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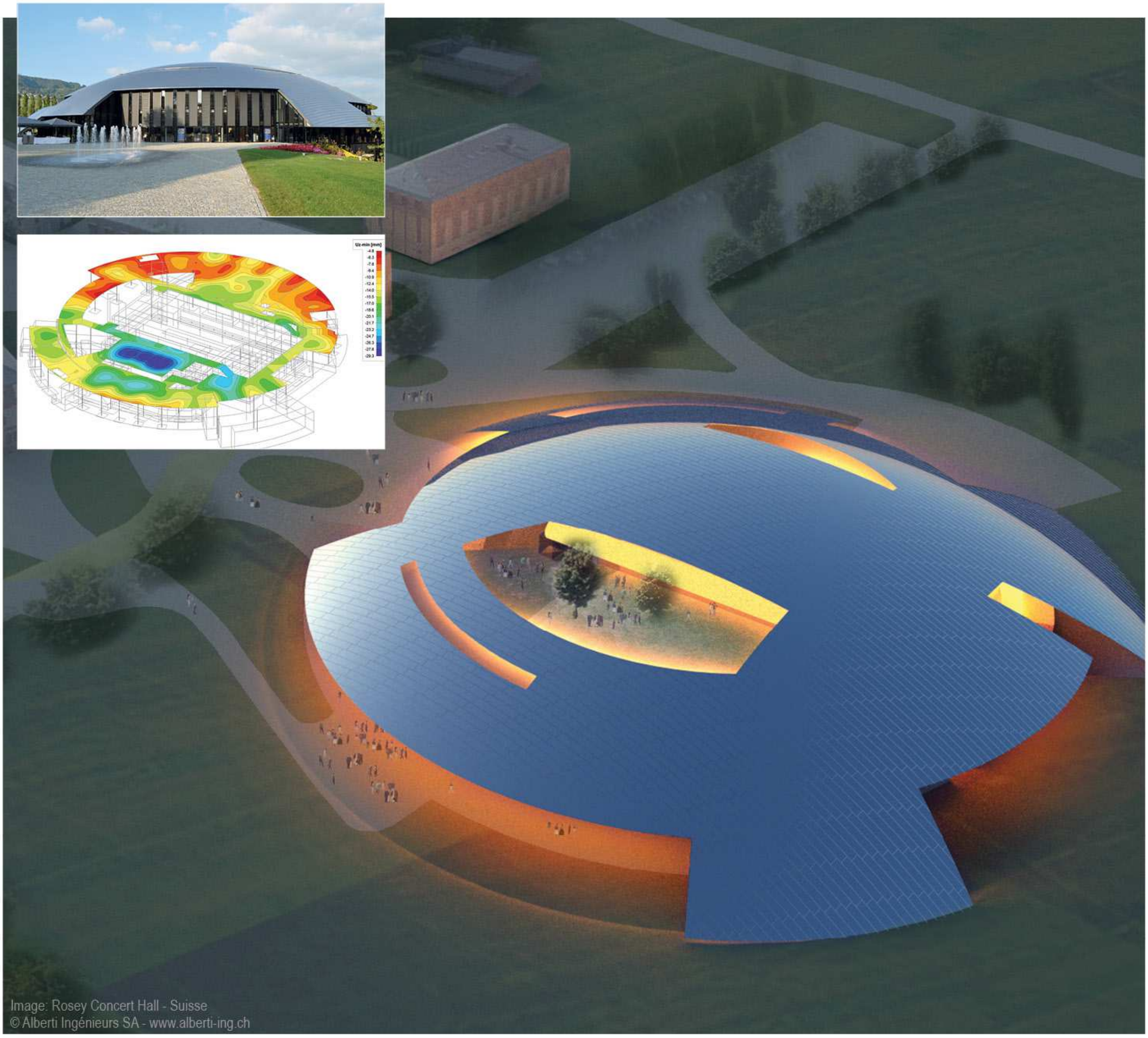

Advanced Concept Training

Reinforced concrete (EN 1992) – 1D members

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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.

For 2D members and advanced concrete calculations reference is made to the respective training documents.

Materials

Verification by the partial factor method

Design values (art. 2.4.2)

Partial factors for materials (art. 2.4.2.4)

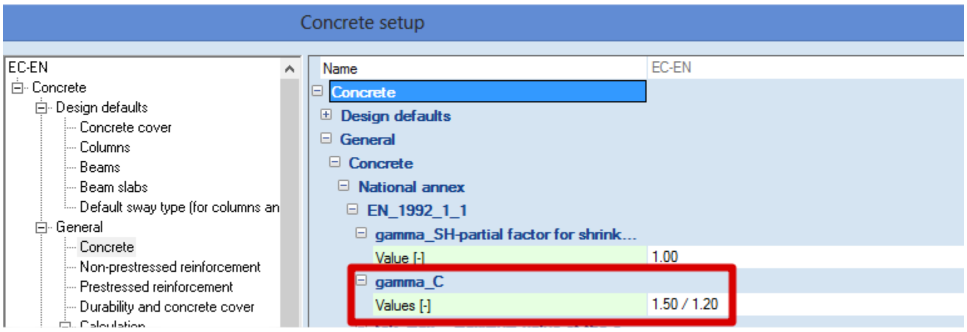
Partial factors for materials for ultimate limit states, γ_c and γ_s should be used.

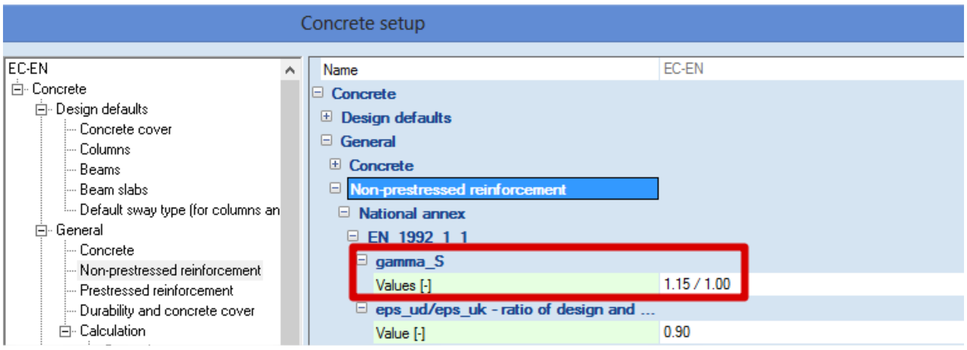
The recommended values of γ_c and γ_s for 'persistent & transient' and 'accidental, design situations are given in the following table. These are not valid for fire design for which reference should be made to EN 1992-1-2.

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:





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

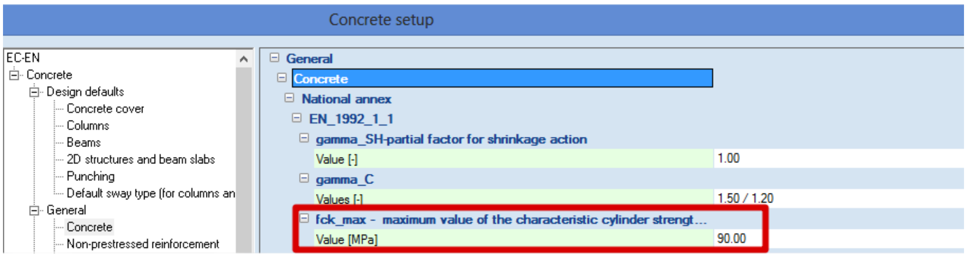
The following clauses give principles and rules for normal and high strength concrete.

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 $f_{ck,cube}$.

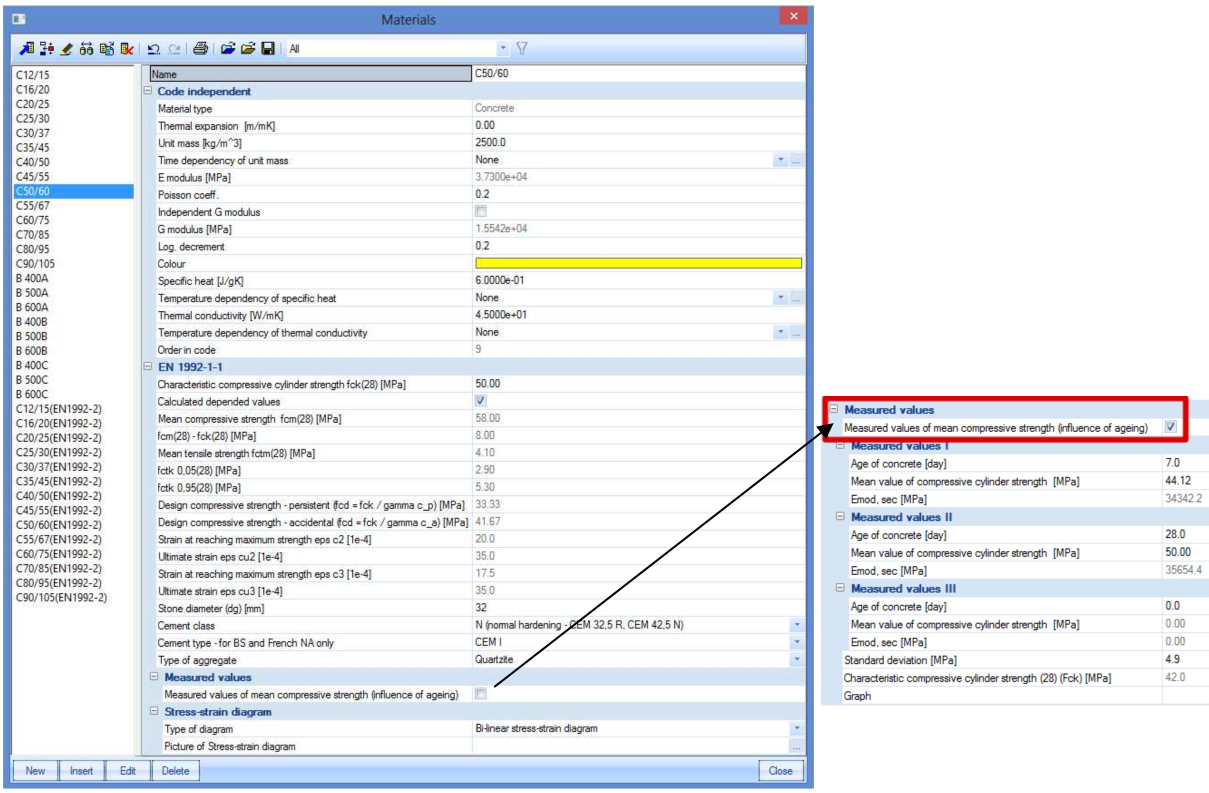
The strength classes in this code are based on the characteristic cylinder strength f_{ck} determined at 28 days with a maximum value of C_{max}.

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:

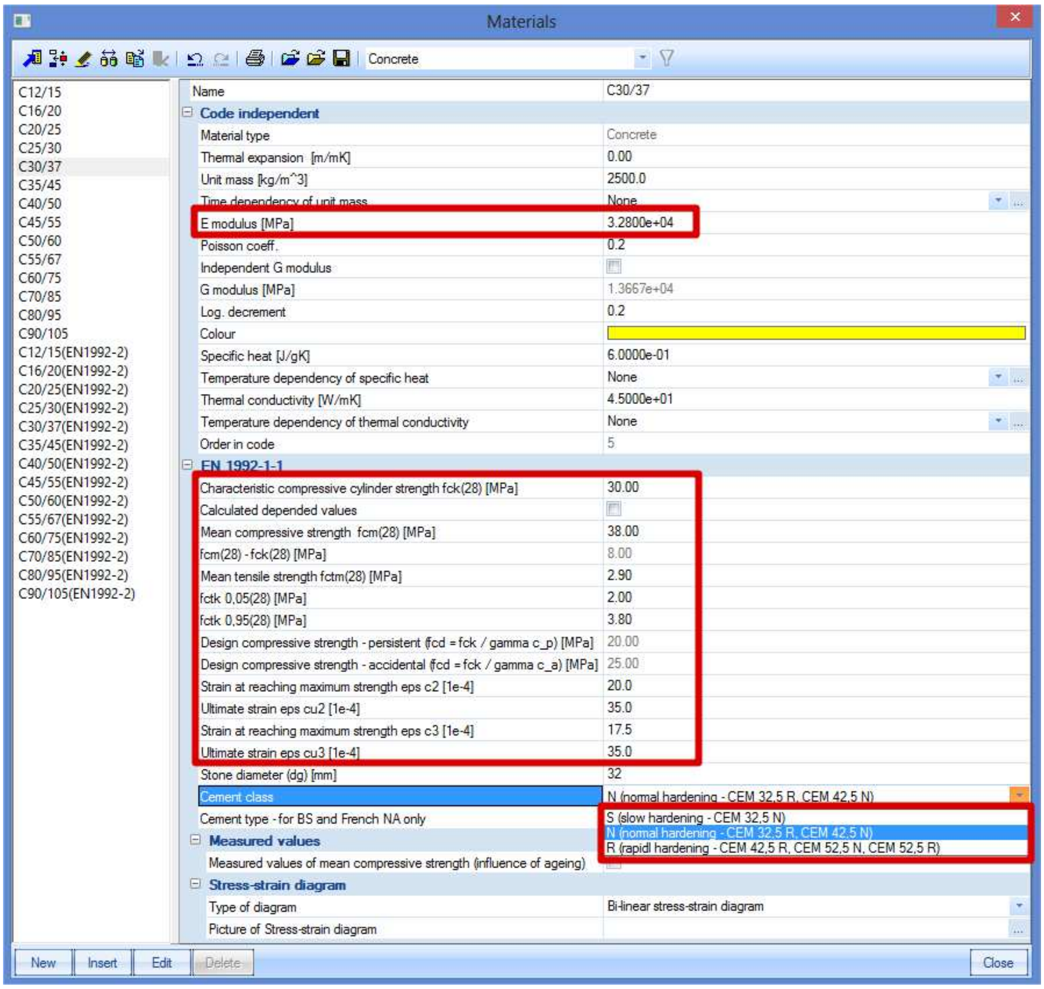
$$f_{cm}(t) = \beta_{cc}(t)\, f_{cm} \qquad (3.1)$$

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

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

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

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

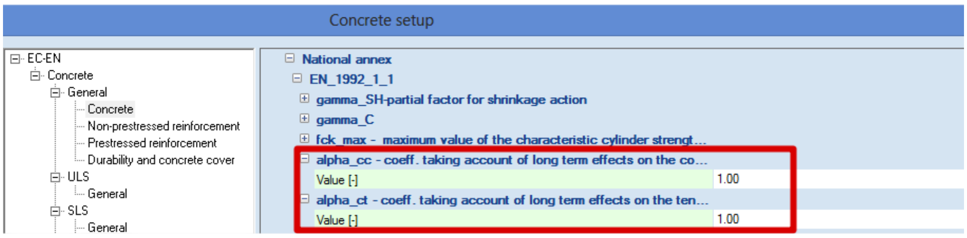
$$f_{ctd} = \alpha_{ct} f_{ctk,0,05} / \gamma_C \qquad (3.16)$$

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:



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%.

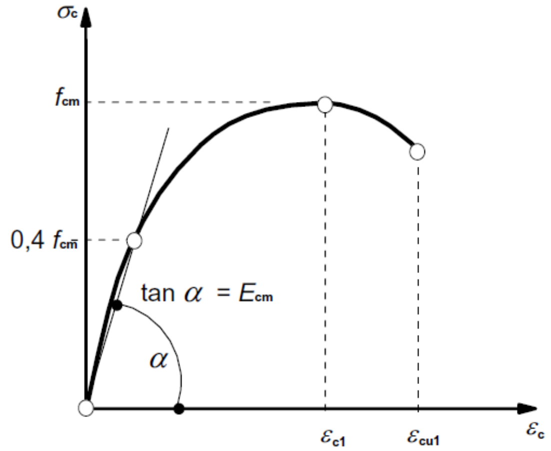


Figure 3.2: Schematic representation of the stress-strain relation for structural analysis (the use $0{,}4f_{cm}$ for the definition of E_{cm} is approximate).

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 creep coefficient, $\varphi(t,t_0)$ is related to E_c, the tangent modulus, which may be taken as 1,05 E_{cm}.

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 $f$ck (t_0) at an age t_0, the age of concrete at the time of loading.

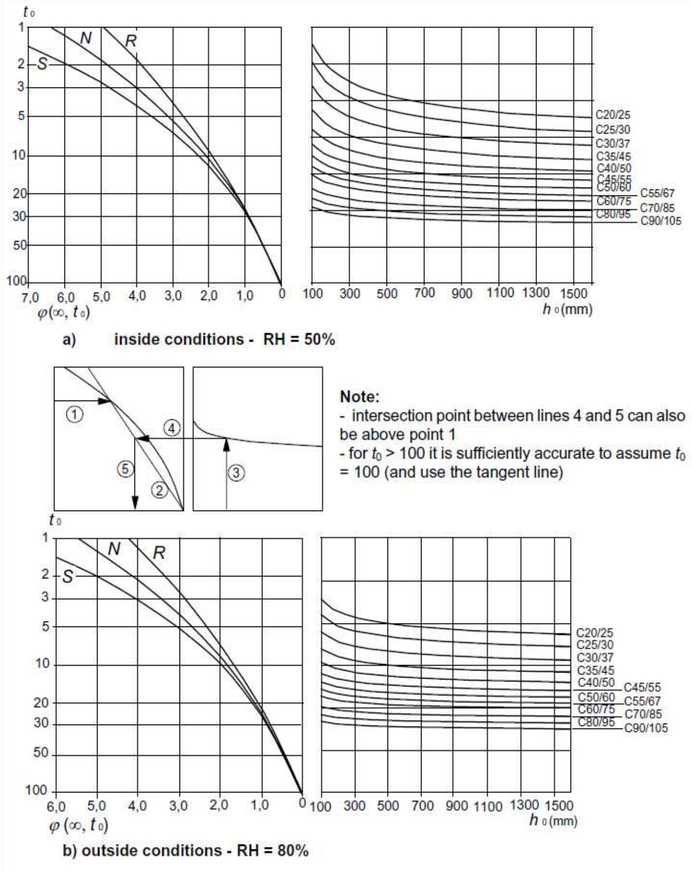
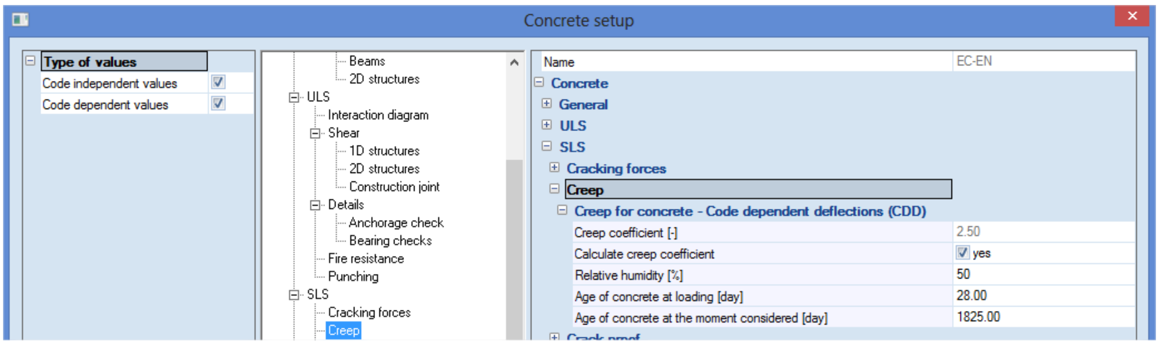
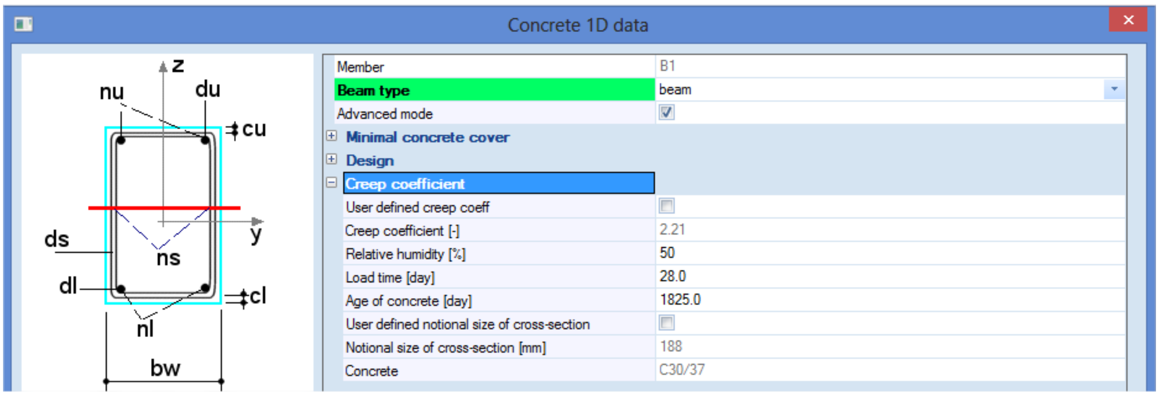


Figure 3.1: Method for determining the creep coefficient $\varphi(\infty, t_0)$ for concrete under normal environmental conditions

All these parameters can be inputted in the setup for concrete or in the beam concrete data:





The creep deformation of concrete $\varepsilon_{cc}(\infty,t_0)$ at time $t = \infty$ for a constant compressive stress σ_c applied at the concrete age t_0, is given by:

$$\varepsilon_{cc}(\infty,t_0) = \varphi(\infty,t_0) \cdot (\sigma_c / E_c) \tag{3.6}$$

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:

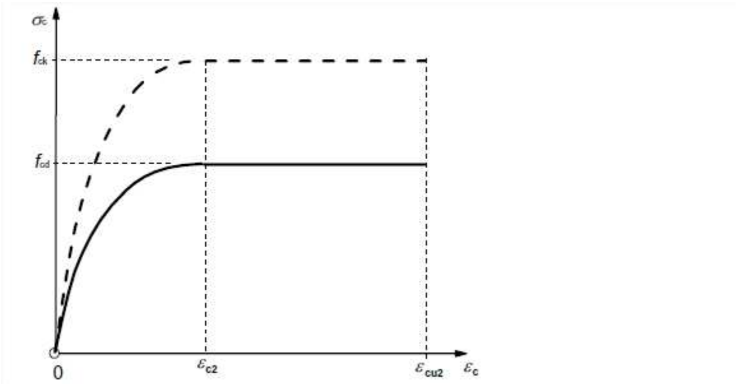


Figure 3.3: Parabola-rectangle diagram for concrete under compression.

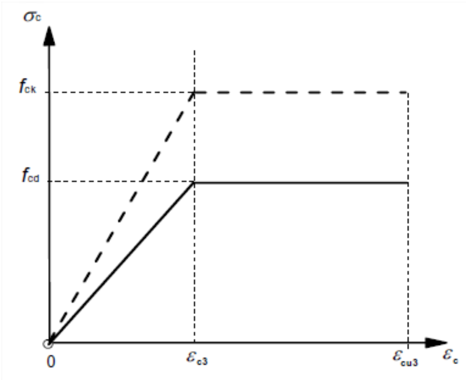


Figure 3.4: Bi-linear stress-strain relation.

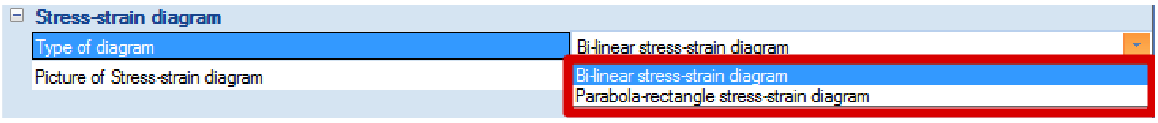
ε_{c2} is the strain at reaching the maximum strength in the parabola-rectangle diagram

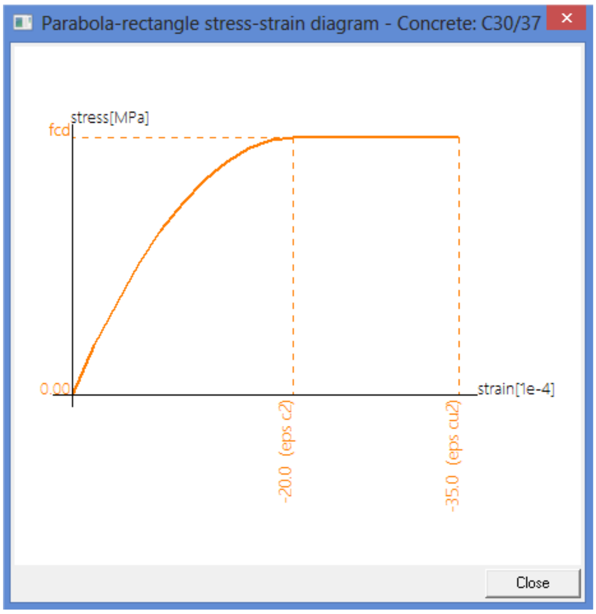
ε_{cu2} is the ultimate strain in the parabola-rectangle diagram

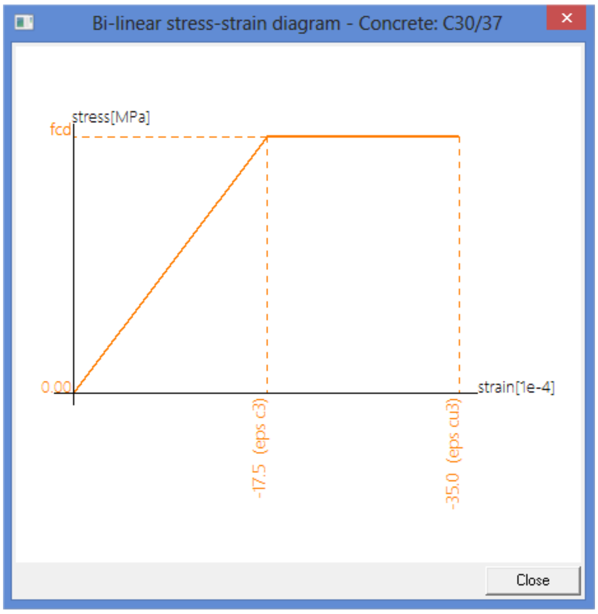
ε_{c3} is the strain at reaching the maximum strength in the bi-linear diagram

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

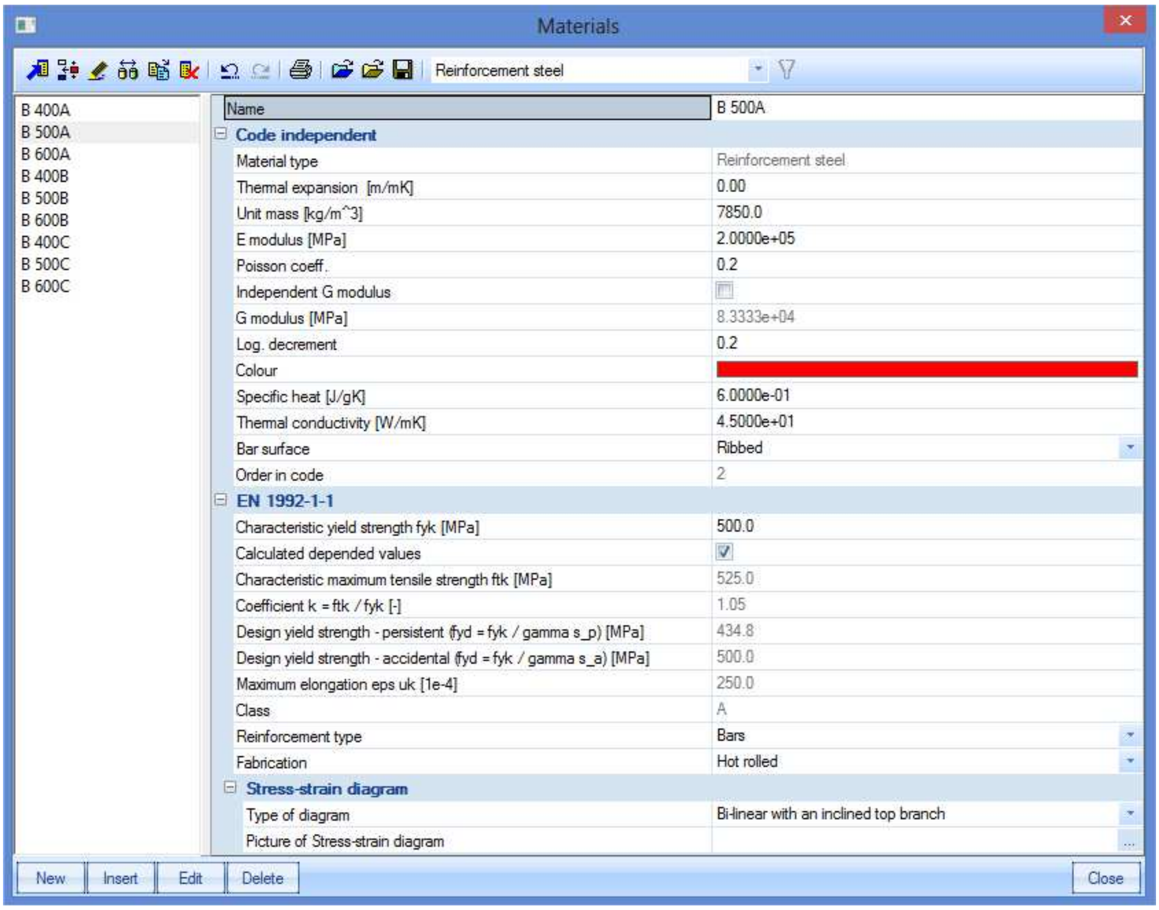
The following clauses give principles and rules for reinforcement which is in the form of bars, de-coiled rods, welded fabric and lattice girders. They do not apply to specially coated bars.

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.

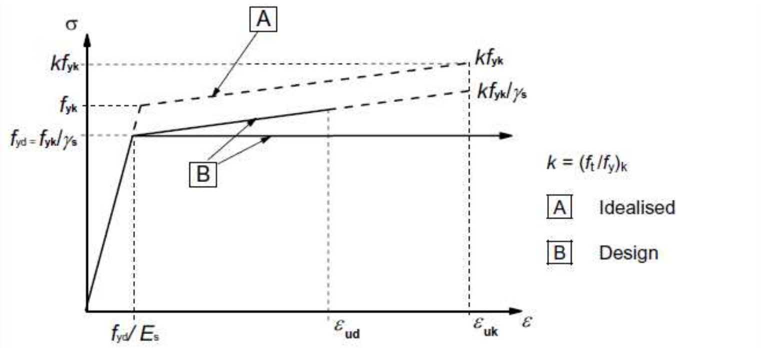
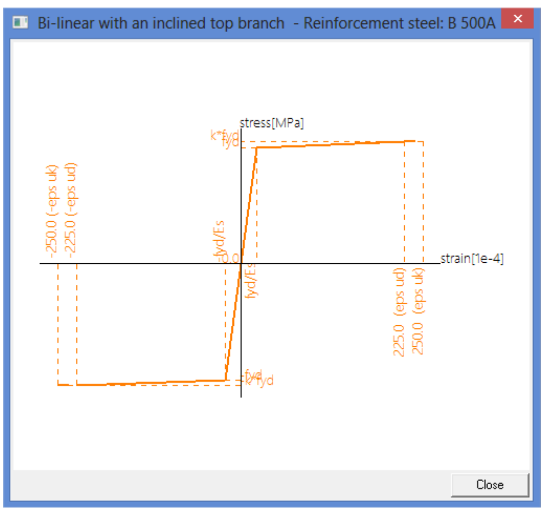
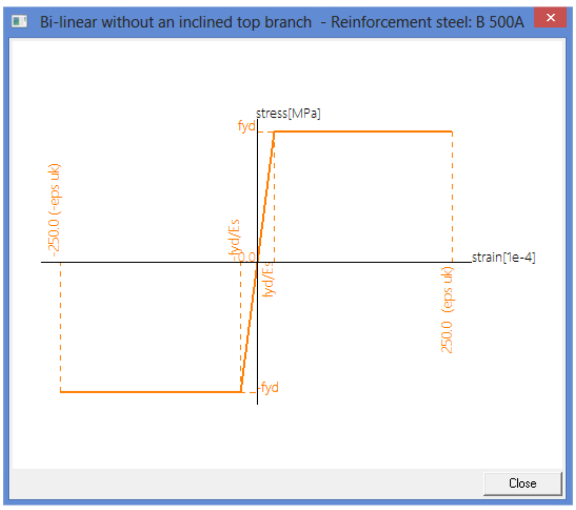


Figure 3.8: Idealised and design stress-strain diagrams for reinforcing steel (for tension and compression)

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





Durability and cover to reinforcement

Environmental conditions (art 4.2)

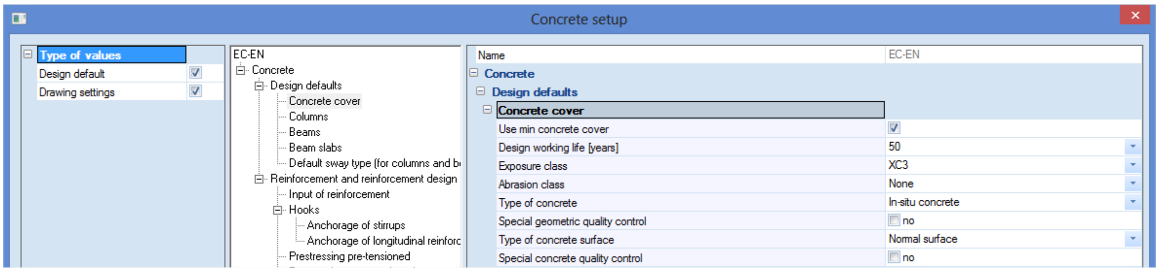
Exposure conditions are chemical and physical conditions to which the structure is exposed in addition to the mechanical actions.

