reinforcement of ϕ = 16mm could be proposed, since the detailing provisions are not met (bar distance too small).

If the default diameter is set to 20mm, the following results are obtained:



Longitudinal required reinforcement

Name	dx [m]	Case	Member	A_{sz_req+} [mm²] $A_{sz\_req\_bar+}$ [mm²]	A_{sz_req-} [mm²] $A_{sz\_req\_bar-}$ [mm²]	A_{sy_req+} [mm²] $A_{sy\_req\_bar+}$ [mm²]	A_{sy_req-} [mm²] $A_{sy\_req\_bar-}$ [mm²]	A_{sz_req} [mm²] $A_{sz\_req\_bar}$ [mm²]	A_{sy_req} [mm²] $A_{sy\_req\_bar}$ [mm²]	A_{s_req} [mm²] $A_{s\_req\_bar}$ [mm²]	ReinfReq
S1	2,333-	ULS	Beam	0 0	**1343** **1571**	**0** **0**	**0** **0**	1343 **1571**	**0** **0**	1343 **1571**	**[z-]5ϕ20**
S1	4,833-	ULS	Beam	**1479** **1571**	0 0	0 0	0 0	**1479** 1571	0 0	**1479** 1571	[z+]5ϕ20

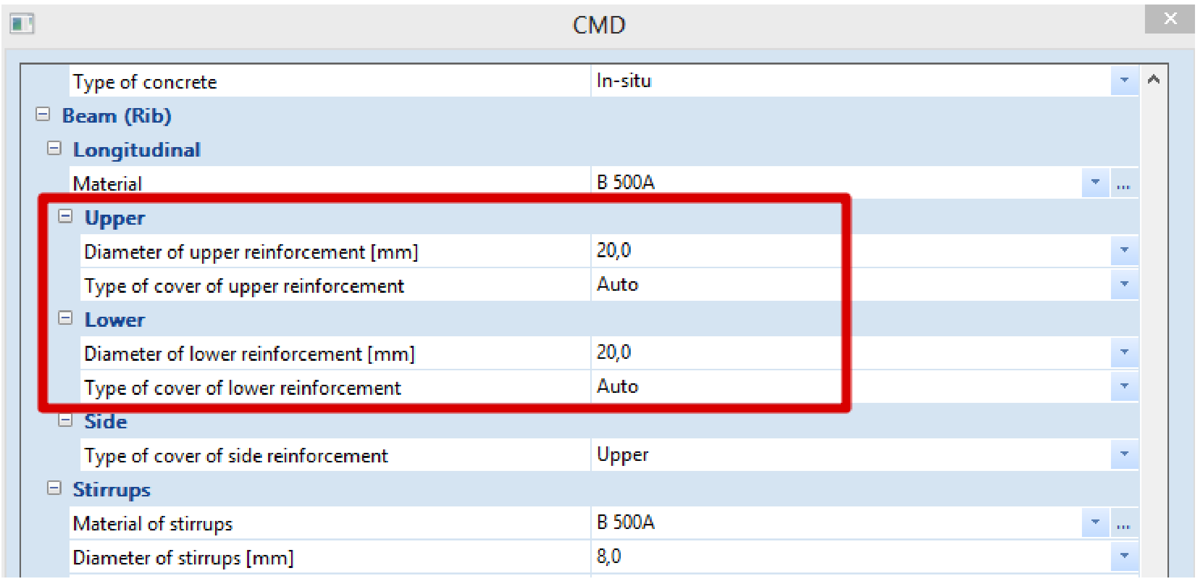
If you take a close look at these results, you can see that also the value for $A_{s,req}$ has changed.

This is because the lever arm d has decreased:

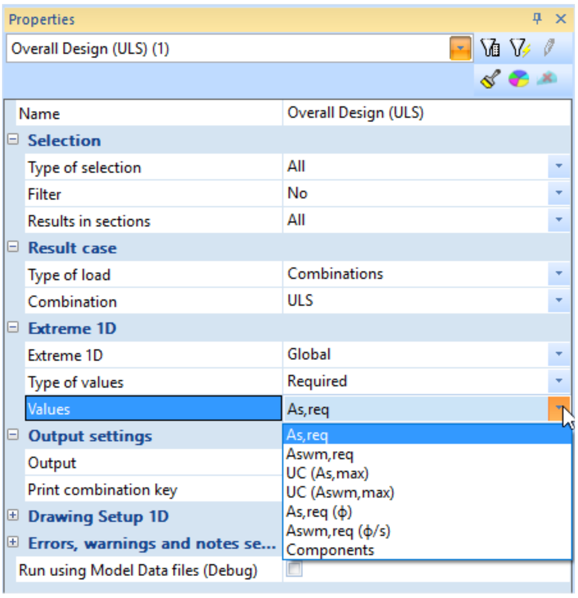
$d = h - cover - \Phi_{stirrup} - \Phi_{longitudinal\ beam}/2 = 500 - 35 - 8 - \mathbf{20}/2 = 447$ mm

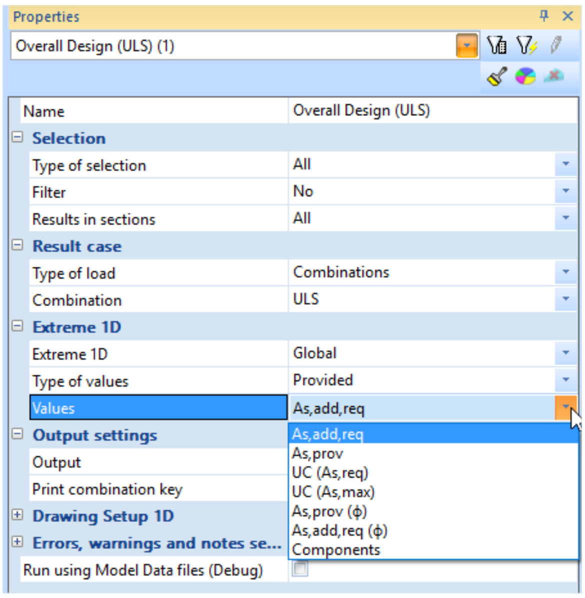
As you can see, the default diameter has also a slight effect on the amount of reinforcement that is required, because of the changed lever arm.

Note that 1D member data can be used to change the default diameter for the bar to which these data are assigned. It is obvious that the 1D member data have higher priority than the Concrete settings.



Next to the required reinforcement area, also the unity check UC can be viewed to check for maximum reinforcement area and $A_{s,req}$ (ϕ) for the reinforcement translated into bars.





The provided reinforcement $A_{s,prov}$ gives the amount of reinforcement or in bars ($A_{s,prov}$), determined by the template. $A_{s,add,\ req} = A_{s,req} - A_{s,prov}$, thus the amount of reinforcement which still has to be added to the template to resist the (recalculated) internal forces. If $A_{s,prov} > A_{s,req}$, $A_{s,add,\ req} = 0$.
Also unity checks can be performed on the provided reinforcement.

Calculation of shear reinforcement Aswm

Shear reinforcement

Name	dx [m]	Case	Member	A_{swm_req} [mm^2/m]	A_{swm_prov} [mm^2/m]	ShearReinf
S1	7,333-	ULS	Beam	**298**	**309**	**ϕ8/325mm, (ns=2)**
S1	4,900	ULS	Beam	**1315**	**1340**	**ϕ8/75mm, (ns=2)**

V_{Ed} = design shear force resulting from external loading
$V_{Rd,c}$ = design shear resistance of the member without shear reinforcement
$V_{Rd,s}$ = design value of the shear force which can be sustained by the yielding shear reinforcement
$V_{Rd,max}$ = design value of the maximum shear force which can be sustained by the member, limited by crushing of the compression struts

In general we can have three cases:

- $V_{Ed} > V_{Rd,max}$ — Concrete strut failure
- $V_{Ed} \leq V_{Rd,c}$ — Shear force carried by concrete. No shear reinforcement necessary (minimum shear reinforcement according to detailing provisions)
- $V_{Ed} > V_{Rd,c}$ and $V_{Ed} < V_{Rd,max}$ — Shear reinforcement necessary in order that: $V_{Ed} \leq V_{Rd}$

Members NOT requiring design shear reinforcement: VEd < VRd,c (art 6.2.2)

$$V_{Rd,c} = [C_{Rd,c}\, k(100\, \rho_l\, f_{ck})^{1/3} + k_1\, \sigma_{cp}]\, b_w\, d \qquad (6.2.a)$$

with a minimum of

$$V_{Rd,c} = (v_{min} + k_1\, \sigma_{cp})\, b_w\, d \qquad (6.2.b)$$

where:
f_{ck} = characteristic concrete compressive strength [MPa]
k = size factor: $k = 1 + \sqrt{(200/d)} \leq 2{,}0$ (with d in mm)
ρ_l = longitudinal reinforcement ratio: $\rho_l = A_{sl}/b_w d \leq 0{,}02$
b_w = smallest web width of the cross-section in the tensile area [mm]
σ_{cp} = concrete compressive stress due to loading: $\sigma_{cp} = N_{Ed}/A_c < 0{,}2\, f_{cd}$ [MPa]
d = effective height of cross section

The recommended value for $C_{Rd,c}$ is $0{,}18/\gamma_c$, that for k_1 is 0,15 and that for v_{min} is given by expression:

$$v_{min} = 0{,}035\, k^{3/2} .\, f_{ck}^{1/2} \qquad (6.3N)$$

The shear force V_{Ed}, calculated without reduction by β, should always satisfy the condition:

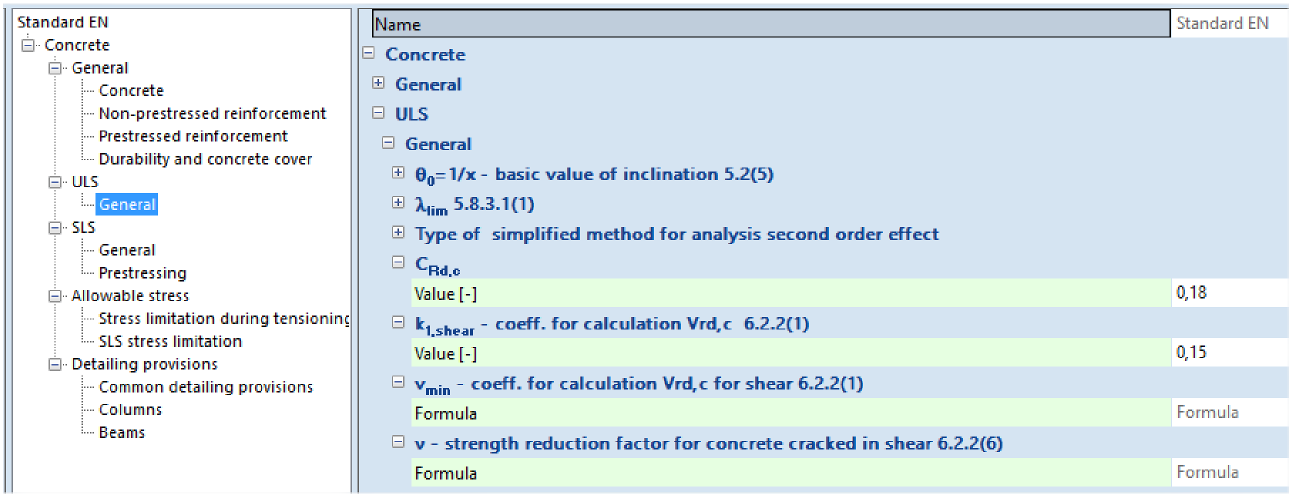
$$V_{Ed} \leq 0{,}5\, b_w\, d\, v\, f_{cd} \qquad (6.5)$$

where v is a strength reduction factor for concrete cracked in shear.

The recommended value for v follows from:

$$v = 0,6\left[1-\frac{f_{ck}}{250}\right] \qquad (6.6N)$$

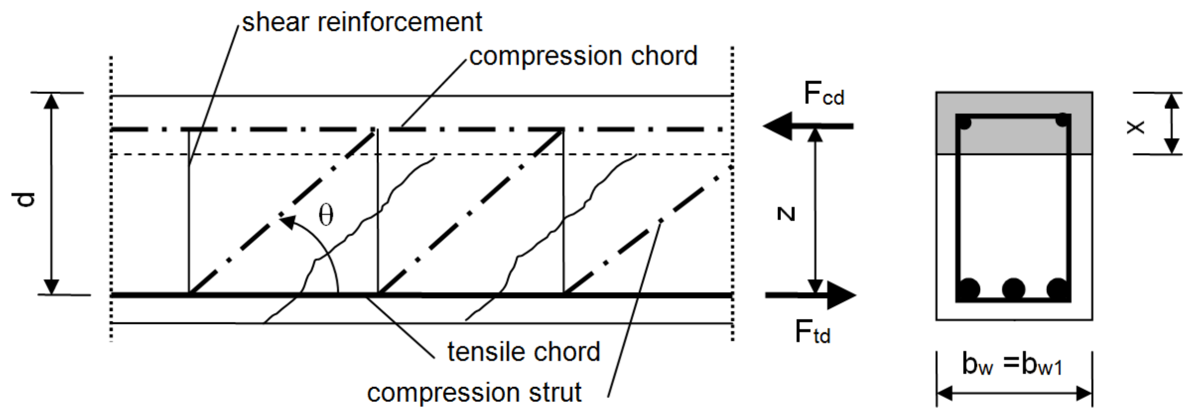
In SCIA Engineer, it is possible to input the following parameters:



Note that the green values are according to EN code.

Members requiring design shear reinforcement: VEd > VRd,c (art 6.2.3)

The design of members with shear reinforcement is based on the theory of the concrete truss-model. In this theory, a virtual truss-model is imagined in a concrete beam. This truss-model has a set of vertical (or slightly diagonal), horizontal and diagonal members. The vertical bars are considered to be the stirrups, the horizontal bars are the longitudinal reinforcement bars and the diagonal bars are the concrete struts.

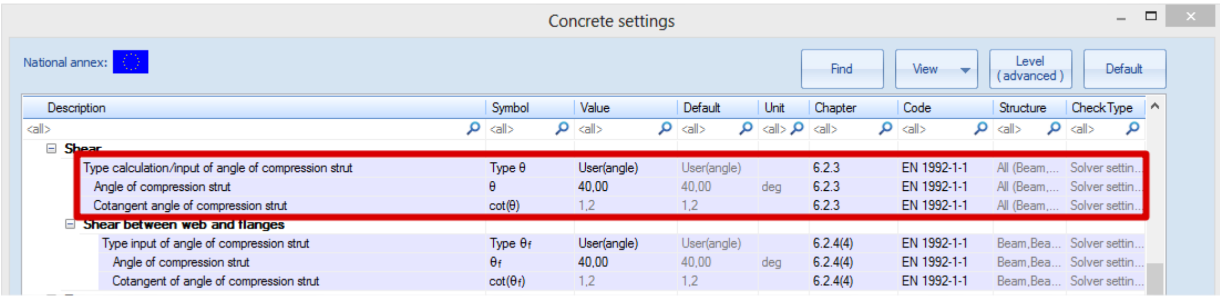


The angle θ should be limited.

The recommended limits of cot θ are given:

$$1 \leq \cot\theta \leq 2,5 \qquad (6.7N)$$

The angle θ can be inserted in SCIA Engineer:



For members with vertical shear reinforcement, the shear resistance V_{Rd} is the smaller value of:

$$V_{Rd,s} = \frac{A_{sw}}{s} z\, f_{ywd} \cot\theta \qquad (6.8)$$

and

$$V_{Rd,max} = \alpha_{cw}\, b_w\, z\, v_1\, f_{cd}/(\cot\theta + \tan\theta) \qquad (6.9)$$

where:

A_{sw} = cross-sectional area of the shear reinforcement
s = spacing of the stirrups
f_{ywd} = design yield strength of the shear reinforcement
v_1 = strength reduction factor for concrete cracked in shear
α_{cw} = coefficient taking account of the state of the stress in the compression chord

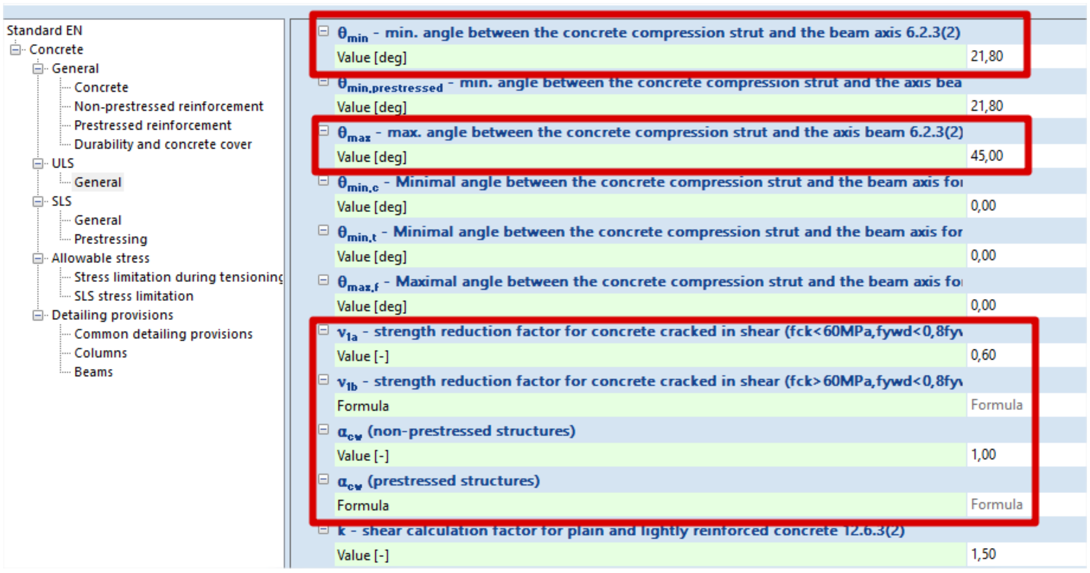
The recommended value of v_1 is v (see Expression 6.6N)

If the design stress of the shear reinforcement is below 80% of the characteristic yield stress f_{yk}, v_1 may be taken as:

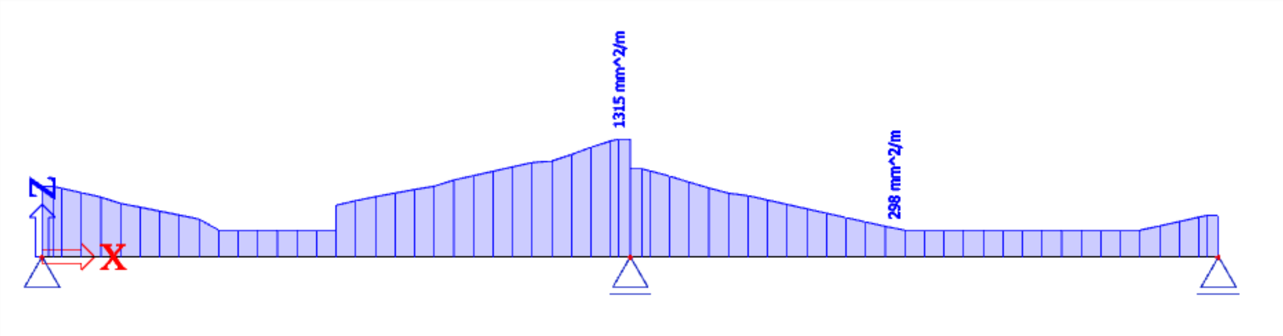
$v_1 = 0,6$ for $f_{ck} \le 60$ MPa (6.10.aN)
$v_1 = 0,9 - f_{ck}/200 > 0,5$ for $f_{ck} \ge 60$ MPa (6.10.bN)

The recommended value of α_{cw} is 1 for non-prestressed structures.

These code related parameters can be found in the Concrete setup:



If we go back to our example in SCIA Engineer, we find the following $A_{swm,req}$ for the whole beam:



Shear reinforcement

Name	dx [m]	Case	Member	A_{swm_req} [mm²/m]	A_{swm_prov} [mm²/m]	ShearReinf
S1	7,333-	ULS	Beam	**298**	**309**	**ϕ8/325mm, (ns=2)**
S1	4,900	ULS	Beam	**1315**	**1340**	**ϕ8/75mm, (ns=2)**

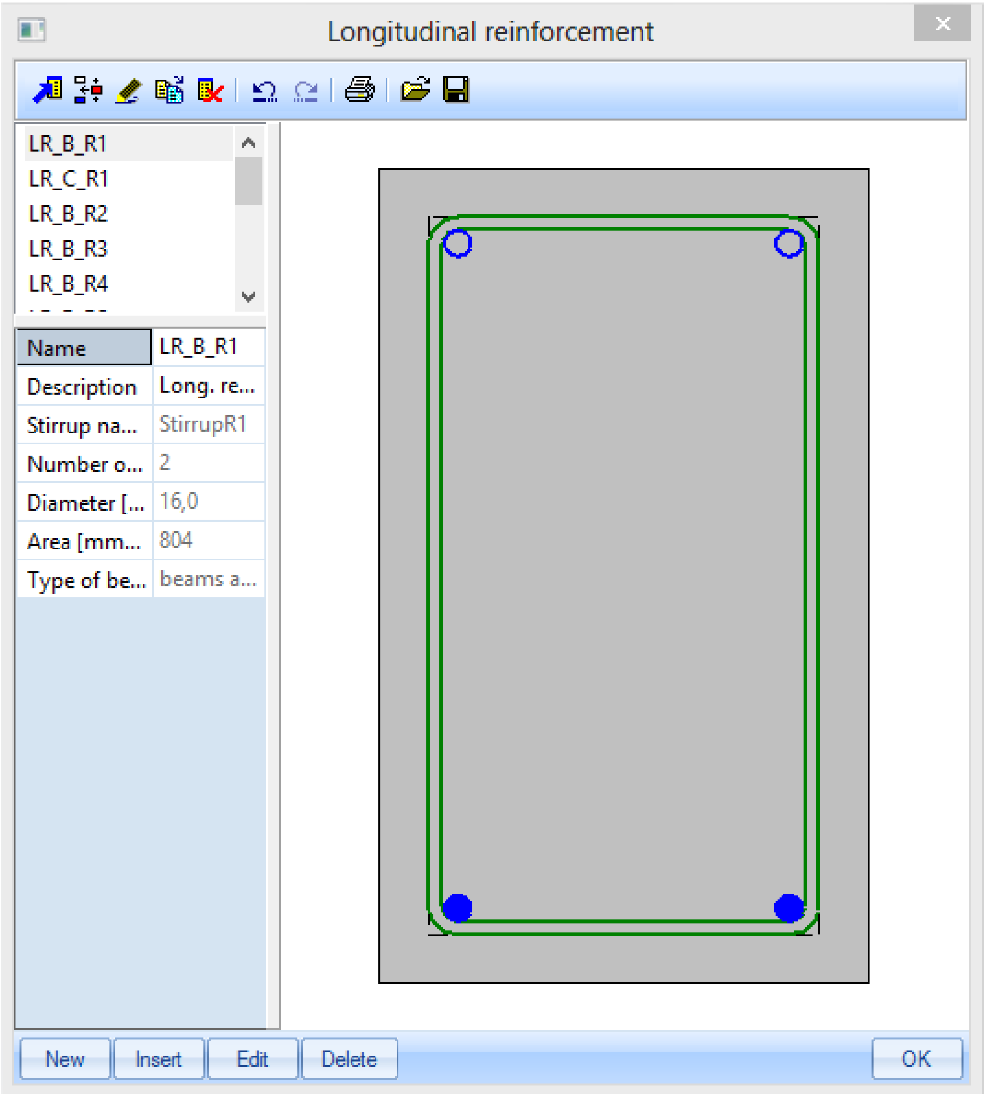
The maximum value of 1315 mm² corresponds to a two section stirrup of ϕ = 8mm every 75 mm.

Practical reinforcement

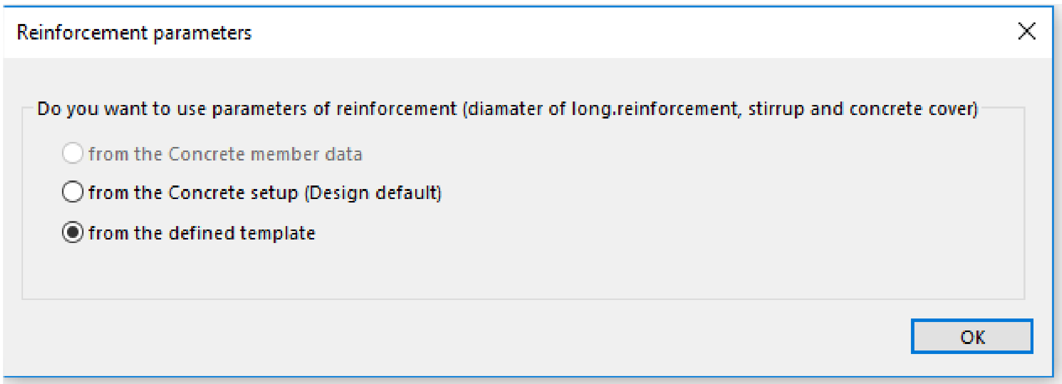
We will now pass on to the level of practical reinforcement. This will allow us to specify the reinforcement locally over the beam.

In the theoretical reinforcement design, we have calculated where reinforcement is needed.
This allows us to input manually the practical reinforcement by adding New reinforcement for the whole length of the beam.

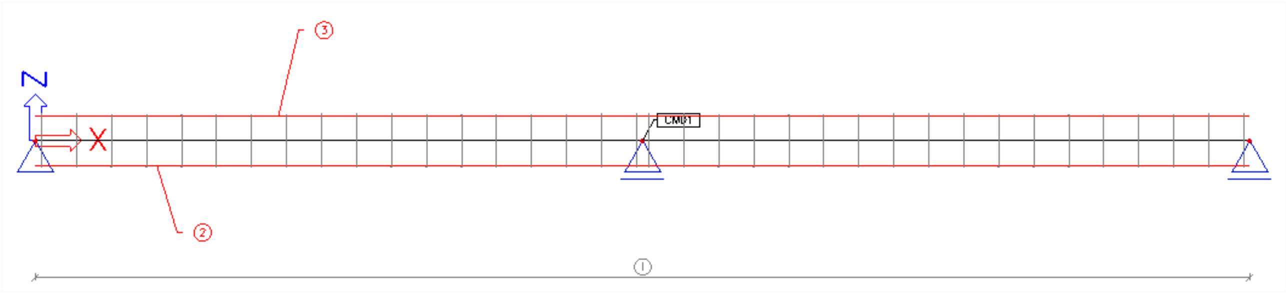
We can first select a template for the longitudinal reinforcement:



Next, we have to decide where the parameters of reinforcement are coming from:

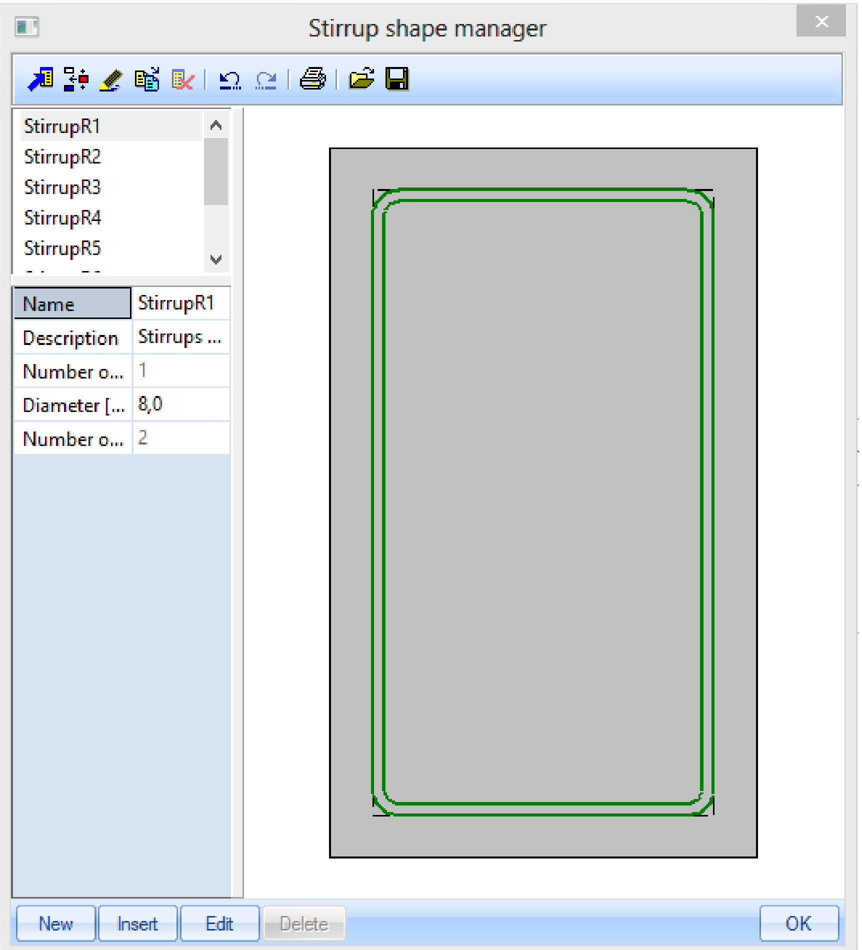


The practical reinforcement is shown graphically on the screen:

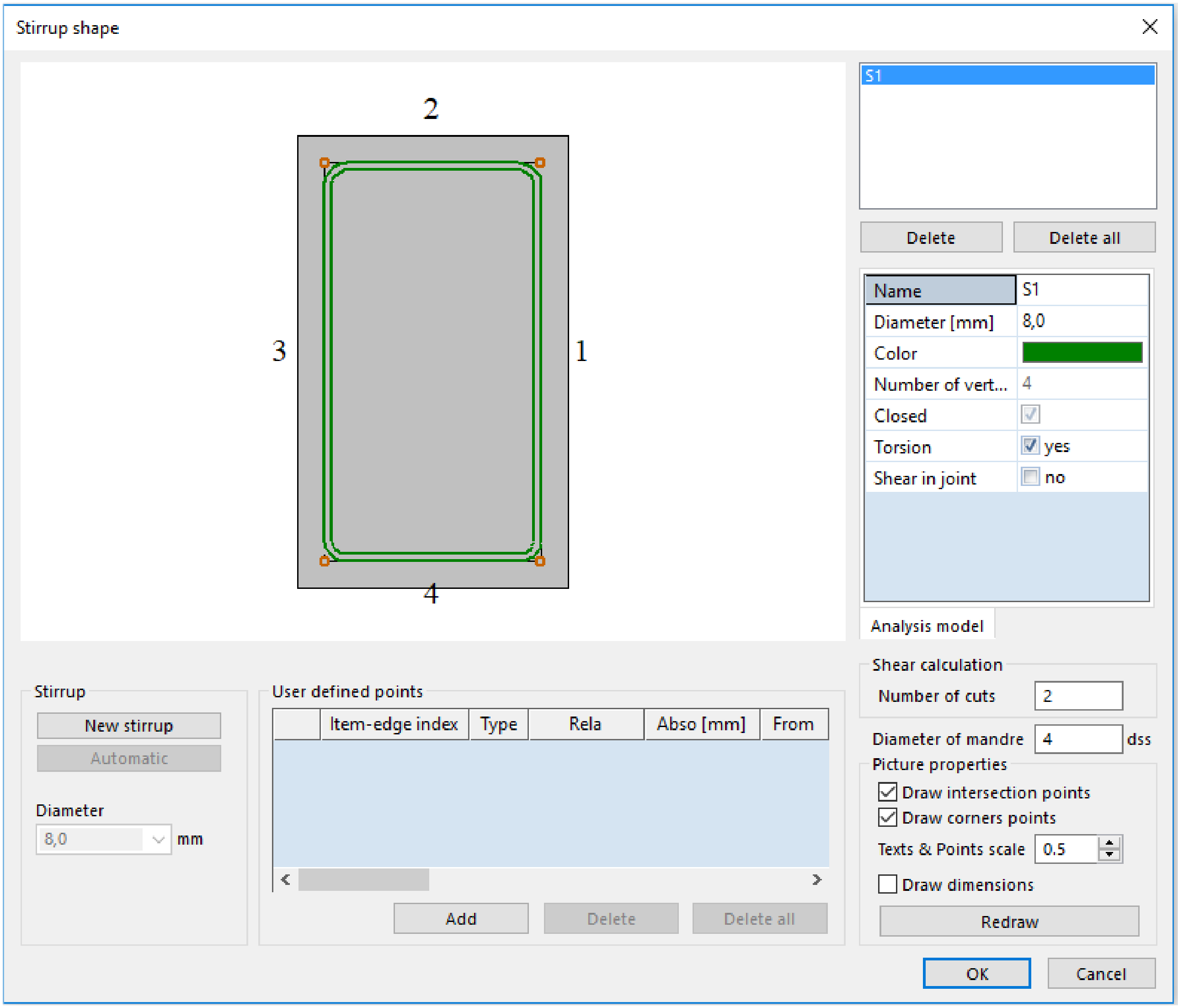


As a user, you can add locally New stirrups or New longitudinal bars.

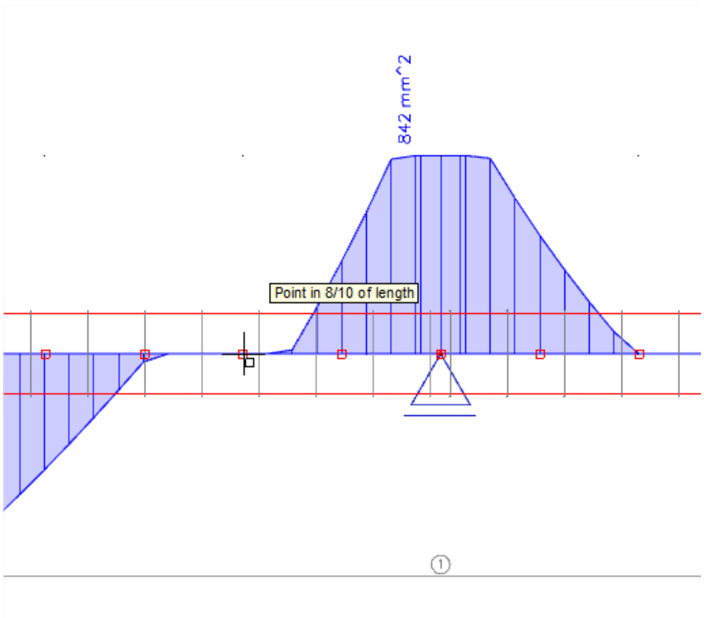
For the stirrups, you can select a certain stirrup shape:

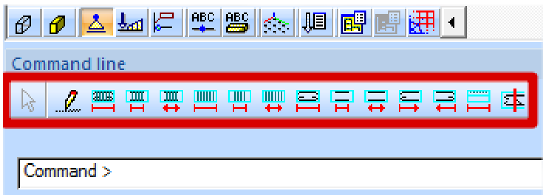


The stirrup shape can be edited or a new one can be made. Therefore user points may be added.