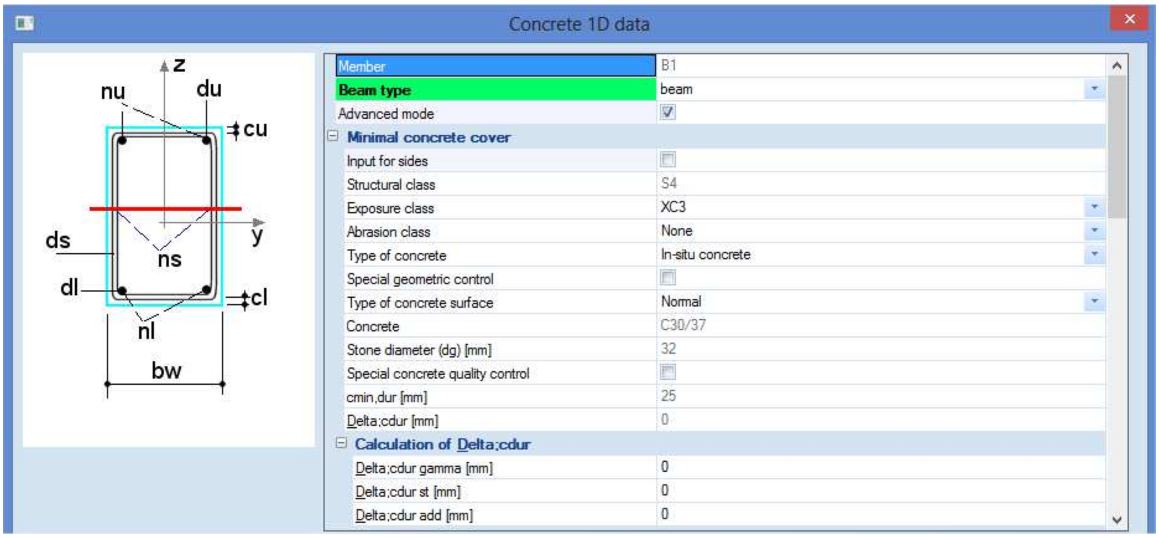
Environmental conditions are classified according to Table 4.1:

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 setup the user can choose the desired exposure class. All items in blue can be overwritten in the data concrete on a member.





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:

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

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 $c_{min,dur}$.

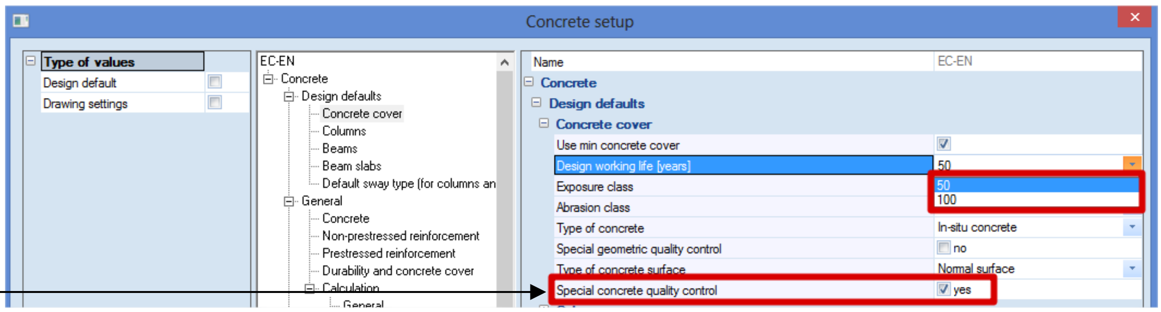
The recommended Structural Class (design working life of 50 years) is S4 for the indicative concrete strengths (given in Annex E of EN 1992-1-1). The recommended minimum Structural Class is S1.

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 can be defined in the design defaults:



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.

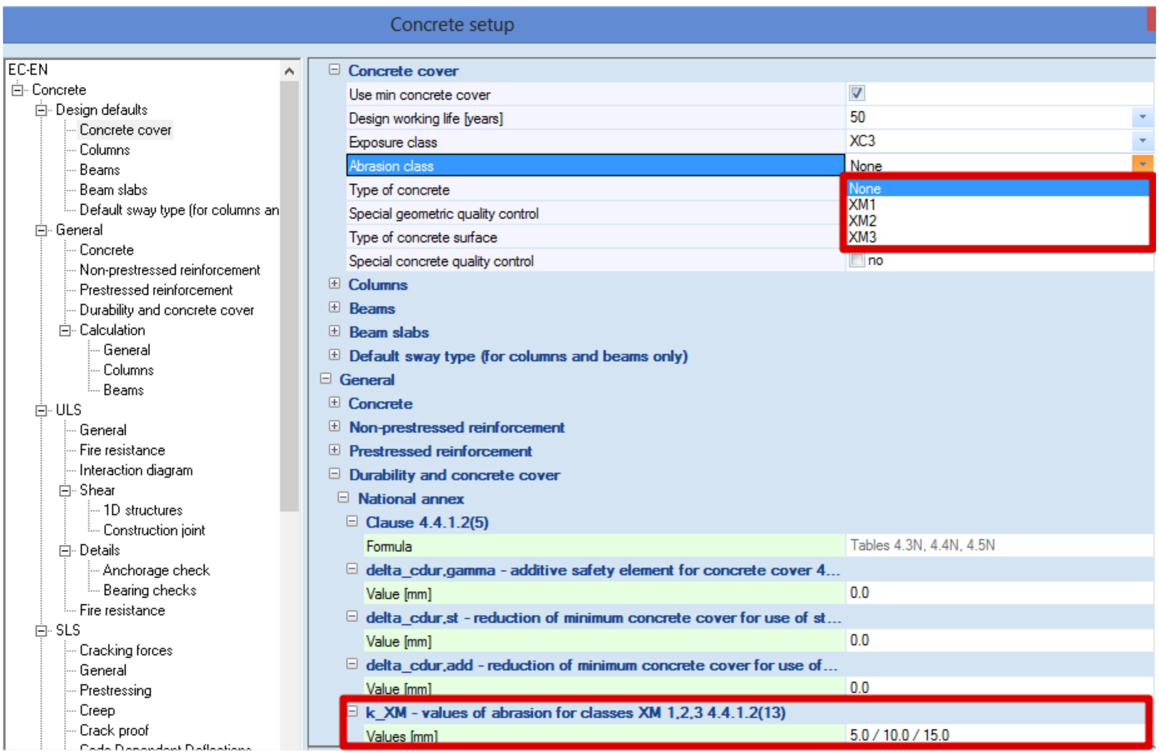
For concrete with additional protection (e.g. coating) the minimum cover may be reduced by $\Delta c_{dur,add}$.

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 and the values of k_1, k_2 and k_3 can be inputted in the concrete setup:



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:

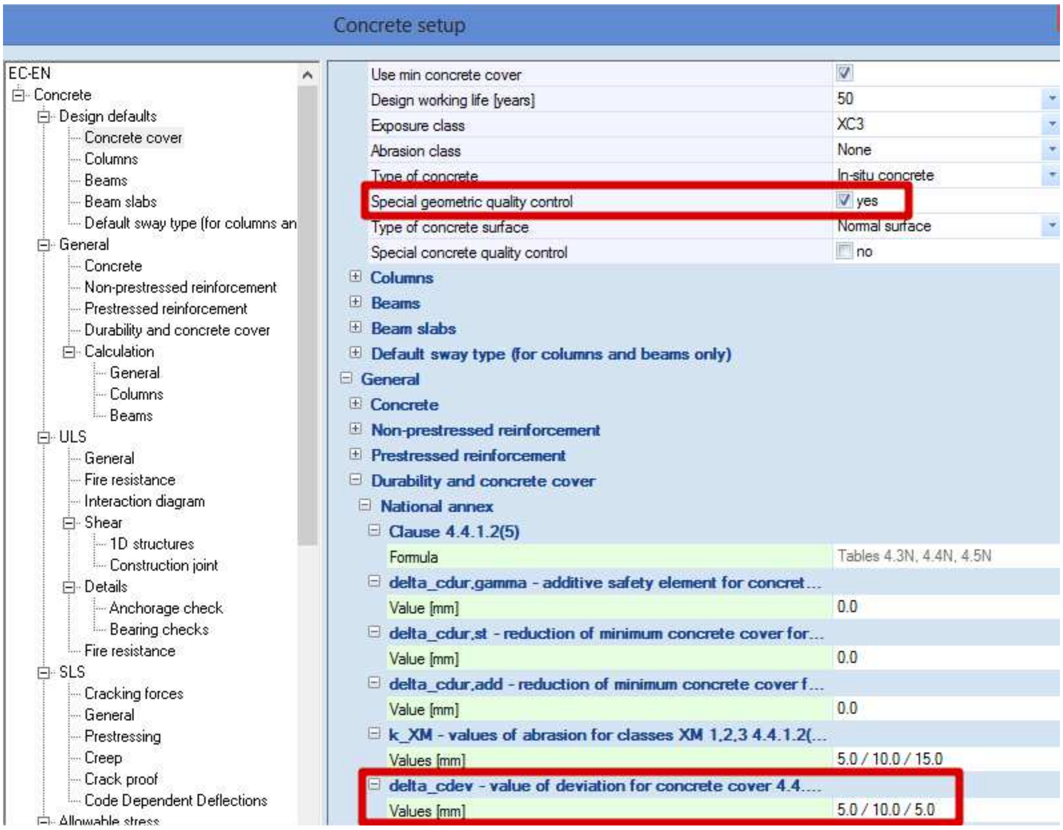
- *where fabrication is subjected to a quality assurance system, in which the monitoring includes measurements of the concrete cover, the allowance in design for deviation Δc_{dev} may be reduced:*

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

- *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:*

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

Also these values can be found in the concrete setup:



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.

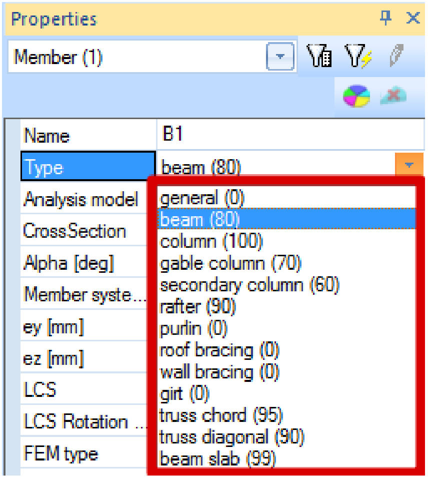
5) A column is a member for which the section depth does not exceed 4 times its width and the height is at least 3 times the section depth. Otherwise it should be considered as a wall.

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:

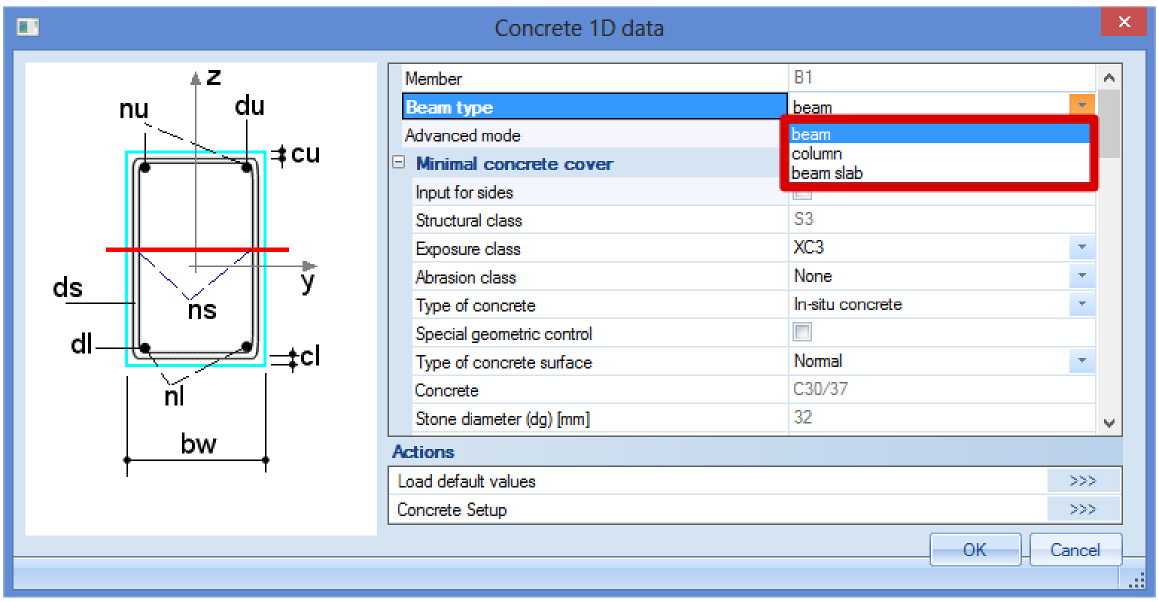


The Beam calculation is used for the Types 'General', 'Beam, 'Rafter', 'Purlin', 'Roof bracing', 'Wall bracing', 'Girt', 'Truss chord' and 'Truss diagonal'.

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'.

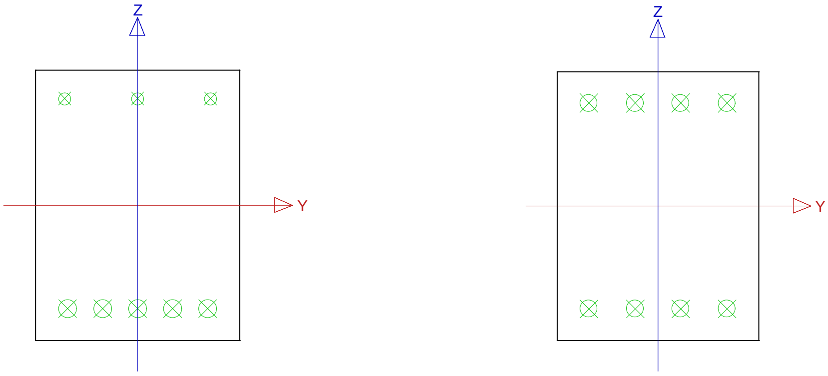
Be careful when Member data are added to an element, via Concrete menu > 1D member > Member data. Also there, the user has the choice for the 3 different analysis models, by means of the option Beam type:



These Member data *overwrite* both the element properties and the default settings in the Concrete Setup.

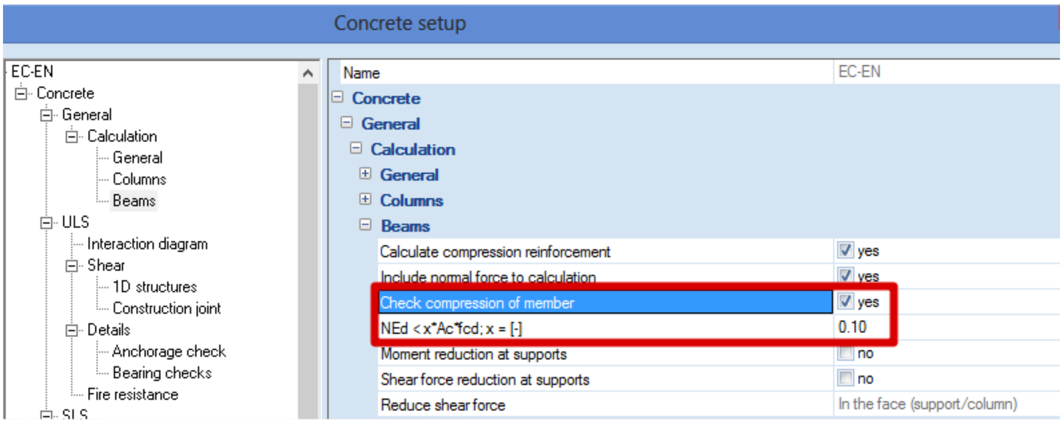
Difference between Beam and Column analysis model

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 Setup an option is available to help the user determine whether the Beam calculation type is valid or not. Go to Setup > Concrete solver > General > Calculation > Tab Beams:



This option ‘Check compression of member’ will check how important the contribution of the axial compression force is:

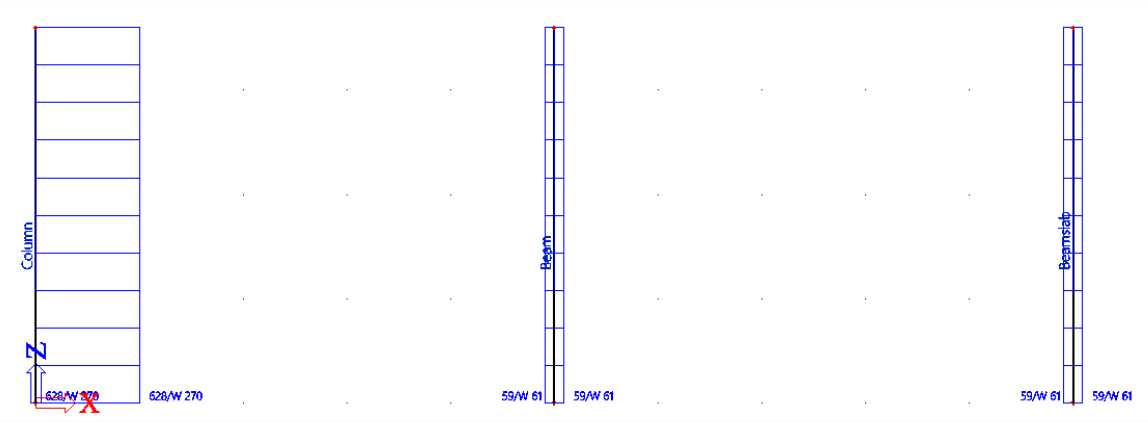
- If the axial compression load $N_{ed} < 0,1*A_c*f_{cd}$, **warning 60** will appear after reinforcement design:

‘The member is not considered to be in compression’, which means the type ‘Beam’ is the right choice.

- If the axial compression load Ned > 0,1*Ac*fcd, **warning 61** will appear after reinforcement design:

‘The member is considered to be in compression’, which means the beam has to be modelled as type ‘Column’.

Example



Design As EN 1992-1-1

Linear calculation, Extreme : Member
Selection : All
Load cases : LC1

Main upper reinforcement for selected beams

Member	d_x [m]	Case	N_d [kN]	M_{yd} [kNm]	x_u [mm]	d [mm]	$A_{s,req}$ [mm²]	Reinf.[no.]	W/E
Beam	0.000	LC1	-100.00	0.00	0	247	59	1x20.0(314)	61
Beamslab	0.000	LC1	-100.00	0.00	0	265	59	10.0-300(59)	61

Main lower reinforcement for selected beams

Member	d_x [m]	Case	N_d [kN]	M_{yd} [kNm]	x_u [mm]	d [mm]	$A_{s,req}$ [mm²]	Reinf.[no.]	W/E
Beam	0.000	LC1	-100.00	0.00	0	247	59	1x20.0(314)	61
Beamslab	0.000	LC1	-100.00	0.00	0	265	59	10.0-300(59)	61

Main reinforcement for selected columns

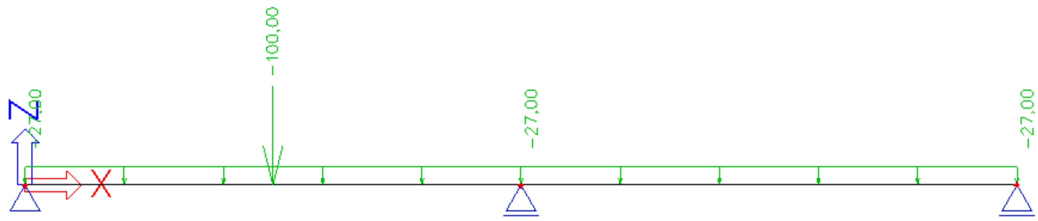
Member	d_x [m]	Case	N_d [kN]	M_{yd} [kNm]	M_{zd} [kNm]	Calc. type	Interaction check [%]	Ratio y/z [%]	$A_{s,req}$ [mm²]	$Reinf_{req}$	$Reinf_{tot}$	W/E
Column	0.000	LC1	-100.00	12.95	12.95	B	47 < 100	50/50	1257	4(4/4)x20.0	4x20.0(1257)	270

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

1_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 moment line (art 9.2.1.3)

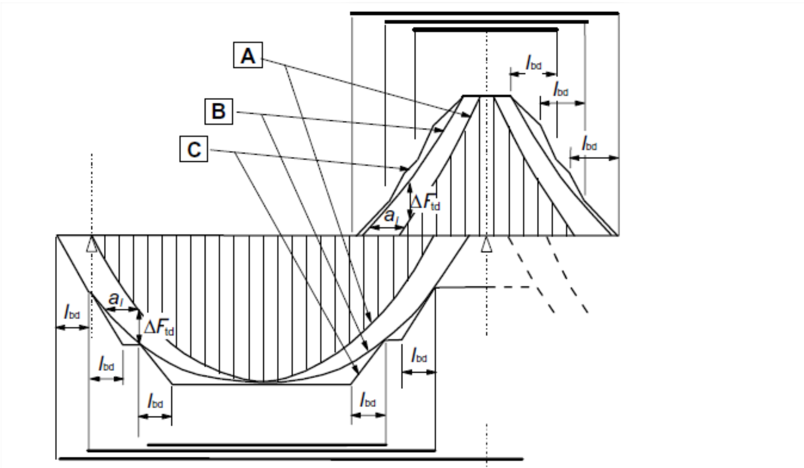
The first transformation of internal forces is the shifting of moment line.

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.

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. This "shift rule" may also be used as an alternative for members with shear reinforcement, where:

$$a_l = z (\cot \theta - \cot \alpha)/2 \qquad (9.2)$$

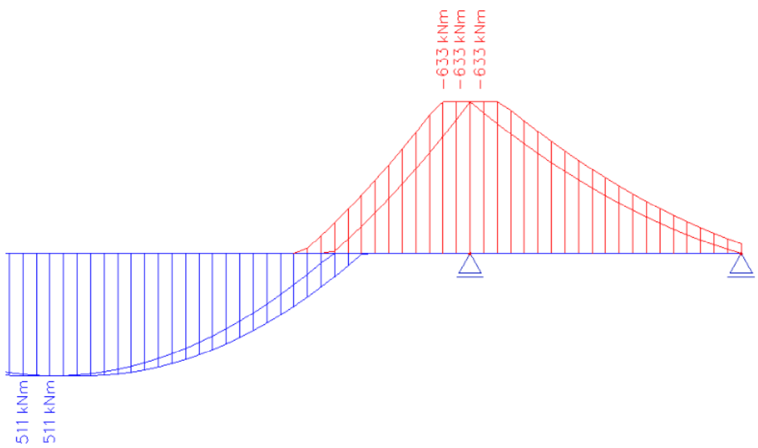
The additional tensile force is illustrated in Figure 9.2:



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 automatically taken into account for recalculated internal forces and by this also for calculation of longitudinal reinforcement. It cannot be found in the setup.

(The picture is taken from another example on which the difference is shown between My and $My_{,recalc}$. Note that the value does not change since no moment capping at supports is taken into account.)

Moment capping at supports (art 5.3.2.2 (4))

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

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:

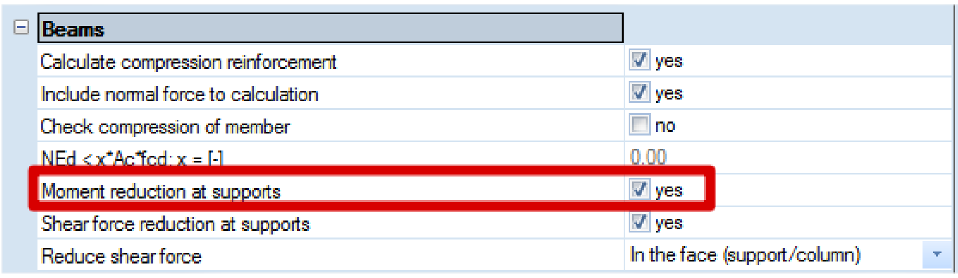
$$\Delta M_{Ed} = F_{Ed,sup}\, t / 8 \qquad (5.9)$$

where:

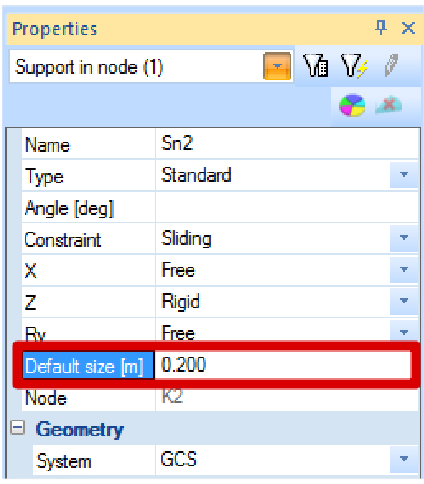
$F_{Ed,sup}$ is the design support reaction

t is the width of the support

In SCIA Engineer this moment capping is only taken into account if it is activated in the concrete setup:



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

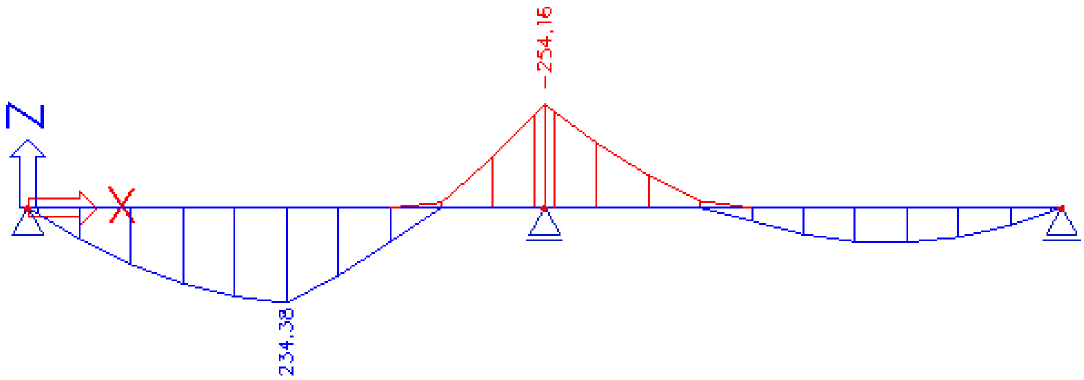


If the support is provided by a column then of course the program will take the width of the column as t.

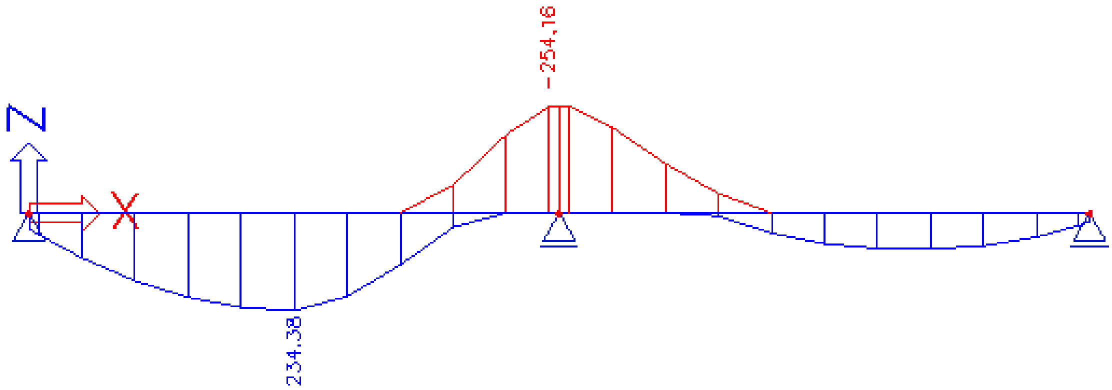
The reduction of moment by moment capping at supports is illustrated for our example below:

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

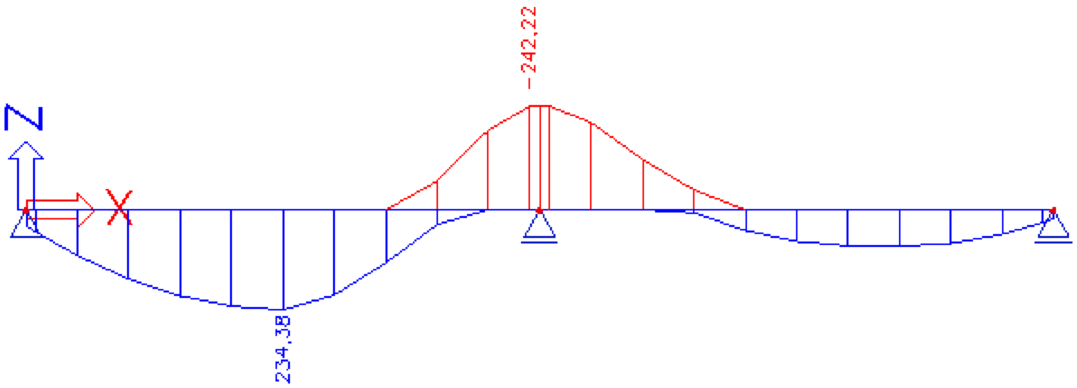
The original moment My at the support was 254.16 kNm.



The recalculated moment clearly shows the shifting of the moment line



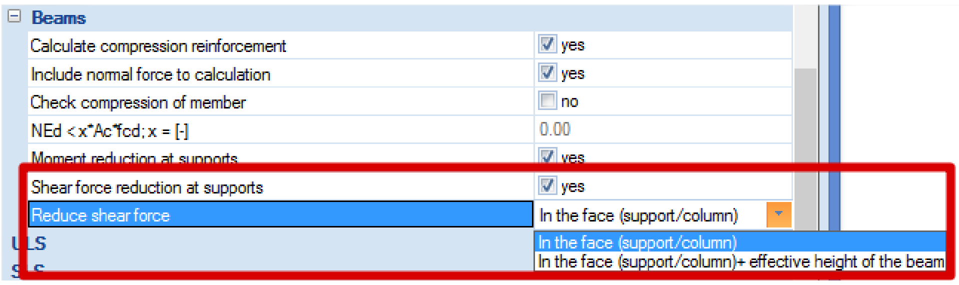
With moment capping at support taken into account the recalculated moment is 242.22 kNm.



Shear force capping at supports (art 6.2.1 (8))

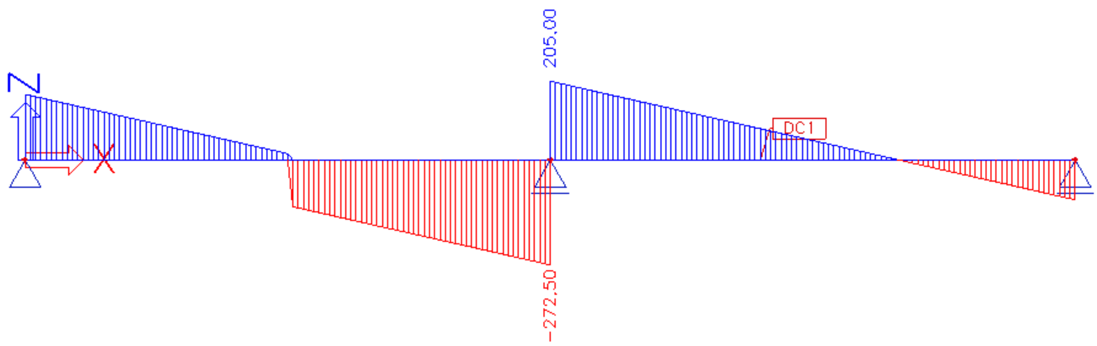
For members subject to predominantly uniformly distributed loading, the design shear force need not to be checked at a distance less than d from the face of the support. Any shear reinforcement required should continue to the support. In addition it should be verified that the shear at the support does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

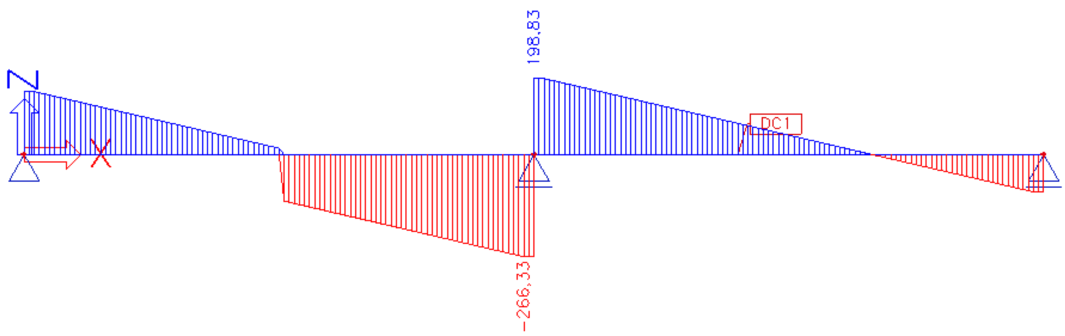
In SCIA Engineer it is possible to choose between at the face of the support or at a distance d from the face of the support:

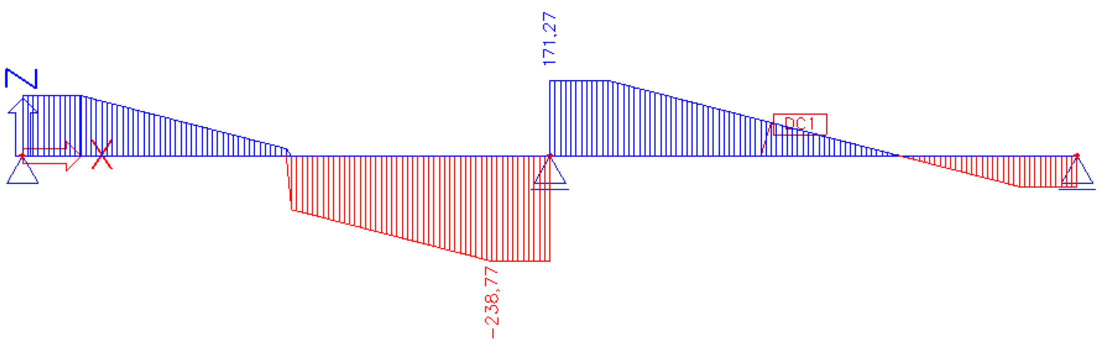


In the figures below it is clear that the shear force is kept constant over the support.

The first figure displays original V_z, the second shows the capping at the face, and the third shows the capping at effective depth from the face:

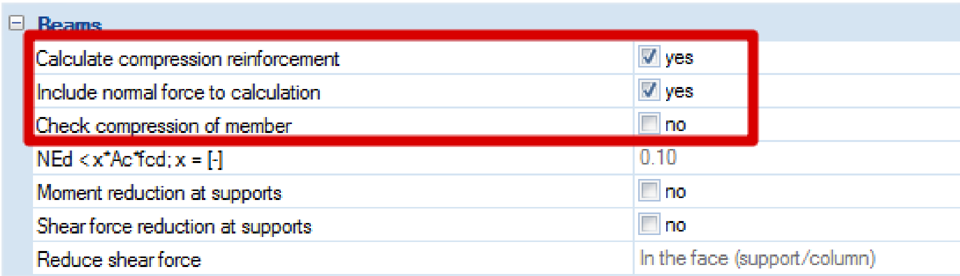






Other calculation settings

For the calculation of theoretical reinforcement some other settings can still be made in the setup:



1) Calculate compression reinforcement : default is **ON**

   By de-activating this option, compression reinforcement will never be calculated and can thus lead to undesignabilites or uneconomical high amounts of tension reinforcement.

2) Include normal force to calculation: default is **ON**

   By de-activating this option, normal force will be neglected (if there is any).

3) Check compression of member:

   If this is on, the value of normal force will be checked.

   This check can be interesting for two reasons.

   - If it is a compression force with an absolute value higher than $0,1 \cdot A_g \cdot f_c'$, then this should be calculated as a column. The beam has to be modelled as type 'column'.
   - If the absolute value normal force is lower than $0,1 \cdot A_g \cdot f_c'$, then the structure can be assumed as primarily subject to bending. In zones primarily subject to bending redistribution of bending moments is allowed. This redistribution is a new feature of SCIA Engineer 2010.

2_Theoretical reinforcement

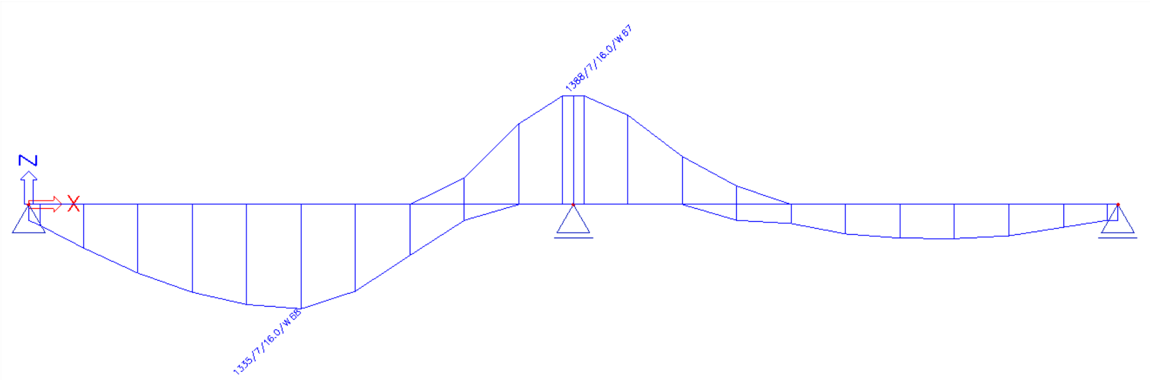
Calculation of longitudinal reinforcement As

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

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

- Material quality is set to B 500A.
- The default diameter is set to 16mm.

The following results are obtained with these settings:



Design As EN 1992-1-1

Linear calculation, Extreme : Member
Selection : All
Combinations : ULS

Main upper reinforcement for selected beams

Member	d_x [m]	Case	N_d [kN]	M_{yd} [kNm]	x_u [mm]	d [mm]	b_{tor} [mm]	$A_{s,b}$ [mm²]	$A_{s,t}$ [mm²]	$A_{s,req}$ [mm²]	Reinf.[no.]	W/E
S1	4.900	ULS/1	0.00	-242.22	136	449	300	1388	0	1388	7x16.0(1407)	67

Main lower reinforcement for selected beams

Member	d_x [m]	Case	N_d [kN]	M_{yd} [kNm]	x_u [mm]	d [mm]	b_{tor} [mm]	$A_{s,t}$ [mm²]	$A_{s,req}$ [mm²]	Reinf.[no.]	W/E
S1	2.500	ULS/1	0.00	234.38	131	449	300	0	1335	7x16.0(1407)	68

Detailing reinforcement for selected member

Member	d_x [m]	Case	T_{Ed} [kNm]	A_k [mm²]	u_k [mm]	$A_{s,tors}$ [mm²]	$A_{s,det}$ [mm²]	Reinf.[no.]	W/E
S1	2.500	ULS/1	0.00	78804	1192	0	0	2x8.0(101)	78
S1	4.900	ULS/1	0.00	78804	1192	0	0	2x8.0(101)	78

Explanation of warnings and errors

67	The bar distance for the upper reinforcement is too small.
68	The bar distance for the lower reinforcement is too small.
78	The reinforcement was designed according to detailing provisions.

d: lever arm of reinforcement.

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

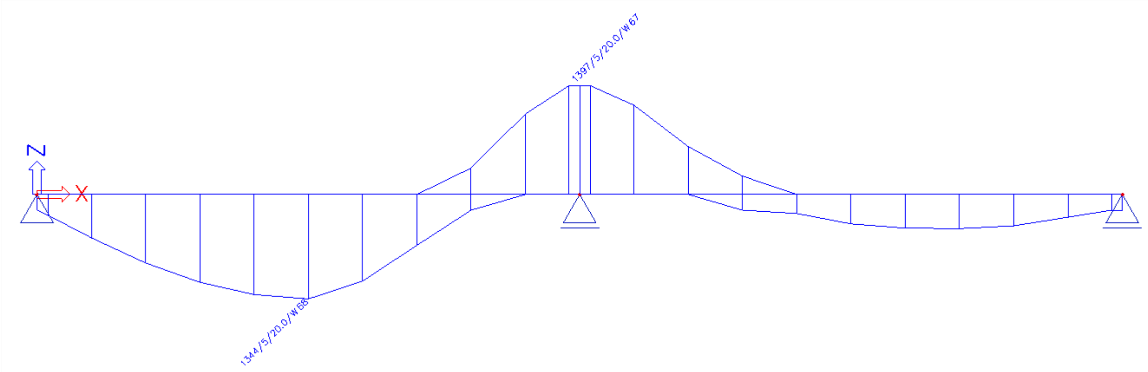
(the cover is defined by the environmental class and is 35 mm for XC3)

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

For detailing reinforcement (the reinforcement in the side of the beam), the theoretically required is zero because there is no torsion on this beam (the 2x8 refers to the stirrups).

The warnings 67 and 68 are typical warnings that appear when the chosen diameter is too small. It means that with this diameter too many bars are needed resulting in a bar distance which is too small. This gives an indication that it is better to choose a bigger default diameter.

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



Design As EN 1992-1-1

Linear calculation, Extreme : Member
Selection : All
Combinations : ULS

Main upper reinforcement for selected beams

Member	d_x [m]	Case	N_d [kN]	M_{yd} [kNm]	x_u [mm]	d [mm]	b_{tor} [mm]	$A_{s,b}$ [mm²]	$A_{s,t}$ [mm²]	$A_{s,req}$ [mm²]	Reinf.[no.]	W/E
S1	4.900	ULS/1	0.00	-242.22	137	447	300	1397	0	1397	5x20.0(1571)	67

Main lower reinforcement for selected beams

Member	d_x [m]	Case	N_d [kN]	M_{yd} [kNm]	x_u [mm]	d [mm]	b_{tor} [mm]	$A_{s,t}$ [mm²]	$A_{s,req}$ [mm²]	Reinf.[no.]	W/E
S1	2.500	ULS/1	0.00	234.38	132	447	300	0	1344	5x20.0(1571)	68

Detailing reinforcement for selected member

Member	d_x [m]	Case	T_{Ed} [kNm]	A_k [mm²]	u_k [mm]	$A_{s,tors}$ [mm²]	$A_{s,det}$ [mm²]	Reinf.[no.]	W/E
S1	2.500	ULS/1	0.00	76436	1176	0	0	2x8.0(101)	78
S1	4.900	ULS/1	0.00	76436	1176	0	0	2x8.0(101)	78

Explanation of warnings and errors

67	The bar distance for the upper reinforcement is too small.
68	The bar distance for the lower reinforcement is too small.
78	The reinforcement was designed according to detailing provisions.

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:

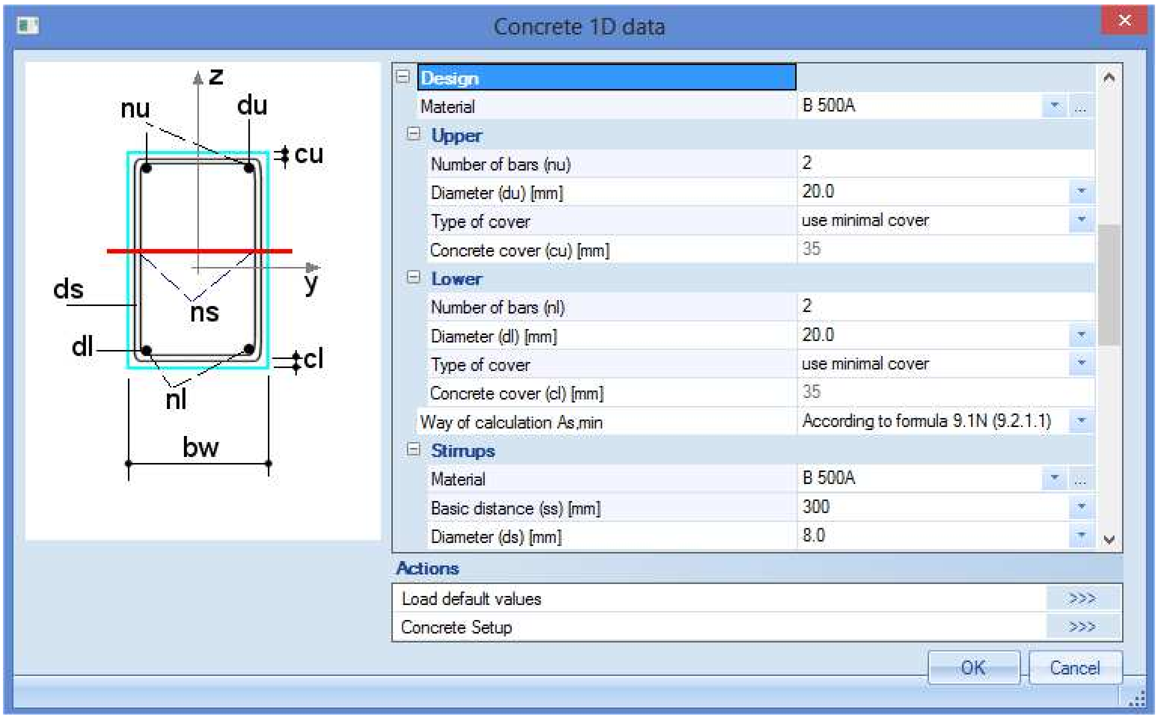
$$d = h - cover - \Phi_{stirrup} - \Phi_{longitudinal\ beam}/2 = 500 - 35 - 8 - \mathbf{20}/2 = 447 \text{ mm}$$

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.

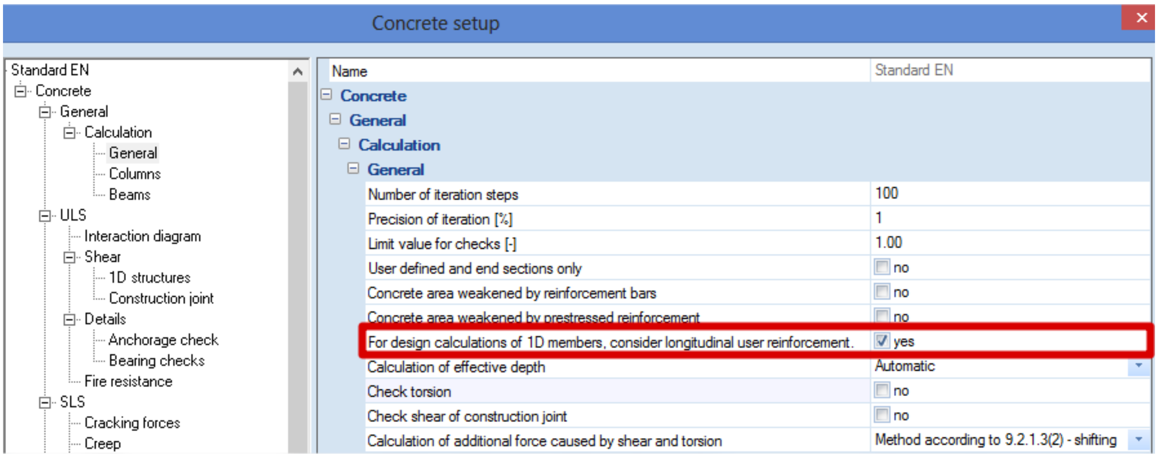
We will now input a basic reinforcement and calculate then the additional reinforcement which is needed.

As basic reinforcement we will use 2 bars of 20mm on the upper and lower side of the beam.
Basic reinforcement can be inserted in the beam by use of member data.

(Note that concrete data can also be used without setting a number of bars.
In that case concrete data are used to change the default diameter for the bar to which these data are assigned. It is obvious that the member data have higher priority than the general setup.)

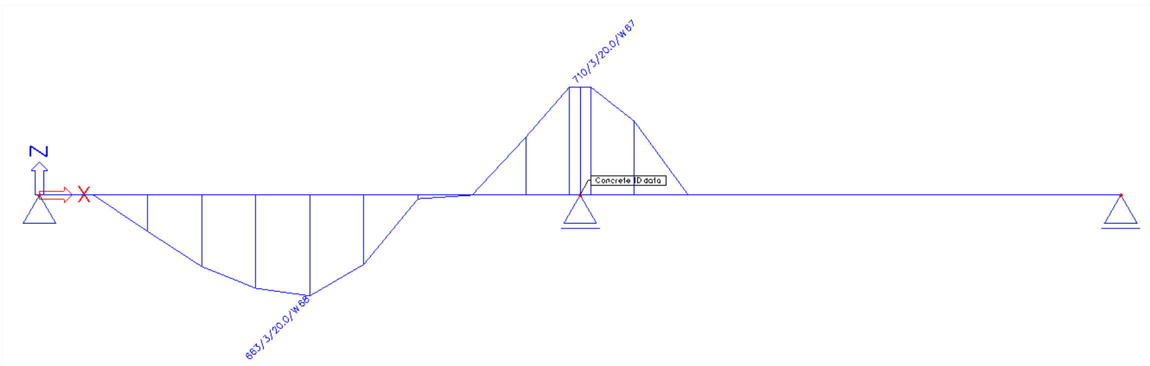


To take the user reinforcement into account, the following setting has to be on (default it is on):



The additional required reinforcement equals the total reinforcement minus the basic user reinforcement.

The result is shown below:



Design As EN 1992-1-1

Linear calculation, Extreme : Member
Selection : All
Combinations : ULS

Main upper reinforcement for selected beams

Member	d_x [m]	Case	N_d [kN]	M_{yd} [kNm]	x_u [mm]	d [mm]	$A_{s,b}$ [mm²]	$A_{s,t}$ [mm²]	$A_{s,req}$ [mm²]	$A_{s,user}$ [mm²]	Reinf.[no.]	W/E
S1	4.900	ULS/1	0.00	-242.22	91	447	710	0	710	628	3x20.0+2x20.0(1571)	67

Main lower reinforcement for selected beams

Member	d_x [m]	Case	N_d [kN]	M_{yd} [kNm]	x_u [mm]	d [mm]	$A_{s,b}$ [mm²]	$A_{s,t}$ [mm²]	$A_{s,req}$ [mm²]	$A_{s,user}$ [mm²]	Reinf.[no.]	W/E
S1	2.500	ULS/1	0.00	234.38	89	447	663	0	663	628	3x20.0+2x20.0(1571)	68

Detailing reinforcement for selected member

Member	d_x [m]	Case	T_{Ed} [kNm]	A_k [mm²]	u_k [mm]	$A_{s,tors}$ [mm²]	$A_{s,det}$ [mm²]	Reinf.[no.]	W/E
S1	2.500	ULS/1	0.00	76436	1176	0	0	(0)	221
S1	4.900	ULS/1	0.00	76436	1176	0	0	(0)	221

Explanation of warnings and errors

67	The bar distance for the upper reinforcement is too small.
68	The bar distance for the lower reinforcement is too small.
221	It is not possible to design of torsion reinforcement including user or practical reinforcement.

As you can see graphically, we will only need additional reinforcement at the upper face over the support and at the lower face in the field of the beam.

We can input this additional reinforcement by means of practical reinforcement. The manual input of practical reinforcement will be described in chapter 3.

Calculation of shear reinforcement Ass

Design As EN 1992-1-1

Linear calculation, Extreme : Member
Selection : All
Combinations : ULS

Shear reinforcement for selected members

Member	d_x [m]	Case	V_{Ed} [kN]	b_w [mm]	d [mm]	$V_{Rd,c}$ [kN]	$V_{Rd,max}$ [kN]	A_{ss} [mm²/m]	Reinf.[no.]
S1	4.900	ULS/1	-266.33	300	447	83.40	627.56	1278	2x8.0-79

$\mathbf{V_{Ed}}$ = design shear force resulting from external loading
$\mathbf{V_{Rd,c}}$ = design shear resistance of the member without shear reinforcement
$\mathbf{V_{Rd,s}}$ = design value of the shear force which can be sustained by the yielding shear reinforcement
$\mathbf{V_{Rd,max}}$ = 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:

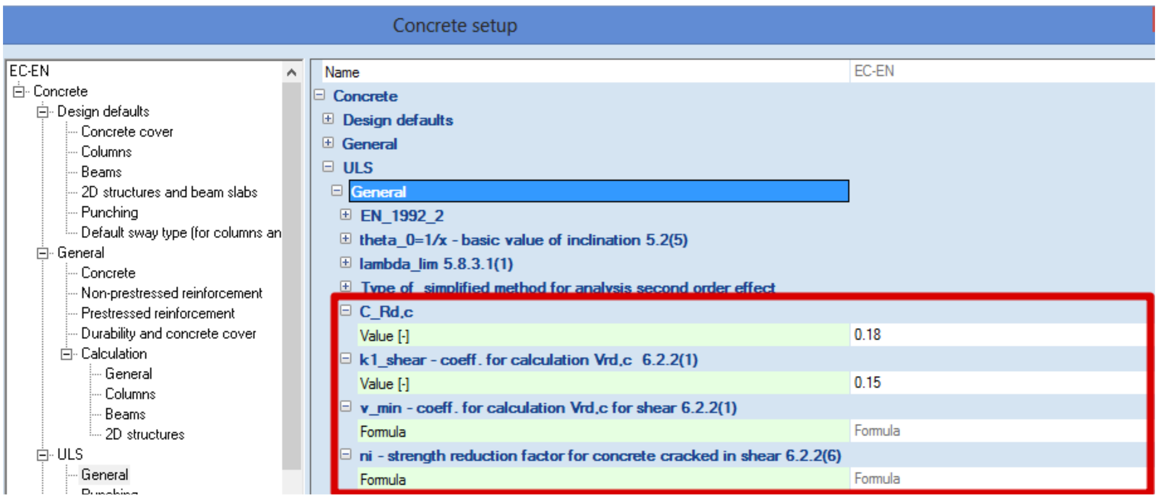
$$V_{Ed} \leq 0{,}5\, b_w\, d\, v\, f_{cd} \qquad (6.5)$$

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 a truss model:

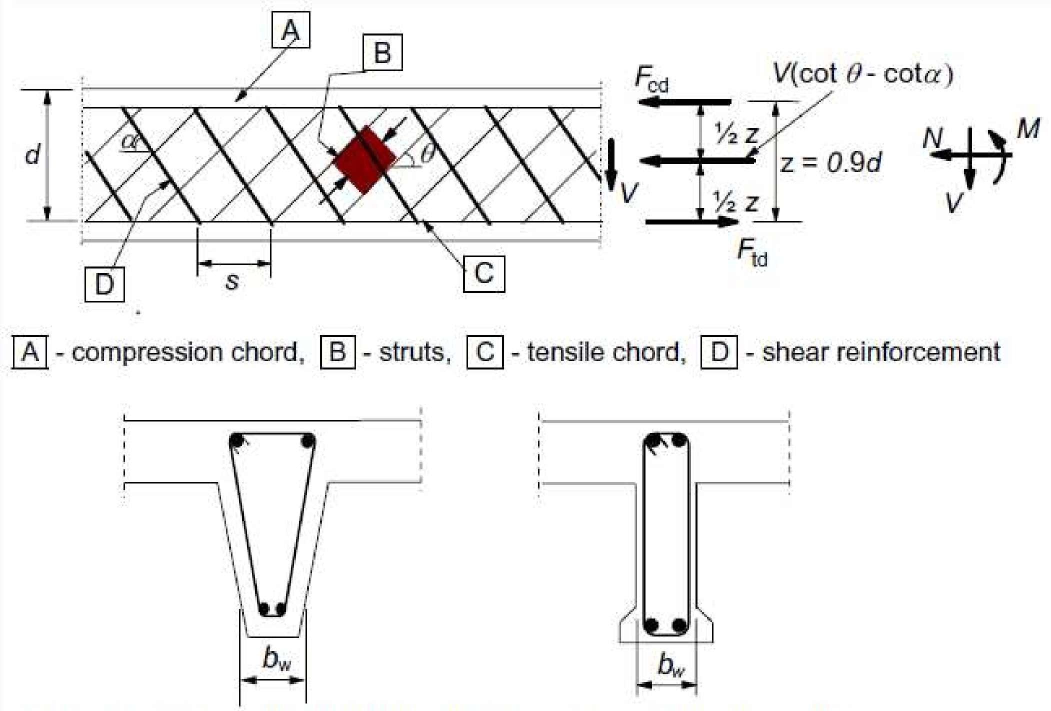


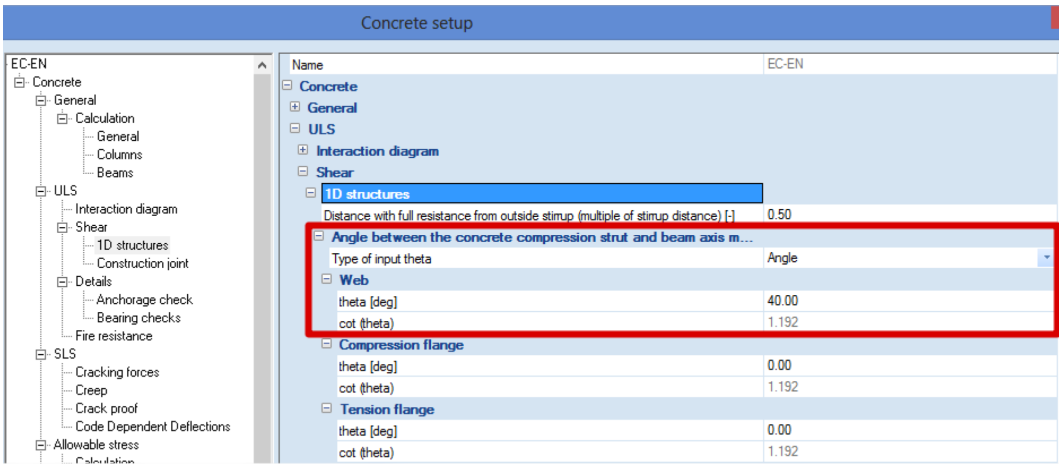
Figure 6.5: Truss model and notation for shear reinforced members

The angle θ should be limited.

The recommended limits of cot θ are given:

$1 \leq \cot\theta \leq 2,5$ (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

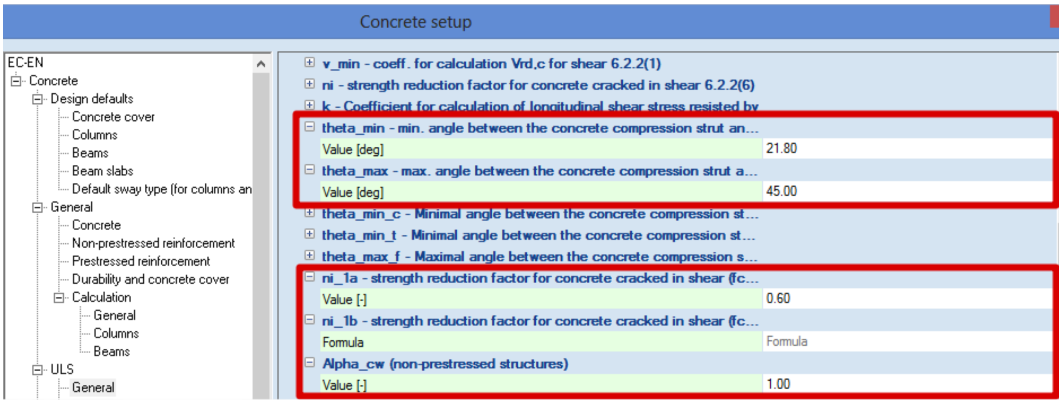
The recommended value of v_1 is v (see Expression 6.6N)

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

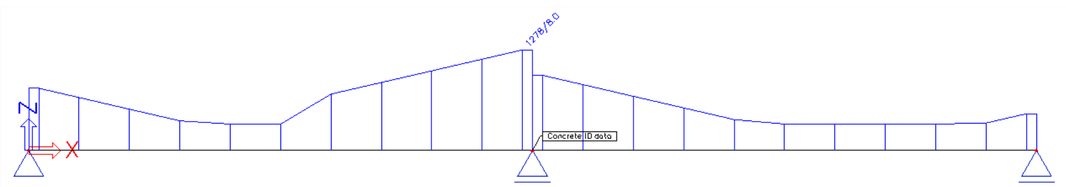
$v_1 = 0{,}6$ for $f_{ck} \leq 60$ MPa (6.10.aN)
$v_1 = 0{,}9 - f_{ck}/200 > 0{,}5$ for $f_{ck} \geq 60$ MPa (6.10.bN)

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

These code related parameters can be found in the setup:



If we come back to our example in SCIA Engineer, we find the following A_{ss} for the whole beam:



Design As EN 1992-1-1

Linear calculation, Extreme : Member
Selection : All
Combinations : ULS

Shear reinforcement for selected members

Member	d_x [m]	Case	V_{Ed} [kN]	b_w [mm]	d [mm]	$V_{Rd,c}$ [kN]	$V_{Rd,max}$ [kN]	A_{ss} [mm²/m]	Reinf.[no.]
S1	4.900	ULS/1	-266.33	300	447	83.40	627.56	1278	2x8.0-79

In the middle of the span you can see that there is everywhere a minimum of 335mm^2. This is because we defined earlier in the concrete data that there is a two section stirrup of 8mm every 300mm. The maximum value of 1278mm^2 corresponds to a two section stirrup of 8mm every 79mm.

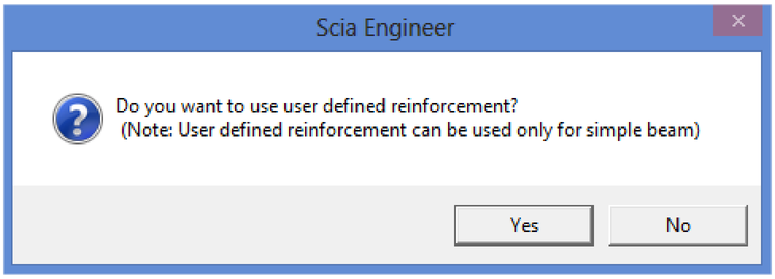
3_Practical reinforcement

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

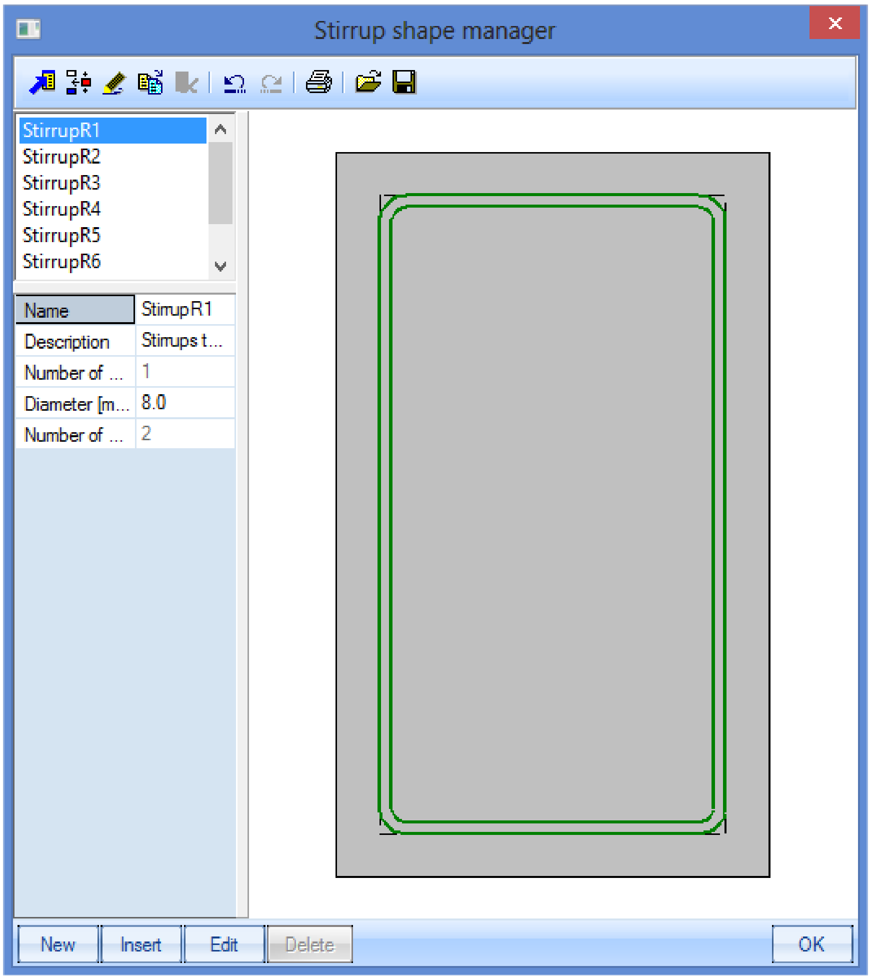
In the theoretical reinforcement design, we have calculated where we needed additional reinforcement. This will allow us to manually input the practical reinforcement.