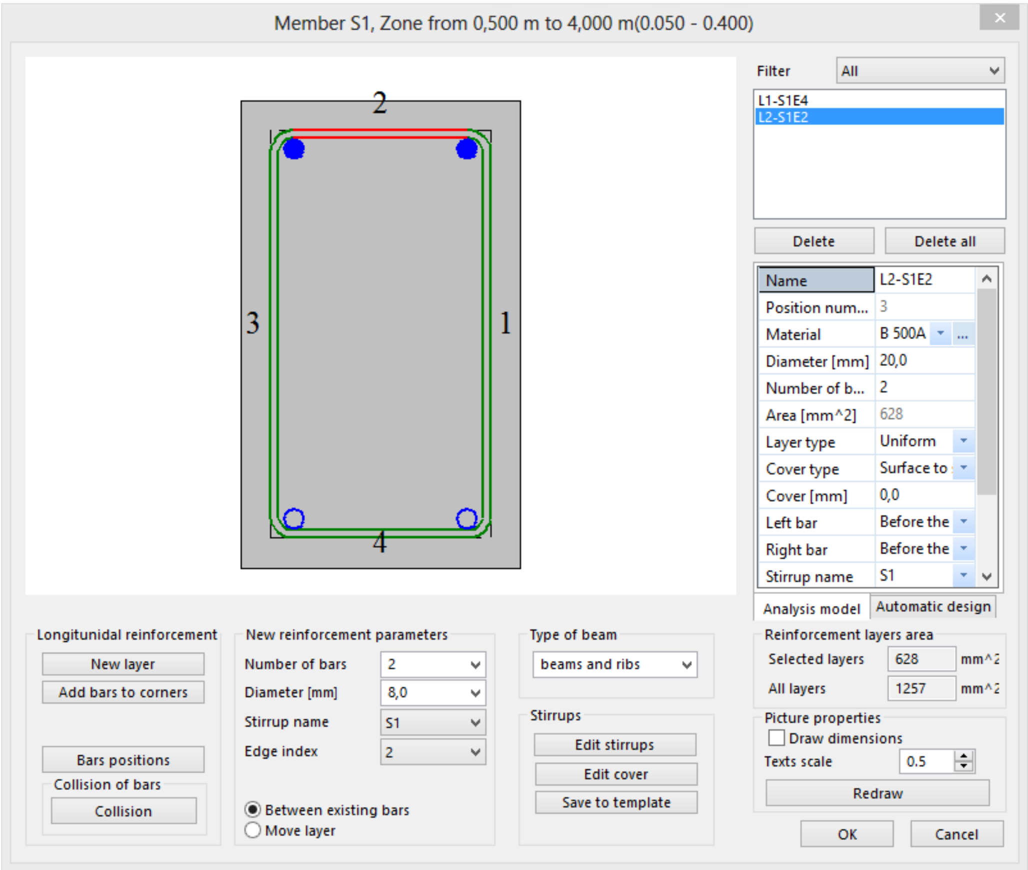


For the longitudinal reinforcement, we can define precisely where the extra practical reinforcement needs to be putted:

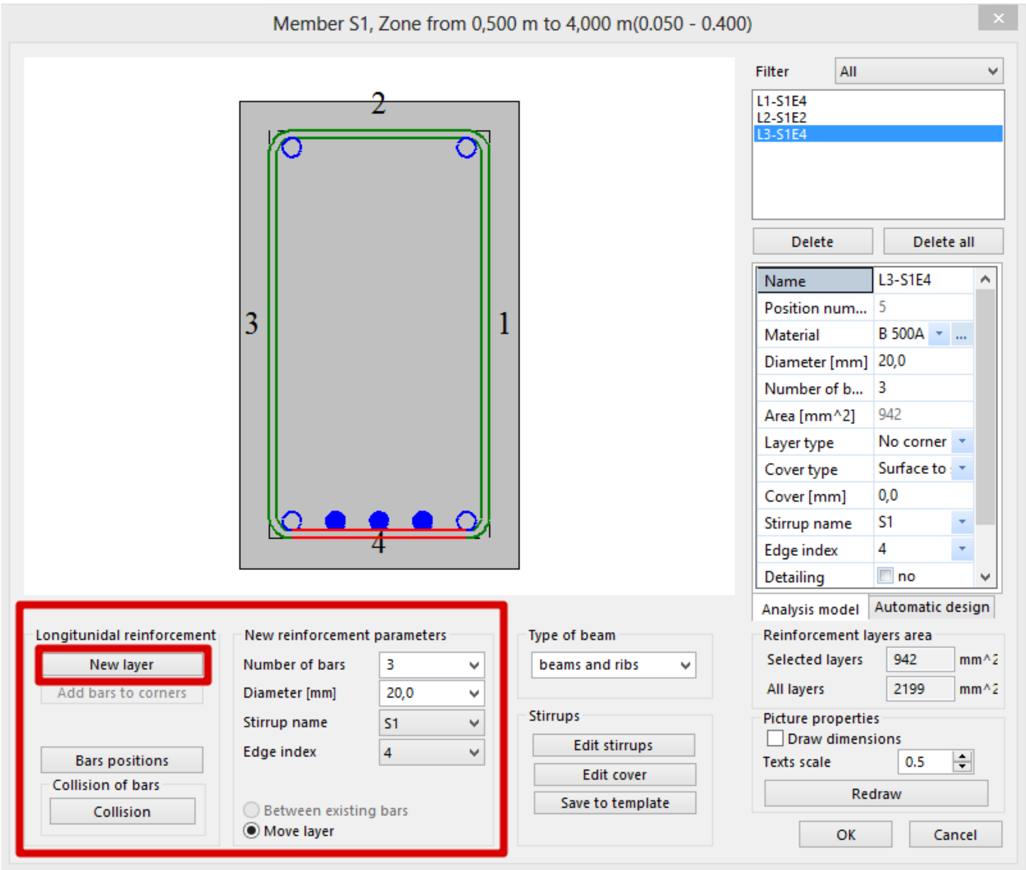




The configuration for the selected zone of the member is shown:



Here can be set on which face extra reinforcement needs to be added:

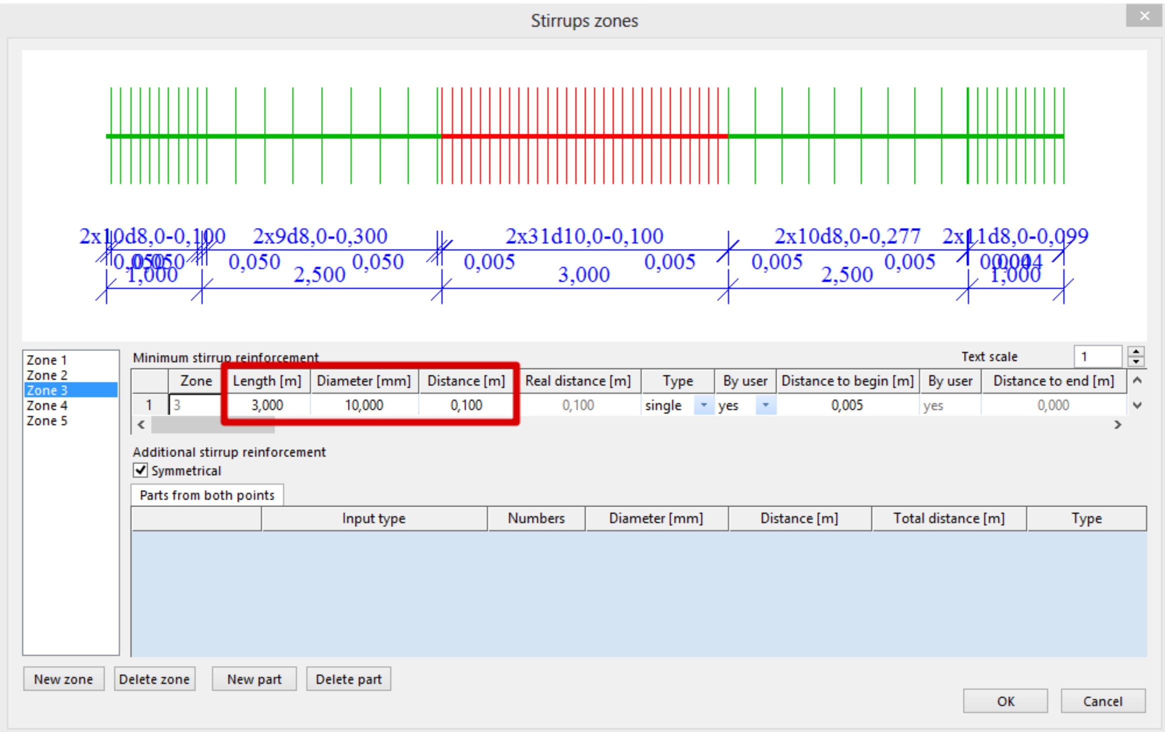


For reasons of simplicity we will add 3 bars of 20mm that are still needed over the whole area where extra reinforcement is required. This can of course be done more detailed.

The same procedure will be repeated for the upper reinforcement over the support.

Also the shear reinforcement needs to be increased in the zones over the support. This can be done by increasing the diameter of the stirrups or by decreasing the distance between the stirrups.

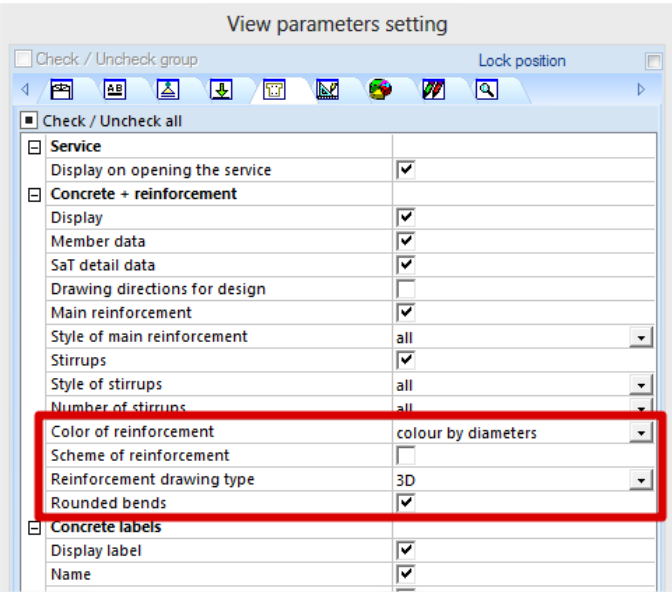
Different stirrup zones can be created:



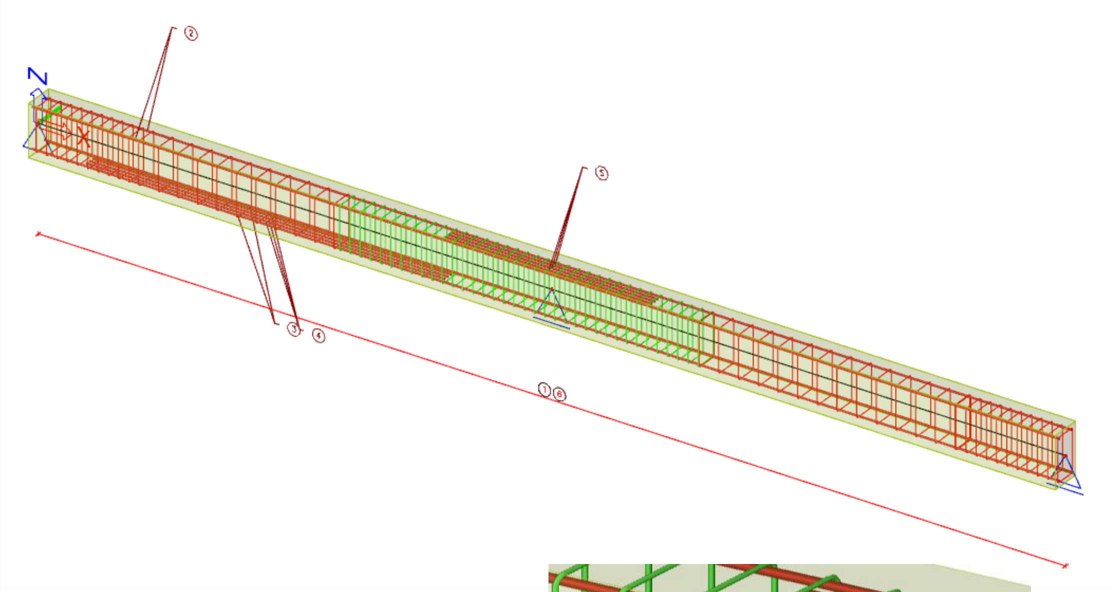
To check if there is enough shear reinforcement, a capacity check needs to be performed. This will be explained in the next chapter.

By selecting the reinforcement it is always possible to change the parameters afterwards through the property window.

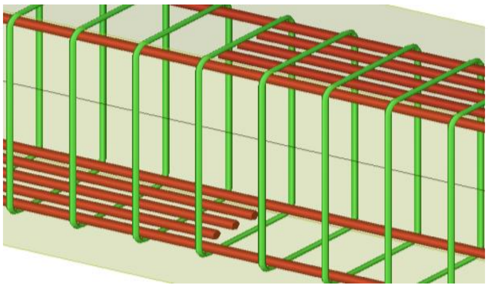
Through view parameter settings a 3D representation of the reinforcement can be obtained:



The total practical reinforcement of the beam is shown below:



A zoomed view shows the 3D representation:



Checks

In SCIA Engineer, checks can be performed in three different ways:

1. With practical reinforcement inputted on the member, checks can be done one by one for all sections of the member
2. With practical reinforcement inputted on the member, overall ULS or SLS checks can be done for a specific section of the member with the tool “Section check”
3. Without practical reinforcement, overall ULS or SLS checks can be done for a specific section of the member with the tool “Section Check”. Reinforcement will then be added locally in the Section check tool to be able to perform the various available checks.

First you get an overview of the input data for the checks:

- **Internal forces:** displaying the characteristic and design values
- **Slenderness:** determining if 2nd order effects need be considered (for member type 'column')
- **Stiffnesses:** displaying the values EA, EI_y and EI_z

Available checks at the Ultimate Limit State are:

- **Capacity check:** for N-My-Mz interaction - based on resistance calculated from interaction diagram
- **Response check:** based on check of ultimate stresses and strains for N-My-Mz interaction
- **Check of shear and torsion**
- **Check of interaction of shear, torsion, bending and normal force**

Available checks at the Serviceability Limit State are:

- **Stress limitation** (for concrete as well as reinforcing steel)
- **Crack width limitation**
- **Simple check for deflection:** based on calculation of stiffness ratio, without necessity to calculate Code Dependent Deflection (CDD)

The capacity, response and shear + torsion check should be okay if no additional reinforcement is required.

However, these checks give interesting information on the efficiency of reinforcement. For instance, if in a section only 50% of reinforcement is used, then we can conclude that here less reinforcement would have been sufficient.

The detailing provisions and the crack limitation are extra checks that are not accounted in the reinforcement design. If these checks are not okay, then the practical reinforcement needs to be changed.

In the following chapters, we will explain the checks one by one when practical reinforcement is inputted. It corresponds to the 1st method to perform a check (see above).

Example 1: ‘beam_practical reinforcement.esa’

The last chapter will be focused on the Section check tool, corresponding to 2nd and 3rd methods to perform a check (see above).

Example 2: ‘beam_without practical reinforcement.esa’

Capacity response

The Capacity - response is based on the calculation of strain and stress in a particular component (concrete fibre or reinforcement bar).
The check consists of the comparison of those strains and stresses with the limited values according to EN 1992-1-1 requirements.

However, this method does not calculate extremes (capacities of the cross-section) like the interaction diagram, but calculates the state of equilibrium for that section (response).
For capacities of the member, please refer to the “Capacity – diagram” check.

The following checks are performed:

- Check of compressive concrete (cc)
- Check of compressive reinforcement (sc)
- Check of tensile reinforcement (st)

The Unity Check, UC, displayed on the screen will be the maximum value of those 3 checks.

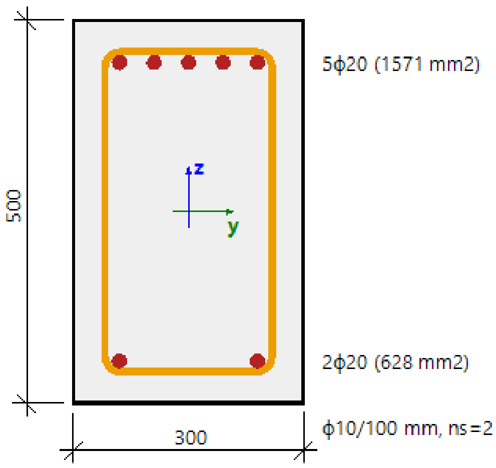
Example: ‘beam_practical reinforcement.esa’

Run the Capacity – Response check in Concrete menu > Reinforcement check (ULS+SLS) > Capacity – Response (ULS)

The maximum value of the check is given on the middle support. The Standard output gives:

Beam S1	RECT (500; 300)
EC EN 1992-1-1:2004/AC:2008	Section 19 [dx = 5 m]

Member length: L = 10 m
Buckling y-y L_y = 10 m (sway)
Buckling z-z L_z = 10 m (sway)



Concrete: C30/37
Bi-linear stress-strain diagram
Exposure class: XC3
Longitudinal reinforcement: B 500A
Bi-linear with an inclined top branch
7ϕ20 mm (A_s = 2199 mm^2)
ρ_l = 1.466 % (17.3 kg/m)
Shear reinforcement: B 500A
Bi-linear with an inclined top branch
ϕ10/99.7 mm (n_s = 2) (A_{sw} = 157 mm^2)
ρ_w = 1.051 % (12.4 kg/m) (A_{swm} = 1576 mm^2/m)
Cover (stirrup)
Top: 36 mm
Bottom: 36 mm
Left: 36 mm
Right: 36 mm

Summary of check

Type of component	Fibre / Bar	ε_{extr} [‰]	σ_{extr} [MPa]	Check strain [-]	Check stress [-]	UC [-]	Limit [-]	Status
Concrete	1	-1.63	-18.7	0.47	0.93	0.95	1	OK
Reinf.	1	2.17	434	0.10	0.95			

In the Standard output you can read the UC, and the extreme strain and stress in the studied section.

In the Detailed output you will get all the strains and stresses and the limit strains and stresses:

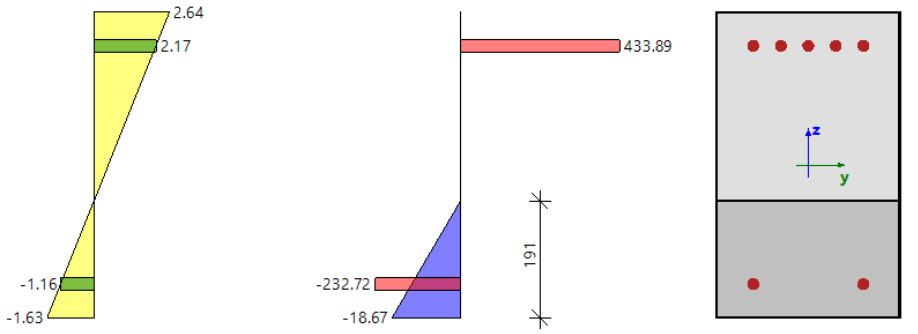
Extreme values of stress/strain in component

Type of component	Fibre / Bar	ε [‰]	ε_{lim} [‰]	σ [MPa]	σ_{lim} [MPa]	UC [-]	Status
Concrete - compression	1	-1.63	-3.5	-18.7	-20	0.93	OK
Concrete - tension	3	2.64	0	0	0	0.00	OK
Reinforcement - compression	3	-1.16	-22.5	-233	-454	0.51	OK
Reinforcement - tension	1	2.17	22.5	434	454	0.95	OK

Note that the tensile stress in concrete is not considered, therefore the corresponding UC is 0.

Stress and strain diagrams are also available in the Detailed output:

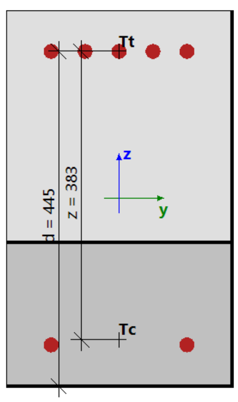
Stress and strain distribution



Settings that might influence the check:

- Effective depth of cross-section - d

It is usually defined as distance of the most compressive fibre of concrete to centre of gravity of tensile reinforcement. In SCIA Engineer, the effective depth of cross-section is defined as distance of the most compressive fibre of concrete to position resultant of forces in tensile reinforcement.



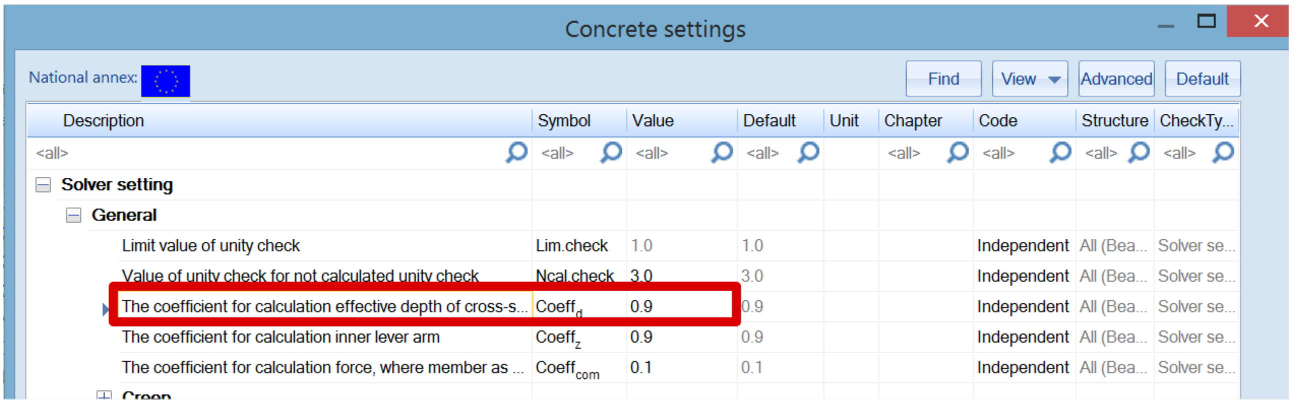
The effective depth d cannot be calculated in the following cases:

- The most compressive fibre cannot be determined (the whole cross-section is in tension)
- Resultant of forces in tensile reinforcement cannot be determined (whole section is in compression)
- Equilibrium is not found
- Distance of the most compressive fibre and Resultant of forces in tensile reinforcement is less than 0,5·h

In those cases, the effective depth is calculated according to formula: $d = Coeff_d * h_l$

$Coeff_d$ by default 0.9 in Concrete settings > Solver settings > General

h_l height of cross-section perpendicular to neutral axis



- Inner lever arm

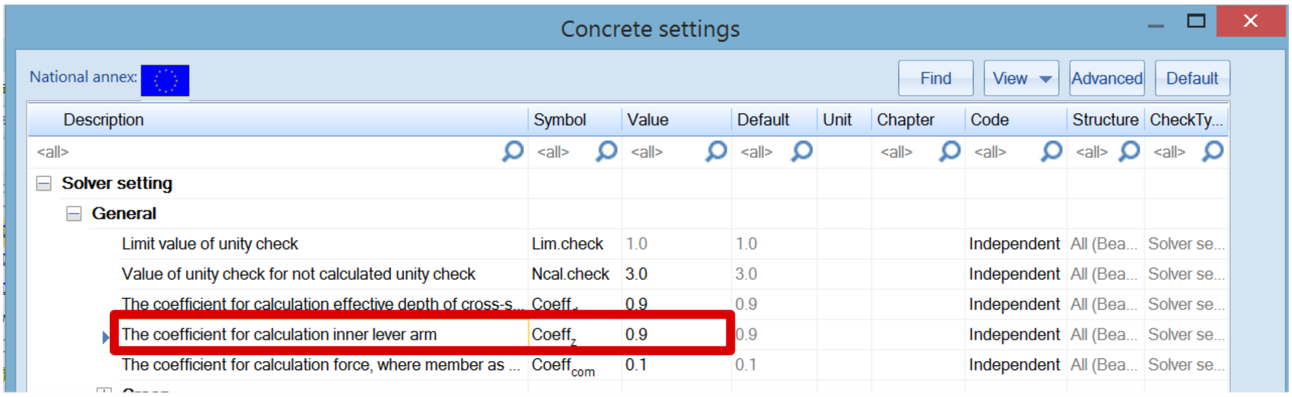
z is defined in EN 1992-1-1, clause 6.2.3 (3) as the distance between position resultant of tensile force (tensile reinforcement) and position of resultant of compressive force (compressive reinforcement and compressive concrete).

The inner lever arm cannot be calculated in the following cases:

- The most compressive fibre cannot be determined (the whole cross-section is in tension)
- Resultant of forces in tensile reinforcement cannot be determined (whole section is in compression)
- Equilibrium is not found

In those cases, it is calculated according to formula: $z = Coeff_z * d$

$Coeff_z$ by default 0.9 in Concrete settings > Solver settings > General



For additional information about this check and the theoretical background, please refer to our web help.

Capacity diagram

Capacity - diagram services uses the creation of interaction diagram (graph presenting the capacity of a concrete member to resist a set of N+My+Mz).

This check calculates the extreme allowable interaction between the normal force N and bending moments My and Mz.

Example: 'beam_practical reinforcement.esa'

Run the Capacity – Diagram check in Concrete menu > Reinforcement check (ULS+SLS) > Capacity – diagrapm (ULS)

The standard output gives the summary result of the check:

Summary of check

N	N_{Ed}	N_{Rd+}	M_y	M_{Edy}	M_{Rdy+}	M_{Rdy-}	UC	Status
		N_{Rd-}	M_z	M_{Edz}	M_{Rdz+}	M_{Rdz-}		
[kN]	[kN]	[kN]	[kNm]	[kNm]	[kNm]	[kNm]	[-]	
0	0	0	-261	-261	119	-278	0.939	OK
		0	0	0	0	0		M_{Edz}/M_{Rdz}

The Detailed output gives additional info about how the check is performed:

Summary of check

Forces: N_{Ed} = 0 kN M_{Edy} = -261 kNm M_{Edz} = 0 kNm

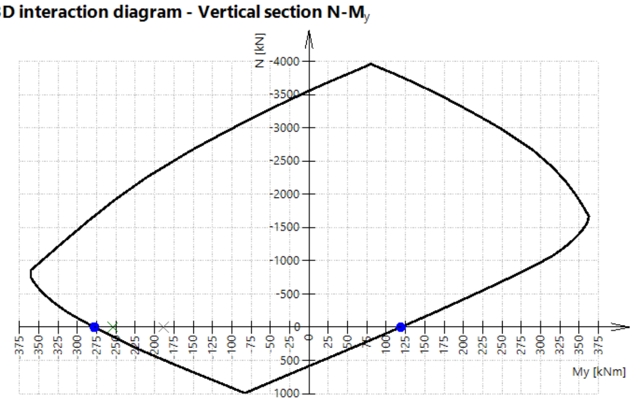
Resistance: N_{Rd} = 0 kN M_{Rdy} = -278 kNm M_{Rdz} = 0 kNm

Calculation of unity check

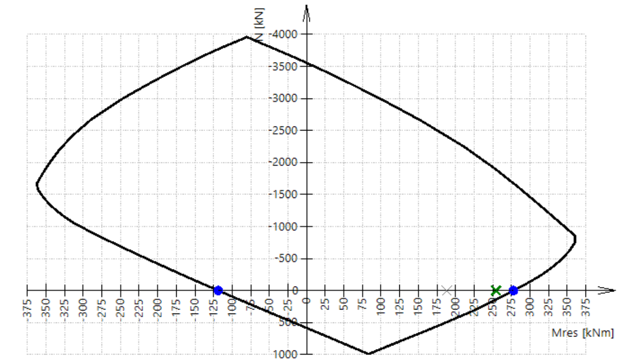
$$UC = \frac{\sqrt{N_{Ed}^2 + M_{Edy}^2 + M_{Edz}^2}}{\sqrt{N_{Rd}^2 + M_{Rdy}^2 + M_{Rdz}^2}} = \frac{\sqrt{0^2 + -261^2 + 0^2}}{\sqrt{0^2 + -278^2 + 0^2}} = 0.939 \quad <= 1 \quad \text{OK}$$

Interaction diagrams are also drawn in the Detailed output:

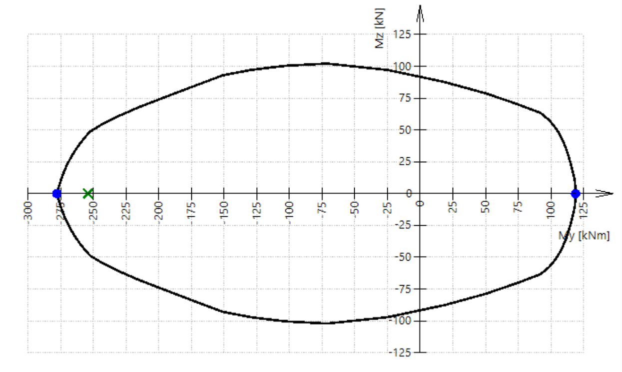
3D interaction diagram - Vertical section N-M_y



3D interaction diagram - Vertical section N-M_{res}



3D interaction diagram - Horizontal section M_y-M_z



Settings that might influence the check:

- Interaction diagram method
- Division of strain
- Number of points in vertical cuts

For additional information about this check and the theoretical background, please refer to our web help.

Shear + Torsion

Check of Interaction shear and torsion consists of three checks according to clause 6.1 - 6.3 in EN 1992-1-1:

- check of shear
- check of torsion
- check of interaction of shear and torsion

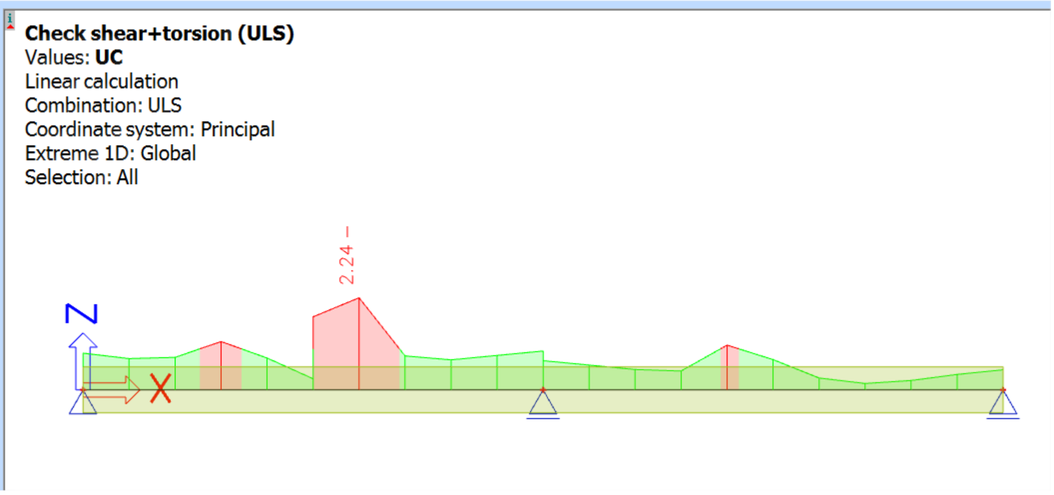
This check can be performed if the following conditions are met:

- The material of all reinforcement bars and stirrups are the same
- The angle between gradient of the strain plane and the resultant of shear forces is not greater than 15°
- Cross-section with one polygon and one material

Example: 'beam_practical reinforcement.esa'

Run the Shear + Torsion check in Concrete menu > Reinforcement check (ULS+SLS) > Shear + Torsion (ULS)

Some parts of the beam do not satisfy:



The Standard output allows us to identify which specific check is not satisfied:

Forces

Content of combination: 1.35*LC1+1.35*LC2+1.50*LC3

N_{Ed} = 0 kN M_{Edy} = 203 kNm M_{Edz} = 0 kNm V_{Edy} = 0 kN V_{Edz} = -152 kN T_{Ed} = 0 kNm

Resultant of shear force

$$V_{Ed} = \sqrt{V_{Edy}^2 + V_{Edz}^2} = \sqrt{0^2 + -152^2} = 152 \text{ kN}$$

Difference between angles α_M and α_V

$$\alpha_{MV} = abs(\alpha_M - \alpha_V) = abs(90 - 90) = 0\,°$$

Summary of check

d = 445 mm z = 383 mm b_w = 300 mm b_{w1} = 300 mm V_{Rdc} = 87.8 kN V_{Rds} = 66.5 kN V_{Edmax} = 705 kN V_{Rdmax} = 598 kN

Type of check	Forces	Resistances	UC [-]	Status
Check shear Vy+Vz	151.7 kN	66.5 kN	2.28	Not OK
Check torsion	0.0 kNm	0.0 kNm	0.00	OK
Interaction check Vy+Vz+T (concrete)			0.00	OK
Interaction check Vy+Vz+T (shear)	0.0 kN	0.0 kN	0.00	OK
Interaction check Vy+Vz+T (long. reinf.)	0.0 kN	0.0 kN	0.00	OK
Summary of check			2.28	Not OK

Here the shear forces cause a unity check >1.

In the Detailed output we can read notes, warning and errors about the design. For example, for the shear forces check not satisfied, the report clearly explains that the shear reinforcement is not sufficient and that we have to increase it.

Shear check

Check V_{Rdmax}

V_{Ed} = 152 kN ≤ $V_{Rdmax} + V_{ccd} + V_{td}$ = 598 kN

Note: The check satisfies for crushing of the compression strut ($V_{Ed} \leq V_{Rd.max} + V_{td} + V_{ccd}$).

Check V_{Edmax}

V_{Ed} = 152 kN ≤ $V_{Edmax} + V_{ccd} + V_{td}$ = 705 kN

Note: The check satisfies for shear force near the support ($V_{Ed} \leq V_{Ed.max} + V_{td} + V_{ccd}$).

Check V_{Rdc} and V_{Rds}

V_{Ed} = 152 kN > V_{Rdc} = 87.8kN and V_{Ed} = 152 kN > $V_{Rds} + V_{ccd} + V_{td}$ = 66.5 kN

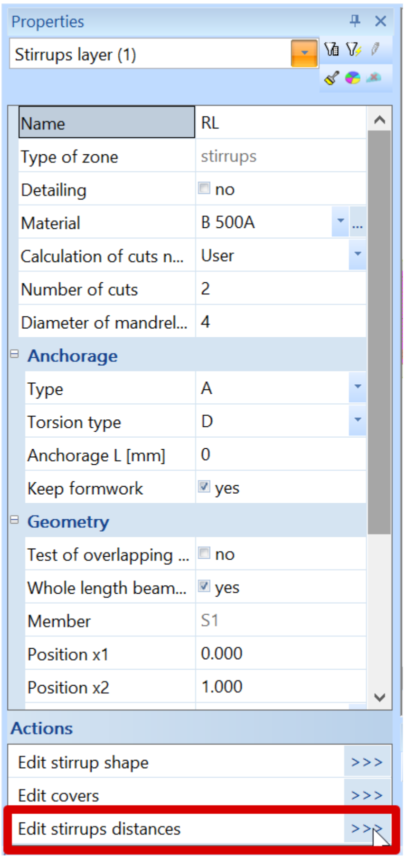
Error: The check does not satisfy, because of shear reinforcement ($V_{Ed} > V_{Rds} + V_{ccd} + V_{td}$). It is necessary to increase area of shear reinforcement or to increase dimensions of the cross-section or quality of shear reinforcement.

Unity check

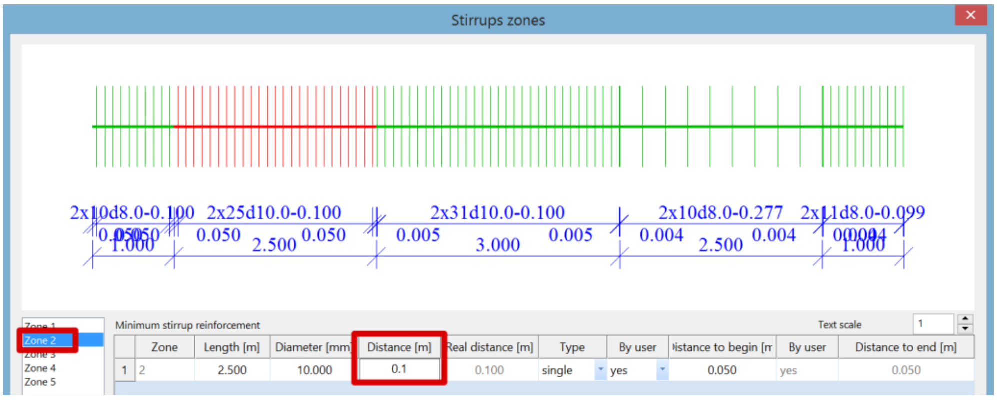
$$UC = \frac{abs(V_{Ed})}{V_{Rd}} = \frac{abs(152\ kN)}{66.5\ kN} = 2.28$$

Various actions can be done to fix this issue. In this example, we choose to decrease the spacing of the stirrups in the section where there is an issue.

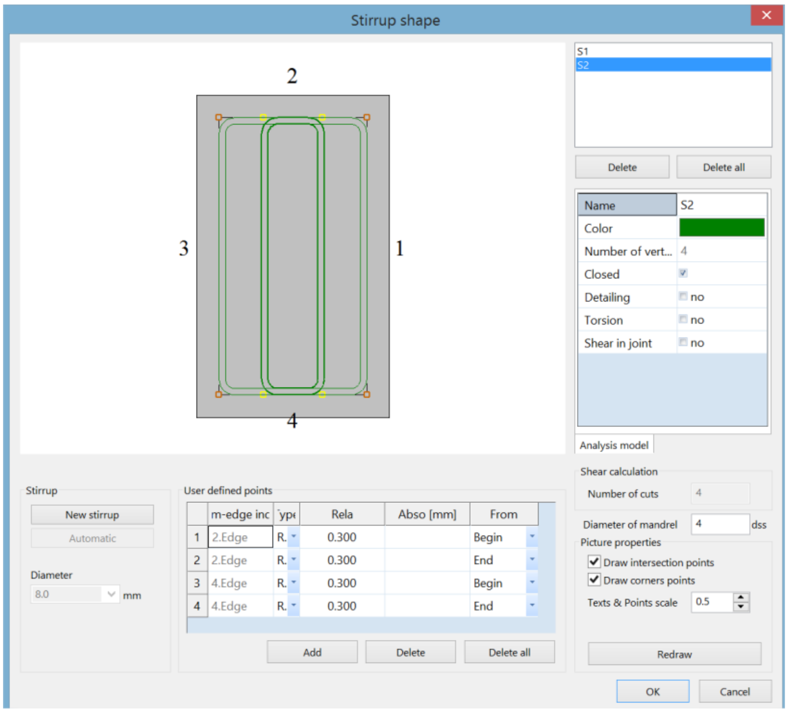
Select stirrups and click on “Edit stirrups distances” at the bottom of the Properties of the stirrup layers:



Select “Zone 2” and change the distance between stirrups from 0.3m to 0.1m:

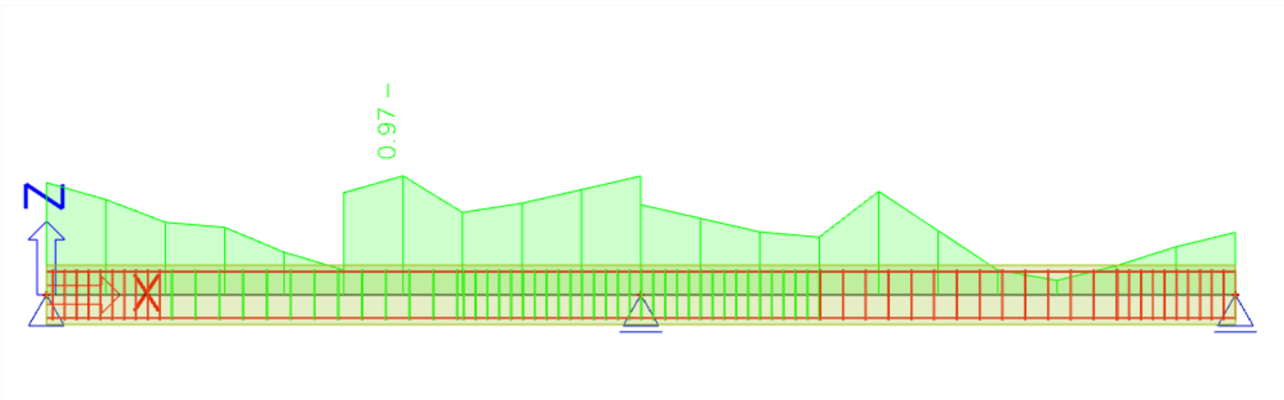


We could also have added more stirrups like below:



Changing the stirrup shape allows us to keep a bigger distance of 0.2m between stirrups in “Zone 2”.