In chapter 5 we will see that also automatic input of practical reinforcement is possible.

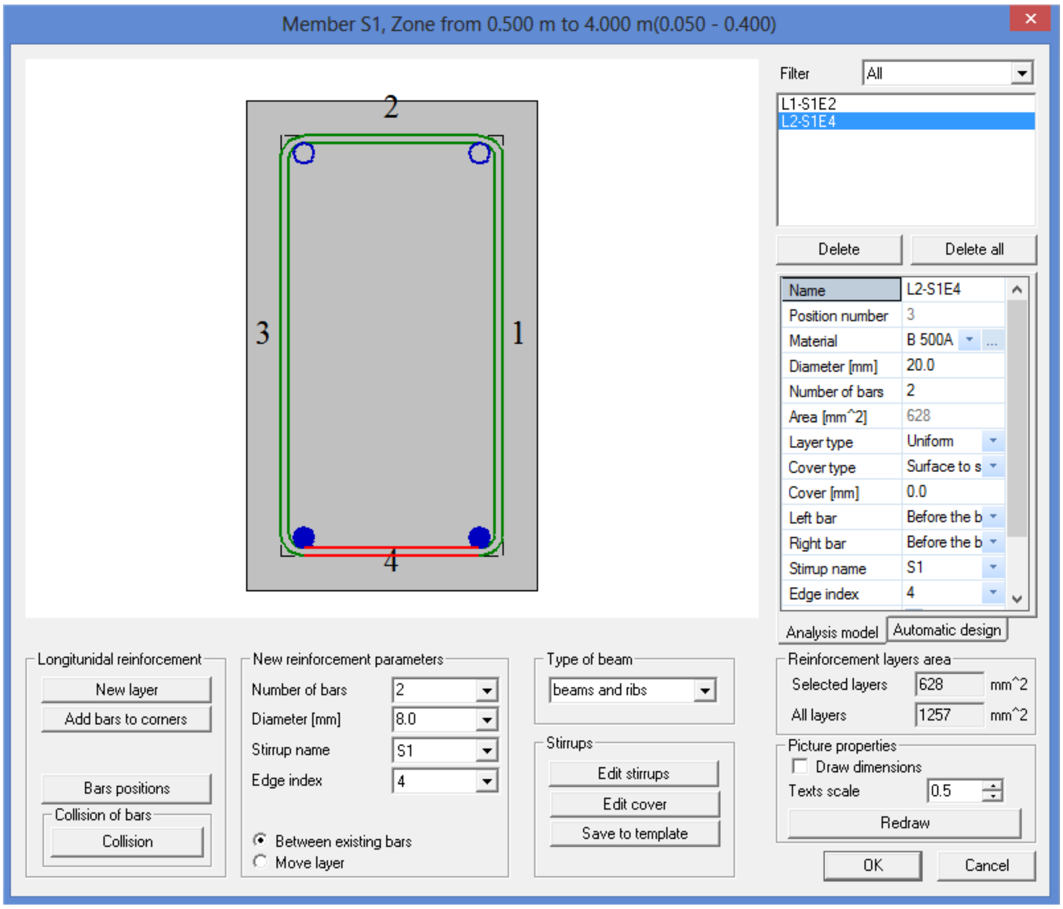
We will start by converting the user defined reinforcement into practical reinforcement:



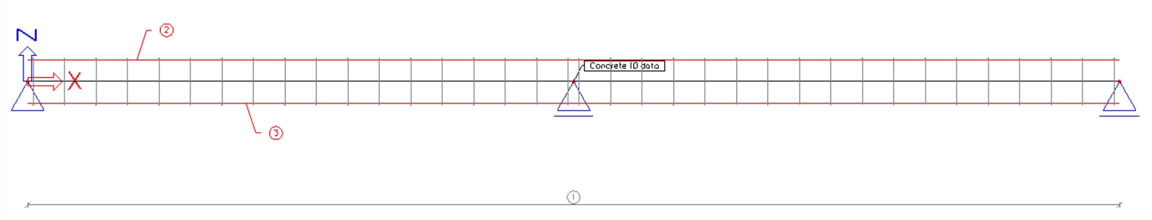
We can first select a stirrup shape:



Then the configuration of longitudinal reinforcement is shown:

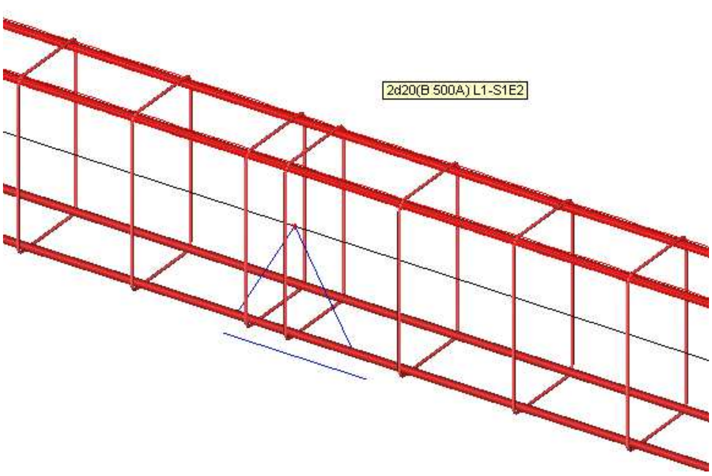


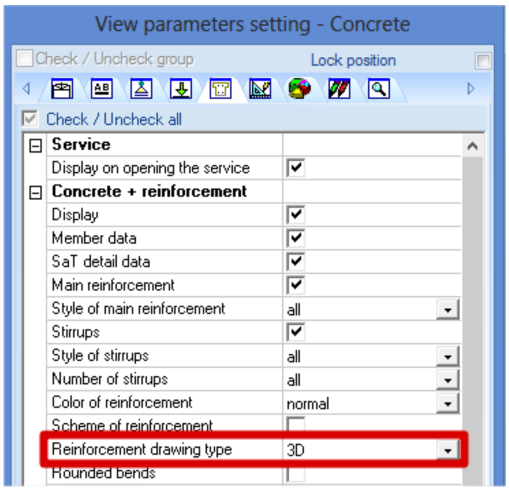
The practical reinforcement is shown graphically on the screen:



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

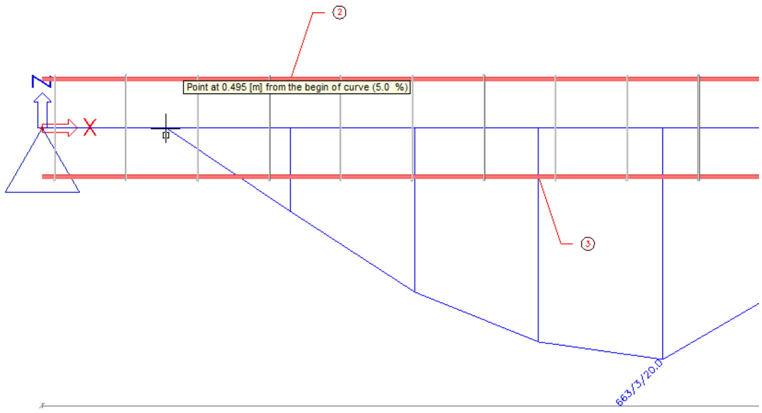
Through view parameter settings a 3D representation of the reinforcement can be obtained:

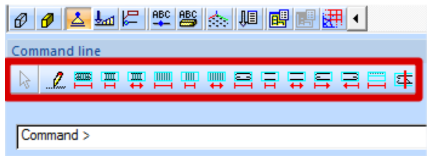




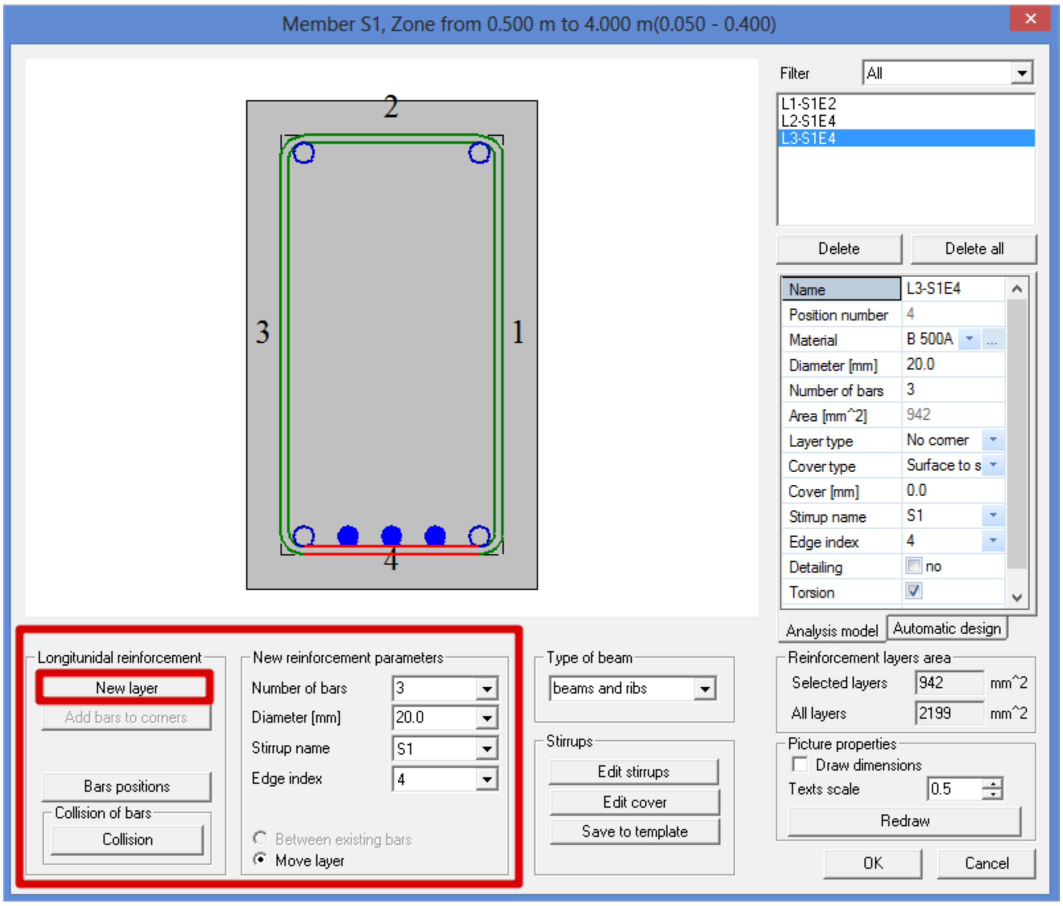
To input locally extra practical reinforcement, we take back the results for additional reinforcement.

On this picture we can precisely define where the extra practical reinforcement needs to be putted:





To add a new layer of longitudinal reinforcement the following screen will pop up. Here can be set on which face extra reinforcement needs to be added:



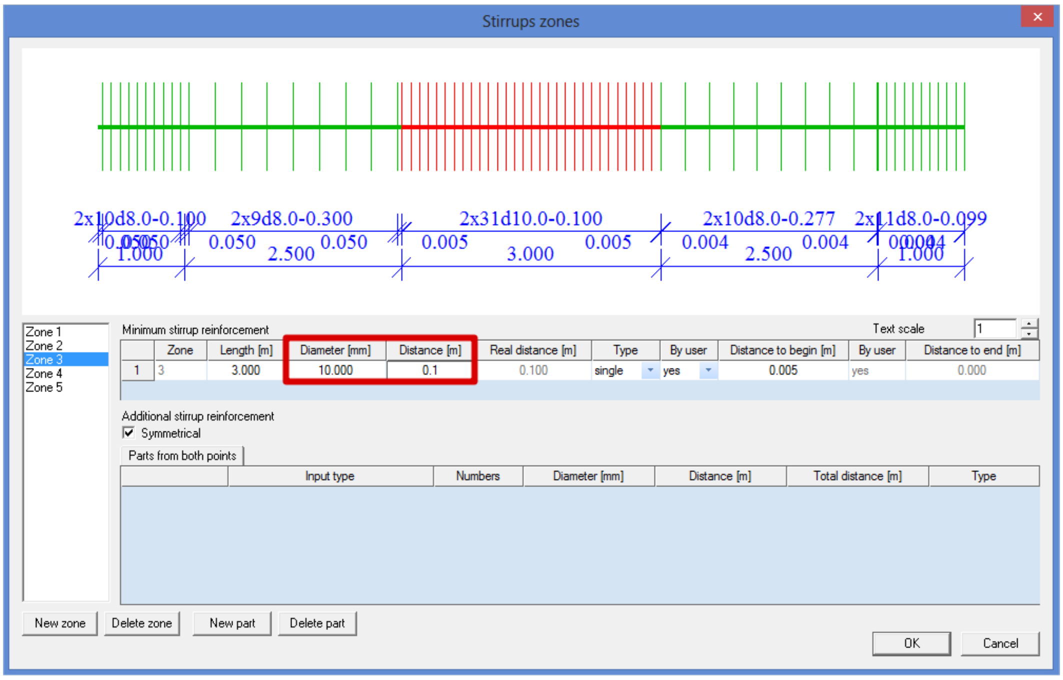
For reasons of simplicity we will add 3 bars of 20mm that are still needed over the whole area where extra reinforcement is required. This can of course be done more detailed.

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

After adding the extra longitudinal reinforcement, we can verify if this is enough through verifying if still additional reinforcement is required or not.

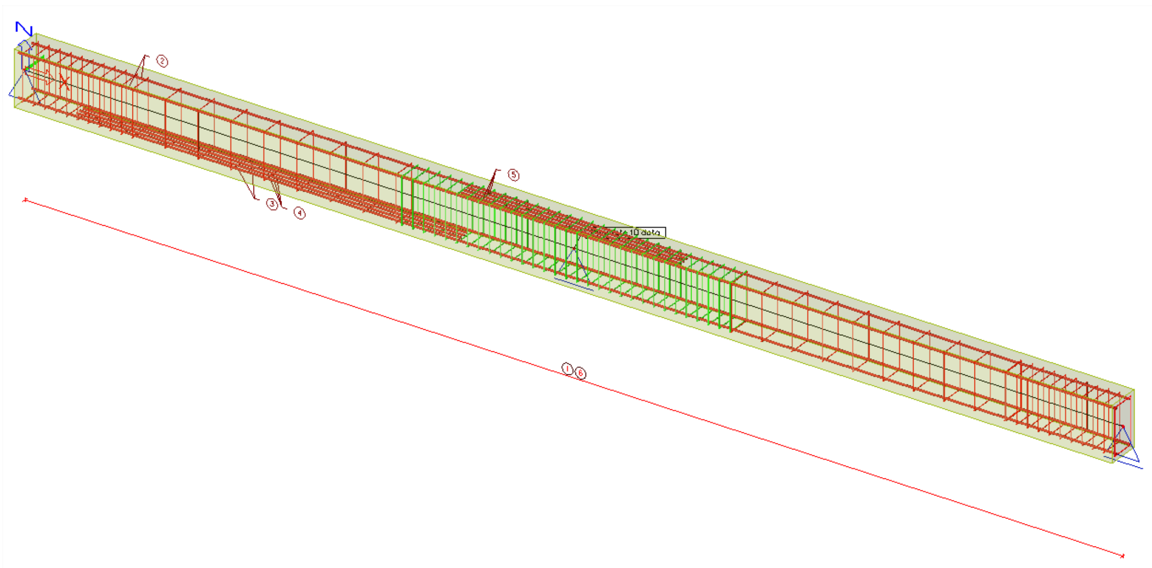
Also the shear reinforcement needs to be increased in the zones over the support. This can be done by increasing the diameter of the stirrups or by decreasing the distance between the stirrups.

Different stirrup zones can be created:

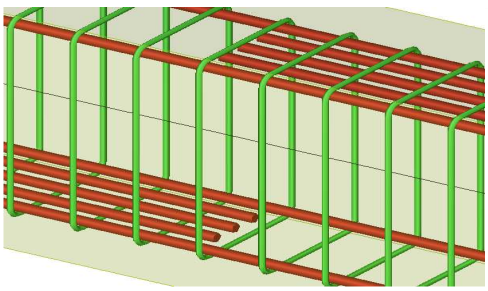


To check if there is enough shear reinforcement, a capacity check needs to be performed. This will be explained in the next chapter.

The total practical reinforcement of the beam is shown below:



A zoomed view shows the 3D representation:



4_Checks

There are some typical checks that can be performed on a beam with practical reinforcement.

The following checks are implemented in SCIA Engineer:

- **capacity check**: checks the interaction of internal forces
- **response check**: checks the maximal strains based on the stress/strain diagram
- **detailing provisions**: checks if detailing provisions described in EN chapter 8 and 9 are met
- **crack control**: checks if maximum crack width is not exceed, according to EN chapter 7.3
- **deflections check**: checks if the long term deflections don't exceed max values (described in chapter CDD)

The capacity and response 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 check and the crack control check 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.

We will explain the checks one by one on our example.

We can start from the esafile: 'beam_practical reinforcement.esa'

Capacity check

We will start by controlling if there is enough shear force reinforcement by asking the report for Vzu:

Check capacity EN 1992-1-1

Linear calculation, Extreme : Member
Selection : All
Combinations : ULS

Check of shear for selected members

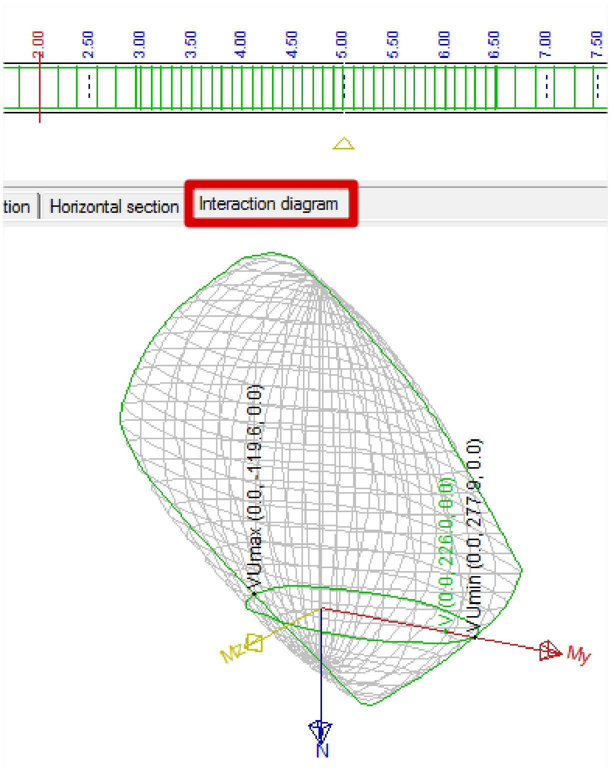
Member	d_x [m]	Case	Method	V_{Ed} [kN]	stirr dist [mm] transv dist [mm]	diam. [mm]	A_{ss} [mm^2/m]	$V_{Rd,c}$ [kN] $V_{Rd,max}$ [kN]	$V_{Rd,s}$ [kN] V_{Rd} [kN]	$Check_{calc}$ [-] $Check_{lim}$ [-]	Check W/E
S1	3.000	ULS/1	formula 6.2a.b) EN1992-1-1	-149.16	300	8.0	335	87.78	69.54	2.14	NOT OK
					218			624.75	**69.54**	1.00	678
S1	7.000	ULS/1	formula 6.2a.b) EN1992-1-1	81.66	277	8.0	363	64.68	75.35	1.08	NOT OK
					218			624.75	**75.35**	1.00	678

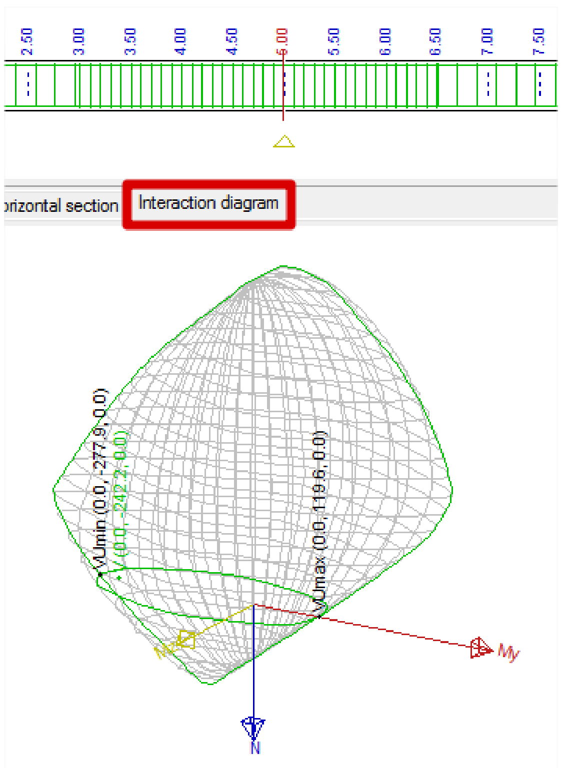
As expected the unity check is higher than one, so this is not okay.

We will increase shear force reinforcement also in the middle zones to solve this problem.

This is done in the new file ‘beam_practical reinforcement2.esa’.

It is also possible to do a single check in a specific section of the beam.
This gives very detailed information on the interaction diagram as shown below:

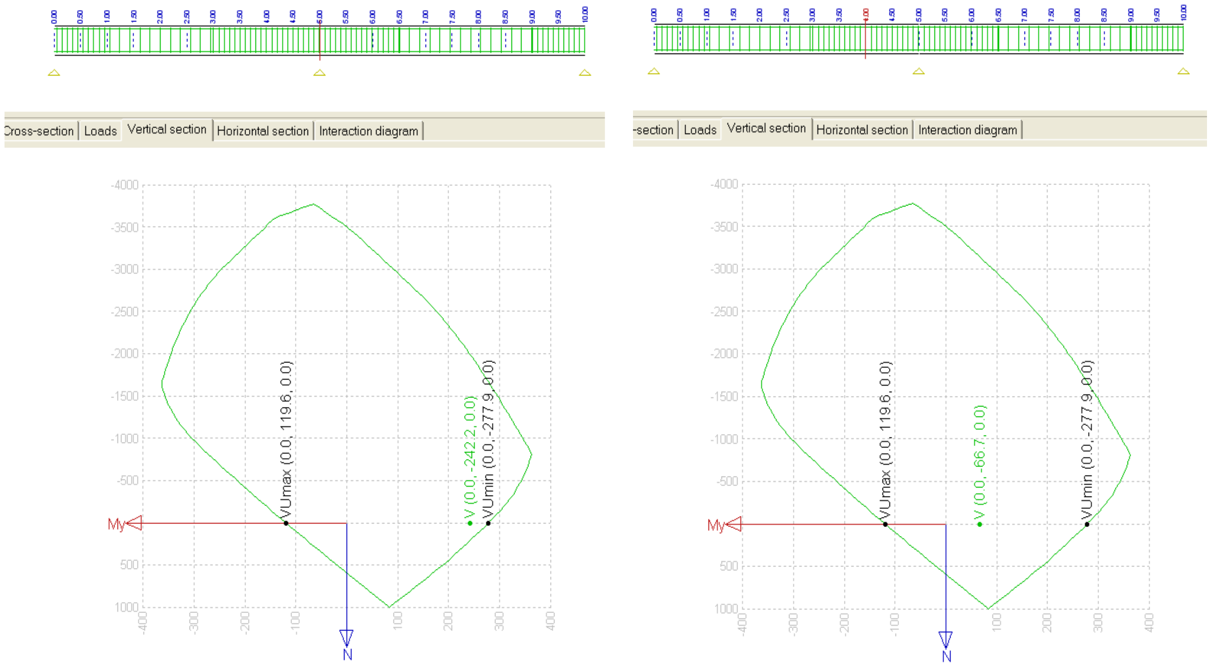




Note the positive moment My in the middle of the span and the negative moment My over the support. (make sure you set the extreme to My+ and My- respectively if you look at these interaction diagrams)

Since Mz and N are zero in this case we could better look at the vertical section of the interaction diagram.

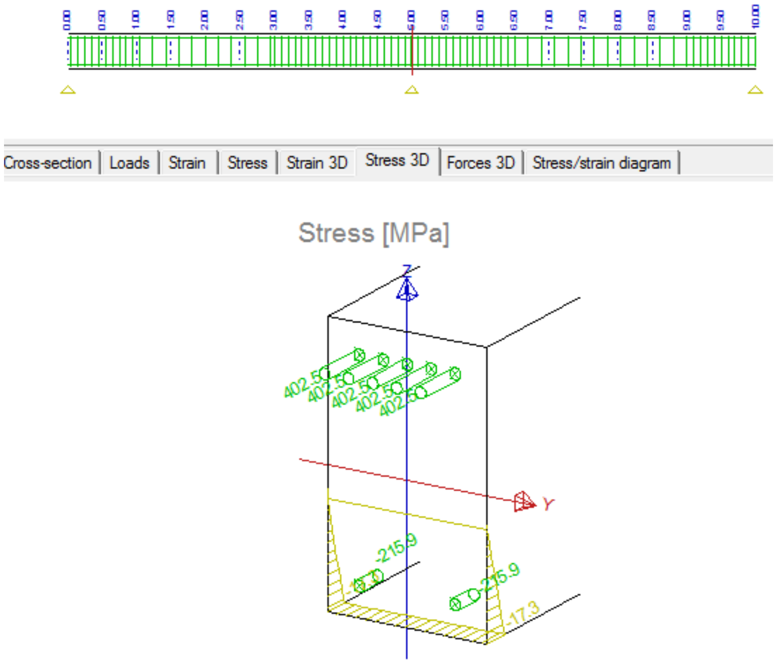
If we compare the results at x = 5m (over support) and at x = 4m (1m next to support), we can see that at x = 4m we could have succeeded easily with less reinforcement:



Response check

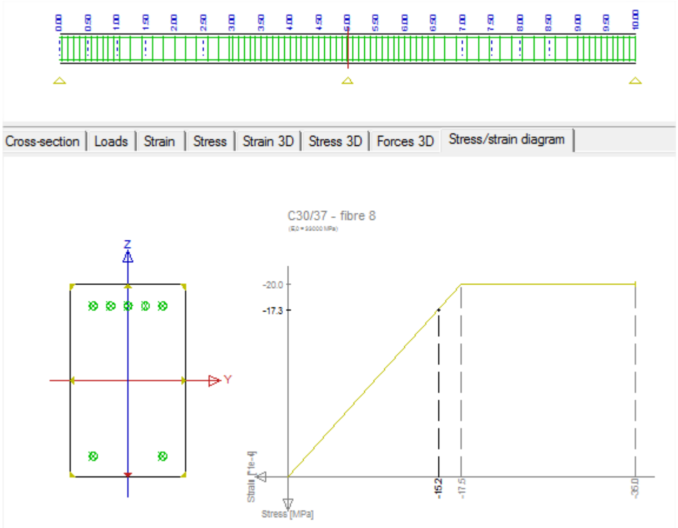
The response check is related to the stress strain diagram and checks if maximal strains are not exceeded. As mentioned above, this check is satisfied if no additional reinforcement was required.

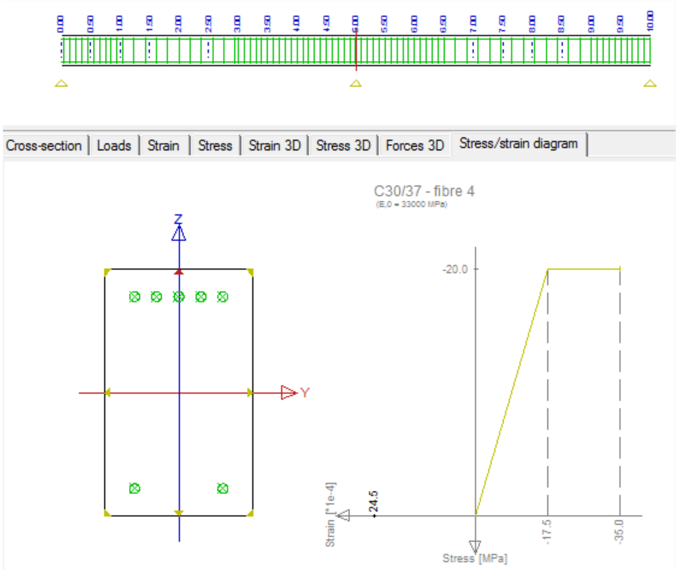
The single check over the support is shown:

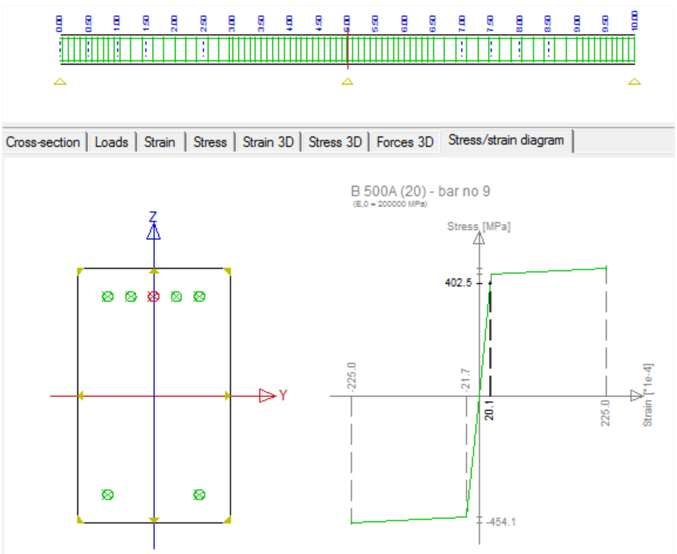


It is clear from this picture that the upper side is in tension and that the concrete does not work in tension. All tension has to be taken by the upper reinforcement bars.

The stress/strain diagrams are shown below for the lower and upper fibre of concrete and for the upper reinforcement bar:



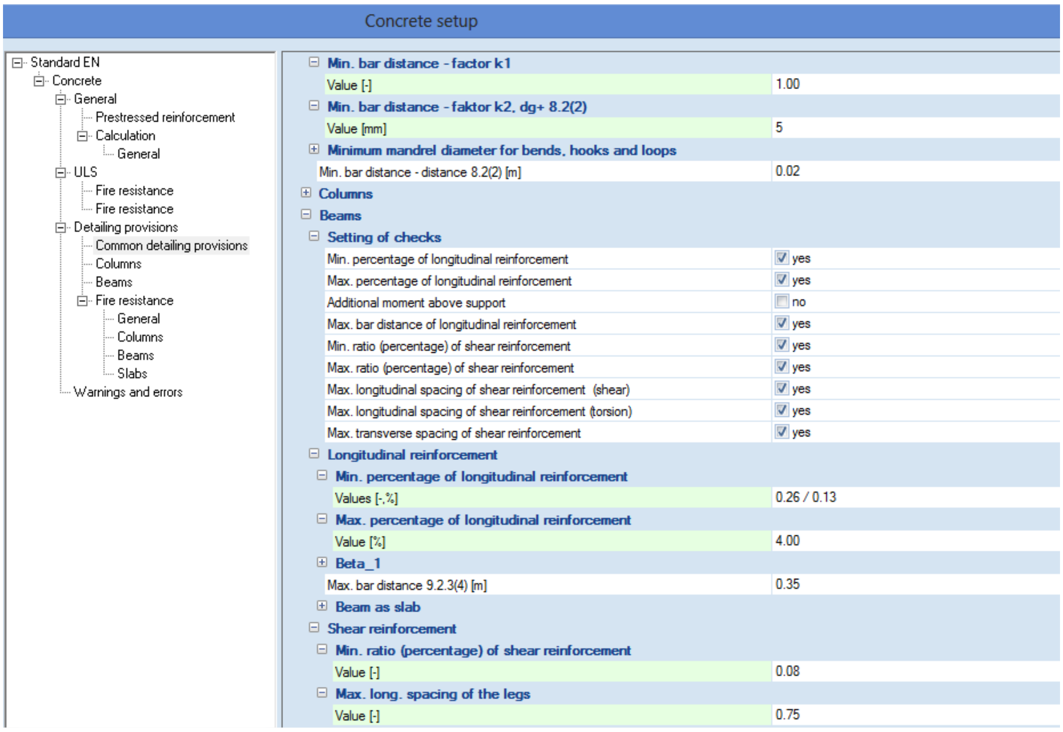




Detailing provisions check

The check of detailing provisions checks the detailing provisions activated in the concrete setup.

Since the EN requires checking these detailing provisions, it is advised to activate them all (note that default they are all activated):



The check of detailing provisions of longitudinal reinforcement is shown below:

Detailing provisions EN 1992-1-1

Linear calculation, Extreme : Member
Selection : All
Combinations : ULS

Detailing provisions of longitudinal reinforcement for selected members

Member	d_x [m]	Case	$\mu_{lc,min}/\mu_{l,min}$ [-]	$\mu_{l,max}/\mu_{lc,max}$ [-]	$s_{lc,min}/s_{l,min}$ [-]	$s_{l,max}/s_{lc,max}$ [-]	$Check_{calc}$ [-]	$Check_{lim}$ [-]	Check	W/E
S1	0.497	ULS/1	0.13	0.37	1.48	0.51	1.48	1.00	NOT OK	868

Explanation of warnings and errors

868	Minimum surface to surface spacing of longitudinal reinforcement is not satisfactory.

The problem in this example is that the surface to surface spacing of longitudinal reinforcement is less than the minimum.

Spacing of bars (art 8.2)

The clear distance (horizontal and vertical) between individual parallel bars or horizontal layers of parallel bars should be not less than the maximum of k_1 . bar diameter, (d_g + k_2 mm) or 20 mm where d_g is the maximum size of aggregate (d_g can be set in the material properties).

The recommended values of k_1 and k_2 are respectively 1 and 5 mm.

A possible solution would be to replace the 5 bars of 20 mm by a smaller amount of bigger bars.

Crack control check

The unity check for crack control in SCIA Engineer is the biggest of three checks:

- crack width ($w > w_{lim}$)
- minimum reinforcement area ($As_{prov} > As_{min}$)
- maximum bar diameter and distances (it is sufficient that either the distances or the diameters satisfy)

Calculation of crack widths (art 7.3.4)

The crack width, w_k, may be calculated from:

$$w_k = s_{r,max}(\varepsilon_{sm} - \varepsilon_{cm}) \qquad (7.8)$$

where

$s_{r,max}$ = maximum crack spacing
ε_{sm} = mean strain in the reinforcement under the relevant combination of loads
ε_{cm} = mean strain in the concrete between cracks

Limiting calculated crack (art 7.3.1(5))

A limiting calculated crack width, w_{max}, taking into account the proposed function and nature of the structure and the costs of limiting cracking, should be established.

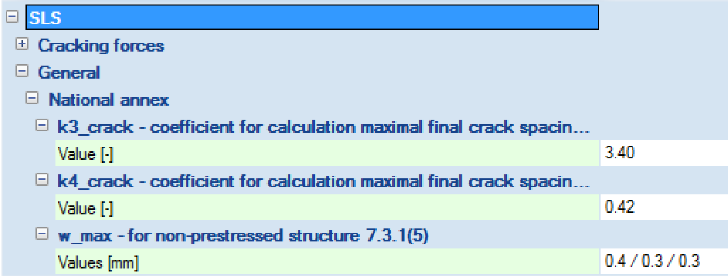
The recommended values of w_{max} for relevant exposure classes are given in following table:

Table 7.1N Recommended values of w_{max} (mm)

<table>
<tr><th>Exposure Class</th><th>Reinforced members and prestressed members with unbonded tendons</th><th>Prestressed members with bonded tendons</th></tr>
<tr><td></td><td>Quasi-permanent load combination</td><td>Frequent load combination</td></tr>
<tr><td>X0, XC1</td><td>0,4[1]</td><td>0,2</td></tr>
<tr><td>XC2, XC3, XC4</td><td rowspan="2">0,3</td><td>0,2[2]</td></tr>
<tr><td>XD1, XD2, XS1, XS2, XS3</td><td>Decompression</td></tr>
</table>

Note 1: For X0, XC1 exposure classes, crack width has no influence on durability and this limit is set to guarantee acceptable appearance. In the absence of appearance conditions this limit may be relaxed.
Note 2: For these exposure classes, in addition, decompression should be checked under the quasi-permanent combination of loads.

The allowable crack width can be set in the setup:



For environmental class XC3, w_{lim} equals 0.3 mm:

Crack proof EN 1992-1-1

Linear calculation, Extreme : Global
Selection : All
Combinations : SLS

Crack width calculation for selected members

Member	d_x [m]	Case	N [kN] N_r [kN]	M_y [kNm] M_{yr} [kNm]	σ_s [MPa] $f_{ct,eff}$ [MPa]	$s_{r,max}$ [mm] $\varepsilon_{sm}-\varepsilon_{cm}$ [1e-4]	w [mm] w_{lim} [mm]	$Check_{calc}$ [-] $Check_{lim}$ [-]	Check
S1	8.500	SLS/1	0.00	63.43	244.7	375	0.275	0.92	OK
			0.00	39.61	2.90	7.3	0.300	1.00	
S1	5.000	SLS/1	0.00	-188.27	300.0	232	0.289	0.96	OK
			0.00	-43.22	2.90	12.4	0.300	1.00	

Minimum reinforcement area (art 7.3.2)

The required minimum areas of reinforcement may be calculated as follows:

$$A_{s,min}\,\sigma_s = k_c\,k\,f_{ct,eff}\,A_{ct} \tag{7.1}$$

where:

$A_{s,min}$ = minimum area of reinforcing steel within the tensile zone
A_{ct} = area of concrete within tensile zone
σ_s = absolute value of the maximum stress permitted in the reinforcement immediately after formation of the crack
$f_{ct,eff}$ = mean value of the tensile strength of the concrete effective at the time when the cracks may first be expected to occur
k = coefficient which allows for the effect of non-uniform self-equilibrating stresses, which lead to a reduction of restraint forces
k_c = coefficient which takes account of stress distribution within the section immediately prior to cracking and of the change of the lever arm

The check of minimum reinforcement can be shown in SCIA Engineer:

Crack proof EN 1992-1-1

Linear calculation, Extreme : Global
Selection : All
Combinations : SLS

Minimum reinforcement for selected members

Member	d_x [m]	Case	k_c [-] k [-]	h [mm] h^* [mm]	σ [MPa] $f_{ct,eff}$ [MPa]	$A_{s,min}$ [mm^2] $A_{s,prov}$(P) [mm^2]	$Check_{calc}$ [-] $Check_{lim}$ [-]	Check
S1	0.500	SLS/1	0.40 0.86	500 500	213.2 2.90	351 1571	0.22 1.00	OK

Control of cracking without direct calculation (art 7.3.3)

Where the minimum reinforcement is provided, crack widths are unlikely to be excessive for cracks caused mainly by loading; either the provisions of Table 7.2N OR the provisions of Table 7.3N are complied with:

Table 7.2N Maximum bar diameters ϕ^*_s for crack control[1]

Steel stress[2] [MPa]	Maximum bar size [mm] w_k= 0,4 mm	w_k= 0,3 mm	w_k= 0,2 mm
160	40	32	25
200	32	25	16
240	20	16	12
280	16	12	8
320	12	10	6
360	10	8	5
400	8	6	4
450	6	5	-

Notes: 1. The values in the table are based on the following assumptions:
c = 25mm; $f_{ct,eff}$ = 2,9MPa; h_{cr} = 0,5; $(h-d)$ = 0,1h; k_1 = 0,8; k_2 = 0,5; k_c = 0,4; k = 1,0; k_t = 0,4 and k' = 1,0
2. Under the relevant combinations of actions

Table 7.3N Maximum bar spacing for crack control[1]

Steel stress[2] [MPa]	Maximum bar spacing [mm] w_k=0,4 mm	w_k=0,3 mm	w_k=0,2 mm
160	300	300	200
200	300	250	150
240	250	200	100
280	200	150	50
320	150	100	-
360	100	50	-

For Notes see Table 7.2N

We can also see the maximum bar spacing and maximum diameters:

Crack proof EN 1992-1-1

Linear calculation, Extreme : Global
Selection : All
Combinations : SLS
Maximum bar spacing for selected members

Member	d_x [m]	Case	N [kN] N_r [kN]	M_y [kNm] M_{yr} [kNm]	σ [MPa]	s_{max} [mm]	s_{prov}(P) [mm]	$Check_{calc}$ [-] $Check_{lim}$ [-]	Check
S1	8.500	SLS/1	0.00 0.00	63.43 39.61	244.7	194	180	0.93 1.00	OK

Maximum diameters for selected members

Member	d_x [m]	Case	N_r [kN] N_r [kN]	M_{yr} [kNm] M_{yr} [kNm]	σ [MPa]	$φ^*$ [mm]	$Φ_s$ [mm]	$Φ_{prov}$(P) [mm]	$Check_{calc}$ [-] $Check_{lim}$ [-]	Check	W/E
S1	5.000	SLS/1	0.00 0.00	-43.22 -43.22	300.0	11.0	**10.0**	20.0	2.00 1.00	NOT OK	669

Note that **only one of these two conditions should be satisfied**. In this case the maximum bar spacing is not exceeded, so also this third check is passed.

Since all three conditions are satisfied, we can conclude that our example passes the crack control.

5_Automatic member reinforcement design (AMRD)

In chapter two of the chapter beam calculation, the theoretical design of reinforcement was explained. In chapter three, practical reinforcement was inserted manually, based on the amount of required reinforcement. In chapter four, a response check and capacity check was performed.

This procedure is collected in the option ‘Automatic reinforcement design’.
Automatic reinforcement design takes into account:

- capacity check
- response check
- detailing provisions: minimum and maximum percentage and maximum bar distances.

The method of AMRD is explained by means of the same example explained in ‘1_description’ of used example. Open this example ‘beam.esa’.

First of all, we have a look at the theoretical design. It’s always recommended to do this before you perform an AMRD. On that way you have an idea of the amount of reinforcement and the possible minimum diameter.

For example: a basic diameter of 10 mm and quality B 600C is taken in the setup of concrete. The following output is obtained:

Design As EN 1992-1-1

Linear calculation, Extreme : Member
Selection : All
Combinations : ULS

Main upper reinforcement for selected beams

Member	d_x [m]	Case	N_d [kN]	M_{yd} [kNm]	x_u [mm]	d [mm]	b_{tor} [mm]	$A_{s,b}$ [mm²]	$A_{s,t}$ [mm²]	$A_{s,req}$ [mm²]	Reinf.[no.]	W/E
S1	4.900	ULS/1	0.00	-242.22	135	452	300	1148	0	1148	15x10.0(1178)	67

Main lower reinforcement for selected beams

Member	d_x [m]	Case	N_d [kN]	M_{yd} [kNm]	x_u [mm]	d [mm]	b_{tor} [mm]	$A_{s,t}$ [mm²]	$A_{s,req}$ [mm²]	Reinf.[no.]	W/E
S1	2.500	ULS/1	0.00	234.38	130	452	300	0	1105	15x10.0(1178)	68

Detailing reinforcement for selected member

Member	d_x [m]	Case	T_{Ed} [kNm]	A_k [mm²]	u_k [mm]	$A_{s,tors}$ [mm²]	$A_{s,det}$ [mm²]	Reinf.[no.]	W/E
S1	2.500	ULS/1	0.00	82416	1216	0	0	2x8.0(101)	78
S1	4.900	ULS/1	0.00	82416	1216	0	0	2x8.0(101)	78

Explanation of warnings and errors

67	The bar distance for the upper reinforcement is too small.
68	The bar distance for the lower reinforcement is too small.
78	The reinforcement was designed according to detailing provisions.

It is obvious that this diameter cannot be used for a design because of the bar distances. So, we increase the diameter in the setup to 16 mm:

Design As EN 1992-1-1

Linear calculation, Extreme : Member
Selection : All
Combinations : ULS

Main upper reinforcement for selected beams

Member	d_x [m]	Case	N_d [kN]	M_{yd} [kNm]	x_u [mm]	d [mm]	b_{tor} [mm]	$A_{s,b}$ [mm²]	$A_{s,t}$ [mm²]	$A_{s,req}$ [mm²]	Reinf.[no.]	W/E
S1	4.900	ULS/1	0.00	-242.22	136	449	300	1159	0	1159	6x16.0(1206)	67

Main lower reinforcement for selected beams

Member	d_x [m]	Case	N_d [kN]	M_{yd} [kNm]	x_u [mm]	d [mm]	b_{tor} [mm]	$A_{s,t}$ [mm²]	$A_{s,req}$ [mm²]	Reinf.[no.]	W/E
S1	2.500	ULS/1	0.00	234.38	131	449	300	0	1115	6x16.0(1206)	68

Detailing reinforcement for selected member

Member	d_x [m]	Case	T_{Ed} [kNm]	A_k [mm²]	u_k [mm]	$A_{s,tors}$ [mm²]	$A_{s,det}$ [mm²]	Reinf.[no.]	W/E
S1	2.500	ULS/1	0.00	78804	1192	0	0	2x8.0(101)	78
S1	4.900	ULS/1	0.00	78804	1192	0	0	2x8.0(101)	78

Explanation of warnings and errors

67	The bar distance for the upper reinforcement is too small.
68	The bar distance for the lower reinforcement is too small.
78	The reinforcement was designed according to detailing provisions.

Also, in this case, the bar distance is too small. However, for an automatic design, one layer can contain a mix of diameters. So, we keep the diameter of 16 mm as minimum.

Now, we can do an automatic reinforcement design. The non-prestressed reinforcement is designed in the selected beam. Both longitudinal and stirrups are designed.

There are 3 manners to do this:

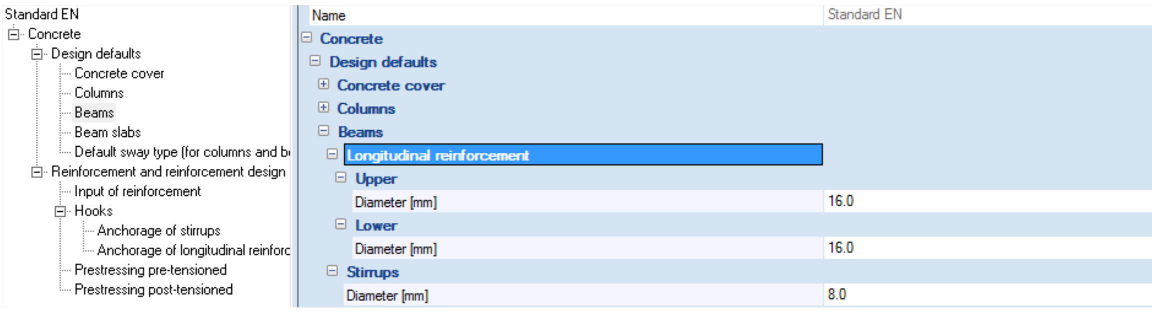
- By means of the setup
- By means of the member data
- By means of a reinforcement template

The 3 options will be explained.

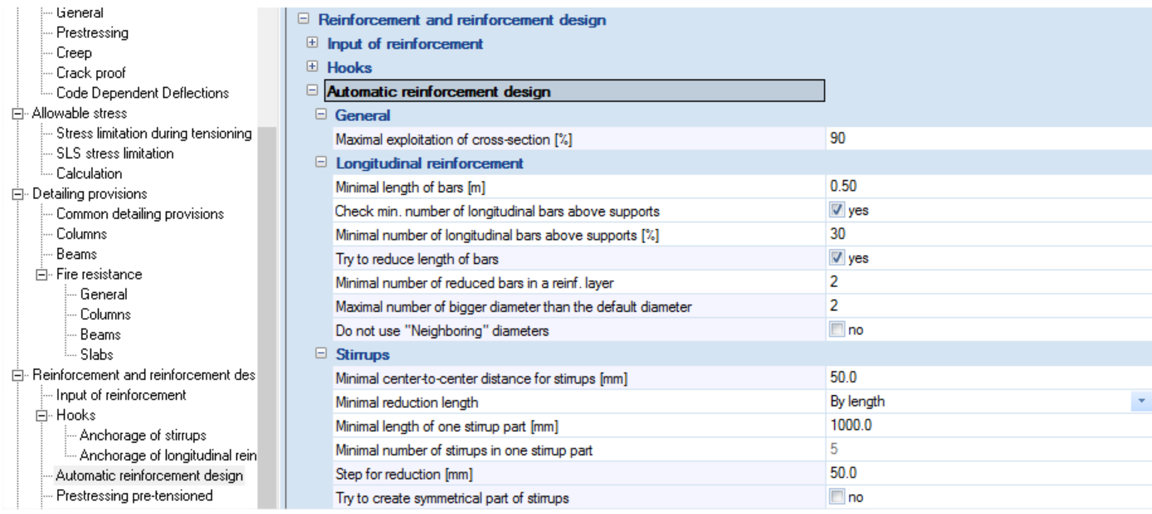
AMRD by means of the setup

Project: AMRD01.esa

First of all, the minimum diameter has to be determined in the setup. This was done in the previous step:



After this, some settings of the AMRD can be adapted in the setup of concrete:



<u>Explanation of the AMRD-parameters</u>

General

Maximum exploitation of cross-section [%]
Specifies the maximal utilization of the cross-section in the automatically reinforced beam.
The value may be between 1 and 100%.

Longitudinal reinforcement

Minimal length of bars [m]
Defines the minimal length of bars automatically inserted into the reinforced beam. The algorithm tries to shorten the reinforcement bars if possible so that they do not extent along the whole length of the beam. The shortened bar cannot be shorter than the value of this parameter.

Check min. number of longitudinal bars above supports

If ON, the number of reinforcement bars above supports is checked and compared to the values specified below.

Minimal number of longitudinal bars above supports [%]
Defines the minimal required amount of reinforcement bars above supports.

Try to reduce length of bars
If OFF, the program uses only bars that extent along the whole length of the beam.
If ON, some bars may be shortened if the unity check is satisfied without them.

Minimal number of reduced bars a in reinforcement layer
Imagine a simply supported beam. There must be 6 bars in the middle of the span. Only 5 bars are required in a section that is closer to the support. And only 4 bars are necessary in another section that is still closer to the support. And so on. So in general, you could shorten the bars one by one. This may be sometimes impractical as it would lead to a large number of different bars.
The value in this parameter defines the minimal number of bars that may be shortened at the same time. The default value is 2. It means that, in our example, you would have 6 bars in the middle, there still will be 6 bars in the section where 5 is sufficient, and only farther towards the support the number will be reduced to 4 bars. And so on.

Maximal number of bigger diameter than the default diameter
Defines how many different (bigger) diameters of the reinforcement can be used for the optimization.
Let us assume that the default diameter specified in the Design default tab is 10mm.
If this parameter is set to 2, the program can use diameters 10, 12 (i.e. +1 item in the manufacturing program) and 14 (i.e.+2 item in the manufacturing program) for the design.

Do not use "Neighboring" bars
Some standards recommend that "neighboring" profiles from the manufacturing program should not be used in one beam (in order to avoid unintentional interchange of the profiles).
Let us assume that the default diameter specified in the Design default tab is 10mm. Further assume that Maximal number of bigger diameters than the default is set to 2.
If this option is ON, the following bars can be inserted into the beam: (i) either 10mm, (ii) or 12mm, (iii) or 14mm, (iv) or 10mm and 14mm can be combined together. 10mm and 12mm are not permitted to be combined in one beam.

Stirrups

Minimal center-to-center distance for stirrups [mm]
Specifies the minimal distance between stirrups measured from the centre of a bar to the centre of an adjacent bar.

Minimal reduction length
Defines the minimum reduction length. It may be defined by means of length or number of stirrups in the part. See the next two parameters.

Minimal length of one stirrup part [mm]
Defines the length of one part of the beam where stirrups are distributed uniformly. This parameter ensures than a situation the distance between two adjacent stirrups is different for every two adjacent stirrups.

Minimal number of stirrups in one stirrup part
Analogous to the parameter above.

Step for reduction [mm]
Defines the step for the reduction of the distance between two adjacent stirrups. This insures that the distance between stirrups is always a "rounded" number – e.g. 200mm, then 250mm, then 300mm, etc. (and not e.g. 200, 246mm, 298mm,etc.).

Try to create symmetrical part of stirrups
This parameter may enforce that the stirrup part are symmetrical along the length of the beam.

To see the influence of the diameter, we change the maximal number of bigger diameters than the default diameter into 0:

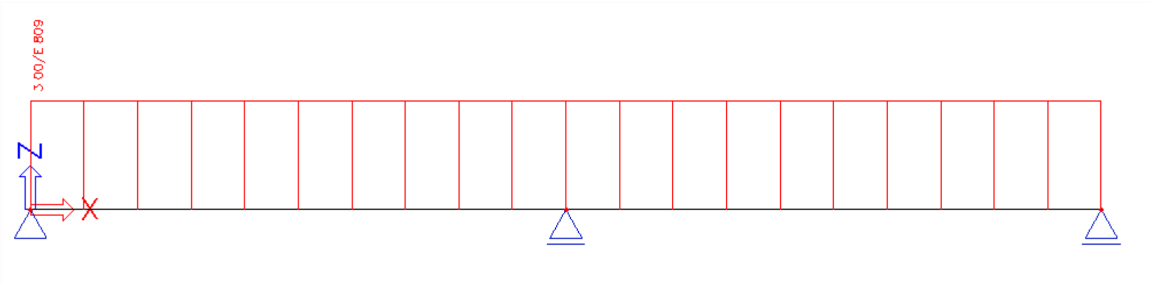
Longitudinal reinforcement	
Minimal length of bars [m]	0.50
Check min. number of longitudinal bars above supports	yes
Minimal number of longitudinal bars above supports [%]	30
Try to reduce length of bars	yes
Minimal number of reduced bars in a reinf. layer	2
Maximal number of bigger diameter than the default diameter	0
Do not use "Neighboring" diameters	no

Now, we can perform a Reinforcement design.

Go to Automatic member reinforcement design > Reinforcement design.

And select combination ULS.

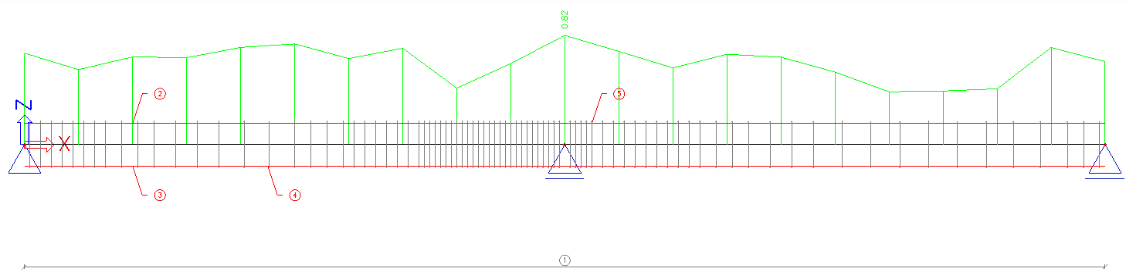
After refreshing, the following result is obtained:

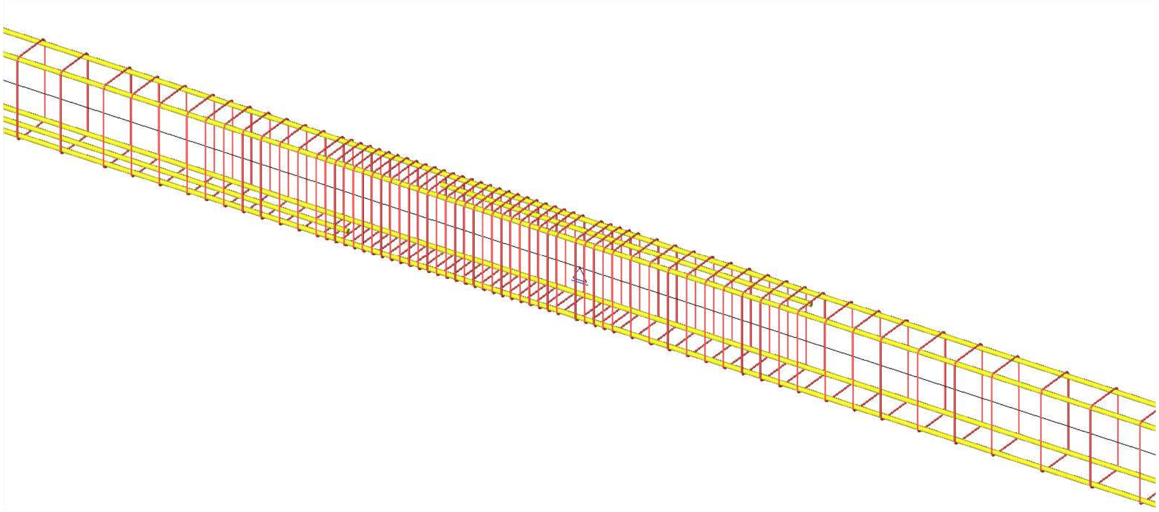


SCIA Engineer could not find a solution. This was expected, because we stated that only the diameter 16 could be used. The bar distances will be too small.

So, we go back to the concrete setup and change the Maximal number of bigger diameters than the default diameter into 3. So 16, 18, 20 and 25 mm can be used.

Now, after refreshing, the amount of reinforcement is:



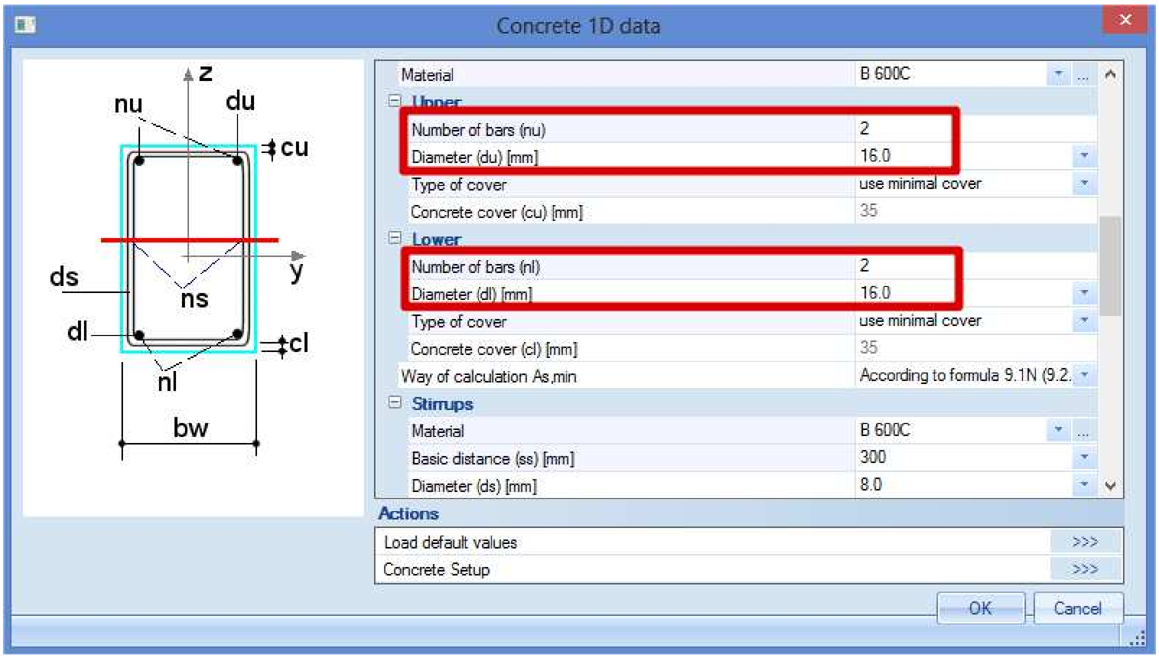


AMRD by means of the concrete data

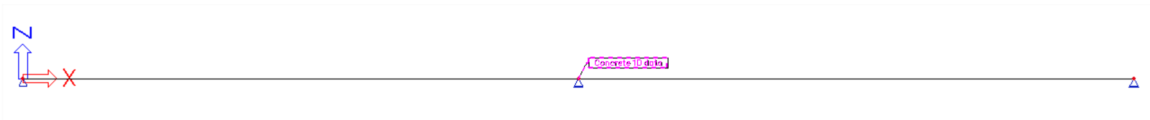
Project: AMRD02.esa

In the first step, Member data are inputted on the beam. So go to Member data in the concrete menu and select the beam.

Next, some basic reinforcement is chosen, namely 2 bars of diameter 16 mm at the upper and lower side:



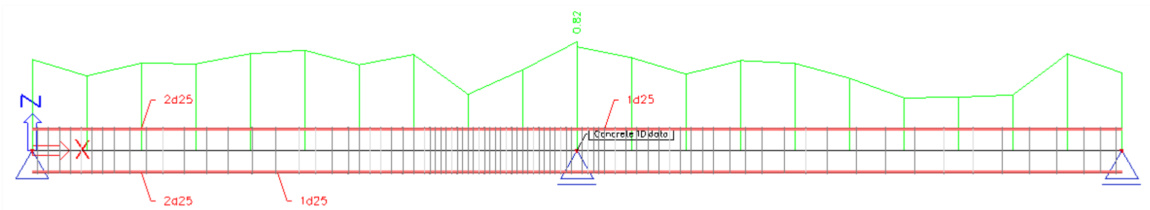
Remark: it is not obligated to insert a number of bars, it is also possible to insert only a basic diameter.



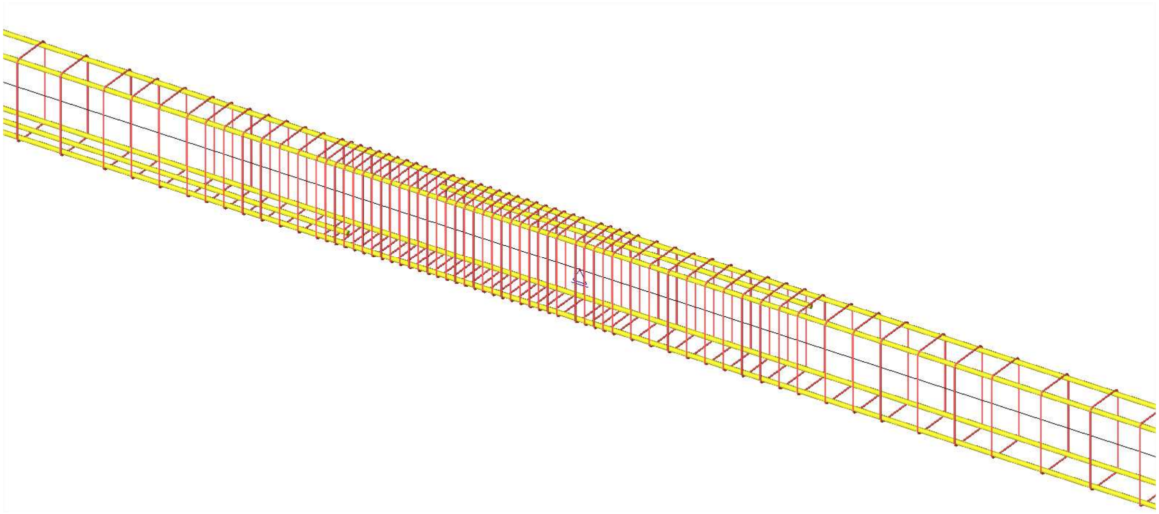
Next, select Automatic member reinforcement design > Reinforcement design.