After modification, the shear + torsion check is satisfied:



Settings that might influence the check:

- Coefficient for calculation of effective depth of cross-section

Default value 0.9 in Concrete settings > Solver settings > General

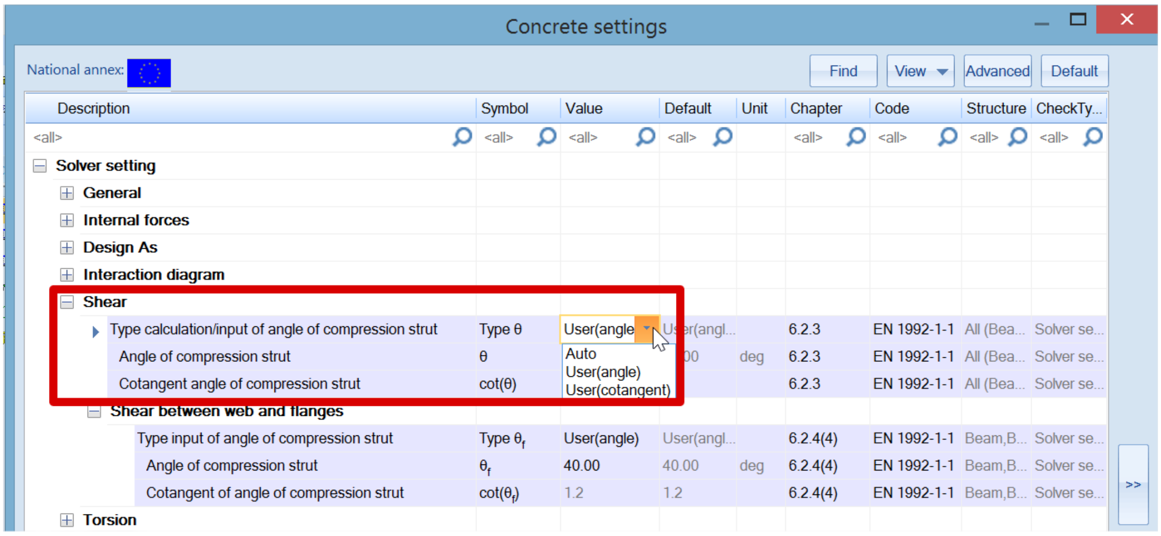
- Coefficient for calculation of inner lever arm

Default value 0.9 in Concrete settings > Solver settings > General

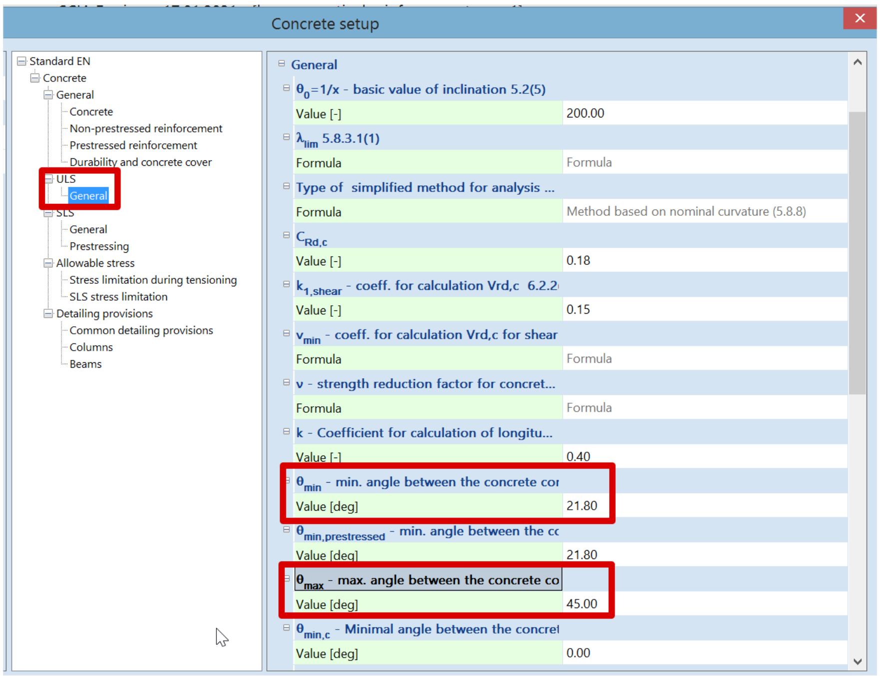
- Angle of concrete compression strut

3 types of input in Concrete settings > Solver settings > Shear:

- User (angle) user input of the angle – by default
- User (cotangent) user input of the cotangent
- Auto automatic calculation of the angle fulfilling equation 6.29



The angle should be between θ_{min} and θ_{max} defined in the NA for EN1992-1-1.



- Angle of shear reinforcement

Practical reinforcement can only be introduced at 90°.

- Type for determination equivalent thin-walled cross-section

For additional information about this check and the theoretical background, please refer to our web help.

Stress limitation

Stress limitation is based on the verification of:

- **compressive stress in concrete** - the high value of compressive stress in concrete could lead to appearance of longitudinal cracks, spreading of micro-cracks in concrete and higher values of creep (mainly nonlinear). This effect can lead to a state where the structure is unusable.
- **tensile stress in reinforcement** - stress in reinforcement is verified due to limitation of unacceptable strain existence and thus appearance of cracks in concrete.

Example: 'beam_practical reinforcement.esa'

The stress limitation check is done according to the following steps:

1. Verification of crack appearance
2. Verification of the stresses

The Standard output shows those 2 steps:

Verification of cracks in cross-section

Load	Type of module	E_c [MPa]	Combi.	N_{Ed} [kN]	M_{Edy} [kNm]	M_{Edz} [kNm]	σ_{ct} [MPa]	h [mm]	$f_{ct,eff}$ [MPa]	Cracks appear
Short	E_c	0	Char.	0	-188	0	12.6	500	2.9	YES

Stress limitation in concrete

Check type	Load	N_{Ed} [kN]	M_{Edy} [kNm]	M_{Edz} [kNm]	y_i [mm]	z_i [mm]	σ_c [MPa]	$\sigma_{c,lim}$ [MPa]	$\sigma_c/\sigma_{c,lim}$ [-]	Status
§7.2(2) Char.	Short	0	-188	0						OFF
§7.2(3) Q.-P.	Short	0	-188	0	0.15	-0.25	-21.2	-13.5	1.57	Not OK

Stress limitation in non-prestressed reinforcement

Check type	Load	N_{Ed} [kN]	M_{Edy} [kNm]	M_{Edz} [kNm]	y_i [mm]	z_i [mm]	σ_s [MPa]	$\sigma_{s,lim}$ [MPa]	$\sigma_s/\sigma_{s,lim}$ [-]	Status
§7.2(5) Char.	Short	0	-188	0	0.09	0.2	300	400	0.75	OK

Verification of crack appearance

Crack appearance is verified for characteristic load combination in accordance to chapter 7.1(2) in EN1992-1-1:

- $\sigma_{ct} \leq f_{ct,eff}$ **no crack appears**
- $\sigma_{ct} > f_{ct,eff}$ **crack appears**

σ_{ct} maximal tensile stress in concrete fibre
$f_{ct,eff}$ effective concrete tensile strength

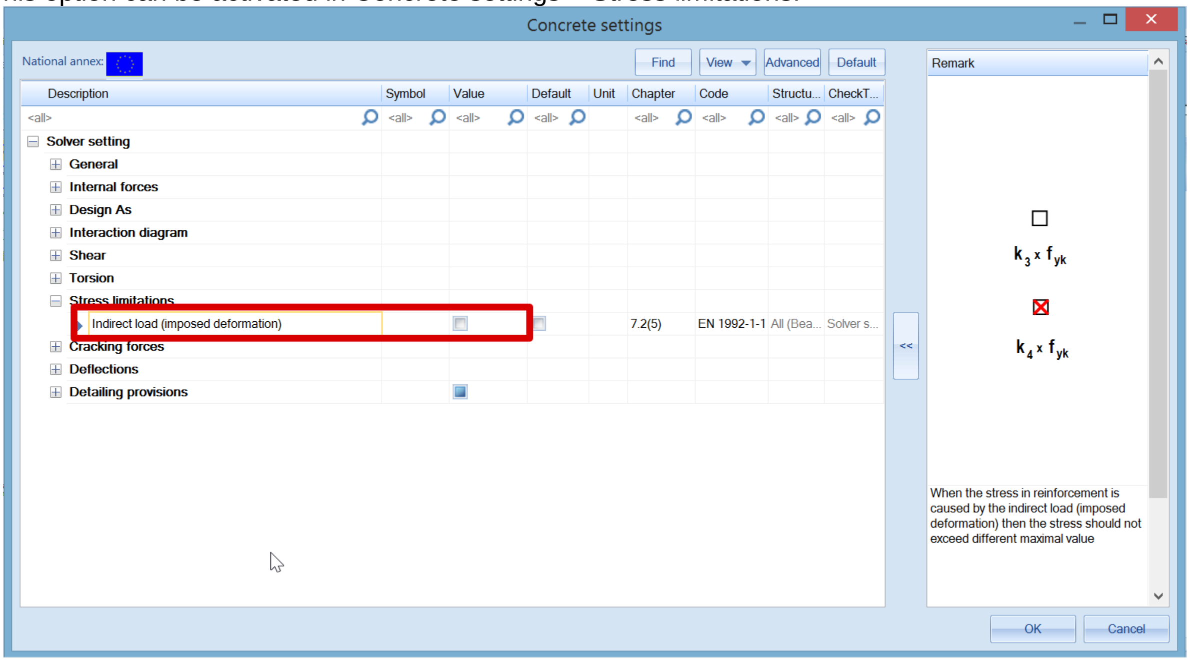
Verification of stresses

There are 3 stress limitations checked:

- $\sigma_{c,char,lim} \leq k_1 \times f_{ck}$ concrete stress under Char. load – 7.2(2) - exposure classes XD, XF, XS
- $\sigma_{c,qp,lim} \leq k_2 \times f_{ck}$ concrete stress under Quasi Perm. load – chapter 7.2(3)
- $\sigma_{s,char,lim} \leq k_3 \times f_{yk}$ reinforcement stress under Char. Load – chapter 7.2(5)

Values of k_1, k_2, k_3, are defined in the NA, standard values are respectively 0.6, 0.45, 0.8

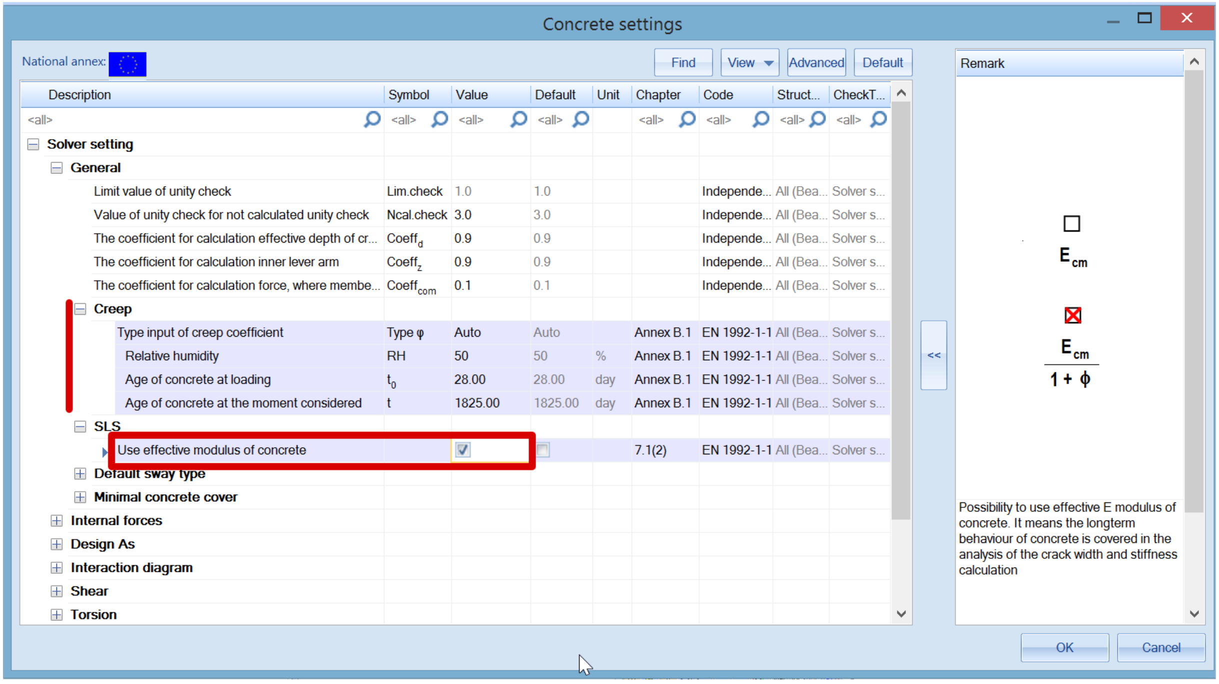
Additionally, when the stress in the reinforcement is caused by an imposed deformation, then the maximal strength is increased to $k_4 \times f_{yk}$, where k_4 is NA parameter with standard value k_4 = 1,0.
This option can be activated in Concrete settings > Stress limitations:



By default, stress limitation check is done for short-term state.
It is possible to perform a long-term state. Effective E modulus of elasticity is calculated as follows, using the creep coefficient:

$$E_{c,eff} = E_{cm} / (1+ \varphi)$$

Long-term behaviour can be activated in Concrete Setting > Solver settings > General > SLS > Use effective modulus of elasticity.
The creep coefficient can whether be calculated by the software or inputted manually in the Concrete settings.



Note: *Scia Engineer is not able to use characteristic or quasi-permanent combinations together in one step. Therefore, the same forces (load combination) are used for crack appearance and final stress values.*

Crack width

The crack width is calculated according to clause 7.3.4 in EN 1992-1-1.

The following preconditions are used for calculation:

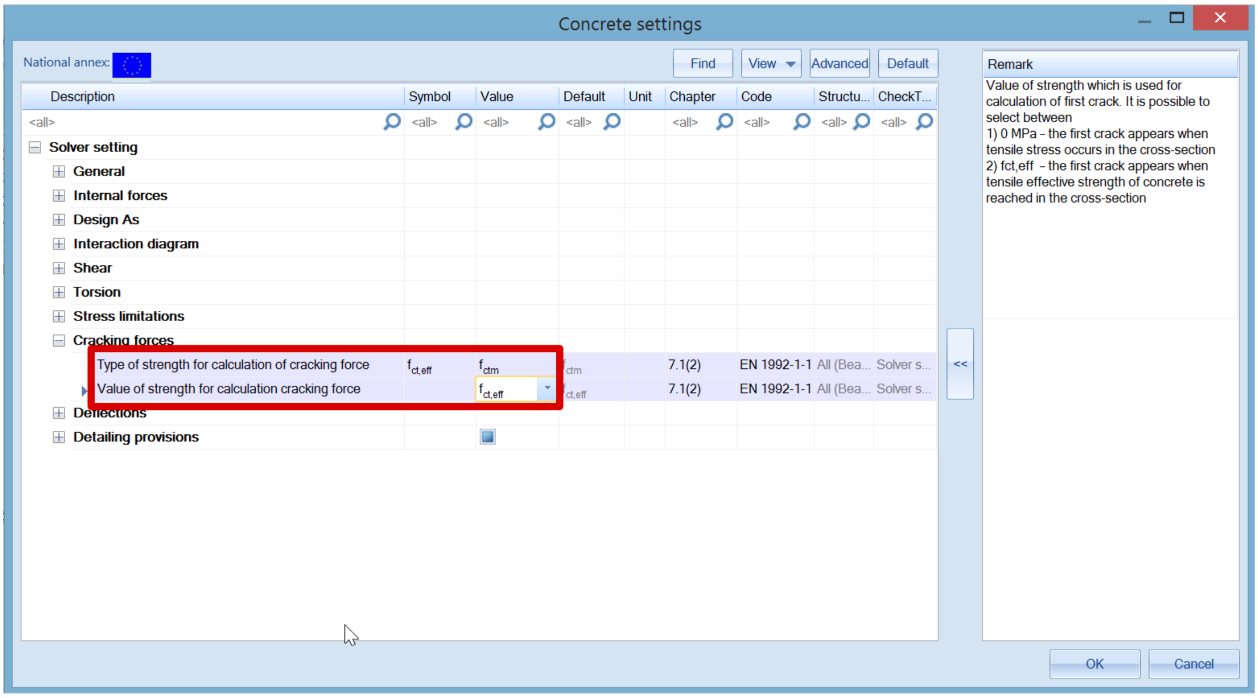
- The crack width is calculated for beams and columns and for general loads (N+My+Mz)
- Cross-section with one polygon and one material is considered in version SEN 17
- The material of all reinforcement bars must be the same in SEN 17
- Appearance of cracks should be calculated for a characteristic combination according to EN 1992-1-1, clause 7.2(2). A simplification is made in SEN 17 that the normal stress is calculated for the same type of combination as used for the calculation of crack width, inputted in service Crack control.

Example: 'beam_practical reinforcement.esa'

First a determination whether the section is cracked or un-cracked is performed by comparing:

- $\sigma_{ct} \leq \sigma_{cr}$ Uncracked
- $\sigma_{ct} > \sigma_{cr}$ Cracked

Value for σ_{cr} can be set in the Concrete settings > Cracking forces. Two options can influence this value:

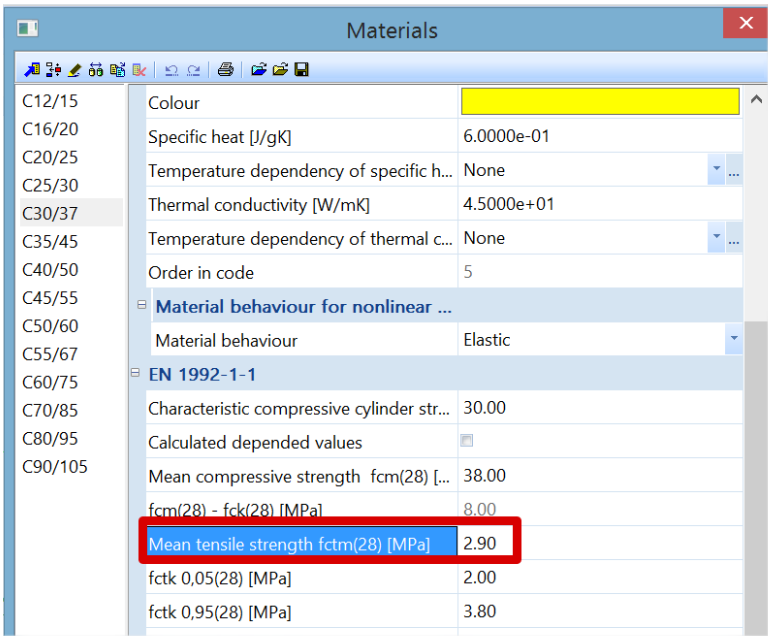


Value of strength for calculation of cracking forces:

- σ_{cr} = 0 Mpa   cracks appear when tensile stress occurs in the section
- $\sigma_{cr} = f_{ct,eff}$ cracks appear when tensile effective strength of concrete is reached in the section

Type of strength for calculation of cracking forces:
If previous option is set on $\sigma_{cr} = f_{ct,eff}$, which is the default value then:

- $f_{ct,eff} = f_{ctm}$ mean tensile strength of concrete at 28 days set in the material properties.
- $f_{ct,eff} = f_{ctm,fl}$ mean flexural tensile strength (EN 1992-1-1,clause 3.1.8(1)). This value should be used if restrained deformations such as shrinkage or temperature movements are considering for calculation crack width.



Note: The value presented in material properties (picture above) is the mean tensile strength at 28 days. If cracking is expected earlier than 28 days, it is necessary to input this value $f_{ctm}(t)$ into the material properties (EN 1992-1-1,clause 3.1.2(9)).

The check of crack appearance, with values of cracking forces (N_{cr}, M_{cry}, M_{crz}) can be read in the Detailed output:

Material characteristics

Effective strength of concrete:

$f_{ct.eff} = f_{ctm} = 2.9$ MPa

Strength in concrete , when crack is appeared:

$\sigma_{cr} = 2.9$ MPa

Modulus of elasticity of concrete:

$E_c = E_{cm} = 33$ GPa

Forces

Content of combination:

LC1+LC2+LC3

Characteristic values

$N_{char} = 0$ kN $M_{y.char} = -188$ kNm $M_{z.char} = 0$ kNm

Quasi-permanent values

$N_{qp} = 0$ kN $M_{y.qp} = -188$ kNm $M_{z.qp} = 0$ kNm

Angle of bending moment resultant

$\alpha_M = -90°$

Cross-section characteristics

Type	Css-uncracked	Css cracked
t_{iy} [m]	0	0
t_{iz} [m]	$6.82 \cdot 10^{-3}$	-0.117
A_i [m^2]	0.163	0.0533
I_{iy} [m^4]	$3.63 \cdot 10^{-3}$	$1.91 \cdot 10^{-3}$
I_{iz} [m^4]	$1.19 \cdot 10^{-3}$	$370 \cdot 10^{-6}$

Calculation of cracking forces (uncracked section)

Maximal stress in concrete

$\sigma_{ct} = 12.6$ MPa

Cracking forces

$N_{cr} = 0$ kN $M_{cry} = -43.3$ kNm $M_{crz} = 0$ kNm

$\sigma_{ct} = 12.6$ MPa > $\sigma_{cr} = 2.9$ MPa => Cracks appear

Note: The crack is appeared, because maximal tensile stress is greater than cracking strength.

Here, modulus E is taken for short-term state. As mentioned previously, long-term state with an effective modulus E_{eff} can be chosen in Concrete settings > General > SLS > Use effective modulus E.

In this example, cracks appear.
Crack width is then calculated according to EN 1992-1-1, formula 7.8: $w = s_{r,max} \cdot (\varepsilon_{sm} - \varepsilon_{cm})$

For further details about the calculation, the Detailed output can be analysed. The following picture shows only a part of the report:

Maximum crack spacing

s_{max} = 45 mm ≤ 5*(c+0.5*ϕ_{eq}) = 275 mm or $\rho_{p.eff}$ = 0, therefore:

$$s_{r.max} = k_3 \cdot c + \frac{k_1 \cdot k_2 \cdot k_4 \cdot \phi_{eq}}{\rho_{p.eff}} = 3.4 \cdot 0.045 + \frac{0.8 \cdot 0.5 \cdot 0.425 \cdot 0.02}{0.0428} = 232 \text{ mm} \quad (7.11)$$

Mean strain in the reinforcement

$$\varepsilon_{sm\_cm} = \max\left(\frac{\sigma_s - k_t \cdot \left(\frac{f_{ct.eff}}{\rho_{p.eff}}\right) \cdot \left(1 + \alpha_E \cdot \rho_{p.eff}\right)}{E_s} ; \frac{0.6 \cdot \sigma_s}{E_s} \right) \quad (7.9)$$

$$= \max\left(\frac{300 \cdot 10^6 - 0.46 \cdot \left(\frac{2.9 \cdot 10^6}{0.0428}\right) \cdot \left(1 + 6.06 \cdot 0.0428\right)}{200 \cdot 10^9} ; \frac{0.6 \cdot 300 \cdot 10^6}{200 \cdot 10^9} \right) = 1.3 ‰$$

Calculated crack width

$$w = \varepsilon_{sm\_cm} \cdot s_{r.max} = 1.3 \cdot 232 = 0.303 \text{ mm} \quad (7.8)$$

Limit value of crack width

w_{max} = 0.4 mm

Unity check

Calculation unity check

$$UC = \frac{w}{w_{max}} = \frac{0.303 \text{ mm}}{0.4 \text{ mm}} = \mathbf{0.757}$$

Check crack width

w = 0.303 mm = < w_{max} = 0.4 mm

Note: Check crack width satisfies, because the crack width is lesser than limit value.

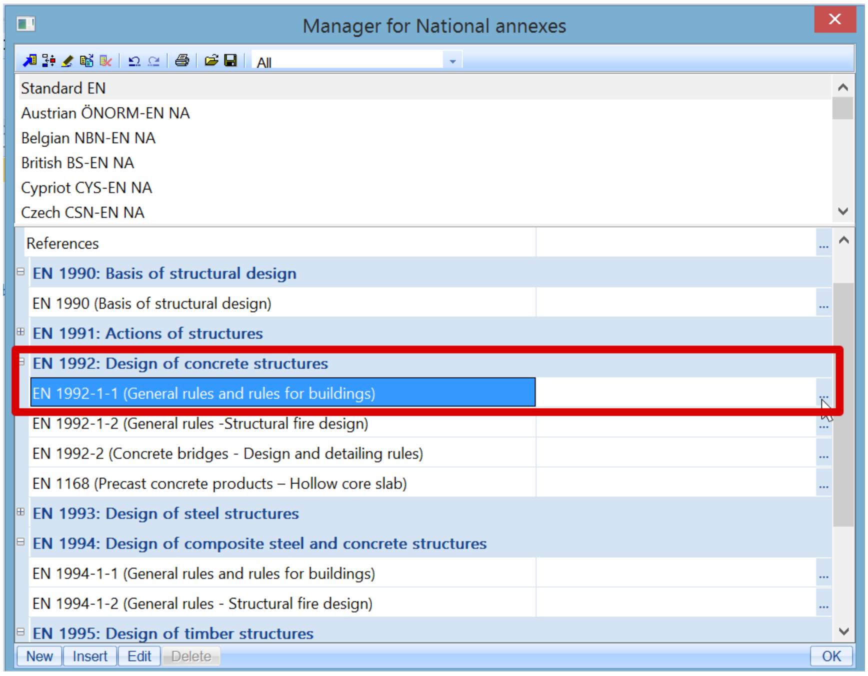
Standard output will give the summary values:

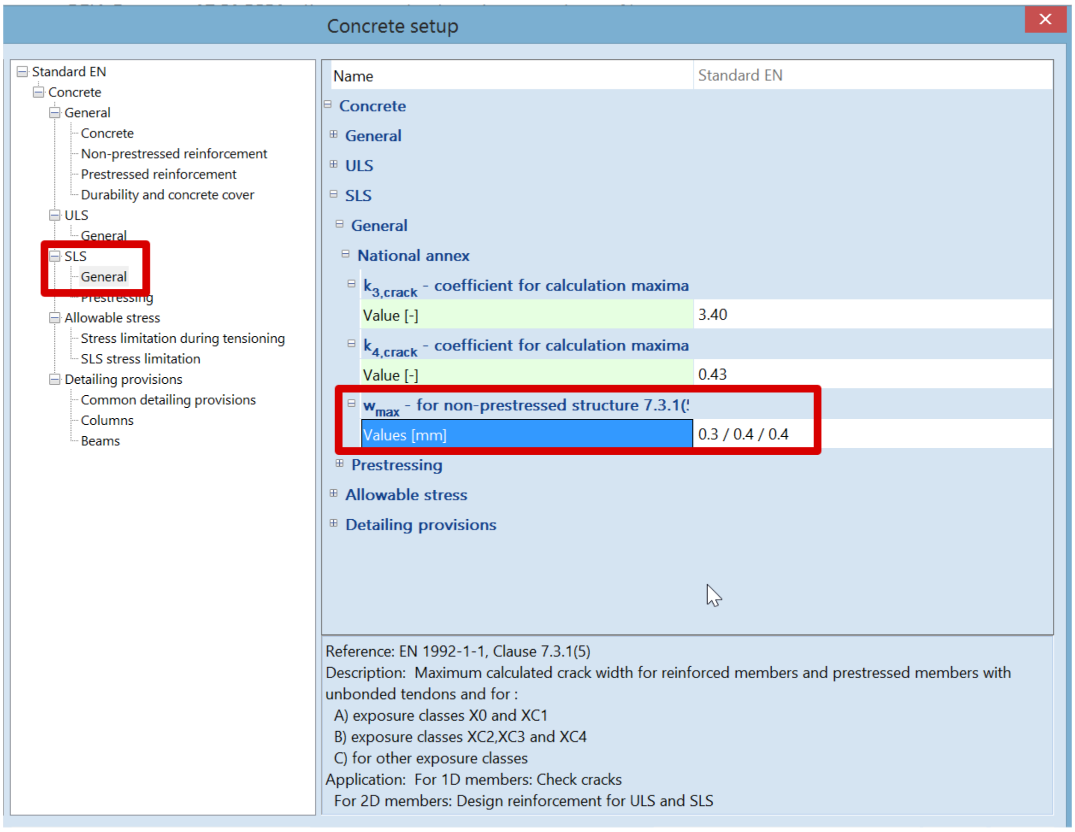
Summary of check

N_{cr} = 0 kN M_{cry} = -43.3 kN M_{crz} = 0 kN σ_s = 300 MPa $s_{r.max}$ = 232 mm ε_{sm_cm} = 1.3 ‰

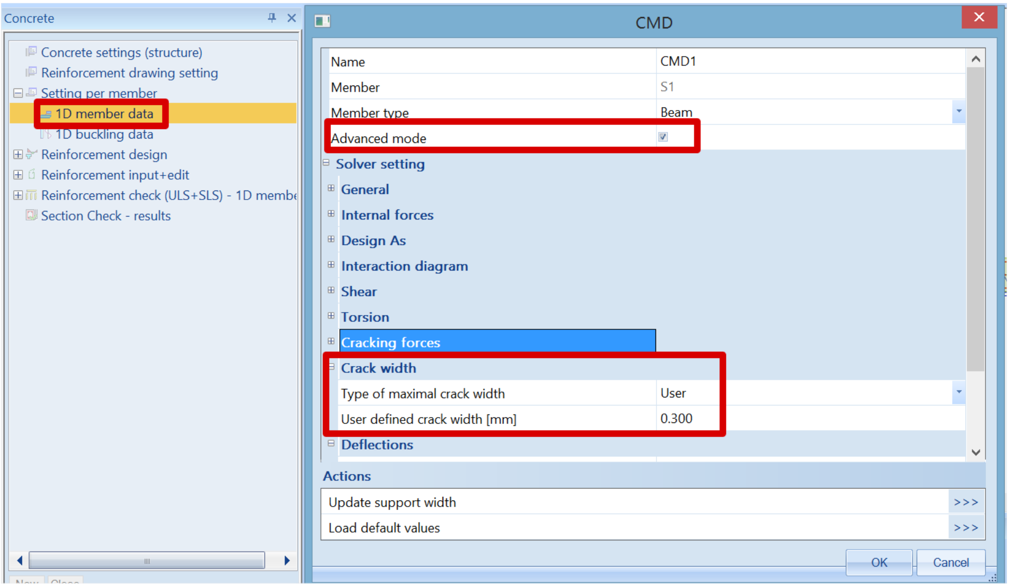
σ_{ct} [MPa]	σ_{cr} [MPa]	Cracked	w [mm]	w_{lim} [mm]	UC [-]	Limit check [-]	Status
12.6	2.9	YES	0.303	0.4	0.76	1	OK

The limit value of the crack width w_{max} is by default automatically calculated according to EN 1992-1-1 (Table 7.1N). The allowable crack width can be seen in the NA setup:





The user can manually input the limiting crack width in Settings per member > 1D member data:



Deflection

The calculation of deflection is done according to chapter 7.4.3 from EN 1992-1-1.
Two kinds of deflection calculations are possible in the software:

- Simplified method where the calculation is done twice, assuming the whole member to be uncracked and fully cracked, and then interpolating formula 7.18 according to clause 7.4.3(7). This is the default used method.
- Code dependent deflection. This is the most rigorous method to calculate deflection by computing the calculation of curvatures at frequent sections along the member and then calculate the deflection by numerical integration. This method is available if additional functionalities are activated.

Simplified method

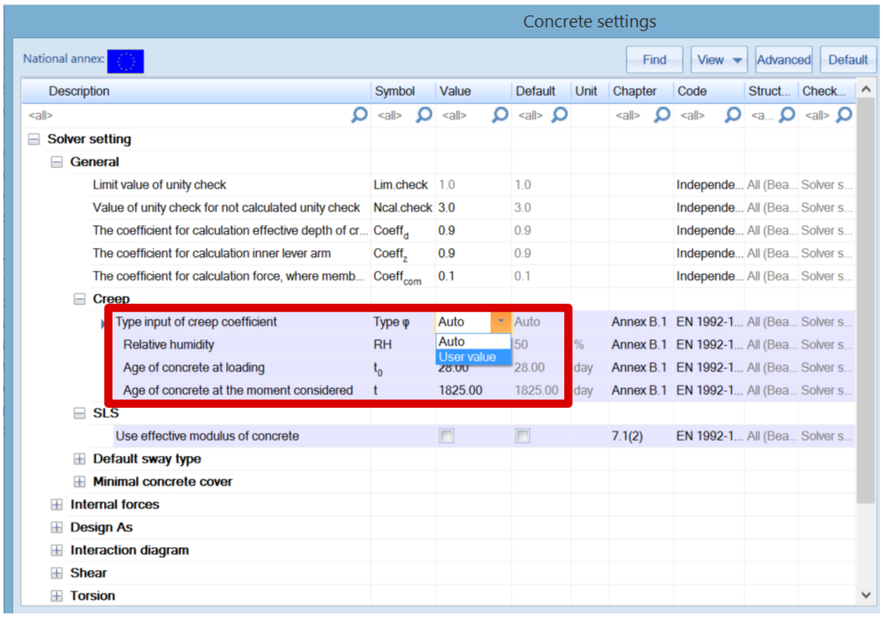
The calculation procedure for the simplified method can be described in the following steps:

1. **Calculation of short-term stiffness** using E modulus at 28 days.
2. **Calculation of long-term stiffness** using effective E modulus based on creep coefficient.

In the current version of the software, 17.1, it is not possible to distinguish between the short-term and long-term part of the load in a combination. Therefore, some preconditions have been established for determination of the long-term part of the load. The long-term part of the load (LongTermPercentage) is estimated based on the type of combination. There are three main SLS combinations:

SLS characteristics - LongTermPercentage = 70 %
SLS frequent - LongTermPercentage = 85 %
SLS quasi-permanent- LongTermPercentage = 100 %

The creep-factor is calculated by the software depending on the relative humidity, outline of the cross-section, reinforcement percentage, concrete class, etc. It can also be manually inputted in the Concrete setup > General > Creep:



3. **Calculation of stiffness ratios** between each state, short and long term.
   It is the ratio of linear stiffness of the concrete component divided by the resultant stiffness taking cracks into account. The calculation of resultant stiffness is based on clause 7.4.3 (3),

formula 7.18. For additional information about the resultant stiffness, please refer to our webhelp or to the Concrete menu > Reinforcement check > Stiffnesses.

ratio = $Stiffness_{lin}$ / $Stiffness_{res}$, for example $ratio_{uz} = EI_{z,lin} / EI_{z,res}$

4. **Calculation of deflection components**

Several components are needed to calculate the total and additional deflection.

In the following part we will note “s” for short term and “l” for long term.

The components are:
δ_{lin} linear (elastic) deflection, $\delta_{lin} = \delta_{lin,s} + \delta_{lin,l}$
δ_{imm} immediate deflection, $\delta_{imm} = \delta_{lin,l} \cdot ratio_s$
δ_s short-term deflection, $\delta_s = \delta_{lin,s} \cdot ratio_s$
$\delta_{l,creep}$ long-term deflection + creep, $\delta_{l,creep} = \delta_{lin,l} \cdot ratio_l$
δ_{creep} creep deflection, $\delta_{creep} = \delta_{lin,l} \cdot (ratio_l - ratio_s)$
δ_l long-term deflection, $\delta_l = \delta_{l,creep} - \delta_{creep}$
δ_{add} additional deflection, $\delta_{add} = \delta_s + \delta_{l,creep} - \delta_{imm}$
δ_{tot} total deflection, $\delta_{tot} = \delta_s + \delta_{l,creep}$

5. **Check of deflections**

Two deflections are checked:
Total deflection: The appearance and general utility of the structure could be impaired when the calculated sag of a beam, slab or cantilever subjected to quasi-permanent loads exceeds span/250.
$\delta_{tot,lim} = L / 250$

Additional deflection: Deflections that could damage adjacent parts of the structure should be limited.

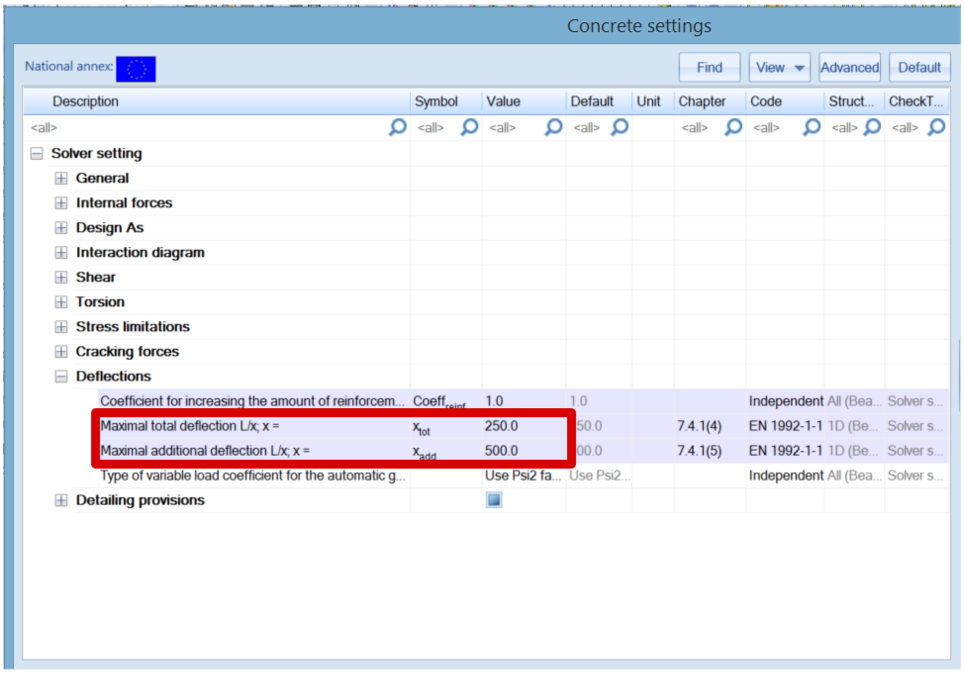
$\delta_{add,lim} = L / 500$

L is the buckling length multiplied by a β factor of the member in the corresponding direction.