And select combination ULS.

Now, the basic reinforcement of the member data will be used because they overwrite the diameters given in the Concrete Setup:



This results in the following reinforcement configuration:



As expected, we will get the same result as in the previous example: a diameter of 16 mm is implemented but we may use also diameter 18 mm, 20 mm and 25 mm.

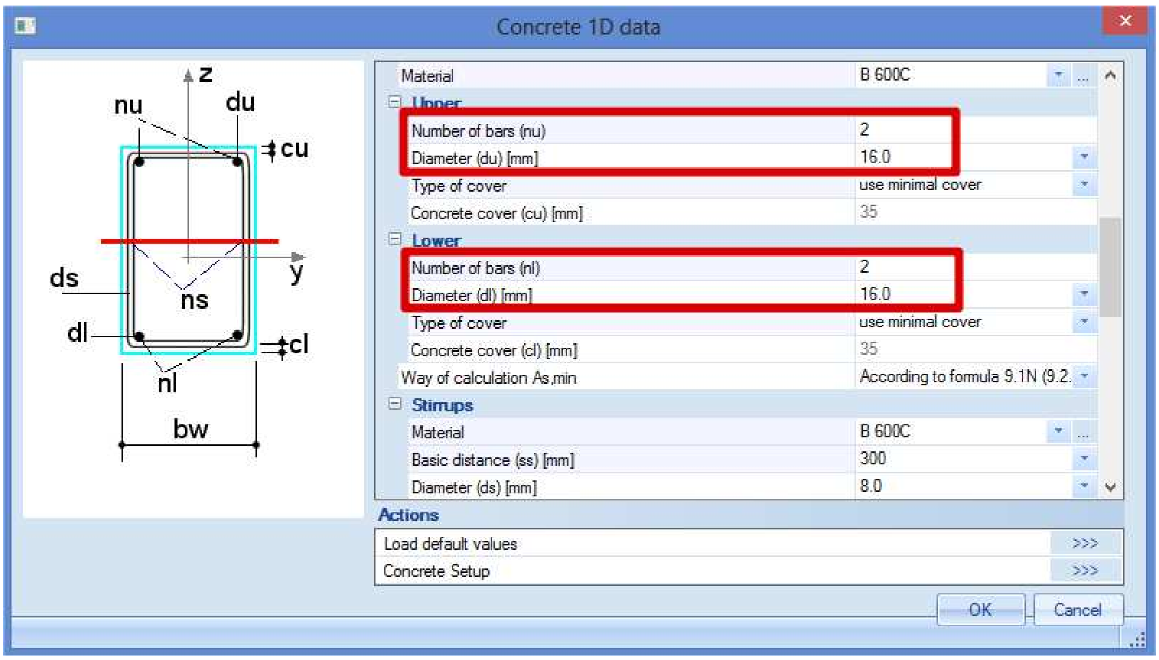
AMRD by means of a reinforcement template

Project: AMRD03.esa

A last option to do an AMRD design is according to a reinforcement template. This can be determined in the AMRD data.

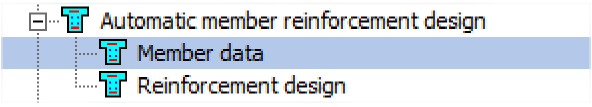
Here you can choose if you use the diameters of the Concrete Setup (AMRD01.esa) or if you use the diameters defined in the Member data (AMRD02.esa).

Suppose that we use the same member data as in the previous example: 2 bars of diameter 16mm at the upper side and 2 bars of diameter 16mm at the lower side:

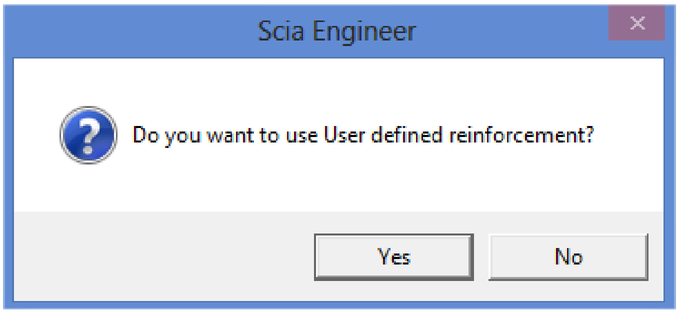


In the second step, the member data of the Automatic member reinforcement design can be inputted.

Select Automatic member reinforcement design > Member data:

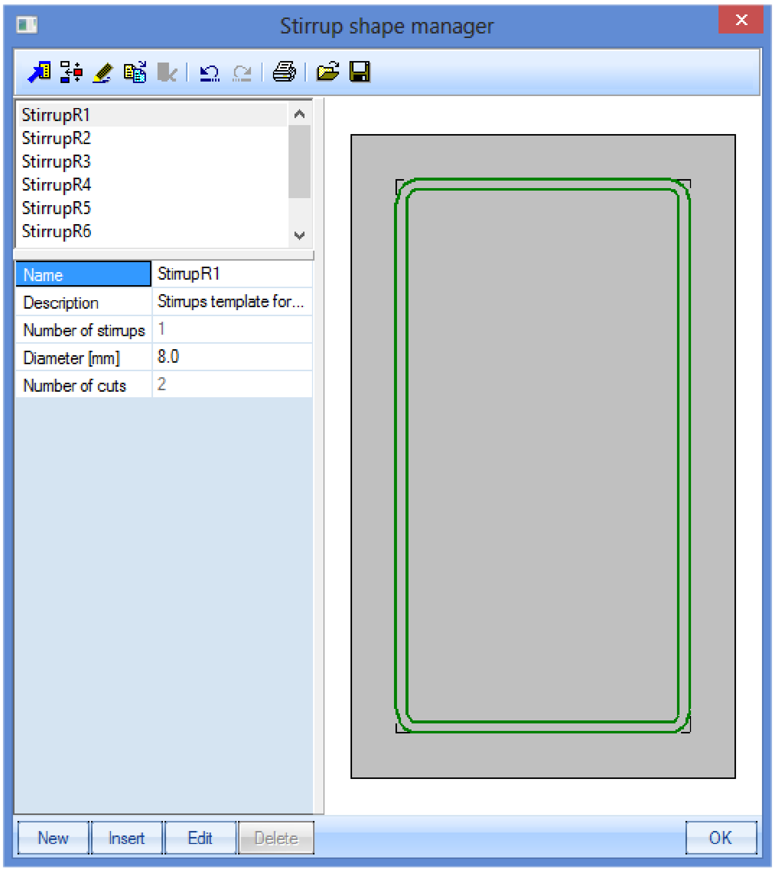


The following question arises:

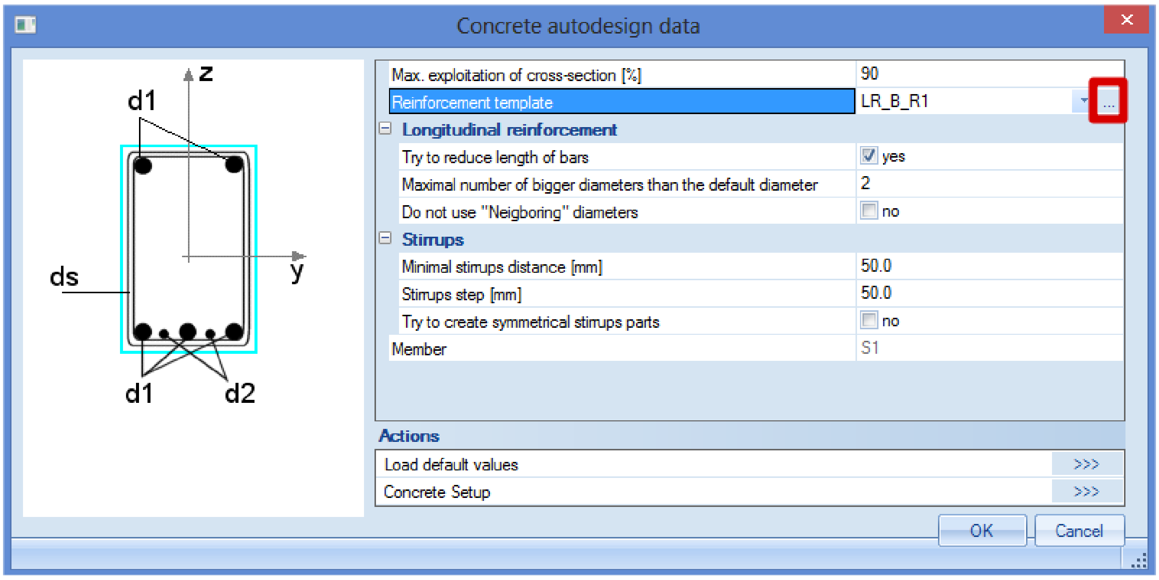


Confirm this by clicking “Yes”.

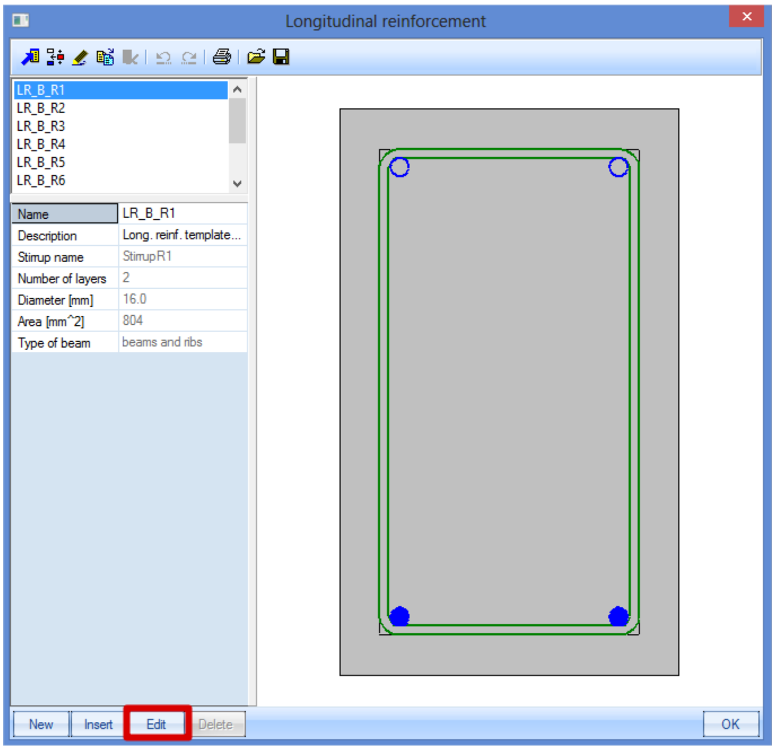
Then, the dialog box of the stirrup shape manager opens, where you can choose a stirrup template:



Click OK to see the autodesign member data:



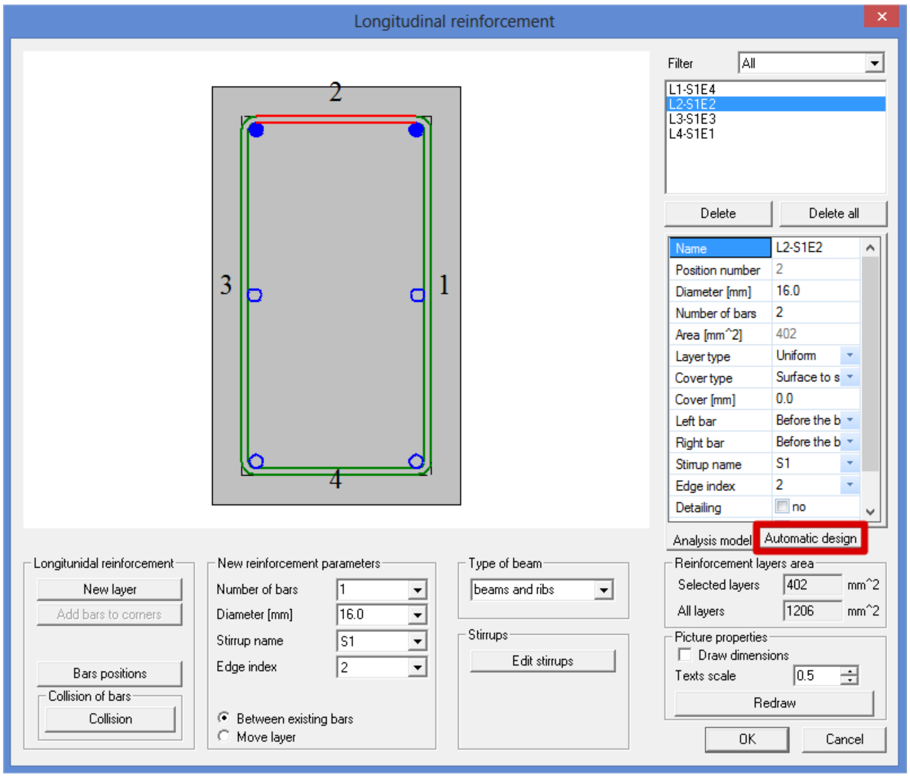
Here you can click on the button behind Reinforcement template to see the templates for the longitudinal reinforcement:



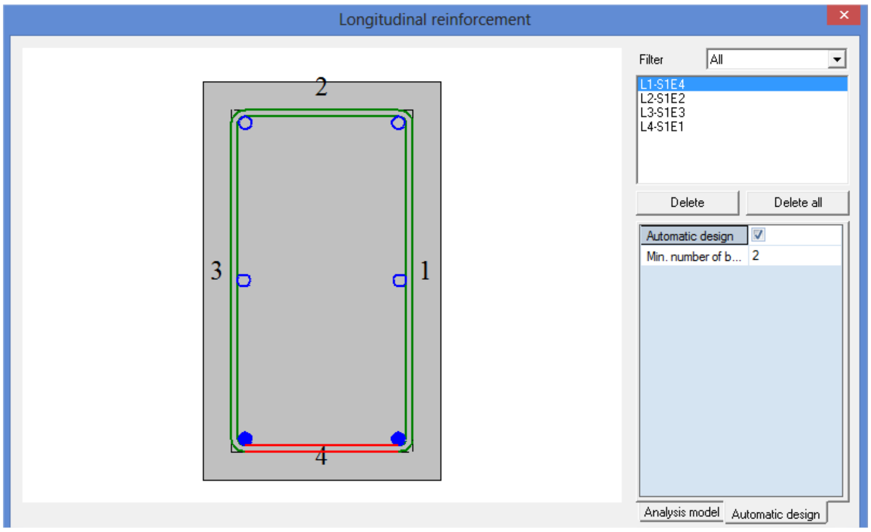
In that case, the selected template LR_B_R1 is used as basic reinforcement. Click on the button ‘Edit’ to see the reinforcement parameters.

Create the following configuration:
  Upper: 2 bars

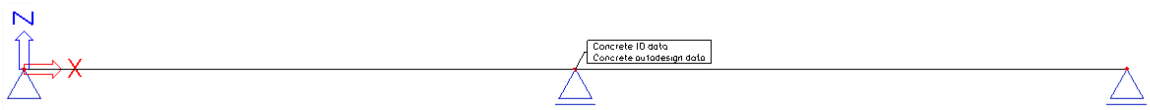
Lower: 2 bars
Middle left/right: 1 bar at each side



Select the tab Automatic design:

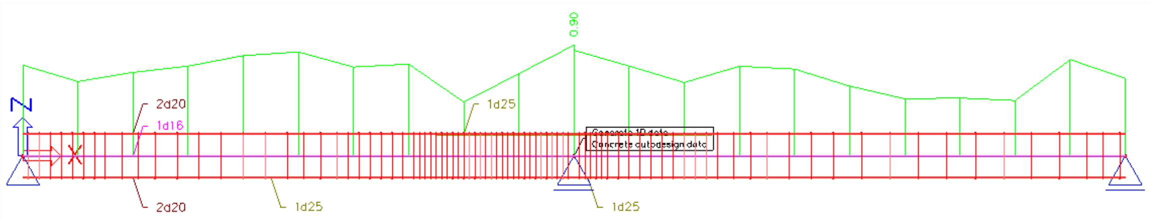


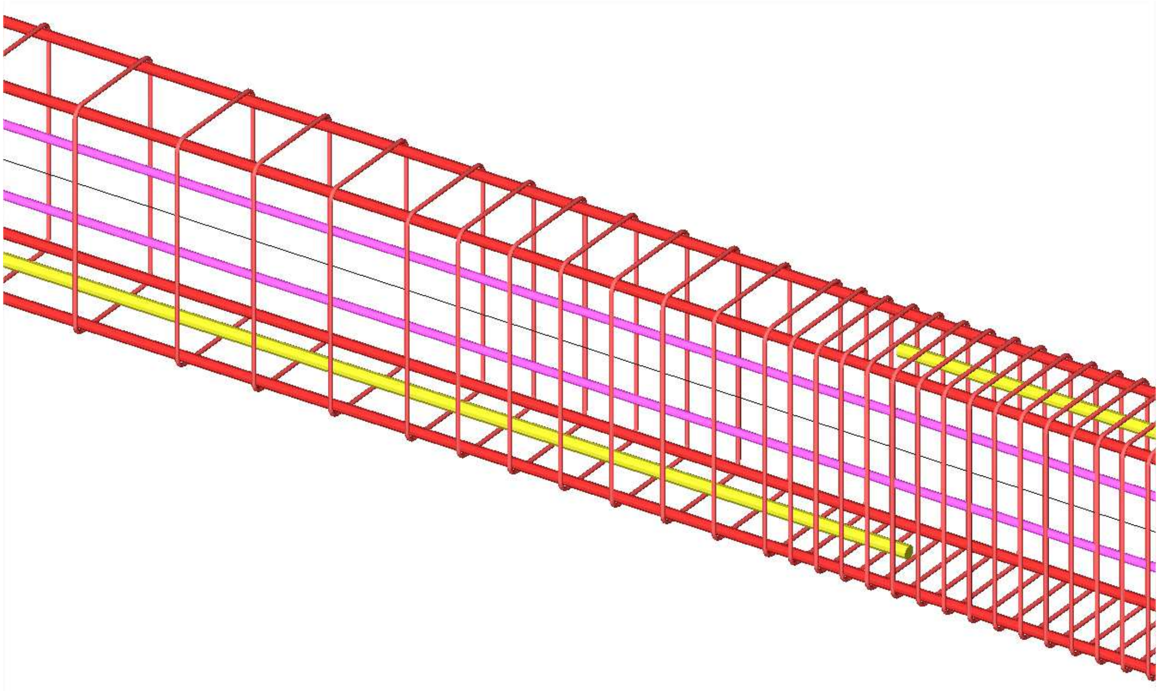
Note that this option is checked for the upper and lower bars. The bars in the middle are not taken into account for the automatic design. This means that the upper and lower bars will be used to vary during the design process.



For example, the first configuration that will be verified is the configuration of the template. Suppose that this gives a capacity check higher than 1. So, new bars are added or diameters are changed (only 3 diameter higher regarding the member data!) to satisfy the check.

The two flank bars however are not taken into account in the automatic reinforcement design. This means that they are included for the checks, but their diameter or number will not vary during the process.





Column Design

Calculation Types

For column design, there are 3 types of calculation:

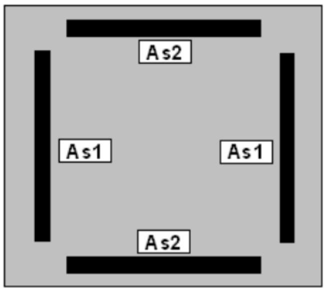
- Axial compression only
- Uni-axial bending
- Bi-axial bending

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

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

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

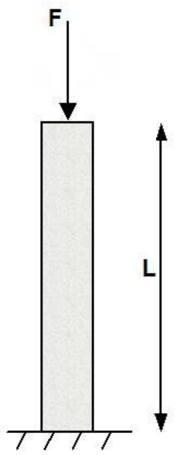
The bi-axial bending calculation has as result a 2-directional reinforcement configuration. The number of bars may differ per direction, but is always the same for parallel sides:



The uni-axial bending calculation is a relatively simple calculation type, while the bi-axial bending calculation requires an iterative process.

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

Axial compression only

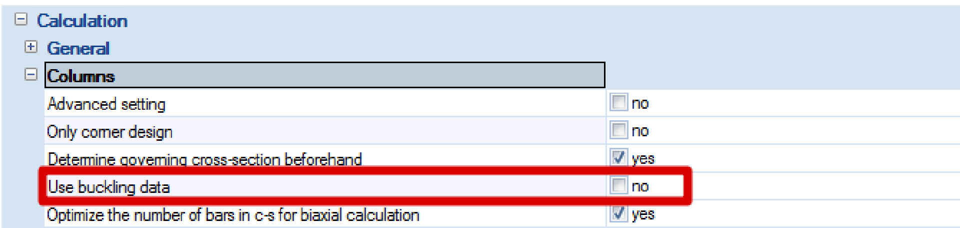


No reinforcement required: $N_{Ed} < N_{Rd}$

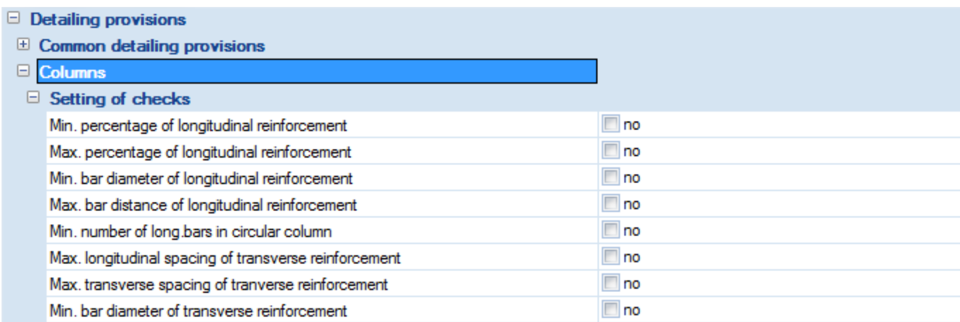
Example: open the example 'Axial force – uni ax bending.esa'

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

Concrete Setup
Item General: Calculation > Tab Columns: 'Buckling data' are not taken into account (only 1st order moments are considered).



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



Loads
BG1: Permanent load > F = 1100 kN
BG2: 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 * BG1 + 1,50 * BG2
Design normal force N_d = 1,35 * 1100 + 1,50 * 1000 = 2985 kN

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

Design defaults	
Concrete cover	
Use min concrete cover	☑
Design working life [years]	50
Exposure class	XC3
Abrasion class	None
Type of concrete	In-situ concrete
Special geometric quality control	☐ no
Type of concrete surface	Normal surface
Special concrete quality control	☐ no
Columns	
Longitudinal reinforcement	
Diameter [mm]	20.0
Stirrups	
Diameter [mm]	8.0

By default, the diameter for the main column reinforcement is put to 20 mm. 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.

Results
Go to Concrete menu > Member design - Design, ask the value for A_s for member B1, and click the action button [Refresh]. The value 0 appears on the screen, followed by warning 77.

Select now the option 'Print explanation of errors and warnings' in the Properties menu, and click the action [Preview]. The following screen will appear:

Design As EN 1992-1-1

Linear calculation, Extreme : Member
Selection : B1
Combinations : ULS

Main reinforcement for selected columns

Member	d_x [m]	Case	N_d [kN]	M_{yd} [kNm]	M_{zd} [kNm]	Calc. type	$A_{s,req}$ [mm^2]	W/E
B1	0.000	ULS/1	-2985.00	0.00	0.00	N/A	0	77

Explanation of warnings and errors

77	The calculation of the main reinforcement is done and the theoretical reinforcement is zero.

No theoretical reinforcement is required.

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

Check of reinforcement

$N_{Rd} = f_{cd} \cdot \alpha \cdot A_c$
$= 30 \cdot 0{,}85 \cdot 350^2 / 1000 = 3124$ kN

Since N_{Rd} = 3124 kN > N_d = 2985 kN, indeed no theoretical reinforcement is required.

Reinforcement required: $N_{Ed} > N_{Rd}$

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

Loads
BG1: Permanent load > F = 2000 kN
BG2: Variable load > F = 1000 kN

Combination according to the Eurocode:
ULS Combination = 1,35 * BG1 + 1,50 * BG2
Design normal force N_d = 1,35 * 2000 + 1,50 * 1000 = 4200 kN

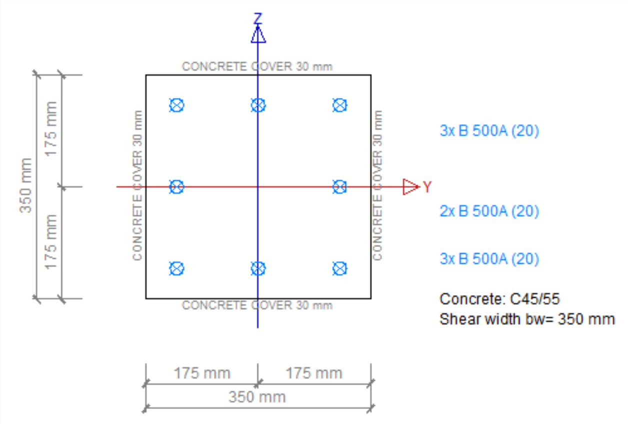
Results
Remark that SCIA Engineer shows on the screen the reinforcement per direction. The total reinforcement area is in fact 750 + 750 = 1500 mm^2.

Design As EN 1992-1-1

Linear calculation, Extreme : Member
Selection : B2
Combinations : ULS
Main reinforcement for selected columns

Member	d_x [m]	Case	N_d [kN]	M_{yd} [kNm]	M_{zd} [kNm]	Calc. type	Ratio y/z [%]	$A_{s,req}$ [mm^2]	$Reinf_{req}$	$Reinf_{tot}$
B2	0.000	ULS/1	-4200.00	0.00	0.00	N/A	50/50	1500	8(6/6)x20.0	8x20.0(2513)

When going to Concrete menu > Member design - Design, the user has the choice for the action Single Check at the bottom of the Properties menu. There the proposed configuration can be found:

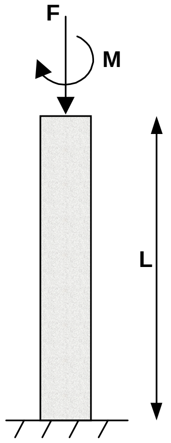


Explanation of the number of reinforcement bars

In the previous example, the configuration of required reinforcement bars was noted as 8(6/6), which stands for “6 bars in the y direction (3 bars per side //y), 6 bars in the z direction (3 bars per side //z)”.

This is a simple representation of the bar configuration, so the user can easily understand how the bars should be inputted.

Uni-axial bending



For uni-axial bending, 2 types of calculation are supported: the (sum) and the (max) calculation.

Uni-axial bending calculation (sum)

Principle
The reinforcement is designed separately around both axes, and the final area of reinforcement is the sum of reinforcement for both directions:

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

This calculation often gives uneconomic solutions for the reinforcement. This is the reason another calculation type has been implemented, uni-axial bending calculation (max), which may be used in most situations.

See paragraph Calculation methods: Overview – Automatic design, to understand how it is determined which calculation may be used.

Uni-axial bending calculation (max)

Principle
The reinforcement is designed only around the axis with the bigger bending moment:

- 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 $M_{Ed,z}$

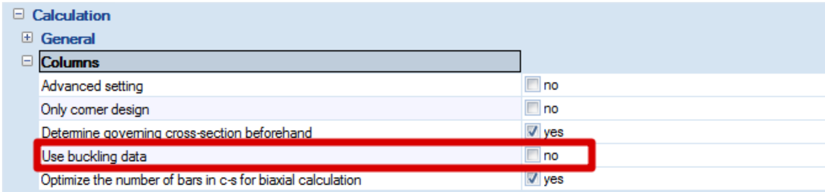
Example

Example: open the example 'Axial force – uni ax bending.esa'

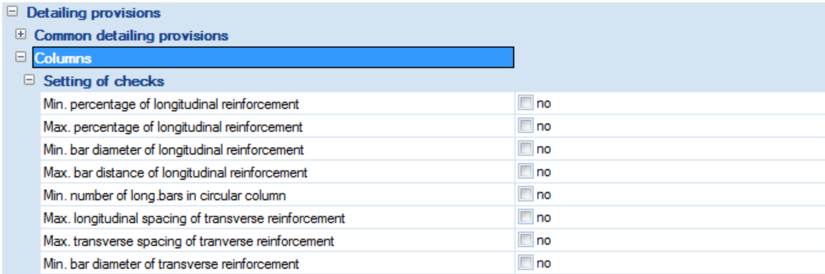
Geometry
Column cross-section: RECT 350x350mm²
Height: 4,5 m
Concrete grade: C45/55

Concrete Setup

Item General: Calculation > Tab Columns: ‘Buckling data’ are not taken into account (only 1st order moments are considered).



Item Detailing provisions: not taken into account, to view the pure results.



Loads

BG1: Permanent load > F = 500 kN; M_y = 100 kNm
BG2: Variable load > F = 1000 kN; M_y = 100 kNm

Combination according to the Eurocode:
ULS Combination = 1,35 * BG1 + 1,50 * BG2
Design normal force N_d = 1,35 * 500 + 1,50 * 1000 = 2175 kN
Design moment M_{yd} = 1,35 * 100 + 1,50 * 100 = 285 kNm

Results

Go to Concrete menu > Member design - Design, ask the value for As, and click the action buttons [Refresh] and [Preview]. The numerical results of the calculation are as follows:

Design As EN 1992-1-1

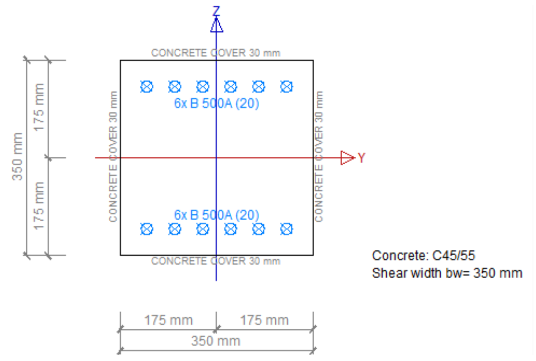
Linear calculation, Extreme : Member
Selection : B3
Combinations : ULS

Main reinforcement for selected columns

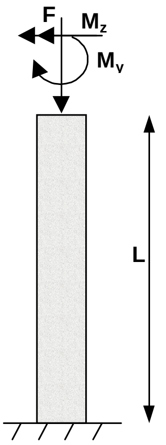
Member	d_x [m]	Case	N_d [kN]	M_{yd} [kNm]	M_{zd} [kNm]	Calc. type	Ratio y/z [%]	$A_{s,req}$ [mm²]	$Reinf_{req}$	$Reinf_{tot}$	W/E
B3	0.000	ULS/1	-2175.00	-285.00	0.00	Um	100/0	3160	12(12/4)x20.0	12x20.0(3770)	134

Remark the calculation type: Um (Uni-axial bending (max)).
Remark that ratio y/z is 100/0: all of the reinforcement works in 1 direction.

When going to the Concrete menu > Member design - Design, the user has the choice for the action Single Check at the bottom of the Properties menu. There the proposed configuration can be found:



Bi-axial bending



Bi-axial bending (art 5.8.9)

To decide if a bi-axial bending calculation is required, the following conditions should be checked:

No further check is necessary if the slenderness ratios satisfy the following two conditions:

$$\lambda_y/\lambda_z \leq 2 \text{ and } \lambda_z/\lambda_y \leq 2 \qquad (5.38a)$$

and if the relative eccentricities e_y/h and e_z/b satisfy one of the following conditions:

$$\frac{e_y/h_{eq}}{e_z/b_{eq}} \leq 0{,}2 \text{ or } \frac{e_z/b_{eq}}{e_y/h_{eq}} \leq 0{,}2 \qquad (5.38b)$$

where:

- b, h are the width and depth of the section
- $b_{eq} = i_y\sqrt{12}$ and $h_{eq} = i_z\sqrt{12}$ for an equivalent rectangular section
- λ_y , λ_z are the slenderness ratios l_0/i with respect to y- and z-axis respectively
- i_y, i_z are the radii of gyration with respect to y- and z-axis respectively
- e_z = M_{Edy} / N_{Ed}; eccentricity along z-axis
- e_y = M_{Edz} / N_{Ed}; eccentricity along y-axis
- M_{Edy} is the design moment about y-axis, including second order moment (if required)
- M_{Edz} is the design moment about z-axis, including second order moment (if required)
- N_{Ed} is the design value of axial load in the respective load combination

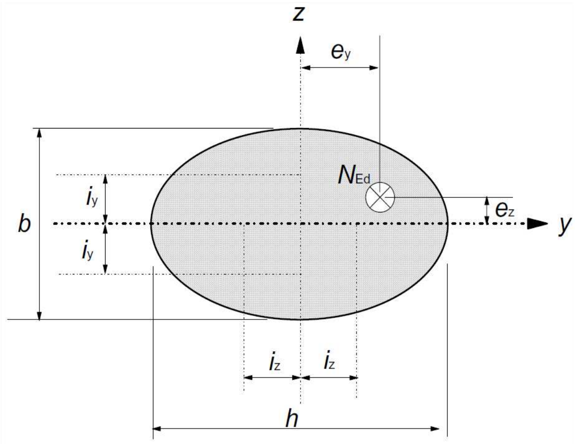


Figure 5.8. Definition of eccentricities e_y and e_z.

If the conditions of expression (*5.38a*) and (*5.38b*) are **NOT** fulfilled, bi-axial bending should be taken into account including the 2nd order effects in each direction (unless they may be ignored).
In the absence of an accurate cross-section design for bi-axial bending, the following simplified criterion may be used:

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

where:

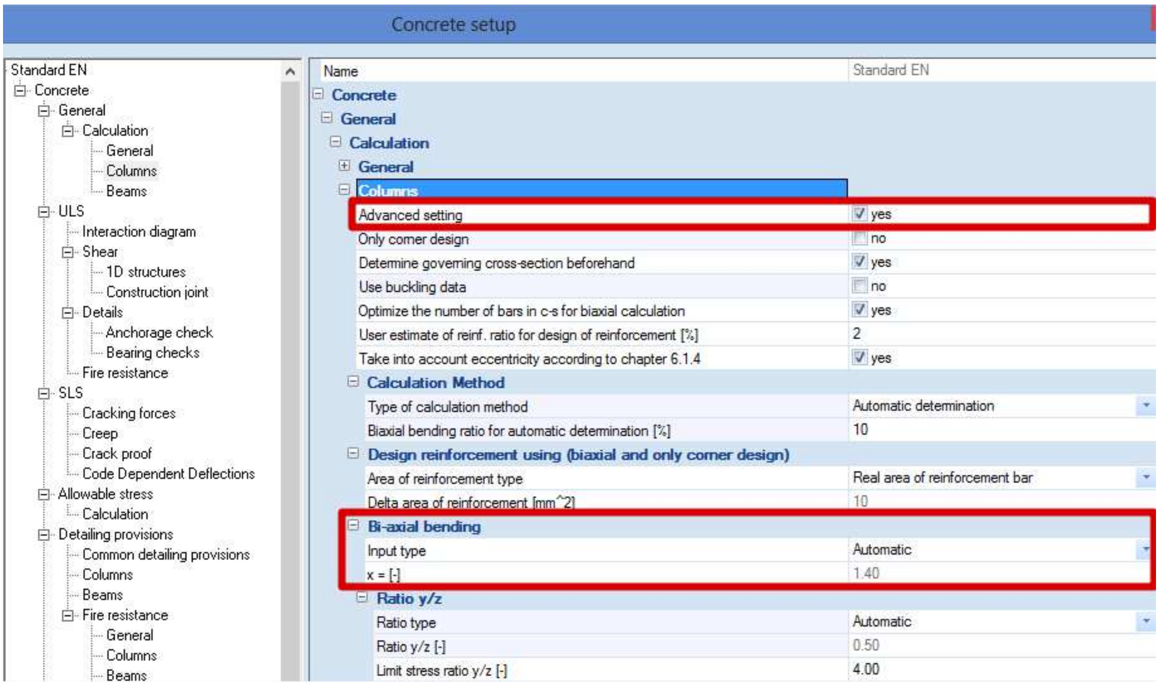
$M_{Edz/y}$ is the design moment around the respective axis, including a 2nd order moment (if required)
$M_{Rdz/y}$ is the moment resistance in the respective direction
A is the exponent;
for circular and elliptical cross sections: a = 2
for rectangular cross sections:

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} is the design value of axial force
N_{Rd} = $A_c f_{cd} + A_s f_{yd}$, the design axial resistance of the section
where:
A_c is the gross area of the concrete section
A_s is the area of longitudinal reinforcement

The criterion described by *Expression 5.39* – which is known as the interaction formula – is also implemented in SCIA Engineer as basis for the bi-axial bending calculation.
Only the notation ‘a’ for the exponent (= safety factor) of the interaction formula has been replaced by an ‘x’.

See Concrete Setup > General: Calculation > Tab Columns:



By default the safety factor is calculated automatically by SCIA Engineer, according to the requirements of the Eurocode mentioned in the previous paragraph.

Also, the notation for M_{Rdy} and M_{Rdz} has been replaced by M_{uy} and M_{uz} in SCIA Engineer. These are the ultimate moments, the moments that can be carried by the concrete + the present reinforcement bars.

Iteration process for bi-axial bending calculation

The increment routine for the number of bars, as implemented in SCIA Engineer, is as follows:

Start situation:

One reinforcement bar in each corner.
The diameter of the reinforcement bars is taken from the Concrete Setup, or from Member data.

Check of interaction formula:

SCIA Engineer determines the values for M_{yu} and M_{zu} for the present reinforcement layout, and calculates the interaction formula.

If the value is less than 1, the calculation is stopped.

Increment step 1:

For the 'weakest' direction, 1 bar per side is added.

Check of interaction formula:

SCIA Engineer determines the new values for M_{yu} and M_{zu} and recalculates the interaction formula.

If the value is less than 1, the calculation is stopped.

Calculation of moment ratios:

For each one of the design moments (M_{yd} and M_{zd}), SCIA Engineer determines the normal stress in the exterior fibre by dividing the design moment by the resistance moment W for that direction. This means that $\sigma_y = M_{yd} / W_y$ and $\sigma_z = M_{zd} / W_z$.

Based on these σ's, the ratio of the moments can be determined per direction:
$r_y = \sigma_y / (\sigma_z + \sigma_y)$ and $r_z = \sigma_z / (\sigma_z + \sigma_y)$

Increment step 2 to …:

According to the values for r_y and r_z the reinforcement is now increased per direction:

- For the 'strongest' direction, 1 bar per side is added.
- For the 'weakest' direction, the new number of bars in the 'strongest' direction (n) is taken and multiplied by 1/r. The value of n*1/r is rounded off to integer even values.

Check of interaction formula – after each increment step:

SCIA Engineer determines the new values for M_{yu} and M_{zu} and recalculates the interaction formula.

If the value is less than 1, the calculation is stopped.
If this is not the case, increment step 2 is repeated until the interaction check does satisfy.

Iteration process – Example

The iteration process used by SCIA Engineer can be simulated by means of an example where each time the values for the loads are increased.

Remark that the ratio between the design moments stays the same while the values are increased.

Example: open the example 'Bi axial bending.esa'

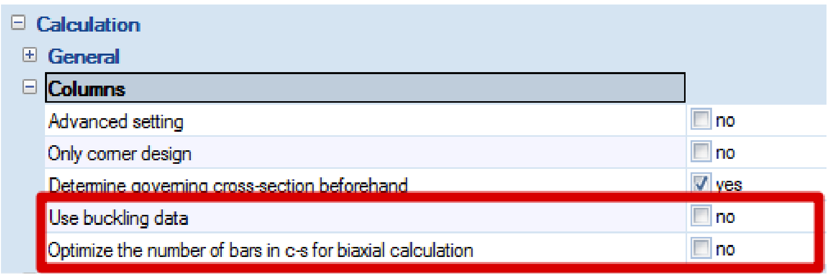
Geometry

Column cross-section: RECT 350x350mm²
Height: 4,5 m
Concrete grade: C25/30
Elastic resistance moments: $W_y = 1/6 \cdot 350^3 = 7{,}15 \cdot 10^6$ [mm^3] $= W_z$

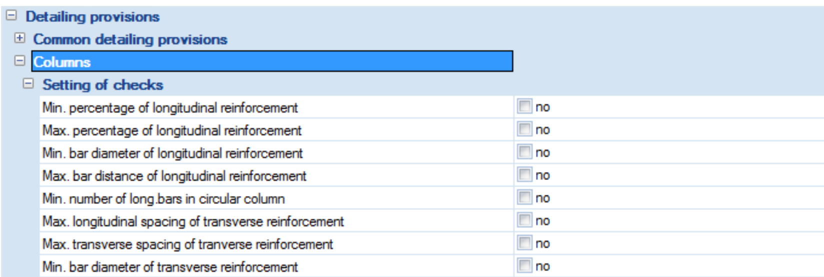
Concrete Setup

Item General: Calculation > Tab Columns: 'Buckling data' are not taken into account (only 1^{st} order moments are considered).

Item General: Calculation > Tab Columns: 'Optimize the number of bars in cross-section for bi-axial calculation' is deselected.



Item Detailing provisions: All of the provisions are deselected (to have a look at the pure results).



Loads

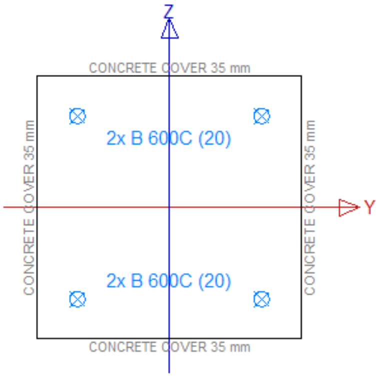
Final load configuration: N_d = 1050,00 kN

M_{yd} = 128,45 kNm
M_{zd} = 216,25 kNm

Start situation:

The start situation is always one reinforcement bar per corner. The bar diameter is taken from the Concrete Setup, or from Member data (if applied).

<u>Simulation</u>: Column B1



N_d = 1050,00 kN
M_{yd} = 51,23 kNm
M_{zd} = 86,25 kNm

Reinforcement configuration: (4/4) – This configuration can be found when going to Concrete menu > Member design - Design, and choose the action Single Check for column B1 at the bottom of the Properties menu.

Check of interaction formula:

M_{yu} = M_{zu} = 148,4 kNm

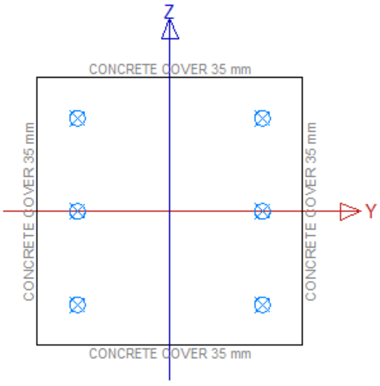
These values can be found when going to Concrete menu > Member design - Design, and choose the action Single Check for column B1 at the bottom of the Properties menu. On the tabs 'Vertical section M_y' (where M_{dz} = 0 kNm) & 'Vertical section M_z' (where M_{dy} = 0 kNm), these specific sections from the interaction diagram are shown and the ultimate values M_{yu} and M_{zu} are indicated to it.

Interaction formula: $(128{,}45/148{,}4)^{1{,}4} + (216{,}25/148{,}4)^{1{,}4} = 2{,}51 > 1$

Increment step 1:

For the 'weakest' direction, // z, 1 bar per side is added.

<u>Simulation</u>: Column B2



N_d = 1050,0 kN

M_{yd} = 63,11 kNm
M_{zd} = 106,25 kNm

Reinforcement configuration: (4/6)

Check of interaction formula:
M_{yu} = 150,3 kNm M_{zu} = 182,9 kNm

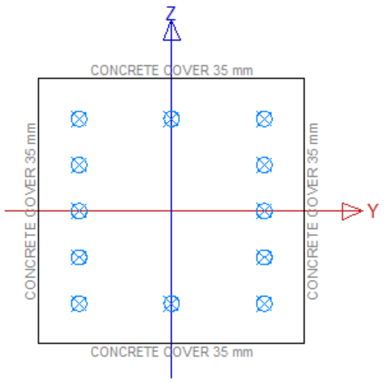
Interaction formula: $(128{,}45/150{,}3)^{1{,}4} + (216{,}25/182{,}9)^{1{,}4} = 2{,}07 > 1$

Calculation of moment ratios:
r_y = 128,45 / 344,70 = 0,37   and r_z = 216,25 / 344,70 = 0,63

The values for M_{dy} and M_{dz} are taken from the final load configuration.

Increment step 2 – 3 – 4:
- For the 'strongest' direction, // y, 1 bar per side is added.
- For the 'weakest' direction, the new number of bars in the ‘strongest’ direction (n) is taken and multiplied by $1/r_z$ (= 1/0,63 = 1,59). The value of $n*1/r_z$ is rounded off to integer even values.

<u>Simulation</u>: Column B3

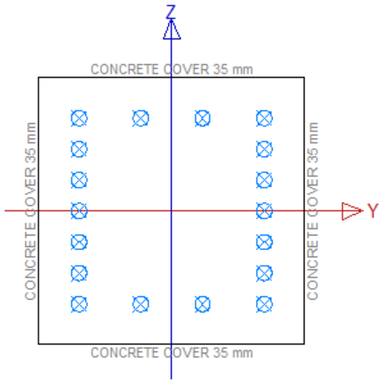


N_d = 1050,00 kN
M_{yd} = 86,86 kNm
M_{zd} = 146,25 kNm

// y   4+2 = 6 bars
// z   6*1,59 = 9,54 ~ 10 bars

Reinforcement configuration: (6/10)

<u>Simulation</u>: Column B4

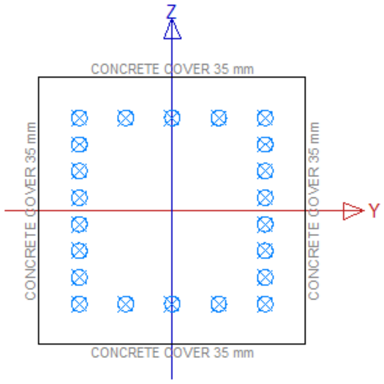


N_d = 1050,00 kN
M_{yd} = 98,74 kNm
M_{zd} = 166,25 kNm

// y 6+2 = 8 bars
// z 8*1,59 = 12,72 ~ 14 bars

Reinforcement configuration: (8/14)

Simulation: Column B5

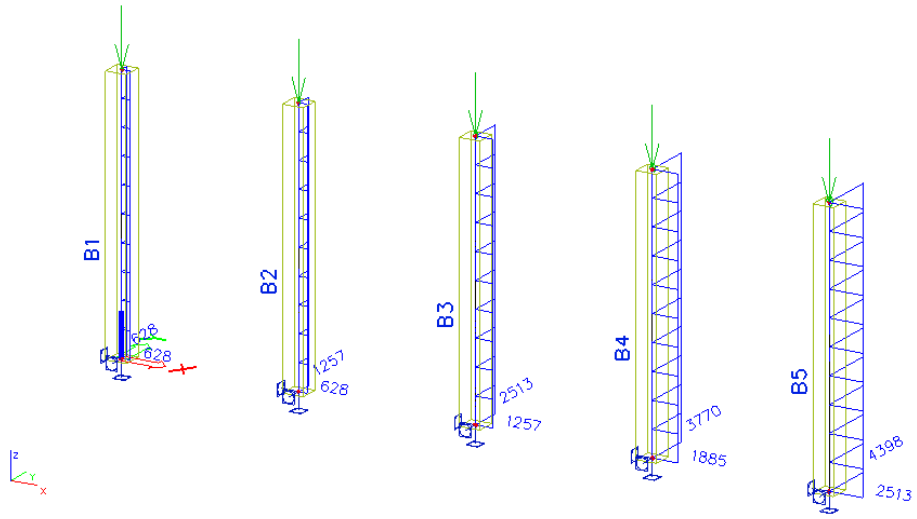


N_d = 1050,00 kN
M_{yd} = 128,45 kNm
M_{zd} = 216,25 kNm

// y 8+2 = 10 bars
// z 10*1,59 = 15,90 ~ 16 bars

Reinforcement configuration: (10/16)

The detailed results can be found in the Concrete menu > Member design - Design section, when you ask the value for As,total req and open the action Preview at the bottom of the Properties menu:



Design As EN 1992-1-1

Linear calculation, Extreme : Member
Selection : All
Load cases : LC1

Main reinforcement for selected columns

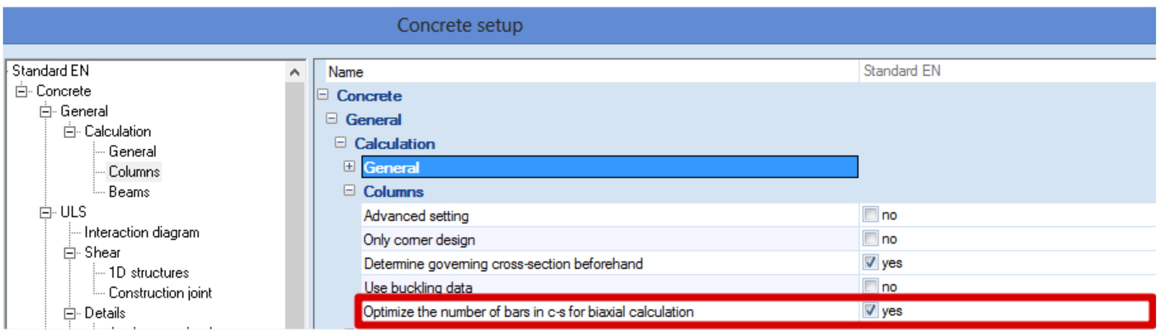
Member	d_x [m]	Case	N_d [kN]	M_{yd} [kNm]	M_{zd} [kNm]	Calc. type	Interaction check [%]	Ratio y/z [%]	$A_{s,req}$ [mm²]	$Reinf_{req}$	$Reinf_{tot}$
B1	0.000	LC1	-1050.00	-51.23	-86.25	B	81 < 100	50/50	1257	4(4/4)x20.0	4x20.0(1257)
B2	0.000	LC1	-1050.00	-63.11	-106.25	B	89 < 100	33/67	1885	6(4/6)x20.0	6x20.0(1885)
B3	0.000	LC1	-1050.00	-86.86	-146.25	B	88 < 100	33/67	3770	12(6/10)x20.0	12x20.0(3770)
B4	0.000	LC1	-1050.00	-98.74	-166.25	B	75 < 100	33/67	5655	18(8/14)x20.0	18x20.0(5655)
B5	0.000	LC1	-1050.00	-128.45	-216.25	B	86 < 100	36/64	6912	22(10/16)x20.0	22x20.0(6912)

For the final load configuration, represented by column B5, the interaction check is 86% < 100%.
As the requirement for the interaction check is met, the iteration process is stopped and the solution for the reinforcement configuration is (10/16).

Optimization

There might be a more optimal solution, with an interaction check even closer to 100%. A specific option has been implemented to make SCIA Engineer look for such a solution:

Go to Concrete Setup > General: Calculation > Tab Columns and activate the option ‘Optimize the number of bars in cross-section for bi-axial calculation’:



Return now to Member design - Design, and open the Preview again:

Design As EN 1992-1-1

Linear calculation, Extreme : Member
Selection : B5
Load cases : LC1

Main reinforcement for selected columns

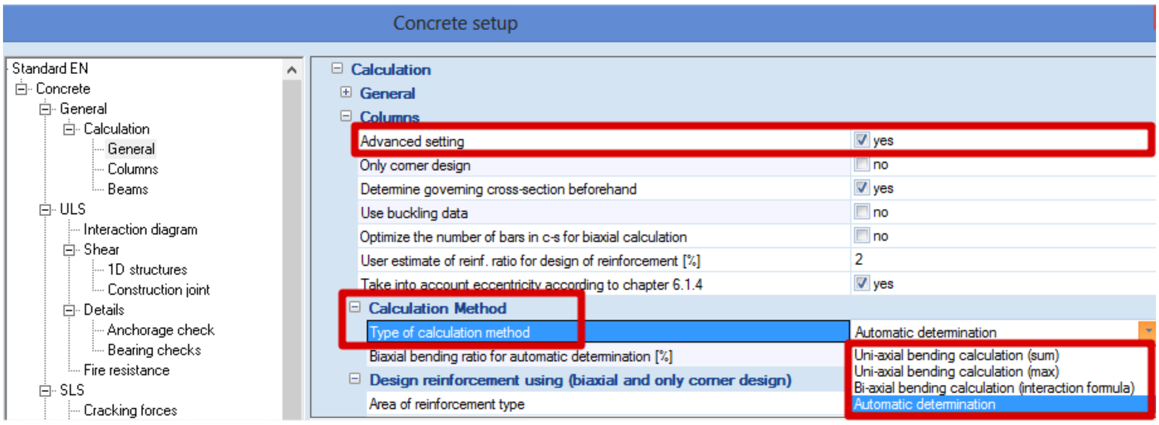
Member	d_x [m]	Case	N_d [kN]	M_{yd} [kNm]	M_{zd} [kNm]	Calc. type	Interaction check [%]	Ratio y/z [%]	$A_{s,req}$ [mm^2]	$Reinf_{req}$	$Reinf_{tot}$
B5	0.000	LC1	-1050.00	-128.45	-216.25	B	94 < 100	40/60	6283	20(10/14)x20.0	20x20.0(6283)

We can conclude that indeed a more optimal solution is found, where the reinforcement configuration is only (10/14) instead of (10/16), and the interaction check is now 94% instead of 86%.

Calculation methods: Overview – Automatic design

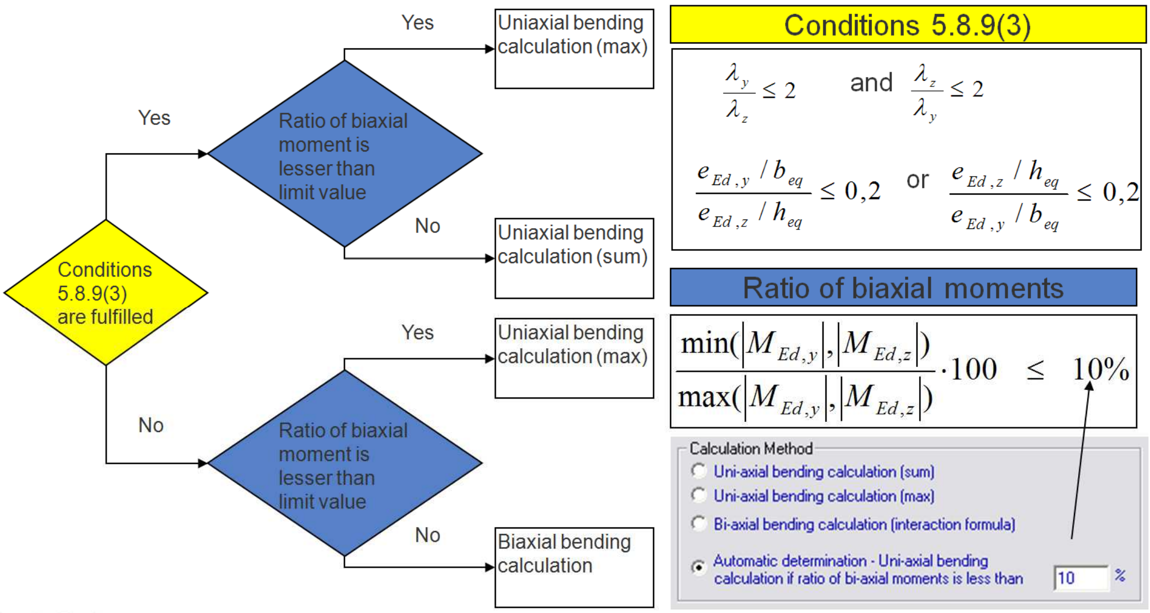
Concrete Setup > General: Calculation > Tab Column:

After selecting the option 'Advanced setting', the option Calculation Method becomes available. The user can enforce now 1 of the 3 calculation methods, if desired. By default the option is put to 'Automatic determination'. In that case, SCIA Engineer automatically determines which type of calculation fits best for each column in the structure:



- First is checked if the conditions according to *EC art 5.8.9(3)* are fulfilled.
- Secondly, a ratio check of the bending moments in the different directions is carried out. The check value of 10% forms (by default) the boundary between uni-axial and bi-axial bending calculation.

The Automatic determination in SCIA Engineer can be summarized in a flow chart as follows:



Circular columns

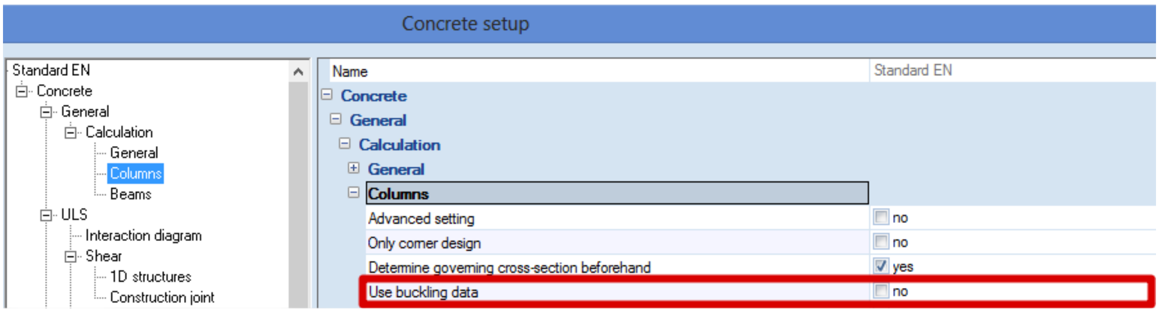
SCIA Engineer is able to calculate reinforcement not only for rectangular but also for circular columns. The calculation method for circular columns is the simplest one, because the required number of reinforcement bars is spread equally along the face of the column.

Example

Example: open the example 'Circular column.esa'

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

Concrete Setup
Item General: Calculation > Tab Columns: 'Buckling data' are not taken into account (only 1^{st} order moments are considered).



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

Results
Go to Concrete menu > Member design - Design, and open the action Preview at the bottom of the Properties menu:

Design As EN 1992-1-1

Linear calculation, Extreme : Member
Selection : All
Class : Alle UGT
Main reinforcement for selected columns

Member	d_x [m]	Case	N_d [kN]	M_{yd} [kNm]	M_{zd} [kNm]	Calc. type	$A_{s,req}$ [mm²]	$Reinf_{req}$	$Reinf_{tot}$
B1	0.000	CO1/1	-2175	-143	0	C	1571	5x20.0	5x20.0(1571)

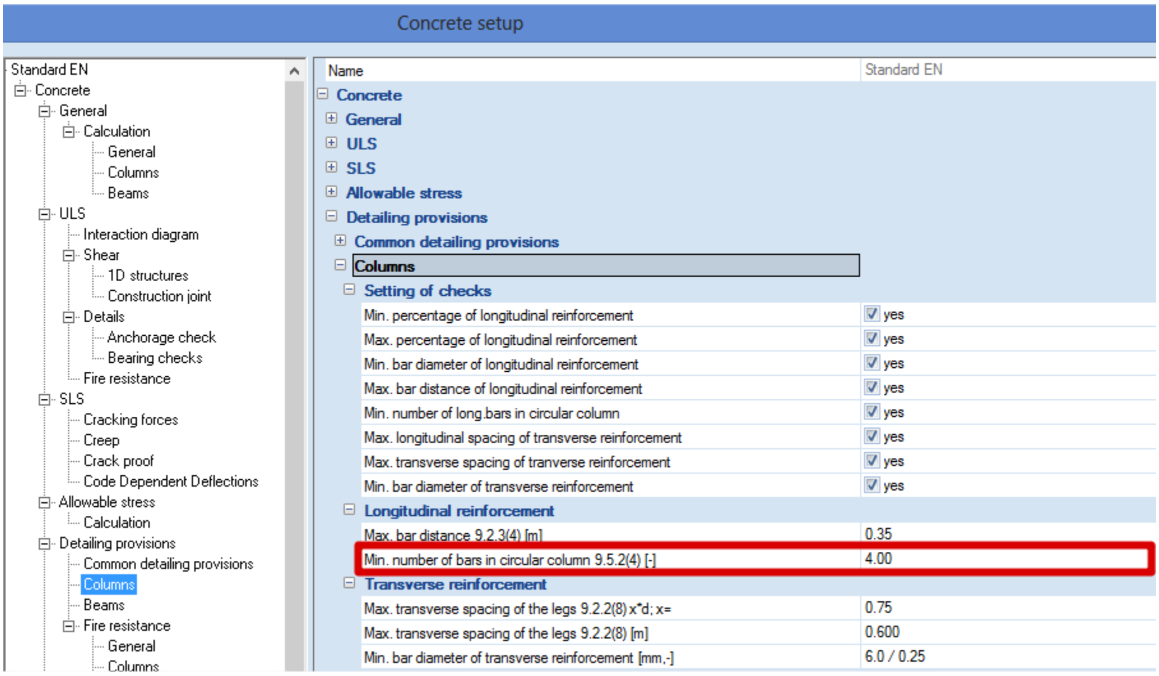
Calculation type = C (Circular column)

As,req = 1571 mm², which equals exactly 5 bars of ∅20mm (5*314mm²).

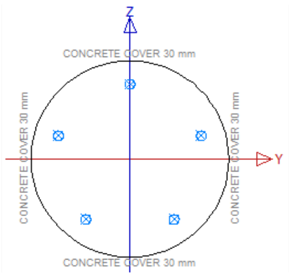
The bar diameter is taken from the Concrete Setup > Design defaults > Tab Columns, or from Member data if applied. The bar diameter is set to 20mm in the concrete setup.

Remark that SCIA Engineer uses the real area of the bars to calculate the required reinforcement area.

The minimum number of bars in a circular column is also taken from the Concrete Setup > Detailing provisions. This is ‘4’ by default, according to *EC art 9.5.2(4)*.



When going to Concrete menu > Member design - Design > action Single Check, the proposed configuration can be found:



When the value for the variable point load is increased to 1250 kN, and for the variable moment to 100 kNm (so that N_d = 2550,00 kN; M_{yd} = 217,50 kNm; M_{zd} = 0 kNm), the results are as follows:

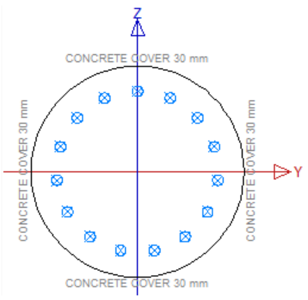
Design As EN 1992-1-1

Linear calculation, Extreme : Member
Selection : All
Class : Alle UGT

Main reinforcement for selected columns

Member	d_x [m]	Case	N_d [kN]	M_{yd} [kNm]	M_{zd} [kNm]	Calc. type	$A_{s,req}$ [mm^2]	$Reinf_{req}$	$Reinf_{tot}$
B1	0.000	CO1/1	-2550	-218	0	C	4712	15x20.0	15x20.0(4712)

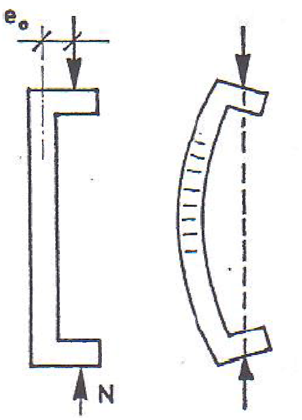
The corresponding bar configuration is the following:



2^{nd} order calculation

2^{nd} order effects:

- Caused by structural deformations
- Important for walls, columns, piles, ...



Eurocode

According to the Eurocode, 2^{nd} order effects have to be taken into account in each direction, unless they may be ignored according to *art 5.8.2 (6)* or *art 5.8.3*.

General (art 5.8.2 (6))

Second order effects may be ignored if they are less than 10 % of the corresponding first order effects.

Simplified criteria for second order effects (art 5.8.3)

Slenderness criterion for isolated members (art 5.8.3.1)

As an alternative to *5.8.2 (6)*, second order effects may be ignored if the slenderness λ is below a certain value λ_{lim}.