Final unity check is:

$$Unity\ check\ =\ max\left\{\frac{\delta tot}{\delta tot, lim}; \frac{\delta add}{\delta add, lim}\right\}$$

The limits of deflection can be changed in Concrete setup > Deflections:



Example: 'beam_practical reinforcement.esa'

Look at deflection check for the "SLS qp" combination.
Various results can be displayed on the screen: UC, total and additional deflection or limits for total and additional deflection.

Open the Standard output for the UC. At position dx = 2.5m we have the following result:

Basic values of deflections

Type of deflection	Ratio short [-]	Ratio long [-]	δ_{lin} [mm]	δ_{imm} [mm]	δ_{add} [mm]	δ_{short} [mm]	δ_{long} [mm]	$\delta_{long+creep}$ [mm]	δ_{creep} [mm]
u_y	2.88	5.22	0	0	0	0	0	0	0
u_z	2.5	3.38	-3.08	-7.7	-2.69	0	-7.7	-10.4	-2.69

Check of additional and total deflections

Type of deflection	L [m]	δ_{add} [mm]	$\delta_{add,lim}$ [mm]	UC_{add} [-]	δ_{tot} [mm]	$\delta_{tot,lim}$ [mm]	UC_{tot} [-]	UC [-]	Limit [-]	Status
u_y	10	0	0	0	0	0	0	0	1	OK
u_z	10	-2.69	-20	0.13	-10.4	-40.1	0.26	0.26	1	OK

List of errors/warnings/notes: NO

All ratio of stiffnesses and deflection components are resumed in a table.

Open the Detailed output, for the same position dx = 2.5m.
All previously mentioned steps for the calculation of the deflections can be found here.

For example for the long-term stiffness, we can obtain the long-term part of the loads and the calculated creep coefficient:

Long-term stiffnesses and curvatures under total load

Settings

Long-term part of applied load = 100%

Creep coefficient $\varphi = 2.21$

Uncracked (state I) and cracked (state II) cross section properties are also shown in a table:

Cross-section characteristics

Type of component	t_y [m]	t_z [m]	A [m^2]	I_y [m^4]	I_z [m^4]	x_i [m]	A_{st} [m^2]	A_{sc} [m^2]	A_s [m^2]
Linear	0	0	0.15	$3.13 \cdot 10^{-3}$	$1.13 \cdot 10^{-3}$	0.25	-	-	-
Uncracked	0	-0.019	0.193	$4.69 \cdot 10^{-3}$	$1.35 \cdot 10^{-3}$	0.27	$1.57 \cdot 10^{-3}$	$628 \cdot 10^{-6}$	$2.2 \cdot 10^{-3}$
Cracked	0	0.053	0.102	$2.89 \cdot 10^{-3}$	$667 \cdot 10^{-6}$	0.197	$1.57 \cdot 10^{-3}$	$628 \cdot 10^{-6}$	$2.2 \cdot 10^{-3}$

Check of concrete stresses and calculation of cracking forces

Maximal tensile stress in concrete fibre

$\sigma_{ct} = 7.76$ MPa

Cracking status

$\sigma_{ct} > f_{ct,eff} = 7.76 \text{ MPa} > 2.9 \text{ MPa}$ => Cracks appear.

Stress in reinforcement for cracking load

$\sigma_{sr} = 99.3$ MPa

Stress in reinforcement for acting load

$\sigma_s = 262$ MPa

Distribution coefficient

$$\zeta = \max\left(0; 1 - \beta \cdot \left(\frac{\sigma_{sr}}{\sigma_s}\right)^2\right) = \max\left(0; 1 - 0.5 \cdot \left(\frac{99.3}{262}\right)^2\right) = 0.928 \qquad (7.19)$$

N_{cr} [kN]	$M_{y,cr}$ [kNm]	$M_{z,cr}$ [kNm]	σ_{ct} [MPa]	$f_{ct,eff}$ [MPa]	Cracked section	σ_{sr} [MPa]	σ_s [MPa]	β [-]	ζ [-]	E_c [GPa]
0	59.6	0	7.76	2.9	YES	99.3	262	0.5	0.928	33

Which allows to calculate the stiffness's ratio, for example the bending stiffness's ratio:

Bending stiffness EIy

$EI_{y,lin} = E_c \cdot I_y = 33 \cdot 3.13 \cdot 10^9 = 103 \text{ MNm}^2$

$EI_{y,I} = E_{c,eff} \cdot I_{y,I} = 10.3 \cdot 4.69 \cdot 10^9 = 48.1 \text{ MNm}^2$

$EI_{y,II} = E_{c,eff} \cdot I_{y,II} = 10.3 \cdot 2.89 \cdot 10^9 = 29.7 \text{ MNm}^2$

$$EI_y = \frac{1}{\frac{\zeta}{EI_{y,II}} + \frac{1-\zeta}{EI_{y,I}}} = \frac{1}{\frac{0.928}{29.7} + \frac{1-0.928}{48.1}} = 30.5 \text{ MN} \cdot \text{m}^2 \qquad (7.18)$$

$$RatioEIy = \frac{EI_y}{EI_{y,lin}} = \frac{30.5}{103} = 0.296$$

Bending stiffness EIz

$EI_{z,lin} = E_c \cdot I_z = 33 \cdot 1.13 \cdot 10^9 = 37.1 \text{ MNm}^2$

$EI_{z,I} = E_{c,eff} \cdot I_{z,I} = 10.3 \cdot 1.35 \cdot 10^9 = 13.8 \text{ MNm}^2$

$EI_{z,II} = E_{c,eff} \cdot I_{z,II} = 10.3 \cdot 667 \cdot 10^6 = 6.85 \text{ MNm}^2$

$$EI_z = \frac{1}{\frac{\zeta}{EI_{z,II}} + \frac{1-\zeta}{EI_{z,I}}} = \frac{1}{\frac{0.928}{6.85} + \frac{1-0.928}{13.8}} = 7.11 \text{ MN} \cdot \text{m}^2 \qquad (7.18)$$

$$RatioEIz = \frac{EI_z}{EI_{z,lin}} = \frac{7.11}{37.1} = 0.191$$

And final the short and long-term ratios:

Short-term ratios

Bending stiffness Ely

$$RatioElys = \frac{EI_{y,s}}{EI_{y,lin}} = \frac{41.2 \cdot 10^6}{103 \cdot 10^6} = 0.4$$

Bending stiffness Elz

$$RatioElzs = \frac{EI_{z,s}}{EI_{z,lin}} = \frac{12.9 \cdot 10^6}{37.1 \cdot 10^6} = 0.347$$

Ratios

$$ratio_{uys} = \frac{1}{RatioElzs} = \frac{1}{0.347} = 2.88$$

$$ratio_{uzs} = \frac{1}{RatioElys} = \frac{1}{0.4} = 2.5$$

Long-term ratios

Bending stiffness Ely

$$RatioElyl = \frac{EI_{y,l}}{EI_{y,lin}} = \frac{30.5 \cdot 10^6}{103 \cdot 10^6} = 0.296$$

Bending stiffness Elz

$$RatioElzl = \frac{EI_{z,l}}{EI_{z,lin}} = \frac{7.11 \cdot 10^6}{37.1 \cdot 10^6} = 0.191$$

Ratios

$$ratio_{uyl} = \frac{1}{RatioElzl} = \frac{1}{0.191} = 5.22$$

$$ratio_{uzl} = \frac{1}{RatioElyl} = \frac{1}{0.296} = 3.38$$

Then all deflection components are calculated together with the limit deflections:

Deflections

Linear deflection

$$\delta_{lin,y} = u_{ys} + u_{yl} = 0 + 0 = 0 \text{ mm}$$

$$\delta_{lin,z} = u_{zs} + u_{zl} = 0 + -3.08 = -3.08 \text{ mm}$$

Immediate deflection

$$\delta_{imm,y} = u_{yl} \cdot ratio_{uys} = 0 \cdot 2.88 = 0 \text{ mm}$$

$$\delta_{imm,z} = u_{zl} \cdot ratio_{uzs} = -3.08 \cdot 2.5 = -7.7 \text{ mm}$$

Short-term deflection

$$\delta_{short,y} = u_{ys} \cdot ratio_{uys} = 0 \cdot 2.88 = 0 \text{ mm}$$

$$\delta_{short,z} = u_{zs} \cdot ratio_{uzs} = 0 \cdot 2.5 = 0 \text{ mm}$$

Long-term + creep deflection

$$\delta_{long,creep,y} = u_{yl} \cdot ratio_{uyl} = 0 \cdot 5.22 = 0 \text{ mm}$$

$$\delta_{long,creep,z} = u_{zl} \cdot ratio_{uzl} = -3.08 \cdot 3.38 = -10.4 \text{ mm}$$

Creep deflection

$$\delta_{creep,y} = u_{yl} \cdot \left(ratio_{uyl} - ratio_{uys} \right) = 0 \cdot \left(5.22 - 2.88 \right) = 0 \text{ mm}$$

$$\delta_{creep,z} = u_{zl} \cdot \left(ratio_{uzl} - ratio_{uzs} \right) = -3.08 \cdot \left(3.38 - 2.5 \right) = -2.69 \text{ mm}$$

Long-term deflection

$$\delta_{long,y} = \delta_{long,creep,y} - \delta_{creep,y} = 0 - 0 = 0 \text{ mm}$$

$$\delta_{long,z} = \delta_{long,creep,z} - \delta_{creep,z} = -10.4 - -2.69 = -7.7 \text{ mm}$$

Additional deflection

$$\delta_{add,y} = \delta_{short,y} + \delta_{long,creep,y} - \delta_{imm,y} = 0 + 0 - 0 = 0 \text{ mm}$$

$$\delta_{add,z} = \delta_{short,z} + \delta_{long,creep,z} - \delta_{imm,z} = 0 + -10.4 - -7.7 = -2.69 \text{ mm}$$

Limit additional deflection

$$\delta_{add,lim,y} = 0 \text{ mm}$$

$$\delta_{add,lim,z} = \frac{-l_{0z}}{Lim_{add}} = \frac{-10}{500} = -20 \text{ mm}$$

Total deflection

$$\delta_{tot,y} = \delta_{short,y} + \delta_{long,creep,y} = 0 + 0 = 0 \text{ mm}$$

$$\delta_{tot,z} = \delta_{short,z} + \delta_{long,creep,z} = 0 + -10.4 = -10.4 \text{ mm}$$

Limit total deflection

$$\delta_{tot,lim,y} = 0 \text{ mm}$$

$$\delta_{tot,lim,z} = \frac{-l_{0z}}{Lim_{tot}} = \frac{-10}{250} = -40 \text{ mm}$$

Limitations of the deflection check:

- Deformation caused by shrinkage is not automatically considered.
- Verification based on limiting span / depth ratio according to 7.4.2 is not implemented.
- Calculation of deflection depends on the internal forces used for the reduced stiffness. Therefore, the check of deflection doesn't work for cases where the internal forces are equal to zero but deflections are not zero. Typically, this is the case for a cantilever structure with free overhang.

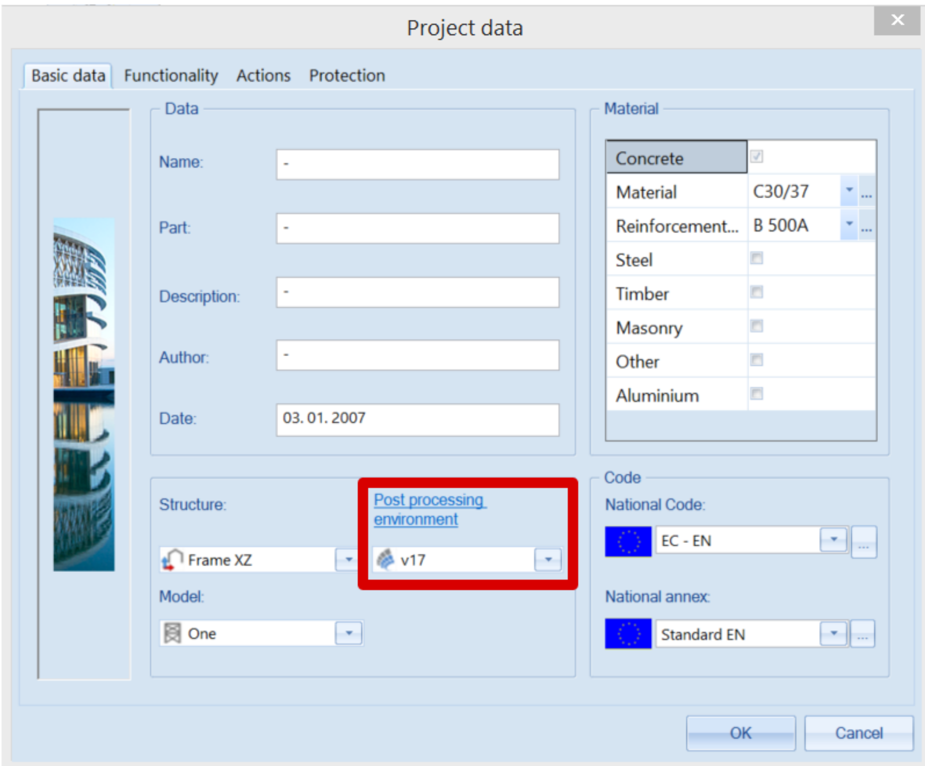
Code dependent deflection CDD

The CDD calculation is a more rigorous calculation of the deflection. The calculation procedure is the same as for the simplified method, but with following differences:

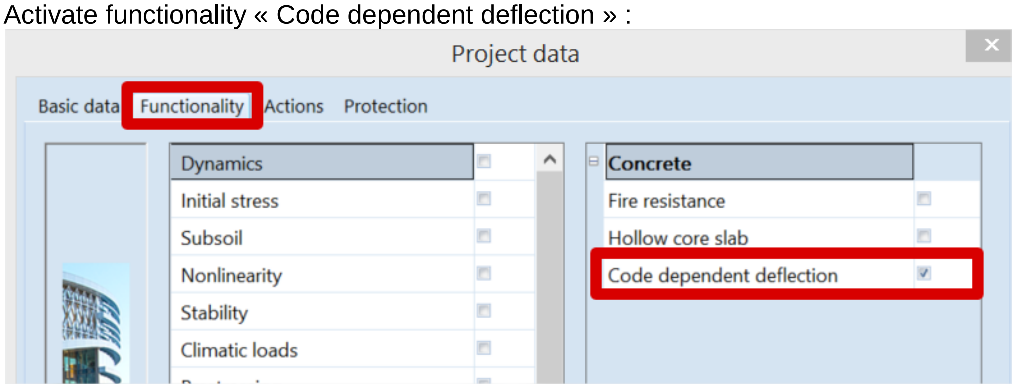
- 3 types of combinations are used to calculate the deflections
- Calculation of stiffness is more precise

To be able to use this method in SCIA Engineer, the following settings should be set beforehand:

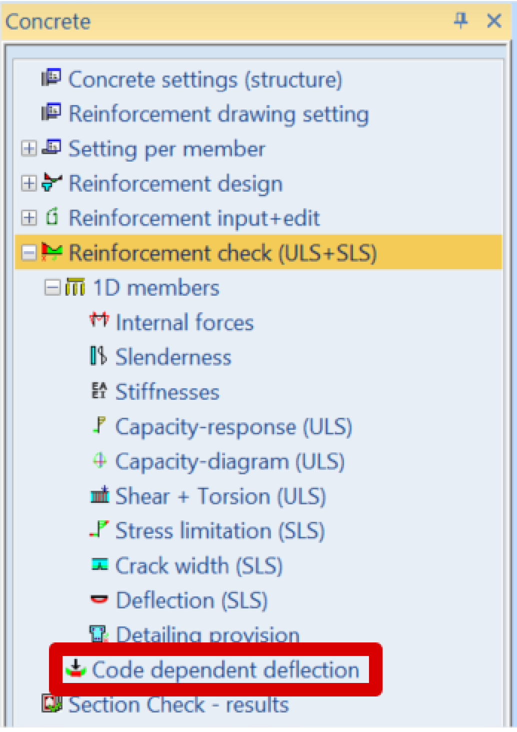
1. Use the post processing environment v17 in the Project menu:



2. Activate functionality « Code dependent deflection » :



3. In the Concrete menu > Reinforcement check, you will then see a new check named Code dependent deflection:

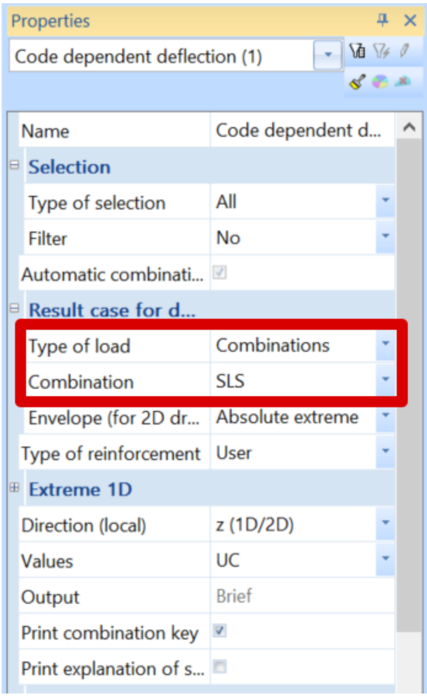


Types of combinations

Three different combinations are automatically created by the software in the background to calculate the deflection:

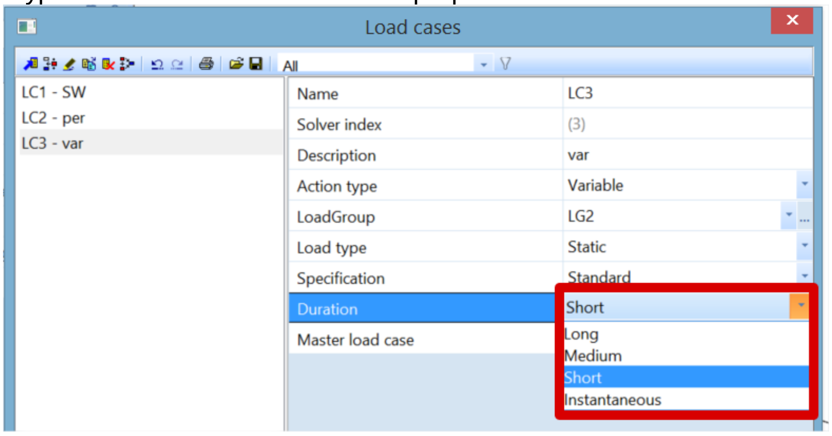
1. Combination for calculation of total deflection

   Generated directly from the user choice of combination in the CDD check, properties window:



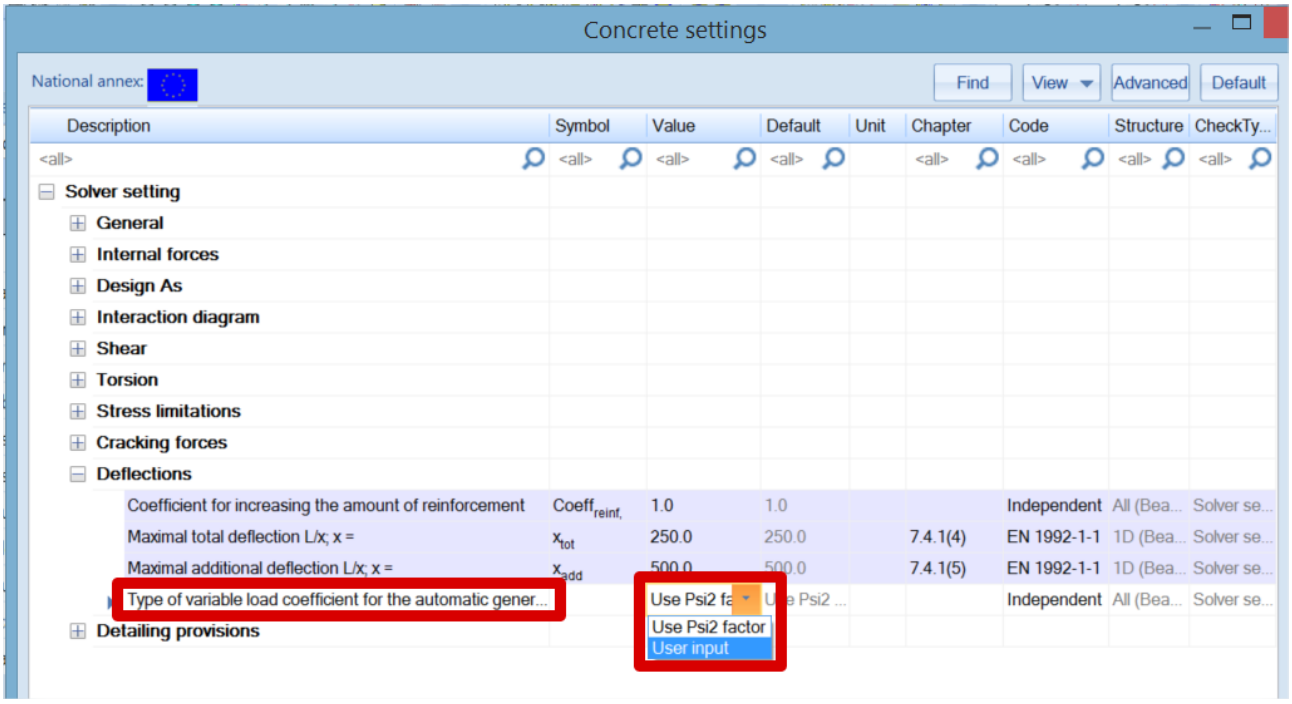
2. Combination for calculation of immediate deflection

   Uses the generated combination for total deflection and removes variable load cases with duration type Medium, Short or Instantaneous.
   Duration type is defined in the Load cases properties:



3. Combination for calculation of deflection due to creep

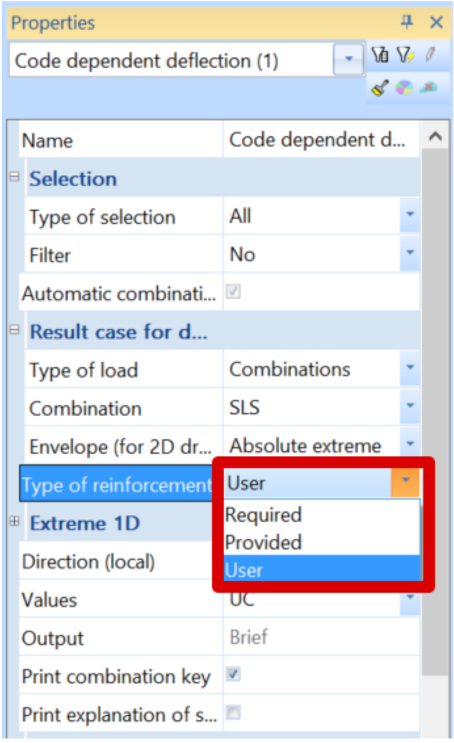
   Uses the generated combination for total deflection and multiplies variable load cases by a coefficient defined in Concrete settings > Deflections:



Additional characteristic combinations are generated for each previously mentioned combination to determine if the section is cracked or uncracked.

Types of reinforcement

For the CDD method, it is possible to calculate the deflection with required, provided or user inputted reinforcement. This choice is done in the Properties window of the CDD check:



Calculation of deflection

The following deflections are calculated in the CDD check:

δ_{lin} linear (elastic) deflection, calculated for the total combination and for linear stiffness.

δ_{imm} immediate deflection, the deflection after applying permanent and long-term variable loads which means calculated for short-term stiffness and immediate combination

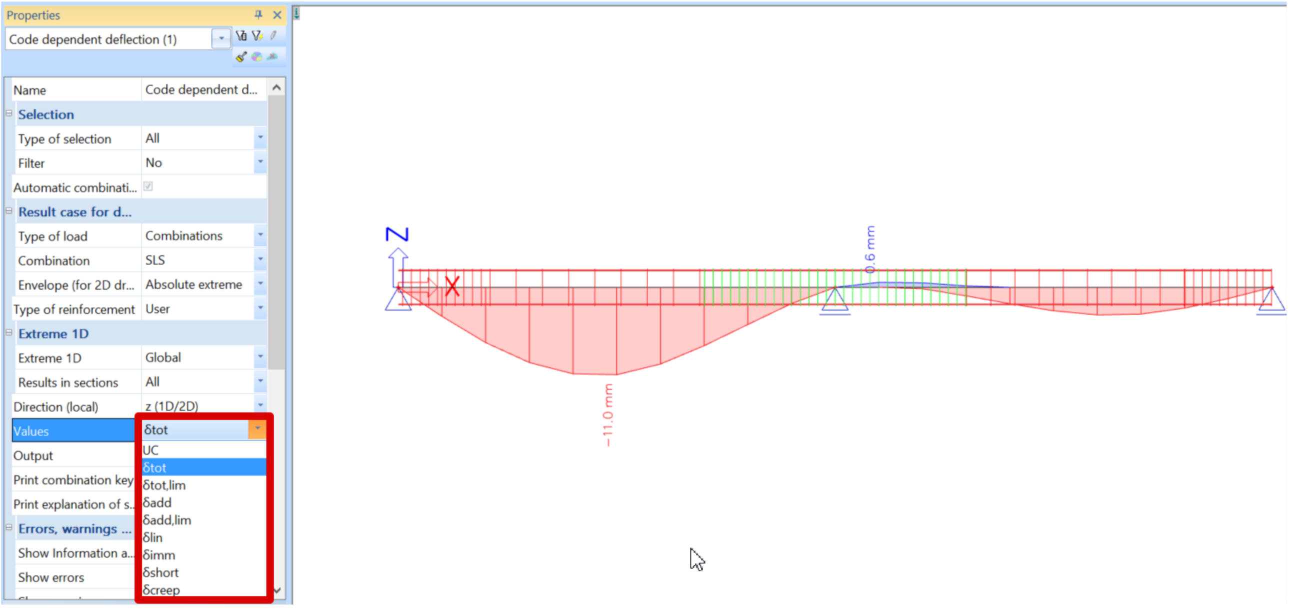
δ_{short} short-term deflection, the deflection which considers cracking of cross-section calculated for short-term stiffness and total combination

δ_{creep} creep deflection, calculated as the difference between deflection calculated for long-term and short-term stiffness for the creep combination. $\delta_{creep} = \delta_{creep, long} - \delta_{creep, short}$

δ_{add} additional deflection, the deflection after applying a variable load and considering creep calculated as the difference between total and immediate deflection. $\delta_{add} = \delta_{tot} - \delta_{imm}$

δ_{tot} total deflection, the deflection which considers creep and cracking calculated as the sum of short-term deflection and deflection caused by creep. $\delta_{tot} = \delta_{short} + \delta_{creep}$

All those values can be displayed on the screen:



For the CDD check, only a brief output is available in 17.1:

For 1D member

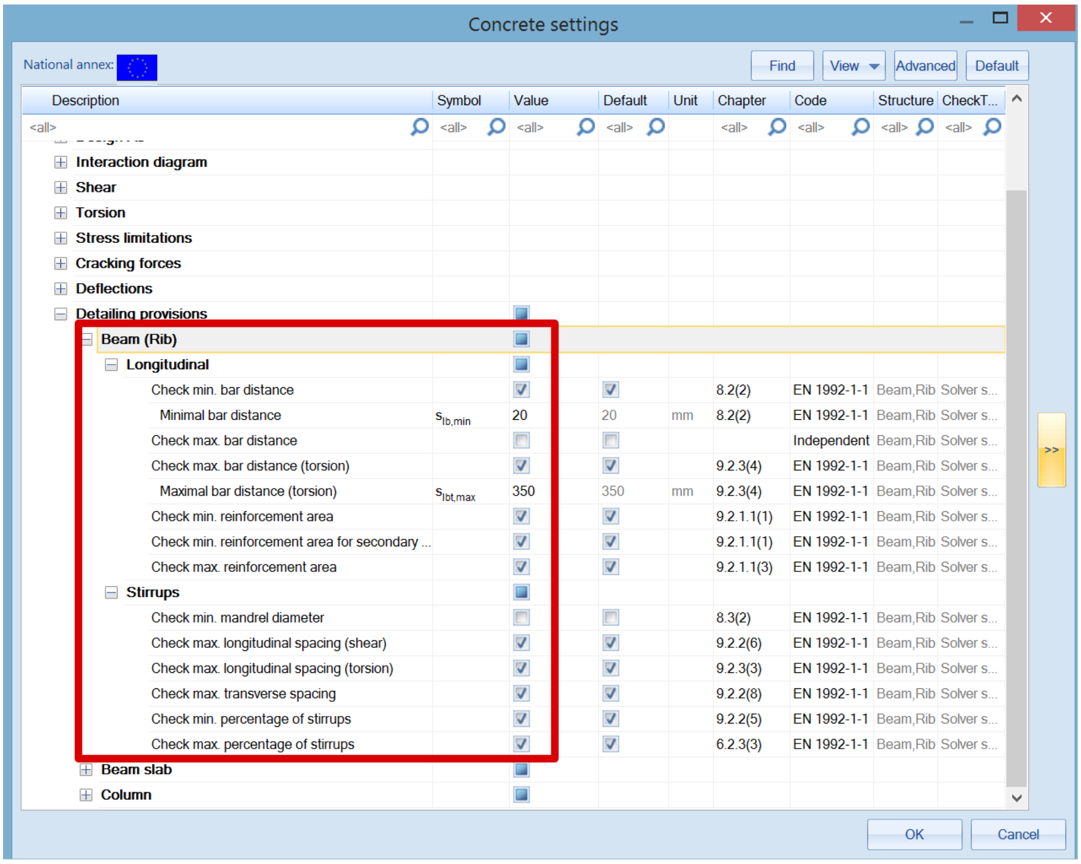
Name	dx [m]	Case Type of reinf.	$\delta_{lin,y}$ [mm] $\delta_{lin,z}$ [mm]	$\delta_{imm,y}$ [mm] $\delta_{imm,z}$ [mm]	$\delta_{short,y}$ [mm] $\delta_{short,z}$ [mm]	$\delta_{creep,y}$ [mm] $\delta_{creep,z}$ [mm]	$\delta_{add,y}$ [mm] $\delta_{add,z}$ [mm]	$\delta_{add,lim,y}$ [mm] $\delta_{add,lim,z}$ [mm]	$\delta_{tot,y}$ [mm] $\delta_{tot,z}$ [mm]	$\delta_{tot,lim,y}$ [mm] $\delta_{tot,lim,z}$ [mm]	UC [-] Check
S1	2.500-	SLS/1 User	0.0 -3.4	0.0 -6.8	0.0 -8.0	0.0 -3.0	0.0 -4.2	20.0 10.0	0.0 **-11.0**	40.0 20.0	0.55 OK
S1	5.500-	SLS/2 User	0.0 0.2	0.0 0.5	0.0 0.5	0.0 0.1	0.0 0.1	20.0 10.0	0.0 **0.6**	40.0 20.0	0.03 OK

Detailing provisions

Scia Engineer distinguishes three types of member with their detailing provisions:

- Beam - verification of longitudinal and shear reinforcement
- Column - verification of main and transverse reinforcement
- Beam slab - verification of longitudinal reinforcement only

All detailing provisions are taken into account automatically in Concrete settings > Detailing provisions:



Following table shows which checks of detailing provisions are performed:

Member type	Longitudinal (main)	Shear (transverse)
Beam	8.2(2) - Minimal clear spacing of bars 9.2.1.1(1) - Minimal area of longitudinal reinforcement 9.2.1.1(3) - Maximal area of longitudinal reinforcement 9.2.3(4) - Maximal center-to-center bar distance based on torsion Code-Independent - Maximal clear spacing	6.2.3(3) - Maximal percentage of shear reinforcement 9.2.2(5) - Minimal percentage of shear reinforcement 9.2.2(6) - Maximal longitudinal spacing of stirrups (shear) 9.2.2(8) - Maximal transverse spacing of stirrups (shear) 9.2.3(3) - Maximal longitudinal spacing of stirrups (torsion)
Column	8.2(2) - Minimal clear spacing of bars 9.5.2(1) - Minimal bar diameter of longitudinal reinforcement 9.5.2(2) - Minimal area of longitudinal reinforcement 9.5.2(3) - Maximal area of longitudinal	9.2.3(3) - Maximal longitudinal spacing of stirrups (torsion) 9.5.3(1) - Minimal diameter of transverse reinforcement 9.5.3(3) - Maximal longitudinal spacing of transverse reinforcement

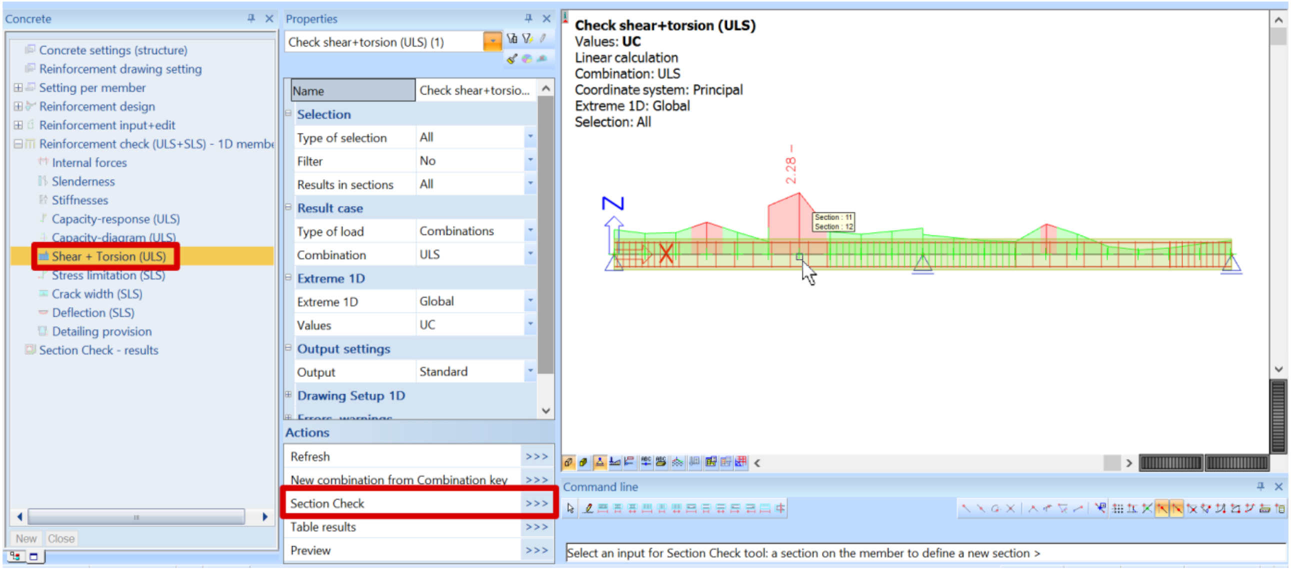
Member type	Longitudinal (main)	Shear (transverse)
	reinforcement 9.5.2(4) - Minimal number of longitudinal reinforcement bars	
Beam Slab	8.2(2) - Minimal clear spacing of bars 9.3.1.1(3) - Maximal bar distance of longitudinal reinforcement	-

Section check

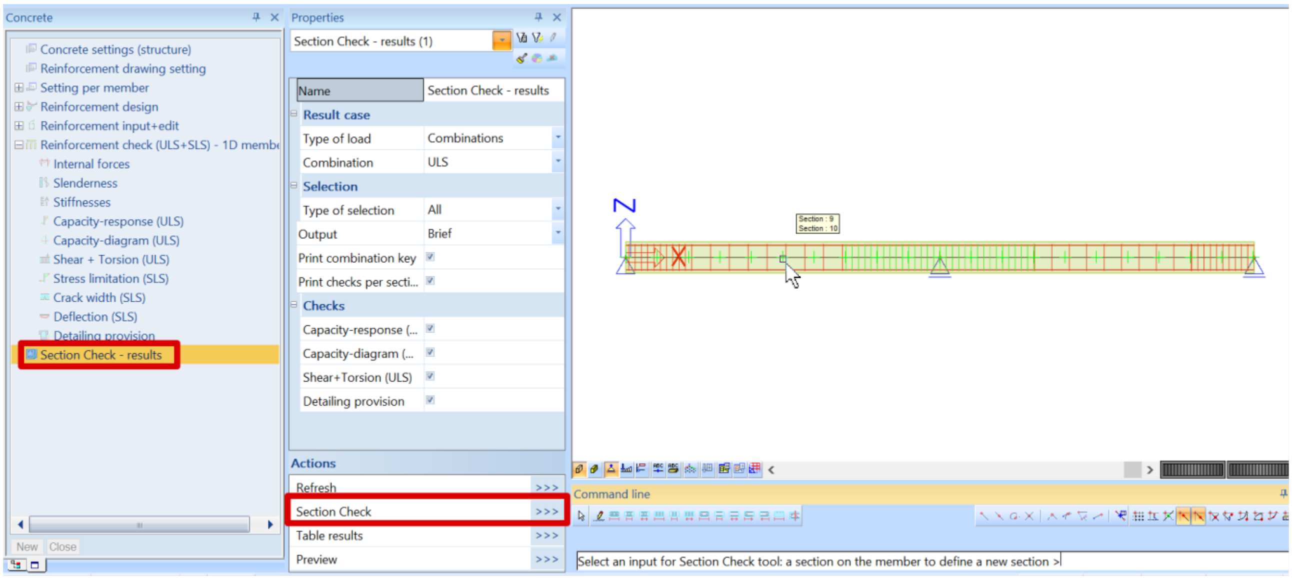
The Section check tools can be used in two different ways: with or without practical reinforcement inputted beforehand.

Section check can be launched:

- In the properties window for an individual check



- In the properties window for the Section Check – results service



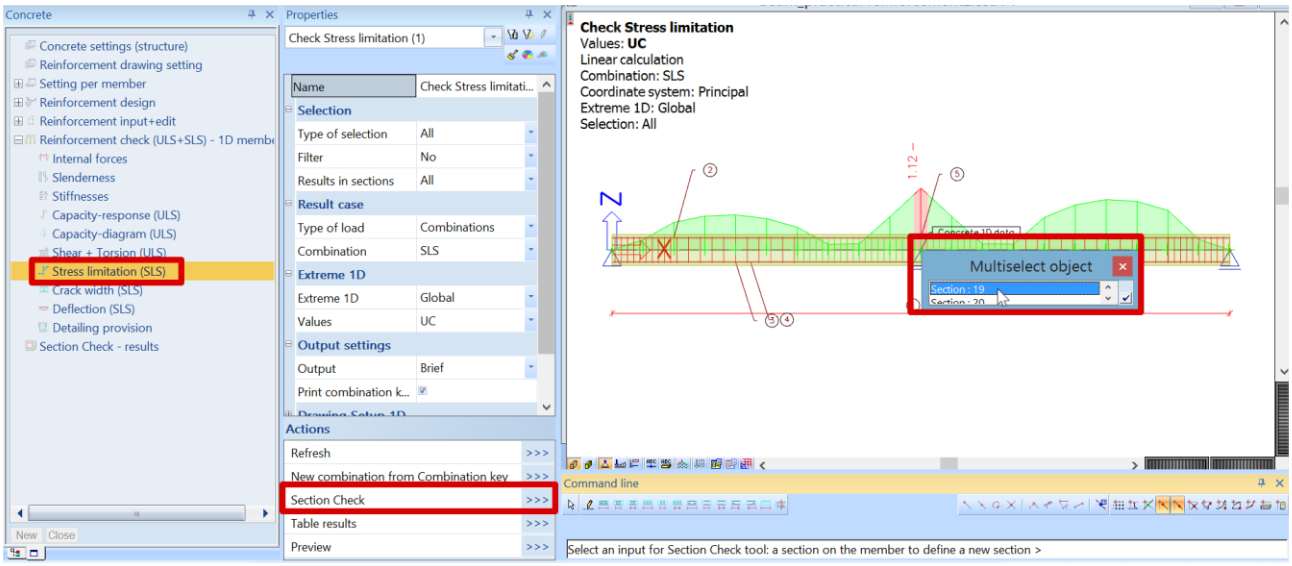
With practical reinforcement

Example 1: 'beam_practical reinforcement SC.esa'

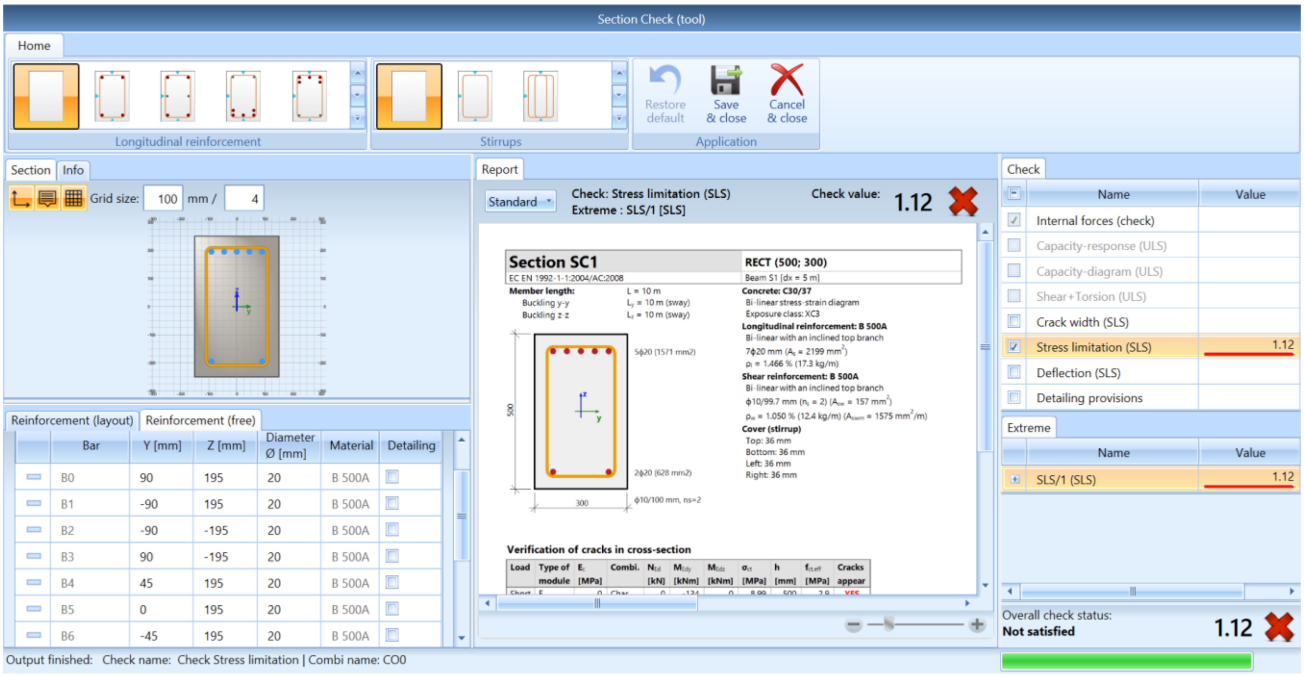
Section check can be opened from all individual checks.

In this example, select Stress limitation check (SLS) and click on "Section check" in the Properties window:

Select the beam and then click on the position for which the check should be done. Choose section 19 at the middle of the beam:



The Section check tool opens:



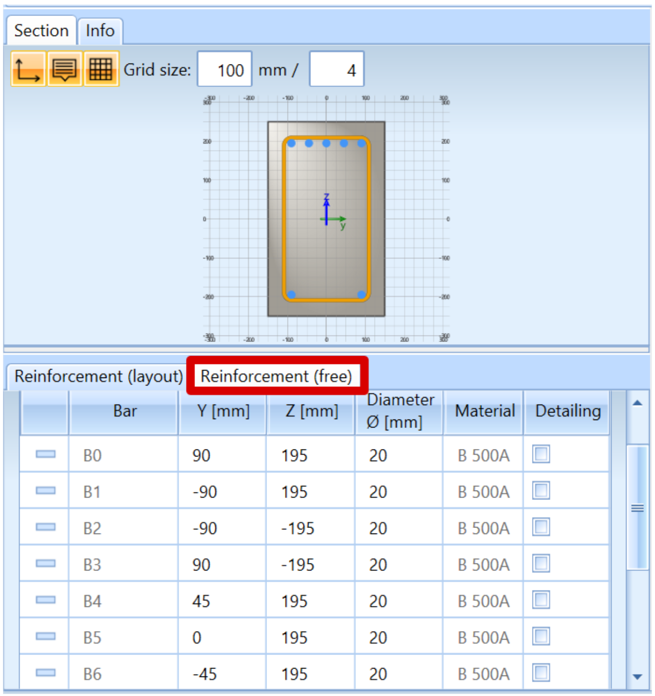
This window is composed of 3 mains parts:

1. Definition / modification of the reinforcement
2. Preview of the report
3. Checks to be performed according to the previous selected combinations or load cases. By default, only the individual selected check will be performed. The user can activate more checks if wanted.

When selecting a SLS combination in the Properties windows, only SLS checks will be available. When selecting a ULS combination in the Properties windows, only ULS checks will be available.

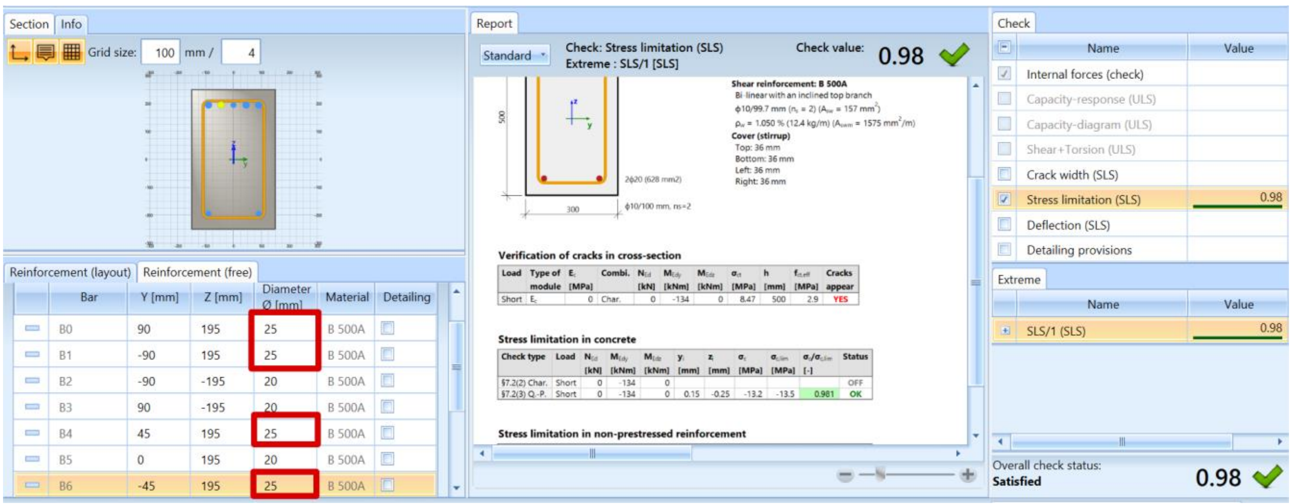
In this example, stress limitation in the concrete is not OK. One solution is to redesign the longitudinal reinforcement to satisfy the SLS stress limitations. We could then close the Section check tool and change the practical reinforcement for this beam or we can adapt locally the reinforcement in the studied section (Section 19). We will choose to adapt the reinforcement in the Section check tool itself.

When practical reinforcement was already inputted, it can be edited in the tab “Reinforcement (free)”:



Each present bar, position and diameter, is listed in the table. They can be modified, deleted or new bars can be added.

Increase the diameter of top layer bars B0, B1, B4 and B6 from 20mm to 25mm:



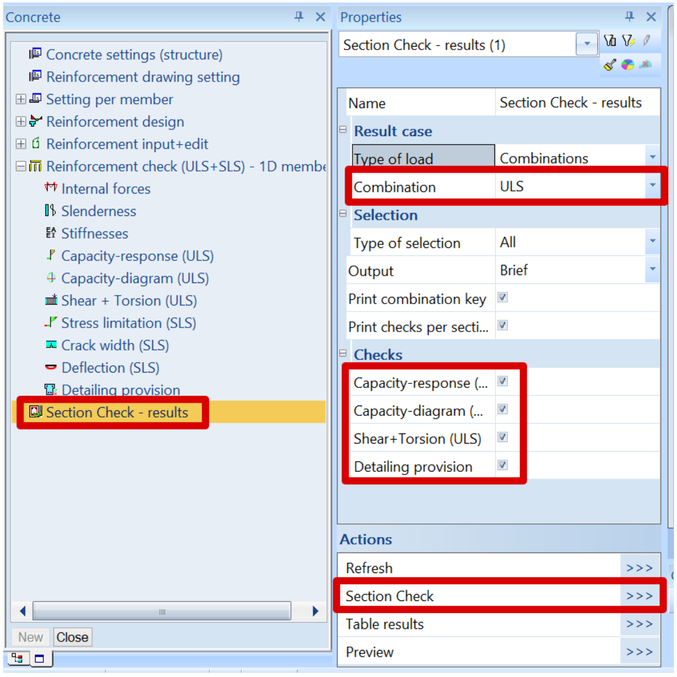
The stress limitation check is now satisfied.

Without practical reinforcement

Example 2: 'beam_without practical reinforcement SC.esa'

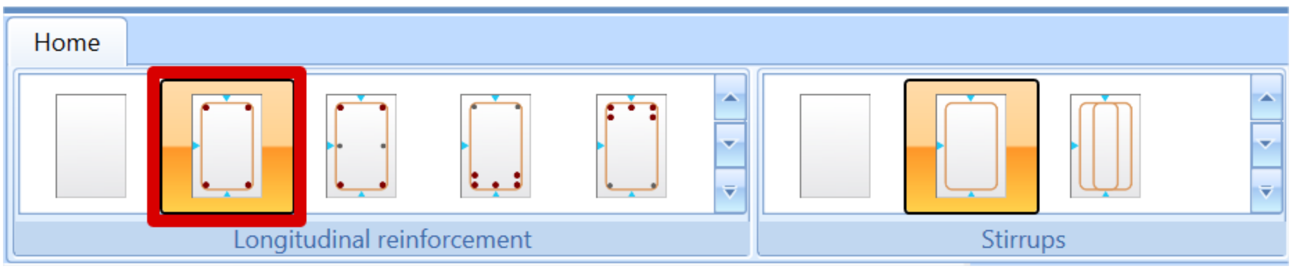
When no practical reinforcement was inputted beforehand, it is possible to run the section check tool in order to check a specific section of a member with a local reinforcement on this specific section.

In the Concrete tree, select "Section check – results".
In the properties window, choose the ULS combination to perform all ULS checks:

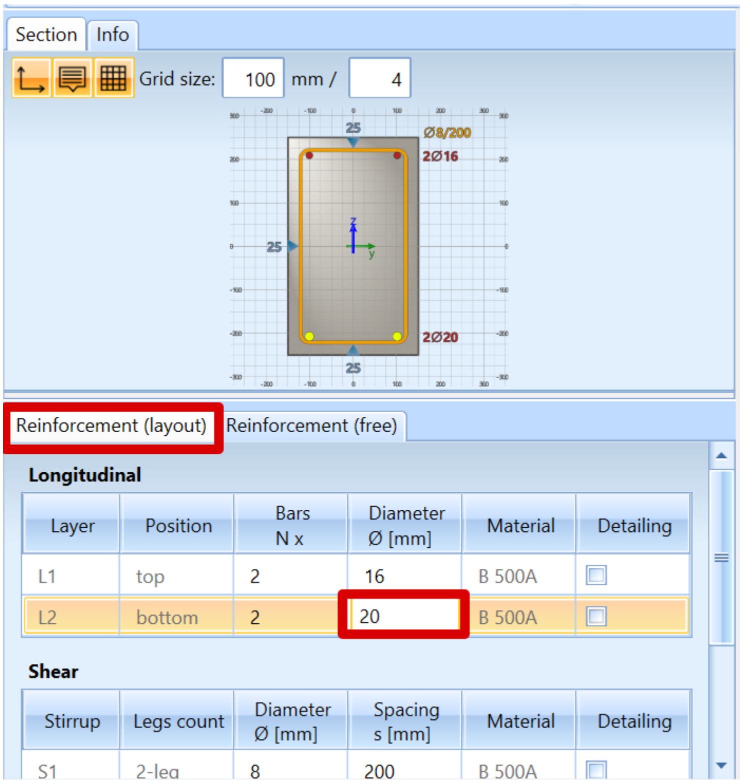


Select Section 9, in the middle of the first span.
All checks are not satisfied, and the overall UC is 3. The value 3 means that the check could not be performed due to an error in the calculation. In this case, it is because there is no reinforcement yet.

We will start by inserting the reinforcement. First choose the reinforcement template:

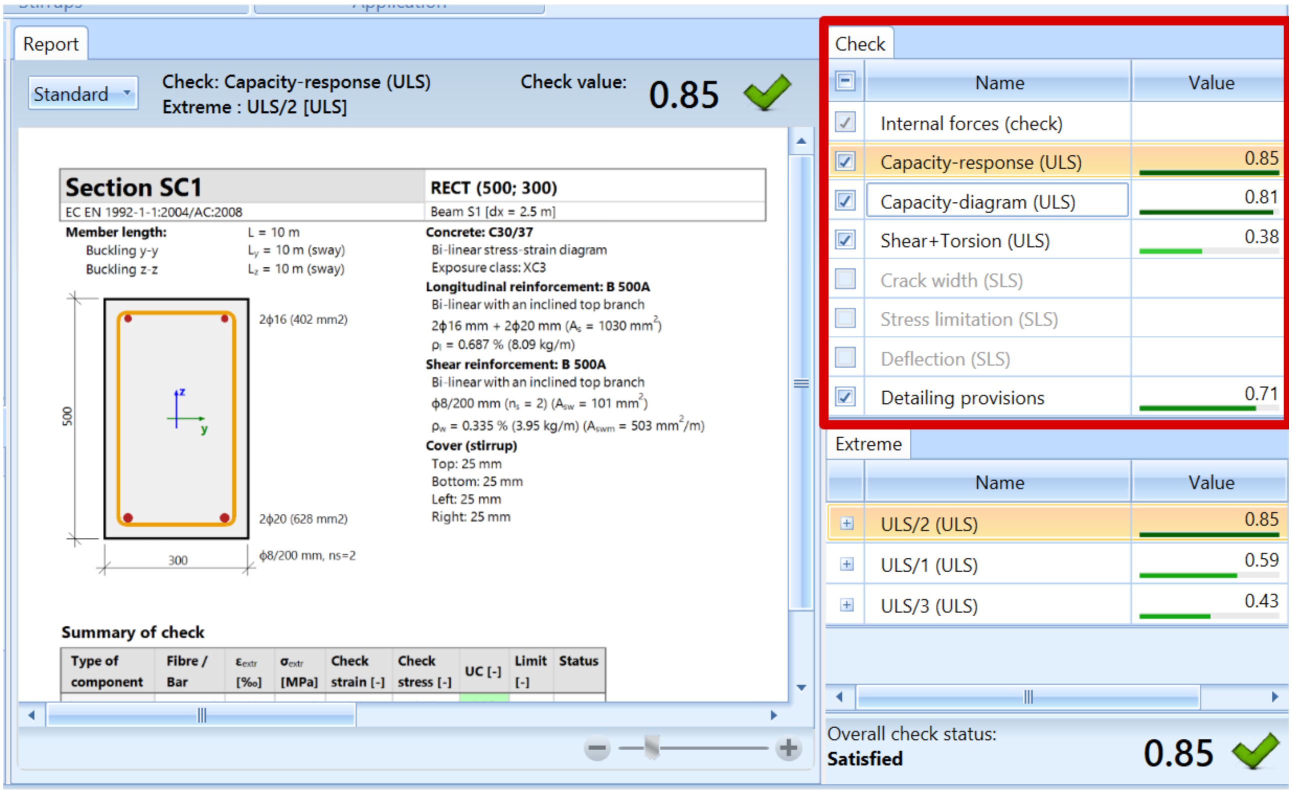


Then change the diameter of the reinforcement template. For bottom longitudinal bars, change diameter to 20mm in the tab "Reinforcement (layout):



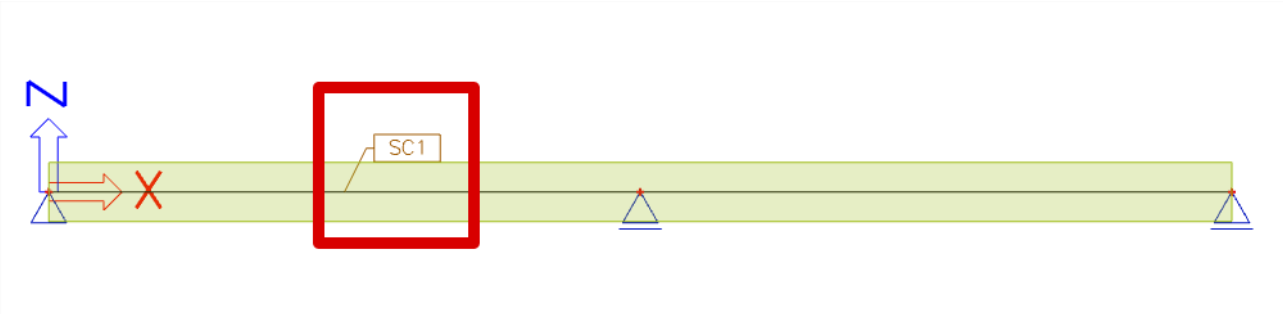
Note that it is also possible to define the shear reinforcement in this window.

The results for all ULS checks are now:

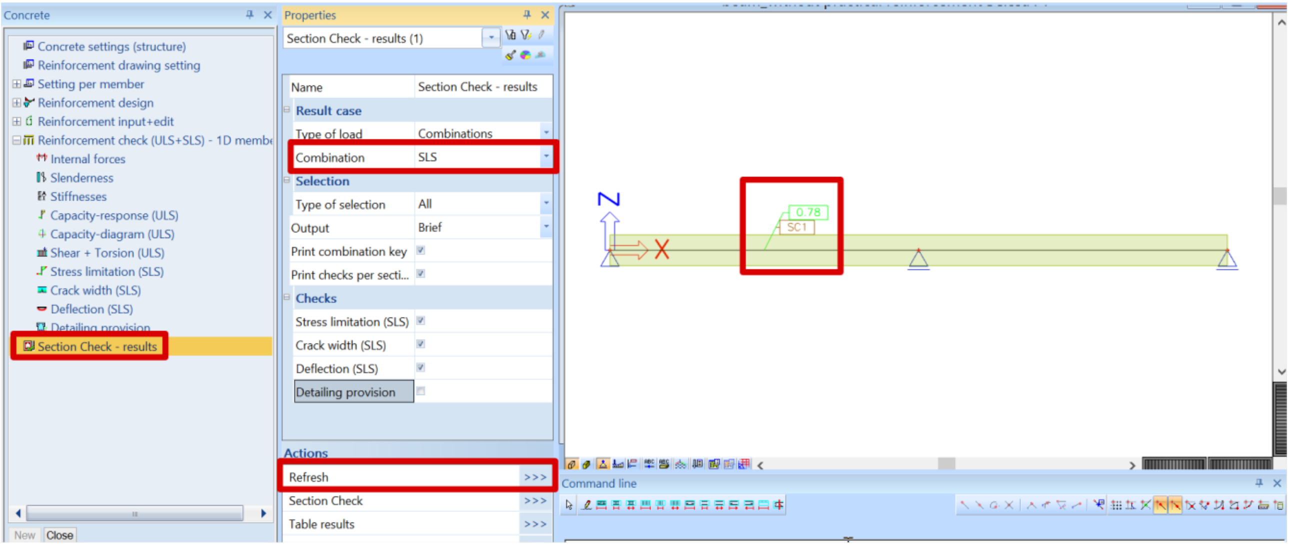


Once the section is reinforced and checks are satisfied, the user can save the design of this section with the option “Save and close”:

A label will then be added on the beam:



It is possible to run the Section check for SLS combination as below:



If required, Section check tool can still be opened to redesign the section to satisfy the SLS checks by clicking on Section check in the Properties window.

Column Design

Reinforcement design methods

For column design, there are 3 types of calculation:

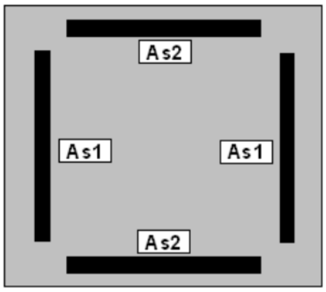
- Axial compression only
- Uniaxial bending
- Biaxial bending

When taking a closer look at the column calculation, 2 different approaches can be distinguished:

- For the ‘Axial compression only’ and ‘Uniaxial bending’ calculation, SCIA Engineer uses the same computing heart as for beams.
- For ‘Biaxial bending’ calculations, SCIA Engineer uses a combination of the computing heart for beams and the so-called interaction formulas.

Furthermore, the uniaxial bending calculation always has as result a 1-directional reinforcement configuration, with the same number of reinforcement bars at parallel sides.

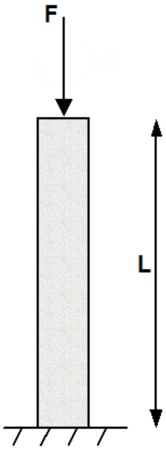
The biaxial bending calculation has as result a 2-directional reinforcement configuration. The number of bars may differ per direction, but is always the same for parallel sides:



The uniaxial bending calculation is a relatively simple calculation type, while the biaxial bending calculation requires an iterative process.

Keep this in mind as the reason why the uniaxial bending calculation will go a lot faster.

Design with axial compression only



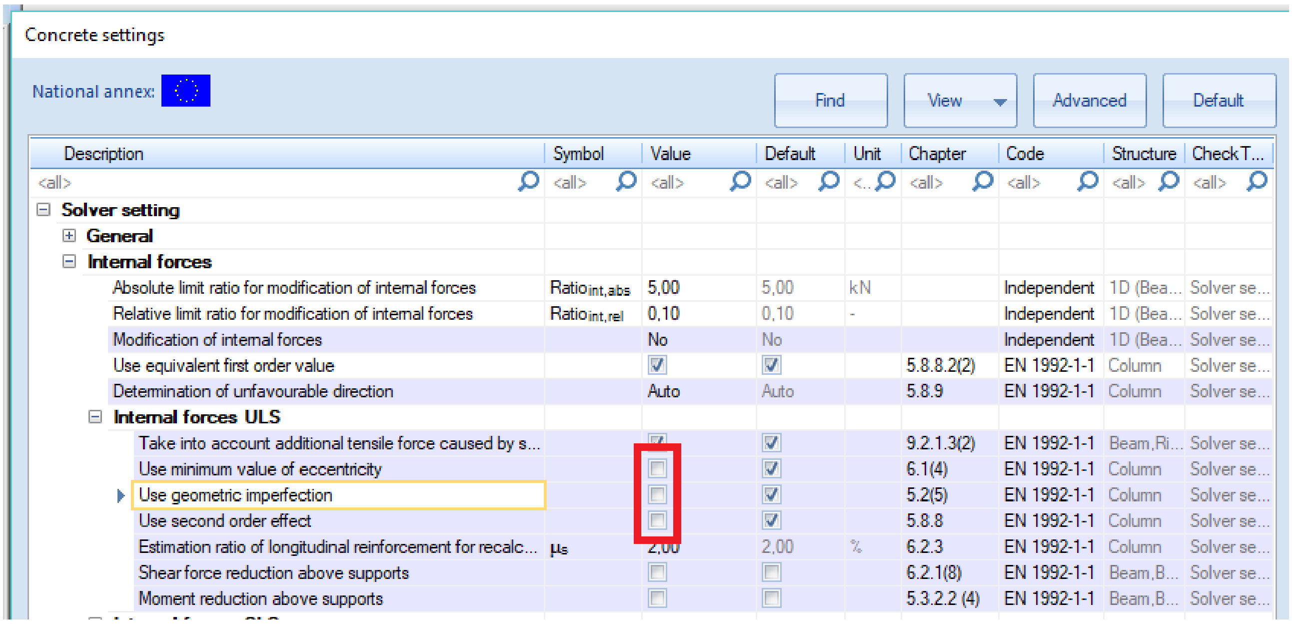
No reinforcement required: $N_{Ed} < N_{Rd}$

Example: 'Axial compression only.esa'

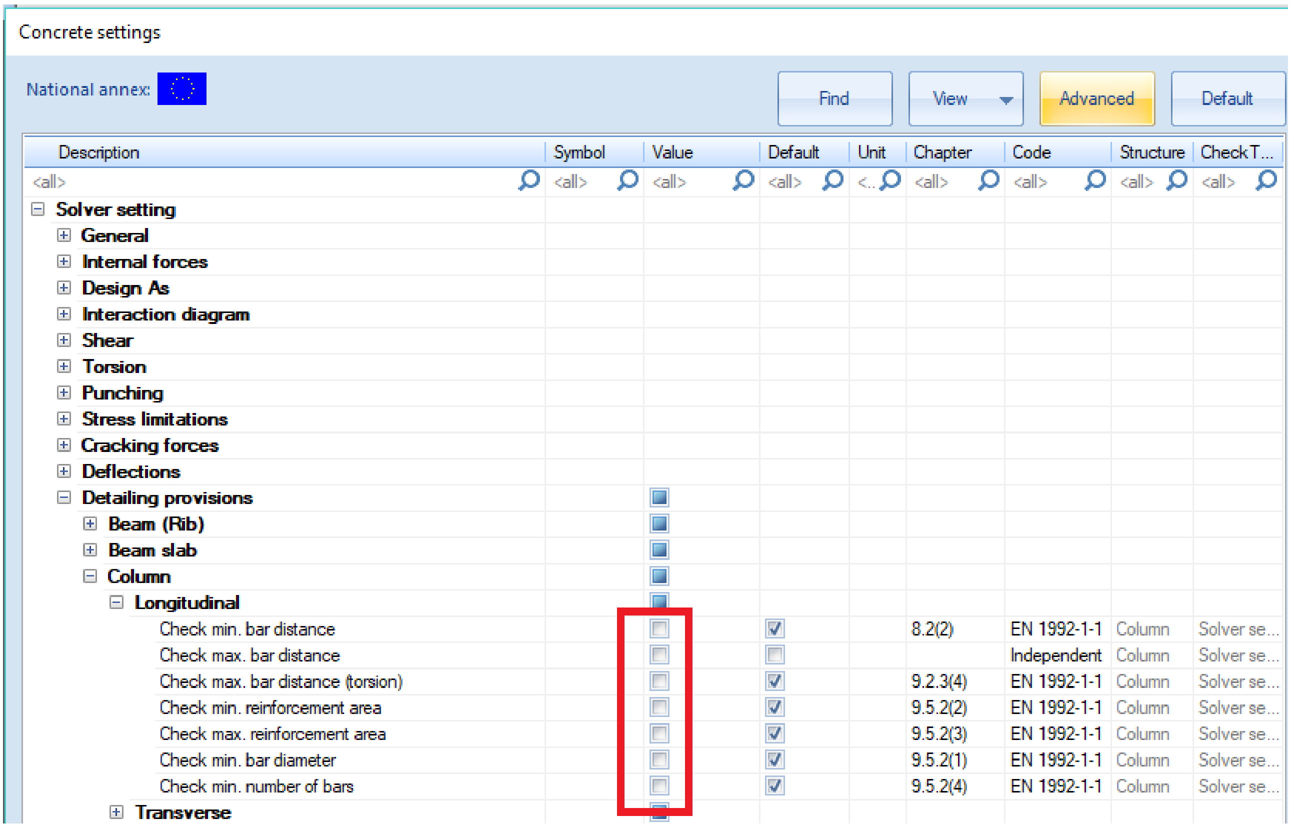
Studied column: B1

Geometry
Column cross-section: RECT 350x350 mm²
Height: 4,5 m
Concrete grade: C45/55

Concrete Setup
Item Concrete settings > Internal forces ULS: 'eccentricities' are not taken in account.



The Detailing provisions are not taken in account, in order to view the pure results (according to the Eurocode, always a minimum reinforcement percentage must be added).



Loads

LC1: Permanent load > F = 1100 kN

LC2: Variable load > F = 1000 kN

This means the column is loaded with a single compression force.

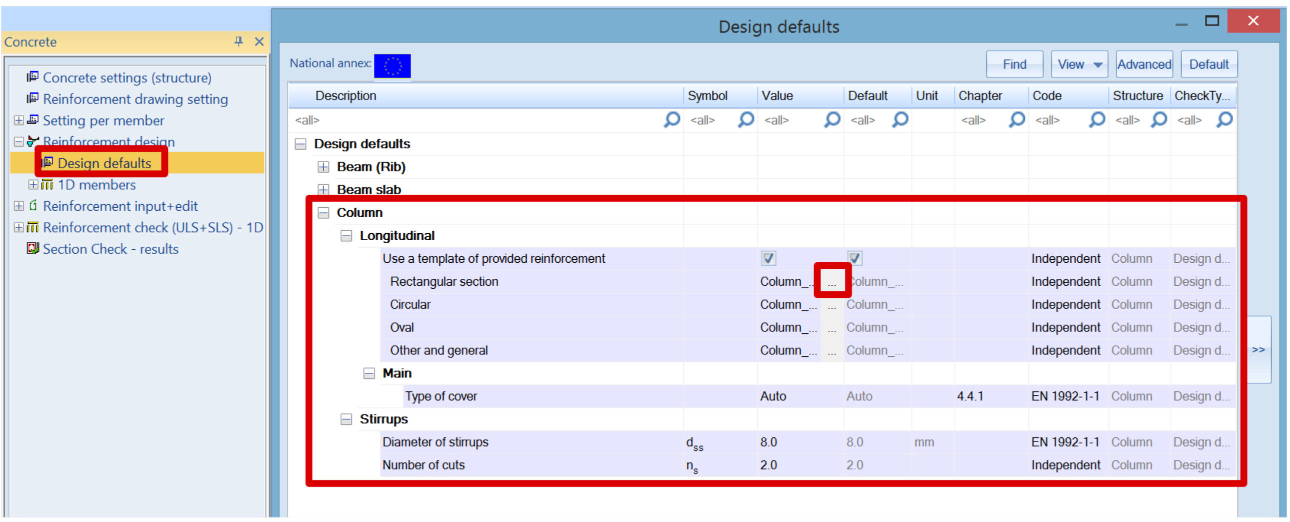
Combination according to the Eurocode:

ULS Combination = 1,35 * LC1 + 1,50 * LC2

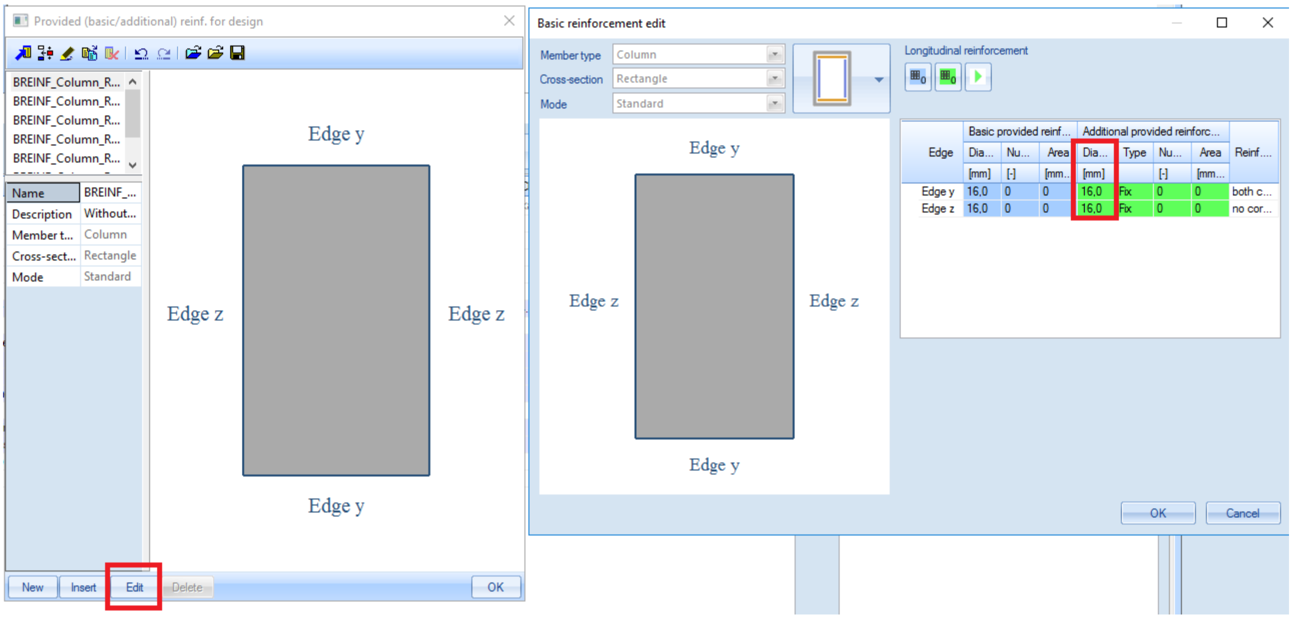
Design normal force N_{Ed} = 1,35 * 1100 + 1,50 * 1000 = 2985 kN

Bar diameter

The bar diameter is taken from the Reinforcement design > Design defaults > Tab Columns, or from 1D member data if applied (1D member data always overwrite the Concrete Setup data, for the specific member they are assigned to).



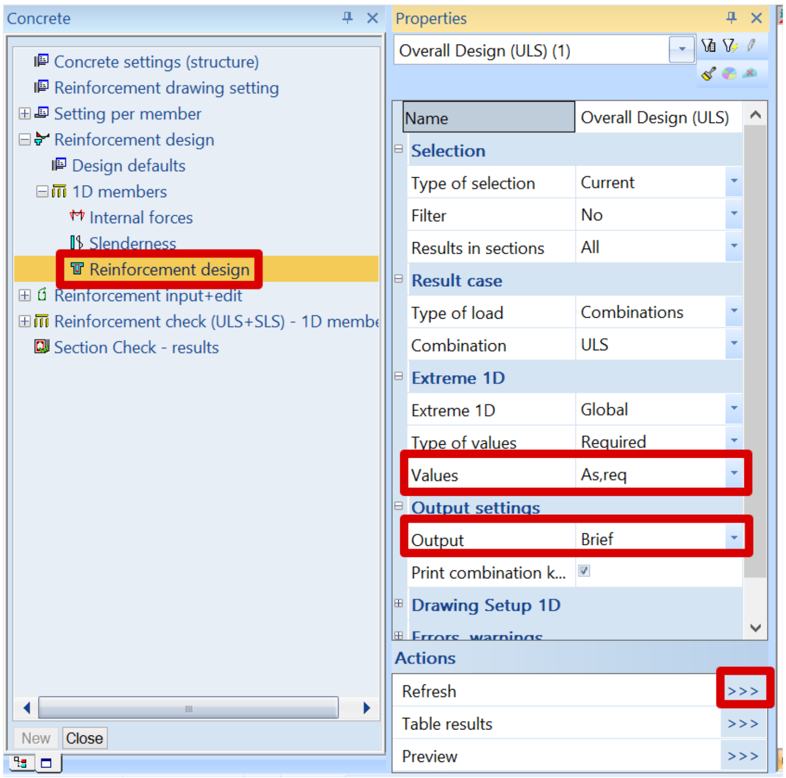
By default, the diameter for the main column reinforcement is put to ϕ16mm. Based on this diameter and the exposure class (by default XC3), the concrete cover is calculated. This information is necessary to be able to calculate the lever arm of the reinforcement bars.



Note: To change the default diameter from ϕ16mm to ϕ20mm for example, edit the template "Column_Rect_Empty" (or the corresponding empty template for the specific columns shape), and change the value of the diameter to be taken into account (additional provided reinforcement).

Results

Go to Reinforcement design > 1D members > Reinforcement design.
Ask the value of $A_{s\,reg}$ for member B1, and click the action button [Refresh].



The graph appears to be null on the screen. The Brief output (Preview button), gives As,req = 0.