The recommended value for λ_{lim} follows from:

$$\lambda_{lim} = 20.A.B.C/\sqrt{n} \qquad (5.13N)$$

where:

A = $1 / (1+0,2\ \varphi_{ef})$ (if φ_{ef} is not known, A = 0,7 may be used)
B = $\sqrt{1+\ 2\omega}$ (if ω is not known, B = 1,1 may be used)
C = $1,7 - r_m$ (if r_m is not known, C = 0,7 may be used)
φ_{ef} effective creep ratio; see 5.8.4;
ω = $A_s f_{yd} / (A_c f_{cd})$; mechanical reinforcement ratio;
A_s is the total area of longitudinal reinforcement
n = $N_{Ed} / (A_c f_{cd})$; relative normal force
r_m = M_{01}/M_{02}; moment ratio
M_{01}, M_{02} are the first order end moments, $|M_{02}| \geq |M_{01}|$

In cases with biaxial bending, the slenderness criterion may be checked separately for each direction. Depending on the outcome of this check, second order effects (a) may be ignored in both directions, (b) should be taken into account in one direction, or (c) should be taken into account in both directions.

Slenderness and effective length of isolated members (art 5.8.3.2)

The slenderness ratio λ is defined as follows:

$$\lambda = l_0 / i \qquad (5.14)$$

where:

l_0 is the effective length

i is the radius of gyration of the uncracked concrete section

Examples of effective length for isolated members with constant cross section are given:

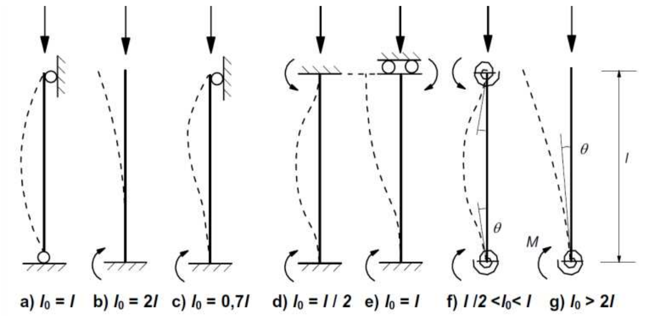


Figure 5.7: Examples of different buckling modes and corresponding effective lengths for isolated members

Geometric imperfections (art 5.2)

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

For isolated members, the effect of imperfections may be taken into account in two alternative ways:

a) as an eccentricity, e_i, given by:

$$e_i = \theta_i \, l_0 / 2 \qquad (5.2)$$

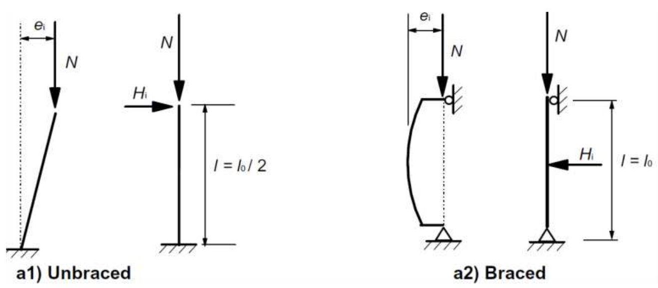
where l_0 is the effective length.

For walls and isolated columns in braced systems, $e_i = l_0/400$ may always be used as a simplification, corresponding to $\alpha_h = 1$.

b) as a transverse force, H_i, in the position that gives the maximum moment:

- for unbraced members:
  $H_i = \theta_i N$ (5.3a)
- for braced members:
  $H_i = 2 \theta_i N$ (5.3b)

where N is the axial load.



According to *EC art 5.8.9 (2)*, imperfections need to be taken into account only in the direction where they will have the most unfavourable effect.

Method based on nominal curvature (art 5.8.8)

Several methods are described by the Eurocode to take into account 2nd order effects. The one implemented in SCIA Engineer is the method based on the nominal curvature.

In fact this is an approximation of a genuine 2nd order calculation: only 1st order moments are calculated, and then increased by 2nd order moments which are calculated based on the 1st order internal forces.

General (art 5.8.8.1)

This method is primarily suitable for isolated members with constant normal force and a defined effective length l_0. The method gives a nominal 2nd order moment based on a deflection, which in turn is based on the effective length and an estimated maximum curvature.

The resulting design moment is used for the design of cross sections with respect to bending moments and axial force.

Bending moments (art 5.8.8.2)

The design moment is:

$$M_{Ed} = M_{0Ed} + M_2 \qquad (5.31)$$

where:

M_{0Ed} is the 1st order moment, including the effect of imperfections, see (1)
M_2 is the nominal 2nd order moment, see (2)

The maximum value of M_{Ed} is given by the distributions of M_{0Ed} and M_2; the latter may be taken as parabolic or sinusoidal over the effective length.

(1) Differing first order end moments M_{01} and M_{02} may be replaced by an equivalent first order end moment M_{0e}:

$$M_{0e} = 0,6\ M_{02} + 0,4\ M_{01} \geq 0,4\ M_{02} \qquad (5.32)$$

M_{01} and M_{02} should have the same sign if they give tension on the same side, otherwise opposite signs. Furthermore, $|M_{02}| \geq |M_{01}|$.

(2) The nominal second order moment M_2 is:

$$M_2 = N_{Ed} * e_2 \qquad (5.33)$$

where:

N_{Ed} is the design value of axial force
e_2 is the deflection = $(1/r)\ l_0^2/\ c$
$1/r$ is the curvature
l_o is the effective length
c is a factor depending on the curvature distribution

For constant cross section, $c = 10$ ($\approx\pi^2$) is normally used. If the first order moment is constant, a lower value should be considered (8 is a lower limit, corresponding to constant total moment).

Curvature (art 5.8.8.3)

The curvature *1/r* can be calculated as follows for members with constant symmetrical cross sections (incl. reinforcement):

$$1/r = K_r \, K_\varphi \, 1/r_0 \qquad (5.34)$$

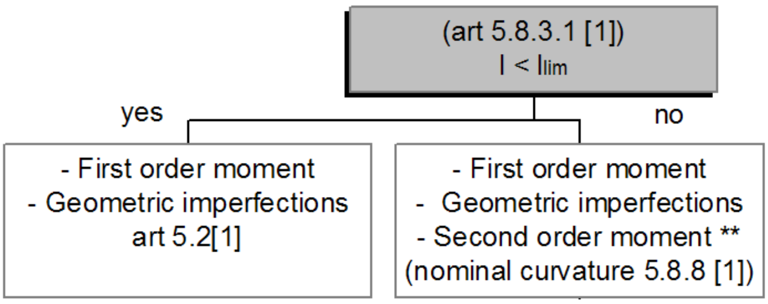
where:

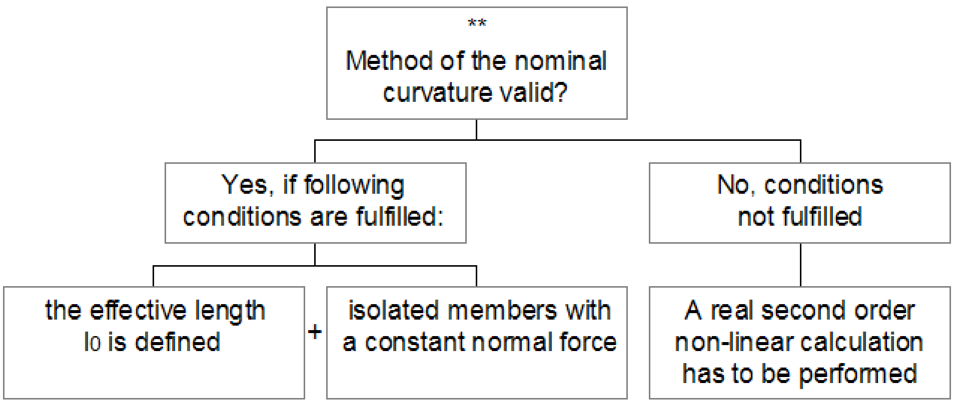
K_r is a correction factor depending on axial load
K_φ is a factor for taking account of creep
$1/r_0$ $= \varepsilon_{yd} / (0{,}45\ d)$
ε_{yd} $= f_{yd} / E_s$
d is the effective depth

Overview

All of this can be summarized as follows:

1) Check if $\lambda < \lambda_{lim}$.
2) If 2nd order effects have to be taken into account: Check if the method based on the nominal curvature is valid, so a nominal 2nd order calculation can be carried out. If not, a real non-linear 2nd order calculation has to be performed.

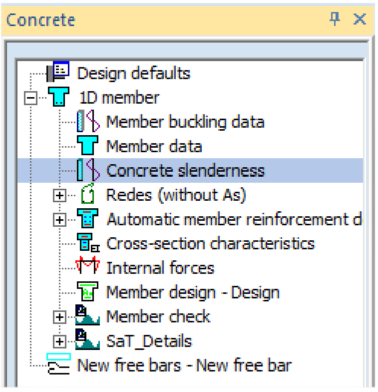


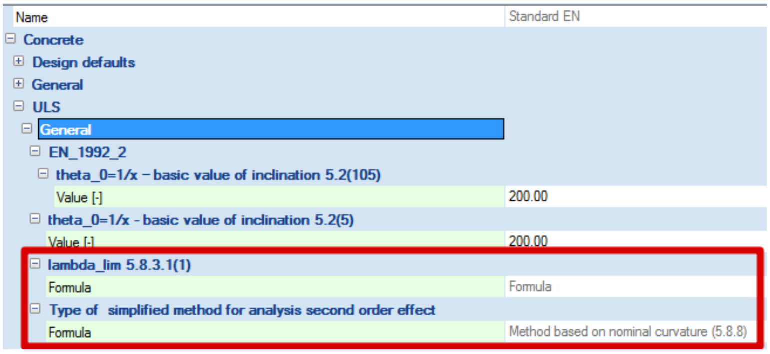


SCIA Engineer

Slenderness

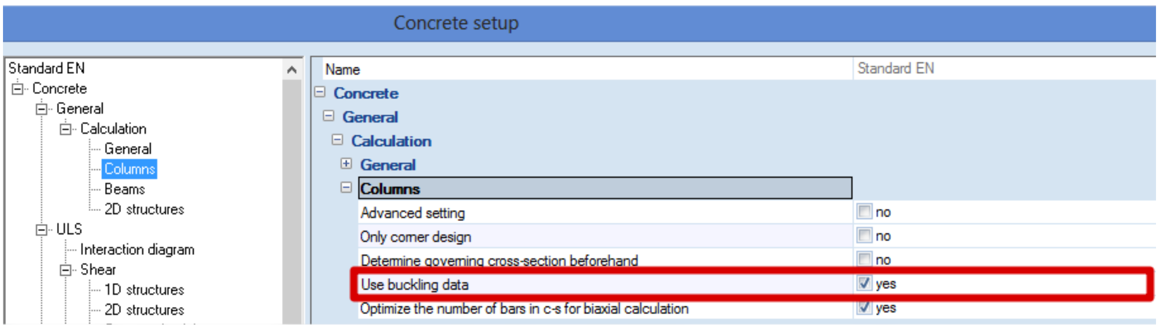
If $\lambda > \lambda_{lim}$, 2nd order effects have to be taken into account. The values for λ and λ_{lim}, and the corresponding check, can be found in the Concrete menu > Concrete slenderness.





Buckling data

In the Concrete Setup > General: Calculation > Tab columns, the option 'Use buckling data' can be found. By default this option is activated:



It is important to know that:

- When this option is not activated, only the 1st order bending moments are taken into account.
- When this option is activated, also the moments caused by geometrical imperfections and the nominal 2nd order moments are taken into account. This means the eccentricities e_i and e_2 have to be calculated supplementary, and in this way the problem of buckling is also taken care of (e_2 depends of the buckling length l_0):

$$e_1 = \frac{M_d}{N_d}$$

$$e_i = \frac{\theta_i . l_0}{2}$$

$$e_2 = \frac{1}{r} \cdot \frac{l_0^2}{c}$$

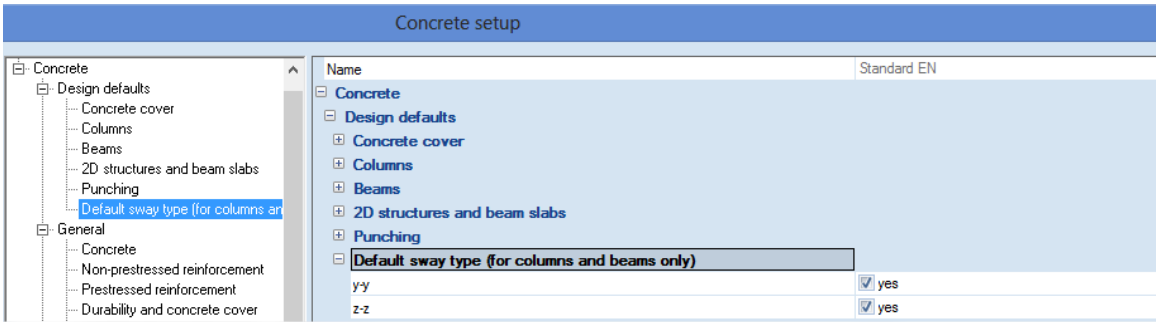
(Where M_d, N_d are the values from 1st order calculation.)

Note that if no second order moment is required according to art 5.8.3.1, only the moment caused by geometrical imperfections is taken into account and not the second order moment.

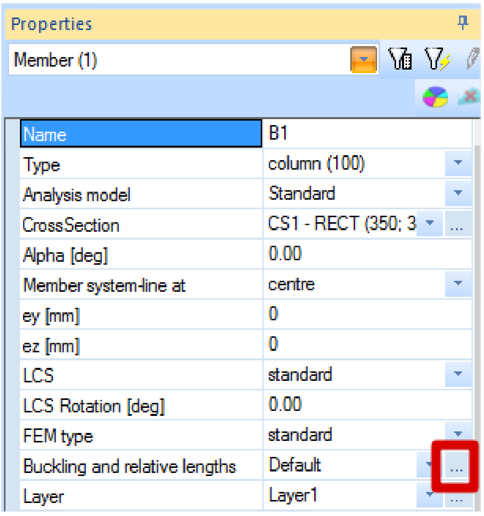
Effective length

The effective length (= buckling length) is calculated by default by SCIA Engineer.
Attention: this is only valid for simple structures!

In Concrete Setup > Design defaults, can be found that for buckling around the local y axis (Y-Y) ánd for buckling around the local z axis (Z-Z), the structure is by default considered as 'sway':



Of course, the user can assign specific buckling data to each member separately. To do so, select a member and click in the Properties menu on the [...] button behind 'Buckling and relative lengths'. Here you can choose to input your own buckling factor (beta) or length (l).



The effective length based on these buckling data is required to calculate e_2 and the corresponding moment M_2.

Recalculated internal forces

In the Concrete menu > Internal forces, the following results can be asked for:

- The values for Mx, My, Mz, which are the 1st order design moments.
- The values for Mx,recalc , My,recalc , Mz,recalc , which are the recalculated design moments where the moments caused by geometrical imperfections and the nominal 2nd order moments are taken into account ($M_{Ed} = M_{0Ed} + M_2$).

Reinforcement design

In the Concrete menu > Member design - Design, the values for As,req are calculated based on the recalculated design values of the internal forces (My,recalc and Mz,recalc).

Example

Example: open the 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.

Loads

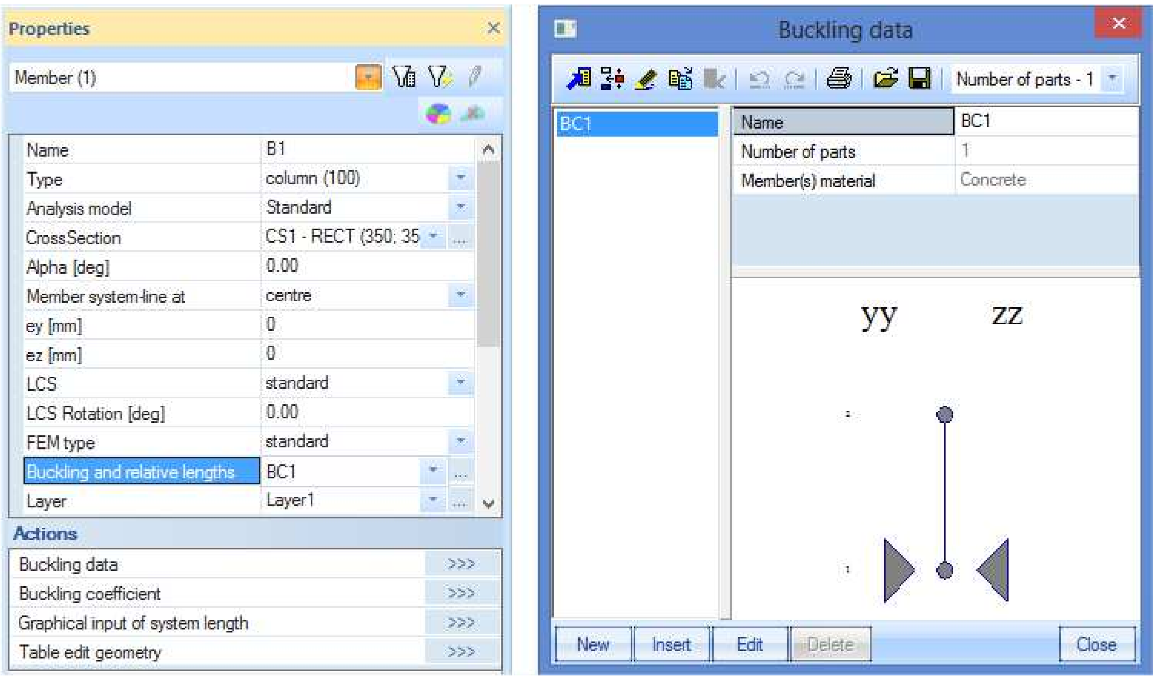
Load configuration: N_d = 405,00 kN
M_{yd} = 40,50 kNm
M_{zd} = 0 kNm

Buckling data
Note that the option ‘buckling data’ is by default activated. This means that for reinforcement design automatically the geometrical imperfections are taken into account.

Since geometrical imperfections are taken into account, no extra buckling check is required.

The calculation of geometrical imperfections is based on the buckling length l_0.

To view or edit the basic settings for calculation of buckling length l_0, the user can access the buckling properties of a member through the Properties > Buckling and relative lengths:



Slenderness criterion

Check if second order calculation is required following art 5.8.3.1:

Concrete slenderness

Concrete slenderness
Linear calculation, Extreme : No
Selection : All
Load cases : LC1

Member	CS Name	Part	$Sway_y$	l_y [m]	β_y [-]	$l_{0,y}$ [m]	i_y [mm]	λ_y [-]	$\lambda_{lim,y}$ [-]	$Check_{calc}$ [-]	Check
			$Sway_z$	l_z [m]	β_z [-]	$l_{0,z}$ [m]	i_z [mm]	λ_z [-]	$\lambda_{lim,z}$ [-]	$Check_{lim}$ [-]	
B1	CS1	1	Yes	4.500	2.00	9.011	101	89.18	46.55	1.92	Not OK
			Yes	4.500	2.00	9.011	101	89.18	46.55	1.00	

Since λ is greater than λ_{lim}, a second order calculation will be required.

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

Internal forces

Basic:

Internal forces

Member	d_x [m]	Case	N [kN]	M_y [kNm]	M_z [kNm]	N_{rec} [kN]	$M_{y\ rec}$ [kNm]	$M_{z\ rec}$ [kNm]
B1	0.000	ULS/1	**-405.00**	-40.50	0.00	-405.00	-114.85	8.60

Recalculated internal forces taking into account geometrical and second order imperfection (both are automatically calculated based on l_0 if of course the option ‘buckling data’ is not manually de-activated):

Results of recalculated bending moment My,recal

Member	d_x [m]	Case	Type of check	N [kN]	M_y [kNm]	$M_{y,recal}$ [kNm]	$M_{0Ed,y}$ [kNm]	$M_{2,y}$ [kNm]	$e_{tot,z}$ [mm]	$e_{0,z}$ [mm]	$e_{2,z}$ [mm]	$e_{i,z}$ [mm]
B1	0.000	ULS/1	Design ULS	-405.00	-40.50	-114.85	-49.10	-65.75	-284	-121	-162	-21

Results of recalculated bending moment Mz,recal

Member	d_x [m]	Case	Type of check	N [kN]	M_z [kNm]	$M_{z,recal}$ [kNm]	$M_{0Ed,z}$ [kNm]	$e_{tot,y}$ [mm]	$e_{0,y}$ [mm]	$e_{i,y}$ [mm]
B1	0.000	ULS/1	Design ULS	-405.00	0.00	8.60	8.60	21	21	21

Results

The results for the reinforcement design are shown below:

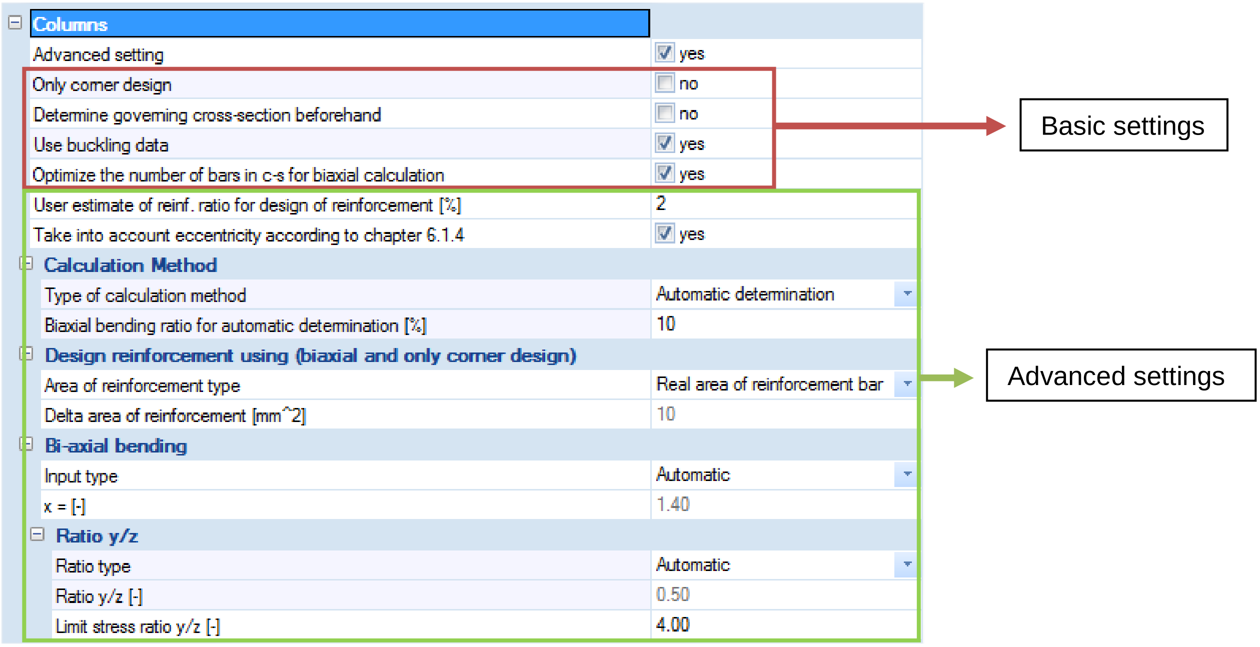
Main reinforcement for selected columns

Member	d_x [m]	Case	N_d [kN]	M_{yd} [kNm]	M_{zd} [kNm]	Calc. type	Ratio y/z [%]	$A_{s,req}$ [mm²]	$Reinf_{req}$	$Reinf_{tot}$	W/E
B1	0.000	ULS/1	-405.00	114.85	8.60	Um	100/0	918	4(4/4)x20.0	4x20.0(1257)	270

Note that uniaxial bending (max) is the calculation type because the ratio of biaxial moment is lesser than the limit value of 10%.

Concrete Setup: Overview

Concrete Setup > General: Calculation > Tab Columns:



Basic Settings

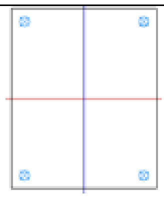
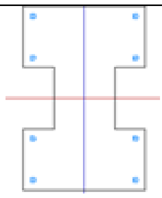
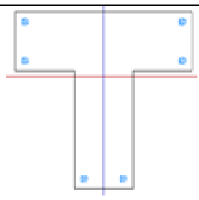
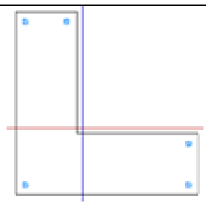
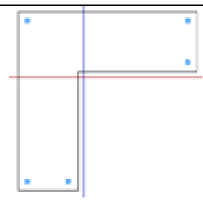
Only corner design

This is a special type of calculation, where the reinforcement is designed only in the corners of the cross-section with an internal angle of 90°. It is an iterative calculation, where the number of bars stays the same, but the diameter or area of reinforcement is increased until the configuration satisfies the ULS checks.

The way of increasing the diameter depends on the setting of the option ‘Design reinforcement by using real area / delta area’ (discussed further in this paragraph):

The position of the corner bars is calculated in both cases from parameters (such as concrete cover) defined in the Concrete Setup or in the Member data, and the position is not changed during the iteration calculation.

Only the basic concrete sections are supported for this calculation (see Libraries > Cross-sections > group Concrete: rectangular, I -, T-, L-sections). The number of bars depends on the shape of the cross-section:

Section	Rectangular section	I section	T section	L section with lower flange	L section with upper flange
n_s	4	8	6	5	5
Shape					

Determine governing cross-section beforehand
When this option is on, the design of the reinforcement is only carried out in the foot and head of the column. When this option is off, the design is carried out in all intermediate sections as well, which may be time consuming and brings no profit in terms of accuracy.
The subsequent check of the designed reinforcement is in both cases performed in all sections.

Remark: the option 'Determine governing cross-section beforehand' is not taken into account in the action Single Check!

Use buckling data
When this option is off, only the 1st order bending moments are taken into account.
When this option is on, also the moments caused by geometrical imperfections and the nominal 2nd order moments are taken into account. This means the eccentricities e_i and e_2 have to be calculated supplementary, and in this way the problem of buckling is also taken care of (e_2 depends of the buckling length l_0).

In fact it is an approximation (described by the Eurocode) of a genuine 2nd order calculation: only 1st order moments are calculated, and then increased by 2nd order moments which are calculated based on the 1st order moments.

Optimize the number of bars in cross-section for biaxial calculation
If this option is on, the number of bars in the cross-section is optimised to achieve the minimum number of bars with the cross-section still passing the ULS checks.

This option only has influence on the reinforcement of members where a bi-axial calculation is performed. In fact, SCIA Engineer looks for the solution where the value for the interaction formula is as close to 1 as possible.

Advanced Settings

These are only available when the option 'Advanced setting' is selected.

User estimate of reinforcement ratio for design of reinforcement [%]
This estimated value for the reinforcement ratio is used to calculate the nominal curvature 1/r according to *EC art 5.8.8.3*, where the value for As has to be filled in. The value for As is then taken as the 'reinforcement ratio * Ac'. Since the nominal curvature is only needed for the calculation of second order effects, this ratio is only calculated for members with "Buckling data" on.

The default value for the ratio is 2%. If the "percentage of additional (required) reinforcement is less than 80% of user estimate ratio", then SCIA Engineer gives warning 270. If the "percentage of additional (required) reinforcement is greater than 120% of user estimate ratio", then SCIA Engineer gives warning 271.

Take into account eccentricity according to chapter 6.1.4
If this option is on, the first order eccentricity including imperfections (e_0) in both directions has to be the maximum of (h/30; 20 mm), where h is the depth of the section.

This option is implemented according to the *EC art 6.1(4)*.

Calculation method
The user has the choice between 3 calculation methods: Uni-axial bending calculation (sum) or (max) and Bi-axial bending calculation.

By default the option 'Automatic determination' is selected; SCIA Engineer follows a certain flow chart to determine which calculation is best suited (See paragraph Bending – Overview for the flow chart).

Design reinforcement by using real area / delta area
The choice made here is only taken into account for bi-axial bending calculation and corner design only, which are both iterative calculations. It defines the way in which the required reinforcement area is increased in each iteration step.

Real area of reinforcement bar

For bi-axial bending calculation
If this option is on, the value for As,req is calculated based on the real sectional area of an individual reinforcement bar. This means that the value for As,req is always a multiple of the area of the used reinforcement bar:

Design As EN 1992-1-1

Linear calculation, Extreme : Member
Selection : B5
Load cases : LC1
Main reinforcement for selected columns

Member	d_x [m]	Case	N_d [kN]	M_{yd} [kNm]	M_{zd} [kNm]	Calc. type	Interaction check [%]	Ratio y/z [%]	$A_{s,req}$ [mm^2]	$Reinf_{req}$	$Reinf_{tot}$
B5	0.000	LC1	-1050.00	-128.45	-216.25	B	94 < 100	40/60	6283	20(10/14)x20.0	20x20.0(6283)

For corner design only
The diameter of the corner bars is increased in each iteration step according to a list of basic diameters, which is for the Eurocode: 5-6-8-10-12-14-16-20-25-28-32-40 mm. As initial value for the bar diameter, the default value from the Concrete Setup or from the Member data (if applied) is taken. If the last diameter from the list doesn't satisfy yet, the calculation is ended with error 508.

Delta area of reinforcement

For bi-axial bending calculation
If this option is on, the value for As,req is calculated based on the user defined delta area. This means that the value for As,req is always a multiple of the value inputted by the user:

Design As EN 1992-1-1

Linear calculation, Extreme : Member
Selection : B5
Load cases : LC1
Main reinforcement for selected columns

Member	d_x [m]	Case	N_d [kN]	M_{yd} [kNm]	M_{zd} [kNm]	Calc. type	Interaction check [%]	Ratio y/z [%]	$A_{s,req}$ [mm^2]	$Reinf_{req}$	$Reinf_{tot}$
B5	0.000	LC1	-1050.00	-128.45	-216.25	B	86 < 100	37/63	6240	22(10/16)x20.0	22x20.0(6912)

In this case a value for delta area of 20mm² has been inputted:

Design reinforcement using (biaxial and only corne...	
Area of reinforcement type	Delta area of reinforcement
Delta area of reinforcement [mm^2]	20

For corner design only
The value for delta area represents the increasing area per reinforcement bar in each iteration step. The 'real' diameter of reinforcement bars ds_i after each iteration step i is calculated according to the formula below and is rounded up to an integer:

$$ds_i = \sqrt{\frac{4 \cdot \Delta A \cdot i}{\pi}}$$

Bi-axial bending – Safety factor

The value for the safety factor x used in the interaction formula can be determined automatically or by user input.

When the option ‘Automatic’ is chosen, SCIA Engineer calculates the factor x by means of the following rules from the Eurocode:

x = 2 for circular and elliptical cross-sections
x = the value ‘a’ according to the following table for rectangular cross-sections (with linear interpolation for intermediate values)

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

Bi-axial bending – Ratio y/z

Automatic

SCIA Engineer calculates the ratio y/z automatically, after the design has been done.

Manual

The user can impose a ratio y/z, so the reinforcement configuration will be adapted to meet this requirement.

From user reinforcement

The ratio of the reinforcement in y and z direction is calculated from existing user reinforcement:

$$\text{ratio } \frac{y}{z} = \left(\frac{n_y}{n_y + n_z}\right) \Big/ \left(\frac{n_z}{n_y + n_z}\right)$$

where: n_y = number of bars in the y direction
n_z = number of bars in the z direction

If no user reinforcement is defined, then the ratio y/z = Automatic.

If the user reinforcement is unsymmetrical about y or z axis, then error 879 will appear: “The calculation is not supported because of unsymmetrical user reinforcement”.

Bi-axial bending – Limit stress ratio y/z

This is the limit value for the ratio y/z of additional (or required) reinforcement for a bi-axial bending calculation.

The default value for $(\text{ratio y/z})_{lim}$ is 4. If ratio y/z > $(\text{ratio y/z})_{lim}$ or ratio y/z < $1/(\text{ratio y/z})_{lim}$, then warning 245 will appear: “An unusual design situation is encountered: the stress ratio y/z exceeds the preset limit or the required reinforcement is hardly acceptable.”

This is due to too many bars in 1 direction, and the solution is to:

- use a different Calculation Method
- use a different Ratio y/z
- switch on the option ‘Optimize the number of bars’ (best solution)

Theoretical background

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