Overall Design (ULS)

Linear calculation
Combination: CO1
Coordinate system: Principal
Extreme 1D: Global
Selection: All

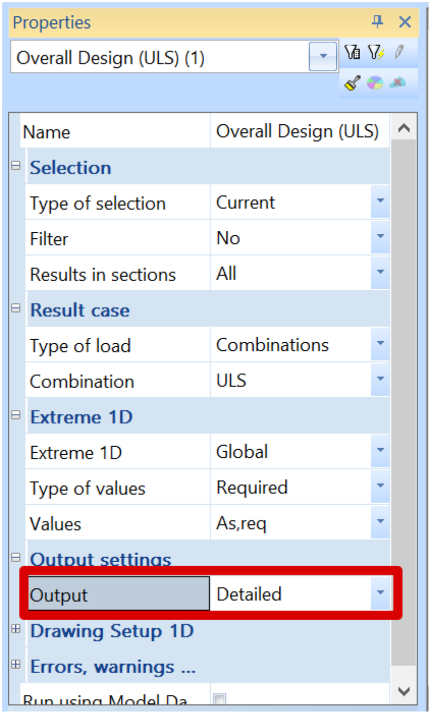
Longitudinal required reinforcement

Name	dx [m]	Case	Member	A_{sz_req+} [mm^2] $A_{sz\_req\_bar+}$ [mm^2]	A_{sz_req-} [mm^2] $A_{sz\_req\_bar-}$ [mm^2]	A_{sy_req+} [mm^2] $A_{sy\_req\_bar+}$ [mm^2]	A_{sy_req-} [mm^2] $A_{sy\_req\_bar-}$ [mm^2]	A_{sz_req} [mm^2] $A_{sz\_req\_bar}$ [mm^2]	A_{sy_req} [mm^2] $A_{sy\_req\_bar}$ [mm^2]	A_{s_req} [mm^2] $A_{s\_req\_bar}$ [mm^2]	ReinfReq
B1	0,000	CO1	Column	0 0	0 0	0 0	0 0	0 0	0 0	0 0	

Shear reinforcement

Name	dx [m]	Case	Member	A_{swm_req} [mm^2/m]	A_{swm_prov} [mm^2/m]	ShearReinf
B1	0,000	CO1	Column	0	0	

If you set output settings on Detailed, you can see the explanation that reinforcement is not necessary.



Explanation errors/warnings and notes

Index	Type	Description	Solution
N1/1	Note	Statically required reinforcement: The reinforcement is not neccessary.	

Remark: this result is obtained only because **all detailing provisions are deactivated** in the Concrete Setup!

Check of reinforcement

$N_{Rd} = f_{cd} \cdot \alpha \cdot A_c$

$= 30 \cdot 1 \cdot 350^2 / 1000 = 3675$ kN

Since N_{Rd} = 3675 kN > N_{Ed} = 2985 kN, indeed no theoretical reinforcement is required.

Reinforcement required: NEd > NRd

Example: 'Axial compression only.esa'

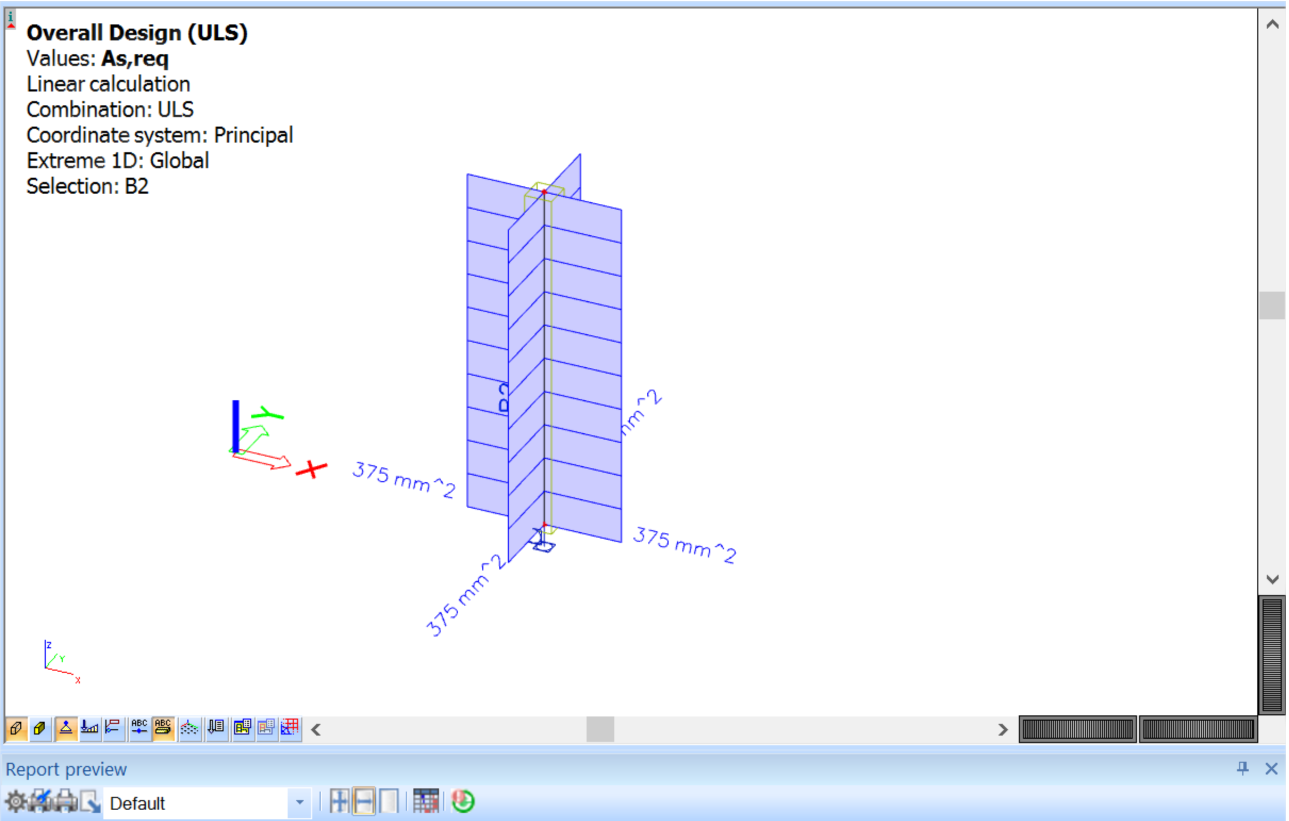
Studied column: B2

For this example, the same configuration as above is used, only the permanent point load is increased to 2000 kN.

Loads
LC1: Permanent load > F = 2000 kN
LC2: Variable load > F = 1000 kN

Combination according to the Eurocode:
ULS Combination = 1,35 * LC1 + 1,50 * LC2
Design normal force N_{Ed} = 1,35 * 2000 + 1,50 * 1000 = 4200 kN

Results
Remark that SCIA Engineer shows on the screen the reinforcement per direction. The total reinforcement area is in fact 750 + 750 = 1500 mm².



Overall Design (ULS)

Linear calculation
Combination: ULS
Coordinate system: Principal
Extreme 1D: Global
Selection: B2
Longitudinal required reinforcement

Name	dx [m]	Case	Member	A_{sz_req+} [mm²] $A_{sz\_req\_bar+}$ [mm²]	A_{sz_req-} [mm²] $A_{sz\_req\_bar-}$ [mm²]	A_{sy_req+} [mm²] $A_{sy\_req\_bar+}$ [mm²]	A_{sy_req-} [mm²] $A_{sy\_req\_bar-}$ [mm²]	A_{sz_req} [mm²] $A_{sz\_req\_bar}$ [mm²]	A_{sy_req} [mm²] $A_{sy\_req\_bar}$ [mm²]	A_{s_req} [mm²] $A_{s\_req\_bar}$ [mm²]	ReinfReq
B2	0.000	ULS	Column	**375** **402**	**375** **402**	**375** **402**	**375** **402**	**750** **804**	**750** **804**	**1500** **1608**	**[z]6ϕ16, [y]6ϕ16**

Shear reinforcement

Name	dx [m]	Case	Member	A_{swm_req} [mm²/m]	A_{swm_prov} [mm²/m]	ShearReinf
B2	0.000	ULS	Column	**0**	**0**	

When asking for the Standard output for Reinforcement design, the proposed configuration can be found:

Overall Design (ULS)

Linear calculation
Combination: CO1
Coordinate system: Principal
Extreme 1D: Global
Selection: All

Column B1	**Rectangle (350; 350)**
EC EN 1992-1-1:2004/AC:2008	Section 0 [dx = 0 m]

		Materials	
Member length	Ld = 4.5 m		
Buckling length y	Ly = 9.01 m	Concrete	C45/55
Buckling length z	Lz = 9.01 m	Reinforcement	B 500B

Longitudinal reinforcement

ϕ = 16 mm, c = 30 mm,

Shear reinforcement

$n_{s,req}$ = 2, $\phi_{s,req}$ = 8 mm, $\alpha_{s,req}$ = 90 °

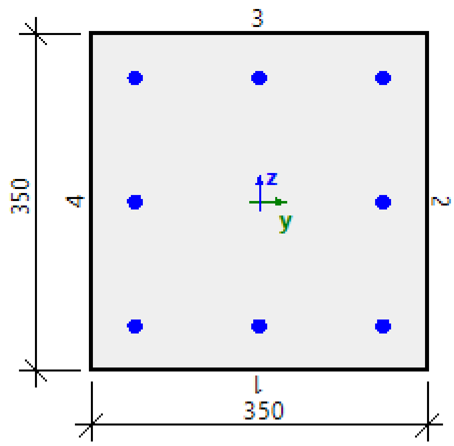
Design of longitudinal reinforcement

A_s: 1.35*LC1+1.50*LC2 : N_{Ed} = -4200 kN, M_{Edy} = 0 kNm, M_{Edz} = 0 kNm

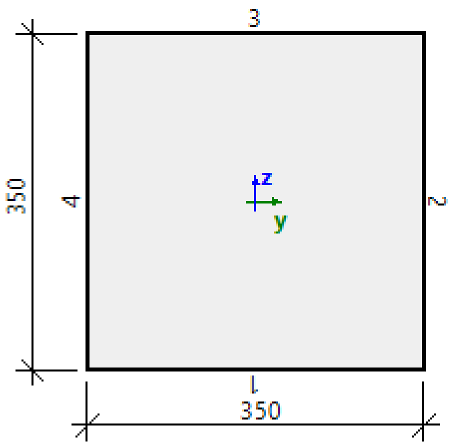
Required

Edge	Layer	y [m]	z [m]	$A_{s,stat}$ [mm^2]	$A_{s,det.min}$ [mm^2]	$A_{s,det.max}$ [mm^2]	$\Delta A_{s,tor}$ [mm^2]	$A_{s,req}$ [mm^2]	$A_{s,req.bar}$ [mm^2]	Reinf
1	1	0	-0.129	375	0	0	0	375	402	3ϕ16
2	1	0.129	0	375	0	0	0	375	402	3ϕ16
3	1	0	0.129	375	0	0	0	375	402	3ϕ16
4	1	-0.129	0	375	0	0	0	375	402	3ϕ16

Required bars



Provided bars



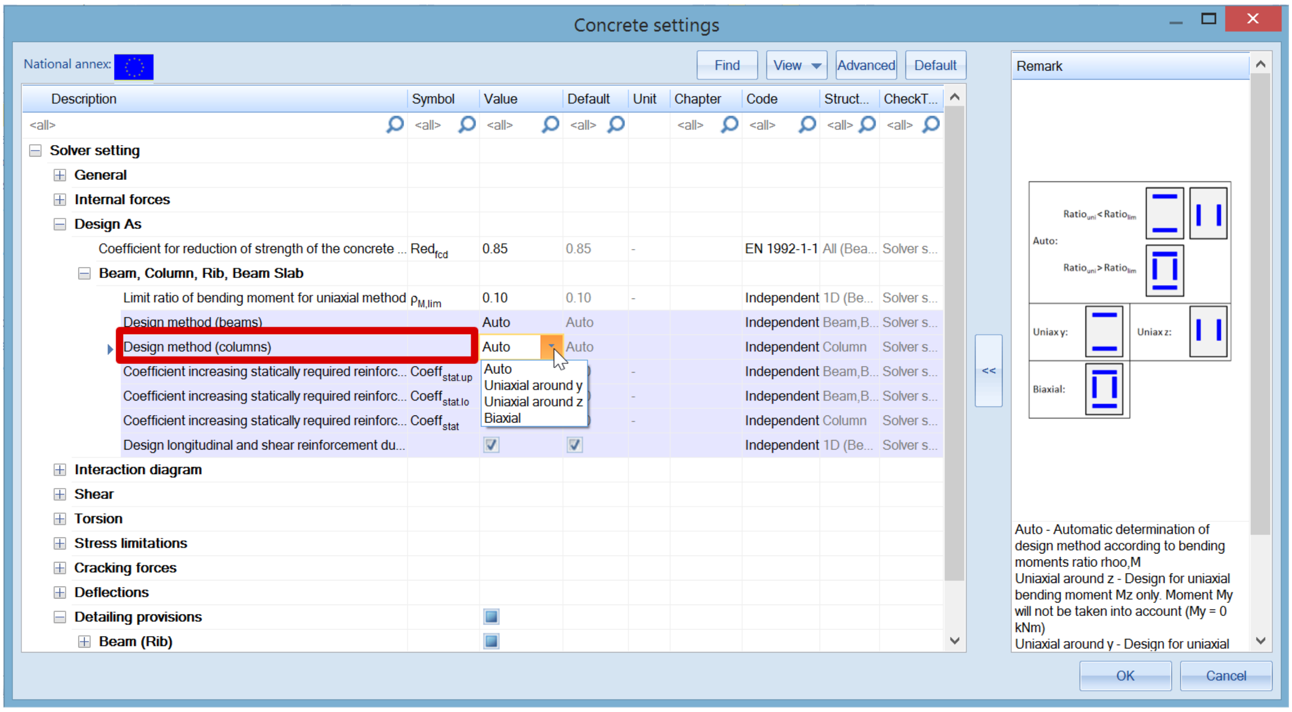
Explanation of the number of reinforcement bars

Default bar diameter has been set to ϕ16 in Design default.
The table indicates that each edge needs 3ϕ16.
On the final picture, this leads to a total of 8ϕ16 in the section of the column.

Design with bending moment and axial force

Four calculation methods are available in SCIA Engineer in concrete settings > Design As > Beam, Column, Rib, … > Design method:

- Auto (by default)
- Uniaxial around y axis
- Uniaxial around z axis
- Biaxial (always used for circular and oval columns)



The “Auto” selection of the design method is based on the limit ratio of bending moment for the uniaxial method. The program will automatically select the uniaxial or biaxial method depending on the values of bending moments around y and z axis.

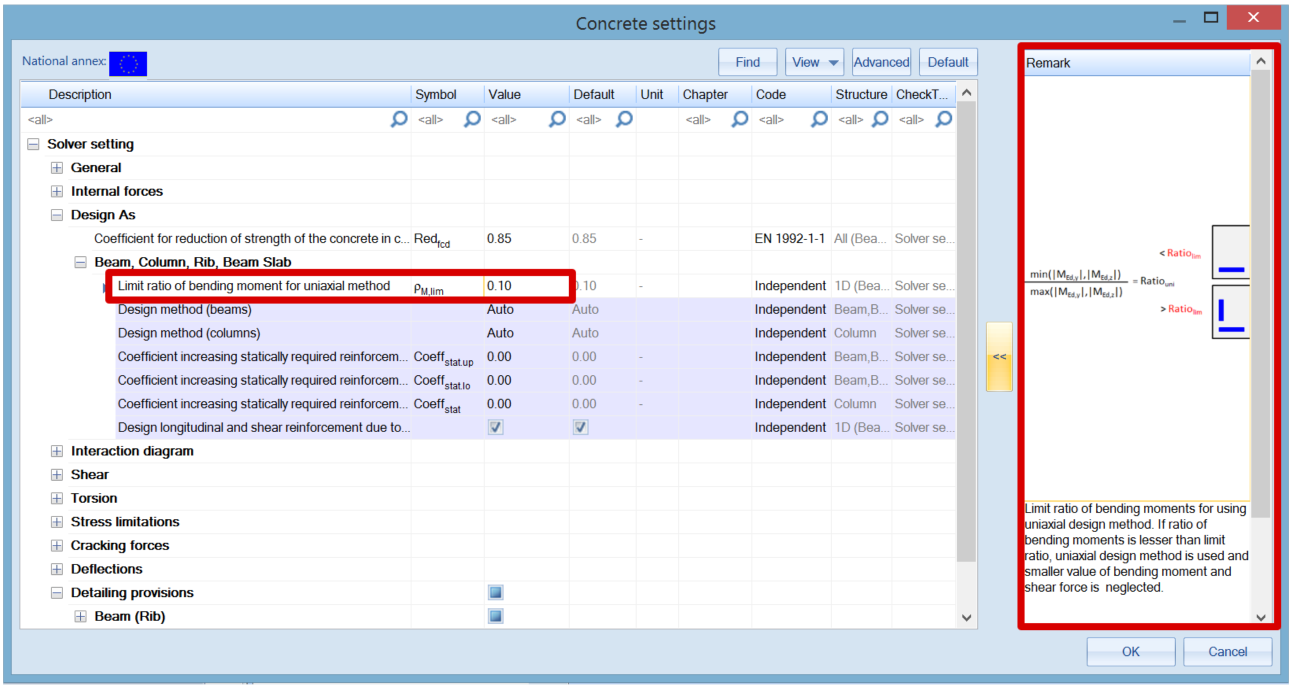
Rule for automatic selection of the design method:

- If $\rho_M \leq \rho_{M,lim}$ Uniaxial method
- If $\rho_M \geq \rho_{M,lim}$ Biaxial method

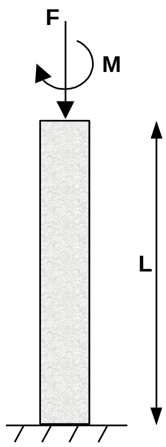
$$\rho_M = \frac{Min\{|MEd_{y,max}|,|MEd_{z,max}|\}}{Max\{|MEd_{y,max}|,|MEd_{z,max}|\}}$$

$MEd_{y,max}$ maximal design moment around y axis from all combinations in current section
$MEd_{z,max}$ maximal design moment around z axis from all combinations in current section
$\rho_{M,lim}$ limit ratio of bending moments for uniaxial method loaded from Concrete setting > Design As

Settings for limit ratio:



Uniaxial bending calculation



Principle

The reinforcement is designed for N_{Ed} and one bending moment $M_{Ed,y}$ or $M_{Ed,z}$:

- Uniaxial around y: M_{Edz} is ignored, the reinforcement is designed only for N_{Ed} and $M_{Ed,y}$
- Uniaxial around z: M_{Edy} is ignored, the reinforcement is designed only for N_{Ed} and $M_{Ed,z}$

If Auto selection of design method is selected and $\rho_M \leq \rho_{M,lim}$, the rule to choose between uniaxial method around y or z is:

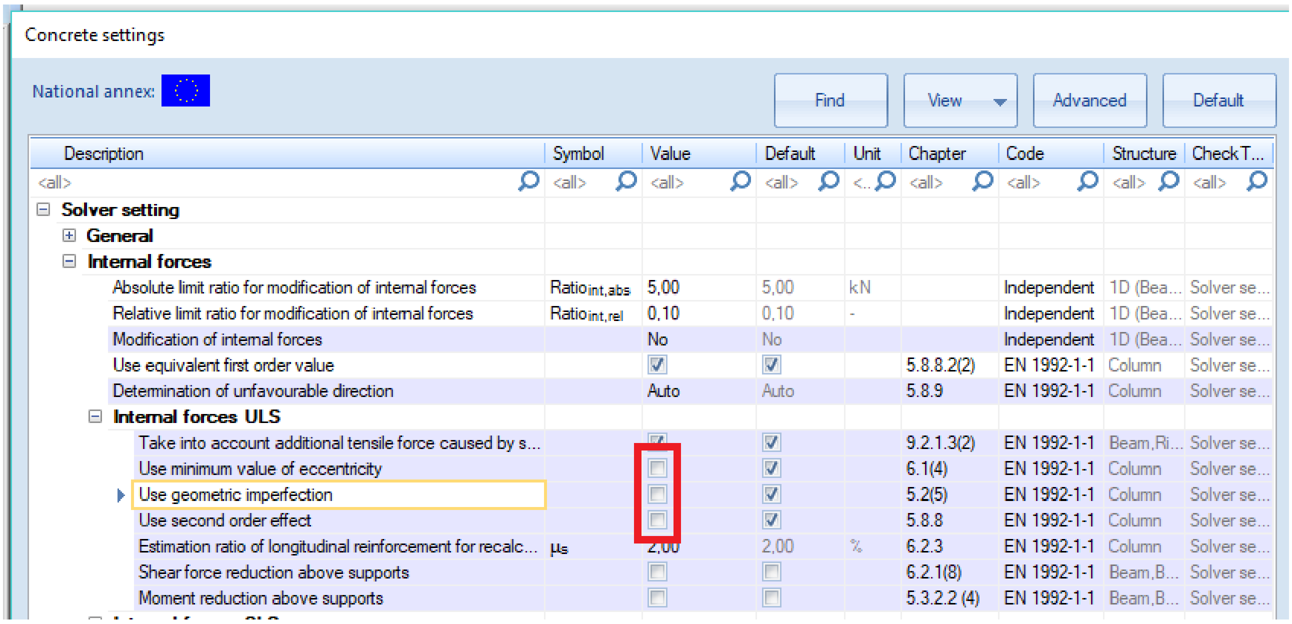
- If $M_{Ed,y} > M_{Ed,z}$ → As = Asy is designed for forces N_{Ed} and $M_{Ed,y}$
- If $M_{Ed,z} > M_{Ed,y}$ → As = Asz is designed for forces N_{Ed} and $M_{Ed,z}$

Example: open the example 'Uniaxial bending.esa'

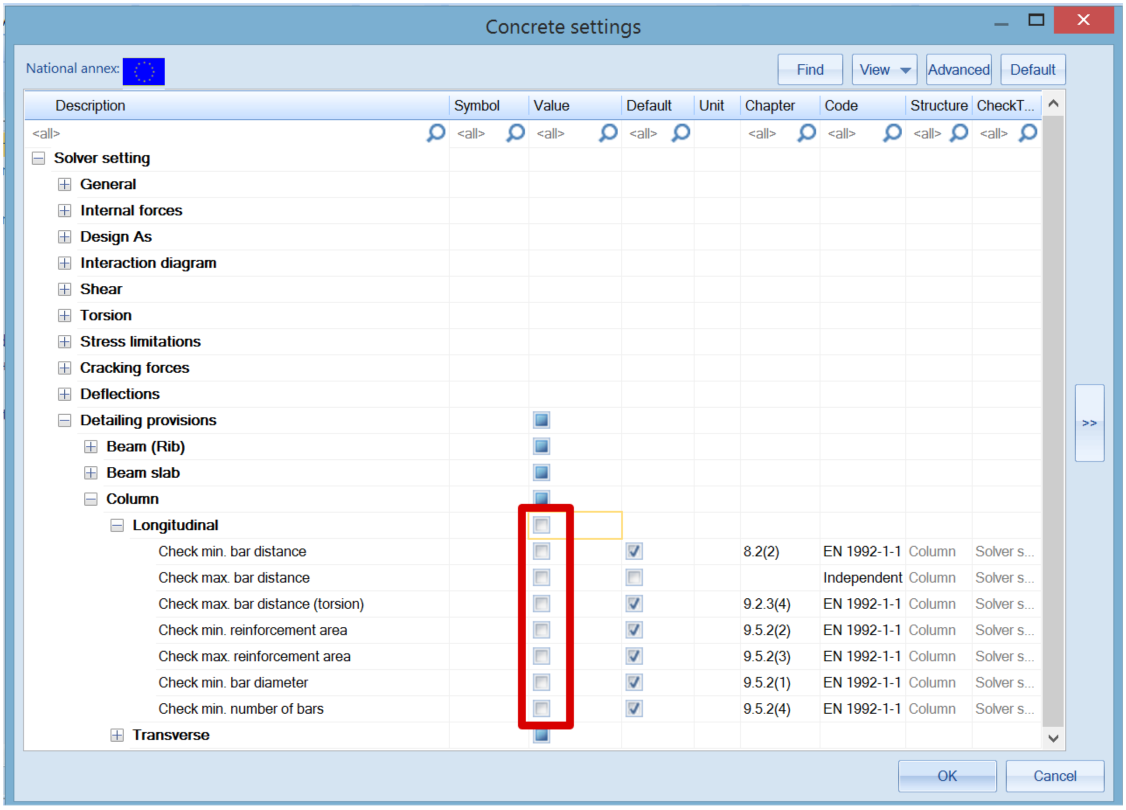
Geometry
Column cross-section: RECT 350x350mm²
Height: 4,5 m
Concrete grade: C45/55

Concrete Setup

Item Concrete settings > Internal forces ULS: 'eccentricities' are not taken in account (only 1st order moments are considered).



Item Detailing provisions are not taken in account, to view the pure results (according to the Eurocode, always a minimum reinforcement percentage must be added).



Loads

Column B1:

LC1: Permanent load > F = 500 kN; M_y = 100 kNm

LC2: Variable load > F = 1000 kN; M_y = 100 kNm

Column B2:

LC1: Permanent load > F = 500 kN; M_y = 100 kNm

LC2: Variable load > F = 1000 kN; M_y = 100 kNm; M_z = 10 kNm

Combination according to the Eurocode:

ULS Combination = 1,35 * LC1 + 1,50 * LC2

Design normal force N_{Ed} = 1,35 * 500 + 1,50 * 1000 = 2175 kN

Design moment M_{yd} = 1,35 * 100 + 1,50 * 100 = 285 kNm

Additional design moment in column B2 M_{zd} = 22.5 kNm

Results

Go to Reinforcement design > 1D members > Reinforcement design, ask the value for $A_{s,req}$, and click the action buttons [Refresh] and [Preview].

Looking at the Detailed output for column B1:

Determination type of calculation

Calculation maximum bending moments around y and z axis

$M_{y.max}$ = -285 kNm $M_{z.max}$ = 0 kNm

Calculation maximum ratio of bending moments

$\rho_M = 0$

Determination type of calculation

$\rho_M = 0 < \rho_{M,lim} = 0.1$ and $|M_{y.max}|$ = 285 kNm > $|M_{z.max}|$ = 0 kNm =>

=> Uniaxial method around y axis. Moment M_z will not take into account (M_z = 0 kNm).

The numerical results of the calculation are as follows (standard output):

Column B1	**RECT (350; 350)**
EC EN 1992-1-1:2004/AC:2008	Section 0 [dx = 0 m]

		Materials	
Member length	Ld = 4.5 m		
Buckling length y	Ly = 9.01 m	Concrete	C45/55
Buckling length z	Lz = 9.01 m	Reinforcement	B 500A

Longitudinal reinforcement

ϕ = 16 mm, c = 30 mm,

Shear reinforcement

$n_{s.req}$ = 2, $\phi_{s.req}$ = 8 mm, $\alpha_{s.req}$ = 90 °

Design of longitudinal reinforcement

$A_{s.z+}$: 1.35*LC1+1.50*LC2 : N_{Ed} = -2175 kN, M_{Edy} = -285 kNm, M_{Edz} = 0 kNm

$A_{s.z-}$: 1.35*LC1+1.50*LC2 : N_{Ed} = -2175 kN, M_{Edy} = -285 kNm, M_{Edz} = 0 kNm

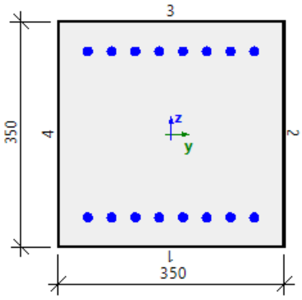
Required

Edge	Layer	y [m]	z [m]	$A_{s.stat}$ [mm^2]	$A_{s.det.min}$ [mm^2]	$A_{s.det.max}$ [mm^2]	$\Delta A_{s.tor}$ [mm^2]	$A_{s.req}$ [mm^2]	$A_{s.req.bar}$ [mm^2]	Reinf
1	1	0	-0.129	1552	0	0	0	1552	1608	8ϕ16
3	1	0	0.129	1552	0	0	0	1552	1608	8ϕ16

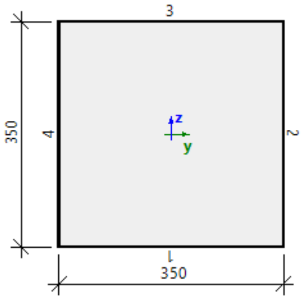
Summary of reinforcement

Top :	$A_{sz,req+}$ = 1552 mm^2	$A_{sz,prov+}$ = 0 mm^2
Bottom :	$A_{sz,req-}$ = 1552 mm^2	$A_{sz,prov-}$ = 0 mm^2
Right :	$A_{sy,req+}$ = 0 mm^2	$A_{sy,prov+}$ = 0 mm^2
Left :	$A_{sy,req-}$ = 0 mm^2	$A_{sy,prov-}$ = 0 mm^2
Total vertical :	$A_{sz,req}$ = 3104 mm^2	$A_{sz,prov}$ = 0 mm^2
Total horizontal :	$A_{sy,req}$ = 0 mm^2	$A_{sy,prov}$ = 0 mm^2
Total :	$A_{s,req}$ = 3104 mm^2	$A_{s,prov}$ = 0 mm^2

Required bars



Provided bars



Looking at the Detailed output for column B2:

Determination type of calculation

Calculation maximum bending moments around y and z axis

$M_{y,max}$ = -285 kNm $M_{z,max}$ = -22.5 kNm

Calculation maximum ratio of bending moments

ρ_M = 0.0789

Determination type of calculation

ρ_M = 0.0789 < $\rho_{M,lim}$ = 0.1 and $|M_{y,max}|$ = 285 kNm > $|M_{z,max}|$ = 22.5 kNm =>

= > Uniaxial method around y axis. Moment M_z will not take into account (M_z = 0 kNm).

And the Standard output:

Column B2

RECT (350; 350)

EC EN 1992-1-1:2004/AC:2008

Section 0 [dx = 0 m]

Member length	Ld = 4.5 m	**Materials**	
Buckling length y	Ly = 9.01 m	Concrete	C45/55
Buckling length z	Lz = 9.01 m	Reinforcement	B 500A

Longitudinal reinforcement

ϕ = 16 mm, c = 30 mm,

Shear reinforcement

$n_{s,req}$ = 2, $\phi_{s,req}$ = 8 mm, $\alpha_{s,req}$ = 90 °

Design of longitudinal reinforcement

$A_{s,z+}$: 1.35*LC1+1.50*LC2 : N_{Ed} = -2175 kN, M_{Edy} = -285 kNm, M_{Edz} = 0 kNm

$A_{s,z-}$: 1.35*LC1+1.50*LC2 : N_{Ed} = -2175 kN, M_{Edy} = -285 kNm, M_{Edz} = 0 kNm

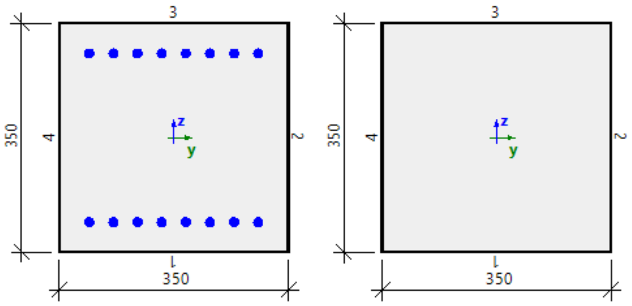
Required

Edge	Layer	y [m]	z [m]	$A_{s,stat}$ [mm^2]	$A_{s,det,min}$ [mm^2]	$A_{s,det,max}$ [mm^2]	$\Delta A_{s,tor}$ [mm^2]	$A_{s,req}$ [mm^2]	$A_{s,req,bar}$ [mm^2]	Reinf
1	1	0	-0.129	1552	0	0	0	1552	1608	8ϕ16
3	1	0	0.129	1552	0	0	0	1552	1608	8ϕ16

Summary of reinforcement

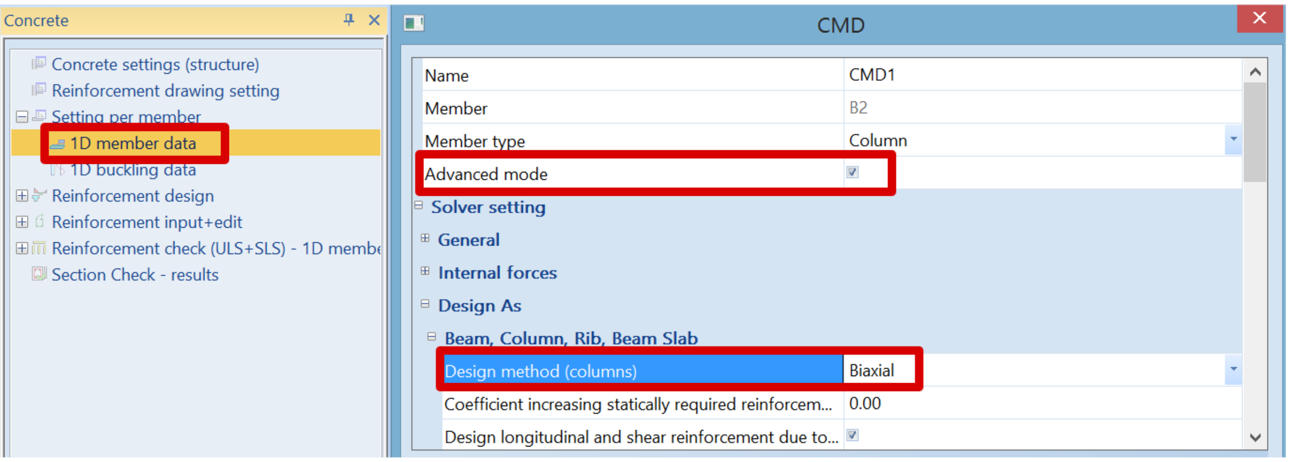
Top :	$A_{sz.req+}$ = 1552 mm^2	$A_{sz.prov+}$ = 0 mm^2
Bottom :	$A_{sz.req-}$ = 1552 mm^2	$A_{sz.prov-}$ = 0 mm^2
Right :	$A_{sy.req+}$ = 0 mm^2	$A_{sy.prov+}$ = 0 mm^2
Left :	$A_{sy.req-}$ = 0 mm^2	$A_{sy.prov-}$ = 0 mm^2
Total vertical :	$A_{sz.req}$ = 3104 mm^2	$A_{sz.prov}$ = 0 mm^2
Total horizontal :	$A_{sy.req}$ = 0 mm^2	$A_{sy.prov}$ = 0 mm^2
Total :	$A_{s.req}$ = 3104 mm^2	$A_{s.prov}$ = 0 mm^2

Required bars **Provided bars**



Even if an additional bending moment in the z direction is present in column B2, according to the limit ratio the uniaxial method was used, and the same amount of reinforcement is required for columns B1 and B2.

The user has the possibility to force the biaxial method design on column B2 using 1D member data in Settings per member > 1D member data:



Amount of required reinforcement will be slightly higher in this case since M_{Edz} is also considered.

Column B2

RECT (350; 350)

EC EN 1992-1-1:2004/AC:2008 — Section 0 [dx = 0 m]

Member length Ld = 4.5 m
Buckling length y Ly = 9.01 m
Buckling length z Lz = 9.01 m

Materials
Concrete C45/55
Reinforcement B 500A

Longitudinal reinforcement
ϕ = 16 mm, c = 30 mm,

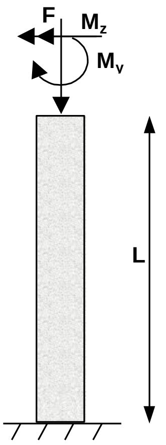
Shear reinforcement
$n_{s.req}$ = 2, $ϕ_{s.req}$ = 8 mm, $α_{s.req}$ = 90 °

Design of longitudinal reinforcement
$A_{s,z+}$: 1.35*LC1+1.50*LC2 : N_{Ed} = -2175 kN, M_{Edy} = -285 kNm, M_{Edz} = -23 kNm
$A_{s,z-}$: 1.35*LC1+1.50*LC2 : N_{Ed} = -2175 kN, M_{Edy} = -285 kNm, M_{Edz} = -23 kNm

Required

Edge	Layer	y [m]	z [m]	$A_{s.stat}$ [mm^2]	$A_{s.det.min}$ [mm^2]	$A_{s.det.max}$ [mm^2]	$ΔA_{s.tor}$ [mm^2]	$A_{s.req}$ [mm^2]	$A_{s.req.bar}$ [mm^2]	Reinf
1	1	0	-0.129	2046	0	0	0	2046	2212	11ϕ16
3	1	0	0.129	2046	0	0	0	2046	2212	11ϕ16

Biaxial bending calculation



This method allows to design reinforcement for a normal force (N_{Ed}) and biaxial bending moments. This method is based on an interaction formula, equation 5.39 in EN 1992-1-1.

$$\left(\frac{M_{Edz}}{M_{Rdz}}\right)^a + \left(\frac{M_{Edy}}{M_{Rdy}}\right)^a \leq 1{,}0 \qquad (5.39)$$

where:

$M_{Edz/y}$ design moment, including a 2nd order moment (if required)

$M_{Rdz/y}$ moment resistance

a exponent:

- for circular and elliptical cross sections: a = 2
- for rectangular cross sections:

N_{Ed}/N_{Rd}	0,1	0,7	1,0
a =	1,0	1,5	2,0

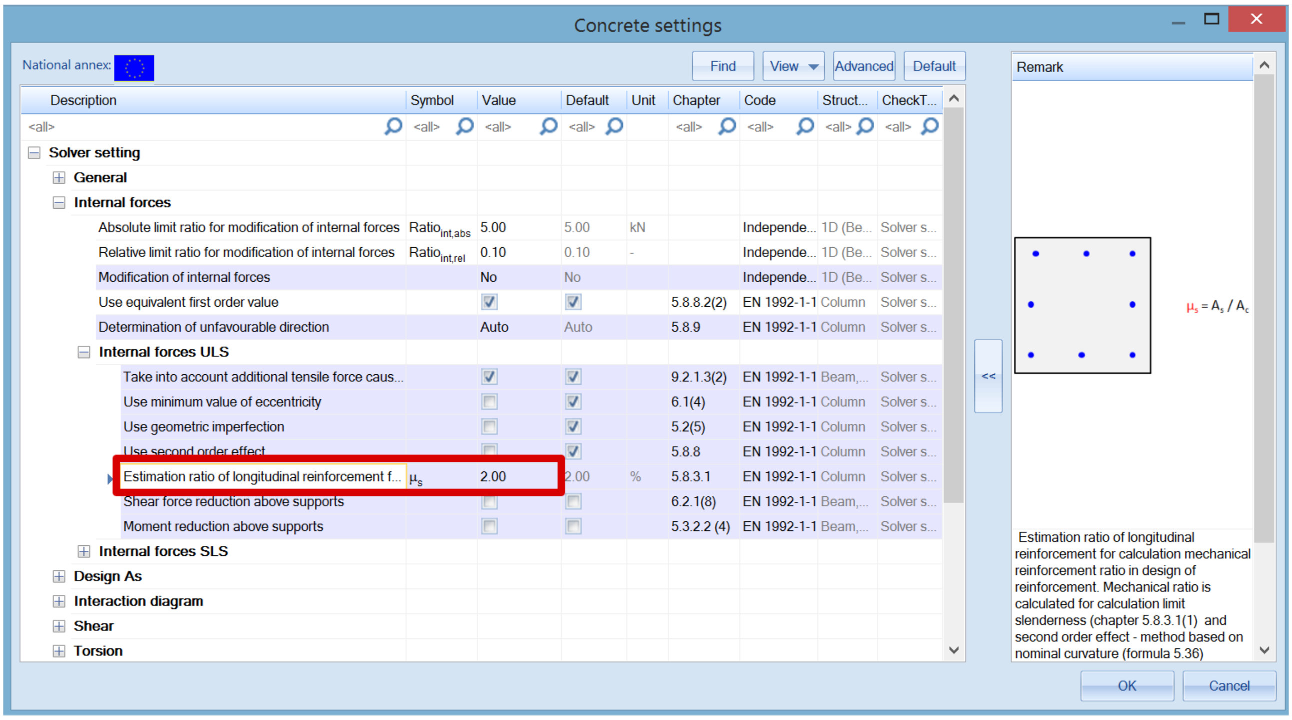
with linear interpolation for intermediate values

N_{Ed} design value of axial force

N_{Rd} = $A_c \cdot (f_{cd} + \mu_s \cdot f_{yd})$, design axial resistance of the section, where:

- A_c gross area of the concrete section
- f_{cd} design value of concrete compressive strength
- f_{yd} design yield strength of reinforcement
- μ_s estimation ratio of longitudinal reinforcement from the Concrete settings or 1D member data

The ratio μ_s can be set in Concrete settings > Solver Settings > Internal forces > Internal forces ULS:


Concrete settings
National annex:
Find
View
Advanced
Default
Description
Symbol
Value
Default
Unit
Chapter
Code
Struct...
CheckT...
Solver setting
General
Internal forces
Absolute limit ratio for modification of internal forces
Relative limit ratio for modification of internal forces
Modification of internal forces
Use equivalent first order value
Determination of unfavourable direction
Internal forces ULS
Take into account additional tensile force caus...
Use minimum value of eccentricity
Use geometric imperfection
Use second order effect
Estimation ratio of longitudinal reinforcement f...
Shear force reduction above supports
Moment reduction above supports
Internal forces SLS
Design As
Interaction diagram
Shear
Torsion
Remark
Estimation ratio of longitudinal reinforcement for calculation mechanical reinforcement ratio in design of reinforcement. Mechanical ratio is calculated for calculation limit slenderness (chapter 5.8.3.1(1) and second order effect - method based on nominal curvature (formula 5.36)
OK
Cancel

Circular columns

For circular and oval columns, the design method is always the biaxial calculation, regardless of the design method set in the Concrete settings.

For circular and oval columns, the required number of reinforcement bars is spread equally along the face of the column.

Example: 'Circular column.esa'

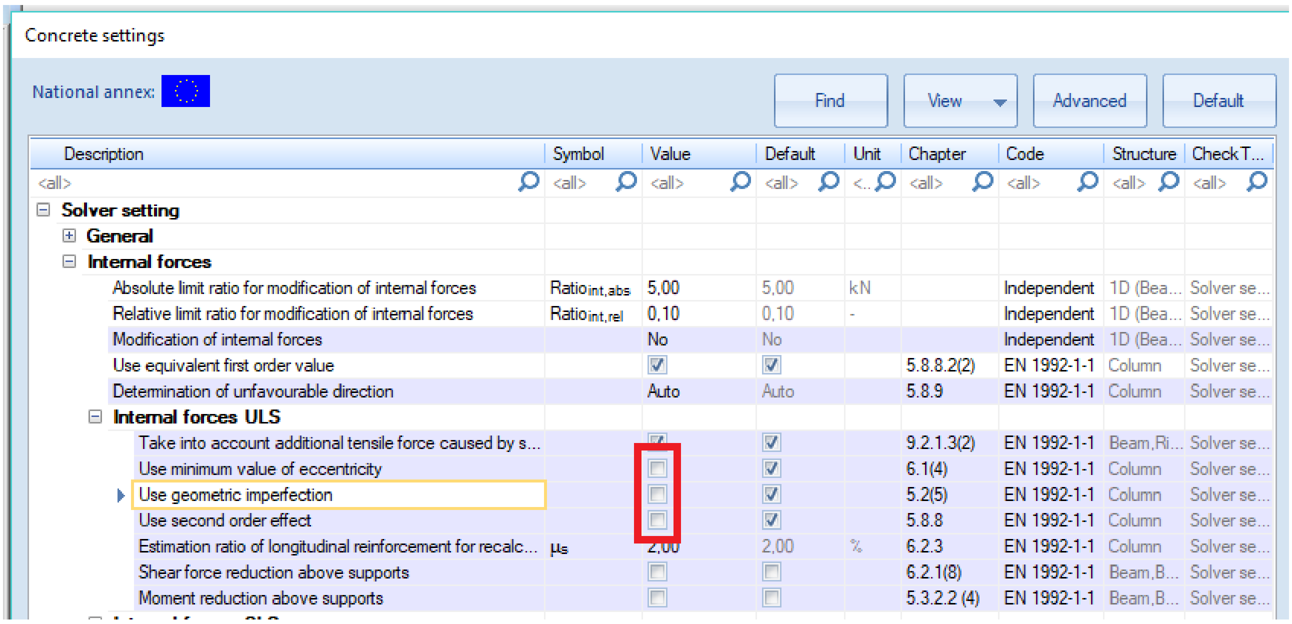
Geometry
Column cross-section: CIRC diameter 400mm
Height: 4,5 m
Concrete grade: C45/55

Loads

Load configuration: N_{Ed} = 2175,00 kN
M_{yd} = 142,50 kNm
M_{zd} = 0 kNm

Concrete Setup

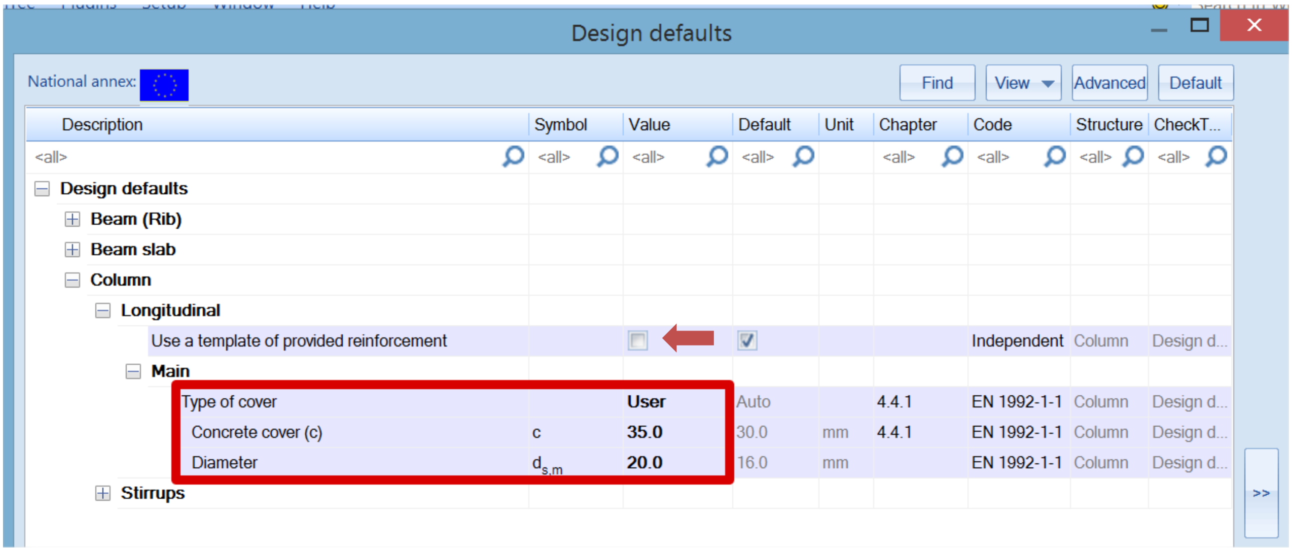
Geometrical imperfection and 2nd order moments are deactivated: Concrete settings > Internal forces ULS:



All detailing provisions are considered.

Design defaults

The bar diameter is set to ϕ20mm in Reinforcement design > Design defaults > Tab Columns, or from 1D Member data if applied.



Results

Go to Reinforcement design > 1D members > Reinforcement design.
Choose Standard output in the Properties window and open the Preview at the bottom of the Properties window:

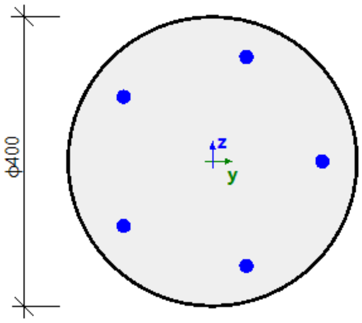
Required

Edge	Layer	y [m]	z [m]	$A_{s,stat}$ [mm²]	$A_{s,det,min}$ [mm²]	$A_{s,det,max}$ [mm²]	$\Delta A_{s,tor}$ [mm²]	$A_{s,req}$ [mm²]	$A_{s,req,bar}$ [mm²]	Reinf
-	-	-	-	1142	1257	5001	0	1257	1571	5ϕ20*

Summary of reinforcement

Total : $A_{s,req}$ = 1257 mm²

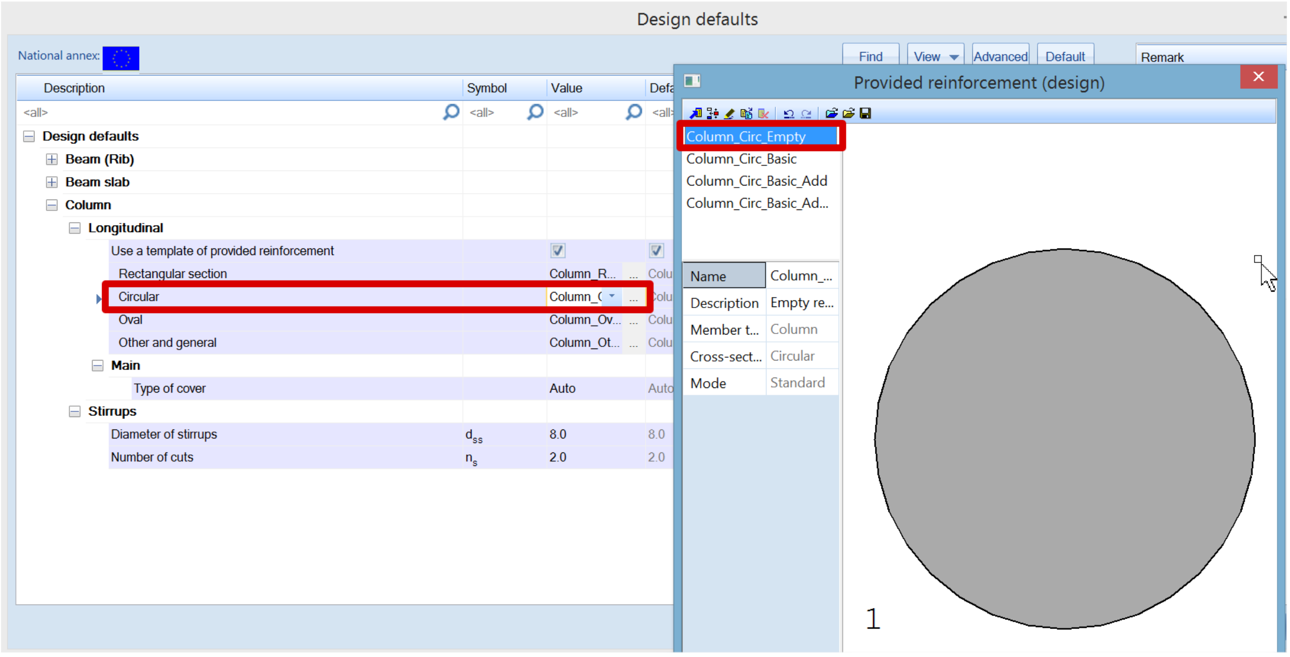
Required bars



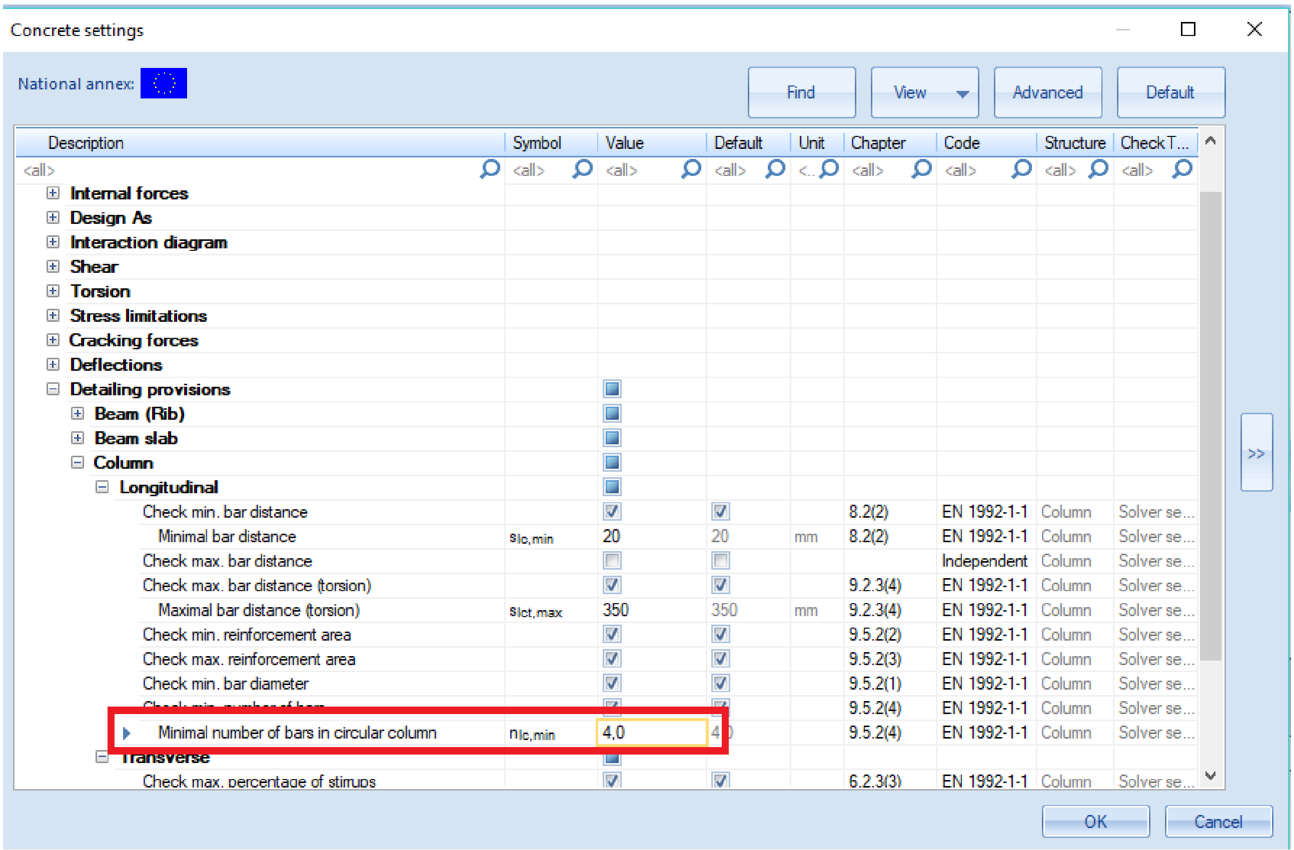
In this example $A_{s,req}$ is determined by the minimum amount of reinforcement according to the detailing provision, $A_{s,det,min}$.
Since $A_{s,req}$ = 1257 mm², the software will propose 5 bars of ϕ20mm (5*314mm² = 1571mm² = $A_{s,req,bar}$) which is the closest amount of bar with $A_{s,req,bar} > A_{s,req}$.

Note that SCIA Engineer uses the real area of the bars to calculate the required reinforcement area. So, the final required reinforcement displayed on the screen is $A_{s,req,bar}$.

Remark 1: If you choose a template without bars predefined in Design Default, for example "Column_Circ-Empty", the software will display only the $A_{s,req}$ and not $A_{s,req,bar}$ as mentioned above.



Remark 2:
According to *EN1992-1-1 art 9.5.2(4)*, there is a minimum number of bars in a circular column. This parameter is set by default to "4" in Concrete Setup > Detailing provisions > Column > Longitudinal.



If we increase the loads:
Fz = -1250 kN
M = 50 kNm
the results are as follows:

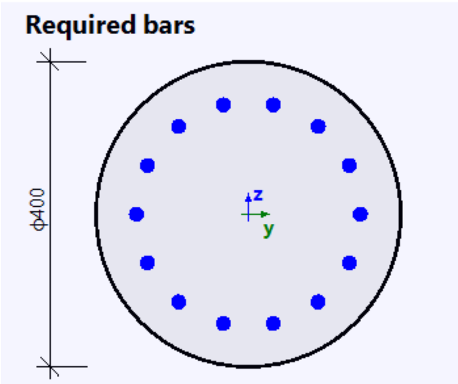
Exemple: "Circular column_increase.esa"

Overall Design (ULS)

Linear calculation
Combination: CO1
Coordinate system: Principal
Extreme 1D: Global
Selection: All
Longitudinal required reinforcement

Name	dx [m]	Case	Member	A_{sz} req+ [mm^2] A_{sz} req bar+ [mm^2]	A_{sz} req- [mm^2] A_{sz} req bar- [mm^2]	A_{sy} req+ [mm^2] A_{sy} req bar+ [mm^2]	A_{sy} req- [mm^2] A_{sy} req bar- [mm^2]	A_{sz} req [mm^2] A_{sz} req bar [mm^2]	A_{sy} req [mm^2] A_{sy} req bar [mm^2]	A_s req [mm^2] A_s req bar [mm^2]	ReinfReq
B1	0,000	CO1	Column	1041 1100	1041 1100	1041 1100	1041 1100	2082 2199	2082 2199	4164 4398	14ϕ20

The corresponding bar configuration is:



Calculation of internal forces

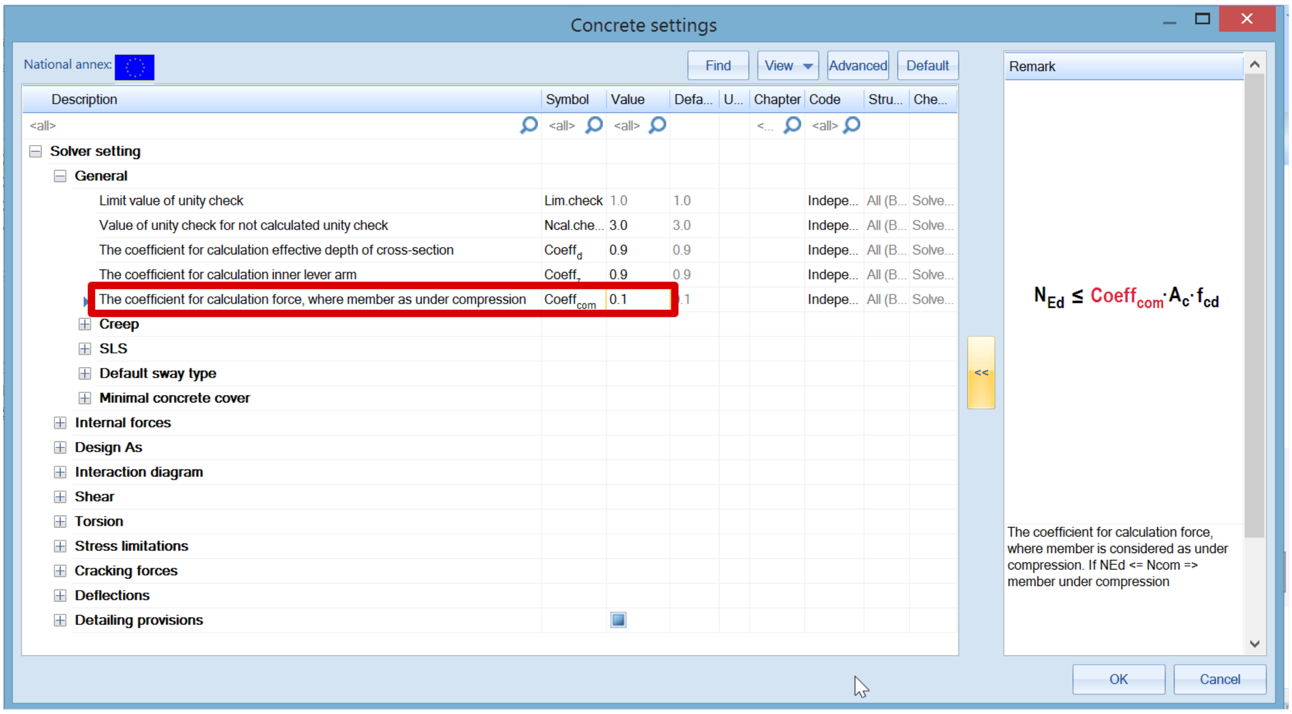
Determining if member is in compression

2nd order effects, geometrical imperfection and minimal eccentricity are considered only if:

- Member type = Column
- Compression in the column is relatively high

In SCIA Engineer, there is a parameter which allows to decide whether a member is in compression or if the compression is too small to be considered.

In Concrete settings > Solver setting > General:



Condition is:

- If $N_{Ed} \leq -\,Coeff_{com} \cdot f_{cd} \cdot A_c$ Member is in compression
- If $N_{Ed} > -\,Coeff_{com} \cdot f_{cd} \cdot A_c$ Compression is not sufficient (zero or relatively small)

This result can be viewed in Reinforcement design > 1D member > Internal forces.

The Detailed output gives:

Compression member

Limit axial force to consider member as compression:

$$N_{com} = -\,Coeff_{com} \cdot \left(f_{cd} \cdot A_c\right) = -0.1 \cdot \left(30 \cdot 10^{6} \cdot 0.123\right) = -368 \text{ kN}$$

Check condition:

$N_{Ed} < N_{com}$ = -1100 kN < -368 kN ... compression member

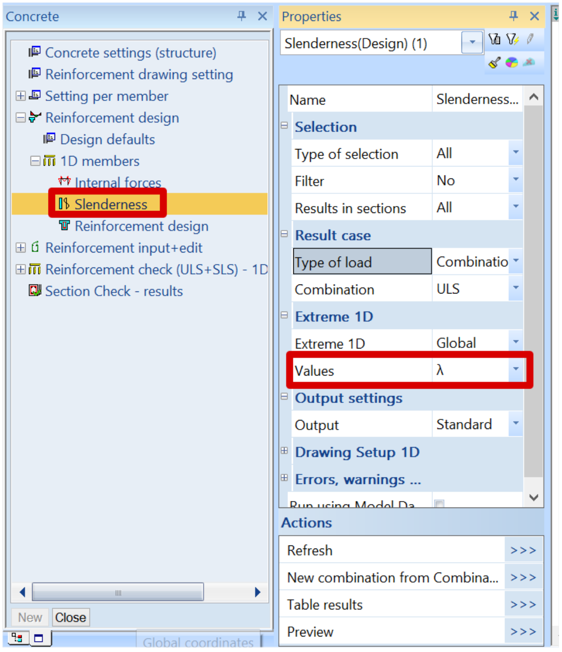
Note: First and second order eccentricity shall be taken into account, because the member is considered as a compression member (significant normal force is presented).

Choice between 1st and 2nd order calculation

Slenderness – Check of the criteria λ < λ lim

- If $\lambda < \lambda_{lim}$, 1st order effects have to be taken into account with geometric imperfection (art 5.2)
- If $\lambda > \lambda_{lim}$, 2nd order effects have to be taken into account with geometric imperfection (art 5.2)

The values for λ and λ_{lim}, and the corresponding check, can be found in the Concrete menu > Reinforcement design > 1D member > Slenderness



The Standard output shows the check of $\lambda > \lambda_{lim}$ and indicates whether a 1st or 2nd order calculation should be done.

Slenderness(Design)

Linear calculation
Load case: LC1
Coordinate system: Principal
Extreme 1D: Global
Selection: All

Column B1	**RECT (350; 350)**
EC EN 1992-1-1:2004/AC:2008	Section 0 [dx = 0 m]

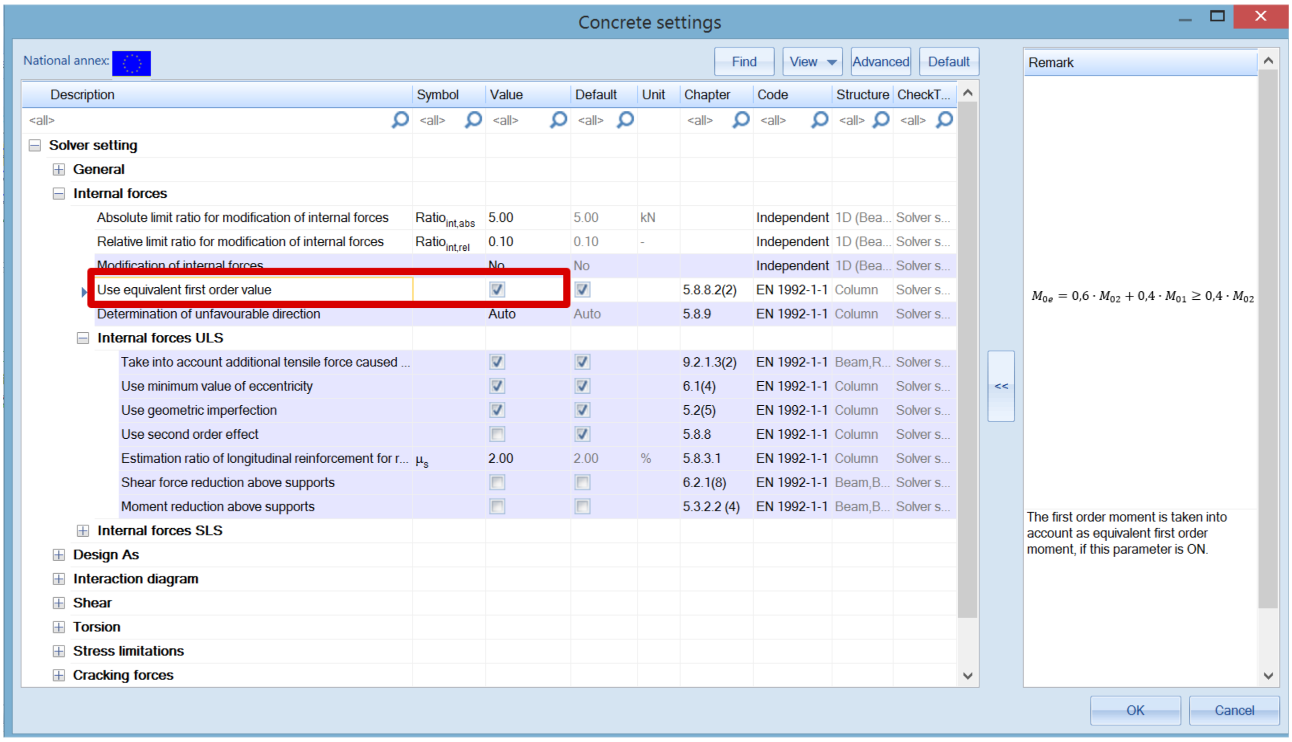
Slenderness

Axis	Braced	$L_{z/y}$ [m]	$\beta_{zz/yy}$ [-]	$l_{0z/y}$ [m]	$\lambda_{z/y}$ [-]	$\lambda_{limz/y}$ [-]	$\lambda_{z/y} > \lambda_{limz/y}$
y-y⊥	No	4.5	2	9.01	89.2	46.5	2nd order
z-z⊥	No	4.5	2	9.01	89.2	46.5	2nd order

1st order effects

1st order effects (eccentricity) are always considered.

There are 2 ways to calculate the 1st order moments and eccentricity in SCIA Engineer depending on check box **Use equivalent first order value** in Concrete Setup > Internal forces.



The 2 options are:

- **Use equivalent first order value = YES**, bending moments at the ends of the column will be taken to calculate an equivalent 1st order bending moment. This leads to the same 1st order bending moment along the whole length of the member.

  $e_{0y} = M_{0ez} / N_{Ed}$, $e_{0z} = M_{0ey} / N_{Ed}$

  With $$M_{0e} = 0{,}6 \cdot M_{02} + 0{,}4 \cdot M_{01} \geq 0{,}4 \cdot M_{02}$$

- **Use equivalent first order value = NO**, 1st order eccentricity is calculated from bending moments in current section. As a result, bending moments in each section can be different.

  $e_{0y} = M_z / N_{Ed}$, $e_{0z} = M_y / N_{Ed}$

Values of the 1st order eccentricities and moments can be viewed in Reinforcement design > 1D member > Internal Forces.

Standard output gives:

Internal forces (Design)

Linear calculation
Combination: ULS
Coordinate system: Principal
Extreme 1D: Global
Selection: All

Column B1	RECT (350; 350)
EC EN 1992-1-1:2004/AC:2008	Section 0 [dx = 0 m]

Internal forces (FEM-based)

Extreme: ULS/1 (ULS)
Type: Combination (linear)
Design situation: EN-ULS (STR/GEO) Set B

Type of load	N [kN]	M_y [kNm]	M_z [kNm]	V_y [kN]	V_z [kN]	M_x [kNm]
Internal forces (FEM-based)	-300.0	-30.0	0.0	0.0	0.0	0.0

Content: LC1

Slenderness

Axis	Braced	$L_{z/y}$ [m]	$\beta_{zz/yy}$ [-]	$l_{0z/y}$ [m]	$\lambda_{z/y}$ [-]	$\lambda_{limz/y}$ [-]	$\lambda_{z/y} > \lambda_{limz/y}$
y-y⊥	No	4.5	2	9.01	89.2	46.5	2nd order
z-z⊥	No	4.5	2	9.01	89.2	46.5	2nd order

Unfavourable direction

Second order effect and imperfections

Axis	N_{Ed} [kN]	$M_{0Edy/z}$ [kNm]	$M_{2y/z}$ [kNm]	$M_{Edy/z}$ [kNm]	$e_{0z/y}$ [mm]	$e_{iz/y}$ [mm]	$e_{0min,z/y}$ [mm]	$e_{0Edz/y}$ [mm]	$e_{2z/y}$ [mm]	$e_{Edz/y}$ [mm]
y-y⊥	-300	-30	0	-30	100	0	0	100	0	100
z-z⊥	-300	0	0	0	0	0	0	0	0	0

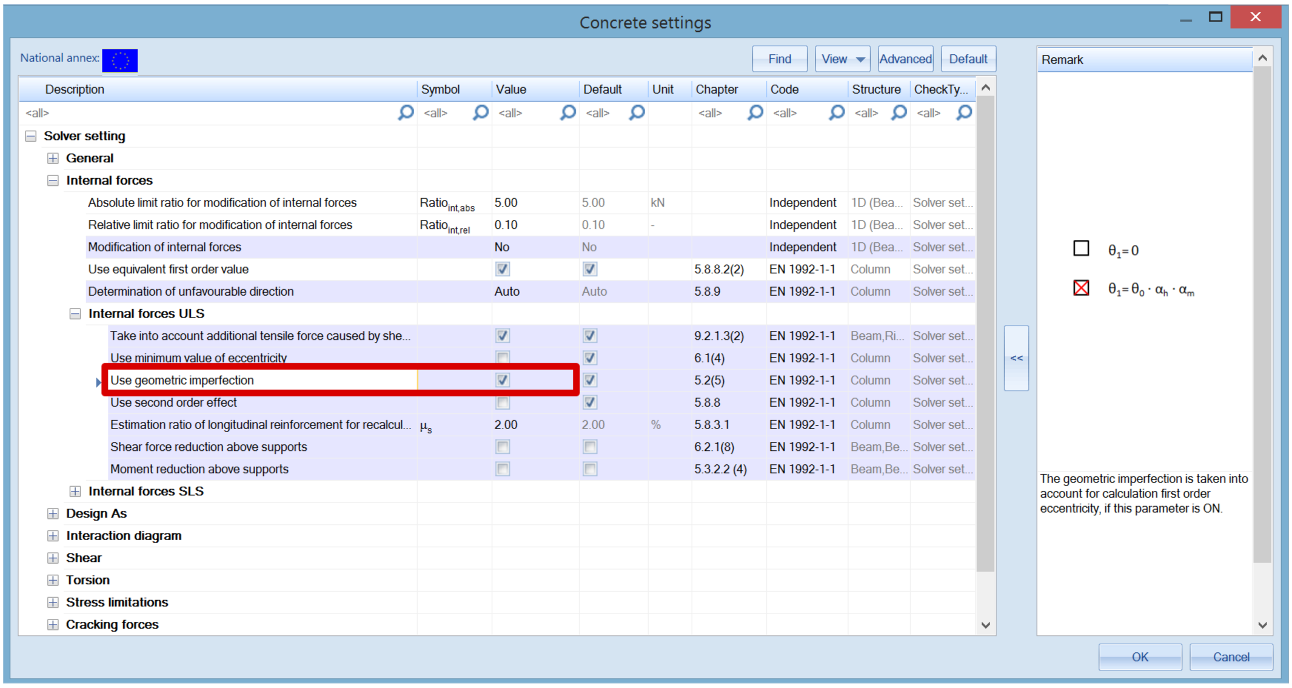
Design forces (recalculated)

Type of load	N_{Ed} [kN]	$M_{Ed,y}$ [kNm]	$M_{Ed,z}$ [kNm]	$V_{Ed,y}$ [kN]	$V_{Ed,z}$ [kN]	$M_{Ed,x}$ [kNm]
Design forces (recalculated)	-300.0	-30.0	0.0	0.0	0.0	0.0

Geometrical imperfection (art.5.2)

The effect of geometric imperfections always have to be taken into account: both in a 1st and 2nd order calculation.

Geometrical imperfection is by default activated in Concrete settings > Internal forces



In SCIA Engineer, the geometrical imperfection is represented by an inclination according to clause 5.2(5) in EN 1992-1-1.

For both axis (y and z of LCS), the inclination is calculated as followed:

$$\theta_{i,y(z)} = \theta_0 \cdot \alpha_h \cdot \alpha_{m,y(z)} \tag{5.1}$$

θ_0 basic value of inclination

α_h reduction factor for length of column or height of structure: $\alpha_h = 2/\sqrt{l}\,;\;\; 2/3 \leq \alpha_h \leq 1$

$\alpha_{m,y(z)}$ reduction factor for numbers of members: $\alpha_{m,y(z)} = \sqrt{(0{,}5 \cdot (1 + 1 / m_{y(z)}))}$

l length of column or height of structure depending on:

- isolated member l = L, where L is the length of the member
- not isolated member l = H, where H is the total height of building (buckling system).

$m_{y(z)}$ number of vertical members contributing to the total effect of the imperfection perpendicular to y(z).

Values of l and $m_{y(z)}$ will be defined in the buckling data.

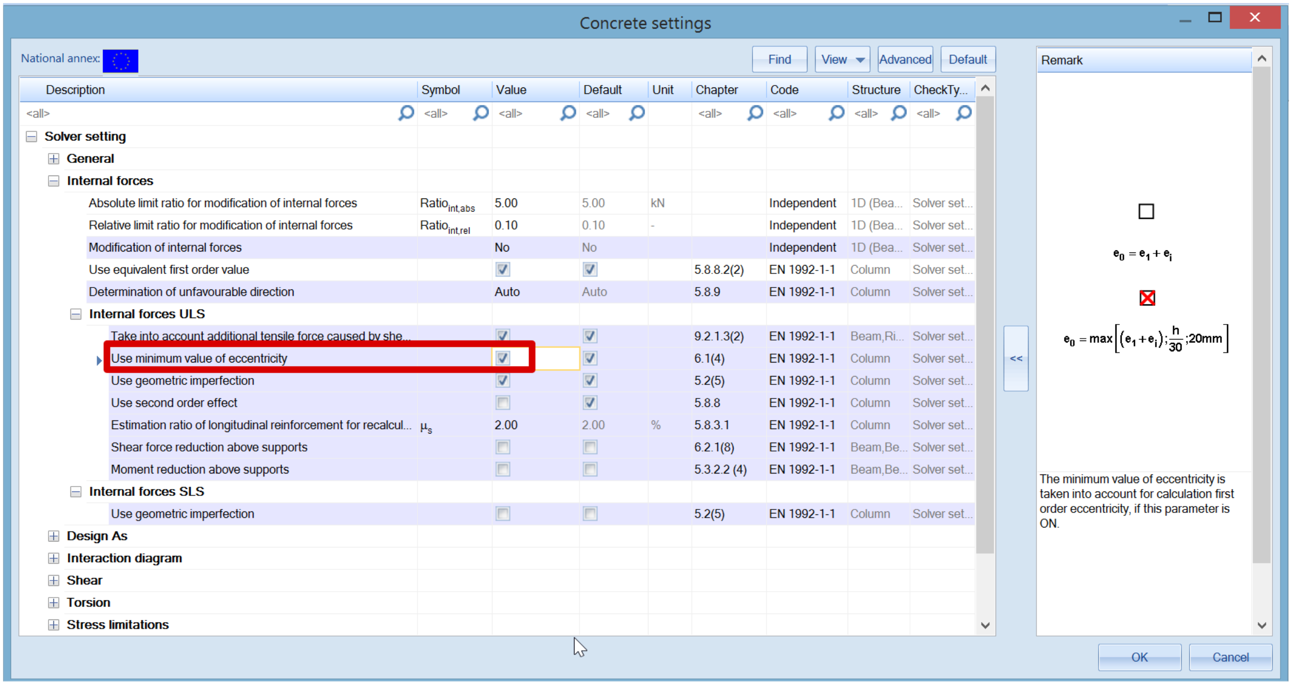
The effect of imperfection for isolated column and for structure is always taken into account as an eccentricity according to clause 5.2(7a) in EN 1992-1-1:

$$e_{i,y} = \theta_{i,z} \cdot l_{0,z} / 2,\; e_{i,z} = \theta_{i,y} \cdot l_{0,y} / 2$$

The imperfection shall be taken into account in ultimate limit states and does not need to be considered for serviceability limit states, see clause 5.2(2P) and 5.2(3) in EN 1992-1-1.

The user can set independently if the imperfection will be taken into account for ULS or SLS in the Concrete settings.

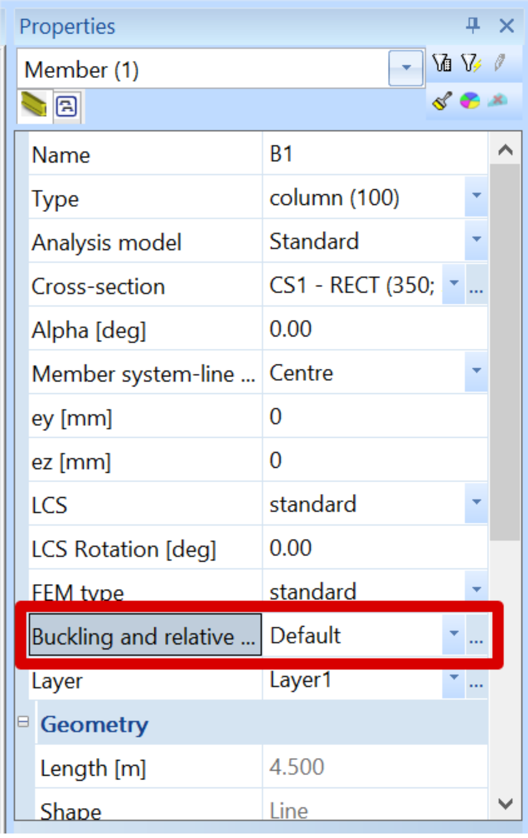
A minimum 1st order eccentricity is also calculated according to clause 6.1(4) in EN 1992-1-1. This can be viewed in Concrete settings > Internal forces > Use minimum value of eccentricity

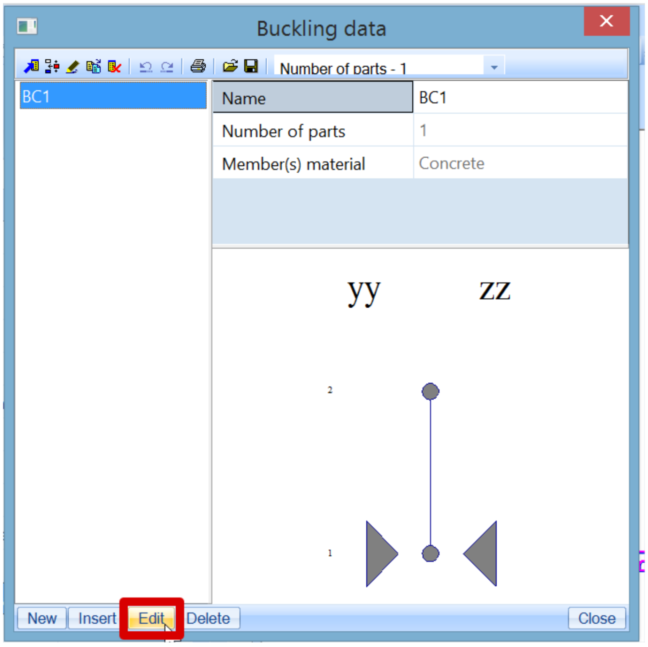


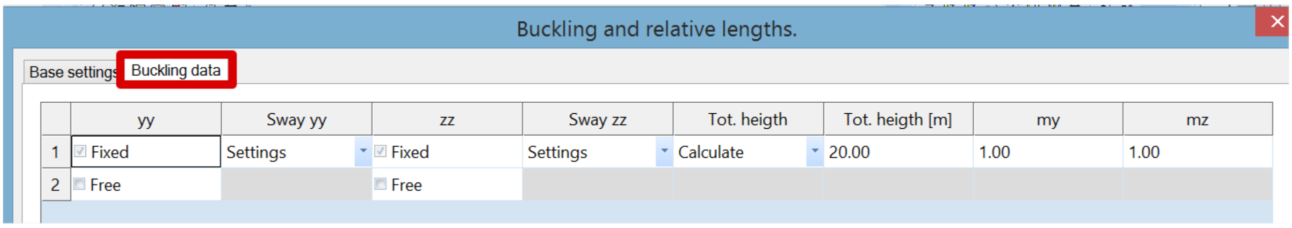
Buckling data for l and my(z)

Settings for l and my(z) for the calculation of the geometrical imperfection can be set in the properties of the columns.

Properties > Buckling and relative length > Edit > tab Buckling data:







	yy	Sway yy	zz	Sway zz	Tot. heigth	Tot. heigth [m]	my	mz
1	Fixed	Settings	Fixed	Settings	Calculate	20.00	1.00	1.00
2	Free		Free					

- **Tot. height**: set type of calculation of total height of building or length of the isolated columns.
  - *Calculate*: H tot will be calculated automatically as sum of lengths of all the members in the buckling system
  - *User*: manual input value for H_{tot} in edit box Tot. height
- **my/z**: number of vertical members contributing to the total effect of the imperfection perpendicular to y/z axis of LCS.

Eccentricities due to geometrical imperfections can be viewed in Reinforcement design > 1D member > Internal Forces:

Second order effect and imperfections

Axis	N_{Ed} [kN]	$M_{0Edy/z}$ [kNm]	$M_{2y/z}$ [kNm]	$M_{Edy/z}$ [kNm]	$e_{0z/y}$ [mm]	$e_{iz/y}$ [mm]	$e_{0min,z/y}$ [mm]	$e_{0Edz/y}$ [mm]	$e_{2z/y}$ [mm]	$e_{Edz/y}$ [mm]
y-y⊥	-405	-49.1	0	-49.1	100	21.2	20	121	0	121
z-z⊥	-405	8.1	0	8.1	0	0	-20	-20	0	-20

After calculation of 1st order eccentricity including effect of imperfection, the 1st order moment, including the effect of imperfections around y (z) axis of LCS is calculated:

$$M_{0Ed,y(z)} = N_{Ed} \cdot e_{oEd,z(y)} \qquad e_{oEd,y(z)} = e_{0,y(z)} + e_{i,y(z)} > e_{0,min,y(z)}$$

$e_{0,y(z)}$ 1st order eccentricity
$e_{i,y(z)}$ eccentricity caused by geometrical imperfection
$e_{0,min}$ minimum first order eccentricity

2nd order effects

The EN 1992-1-1 defines several methods for 2nd order effects with axial loads(general method, simplified method based on nominal stiffness, simplified method based on nominal curvature...).

In SCIA Engineer the following methods are available:

- General method according to clause 5.8.2(2) – based on a nonlinear calculation
- Simplified method based on nominal curvature according to clause 5.8.8

The simplified method is taken into account:

- For ultimate limit state
- For Member type = Column with compression according to "Determination if member is in compression"
- If option “Use second order effect” in switched ON, see Concrete settings > Internal forces.

  This option is activated by default.
- If slenderness $\lambda > \lambda_{lim}$, see chapter "Slenderness criteria"

The nominal 2nd order moment is calculated according to clause 5.8.8.2(3) in EN 1992-1-1:

$$M_{2,y(z)} = N_{Ed} \cdot e_{2,z(y)}$$

N_{Ed} design axial force
$e_{2,z(y)}$ 2nd order eccentricity

When all mentioned criteria above are met for the simplified method, the 2nd order eccentricity is calculated according to formula:

$$e_{2y(z)} = (1/r)_{z(y)} \cdot l_{0z(y)}^2 / c_{z(y)}$$

Otherwise $e_{2y(z)} = 0$

$(1/r)_{z(y)}$ curvature around z(y), calculated according to clause 5.8.8.3
$l_{0,z(y)}$ effective length of the column around z(y) – buckling length
$c_{z(y)}$ factor depending on the curvature distribution around z(y) axis according to clause 5.8.8.2(4)

- = 8, for constant 1st order bending moment (non zero) along the column and in case that equivalent bending moment is taken into account (“Use equivalent first order value” ON).
- = 10 otherwise.

$\lambda_{z(y)}$ slenderness
$\lambda_{z(y),lim}$ limit slenderness

Effective length

The effective length, or buckling length, is by default calculated by SCIA Engineer. Be aware that formulas for automatic calculation are only valid for simple structures!
Otherwise it is also possible to input the value of the effective length manually.

Automatic calculation of effective length

Calculation of effective length depends on the type of structure, sway or non-sway.

Two approximate formulas are used: one formula for a non-sway structure (resulting in a buckling factor $\beta \leq 1$) and one formula for a sway structure (resulting in a buckling factor $\beta \geq 1$):

- For a non-sway structure:

$$\beta = \frac{(\rho_1\rho_2 + 5\rho_1 + 5\rho_2 + 24)(\rho_1\rho_2 + 4\rho_1 + 4\rho_2 + 12)2}{(2\rho_1\rho_2 + 11\rho_1 + 5\rho_2 + 24)(2\rho_1\rho_2 + 5\rho_1 + 11\rho_2 + 24)}$$

- For a sway structure:

$$\beta = x\sqrt{\frac{\pi^2}{\rho_1 x} + 4}$$

with	β	the buckling factor
	L	the system length
	E	the modulus of Young
	I	the moment of inertia
	C_i	the stiffness in node i
	M_i	the moment in node i
	ϕ_i	the rotation in node i

$$x = \frac{4\rho_1\rho_2 + \pi^2\rho_1}{\pi^2(\rho_1 + \rho_2) + 8\rho_1\rho_2}$$

$$\rho_i = \frac{C_i L}{EI}$$

$$C_i = \frac{M_i}{\phi_i}$$

The values for M_i and ϕ_i are approximately determined by the internal forces and the deformations, calculated by load cases which generate deformation forms, having an affinity with the buckling form.

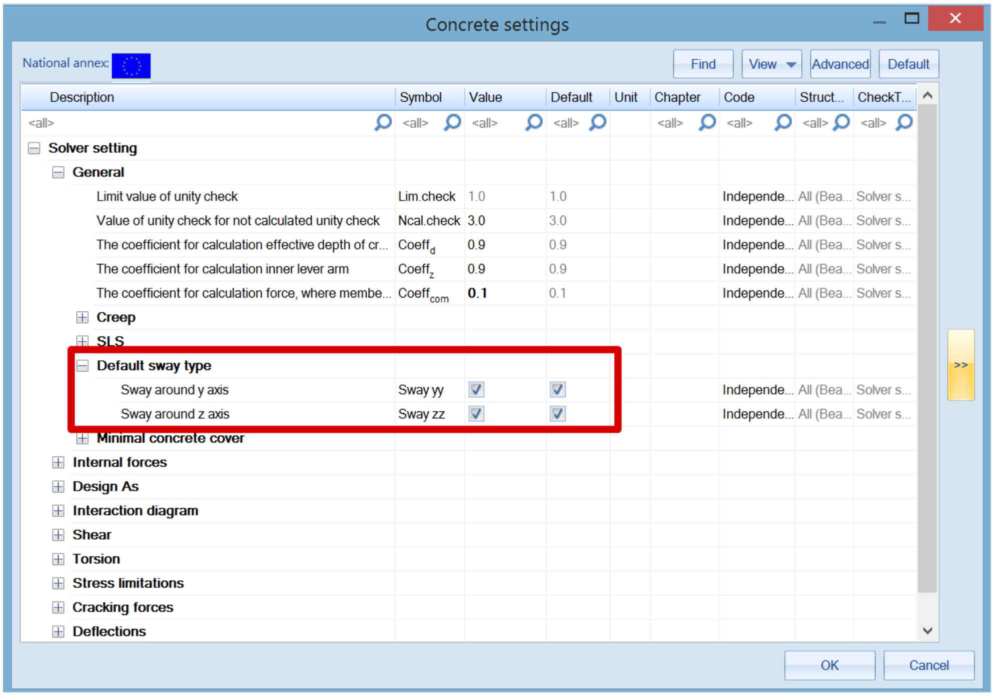
The calculation of the β ratios is automatically done when calculating the structure linearly. For this, two additional load cases are calculated in the background:

- Load case 1:
  - on the beams, the local distributed loads qy=1 N/m and qz=-100 N/m are used
  - on the columns the global distributed loads Qx =10000 N/m and Qy =10000 N/m are used.
- Load case 2:
  - on the beams, the local distributed loads qy=-1 N/m and qz=-100 N/m are used
  - on the columns the global distributed loads Qx =-10000 N/m and Qy=-10000 N/m are used.

Since these load cases, and thus the buckling ratios, are calculated during the linear calculation, it is necessary to always perform a linear calculation of the structure.

Note: The used approach gives good results for frame structures with perpendicular rigid or semi-rigid beam connections. For other cases, the user must evaluate the presented bucking ratios.

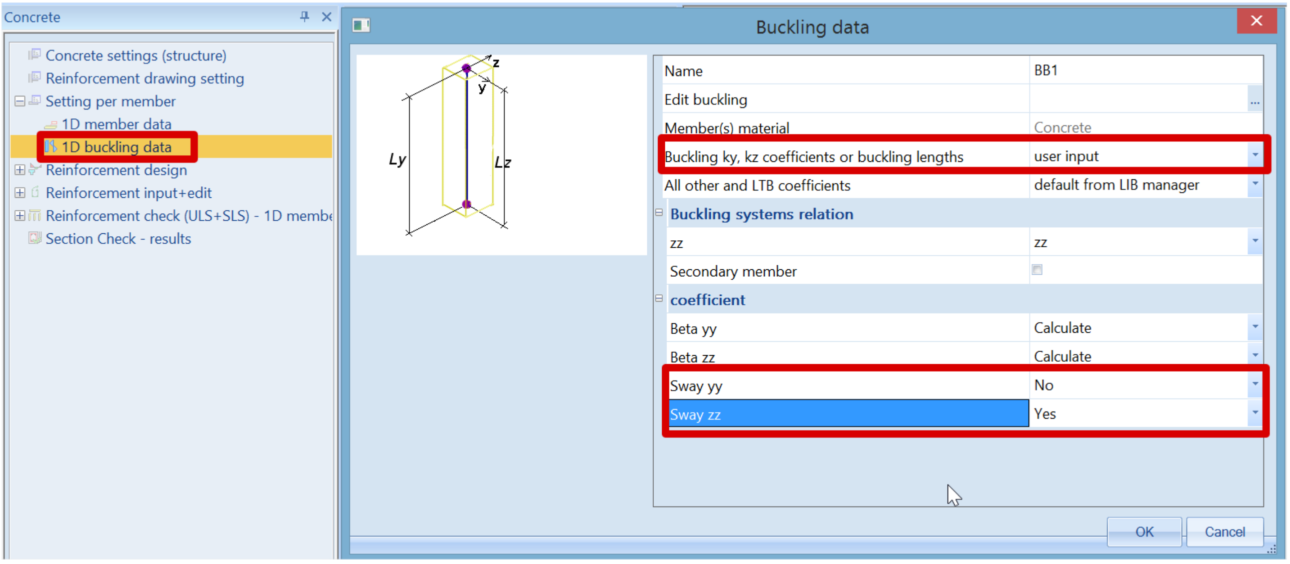
By default, the structure is considered as sway in y and z direction. It can be modified for the whole project in Concrete settings > General > Default sway type.



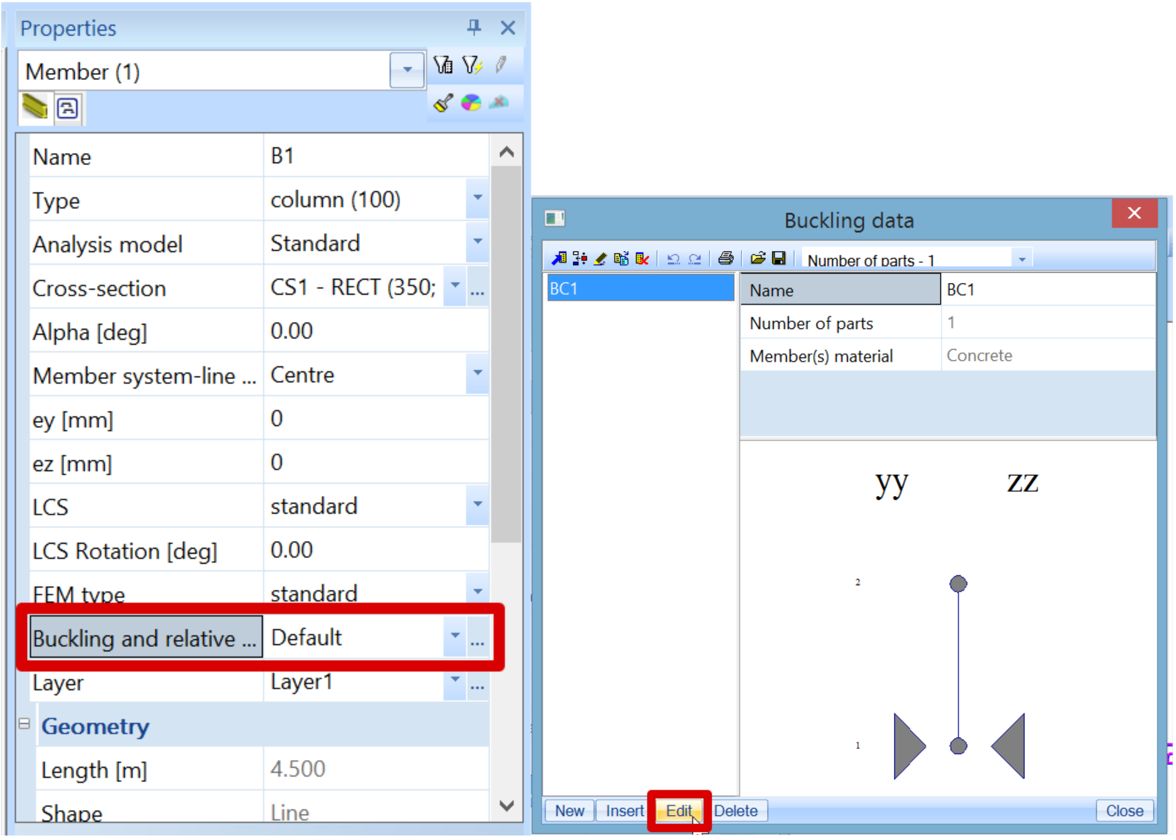
The user can change this default setting for specific columns in the project using either:

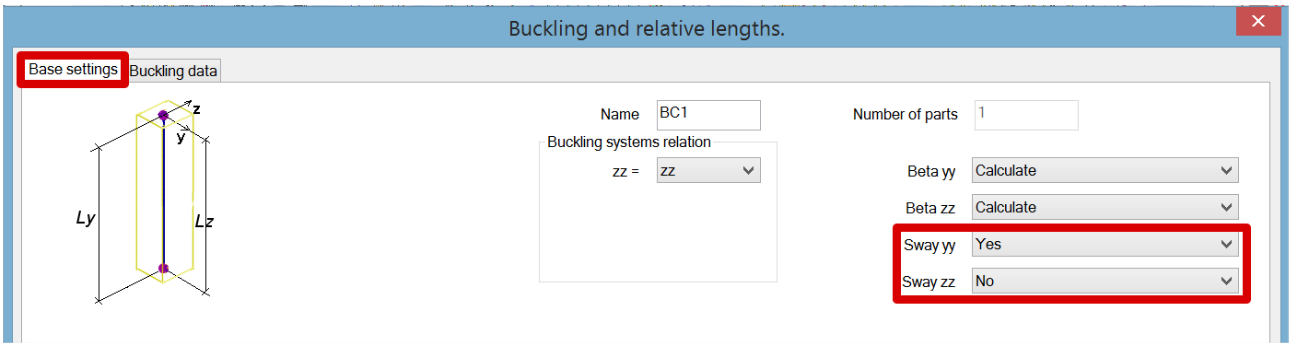
- Setting per member > 1D buckling data.

  Then select the columns on which you want to apply this modification.

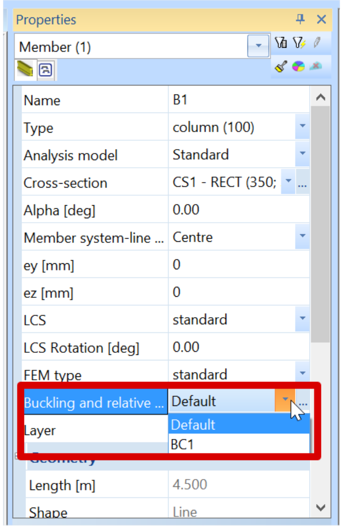


- Properties of columns > Buckling and relative length > Edit > tab Base settings:

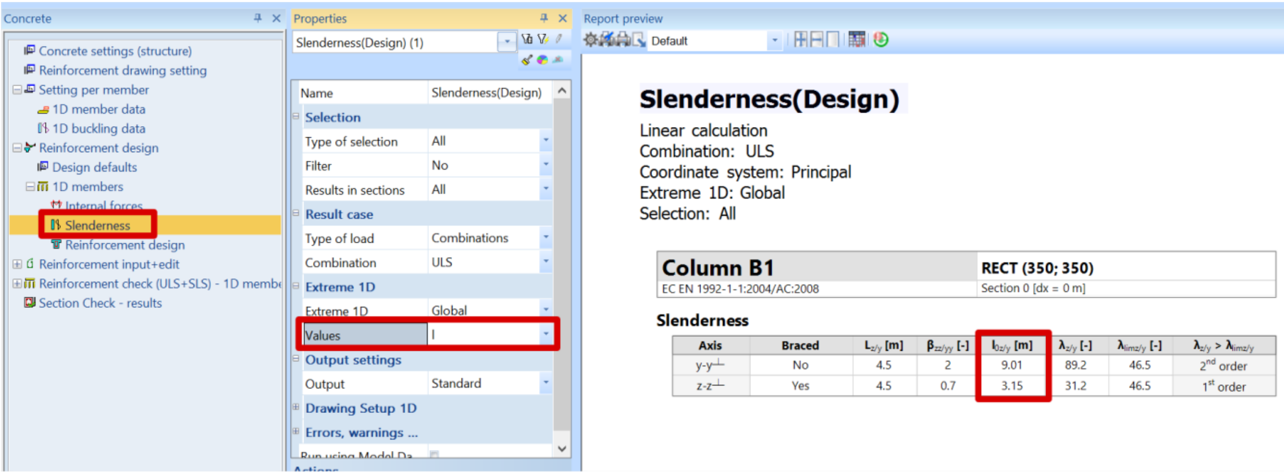




This new setting has the name, here BC1, which you can attribute to others similar columns in their properties window:

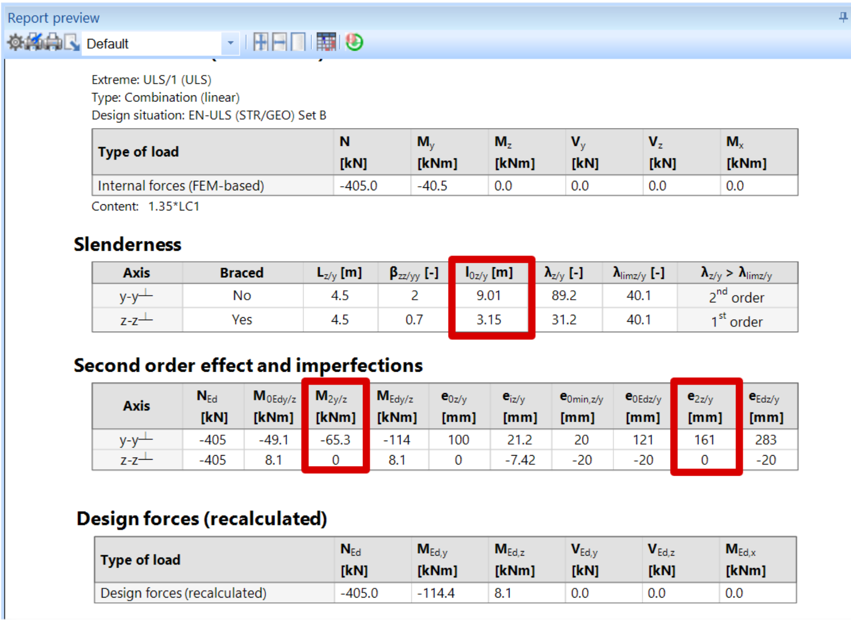


The calculated effective length can be viewed in Concrete menu > Reinforcement Design > 1D members > Slenderness:



Axis	Braced	$L_{z/y}$ [m]	$\beta_{zz/yy}$ [-]	$l_{0z/y}$ [m]	$\lambda_{z/y}$ [-]	$\lambda_{limz/y}$ [-]	$\lambda_{z/y} > \lambda_{limz/y}$
y-y	No	4.5	2	9.01	89.2	46.5	2nd order
z-z	Yes	4.5	0.7	3.15	31.2	46.5	1st order

This value is also displayed in the Standard output for Internal forces, together with the 2nd order eccentricity and the corresponding bending moment:



Extreme: ULS/1 (ULS)
Type: Combination (linear)
Design situation: EN-ULS (STR/GEO) Set B

Type of load	N [kN]	M_y [kNm]	M_z [kNm]	V_y [kN]	V_z [kN]	M_x [kNm]
Internal forces (FEM-based)	-405.0	-40.5	0.0	0.0	0.0	0.0

Content: 1.35*LC1

Slenderness

Axis	Braced	$L_{z/y}$ [m]	$\beta_{zz/yy}$ [-]	$l_{0z/y}$ [m]	$\lambda_{z/y}$ [-]	$\lambda_{limz/y}$ [-]	$\lambda_{z/y} > \lambda_{limz/y}$
y-y	No	4.5	2	9.01	89.2	40.1	2nd order
z-z	Yes	4.5	0.7	3.15	31.2	40.1	1st order

Second order effect and imperfections

Axis	N_{Ed} [kN]	$M_{0Edy/z}$ [kNm]	$M_{2y/z}$ [kNm]	$M_{Edy/z}$ [kNm]	$e_{0z/y}$ [mm]	$e_{iz/y}$ [mm]	$e_{0min,z/y}$ [mm]	$e_{0Edz/y}$ [mm]	$e_{2z/y}$ [mm]	$e_{Edz/y}$ [mm]
y-y	-405	-49.1	-65.3	-114	100	21.2	20	121	161	283
z-z	-405	8.1	0	8.1	0	-7.42	-20	-20	0	-20

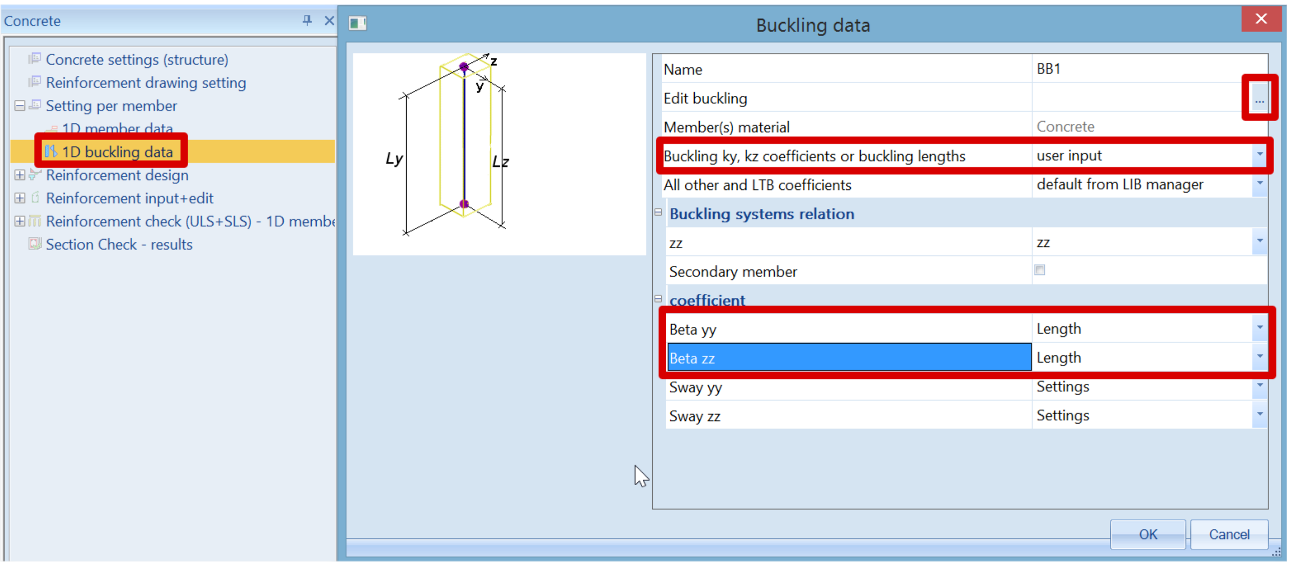
Design forces (recalculated)

Type of load	N_{Ed} [kN]	$M_{Ed,y}$ [kNm]	$M_{Ed,z}$ [kNm]	$V_{Ed,y}$ [kN]	$V_{Ed,z}$ [kN]	$M_{Ed,x}$ [kNm]
Design forces (recalculated)	-405.0	-114.4	8.1	0.0	0.0	0.0

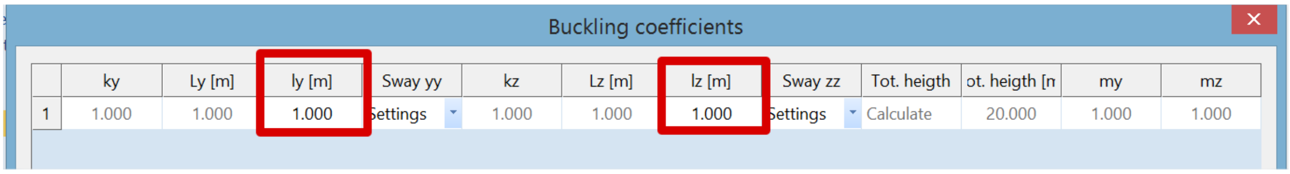
Manual input of effective length

The same 2 options as seen for the automatic calculation, allow to manually input the effective length.

- Setting per member > 1D buckling data.

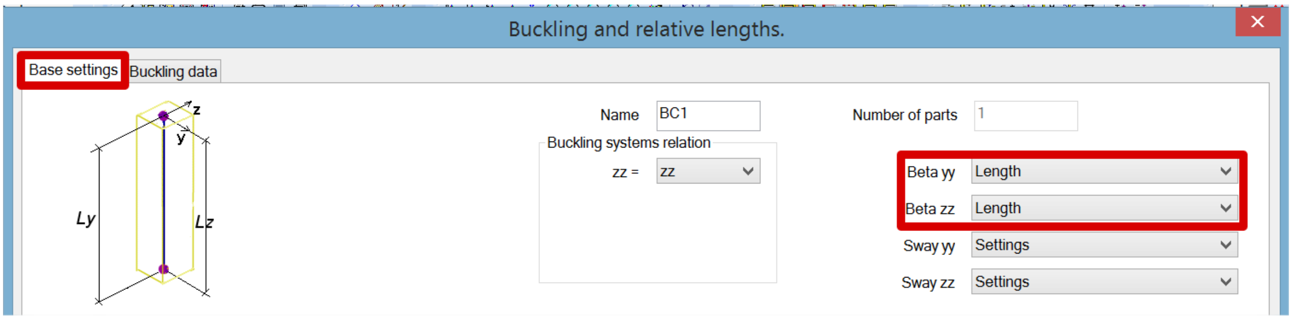


Click on “Edit buckling” to input the values:

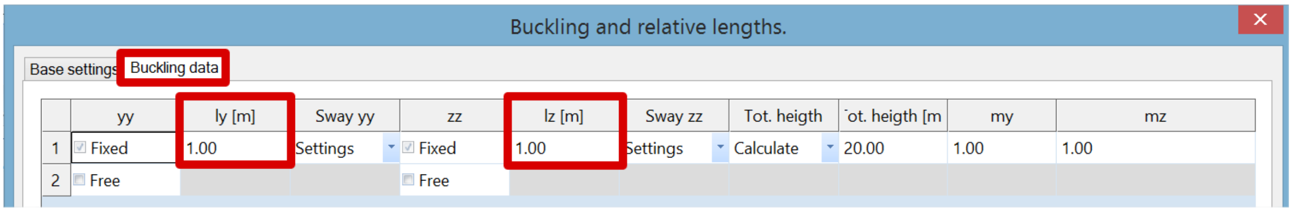


- Properties of columns > Buckling and relative length > Edit:

1st tab: Base settings



2nd tab: Buckling data



Recalculated internal forces

In Concrete Menu > Reinforcement Design > 1D member > Internal forces.

The design moment, M_{Ed}, is equal to $M_{Ed} = M_{0Ed} + M_2$.

M_2 2nd order bending moment
M_{0Ed} bending moment taking into account 1st order and geometrical imperfections

Example: '2nd order.esa'

Geometry
Column cross-section: RECT 350x350mm²
Height: 4,5 m
Concrete grade: C45/55

Concrete Setup
All of the default values are kept.
This means that geometrical imperfection and 2nd order effects are taken into account.

Loads
Load configuration: N_d = 405,00 kN
M_{yd} = 40,50 kNm
M_{zd} = 0 kNm

Buckling data
Sway type is set by default.
Calculation of the effective length is done automatically by the software.

Slenderness criterion
Check if 2nd order calculation is required following art 5.8.3.1:

Since $\lambda > \lambda_{lim}$, a 2nd order calculation will be required.

Note: the program automatically takes into account a second order moment if required. So, this check is just extra information for the user.

Internal forces

Ask for M_{Ed} in Reinforcement design > 1D member > Internal forces.
The Standard output is chosen:

Internal forces (FEM-based)

Extreme: ULS/1 (ULS)
Type: Combination (linear)
Design situation: EN-ULS (STR/GEO) Set B

Type of load	N [kN]	M_y [kNm]	M_z [kNm]	V_y [kN]	V_z [kN]	M_x [kNm]
Internal forces (FEM-based)	-405.0	-40.5	0.0	0.0	0.0	0.0

Content: 1.35*LC1

Slenderness

Axis	Braced	$L_{z/y}$ [m]	$\beta_{zz/yy}$ [-]	$l_{0z/y}$ [m]	$\lambda_{z/y}$ [-]	$\lambda_{limz/y}$ [-]	$\lambda_{z/y} > \lambda_{limz/y}$
y-y┴	No	4.5	2	9.01	89.2	40.1	2nd order
z-z┴	No	4.5	2	9.01	89.2	40.1	2nd order

Second order effect and imperfections

Axis	N_{Ed} [kN]	$M_{0Edy/z}$ [kNm]	$M_{2y/z}$ [kNm]	$M_{Edy/z}$ [kNm]	$e_{0z/y}$ [mm]	$e_{iz/y}$ [mm]	$e_{0min,z/y}$ [mm]	$e_{0Edz/y}$ [mm]	$e_{2z/y}$ [mm]	$e_{Edz/y}$ [mm]
y-y┴	-405	-49.1	-65.3	-114	100	21.2	20	121	161	283
z-z┴	-405	8.6	65.3	73.9	0	-21.2	-20	-21.2	-161	-183

Design forces (recalculated)

Type of load	N_{Ed} [kN]	$M_{Ed,y}$ [kNm]	$M_{Ed,z}$ [kNm]	$V_{Ed,y}$ [kN]	$V_{Ed,z}$ [kN]	$M_{Ed,x}$ [kNm]
Design forces (recalculated)	-405.0	-114.4	73.9	0.0	0.0	0.0

Results

The results for the reinforcement design are shown below:

Design of longitudinal reinforcement

$A_{s,z+}$: 1.35*LC1 : N_{Ed} = -405 kN, M_{Edy} = -114 kNm, M_{Edz} = 74 kNm
$A_{s,z-}$: 1.35*LC1 : N_{Ed} = -405 kN, M_{Edy} = -114 kNm, M_{Edz} = 74 kNm
$A_{s,y+}$: 1.35*LC1 : N_{Ed} = -405 kN, M_{Edy} = -114 kNm, M_{Edz} = 74 kNm
$A_{s,y-}$: 1.35*LC1 : N_{Ed} = -405 kN, M_{Edy} = -114 kNm, M_{Edz} = 74 kNm

Required

Edge	Layer	y [m]	z [m]	$A_{s,stat}$ [mm^2]	$A_{s,detmin}$ [mm^2]	$A_{s,detmax}$ [mm^2]	$\Delta A_{s,tor}$ [mm^2]	$A_{s,req}$ [mm^2]	$A_{s,reqbar}$ [mm^2]	Reinf
1	1	0	-0.129	827	201	1180	0	827	1005	6ϕ16
2	1	0.129	0	458	201	1180	0	458	603	4ϕ16
3	1	0	0.129	827	201	1180	0	827	1005	6ϕ16
4	1	-0.129	0	458	201	1180	0	458	603	4ϕ16

Determination type of calculation

Calculation maximum bending moments around y and z axis

$M_{y,max}$ = -114 kNm $M_{z,max}$ = 73.9 kNm

Calculation maximum ratio of bending moments

ρ_M = 0.646

Determination type of calculation

ρ_M = 0.646 > $\rho_{M,lim}$ = 0.1 or $|M_{y,max}|$ = 0 kNm and $|M_{z,max}|$ = 0 kNm => Biaxial method

Note that biaxial bending method was used for reinforcement calculation.

Theoretical background

An extended manual that also contains some theoretical background can be found through the help menu of SCIA Engineer: