



ADVANCED CONCEPT TRAINING Concrete

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Chapter 1: Materials

1.1. Verification by the partial factor method

Cf 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	$\gamma_{\rm C}$ for concrete	$\gamma_{\rm S}$ for reinforcing steel	$\gamma_{\rm S}$ for prestressing steel
Persistent & Transient	1,5	1,15	1,15
Accidental	1,2	1,0	1,0

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



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

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

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

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

1.2. Concrete

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

1.2.1. 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:

I Materials		×
et -1 🖸 17 P 🖬	🖌 🐟 🗖 😤 🕞 All 🔹 🗸	7
C12/15	Name	C50/60
C16/20	Code independent	
C20/25	FN 1992-1-1	
C25/30	Characteristic compressive sulinder strength fck/28) [MDa]	50.00
C30/37	Characteristic compressive cylinder strength rck(26) [inra]	
C35/45	Calculated depended values	<u></u>
C40/50	Mean compressive strength fcm(28) [MPa]	58.00
C45/55	fcm(28) - fck(28) [MPa]	8.00
C50/60	Mean tensile strength fctm(28) [MPa]	4.10
C55/67	fctk 0,05(28) [MPa]	2.90
C60/75	fctk 0.95(28) [MPa]	5.30
C70/85	Decise composition december associated (find = fold / composition of MDa)	23.23
C80/95	Design compressive su engur - persistent (red - rek / gamma c_p) [wra]	44.67
C90/105	Design compressive strength - accidental (fcd = fck / gamma c_a) [MPa]	41.07
C6/8 (British BS-E	Strain at reaching maximum strength eps c2 [1e-4]	20.0
C8/10 (British BS-E	Ultimate strain eps cu2 [1e-4]	35.0
C28/35 (British BS	Strain at reaching maximum strength eps c3 [1e-4]	17.5
C28/35 (Irish I.S-E	Ultimate strain eps cu3 [1e-4]	35.0
C28/35 (Dutch NE	Stone diameter (dg) [mm]	32
C32/40 (British BS	Compare diameter (og) (mini	N (normal bardening - CEM 32 Ly
C100/115 (Germa	Cement class	Contracting CENTS2, T
C12/15(EN1992-2)	Type of aggregate	Quartzite *
C16/20(EN1992-2)	Measured values	
C20/25(EN1992-2)	Measured values of mean compressive strength (influence of ageing)	
C25/30(EN1992-2)	Stress-strain diagram	
C30/37(EN1992-2)		
New Insert Ed	lit Delete	Close



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:

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

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:

$$f_{cm}(t) = \beta_{cc}(t) f_{cm}$$
(3.1)
with $\beta_{cc}(t) = e^{\left\{s\left[1 - \left(\frac{28}{t}\right)^{\frac{1}{2}}\right]\right\}}$
(3.2)

where:

- fcm(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:

Materials		×
e -1 C = + 8		r
C12/15	Name	C30/37
C16/20	Code independent	
C20/25	Material type	Concrete
C25/30	Thermal expansion [m/mK]	0.01e-003
C30/37	inematespersion (nymy)	3500.00
C35/45	Onic mass [kg/m-5]	3500.00
C40/50	Density in tresh state [kg/m*3]	2600.00
C45/55	E modulus [MPa]	3.2800e+04
C50/60	Poisson coeff.	0.2
C55/67	Independent G modulus	
000/15	G modulus [MPa]	1.3667e+04
CIU/00	Log. decrement (non-uniform damping only)	0.2
C00/105	Colour	
CE/R (Rolling RS-EN NA)	Specific heat [1/ek]	6.0000e-01
CE/10 (British BS-FN	There all conductivity DV/mK1	4 5000e+01
C28/35 (British BS-EN	inernal conductivity (w/mk)	
C28/35 (Irish I.S-EN NA)	Order in code	
C28/35 (Dutch NEN-E	Price per unit [€/m^3]	1.00
C32/40 (British BS-EN	* EN 1992-1-1	
C100/115 (German Dl	Characteristic compressive cylinder strength fck(28) (MPa)	30.00
C12/15(EN1992-2)	Calculated depended values	
C16/20(EN1992-2)	Mean compressive strength fcm(28) [MPa]	38.00
C20/25(EN1992-2)	fcm(28) - fck(28) [MPa]	8.00
C25/30(EN1992-2)	Mean tensile strength fctm(28) [MPa]	2.90
C30/37(EN1992-2)	Erik p. ps/pa) [MPa]	2.00
C35/45(EN1992-2)	List o prime tato a	1.00
C40/50(EN1992-2)	ictk 0/ap(#a) [wira]	20.00
C45/55(EN1992-2)	Design compressive strength - persistent (fcd = fck / gamma c_p) [MPa]	20.00
C50/60(EN1992-2)	Design compressive strength - accidental (fcd = fck / gamma c_a) [MPa]	25.00
C55/67(EN1992-2)	Strain at reaching maximum strength eps c2 [1e-4]	20.0
C50/75(EN1992-2)	Ultimate strain eps cu2 [1e-4]	35.0
C/0/85(EN1992-2)	Strain at reaching maximum strength eps c3 [1e-4]	17.5
C80/95(EN1992-2)	Ultimate strain eps cu3 [1e-4]	35.0
R 4004	Stone diameter (de) [mm]	32
8 500Å	Camant class	N (normal hardening - CEM 32.5 R, CEM 42.5 N)
B 600A	Turn of anymetric	S (slow bardening - CEM 32.5 N)
B 400B	A Measured up has	N (normal hardening - CEM 32,5 R, CEM 42,5 N)
B SOOB	· measured values	R (rapidl hardening - CEM 42,5 R, CEM 52,5 N, CEM 52,5 R)
B 600B	Measured values of mean compressive strength (influence of ageing)	
B 400C	Stress-strain diagram	
B 500C	Type of diagram	Bi-linear stress-strain diagram
8 600C	Picture of Stress-strain diagram	
B 420B (Austrian ONO		
B 550A (Austrian ONO		
B 550B (Austrian ONO		
New Josef Edit	Delete	Cone

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:

Analytical relation / Explanation			$f_{\rm cm} = f_{\rm ok} + 8({\rm MPa})$	$f_{etm} = 0, 30 \times f_{ex}^{(23)} \le C50/60$ $f_{etm} = 2, 12 \cdot \ln(1 + (f_{em}/10))$ > C50/60	$f_{ck;0,05} = 0.7 \times f_{clm}$ 5% fractile	feeco.ce = 1,3×feam 95% fractile	$E_{\rm cm} = 22[(f_{\rm cm})'10]^{0.3}$ ($f_{\rm cm}$ in MPa)	see Figure 3.2 ₆₀₁ (⁹ / ₀₀) = 0.7 f _m ^{0.31} < 2.8	see Figure 3.2 for f _{ix} > 50 Mpa	see Figure 3.3 for f _{6k} ≥ 50 Mpa s ₆₂ (⁰ / ₄₀)=2,0+0,085(f _{4k} -50) ⁰⁵⁸	see Figure 3.3 for <i>t_{ik}</i> ≥ 50 Mpa <i>s</i> _{ouc} (¹ / ₉ m)=2,6+35[(90- <i>t_{ik})</i> /100] ⁴	for f _{ei} ≥ 50 Mpa <i>n</i> =1,4+23,4[(90- <i>f_a</i>)/100] ⁴	$\begin{array}{l} \underset{\mathcal{E}_{c_3}(^{i_{j_{0,0}}})=1}{\text{see Figure 3.4}} \\ \text{for } f_{a_i} \geq 50 \text{ Mpa} \\ \underset{\mathcal{E}_{c_3}(^{i_{j_{0,0}}})=1}{\text{,} 75+0, 55[(f_{a_i}-50)/40]} \end{array}$	see Figure 3.4 for <i>f</i> ₄ ≥ 50 Mpa <i>ɛ</i> _{cot} (²) ₀₀)=2,6+35[(90-f ₆)/100] ⁴
	90	105	<u>98</u>	5,0	3,5	6,6	44	2,8	2,8	2,6	2,6	1,4	2,3	2,6
	80	<mark>96</mark>	88	4,8	3,4	6,3	42	2,8	2,8	2,5	2,6	1,4	2,2	2,6
	70	85	78	4,6	3,2	6,0	41	2,7	2,8	2,4	2,7	1,45	2,0	2,7
	60	75	68	4,4	3,1	5,7	39	2,6	3,0	2,3	2,9	1,6	1,9	2,9
	55	67	63	4,2	3,0	5,5	38	2,5	3,2	2.2	3,1	1,75	1,8	3,1
Icrete	50	8	8	4,1	2,9	5,3	37	2,45	·>				0	2
or con	45	<mark>55</mark>	53	3,8	2,7	4,9	36	2,4						
sses f	40	8	48	3,5	2,5	4,6	35	2,3						
th clas	35	45	43	3,2	2,2	4,2	34	2,25						
streng	30	37	8	2,9	2,0	3,8	33	2,2	3,5	2,0	3,5	2,0	1,75	3,5
0)	25	8	33	2,6	1,8	3,3	31	2,1						
	20	25	28	2,2	1,5	2,9	30	2,0						
	16	20	24	1,9	1,3	2,5	29	1,9						
	12	15	20	1,6	1,1	2,0	27	1,8						
	f ₄ (MPa)	f _{ck cube} (MPa)	fan (MPa)	f _{cm} (MPa)	f _{dk, 0,05} (MPa)	f _{ck 095} (MPa)	E _{cm} (GPa)	Ect (%00)	Eart (100)	E (2 (%0)	Eu2 (%o)	и	E c3 (%0)	Eau3 (%00)

Table 3.1 Strength and deformation characteristics for concrete

1.2.2. Design compressive and tensile strengths (art 3.1.6)

The value of the design compressive strength is defined as

	$f_{cd} = \alpha_{cc} f_{ck} / \gamma_c$	(3.15)
where:		

γc 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.

Remark: the Belgian National Annex recommends the use of the value 0,85.

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

$$f_{ctd} = \alpha_{ct} f_{ctk,0,05} / \gamma_c \tag{3.16}$$

where:

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

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

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



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

The recommended value of k_t is 0,85.

1.2.3. 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%.



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

$$E_{cm}(t) = (f_{cm}(t) / f_{cm})^{0,3} E_{cm}$$
(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.

Another option to take into account cracked concrete, is to deactivate the option 'Calculate dependent values'. This allows you to set any user value for the E modulus.

1.2.4. Creep and shrinkage (art 3.1.4)

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

The value of the creep coefficient can be set in the concrete settings by using the "Code-based settings" view or in the 1D member data. If the type input of the creep coefficient is "**Auto**", the creep coefficient can be calculated automatically by inputting the age of concrete and the relative humidity (see annex B.1. in EN 1992-1-1).

If the type input of the creep coefficient is "User value", you can directly input the creep coefficient.

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Consider drying and autogenous shrinkage Type $\varepsilon_{cs}(t,t)$ Auto Auto S.1.4(6) EN 1992-1-1 All (Bea Solver se Age of concrete at the beginning of drying shrinkage t_s 7.00 7.00 day 3.1.4(6) EN 1992-1-1 All (Bea Solver se 5. Structural analysis Image: Solver se 7.00 7.00 day 3.1.4(6) EN 1992-1-1 All (Bea Solver se 5. Structural analysis Image: Solver se Image: Solver se Image: Solver se Image: Solver se Solver se 5. Structural analysis Image: Solver se 5. Structural analysis Image: Solver se 5. Structural analysis of second order effects with axial load Image: Solver se Image: Solver se Solver se 5. Structurat limit states (ULS) Image: Solver se Image: Solver se Image: Solver se Image: Solver se 6. Solver se Image: Solver se 9. S.8 Aralysis of second order effects with axia	Age of concrete at loading	to	28.00	28.00	day	3.1.4(2),B1	EN 1992-1-1	All (Bea	Solver se	
Age of concrete at the beginning of drying shrinkage ts 7.00 7.00 day 3.1.4(6),B2 EN 1992-1-1 All (Bea Solver se 5. Structural analysis 0 0 0 0 0 0 0 0 5. Structural analysis 0 <t< td=""><td>Consider drying and autogenous shrinkage</td><td>Type s_{cs}(t,ts)</td><td>Auto</td><td>Auto</td><td></td><td>3.1.4(6)</td><td>EN 1992-1-1</td><td>All (Bea</td><td>Solver se</td><td></td></t<>	Consider drying and autogenous shrinkage	Type s _{cs} (t,ts)	Auto	Auto		3.1.4(6)	EN 1992-1-1	All (Bea	Solver se	
5. Structural analysis Image: Structural analysis Ima	Age of concrete at the beginning of drying shrink	age t₅	7.00	7.00	day	3.1.4(6),B2	EN 1992-1-1	All (Bea	Solver se	
b 5.2 Geometric imperfections Image: Construction of the structure Image: Constructure	5. Structural analysis		0							1
5.3 Idealisation of the structure Image: Signal	5.2 Geometric imperfections									
5.3.2 Geometric data Image: Solution above supports Solution above supportes Solution above support supports	 5.3 Idealisation of the structure 	0								
Moment reduction above supports Image: Solution above supports Solution above s	∡ 5.3.2 Geometric data									
> 5.8 Analysis of second order effects with axial load	Moment reduction above supports					5.3.2.2 (4)	EN 1992-1-1	Beam,B	Solver se	
6. Ultimate limit states (ULS) 6.1 Bending with or without axial force 6.2 Shear	5.8 Analysis of second order effects with axial load									
b 6.1 Bending with or without axial force a 6.2 Shear	6. Ultimate limit states (ULS)									
4 6.2 Shear	6.1 Bending with or without axial force									
	4 6.2 Shear									

Note that the concrete member data is automatically added on each 1D and 2D element. It can be accessed by selecting the entity and going to the extra options below:



CMD					×
	Name	CMD1D			^
	Member	B5			
	Member type	Column		~	
Design defaults					
Reinforcement					
Minimum cover					
 Solver setting 					
▲ General					
Creep and shrink	age				
	Age of concrete at the moment considered [day]	18250.00			
	Relative humidity [%]	50		_	
	Type input of creep coefficient	Auto		*	
	Age of concrete at loading [day]	28.00		_	
	Consider drying and autogenous shrinkage	Auto		~	
	Age of concrete at the beginning of drying shrinkage [day]	7.00			
▲ SLS					
	Use effective modulus of concrete				
 Internal forces 		_			
	Isolated member				~
Actions					
			Load default values	>>>	
				_	5
			OK (Cance	

The creep coefficient \u03c8(t,t_0) may be calculated from:

 $\varphi(t,t_0) = \varphi_0 \cdot \beta_0(t,t_0)$ (B.1)

where:

φ₀ is the notional creep coefficient and may be estimated from:

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

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

$$\varphi_{\text{RH}} = 1 + \frac{1 - \text{RH}/100}{0.1 \cdot \sqrt[3]{h_0}}$$
 for $f_{\text{cm}} \le 35 \text{ MPa}$ (B.3a)

$$\varphi_{\text{RH}} = \left[1 + \frac{1 - \text{RH}/100}{0.1 \cdot \sqrt[3]{h_0}} \cdot \alpha_1\right] \cdot \alpha_2 \quad \text{for } f_{\text{cm}} > 35 \text{ MPa}$$
(B.3b)

RH is the relative humidity of the ambient environment in %

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

$$\beta(f_{\rm cm}) = \frac{16.8}{\sqrt{f_{\rm cm}}} \tag{B.4}$$

 f_{cm} is the mean compressive strength of concrete in MPa at the age of 28 days $\beta(t_0)$ is a factor to allow for the effect of concrete age at loading on the notional creep coefficient:

$$\beta(t_0) = \frac{1}{(0,1+t_0^{0.20})}$$
(B.5)

h₀ is the notional size of the member in mm where:

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

Ac is the cross-sectional area

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

β_c(t,t₀) is a coefficient to describe the development of creep with time after loading, and may be estimated using the following Expression:

$$\beta_{c}(t,t_{0}) = \left[\frac{(t-t_{0})}{(\beta_{H}+t-t_{0})}\right]^{0.3}$$
(B.7)

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

to is the age of concrete at loading in days

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

β_H is a coefficient depending on the relative humidity (RH in %) and the notional member size (h₀ in mm). It may be estimated from:

 $\beta_{\rm H} = 1.5 \left[1 + (0.012 \text{ RH})^{18}\right] h_0 + 250 \le 1500$ for $f_{\rm cm} \le 35$ (B.8a)

$$\beta_{\rm H} = 1.5 \left[1 + (0.012 \text{ RH})^{18}\right] h_0 + 250 \alpha_3 \le 1500 \alpha_3 \qquad \text{for } f_{\rm cm} \ge 35 \qquad (B.8b)$$

ana are coefficients to consider the influence of the concrete strength:

$$\alpha_{1} = \left[\frac{35}{f_{cm}}\right]^{0.7} \quad \alpha_{2} = \left[\frac{35}{f_{cm}}\right]^{0.2} \quad \alpha_{3} = \left[\frac{35}{f_{cm}}\right]^{0.5}$$
(B.8c)

Where great accuracy is not required, a value found from a figure (Figure 3.1) may be considered as the creep coefficient, provided that the concrete is not subjected to a compressive stress greater than 0,45 f_{ck} (t₀) at an age t₀, the age of concrete at the time of loading.



1.2.5. 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:



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

 $\epsilon_{\mbox{\scriptsize cu2}}$ $% \epsilon_{\mbox{\scriptsize cu2}}$ 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

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







1.3. 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.

1.3.1. **Properties (art 3.2.2)**

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

- yield strength (fyk or f0,2k)
- maximum actual yield strength (fy,max)
- tensile strength (ft)
- 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/m³.

The design value of the modulus of elasticity E_s may be assumed to be 200GPa.

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.

Product form		Bars a	nd de-coi	led rods	V	Vire Fabri	cs	Requirement or quantile value (%)
Class		A	В	с	А	В	с	
Characteristic yi or f _{0,2k} (MPa)	eld strength f _{yk}			400	to 600			5,0
Minimum value	of $k = (f_V / f_y)_k$	≥1,05	≥1,08	≥1,15 <1,35	≥1,05	≥1,08	≥1,15 <1,35	10,0
Characteristic st maximum force,	≥2,5	≥5,0	≥7,5	≥2,5	≥5,0	≥7,5	10,0	
Bendability		Ber	nd/Rebend	i test		5		
Shear strength			Minimum					
Maximum deviation from nominal mass (individual bar or wire) (%)			5,0					

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

1.3.2. 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 kfyk/ 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.



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



1.4. Durability and cover to reinforcement

1.4.1. 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:

Class designation	Description of the environment	Informative examples where exposure classes may occur
1 No risk of	corrosion or attack	1953
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

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

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/Th	aw Attack	
XF1	Moderate water saturation, without de-icing agent	Vertical concrete surfaces exposed to rain and freezing
XF2	Moderate water saturation, with de-icing agent	Vertical concrete surfaces of road structures exposed to freezing and airborne de-icing agents
XF3	High water saturation, without de-icing agents	Horizontal concrete surfaces exposed to rain and freezing
XF4	High water saturation with de-icing agents or sea water	Road and bridge decks exposed to de-icing agents Concrete surfaces exposed to direct spray containing de-icing agents and freezing Splash zone of marine structures exposed to freezing
6. Chemical	attack	
XA1	Slightly aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water
XA2	Moderately aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water
XA3	Highly aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water

In the Concrete settings, in the "Design defaults" view, you can choose the desired exposure class. All items with a blue background colour can be overwritten in the 1D member data.

Descr	iption	Symbol		Value	Default	Unit	Chapter	Code	Structure	CheckTy
11>	Q	<all></all>	P	<all> ρ</all>	<all> \wp</all>	<p< th=""><th><all> ρ</all></th><th><all> ρ</all></th><th><all> ρ</all></th><th>Design (\times</th></p<>	<all> ρ</all>	<all> ρ</all>	<all> ρ</all>	Design (\times
Desig	n defaults									
⊳ Re	einforcement									
🔺 Mi	inimum cover									
	Design working life			50.00	50.00	year	4.4.1.2(5), t	EN 1992-1-1	All (Bea	Design de
1	Risk of corrosion attack									
	Corrosion induced by carbonation			ХСЗ	XC3		4.4.1.2(5)	EN 1992-1-1	All (Bea	Design de
	Corrosion induced by chlorides Corrosion induced by chlorides from sea water Freeze / thaw attack			None	None		4.4.1.2(5)	EN 1992-1-1	All (Bea	Design de
				None	None		4.4.1.2(5)	EN 1992-1-1	All (Bea	Design de
				None	None		4.4.1.2(12)	EN 1992-1-1	All (Bea	Design de
	Chemical attack			None	None		4.4.1.2(12)	EN 1992-1-1	All (Bea	Design de
	Risk of abrasion attack			None	None		4.4.1.2(13)	EN 1992-1-1	All (Bea	Design de
Þ	Possibility of special control									
	Risk of casting on atypical surface			Standard	Standard		4.4.1.3(4)	EN 1992-1-1	All (Bea	Design de
Þ	Concrete characteristics									

CMD		
Name	CMD1D	
Member	B1	
Member type	Column	,
Design defaults		
A Reinforcement		
▶ Column		
Minimum cover		
Design working life [year]	50.00	
Risk of corrosion attack		
Corrosion induced by carbonation	XC3	r
Corrosion induced by chlorides	None	٢
Corrosion induced by chlorides from sea water	None	٢
Freeze / thaw attack	None	٢
Chemical attack	None	٢
Risk of abrasion attack	None	1
Possibility of special control		
Special geometric control		
Special concrete quality control		
Risk of casting on atypical surface	Standard	٢
Actions		
	Load default values >:	>>
	OK Car	cal
	OK Can	.ei

1.4.2. 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} :

 $C_{nom} = c_{min} + \Delta c_{dev}$

Minimum cover, c_{min} (art 4.4.1.2)

Minimum concrete cover, *c*min, shall be provided in order to ensure:

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

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

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

(4.2)

where:

- cmin,b
 minimum cover due to bond requirement
- cmin,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 $\Delta c_{dur,Y}$, $\Delta c_{dur,st}$ 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 cmin,b given in table 4.2.

Bond Requirement	
Arrangement of bars	Minimum cover c _{min,b} *
Separated	Diameter of bar
Bundled	Equivalent diameter (ϕ_n)(see 8.9.1)

• 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 cmin,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 th	e structural class is	given in Table 4.3N
-------------------------------------	-----------------------	---------------------

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

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

esci	iption	Symbol	Value	Default	Unit	Chapter	Code	Structure	CheckTy
	P	<all></all>	O <all></all>	C <all> D</all>	<p< th=""><th><all> D</all></th><th><all> D</all></th><th><all> D</all></th><th>Design (X</th></p<>	<all> D</all>	<all> D</all>	<all> D</all>	Design (X
esig	n defaults								
R	einforcement								
M	inimum cover								
	Design working life		50.00	50.00	year	4.4.1.2(5), t	EN 1992-1-1	All (Bea	Design de
	Risk of corrosion attack								
	Corrosion induced by carbonation		ХСЗ	XC3		4.4.1.2(5)	EN 1992-1-1	All (Bea	Design de
	Corrosion induced by chlorides Corrosion induced by chlorides from sea water		None	None		4.4.1.2(5)	EN 1992-1-1	All (Bea	Design de
			None None 4.4.1.	4.4.1.2(5)	2(5) EN 1992-1-1	All (Bea	Design de		
	Freeze / thaw attack		None	None		4.4.1.2(12)	EN 1992-1-1	All (Bea	Design de
	Chemical attack		None	None		4.4.1.2(12)	EN 1992-1-1	All (Bea	Design de
	Risk of abrasion attack		None	None		4.4.1,2(13)	EN 1992-1-1	All (Bea	Design de
- 4	Possibility of special control								
	Special geometric control					4.4.1.3(3)	EN 1992-1-1	All (Bea	Design de
	Special concrete quality control					4.4.1.2(5)	EN 1992-1-1	All (Bea	Design de
	Risk of casting on atypical surface		Standard	Standard		4.4.1.3(4)	EN 1992-1-1	All (Bea	Design de
Þ	Concrete characteristics								

II CMD			
Name	CMD1D		^
Member	B1		
Member type	Column	~	10.01
Design defaults			
Reinforcement			
▷ Column			
Minimum cover		-	
Design working life [year]	50.00		
Risk of corrosion attack			
Corrosion induced by carbonation	ХСЗ	*	
Corrosion induced by chlorides	None	~	1
Corrosion induced by chlorides from sea water	None	*	E.
Freeze / thaw attack	None	*	
Chemical attack	None	*	
Risk of abrasion attack	None	~	
Possibility of special control			
Special geometric control		_	
Special concrete quality control			
Risk of casting on atypical surface	Standard	~	-
Actions			
		Load default values >>	>

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

re	einforcen	nent stee	l in accordan	ice with EN	10080.						
Environmental Requirement for c _{min,dur} (mm)											
Structural	Exposu	re Class	according to	Table 4.1							
Class	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				

Table 4 4N: Values of minimum cover, e. . requirements with regard to durability for

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 k₁ for Abrasion Class XM1, by k₂ for XM2 and by k₃ for XM3.

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

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

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

/5:	: Design defaults 🔹 View settings 👻 Load de	ault	Fi	ind					National a	nnex:
De	Description	Symbo	ı	Value	Default	Unit	Chapter	Code	Structure	CheckTy
>		O <all></all>	P	<all></all>	<all> D</all>	<p< th=""><th><all> \wp</all></th><th><all> ρ</all></th><th><all> D</all></th><th>DesigncX</th></p<>	<all> \wp</all>	<all> ρ</all>	<all> D</all>	DesigncX
De D	Design defaults									
	> Reinforcement									
	Minimum cover									
	Design working life			50.00	50.00	year	4.4.1.2(5), t	EN 1992-1-1	All (Bea	Design de
	 Risk of corrosion attack 									
	Corrosion induced by carbonation			ХСЗ	XC3		4.4.1.2(5)	EN 1992-1-1	All (Bea	Design de
	Corrosion induced by chlorides			None	None		4.4.1.2(5)	EN 1992-1-1	All (Bea	Design de
	Corrosion induced by chlorides from sea water			None	None		4.4.1.2(5)	EN 1992-1-1	All (Bea	Design de
	Freeze / thaw attack			None	None		4.4.1.2(12)	EN 1992-1-1	All (Bea	Design de
	Chemical attack			None	None		4.4.1.2(12)	EN 1992-1-1	All (Bea	Design de
	Risk of abrasion attack			None	None		4.4.1.2(13)	EN 1992-1-1	All (Bea	Design de
	 Possibility of special control 			1						
	Special geometric control	2					4.4.1.3(3)	EN 1992-1-1	All (Bea	Design de
	Special concrete quality control						4.4.1.2(5)	EN 1992-1-1	All (Bea	Design de
	Risk of casting on atypical surface			Standard	Standard		4.4.1.3(4)	EN 1992-1-1	All (Bea	Design de
	Concrete characteristics					1				

OK Cancel

CMD		
Name	CMD1D	^
Member	B1	
Member type	Column	*
Design defaults		
Reinforcement		
▶ Column		
Minimum cover		
Design working life [year]	50.00	
A Risk of corrosion attack		
Corrosion induced by carbonation	XC3	*
Corrosion induced by chlorides	None	*
Corrosion induced by chlorides from sea water	None	*
Freeze / thaw attack	None	*
Chemical attack	None	*
Risk of abrasion attack	None	*
Possibility of special control		
Special geometric control		
Special concrete quality control		
Risk of casting on atypical surface	Standard	× .
ctions		
	Load default	values >>>

The values of k1,	k_2 and k_3 are available	able in the National Annex:	
Concrete setup			
 Type of values 	Standard EN		

NA building Concrete ' Type of functionality General Hollow core beams Concrete Prestressing Non-prestressed reinforcement Durability and concrete cover UIS General Concrete Best Sts Sts General Prestressing Sts Bailing provisions Columns Columns Beams Columns General Columns Concrete Value (mm) 0.0 Ac _{dur,st} - reduction of minimum concrete cover for use Value (mm) 0.0	 K_{XM} - values of ablasion for classes AM 1,2,3 -4-1.2.(13) Values [mm] 5.0 / 10.0 / 15.0 Δe_{dev} - value of deviation for concrete cover 4.4.1.3(3) Values [mm] 5.0 / 10.0 / 5.0 k_{omin} - minimum value of concrete cover 4.4.1.3(4) Values [mm] 40.0 / 75.0 / 5.0
--	--

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:

 $10 mm \ge \Delta c_{dev} \ge 5 mm$

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

 $10 mm \ge \Delta c_{dev} \ge 0 mm$

-

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

De	scription S	Symbol	Value	Default	Unit	Chapter	Code	Structure	CheckTy
>	P -	all> D	<all> \wp</all>	<all> D</all>	<	<all> D</all>	<all> D</all>	<all> D</all>	Design (X
Des	sign defaults								
Þ	Reinforcement								
4	Minimum cover								
	Design working life		50.00	50.00	year	4.4.1.2(5), t	EN 1992-1-1	All (Bea	Design de
	A Risk of corrosion attack								
	Corrosion induced by carbonation		ХСЗ	XC3		4.4.1.2(5)	EN 1992-1-1	All (Bea	Design de
	Corrosion induced by chlorides		None	None		4.4.1.2(5)	EN 1992-1-1	All (Bea	Design de
	Corrosion induced by chlorides from sea water		None	None		4.4.1.2(5)	EN 1992-1-1	All (Bea	Design de
	Freeze / thaw attack		None	None		4.4.1.2(12)	EN 1992-1-1	All (Bea	Design de
	Chemical attack		None	None		4.4.1.2(12)	EN 1992-1-1	All (Bea	Design de
	Risk of abrasion attack		None	None		4.4.1.2(13)	EN 1992-1-1	All (Bea	Design de
	Possibility of special control								
	Special geometric control			Mi		4.4.1.3(3)	EN 1992-1-1	All (Bea	Design de
	Special concrete quality control				1	4.4.1.2(5)	EN 1992-1-1	All (Bea	Design de
	Risk of casting on atypical surface		Standard	Standard		4.4.1.3(4)	EN 1992-1-1	All (Bea	Design de
	Concrete characteristics								

CMD			
Name	CMD1D		^
Member	B1		
Member type	Column		٣
Design defaults			
A Reinforcement			
▶ Column			-
Minimum cover			
Design working life [year]	50.00		
 Risk of corrosion attack 			
Corrosion induced by carbonation	ХСЗ		¥
Corrosion induced by chlorides	None		٣
Corrosion induced by chlorides from sea water	None		*
Freeze / thaw attack	None		¥
Chemical attack	None		*
Risk of abrasion attack	None		*
 Possibility of special control 			
Special geometric control			
Special concrete quality control			
Risk of casting on alypical surface	Standard		× .
Actions			
		Load default values	>>>
		11 mar 317	

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



Chapter 2: Design and Check

2.1 Analysis models

2.1.1 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.

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.

2.1.2 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 Settings for slabs). As diameter for the longitudinal reinforcement, the default diameter for 2D structures – and not for beams! – is taken from the Concrete Settings.

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

Also there, you have the choice for the 3 different analysis models, by means of the option "Member type":

CMD			>
	Name	CMD1D	^
	Member	B5	
	Member type	Beam	^
▲ Design defaults		Beam	
Reinforcement		Column Beam slab	
▷ Beam / Rib		- Cull Sub	-1
Minimum cover			
4 Solver setting			
4 General			
Creep and shrinkage			
▷ SLS			
Internal forces			- 1
Design As			
Beam, Column, Rib, Beam Slab			
Conversion to rebars			
Interaction diagram			
▶ Shear			
▶ Torsion			~
Actions			
		Update support width	>>>
		Load default values	>>>
		ОК Сан	ncel

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

LIFFERENCE 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, you should choose to calculate the element as a column. In the concrete settings an option is available to consider if the member is in compression or not. If the member is compressed, the second order effect is taken into account. Go to the Concrete workstation and "Concrete settings", on the "Complete setup" view :

ews: Complete setup 💙 View settings 👻 Load default 🕴 Find						1	National	annex: 🧹	
Description	Symbol	Value	Default	Unit	Chapter	Code	Struc	Check	П
all>	<all> ₽</all>	<all> D</all>	<all> D</all>		<all> ρ</all>	<all> D</all>	< ,D	<all> D</all>	
Design defaults				-					
Reinforcement									1
Minimum cover							1		
Solver setting									1
4 General									
Limit value of unity check	Lim.check	1.0	1.0			Independ	All (Be	Solver	11
Value of unity check for not calculated unity check	Ncal.check	3.0	3.0			Independ	All (Be	Solver	
The coefficient for calculation effective depth of cross-section	Coeff _d	0.9	0.9			Independ	All (Be	Solver	
The coefficient for calculation inner lever arm	Coeffz	0.9	0.9			Independ	All (Be	Solver	
The coefficient for calculation force, where member as under compression	Coeff _{com}	0.1	0.1			Independ	All (Be	Solver	
Creep and shrinkage									
Age of concrete at the moment considered	t	18250.00	18250,00	day	3.1.4.B.1-2	EN 1992	All (Be	Solver	
Relative humidity	RH	50	50	96	3.1.4.B.1-2	EN 1992	All (Be	Solver	
Type input of creep coefficient	Type φ(t,t	Auto	Auto		3.1.4(2)	EN 1992	All (Be	Solver	
Age of concrete at loading	to	28.00	28.00	day	3.1.4(2),	EN 1992	All (Be	Solver	
Consider drying and autogenous shrinkage	$Type\mathfrak{s}_{cS}(t,\!t;\!t)$	Auto	Auto		3.1.4(6)	EN 1992	All (Be	Solver	
Age of concrete at the beginning of drying shrinkage	ts	7.00	7.00	day	3.1.4(6),	EN 1992	All (Be	Solver	
▲ SLS									
Ilee effective modulue of concrete	1				7.1(2)	EN 1002.	All (Re	Soluer	

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

- If the axial compression load N_{Ed} < 0,1*A_c*f_{cd}, the member is not considered to be in compression, which means the type 'Beam' is the right choice.
- If the axial compression load N_{Ed} > 0,1*Ac*f_{cd}, the member is considered to be in compression, which
 means the beam has to be modelled as type 'Column' and the second order effect will be taken into
 account.

2.1.3 Example



Overall Design (ULS)

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

Name	dx	Case	Member	A _{sz req+}	A _{sz req} .	A _{sy req+}	A _{sy req} .	A _{sz req}	A _{sy req}	A _{s req}	ReinfReq
	[m]			[mm ²]	[mm ²]	[mm²]	[mm²]	[mm²]	[mm ²]	[mm ²]	
				Asz reg bar+	Asz reg bar-	Asy reg bar+	A _{sy req bar-}	A _{sz req bar}	A _{sy req bar}	A _{s req bar}	
				[mm ²]	[mm ²]	[mm²]	[mm²]	[mm²]	[mm ²]	[mm ²]	
B1	0,000	LC2	Column	201	201	201	201	402	402	804	[z]4ф16*,
				201	201	201	201	402	402	804	[y]4 ф16 *
B2	0,000	LC2	Beam	0	0	0	0	0	0	0	
				0	0	0	0	0	0	0	
B3	0,000	LC2	Beam slab	108	108	108	108	215	215	430	[z+]2 φ 16*,
				201	201	201	201	402	402	804	[z-]2φ16*,
											[y+]2 φ16* ,
											[y-]2 φ16 *

Shear reinforcement

Name	dx [m]	Case	Member	A _{swm req} [mm²/m]	A _{swm prov} [mm²/m]	ShearReinf
B1	0,000	LC2	Column	0	0	
B2	0,000	LC2	Beam	0	0	
B 3	0,000	LC2	Beam slab	0	0	Not required

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

Compression member

Limit axial force to consider member as compression:

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

Check condition:

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

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

2.2 Beam design

2.2.1 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 10m and it has a dimension of 500x300mm.

The inputted loads are:

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

2.2.2 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 settings of SCIA Engineer.

Shifting of bending moments (art 9.2.1.3)

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

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

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

$$a_i = z (\cot \theta - \cot \alpha)/2$$
 (for beams)

(9.2)

The additional tensile force is illustrated in Figure 9.2:



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



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

Concre	te settings								×
Views:	Complete setup View settings					1	lational a	nnex:	
De	escription	Symbol	Value	Default	Unit Chapter	Code	Stru	Chec	1
<all></all>	Q	<all> D</all>	<all> D</all>	<all> D</all>	<all> D</all>	<all></all>	< P	<a d<="" td=""><td></td>	
A De	sign defaults					10			
Þ	Reinforcement								
Þ	Minimum cover								
a So	lver setting								
Þ	General								
	Internal forces								
	Shear force reduction above supports				6.2.1(8)	EN 1992-1-1	Beam	Solver	1
	Moment reduction above supports			<u> </u>	5.3.2.2 (4)	EN 1992-1-1	Beam	Solver	
	▶ Shifting of moment curve to cover additional tensile force caused by shear				9.2.1.3(2)	EN 1992-1-1	Beam	Solver	
	Geometric imperfection in ULS	e _{i,ULS}	2		5.2(2)	EN 1992-1-1	Column	Solver	1
	Geometric imperfection in SLS	e _{i,SLS}			5.2(3)	EN 1992-1-1	Column	Solver	
	Minimum eccentricity	e _{min}	In first o	In first	6.1(4)	EN 1992-1-1	Column	Solver	
	First order eccentricity with the equivalent moment				5.8.8.2(2)	EN 1992-1-1	Column	Solver	
	Second order eccentricity	e ₂			5.8.8	EN 1992-1-1	Column	Solver	
	Internal forces modifications								
Þ	Design As								
⊳	Conversion to rebars								
Þ	Interaction diagram								
ħ	Chase								



REDUCTION OF BENDING MOMENT (art 5.3.5.5 (3) & 5.3.2.2 (4))

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

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

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

$$\Delta M_{Ed}$$
 = FEd,sup t / 8

where:

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

• t is the width of the support

(5.9)

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

vs: Complete setup 🛛 👻 View settings 👻 Load de	ault	F	Find					Nationa	l annex:
Description	Symbol		Value	Default	Unit	Chapter	Code	Structu	CheckT
Þ ,	<all></all>	P	<all> P</all>	<all></all>	Q P	<all> D</all>	<all> D</all>	<all> D</all>	<all> D</all>
Design defaults									
Reinforcement									
Minimum cover									
Solver setting									
⊳ General									
 Internal forces 									
Shear force reduction above supports				M		6.2.1(8)	EN 1992-1-1	Beam,B	Solver se
Moment reduction above supports						5.3.2.2 (4)	EN 1992-1-1	Beam,B	Solver se
Shifting of moment curve to cover additional tensile forc.	4			S	1	9.2.1.3(2)	EN 1992-1-1	Beam,Ri	Solver se
Geometric imperfection in ULS	ei,ULS					5.2(2)	EN 1992-1-1	Column	Solver se
Geometric imperfection in SLS	ei, SLS					5,2(3)	EN 1992-1-1	Column	Solver se
Minimum eccentricity	e _{min}		In first order	In first or.	-	6.1(4)	EN 1992-1-1	Column	Solver se
First order eccentricity with the equivalent moment				2		5.8.8.2(2)	EN 1992-1-1	Column	Solver se
Second order eccentricity	e ₂					5.8.8	EN 1992-1-1	Column	Solver se
Internal forces modifications									
Design As									
Conversion to rebars									
Interaction diagram							(

M CMD		×						
	Name CMD1D							
	Member B5							
	Member type Beam	~						
Design defaults								
Reinforcement								
Minimum cover								
Solver setting								
4 General								
Creep and shrinkage								
▶ SLS								
 Internal forces 								
Shear force red	luction above supports							
Moment red	luction above supports							
Shifting of moment curve to cover additional tensile	nent curve to cover additional tensile fc Chapter : 5.3.2.2 (4) Code : EN 1992-1.1							
 Internal forces modifications 	Remark : Bending moment above support is reduced if this item is							
▶ Beam	 for standard support, formula 5.9 is used for column support the reduced moment is the same as on the face of 							
Design As								
Conversion to rebars								
Interaction diagram								
Shear								

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



Using formula 5.9 (5.3.2.2 (4))


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

SUPPORT II	N NODE (1)
Name	Sn1
Туре	Standard \lor
Angle [deg]	
Constraint	Sliding \lor
х	Free \vee
Y	Free 🗸
Z	Rigid \checkmark
Rx	Free \vee
Ry	Free \vee
Rz	Free \lor
Default size [m]	0.20

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

			N	ame CMD1D			
			Men	ber B5			
			Member	type Beam			
Design d	efaults						
Reinford	cement						
Minimu	m cover						
Solver se	etting						
General	U						
Internal	forces						
Design	As						
Convers	sion to rebars						
Interact	tion diagram						
Shear							
Torsion							
Stress li	imitations						
Crackin	g forces						
Crack w	ridth						
Deflecti	ons						
ions							
ions				I	Update Load	support wi default val	idth >>: lues >>:
ions				I	Update Load	support wi default val OK	idth >>: lues >>: Cance
ions	upports width			[Update Load	support wi default val OK	idth >>> lues >>> Cance
ions I Su	upports width			[Update Load	support wi default val OK	idth >>: lues >>: Cance
ions I Su	ipports width Name	Position [m]	Width [m]	Shear reduction	Update Load	support wi default val OK	idth >>: lues >>: Cance X tion
ions I Su	ipports width Name B1	Position [m] 0.000	Width [m] 0.200	Shear reduction	Update Load	support wi default val OK ent reduct	idth >>: Lues >>: Cance X tion

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

- t = 0,2m
- F_{Ed,sup} = 477,5kN
- ΔM_{Ed} = 477,5*0,2 / 8 = 11,94kNm

The original moment M_y at the support was 254,16 kNm:



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



With moment capping at support taken into account the recalculated moment is 242,22 kNm.



REDUCTION OF SHEAR FORCES (art 6.2.1 (8))

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

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

icrete settings									
ws: Complete setup 👻 View settings 👻 Load defa	ult	Find					Nationa	il annex:	
Description	Symbol	Value	Default	Unit	Chapter	Code	Structu	CheckT	Π
	<all></all>	all> p	<all> ρ</all>	<p< td=""><td><all></all></td><td> <all> ₽</all></td><td><al⊳ td="" ⊅<=""><td><all> D</all></td><td></td></al⊳></td></p<>	<all></all>	<all> ₽</all>	<al⊳ td="" ⊅<=""><td><all> D</all></td><td></td></al⊳>	<all> D</all>	
Design defaults									
Reinforcement					-				11
Minimum cover							1		
Solver setting									
Ø General							1		1
 Internal forces 							-		11
Shear force reduction above supports					6.2.1(8)	EN 1992-1-1	Beam,B	Solver se	11
Reduce shear forces		On the face 🔺	On the fa		6.2.1(8)	EN 1992-1-1	Beam,B	Solver se	11
Moment reduction above supports		On the face (su	pport/colum	nn)			Beam,B	Solver se	11
Shifting of moment curve to cover additional tensile forc		On the face (su	pport/colum	n <mark>n) +</mark> eff	ective depth	of cross-section	Beam,Ri	Solver se	11
Geometric imperfection in ULS	e _{i,ULS}				5.2(2)	EN 1992-1-1	Column	Solver se	11
Geometric imperfection in SLS	e _{i,SLS}				5.2(3)	EN 1992-1-1	Column	Solver se	
Minimum eccentricity	e _{min}	In first order	. In first or		6.1(4)	EN 1992-1-1	Column	Solver se	
First order eccentricity with the equivalent moment					5.8.8.2(2)	EN 1992-1-1	Column	Solver se	
Second order eccentricity	e ₂				5.8.8	EN 1992-1-1	Column	Solver se	
Internal forces modifications									
Design As									
Conversion to rebars									
> Interaction diagram				2	1				

CMD		
Design defaults		
Reinforcement		
Minimum cover		
Solver setting		
▲ General		
Creep and shrinkage		
▶ SLS		
Internal forces		
	Shear force reduction above supports 🔽	
	Reduce shear forces On the face (support/column)	^
	Moment reduction above supports On the face (support/column)	
Shifting of moment curv	to cover additional tensile force caused by shear	section
Internal forces modificat	ons	
▶ Beam		
Design As		
Conversion to rebars		
Interaction diagram		
▶ Shear		

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



Also for the reduction of shear forces, the support width "t" is taken into account, which is taken from the properties of the support or the 1D member data. The reduction of shear forces at supports is illustrated for our example below with t = 0,2 m.

The first image displays the original V_z:



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







2.2.3 Theoretical reinforcement

CONFIGURATION

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

- **Required reinforcement:** The required reinforcement is a numerical value (mm²) of the reinforcement that is necessary in every section of the beam.
- **Provided reinforcement:** The provided reinforcement is a template added to each beam/column consisting of basic and additional reinforcement.

The configuration of theoretical reinforcement can be found in the Concrete settings, in the "Design defaults" view. Templates of longitudinal reinforcement and stirrups for different shapes of beam are available. The concrete cover can be set for upper, lower and side faces.

iews: Design defa	aults 👻 View settings 👻		Load defa	ult	Find						Natio	onal annex:	
Description			Symbol		Value	Default		Unit	Chapter	Code	Structure	CheckType	ĺ.
<all></all>		P	<all></all>	P	<all> ₽</all>	<all></all>	Э.	< P	<all></all>	<all> ₽</all>	<all> D</all>	Design de $ imes$	
 Design defaul 	its												
A Reinforcer	ment												
🔺 Beam	/ Rib												
Des	sign of provided reinforcement									Independent	Beam,Rib	Design defa	
R	ectangular section				Beam_Rec	Beam_Rec	+++			Independent	Beam,Rib	Design defa	
Т	section				Beam_Tse	Beam_Tse				Independent	Beam,Rib	Design defa	
L	section				Beam_Lse	Beam_Lse				Independent	Beam,Rib	Design defa	
D	section				Beam_Ise	Beam_lse.				Independent	Beam,Rib	Design defa	
0	Other and general				Beam_Ot	Beam_Ot.				Independent	Beam,Rib	Design defa	
∡ Lor	ngitudinal												
4	Upper (z+)												
	Type of cover				Auto	Auto			4.4.1	EN 1992-1-1	Beam,Rib	Design defa	
	Diameter		d _{s,u}		16.0	16.0	ſ	mm		EN 1992-1-1	Beam,Rib	Design defa	
	Lower (z-)												
	Type of cover				Auto	Auto			4.4.1	EN 1992-1-1	Beam,Rib	Design defa	
	Diameter		d _{s,l}		16.0	16.0	ſ	mm		EN 1992-1-1	Beam,Rib	Design defa	
4	Side (y±)												
	Type of cover				Upper	Upper			4.4.1	EN 1992-1-1	Beam,Rib	Design defa	
D	Detailing (det)												L

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



This template exists of basic, additional and shear reinforcement. The purpose is to compare these templates with the required reinforcement, to model the user reinforcement that is introduced later on or to convert it automatically to user reinforcement.

⇒ Longitudinal reinforcement

The basic reinforcement is present along the whole length of the beam; the additional reinforcement is present only at the zones where basic reinforcement is not sufficient to withstand (recalculated) internal forces.

A choice can be made between fixed additional bars (diameter and number) or a list with different numbers of bars with a fixed diameter. SCIA Engineer uses the least amount of necessary additional bars or places the maximum if this template is still not sufficient to resist the (recalculated) internal forces. Next to the basic and additional reinforcement you can also set a diameter for the detailing reinforcement. The detailing reinforcement is reinforcement that statically is not required but that needs to be added to the cross-section to fulfil the detailing provisions.



⇒ Shear reinforcement

For the shear reinforcement the number of cuts, the maximum number of stirrup zones, the diameter and the spacing can be set. For the spacing different types of input can be used: **Multiple** and **User defined**. Multiple means that the spacing between the stirrups will be the multiple of a set value. With User defined reinforcement you can set the spacings that can be used. SCIA Engineer will automatically select the spacing depending on this template and the general settings in the design defaults. The option **Symmetrical** allows you to define whether the zones in each span will be symmetrical or not.



4 CONFIGURATION FOR CONVERSION TO REBARS

The configuration for conversion to rebars can be found in the Concrete settings, in the "Complete setup" view. Different options are available:

Co	ncre	te settings									- 🗆	×
Vi	ws:	Complete setup 👻 View settings 💌 Load o	lefault		Find					Natio	nal annex: 🛛	
	De	escription	Symbol		Value	Default	Unit	Chapter	Code	Structu	CheckType	Π
<	all>	Q	<all></all>	ρ	<all></all>	<all> 🔎</all>	< <i>P</i>	<all> 🔎</all>	<all> 🔎</all>	<all> 🔎</all>	<all> 🔎</all>	
	De	sign defaults										
	⊳	Reinforcement										
	⊳	Minimum cover										
	So	lversetting										
	⊳	General										
	⊳	Internal forces										
	⊳	Design As										
		Conversion to rebars										
		Unify upper reinforcement above middle support			Z				Independent	Beam,B	Solver setti	>>
		Minimum length of long.reinforcement			1000	1000	mm		Independent	1D (Bea	Solver setti	
		Uniformly distributed reinforcement for the column			~				Independent	Column	Solver setti	
		Number of corrected bars (neighbouring sections)			2				Independent	1D (Bea	Solver setti	
		Type of zone for corrected shear reinforcement			Geometrical	Geometri			Independent	1D (Bea	Solver setti	
	⊳	Interaction diagram										
	⊳	Shear										
	⊳	Torsion										
	⊳	Punching										
	⊳	Stress limitations										
_	ь	Cracking forces										
											ок с	Cancel

⇒ Unify upper reinforcement above middle support

Unifies the number of bars of upper reinforcement at the middle support. The maximum number of bars from the left and right side of the support are taken into account.



⇒ Minimum length of longitudinal reinforcement

Sets a minimum length for the longitudinal reinforcement.



⇒ Uniformly distributed reinforcement for the column

Uniform distribution of reinforcement along the whole length of the column, with maximum area from y and z edges in all sections taken into account.



⇒ Number of corrected bars (neighbouring sections)

Additional reinforcement is tested in each section for number of bars and diameter in neighbouring sections. If the additional reinforcement can be distributed to the stirrup links between basic reinforcement bars, the number of bars and diameter of additional reinforcement is increased to fulfil conditions. The reason for the correction of the number of bars of additional provided reinforcement is to have logic and symmetrical reinforcement in the cross-section along the beam.



⇒ Type of zone for corrected shear reinforcement

None - Zones for shear reinforcement are not created. Conversion of provided reinforcement to real bars is not possible.

- (A) Geometrical Member is in every span divided geometrically in zones with the same length.
- (B) Spacing Member is in every span divided in zones according to the most occurrent spacing.

	(A)
(s,prov,min1)	(s,prov,min2)
L1=0,5L	L2=0,5L
l,zone,1 x,zone=x,be(,,most x,end,most 1,zone,2
s,prov,min1)	(B) rov,most (s,prov,min2)
L1=l,zone,1	L2=L-l,zone,1

4 CALCULATION OF LONGITUDINAL REINFORCEMENT As

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

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

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

The following results are obtained with these settings:



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

Longitud	Longitudinal required reinforcement													
Name	dx	Case	Member	Asz_req+	Asz_req-	Asy_req+	Asy_req-	Asz_req	Asy_req	As_req	ReinfReq			
	[m]			[mm²]	[mm²]	[mm²]	[mm²]	[mm²]	[mm ²]	[mm²]	-			
				Asz_req_bar+	A sz_req_bar-	A _{sy_req_bar+}	A _{sy_req_bar} -	Asz_req_bar	A _{sy_req_bar}	As_req_bar				
				[mm²]	[mm²]	[mm²]	[mm²]	[mm²]	[mm ²]	[mm ²]				
S1	2,333-	ULS	Beam	0	1335	0	0	1335	0	1335	[z-]7φ16			
				0	1407	0	0	1407	0	1407				
S1	4,833-	ULS	Beam	1470	0	0	0	1470	0	1470	[z+]8¢16			
				1608	0	0	0	1608	0	1608				

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

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

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

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

 $A_{sy_req} = 0$ because there is no torsion on this beam.

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

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



Longitudinal required reinforcement

Name	dx [m]	Case	Member	Asz_req+ [mm²]	Asz_req- [mm²]	A _{sy_req+} [mm²]	A _{sy_req} . [mm²]	Asz_req [mm²]	A sy_req [mm ²]	A₅_req [mm²]	ReinfReq
				A _{sz_req_bar+} [mm ²]	A sz_req_bar- [mm²]	A _{sy_req_bar+} [mm ²]	A _{sy_req_bar} . [mm²]	A _{sz_req_bar} [mm ²]	A _{sy_req_bar}	A _{s_req_bar} [mm²]	
S1	2,333-	ULS	Beam	0	1343	0	0	1343	0	1343	[z-]5φ20
				0	1571	0	0	1571	0	1571	
S1	4,833-	ULS	Beam	1479	0	0	0	1479	0	1479	[z+]5¢20
				1571	0	0	0	1571	0	1571	

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:

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

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

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

CMD1D	
BS	
Beam	~
: 🛃	
Beam_Rect_Empty v	• •••
B 500B	
User	٣
30.0	
16.0	٣
User	٣
1 30.0	
16.0	*
12.57497.1	
Upper	*
10.0	*
B 500B	
Update support width	>>>
Load default values	>>>
	CMD1D B5 Beam Beam_Rect_Empty SooB User 30.0 16.0 16.0 10.0 16.0 10.0 10.0 Upper 10.0 Update support width Load default values

Next to the required reinforcement area, also the provided reinforcement can be viewed. Both can be displayed as a value in reinforcement area (value As or Aswm) or as the amount of bars (value N ϕ or N ϕ w). Also the weight of the reinforcement can be shown (value GI or Gw).

	(1)	0 ×	6.42			
RESUL	(I)		=	RESULT	S (1)	5
Name	Reinforcement 1D design			Name	Reinforcement 1D design	
SELECTION			▼ SELECT	ION		_
Type of selection	AII	<u> </u>		Type of selection	All	
Filter	No	\sim		Eiltor	No	
Results in sections	All	\sim		Plice		_
RESULT CASE				Results in sections	All	
Type of load	Combinations	~	▼ RESULT	CASE		
Combination	ULS-Set B (auto)			Type of load	Combinations	
EVTDENE 1D				Combination	ULS-Set B (auto)	
EXTREME ID Extreme 10	Global	~	▼ EXTREM	IE 1D		_
Transfer la	Desuined			Extreme 1D	Global	
Type of values	Required	II		Type of values	Provided	
Values	As,rec	Ň		Values	Asiptov	_
Interval	As,req			Interval	As,prov	
LIMIT STATE CONDITION	Aswm,reg			intervat	As,prov (∑)	
Design ULS	Nø,req		▼ LIMIT ST	TATE CONDITION	Aswm,prov	
OUTPUT SETTINGS	Nø,req (Σ)			Design ULS	Nø,prov	
Output	Nøw,req		▼ OUTPUT	TSETTINGS	Nøw prov (2)	
DRAWING SETUP 1D	Gw.reg	1		Output	Gl,prov	
Display value name	Components		▼ DRAWIN	IG SETUP 1D	Gw,prov	
Display values				Display value name	Components	

The option Required – not covered will show the amount of provided reinforcement that is missing. For example: $\Delta A_{s,req} = A_{s,req} - A_{s,prov}$, thus the amount of reinforcement which still has to be added to the template to resist the (recalculated) internal forces. If $A_{s,prov} > A_{s,req} = 0$

= RESULT	S (1)	6	×
	Deinfersenent 1D desim	PN	-
Name	Reinforcement 1D design		4
▼ SELECTION			
Type of selection	All	~	2
Filter	No	\sim	1
Results in sections	All	~	
 RESULT CASE 			
Type of load	Combinations	\sim	
Combination	ULS-Set B (auto)	~	/
▼ EXTREME 1D			
Extreme 1D	Global	\sim	
Type of values	Required - Not covered	~	2
Values	ΔAs,req	T	Ś
Interval	ΔAs,req	u	9
 LIMIT STATE CONDITION 	ΔAs,req (Σ)		1
Design ULS	ΔNø,req		1
▼ OUTPUT SETTINGS	ΔNøw,req		1
Output	Components	-	.1

Unity checks can be performed on the provided reinforcement compared to the required reinforcement. This will give you an idea of the efficiency of the reinforcement.

(P)			
Ξ	RESULTS (1)		
	Name	Reinforcement 1D design	n
▼ SELECTION			
Type of	selection	All	\sim
	Filter	No	\sim
Results in	sections	All	~
▼ RESULT CASE			
Тур	e of load	Combinations	\sim
Com	bination	ULS-Set B (auto)	~
▼ EXTREME 1D			-
Ex	treme 1D	Global	\sim
Туре	of values	Provided - Utilization	~
	Values	UC(As,prov)	X
	Interval	UC(As,prov)	45
▼ LIMIT STATE CONDITIO	N	UC(Asw,prov)	
De	sign ULS	As,req-prov	
▼ OUTPUT SETTINGS		As,prov	
	Output	Aswm,req	
DRAWING SETUP 1D		Aswm,prov	
Display val	ue name	Components	

4 CALCULATION OF SHEAR REINFORCEMENT Aswm

Shear reinforcement								
Name	dx	Case	Member	A _{swm_req}	Aswm_prov	ShearReinf		
	m			mm²/ m	mm²/m			
S1	7,333-	ULS	Beam	298	309	ф8/325mm,		
						(ns=2)		
S1	4,900	ULS	Beam	1315	1340	φ8/75mm,		
						(ns=2)		

VEd = design shear force resulting from external loading

V_{Rd,c} = design shear resistance of the member without shear reinforcement

V_{Rd.s} = design value of the shear force which can be sustained by the yielding shear reinforcement

 $V_{\text{Rd,max}}$ = 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:

Concrete strut failure

- VEd > VRd,max • V_{Ed} ≤ 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)

$$V_{Rd,c} = [C_{Rd,c} k (100 \ \rho_{I} \ f_{ck})^{1/3} + k_{1} \ \sigma_{cp}] \ b_{w} d$$
(6.2.a)

with a minimum of:

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

where:

- = characteristic concrete compressive strength [MPa] • f_{ck}
- = size factor: $k = 1 + \sqrt{(200/d)} \le 2.0$ (with d in mm) • k
- = longitudinal reinforcement ratio: $p_l = A_{sl}/b_w d \le 0.02$ • ρι
- = smallest web width of the cross-section in the tensile area [mm] . bw
- σ_{cp} = concrete compressive stress due to loading: $\sigma_{cp} = N_{Ed}/A_c < 0.2 f_{cd}$ [MPa]
- = effective height of cross section d •

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

$$V_{min} = 0.035 \text{ k}^{3/2} \cdot f_{ck}^{1/2}$$
 (6.3N)

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

V _{Ed} ≤ 0,5 b _w d v f _{cd}	(6.5)
--	-------

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

The recommended value for v follows from:

$$V = 0.6 \left[1 - \frac{f_{ck}}{250} \right] \tag{6.6N}$$

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

Concrete setup			×
 Type of members ID 2D 20 Type of values	Standard EN General General Durability and concrete cover DUS General Punching Stress limitation during tensioning Stress limitation during tensioning Stress limitation during tensioning Stress limitation Detailing provisions Columns Beams 20 structures and slabs Punching	Name Standard EN Concrete General ULS General Øg=1/x - basic value of inclination 5.2(5) λ_{lim} 5.8.3.1(1) Øg=1/x - basic value of inclination 5.2(5) λ_{lim} 5.8.3.1(1) Type of simplified method for analysis second order ef CRd,o Value[-] 0.18 k1,shear - coeff. for calculation Vrd,c 6.2.2(1) Value[-] Value [-] 0.15 Vmin - coeff. for calculation Vrd,c for shear 6.2.2(1) Formula Formula Formula Formula Formula V - strength reduction factor for concrete cracked in s Formula Formula Ømin - min. angle between the concrete compression strut Ømin,prestressed - min. angle between the concrete compre Ømax Minimal angle between the concrete compression strut Ømin,o - Minimal angle between the concrete compression strut Ømin, - Minimal angle between the concrete compression strut Ømin, - Minimal angle between the concrete compression strut	
	۲ ×	v _n - strength reduction factor for concrete cracked in she	v
Select all Unselect all	Refresh	Load default NA parameters OK	Cancel

Note: the green values are according to EN code.

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

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



The angle θ should be limited.

The recommended limits of $\cot \theta$ are given:

 $1 \le \cot \theta \le 2,5$

(6.7N)

The angle θ can be inserted in SCIA Engineer:

s: Complete setup 👻 View settings 🔻 Load defa	ult	Find					Nationa	al annex:
Description	Symbol	Value	Default	Unit	Chapter	Code	Structu	CheckT
>	<all> ρ</all>	<all> ρ</all>	<all> \wp</all>	<p< th=""><th><all> D</all></th><th><all> \wp</all></th><th><all> D</all></th><th><all> ρ</all></th></p<>	<all> D</all>	<all> \wp</all>	<all> D</all>	<all> ρ</all>
Design defaults								
Reinforcement								
Minimum cover								
Solver setting								
⊳ General								
Internal forces		-						
Design As								
Conversion to rebars								
Interaction diagram								
4 Shear	,							
Type calculation/input of angle of compression strut	Туре Ө	User(angle)	User(angle)		6.2.3	EN 1992-1-1	All (Bea	Solver se
Angle of compression strut	θ	40.00	40.00	deg	6.2.3	EN 1992-1-1	All (Bea	Solver se
Cotangent angle of compression strut	$\cot(\theta)$	1.2	1.2		6.2.3	EN 1992-1-1	All (Bea	Solver se
Consider effect of axial force in nonprestressed shear che	Type a _{cw}				6.2.2(1)	EN 1992-1-1	1D (Bea	Solver se
 Shear between web and flanges 							1	
Type input of angle of compression strut	Type θ _f	User(angle)	User(angle)		6.2.4(4)	EN 1992-1-1	Beam,B	Solver se
Angle of compression strut	θ _f	40.00	40.00	deg	6.2.4(4)	EN 1992-1-1	Beam,B	Solver se
Cotangent of angle of compression strut	$\cot(\theta_f)$	1.2	1.2		6.2.4(4)	EN 1992-1-1	Beam,B	Solver se

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

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

$$V_{Rd,max} = \alpha_{cw} b_w z v_1 f_{cd} / (\cot \theta + \tan \theta)$$
(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
- v₁ = 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 $f_{ck} \leq 60 \text{ MPa}$	(6.10.aN)
$v_1 = 0.9 - f_{ck}/200 > 0.5$	for $f_{ck} \ge 60 \text{ MPa}$	(6.10.bN)

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

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

Type of members	Standard EN	Name Standard EN							
1D 🔽	🖻 Concrete	4 Concrete							
2D 🔽	- General	▶ General							
Type of values	Non-prestressed reinforcement	4 ULS							
NA building	Prestressed reinforcement Durability and constants	4 General							
Type of functionality		θ _n =1/x - basic value of inclination 5.2(5)							
Hollow core beams 🔽	General	⊳ λ _{lim} 5.8.3.1(1)							
Prestressing 🔽	E Punching	Type of simplified method for analysis second order ef							
	General	⊳ c _{Bd c}							
	Prestressing	k _{1 shear} - coeff. for calculation Vrd,c 6.2.2(1)							
 Stress limitation during tensioning Stress limitation Stress limitation Detailing provisions Common detailing provisions Columns Beams 2D structures and slabs Punching 	v _{min} - coeff. for calculation Vrd,c for shear 6.2.2(1)								
	v - strength reduction factor for concrete cracked in s								
	k - Coefficient for calculation of longitudinal shear stre								
	θ _{min} - min. angle between the concrete compression strut								
	Value [deg] 21.80 ^b ® _{min,prestressed} - min. angle between the concrete compression strul ^d ® _{max} - max. angle between the concrete compression strul								
									Value [deg] 45.00
									θ _{min,c} - Minimal angle between the concrete compression
		θ _{min,t} - Minimal angle between the concrete compression s							
		• • • • • • • • • • • • •							
		v _{1a} - strength reduction factor for concrete cracked in she							
		Value [-] 0.60							
		v _{1b} - strength reduction factor for concrete cracked in she							
		Formula Formula							
		▲ a _{cv} (non-prestressed structures)							
		Value [-] 1.00							
	a _{cw} (prestressed structures)								
		Formula Formula							
		k - shear calculation factor for plain and lightly reinfo							
		k ₁ -coefficient for calculation g _{Rd,max} 6.5.4(4)							
	< >>	k ₂ -coefficient for calculation opd man 6.5.4(4)							

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



Shear reinforcement

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

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

2.2.4 Practical reinforcement

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

In the theoretical reinforcement design, we have calculated where reinforcement is needed.

This allows us to input manually the practical reinforcement by adding New reinforcement for the whole length of the beam.

We can first select a template for the longitudinal reinforcement:



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

Reinforcement parameters	×
Do you want to use parameters of reinforcement (diameter of long.reinforcement, stirrup and concrete cover)
O from the Design defaults	
If from the defined template	
0	К

The practical reinforcement is shown graphically on the screen:



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

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

Stirrup shape	manager			Х
et -: 🖸 🕩	1	I 🕞 🖸		
StirrupR9 StirrupR10 StirrupR11 StirrupR12 StirrupR13 StirrupR14 StirrupR15 StirrupR16 StirrupR17				
Name Description Number of stirru Diameter (mm)	StirrupR9 Stirrups tem; 1 8.0			
Number of cuts	2			
New Insert	Edit Dele	te		ОК

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



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



The selected zone of the member can be modified by the properties panel or by the menu Library / Concrete, Reinforcement / Longitudinal Reinforcement Library:

gitudinal reinforcement							
	-	2			Filter <u>All</u> L1-S1E4 L2-S1E2		
					Delete	Delete	all
					Position numbe	2	-1
	3		1		Diameter [mm]	16.0	
					Number of bars	2	
					Area [mm^2]	40.2	
					Layer type	Uniform	۷
					Cover type	Surface to	۷
					Cover [mm]	0.0	
					Left bar	Before the	۷
		Q			Right bar	Before the	۷
					Stirrup name	S1	٧
						A	
		ENT DADAMETERS			Analysis model		EN
Now laws	Number of bars		herees	datha w	Selected lavers	402	mn
Add bars to corpore	Diameter [mm]	2 .	Dearns and	TTIDS	All lavers	804	mn
Aud Dars to corners	Channeter [mini]	0.0 51	STIRRUPS		PICTURE PROPER	RTIES	an
	Surrup name	31 *	Edits	tirrups	Draw dimen	sions	
Bars positions	Luge index				Texts scale	0.5	+
COLLISION OF BARS					Rec	Iraw	
C III I I I I I I I I I I I I I I I I I	 Batwaan evictin 	o pars					

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

Longitudinal reinforcement					×
	2	٩]	Filter <u>All</u> L1-51E4 L2-51E2 L3-51E4	¥
				Delete	Delete all
	3	1		Name Position numbe Diameter [mm] Number of bars Area [mm^2] Layer type Cover type Cover type Cover [mm] Stirrup name Edge index Detailing	L3-S1E4 7 7 20.0 3 942 No corner * Surface to * 0.0 S1 * 4 *
		METERS	TYPE OF REAM	Analysis model	Automatic design
New layer	Number of bars 3 Diameter [mm] 20.0	* *	beams and ribs	Selected layers All layers	942 mm ^A 1747 mm ^A
	Stirrup name S1	* *	STIRRUPS Edit stirrups	PICTURE PROPEI	RTIES
Bars positions COLLISION OF BARS				Texts scale Rec	0.5
	Move layer			OK	Cancel

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

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

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

Different stirrup zones can be created:

Stirrups zones		×
+		
2x1	048.0-0.100 2x9d8.0-0.300 2x31d8.0-0.100 2x10d8.0-0.277 2x11d8.0-0.099 0.05050 0.050 0.004 0.004 0.004 2.500 0.004 0.004 1.000 1.000 2.500 0.050 0.004 3.000 0.004 2.500 0.004 1.000 1.000	
Zone 1 Zone 2	Minimum stirrup reinforcement Text scale 1 Zone Length [m] Diameter [mm] Distance [m] Real distance [m] Type By user Distance to begin [m] By user Distance to end Dis	nd [m]
Zone 4 Zone 5	1 3 3.000 8.000 0.100 0.100 single v yes v 0.004 yes 0.000	
	Additional stirrup reinforcement Symmetrical	
	Parts from both points Input type Numbers Diameter [mm] Distance [m] Total distance [m] Type	
New zone	Delete zone New part Delete part OK Ca	ncel

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

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

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

Vi	ew parameters setting - Concrete		
	Check / Uncheck group		Lock position
4	🔲 Structure 🕮 Labels 🖾 Model	Concrete Composite Modelling/E	Drawing 😌 Attributes 🖉 Misc. 🔍 View 🕟
	Check / Uncheck all		
	Service		
	Display on opening the service	7	
E	Concrete + reinforcement		
	Display	2	
	Member data	7	
	SaT detail data	~	
	Drawing directions for design		
	Main reinforcement	7	
	Style of main reinforcement	11	•
	Stirrups	7	
	Style of stirrups	11	•
Ш.	Number of stirrups	11	-
	Color of reinforcement	olour by diameters	•
	Scheme of reinforcement		
	Reinforcement drawing type	D	•
	Rounded bends	7	
	Concrete labels		
	Display label	7	
	Name	7	
	User defined reinforcement		

The total practical reinforcement of the beam is shown below:



A zoomed view shows the 3D representation:



2.2.5 Conversion of theoretical reinforcement into practical reinforcement

Since SCIA Engineer 19 it is also possible to convert theoretical reinforcement into practical reinforcement. As mentioned in previous chapter there are two types of theoretical reinforcement: **Required reinfocement** (= mm² necessary in each section) and **Provided reinforcement** (= template of reinforcement with various ammounts of additional reinforcement possible). It is only possible to convert **Provided reinforcement** into practical (=user) reinforcement.

Let's have a look at this example : open beam.esa

Set the template of provided reinforcement.



Go to Reinforcement design and look at the value As, $prov(\phi)$. This is the provided reinforcement that will be converted into practical reinforcement.



Press 'Conversion for real bars'



The following reinforcement is generated.



The practical reinforcement is added as reinforcement data. You can edit the reinforcement by selecting it and then click on 'Edit reinforcement'.



Now the parts of the reinforcement that needs edditing can be slected. The diameter, number of bars, length, spacing, ... can be changed in the properties window.

Remark:

It might occur the error message 'Conversion of reinforcement was not done because the Type of zone of shear reinforcement is set to 'None' in the Design defaults' appears within the summary after conversion when converting the provided reinforcement into real bars. This behaviour is caused due to the option 'None' is selected for the setting 'Type of zone for corrected shear reinforcement' within the design defaults.

Summary	after conversion		
Member S1	Additional data	Status Not OK	Explanation Conversion of reinforcement was not done because the Type of zone for shear reinforcement is set to 'None' in the Design defaults.
<			OK

In the example, we will increase the length and diameter of reinforcement area 5.

Name	Long5
Position number	5
Туре	Additional 🗸
Torsional	
BASIC PARAMETERS	
No. of bars	2
Diameter [mm]	22.00
Area As [mm^2]	760.27
Edge type	Upper 🗸
Material	B 600C 🗸
GEOMETRY	
Edge	3
Stirrup	Shear1
Coord. definition	Absolute \vee
Begin [m]	4.00
End [m]	6.00
Length [m]	2.00
Edge distance [mm]	43.00
Anchorage length at begin [m]	0.00
Anchorage length at end [m]	0.00
DETAILED INFO BAR - 1	
Info	φ22.0(8 600C);y=0.048;z=0.19
BAR - 2	
Info	@22.0(8 600C);v=-0.048:z=0.15



Note that the converted reinforcement cannot be put together with the practical user reinforcement on the same element. You can either input your own reinforcement template using the first option or convert the reinforcement template using the second option. In general, it is advisable to use the first option for elements with a more difficult layout of the reinforcement (multiple reinforcement layers) as this is easier to adjust. For elements that have a very similar reinforcement layout, the converted reinforcement can be a useful tool.

2.2.6 Checks

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

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

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

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

Available checks at the Ultimate Limit State are:

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

Available checks at the Serviceability Limit State are:

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

There is also an Overall check available. This will simply check all the checks you have activated, but for a more detailed report, you will need to go into the check itself.

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

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

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

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

Example 1: beam practical reinforcement.esa

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

Example 2: beam_without practical reinforcement.esa

GAPACITY RESPONSE

The Capacity - response is based on the calculation of strain and stress in a particular component (concrete fibre or reinforcement bar).

The check consists of the comparison of those strains and stresses with the limited values according to EN 1992-1-1 requirements.

However, this method does not calculate extremes (capacities of the cross-section) like the interaction diagram but calculates the state of equilibrium for that section (response).

For capacities of the member, please refer to the "Capacity – diagram" check.

The following checks are performed:

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

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

Example: beam_practical reinforcement.esa

Run the Capacity – Response check in Design > Concrete 1D > ULS response check.

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

Beam S1					RECT	(500; 3	300)		
CEN 1992-1-1:2	2004/AC:200	8			Section 26 [dx = 5 m]				
ECEN 1992-1-1:2004/AC:2008 Member length: L = 1 Buckling y-y $^{\perp}$ L _y = Buckling z-z $^{\perp}$ L _z = L_z = 5¢		0 m 10 m (swa 0 m (swa 20 (1571 n	y) y) 1m2)	Section Concret Bi-linea Exposu Longitu Bi-linea 7\\$20 m \$\rho_1 = 1.4\$ Shear re Bi-linea \$\phi10/95\$ \$\rho_w = 1\$. Cover (6) Tone 36	te: C30/3 ar stress ure class:) dial rei ar with an $nm (A_s =$ 466 % (17) einforcer ar with an 0.7 mm (n 051 % (1) stirrup) b mm	7 7 strain dia (C3 nforcer inclined 2199 mi .3 kg/m ment: B inclined s = 2) (A 2.4 kg/m	agram hent: B 500A d top branch m ²) 500A d top branch sw = 157 mm ²) 1) (A _{swm} = 1576	ō mm²/m)	
		2ф2	!0 (628 mr	m2)	Botton Left: 36 Right: 3	n: 36 mm 5 mm 36 mm			
mmary of ch	300 neck	φ10)/100 mm,	ns=2					
Type of component	Fibre / Bar	ε _{extr} [‰]	σ _{extr} [MPa]	Check strain [-]	Check stress [-]	UC [-]	Limit [-]	Status	
	1	-1.63	-18.7	0.47	0.93	0.95	1	OK	
Concrete									

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

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

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

Ε

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

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



Settings that might influence the check:

• Effective depth of cross-section - d

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



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

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

In those cases, the effective depth is calculated according to formula :

With:

- Coeff_d by default 0,9 in Concrete settings, in "Complete Setup" view, and in "Solver settings" / "General"
- h height of cross-section perpendicular to neutral axis

Concre	ete	settings								_ D	×
Views:	(Complete setup 👻 View settings 💌 Load defa	ult	Find					Nationa	l annex:	<u>}</u>
D	es	cription	Symbol	Value	Default	Unit	Chapter	Code	Structu	CheckT	
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⊿ D	esi	ign defaults									
Þ	1	Reinforcement									
Þ	Ī	Minimum cover									
⊿ S	olv	versetting									
		General									
		Limit value of unity check	Lim.check	1.0	1.0			Independent	All (Bea	Solver se	
		Value of unity check for not calculated unity check	Ncal.check	3.0	3.0			Independent	All (Bea	Solver se	
		The coefficient for calculation effective depth of cross-sec	Coeff _d	0.9	0.9			Independent	All (Bea	Solver se	
		The coefficient for calculation inner lever arm	Coeff _z	0.9	0.9			Independent	All (Bea	Solver se	>>
		The coefficient for calculation force, where member as u	Coeff _{com}	0.1	0.1			Independent	All (Bea	Solver se	
		Creen and shrinkage									

• Inner lever arm

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

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

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

In those cases, it is calculated according to formula:

 $z = Coeff_z * d$

With:

• Coeff_z by default 0,9 in Concrete settings, in "Complete Setup" view, and in "Solver settings" / "General"

Concrete settings								- 0	×
Views: Complete setup View settings Load def	ault	Find					Nationa	l annex: 🔣	
Description	Symbol	Value	Default	Unit	Chapter	Code	Structu	CheckT	Π
<all></all>	<all> 🔎</all>	<all></all>	<all> 🔎</all>	<,D	<all> 🔎</all>	<all> 🔎</all>	<all> 🔎</all>	<all> 🔎</all>	
Design defaults									
Reinforcement									
Minimum cover									
 Solver setting 									
∡ General									
Limit value of unity check	Lim.check	1.0	1.0			Independent	All (Bea	Solver se	
Value of unity check for not calculated unity check	Ncal.check	3.0	3.0			Independent	All (Bea	Solver se	
The coefficient for calculation effective depth of cross-sec	. Coeff _d	0.9	0.9			Independent	All (Bea	Solver se	
The coefficient for calculation inner lever arm	Coeff _z	0.9	0.9			Independent	All (Bea	Solver se	>>
The coefficient for calculation force, where member as u	Coeff _{com}	0.1	0.1			Independent	All (Bea	Solver se	
Creep and shrinkage									

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

GAPACITY DIAGRAM

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

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

Example: beam_practical reinforcement.esa

Run the Capacity – Diagram check in Design menu > Concrete 1D > ULS capacity diagram check

The standard output gives the summary result of the check:

mma	nmary of check									
N	N _{Ed}	$\mathbf{N}_{\mathrm{Rd}+}$	My	M _{Edy}	M _{Rdy+}	M _{Rdy-}	UC	Status		
		N _{Rd-}	Mz	M _{Edz}	M _{Rdz+}	M _{Rdz-}				
[kN]	[kN]	[kN]	[kNm]	[kNm]	[kNm]	[kNm]	[-]			
0	0	0	-261	-261	119	-278	0.939	ОК		
		0	0	0	0	0		M_{Edz}/M_{Rdz}		

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

Summary of check	
Forces: $N_{Ed} = 0 \text{ kN}$ $M_{Edy} = -261 \text{ kNm}$ $M_{Edz} = 0 \text{ kNm}$	
Resistance: $N_{Rd} = 0 \text{ kN}$ $M_{Rdy} = -278 \text{ kNm}$ $M_{Rdz} = 0 \text{ kNm}$	
Calculation of unity check	
$UC = \frac{\sqrt{N_{Ed}^{2} + M_{Edy}^{2} + M_{Edz}^{2}}}{\sqrt{N_{Rd}^{2} + M_{Rdy}^{2} + M_{Rdz}^{2}}} = \frac{\sqrt{0^{2} + -261^{2} + 0^{2}}}{\sqrt{0^{2} + -278^{2} + 0^{2}}} = 0.939 <= 1 \mathbf{O}$	¢

Interaction diagrams are also drawn in the Detailed output:



Settings that might influence the check:

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

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

SHEAR + TORSION

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

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

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

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

Example: beam_practical reinforcement.esa

Run the Shear + Torsion check in Design > Concrete 1D > ULS Shear and Torsion check

Some parts of the beam do not satisfy:



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

orces						
Content of combination: 1.35*LC1+1.35*LC2+1	1.50*LC3					
N _{Ed} = 0 kN M _{Edy} = 203 kNm M _{Edz} = 0 kNm	V _{Edy} = 0 k	N V _{Edz} = -152	kN T _{Ed}	= 0 kNm		
Resultant of shear force		Difference	between	angles α_M	and α_V	
$V_{Ed} = \sqrt{V_{Edy}^{2} + V_{Edz}^{2}} = \sqrt{0^{2} + -152^{2}} =$	152 kN	α _{MV} = a	bs(α _M –	α_V) = abs(9	0 - 90) = 0 °	
d = 445 mm z = 383 mm b _w = 300 mm b _w	1 = 300 mn	N V _{Rdc} = 87.8	kN V _{Rds}	= 66.5 kN	V _{Edmax} = 705 k	cN V _{Rdmax} = 598 kN
d = 445 mm z = 383 mm b _w = 300 mm b _w Type of check Check shear Vy+Vz	1 = 300 mn Forces 151.7 kN	N V _{Rdc} = 87.8 Resistances 66.5 kN	kN V _{Rds} UC [-] 2.28	= 66.5 kN Status Not OK	V _{Edmax} = 705 k	(N V _{Rdmax} = 598 kN
d = 445 mm z = 383 mm b _w = 300 mm b _w Type of check Check shear Vy+Vz Check torsion	1 = 300 mm Forces 151.7 kN 0.0 kNm	V _{Rdc} = 87.8 Resistances 66.5 kN 0.0 kNm	kN V _{Rds} UC [-] 2.28 0.00	= 66.5 kN Status Not OK OK	V _{Edmax} = 705 k	cN V _{Rdmax} = 598 kN
d = 445 mm z = 383 mm b _w = 300 mm b _w Type of check Check shear Vy+Vz Check torsion Interaction check Vy+Vz+T (concrete)	1 = 300 mm Forces 151.7 kN 0.0 kNm	N V _{Rdc} = 87.8 Resistances 66.5 kN 0.0 kNm	kN V _{Rds} UC [-] 2.28 0.00 0.00	= 66.5 kN Status Not OK OK OK	V _{Edmax} = 705 k	cN V _{Rdmax} = 598 kN
ummary of check d = 445 mm z = 383 mm b _w = 300 mm b _w Type of check Check shear Vy+Vz Check torsion Interaction check Vy+Vz+T (concrete) Interaction check Vy+Vz+T (shear)	1 = 300 mn Forces 151.7 kN 0.0 kNm 0.0 kN	V _{Rdc} = 87.8 Resistances 66.5 kN 0.0 kNm 0.0 kN	KN V _{Rds} UC [-] 2.28 0.00 0.00 0.00	= 66.5 kN Status Not OK OK OK OK	V _{Edmax} = 705 k	cN V _{Rdmax} = 598 kN
ummary of check d = 445 mm z = 383 mm b _w = 300 mm b _w Type of check Check shear Vy+Vz Check torsion Interaction check Vy+Vz+T (concrete) Interaction check Vy+Vz+T (shear) Interaction check Vy+Vz+T (long. reinf.)	1 = 300 mm Forces 151.7 kN 0.0 kNm 0.0 kN 0.0 kN	 V_{Rdc} = 87.8 l Resistances 66.5 kN 0.0 kNm 0.0 kN 0.0 kN 	KN V _{Rds} UC [-] 2.28 0.00 0.00 0.00 0.00	= 66.5 kN Status Not OK OK OK OK OK	V _{Edmax} = 705 k	cN V _{Rdmax} = 598 kN

Here the shear forces cause a unity check >1.

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

Check \	
encer	ramax Vr. = 152 kN \leq Values: + V. + V. = 508 kN
Note:	The check satisfies for crushing of the compression strut ($V_{Ed} \le V_{Rd,max} + V_{td} + V_{ccd}$).
Check \	
	V_{Ed} = 152 kN \leq V_{Edmax} + V_{ccd} + V_{td} = 705 kN
Note	The check satisfies for shear force near the support ($V_{Ed} \leq V_{Ed,max} + V_{td} + V_{ccd}$).
Check \	/ _{Rdc} and V _{Rds}
	V_{Ed} = 152 kN $>$ V_{Rdc} = 87.8kN and V_{Ed} = 152 kN $>$ V_{Rds} + V_{ccd} + V_{td} = 66.5 kN
Error: area	The check does not satisfy, because of shear reinforcement ($V_{Ed} > V_{Rds} + V_{ccd} + V_{td}$). It is necessary to increase of shear reinforcement or to increase dimensions of the cross-section or quality of shear reinforcement.
Unity c	heck
-	$UC = \frac{abs(V_{Ed})}{c} = \frac{abs(152 \text{ kN})}{c} = 2.28$

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

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

REINFORCEM	ENT LAYER (1)	\bowtie
a 3		
Name	RL	
Type of zone	stirrups	
Detailing	0	
Position number	6	
Material	B 500A ∨	=
Calculation of cuts number	User V	
Number of cuts	2.00	
Diameter of mandrel dm =x*ds(s), x =	4.00	
Torsion type	DV	
Anchorage L [mm]	120.00	
Keep formwork		
▼ GEOMETRY		
Test of overlapping stirrups	0	
Member	51	
Whole length beam/span		
Coord. definition	Rela 🗸	
Position x1	0.000	
Position x ₂	1.000	
Origin	From start \checkmark	
 DESCRIPTION POSITIONS 		
Vertical [m]	-0.40	
SCHEME OF REINFORCEMENT		
Horizontal position in X direction [m]	0.00	
Vertical position in Z direction	0.00	
ACTIONS >>>>		
S Edit stirrup shape		
Edit covers		
Edit stirrups distances		

Select "Zone 2" and change the distance between stirrups from 0.3 m to 0.1 m. Apply the same procedure for "Zone 4" and modify the spacing to 0,2 m:



We could also have added more stirrups like below:

000 E 100 E E								;
		2					51 52	
							Delete	Delete all
							Name	52
							Color	
3				1			Number of verte	4
5				•			Closed	~
							Detailing	no
							Torsion	no
		4						
							Analysis mo	
							SHEAR CALCULATI	ION
TIRRUP	JSER DEFINE	D POINTS					Number of cuts	4
New stirrup	em-ed 1 2.Edge	geind∉ Type Re∀	Rela 0.300	ibso [mm	From Begin	*	Diameter of mand	i 4 di
	a set day	Re 🛩	0.300		End	۷	Z Draw intersec	tion points
	Z Z.Edge				Degin	1.64		
Automatic	2 2.Edge 3 4.Edge	Re ¥	0.300		Degin	-	Draw corners	points
Automatic Diameter 8.0 M mm	2 2.Edge 3 4.Edge 4 4.Edge	Re ♥ Re ♥	0.300		End	v	Texts & Points sc	points ε 0.5 ±

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

After modification, the shear + torsion check is satisfied:



Settings that might influence the check:

0

0

- Coefficient for calculation of effective depth of cross-section Default value 0,9 in Concrete settings > Complete Setup view > Solver settings > General
- Coefficient for calculation of inner lever arm Default value 0,9 in Concrete settings > Complete Setup view > Solver settings > General
- Angle of concrete compression strut
 - 3 types of input in Concrete settings > Solver settings > Shear:
 - user input of the angle by default User (angle)
 - User (cotangent) 0 Auto
- user input of the cotangent automatic calculation of the angle fulfilling equation 6.29

Co	ncre	te s	ettings									— D	×
Vie	ws:	Co	omplete setup 👻 View settings 🔻 Load defa	ault	1	Find					Nationa	al annex:	
	D	escr	ription	Symbol		Value	Default	Unit	Chapter	Code	Structu	CheckT	
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	De	sig	n defaults										
	Þ	Re	einforcement										
	Þ	М	inimum cover										
	So	lve	rsetting										
	Þ	G	eneral										
	Þ	In	iternal forces										
	Þ	De	esign As										-
	Þ	Co	onversion to rebars										
	Þ	In	iteraction diagram										>>
	-	Sł	hear	1									
	Е		Type calculation/input of angle of compression strut	Туре Ө		User(angle) 🔺	User(angle)		6.2.3	EN 1992-1-1	All (Bea	Solver se	
	L		Angle of compression strut	θ		Auto	.00	deg	6.2.3	EN 1992-1-1	All (Bea	Solver se	
	L		Cotangent angle of compression strut	$\cot(\theta)$		User(angle)			6.2.3	EN 1992-1-1	All (Bea	Solver se	
	L		Consider effect of axial force in nonprestressed shear che	Typean		User(cotangent	2		6.2.2(1)	EN 1992-1-1	1D (Bea	Solver se	
			Shear between web and flanges										
			Type input of angle of compression strut	Type θ_{f}		User(angle)	User(angle)		6.2.4(4)	EN 1992-1-1	Beam,B	Solver se	
				100		40.00	80.00	291226	CO. 4141	TM toos t t	n	10.00	

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



- Angle of shear reinforcement Practical reinforcement can only be introduced at 90°.
 - Type for determination equivalent thin-walled cross-section

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

STRESS LIMITATION

Stress limitation is based on the verification of:

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

Example: beam_practical reinforcement.esa

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

- Verification of crack appearance
- Verification of the stresses

The Standard output shows those 2 steps:

Load	Type of	Ec	Combi.	N _{Ed}	MEdy	M _{Edz}	σ _{ct}	h	f _{ct,eff}	Cracks
	module	[MPa]		[kN]	[kNm]	[kNm]	[MPa]	[mm]	[MPa]	appear
Short	Ec	0	Char.	0	-188	0	12.6	500	2.9	YES

Stress limitation in concrete

Check type	Load	N _{Ed}	M _{Edy}	M _{Edz}	y i	z _i	σ	σ _{c,lim}	σ _c /σ _{c,lim}	Status
		[KIN]	[KINM]	[KINM]	լՠՠյ	[mm]	[IVIPa]	[IVIPa]	1-1	
§7.2(2) Char.	Short	0	-188	0						OFF
§7.2(3) QP.	Short	0	-188	0	0.15	-0.25	-21.2	-13.5	1.57	Not OK

Stress limitation in non-prestressed reinforcement

Check type	Load	N _{Ed} [kN]	M _{Edy} [kNm]	M _{Edz} [kNm]	y _i [mm]	z _i [mm]	σ _s [MPa]	σ _{s,lim} [MPa]	σ _s /σ _{s,lim} [-]	Status
§7.2(5) Char.	Short	0	-188	0	0.09	0.2	300	400	0.75	ОК

Verification of crack appearance

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

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

With:

•

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

Verification of stresses

There are 3 stress limitations checked:

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

Values of k1, k2, k3, are defined in the NA, standard values are respectively 0.6, 0.45, 0.8

Additionally, when the stress in the reinforcement is caused by an imposed deformation, then the maximal strength is increased to $k_4 * f_{yk}$, where k_4 is NA parameter with standard value $k_4 = 1,0$.

This option can be activated in Concrete settings > Stress limitations:



By default, stress limitation check is done for short-term state.

It is possible to perform a long-term state. Effective E modulus of elasticity is calculated as follows, using the creep coefficient:

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

Long-term behaviour can be activated in Concrete Setting > Complete Setup view > Solver settings > General > SLS > Use effective modulus of elasticity.

The creep coefficient can whether be calculated by the software or inputted manually in the Concrete settings.

s: Co	mplete setup 👻 View settin	ngs 💌	Load defai	ult	Fir	nd	N	ntional a	nnex 🧃		
Descr	iption	Symbol	Value	Defa	U	Chapt	Code	Stru	Chec		Remark
>	Q	<a p<="" td=""><td><all> D</all></td><td>< P</td><td></td><td><a td="" ø<=""><td><all> ρ</all></td><td>< P</td><td>< P</td><td></td><td></td></td>	<all> D</all>	< P		<a td="" ø<=""><td><all> ρ</all></td><td>< P</td><td>< P</td><td></td><td></td>	<all> ρ</all>	< P	< P		
Desig	n de faults										
⊳ Re	inforcement										12210
⊳ M	inimum cover										
Solve	rsetting										1
⊿ Ge	eneral										E
	Limit value of unity check	Lim.ch	1.0	1.0			Indepe	All (B.,	. Solve		Cin
	Value of unity check for not calculated.	. Ncal.c	3.0	3.0			Indepe	All (B.,	Solve		
	The coefficient for calculation effective.	. Coeff _d	0.9	0.9			Indepe	All (B.,	. Solve		
	The coefficient for calculation inner le	Coeffz	0.9	0.9			Indepe	All (B.,	Solve		
	The coefficient for calculation force, w	Coeff _{com}	0.1	0.1			Indepe	All (B.,	. Solve		-
F	Creep and shrinkage									~~	Ecm
	Age of concrete at the moment co	t	1825.00	18250	day	3.1.4.B	EN 1992.	All (B _m	. Solve	- (<u> </u>)	
	Relative humidity	RH	50	50	96	3.1.4.B	EN 1992.	All (B.,	. Solve		1+φ
	Type input of creep coefficient	Туреф	Auto	Auto		3.1.4(2)	EN 1992.	All (B _m	. Solve		
	Age of concrete at loading	t ₀	28.00	28.00	day	3.1.4(2)	EN 1992.	All (B.,	. Solve		Possibility to use effective E modulus of concrete. It
	Consider drying and autogenous s	Typeses	No	Auto		3.1.4(6)	EN 1992.	All (B _m	Solve		means the longterm behaviour of concrete is covere
1	SLS										in the analysis of the crack width and stiffness calculation
. L	Use effective modulus of concrete					7.1(2)	EN 1992	All (B.,	Solve		cheddion
-	Default sway type										
	Sway around y axis	Sway yy		2			Indepe	All (B.,.	. Solve		
_	Sway around z axis	Sway zz		2			Indepe	All (B.,	, Solve		
⊳ In	ternal forces			1					· ·		

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

CRACK WIDTH

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

The following preconditions are used for calculation:

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

Example: beam_practical reinforcement.esa

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

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

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

ews: C	omplete setup 👻 View setting	s ▼ I	load defau	lt	Find		Natio	anal ann	ех:		
Desc	ription	Symbol	Value	Defa	U, C	Chapt	Code	Stru	Chec		Remark
all>	ρ	<al ,<="" td=""><td>) <all> ↓</all></td><td>) < P</td><td><</td><td>:al P</td><td><all> 🔎</all></td><td>< P</td><td>< P</td><td></td><td>Value of strength which is used for calculation of first</td></al>) <all> ↓</all>) < P	<	:al P	<all> 🔎</all>	< P	< P		Value of strength which is used for calculation of first
Desig	gn defaults										 Crack. It is possible to select between 0 MPa – the first crack appears when tensile stress
⊳R	einforcement										occurs in the cross-section
⊳ M	1inimum cover										2) fct,eff - the first crack appears when tensile effective
Solve	ersetting										strength of concrete is reached in the cross-section
ÞG	ieneral										
⊳ h	nternal forces										
⊳ D	Pesign As										
D C	onversion to rebars										
⊳ li	nteraction diagram										
⊳ S	hear										
⊳ T	orsion									<<	
⊳ S	tress limitations				2.1						
- C	racking forces			1							
	Type of strength for calculation of crac	f _{ct,eff}	f _{ctm}	fatm	7.	.1(2)	EN 1992	All (B	Solver		
)	 Value of strength for calculation cracki 		f _{ct,eff}	f _{ot,eff}	7.	.1(2)	EN 1992	All (B	Solver		
D C	rack width										
DD	Deflections										
140.0			-								

Value of strength for calculation of cracking forces:

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

Type of strength for calculation of cracking forces:

If previous option is set on $\sigma_{cr} = f_{ct,eff}$, which is the default value then:

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



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

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

Material characteristics			
Effective strength of concrete:	Modulus of elasticity	of concrete:	
$f_{ct.eff} = f_{ctm} = 2.9 \text{ MPa}$	$E_c = E_{cm} = 33 \text{ GPa}$		
Strength in concrete , when crack is appeared:			
σ_{cr} = 2.9 MPa			
Forces	Cross-sect	ion characteris	tics
Content of combination:	Туре	Css-uncracked	Css cracked
LC1+LC2+LC3	t _{iy} [m]	0	0
Characteristic values	t _{iz} [m]	6.82·10 ⁻³	-0.117
N _{char} = 0 kN M _{y.char} = -188 kNm M _{z.char} = 0 kNm	A _i [m ²]	0.163	0.0533
Quasi-permanent values	l _{iy} [m ⁴]	3.63·10 ⁻³	1.91·10 ⁻³
N _{qp} = 0 kN M _{y,qp} = -188 kNm M _{z,qp} = 0 kNm	$I_{iz}[m^4]$	1.19·10 ⁻³	370·10 ⁻⁶
Angle of bending moment resultant			
α _M = -90°			
Calculation of cracking forces (uncracked sect	ion)		
Maximal stress in concrete			
σ_{ct} = 12.6 MPa			
Cracking forces			
$N_{cr} = 0 \text{ kN}$ $M_{cry} = -43.3 \text{ kNm}$ $M_{crz} = 0 \text{ kNm}$			

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

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

In this example, cracks appear.

Crack width is then calculated according to EN 1992-1-1, formula 7.8:

 σ_{ct} = 12.6 MPa > σ_{cr} = 2.9 MPa = > Cracks appear

$$W = S_{r,max} \bullet (\epsilon_{sm} - \epsilon_{cm})$$

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


Unity check
Calculation unity check
$UC = \frac{W}{W_{max}} = \frac{0.303 \text{ mm}}{0.4 \text{ mm}} = 0.757$
Check crack width
w = 0.303 mm = < w _{max} = 0.4 mm
Note: Check crack width satisfies, because the crack width is lesser than limit value.

Standard output will give the summary values:

Sı	ummary o	f check					Summary of check												
	$N_{cr} = 0 \text{ kN}$	M _{cry} = -43.3 kN	$M_{crz} = 0 \text{ kN}$	σ _s = 300 MPa	$s_{r.max}$ = 232 mm	ε _{sm_cm} = 1.3 %	, 00												
	σ _{ct} [MPa]	σ _{cr} [MPa]	Cracked	w [mm]	w _{lim} [mm]	UC [-]	Limit check [-]	Status											
	12.6	2.9	YES	0.303	0.4	0.76	1	ОК											

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

Manager for National annexes)	×
📑 📲 🗹 🗟 🐟 🗢 🔳 🖨 🖌 All	۷	
Standard EN		Π
Austrian ÖNORM-EN NA		
Belgian NBN-EN NA		
British BS-EN NA		
Cypriot CYS-EN NA		
Czech CSN-EN NA		
Name Standard EN		
National annex Standard EN		
References		
EN 1990: Basis of structural design		
EN 1990 (Basis of structural design)		
EN 1991: Actions of structures		
EN 1991-1-3 (General actions - Snow loads)		
EN 1991-1-4 (General actions - Wind actions)		
EN 1992: Design of concrete structures		٦
EN 1992-1-1 (General rules and rules for buildings)		
EN 1992-1-2 (General rules -Structural fire design)		
EN 1992-2 (Concrete bridges - Design and detailing rules)		
EN 1168 (Precast concrete products – Hollow core slab)		
 EN 1993: Design of steel structures 		
EN 1993-1-1 (General rules and rules for buildings)		
EN 1993-1-2 (General rules - Structural fire design)		
EN 1993-1-3 (General rules - Supplementary rules for cold-f		
EN 1993-1-5 (Plated structural elements)		¥
New Insert Edit Delete	ОК	



You can manually input the limiting crack width in the 1D member data:

CMD	
Name	CMD1D
Member	\$1
Member type	Beam
Design defaults	
 Reinforcement 	
Beam / Rib	
Minimum cover	
Solver setting	
▶ General	
Internal forces	
Design As	
Conversion to rebars	
Interaction diagram	
D Shear	
Torsion	
Stress limitations	
Cracking forces	
Crack width	
Type of maximal crack width	User
User defined crack width [mm]	0.300
Deflections	
Actions	
	Update support width >>>
	Load default values >>>
	OK Cance

DEFLECTION

The calculation of deflection is done according to chapter 7.4.3 from EN 1992-1-1.

Two kinds of deflection calculations are possible in the software:

- Simplified method where the calculation is done twice, assuming the whole member to be uncracked and fully cracked, and then interpolating formula 7.18 according to clause 7.4.3(7). This is the default used method.
- Code dependent deflection. This is the most rigorous method to calculate deflection by computing the calculation of curvatures at frequent sections along the member and then calculate the deflection by numerical integration. More information about this method can be found in the chapter **Code dependent deflections**.

The <u>calculation procedure for the simplified method</u> can be described in the following steps:

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

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

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

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

vs: Complete setup 👻 View settings 👻 🛛 Load defau	lt Fi	nd					Nation	al annex
Description	Symbol	Value	Default	Unit	Chapter	Code	Structure	CheckTy
Q Ib	<all> D</all>	<all> ∫C</all>	<all> D</all>	< P	<all> \wp</all>	<all> \wp</all>	<all> D</all>	<all> ₽</all>
Design defaults								
Reinforcement								
Minimum cover								
Solversetting								
- General								
Limit value of unity check	Lim.check	1.0	1.0			Independent	All (Bea	Solver set
Value of unity check for not calculated unity check	Ncal.check	3.0	3.0			Independent	All (Bea	Solver set
The coefficient for calculation effective depth of cross-secti	Coeffd	0.9	0.9			Independent	All (Bea	Solver set
The coefficient for calculation inner lever arm	Coeffz	0.9	0.9			Independent	All (Bea	Solver set
The coefficient for calculation force, where member as un	Coeff _{com}	0.1	0.1			Independent	All (Bea	Solver set
 Creep and shrinkage 								
Age of concrete at the moment considered	t	1825.00	18250.00	day	3.1.4.B.1-2	EN 1992-1-1	All (Bea	Solver set
Relative humidity	RH	50	50	96	3.1.4.B.1-2	EN 1992-1-1	All (Bea	Solver set
Type input of creep coefficient	$Type \phi(t,\!to)$	Auto	Auto		3.1.4(2)	EN 1992-1-1	All (Bea	Solver set
Age of concrete at loading	t ₀	Auto	28.00	day	3.1.4(2),B1	EN 1992-1-1	All (Bea	Solver set
Consider drying and autogenous shrinkage	Type sos(t,ts	User value	Auto		3.1.4(6)	EN 1992-1-1	All (Bea	Solver set
∠ SLS								
Use effective modulus of concrete					7.1(2)	EN 1992-1-1	All (Bea	Solver set
A Default sway type								
Sway around y axis	Sway yy					Independent	All (Bea	Solver set
Sway around z axis	Sway zz					Independent	All (Bea	Solver set
Internal forces		20 a.T.						

3. Calculation of stiffness ratios between each state, short and long term.

It is the ratio of linear stiffness of the concrete component divided by the resultant stiffness taking cracks into account. The calculation of resultant stiffness is based on clause 7.4.3 (3), formula 7.18. bending stiffness around y-axis (Ely) = $1 / [\zeta/(Ely)_{II} + (1-\zeta) / (Ely)_{I}]$ bending stiffness around z-axis (Elz) = $1 / [\zeta/(Elz)_{II} + (1-\zeta) / (Elz)_{I}]$ axial stiffness (EA) = $1 / [(\zeta/(EA)_{II} + (1-\zeta) / (EA)_{I}]$ In this formula (EI)_I is the linear stiffness, (EI)_{II} is the stiffness of the cracked element (= long term stiffness = $E_{lin} / 1 + \phi$) and ζ is the distribution coefficient.

$$\zeta = 1 - \beta \left(\frac{\sigma_{\rm sr}}{\sigma_{\rm s}}\right)^2$$

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

4. Calculation of deflection components

Several components are needed to calculate the total and additional deflection. In the following part we will note "s" for short term and "l" for long term. The components are:

- δ_{lin} linear (elastic) deflection, $\delta_{\text{lin}} = \delta_{\text{lin,s}} + \delta_{\text{lin,l}}$
- δ_{imm} immediate deflection, $\delta_{imm} = \delta_{lin,l} \cdot ratio_s$
- δ_s short-term deflection, $\delta_s = \delta_{\text{lin},s} \cdot \text{ratio}_s$
- $\delta_{l,creep}$ long-term deflection + creep, $\delta_{l,creep} = \delta_{lin,l} \cdot ratio_{l}$
- δ_{creep} creep deflection, $\delta_{creep} = \delta_{lin,l} \cdot (ratio_l ratio_s)$
- δ_{l} long-term deflection, $\delta_{l} = \delta_{l,creep} \delta_{creep}$
- δ_{add} additional deflection, $\delta_{add} = \delta_s + \delta_{I,creep} \delta_{imm}$
- δ_{tot} total deflection, $\delta_{tot} = \delta_s + \delta_{l,creep}$

5. Check of deflections

Two deflections are checked:

Total deflection: The appearance and general utility of the structure could be impaired when the calculated sag of a beam, slab or cantilever subjected to quasi-permanent loads exceeds span/250.

$$\delta_{tot,lim} = L / 250$$

Additional deflection: Deflections that could damage adjacent parts of the structure should be limited. $\delta_{add,lim} = L / 500$

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

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

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

	escription	Syml	lool	Value	Default	Unit	Chapter	Code	Structure	CheckTy
>	Q	<all></all>	ρ	<all></all>	<all></all>	<p< th=""><th><all></all></th><th><all> ♀</all></th><th><all> ρ</all></th><th><all> \wp</all></th></p<>	<all></all>	<all> ♀</all>	<all> ρ</all>	<all> \wp</all>
D	esign defaults									
P	Reinforcement									
Þ	Minimum cover									
s	olversetting									
D	General									
P	Internal forces									
D	Design As									
P	Conversion to rebars									
D	Interaction diagram									
P	Shear									
Þ	Torsion									
P	Stress limitations									
Þ	Cracking forces									
P	Crack width									
ł	Deflections									
	Coefficient for increasing the amount of reinforcement	Coeff	reinf	1.0	1.0			Independent	All (Bea	Solver set
	Maximal total deflection L/x; x =	×tot		250.0	250.0		7.4.1(4)	EN 1992-1-1	1D (Bea	Solver set
	Maximal additional deflection L/x; x =	Xadd		500.0	500.0		7.4.1(5)	EN 1992-1-1	1D (Bea	Solver set
	Type of variable load coefficient for the automatic generati			Use Psi2 factor	Use Psi2 f			Independent	All (Bea	Solver set
P	Detailing provisions			Ξ.						

Example: beam_practical reinforcement.esa

Look at deflection check for the "SLS qp" combination.

Various results can be displayed on the screen: UC, total and additional deflection or limits for total and additional deflection.

Open the Standard output for the UC. At position dx = 2,5 m we have the following result:

Type of deflection	Ratio short	• R :[-]	atio ong [-]	δ _{lin} [mm]	δ _{imm} [mm]	δ _{add} [mm]	δ _{short} [mm]	δ _{long} [mm]	δ _{long+cre}	_{ep} δ _{creep}
u _y	2	2.88	5.22	0	0	0	0	0		0 (
U _z		2.5	3.38	-3.08	-7.7	-2.69	0	-7.7	-10	.4 -2.6
eck of ad	dition	nal an	d total	defle	ctions					0 (0 10.4 -2.69
eck of ad Type of deflection	dition L [m]	nal an δ _{add} [mm]	d total δ _{add.lim} [mm]	defle UC _{add} [-]	ctions δ _{tot} [mm]	δ _{tot.lim} [mm]	UC _{tot}	UC [-]	Limit [-]	Status
eck of ad Type of deflection	dition L [m] 10	hal an δ _{add} [mm] 0	d tota δ _{add,lim} [mm]	defle UC _{add} [-] 0	ctions δ _{tot} [mm] 0	δ _{tot,lim} [mm]	UC _{tot} [-]	UC [-]	Limit [-] 1	Status OK

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

Open the Detailed output, for the same position dx = 2,5 m.

All previously mentioned steps for the calculation of the deflections can be found here.

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

Long-term stiffne	sses and curvatures under total load
Settings	
Long-term part of applied	pad = 100%
Creep coefficient $\phi = 2.21$	
Uncracked (state I) and cracked (state II) cross section properties are also shown in a table

$\begin{array}{ c c c c c c } \hline linear & 0 & 0 & 0.15 & 3.13 \cdot 10^{-3} & 1.13 \cdot 10^{-3} & 0.25 & - & - & - & - & - & - & - & - & - & $	Type of compone	t _y nt [m]	t _z [m]	A [m ²]	l _y [m ⁴]	l _z [m ⁴]	x _i [m]	A _{st} [m ²]	A _{sc} [m ²]	A _s [m ²]		
$\begin{array}{ c c c c c c c c c c c c c c c c c c c$	Linear	0	0	0.15	3.13.10-3	1.13.10-3	0.25	-				
Cracked00.0530.1022.89 \cdot 10^{-3}667 \cdot 10^{-6}0.1971.57 \cdot 10^{-3}628 \cdot 10^{-6}2.2 \cdot 10^{-3}eck of concrete stresses and calculation of cracking forcesMaximal tensile stress in concrete fibre $\sigma_{ct} = 7.76$ MPatracking status $\sigma_{ct} > f_{cteff} = 7.76$ MPa > 2.9 MPa $= > Cracks$ appear.tress in reinforcement for cracking load $\sigma_{sr} = 99.3$ MPatress in reinforcement for acting load $\sigma_{s} = 262$ MPavistribution coefficient $\zeta = max \left(0:1 - \beta \cdot \left(\frac{\sigma_{sr}}{\sigma_{s}} \right)^2 \right) = max \left(0:1 - 0.5 \cdot \left(\frac{99.3}{262} \right)^2 \right) = 0.928$ Ncr.My, crMz, cr σ_{ct} f_{cteff} Cracked section σ_{sr} β ζ E_c [KNm][KNm][MPa][MPa][I-1][GPa]	Uncracked	0 1	-0.019	0.193	4.69-10 ⁻³	1.35.10-3	0.27	1.57.10 ⁻³	628·10 ⁻⁶	2.2.10	3	
eck of concrete stresses and calculation of cracking forces Aaximal tensile stress in concrete fibre $\sigma_{ct} = 7.76 \text{ MPa}$ tracking status $\sigma_{ct} > f_{cteff} = 7.76 \text{ MPa} > 2.9 \text{ MPa} => Cracks appear.$ tress in reinforcement for cracking load $\sigma_{sr} = 99.3 \text{ MPa}$ tress in reinforcement for acting load $\sigma_{s} = 262 \text{ MPa}$ Vistribution coefficient $\zeta = \max \left(0;1 - \beta \cdot \left(\frac{\sigma_{sr}}{\sigma_{s}} \right)^2 \right) = \max \left(0;1 - 0.5 \cdot \left(\frac{99.3}{262} \right)^2 \right) = 0.928$ N _{cf.} M _{ly,cr} M _{z,cr} σ_{ct} f_{cteff} Cracked section σ_{sr} σ_{s} β ζ E _c [MPa] [MPa] [-1] [-1] [GPa]	Cracked	0	0.053	0.102	2.89.10 ⁻³	667·10 ⁻⁶	0.197	1.57.10 ⁻³	628·10 ⁻⁶	2.2.10	8	
Distribution coefficient $\zeta = \max\left(0; 1 - \beta \cdot \left(\frac{\sigma_{sr}}{\sigma_s}\right)^2\right) = \max\left(0; 1 - 0.5 \cdot \left(\frac{99.3}{262}\right)^2\right) = 0.928$ $\boxed{\begin{array}{ccccccccccccccccccccccccccccccccccc$	Cracking sta σ _{ct} > Stress in reii σ _{sr} =	tus f _{ct,eff} = 7 nforcemer 99.3 MPa	.76 MPa = nt for crac	> 2.9 MP king load	a => Crao	cks appear.						
[kN] [kNm] [kNm] [MPa] [MPa] [MPa] [MPa] [-] [-] [GPa]	itress in rei σ _s =	nforcemei 262 MPa	nt for actir	ng load								
	tress in reli $\sigma_s =$ Distribution $\zeta = n$ N _{cr.}	nforcemen 262 MPa coefficien nax (0;1 – My,cr	It for actin $\beta \cdot \left(\frac{\sigma_{sr}}{\sigma_s}\right)$ $M_{z,cr}$	$\left(\frac{2}{\sigma_{ct}}\right) = \max$	(0;1 - 0.5 · f _{ct.eff}	$\left(\frac{99.3}{262}\right)^2 = \mathbf{Cracke}$	0.928 ed secti	on σ_{sr}	σ,	β	ζ	Ec

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



And final the short and long-term ratios:



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

Deflections

Linear deflection

$$\begin{split} \delta_{lin,y} &= u_{ys} + u_{yl} = 0 + 0 = 0 \ mm \\ \delta_{lin,z} &= u_{zs} + u_{zl} = 0 + -3.08 = -3.08 \ mm \end{split}$$

Immediate deflection

```
δ_{imm,y} = u_{yl} \cdot ratio_{uys} = 0 \cdot 2.88 = 0 mm

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

Short-term deflection

$$\begin{split} \delta_{short,y} &= u_{ys} \cdot ratio_{uys} = 0 \cdot 2.88 = 0 \ mm \\ \delta_{short,z} &= u_{zs} \cdot ratio_{uzs} = 0 \cdot 2.5 = 0 \ mm \end{split}$$

Long-term + creep deflection

 $\delta_{long,creep,y} = u_{yl} \cdot ratio_{uyl} = 0 \cdot 5.22 = 0 \text{ mm}$ $\delta_{long,creep,z} = u_{zl} \cdot ratio_{uzl} = -3.08 \cdot 3.38 = -10.4 \text{ mm}$

Creep deflection

$$\begin{split} \delta_{creep,y} &= u_{yl} \cdot \left(\mathsf{ratio}_{uyl} - \mathsf{ratio}_{uys} \right) = 0 \cdot \left(5.22 - 2.88 \right) = 0 \text{ mm} \\ \delta_{creep,z} &= u_{zl} \cdot \left(\mathsf{ratio}_{uzl} - \mathsf{ratio}_{uzs} \right) = -3.08 \cdot \left(3.38 - 2.5 \right) = -2.69 \text{ mm} \end{split}$$

Long-term deflection

$$\begin{split} \delta_{long,y} &= \delta_{long,creep,y} - \delta_{creep,y} = 0 - 0 = 0 \ mm \\ \delta_{long,z} &= \delta_{long,creep,z} - \delta_{creep,z} = -10.4 - -2.69 = -7.7 \ mm \end{split}$$

Additional deflection

$$\begin{split} \delta_{add,y} &= \delta_{short,y} + \delta_{long,creep,y} - \delta_{lmm,y} = 0 + 0 - 0 = 0 \ mm \\ \delta_{add,z} &= \delta_{short,z} + \delta_{long,creep,z} - \delta_{lmm,z} = 0 + -10.4 - -7.7 = -2.69 \ mm \end{split}$$

Limit additional deflection

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

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

Total deflection

$$\begin{split} \delta_{toty} &= \delta_{shorty} + \delta_{long,creep,y} = 0 + 0 = 0 \text{ mm} \\ \delta_{totz} &= \delta_{shortz} + \delta_{long,creepz} = 0 + -10.4 = -10.4 \text{ mm} \end{split}$$

Limit total deflection

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

Limitations of the deflection check:

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

DETAILING PROVISIONS

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

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

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



Following table shows which checks of detailing provisions are performed:

Member type	Longitudinal (main)	Shear (transverse)
Beam	 8.2(2) - Minimal clear spacing of bars 9.2.1.1(1) - Minimal area of longitudinal reinforcement 9.2.1.1(3) - Maximal area of longitudinal reinforcement 9.2.3(4) - Maximal center-to-center bar distance based on torsion Code-Independent - Maximal clear spacing 	 6.2.3(3) - Maximal percentage of shear reinforcement 9.2.2(5) - Minimal percentage of shear reinforcement 9.2.2(6) - Maximal longitudinal spacing of stirrups (shear) 9.2.2(8) - Maximal transverse spacing of stirrups (shear) 9.2.3(3) - Maximal longitudinal spacing of stirrups (torsion)
Column	 8.2(2) - Minimal clear spacing of bars 9.5.2(1) - Minimal bar diameter of longitudinal reinforcement 9.5.2(2) - Minimal area of longitudinal reinforcement 9.5.2(3) - Maximal area of longitudinal reinforcement 9.5.2(4) - Minimal number of longitudinal reinforcement bars 	9.2.3(3) - Maximal longitudinal spacing of stirrups (torsion) 9.5.3(1) - Minimal diameter of transverse reinforcement 9.5.3(3) - Maximal longitudinal spacing of transverse reinforcement
Beam Slab	8.2(2) - Minimal clear spacing of bars 9.3.1.1(3) - Maximal bar distance of longitudinal reinforcement	-

SECTION CHECK

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

Section check can be launched:

• In the properties window for an individual check



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

			INPUT PANEL	💼 All workstations 🖂		i 🖗 😼	
			All categories 🗸	🥔 Basic modelling 🗸		RESUL	.TS (1) 🔰
			1D member	Ctri+8		Name	Section Check - results
			🥽 Beam			▼ SELECTION	
						Type of selection	All \checkmark
	WF_					▼ RESULT CASE	
	410					Type of load	Combinations V
	18					Combination	ULS \lor
	~				6	▼ OUTPUT SETTINGS	
	H				11	Output	Brief \checkmark
					LIF	Print combination key	
	<i>a</i>				1	Print checks per section	
	~			ي الله الله الله الله الله الله الله الل		▼ CHECKS	
						Capacity-response (ULS)	
						Capacity-diagram (ULS)	
					47	Shear+Torsion (ULS)	
	T					Detailing provision	
	-1 L				U ^r	ACTIONS >>>>	
					Lii (ii	Refresh	
	*					Section Check	
					=	Preview	
	14				G		
14	🕅 📦 📜	SECTION CHECK RESULTS			6		
100		PaP F K A			0 0 8		
(A)	- FM				increase in the second		
X	C. Con						

⇒ With practical reinforcement

Example 1: beam_practical reinforcement SC.esa

Section check can be opened from all individual checks.

In this example, select Design > Concrete 1D > SLS reinforcement stress limitation check (SLS) and click on "Section check" in the Properties window:

Select the beam and then click on the position for which the check should be done. Choose section 20 at the middle of the beam:



The Section check tool opens:

									Section Check (tool)						
Home															
				Ľ				Restore default	Save & close						
Section	Linfo	ongitudinal re	einforcemen	t			Stirrups		Application			Check			
t E	Grid s	ize: 100	nm / 4				Chendred	Check: Stress limitatio	in (SLS)	Check value: 0 00		E	Name	Value	Status
-		p	-10 9				Standard	Extreme : SLS/2 [SLS]		0.89		Inte	ernal forces (check)		
		_		_							^	Cap	pacity-response (ULS)		
	20				300						- 11	Cap	oacity-diagram (ULS)		
					10		S	ection SC1		RECT (500; 300)		She	ear+Torsion (ULS)		
			¥				EC	EN 1992-1-1:2004/AC:	2008	Beam S1 [dx = 5 m]	_	Stre	ess limitation (SLS)	0.89	4
			+ ,				M	Buckling v-v-	L = 10 m $L_v = 10 \text{ m} (swav)$	Bi-linear stress-strain diagram		Cra	ck width (SLS)		
					-10			Buckling z-z	$L_z = 10 \text{ m} \text{ (sway)}$	Exposure class: XC3		Def	flection (SLS)		
							4			Longitudinal reinforcement: B 500A		Det	tailing provisions		
	4	0	<u>e</u>	_	-30				ο 5φ20 (1571 mm2)	$7620 \text{ mm} (A_{2} = 2199 \text{ mm}^{2})$		Extreme			
	4	8								ρ _l = 1.466 % (17.3 kg/m)			Name	Value	Status
Desireday	and the second	AN Deleter	and the st	1						Shear reinforcement: B 500A		🕚 SLS	/2 (SLS)	0.89	4
Reinio	cement (layo	uu Nemioro	ement (iree)					t ^z		Bi-linear with an inclined top branch		* SLS	i/1 (SLS)	0.79	4
Long	itudinal						22	+→ _y		ϕ 10/99.7 mm (n _s = 2) (A _{sw} = 157 mm)					
	Bar	Y (mm)	Z [mm]	Diameter Ø [mm]	Material	Detailing				p _w = 1.050 % (12.4 kg/m) (A _{swm} = 1575) Cover (stirrup)	nu.				
-	BO	90	195	20	8 500A	E				Bottom: 36 mm					
-	B1	-90	195	20	8 500A				2¢20 (628 mm2)	Left: 36 mm					
-	B2.	-90	-195	20	8 500A		4			Right: 36 mm					
-	83	90	-195	20	8 500A			300	φ10/100 mm, ns=2						
-	84	45	195	20	B 500A			21							
-	B5	0	195	20	8 500A		more					4	8		•
-	B6	-45	195	20	B 500A		•				-	Overall che	eck status:	0.00	
						County .					100	Satisfied		0.89	

This window is composed of 3 mains parts:

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

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

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

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

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



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

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

Section Info	Report		Check	
늘 💭 🎬 Grid size: 100 mm / 4	Standard Check: Stress limitation (SLS)	Check value: 0.76	(E) Name	Value Statu:
4° ° ° ° ° ° ° %	Extreme : SLS/2 [SLS]		Internal forces (check)	
		1	Capacity-response (ULS)	
* *****	a .:		Capacity-diagram (ULS)	
	Section SC1	RECT (500; 300)	Shear+Torsion (ULS)	
i i i i i i i i i i i i i i i i i i i	EC EN 1992-1-1:2004/AC:2008	Beam S1 [dx = 5 m]	Stress limitation (SLS)	0.76 🖌
· · · · · · · · · · · · ·	Member length: L = 10 m	Concrete: C30/37	Crack width (SLS)	
	Buckling $z-z^{\perp}$ L _y = 10 m (sway) Buckling $z-z^{\perp}$ L _y = 10 m (sway)	Exposure class: XC3	Deflection (SLS)	
	+	Longitudinal reinforcement: B 500A	Detailing provisions	
• [*****	5¢25 (2454 mm2)	Bi-linear with an inclined top branch	Extreme	
		$p_1 = 2.055 \% (24.2 \text{ kg/m})$	Name	Value Statu:
		Shear reinforcement: B 500A	1 SLS/2 (SLS)	0.76
Longitudinal 4	8	Bi-linear with an inclined top branch $d_{10}/99.7 \text{ mm}(n_{1} = 2) (A_{1} = 157 \text{ mm}^{2})$	± SLS/1 (SLS)	0.68 🖌
	S Ty	$\varphi_{10}(3,5,7) = 1,050 \% (12,4 kg/m) (A_{max} = 1575 m)$		
Bar Y [mm] Z [mm] Ø [mm] Material Detailing		Cover (stirrup)		
📟 B0 90 195 25 B 500A 🖾		Top: 36 mm		
📟 B1 -90 195 25 B 500A 🗐	2420 (628 mm ²)	Left: 36 mm		
📟 B2 -90 -195 20 B 500A 🕅	2420 (020 minz)	Right: 36 mm		
📟 B3 90 -195 20 B 500A 🕅	\$10/100 mm, ns=2			
🚥 84 45 195 25 8.500A 🛄	× 300 ×			
📟 85 0 195 25 8 500A 🖾				
= 86 -45 195 25 8 500A	4	, ''	<u>*</u>	•
1 New 8.500A		e e	Overall check status: Satisfied	0.76 ؇
Ready	B.,			

⇒ Without practical reinforcement

Example 2: beam_without practical reinforcement SC.esa

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

In the Concrete menu, select "Section check results".

In the properties window, choose the ULS combination to perform all ULS checks:

	(I) N
Name	Section Check - results
▼ SELECTION	
Type of selection	All 🗸
 RESULT CASE 	
Type of load	Combinations 💛
Combination	ULS 🗸
 OUTPUT SETTINGS 	
Output	Brief \sim
Print combination key	
Print checks per section	
▼ CHECKS	
Capacity-response (ULS)	
Capacity-diagram (ULS)	
Shear+Torsion (ULS)	
Detailing provision	
	and the second se
ACTIONS >>>	

Select Section 9, in the middle of the first span.

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

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



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

ection	fo				
, 🗐 🖩	Grid size:	100 mm /	4		
	,	00000000	• • • •	au ac	
		»	2Ø16	26	
		0		- 10	
	0	- 25 >	Î.		
				-100	
			2020	-30	
			25		
einforceme	ent (layout)	leinforcement	(free)		
Longitudi	na				
-	i ci i				
Layer	Position	Bars N x	Diameter Ø [mm]	Material	Detailing
Layer L1	Position top	Bars N x 2	Diameter Ø [mm] 16	Material B 500A	Detailing
Layer L1 L2	Position top bottom	Bars N x 2 2	Diameter Ø [mm] 16 20	Material B 500A B 500A	Detailing
Layer L1 L2 Shear	Position top bottom	Bars N x 2 2	Diameter Ø [mm] 16 20	Material B 500A B 500A	Detailing
Layer L1 L2 Shear Stirrup	Position top bottom	Bars N x 2 2 Diameter Ø [mm]	Diameter Ø [mm] 16 20 Spacing s [mm]	Material B 500A B 500A Material	Detailing

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

The results for all ULS checks are now:



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



A label will then be added on the beam:

N ŵ	- 501	Ì	
	1000		
		\land	\square

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



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

2.3 Column design

2.3.1 Reinforcement design methods

For column design, there are 3 types of calculation:

- Axial compression only
- Uniaxial bending
- Biaxial bending

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

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

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

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



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

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

DESIGN WITH AXIAL COMPRESSION ONLY



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

Example: <u>Axial compression only.esa</u>

Studied column: B1

Geometry

Column cross-section: RECT 350x350 mm²

Height: 4,5 m

Concrete grade: C45/55

Concrete Setup

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

/iews: Internal	forces 👻 View settings 💌 Load defa	ult	Find						National annex:		
Description	i i i i i i i i i i i i i i i i i i i	Symbol	N	/alue	Default	Unit	Chapter	Code	Structu	CheckT	
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Solver sett	ling										
🔺 Interna	l forces		1								
She	ar force reduction above supports		1				6.2.1(8)	EN 1992-1-1	Beam,B	Solver se	
Mor	nent reduction above supports		Ū.				5.3.2.2 (4)	EN 1992-1-1	Bearn,B	Solver se	
Shif	ting of moment curve to cover additional tensile forc						9.2.1.3(2)	EN 1992-1-1	Beam,Ri	Solver se	
Geo	metric imperfection in ULS	ei.ULS			2		5.2(2)	EN 1992-1-1	Column	Solver se	
Geo	metric imperfection in SLS	ei.sls					5.2(3)	EN 1992-1-1	Column	Solver se	
Mini	imum eccentricity	e _{min}	h	n first order	In first or		6.1(4)	EN 1992-1-1	Column	Solver se	
▶ First	order eccentricity with the equivalent moment		TE				5.8.8.2(2)	EN 1992-1-1	Column	Solver se	
Sec	ond order eccentricity	e ₂			2		5.8.8	EN 1992-1-1	Column	Solver se	
⊿ Inte	rnal forces modifications		-								
	Limit ratio for uniaxial method	Plim	0	.10	0.10	2		Independent	1D (Bea	Solver se	
Þ	Beam							12920-927-046 (1966)0-			
	Column		-								

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

Concre	te se	ttings														— C	נ	×
Views:	Сог	mplete s	setup 👻	View settings 💌	Load defa	ault	F	Find							Nationa	al annex:	\odot	
D	escri	ption				Symbol		Value		Default		Unit	Chapter	Code	Structu	CheckT		
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V	UR	ack wid	in i						_		_							
⊳	De	flectior	rs														- 1	
- 4	De	tailing	provisions					-									- 1	
	⊳	Beam	/ Rib					-									- 1	
	⊳	Beam	slab					-										
		Colum	n					-										
		I Lor	ngitudinal					-										
			Check min. bar distan	ce			Г			~			8.2(2)	EN 1992-1-1	Column	Solver se		
			Check max. bar distar	nce			Т			~				Independent	Column	Solver se		>>
			Check max. bar distar	nce (torsion)						 Image: A set of the set of the			9.2.3(4)	EN 1992-1-1	Column	Solver se		
			Check min. reinforcen	nent area						~			9.5.2(2)	EN 1992-1-1	Column	Solver se		
			Check max. reinforcer	ment area						~			9.5.2(3)	EN 1992-1-1	Column	Solver se		
			Check min. bar diame	ter					_				9.5.2(1)	EN 1992-1-1	Column	Solver se		
			Check min. number of	fbars									9.5.2(4)	EN 1992-1-1	Column	Solver se		
		⊿ Tra	nsverse					-									-11	
			Check max. percentag	ge of stirrups					_				6.2.3(3)	EN 1992-1-1	Column	Solver se		
			Check min. mandrel d	liameter									8.3(2)	EN 1992-1-1	Column	Solver se		
			Check max. longitudir	nal spacing									9.5.3(3)	EN 1992-1-1	Column	Solver se	_	
		•	Check min. bar diame	ter									9.5.3(1)	EN 1992-1-1	Column	Solver se	_	

Loads

- LC1: Permanent load > F = 1100 kN
- LC2: Variable load > F = 1000 kN

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

Combination according to the Eurocode:

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

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

Bar diameter

The bar diameter is taken from the Concrete Settings > Complete setup View, or from 1D member data if applied (1D member data always overwrite the Concrete Settings data, for the specific member they are assigned to).

Load dera	uit	Find					Nationa	a annex:	
Description	Symbol	Value	Default	Unit	Chapter	Code	Structu	CheckT	
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Design defaults									
A Reinforcement									
Beam / Rib									
▷ Beam slab									
Column			-						1
Design of provided reinforcement		2	2			Independent	Column	Design d	L
Rectangular section		Column	Column			Independent	Cəlumn	Design d	L
Gircular		Column	Column			Independent	Column	Design d	L
Oval		Column	Column			Independent	Cəlumn	Design d	L
Other and general		Column	Column			Independent	Column	Design d	L
 Longitudinal 									L
 Main (m) 									L
Type of cover		Auto	Auto		4.4.1	EN 1992-1-1	Column	Design d	L
Diameter	d _{s,m}	16.0	16.0	mm		EN 1992-1-1	Column	Design d	L
Detailing (det)									L
✓ Stirrups (sw)									
Diameter	d _{ss}	8.0	8.0	mm		EN 1992-1-1	Column	Design d	
Number of cuts	ns	2.0	2.0			Independent	Column	Design d	

By default, the diameter for the main column reinforcement is put to ϕ 16mm. 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.



<u>Note:</u> To change the default diameter from ϕ 16mm to ϕ 20mm for example, edit the template "Column_Rect_Empty" (or the corresponding empty template for the specific columns shape), and change the value of the diameter to be taken into account (additional provided reinforcement).

Results

Go to Steel workstation > 1D Reinforcement design:



Ask the value of As reg for member B1, and click the action button [Refresh].

RESUL	TS (1)	3
Name	Overall Design (ULS)	
SELECTION		
Type of selection	Current \sim	
Filter	No 🗸	
Results in sections	All \sim	
RESULT CASE		
Type of load	Combinations \checkmark	
Combination	ULS \vee	
 EXTREME 1D 		
Extreme 1D	Global \checkmark	
Type of values	Required \vee	
Values	As,req 🗸	
Interval	0	
 OUTPUT SETTINGS 		
Output	Brief 🗸	
DRAWING SETUP 1D		
ERRORS, WARNINGS AND NOTES S	SETTINGS	
Run using Model Data files (Debug)	\bigcirc	
ACTIONS >>>>		
Refresh		
Edit provided reinforcement tem	iplate	
Oncrete setup		
Preview		

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

Overa Linear cal Combinat Coordinat Extreme Selection	culation ion: CO1 te system 1D: Glob All	esign I 1: Princip al auired r	(ULS)	ent							
Name	dx [m]	Case	Member	Asz_req+ [mm ²] Asz_req_bar+ [mm ²]	Asz_req- [mm ²] Asz_req_bar- [mm ²]	Asy_req+ [mm ²] Asy_req_bar+ [mm ²]	Asy_req- [mm ²] Asy_req_bar- [mm ²]	A _{sz_req} [mm ²] A _{sz_req_bar} [mm ²]	A _{sy_req} [mm ²] A _{sy_req_bar} [mm ²]	As_req [mm ²] As_req_bar [mm ²]	ReinfReq
B1	0,000	CO1	Column	0	0	0	0	0	0	0	
Shear reinforcement											
Name	dx [m]	Case	Member	A _{swm_req} [mm²/m]	A _{swm_prov} [mm²/m]	ShearReinf					
B1	0.000	CO1	Column	0	0						

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

RESUL	TS (1)	5
Name	Overall Design (ULS)	
SELECTION		
Type of selection	Current 💛	
Filter	No 🗸	
Results in sections	All \sim	
RESULT CASE		
Type of load	Combinations \checkmark	
Combination	ULS \lor	
EXTREME 1D		
Extreme 1D	Global \checkmark	
Type of values	Required \lor	
Values	As,req \vee	
Interval	\bigcirc	
OUTPUT SETTINGS		
Output	Detailed \lor	
DRAWING SETUP 1D		
 ERRORS, WARNINGS AND NOTES S 	SETTINGS	
Run using Model Data files (Debug)	\bigcirc	
ACTIONS >>>>		
Refresh		
Edit provided reinforcement tem	iplate	
Concrete setup		
~		

Explanation errors/warnings and notes

Index	Туре	Description	Solution
N1 /1	Nota	Statically required reinforcement: The	
IN I/ I	Note	reinforcement is not neccessary.	
		Shear design: Design is not done because	

<u>Remark</u>: this result is obtained only because **all detailing provisions are deactivated** in the Concrete Settings!

Check of reinforcement

 $N_{Rd} = f_{cd} \cdot \alpha \cdot A_c = 30 * 1 * 350^2 / 1000 = 3675 kN$

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

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

Example: Axial compression only.esa

Studied column: B2

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

Loads

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

Combination according to the Eurocode:

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

Design normal force $N_{Ed} = 1,35 * 2000 + 1,50 * 1000 = 4200 kN$

Results

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



Overa Linear cal Combinat	LI Des	sign	(ULS)								
Coordinat Extreme 1	e system 1D: Glob	1: Princip al	bal								
Selection:	B2 linal rec	wired r	einforceme	nt							
Name	dx	Case	Member	Asz_req+	Asz_req-	A _{sy_req+}	A _{sy_req} -	Asz_req	Asy_req	As_req	ReinfReq
	[m]			[mm²] Asz_req_bar+	[mm²] Asz_req_bar-	[mm²] Asy_req_bar+	[mm²] Asy_req_bar-	[mm ²] Asz_req_bar	[mm²] Asy_req_bar	[mm²] As_req_bar	
				[mm²]	[mm ²]	[mm²]	[mm²]	[mm²]	[mm²]	[mm²]	
B2	0.000	ULS	Column	375	375	375	375	750	750	1500	[z]6 ф16 ,
				402	402	402	402	804	804	1608	[y]6 φ16
Shear re	inforce	ment									
Name	dx	Case	Member	Aswm_req	Aswm_prov	ShearReinf					
B2	[m] 0.000	UIS	Column	[mm²/m]	[mm²/m]						
02	0.000	1025	Column	U	0						

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

Luon: A										
Col	umn l	B1				Rec	tangle	(350; 3	50)	
EC EN 1	992-1-1:2	004/AC:20	800			Secti	on 0 [dx =	0 m]		
Mem	ber length		Ld = -	4.5 m		Ma	terials			
Buck	ing length	у	Ly = 9	9.01 m		Cond	rete	C45,	/55	
Buck	ing length	z	Lz = 9	9.01 m		Reinf	forcement	B 50	OB	
Longit	udinal r	einforce	ment					Shear	reinfor	cement
φ = 1	6 mm, c =	30 mm,						n _{s.req}	= 2, φ _{s.req}	= 8 mm, α _{sreg} = 90 °
	• I =	dinal re	inforcer	nent						
sign o A _s : 1.35 [*] quired	Leves	*LC2 : N _{Ed}	= -4200 kl	N, M _{Edy} =	0 kNm, M A _{s.det.min}	_{Edz} = 0 kN	m ΔA _{s.tor}	A _{s.req}	A _{s.req.bar}	Delet
sign o A _s : 1.35* quired Edge	Layer	*LC2 : N _{Ed}	= -4200 kl	N, M _{Edy} = A _{s.stat} [mm ²]	0 kNm, M A _{s.det.min} [mm ²]	Edz = 0 kN A _{s.det.max} [mm ²]	m ΔA _{s.tor} [mm ²]	A _{s.req} [mm ²]	A _{s.req.bar} [mm ²]	Reinf
sign o A _s : 1.35 ^s quired Edge 1	Layer	y [m]	z [m] -0.129	N, M _{Edy} =	0 kNm, M A _{s.det.min} [mm ²] 0	Edz = 0 kN A _{s.det.max} [mm ²] 0	m ΔA _{s.tor} [mm ²] 0	A _{s.req} [mm ²] 375	A _{s.req.bar} [mm ²] 402	Reinf 3¢16
sign o A _s : 1.35* quired Edge 1 2	Layer 1	y [m] 0 0.129	z [m] -0.129 0	N, M _{Edy} = A _{s.stat} [mm ²] 375 375	0 kNm, M A _{s.det.min} [mm ²] 0 0	Edz = 0 kN A _{s.det.max} [mm ²] 0 0	ΔA _{s.tor} [mm ²] 0 0	A _{s.req} [mm ²] 375 375	A _{s.req.bar} [mm ²] 402 402	Reinf 3¢16 3¢16
sign o A _s : 1.35* quired Edge 1 2 3	Layer 1 1	y [m] 0 0.129 0	z [m] -0.129 0.129	N, M _{Edy} = A _{s.stat} [mm ²] 375 375 375	0 kNm, M A _{s.det.min} [mm ²] 0 0 0 0	Edz = 0 kN A _{s.det.max} [mm ²] 0 0 0	m ΔA _{s.tor} [mm ²] 0 0 0	A _{s.req} [mm ²] 375 375 375	A _{s.req.bar} [mm ²] 402 402 402	Reinf 3\$\phi16 3\$\phi16 3\$\phi16



Explanation of the number of reinforcement bars

Default bar diameter has been set to $\phi 16$ in Design default.

The table indicates that each edge needs 3016.

On the final picture, this leads to a total of $8\phi16$ in the section of the column.

LESIGN WITH BENDING MOMENT AND AXIAL FORCE

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

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

ews: Complete setup 👻 View settings 👻 Load default 👘 Fir	id						National a	nnex:	
Description	Symbol	Value	Default	Unit	Chapter	Code	Struc	Check	
all> 🖇	<all> D</all>	<all> ρ</all>	<all> D</all>		<all> D</all>	<all> 🔎</all>	<a)<="" td=""><td><a <math="">\rho</td><td></td>	<a <math="">\rho	
First order eccentricity with the equivalent moment					5.8.8.2(2)	EN 1992	Column	Solver	
Second order eccentricity	e ₂				5.8.8	EN 1992	Column	Solver	
 Internal forces modifications 									
Limit ratio for uniaxial method	Plim	0.10	0.10	21		Independ	1D (Be	Solver	
⊳ Beam			-						
Column									
▶ Type		Auto 🔺	Atto			Independ	Column	Solver	
Axial force (N _{Ed})	N _{Ed}	Auto				Independ	Column	Solver	
Bending moment about Y-axis (M _{Edy})	M _{Edy}	Uniaxial Y-Y				Independ	Column	Solver	
Bending moment about Z-axis (M _{Edz})	M _{Edz}	Biaxial				Independ	Column	Solver	
Torsional moment (T _{Ed})	T _{Ed}	User				Independ	Column	Solver	
Shear force in Y-axis (V _{Edy})	VEdy	User with lin	nit			Independ	Column	Solver	
Shear force in Z-axis (V _{Edz})	V _{Edz}	2				Independ	Column	Solver	
Beam slab									
 Design As 									
Beam, Column, Rib, Beam Slab									
Coefficient increasing statically required reinforcement in beam for u	Coeff _{stat.up}	0.00	0.00			Independ	Beam,	Solver	
Coefficient increasing statically required reinforcement in beam for lo.	. Coeff _{stat.lo}	0.00	0.00			Independ	Beam,	Solver	
Coefficient increasing statically required reinforcement in column	Coeff	0.00	0.00			Independ	Calumn	Sahrer	

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

Rule for automatic selection of the design method:

- If $\rho_M \le \rho_{M,lim}$ Uniaxial method
- If $\rho_M \ge \rho_{M,lim}$ Biaxial method

$$\rho_{M} = \frac{\text{Min}\{|\text{MEd}_{y,\text{max}}|,|\text{MEd}_{z,\text{max}}|\}}{\text{Max}\{|\text{MEd}_{y,\text{max}}|,|\text{MEd}_{z,\text{max}}|\}}$$

With:

- M_{Edy.max} maximal design moment around y axis from all combinations in current section
- MEdz.max maximal design moment around z axis from all combinations in current section
- ρ_{M,lim} limit ratio of bending moments for uniaxial method loaded from Concrete settings

Settings for limit ratio:

-									
Des	cription	Symbol	Value	Default Unit	Chapter	Code	Struc	Check	Î.
<all></all>	Q	<all> D</all>	<all> ρ</all>	<all> 🔎</all>	<all> \wp</all>	<all> \wp</all>	<a <math="">\wp	<a p<="" th=""><th></th>	
Solv	er setting								
D	General								
41	nternal forces								
	Shear force reduction above supports				6.2.1(8)	EN 1992	Beam,	Solver	
	Moment reduction above supports				5.3.2.2 (4)	EN 1992	Beam,	Solver	
	Shifting of moment curve to cover additional tensile force caused by shear				9.2.1.3(2)	EN 1992	Beam,	Solver	
	Geometric imperfection in ULS	ei, ULS			5.2(2)	EN 1992	Column	Solver	
	Geometric imperfection in SLS	ei, sls			5.2(3)	EN 1992	Column	Solver	
	Minimum eccentricity	e _{min}	In first ord	In first	6.1(4)	EN 1992	Column	Solver	
	First order eccentricity with the equivalent moment				5.8.8.2(2)	EN 1992	Column	Solver	
	Second order eccentricity	e ₂			5.8.8	EN 1992	Column	Solver	L
	 Internal forces modifications 								L
	Limit ratio for uniaxial method	ρ _{lim}	0.10	0.10		Independ	1D (B.e.,,	Solver	L
	⊳ Beam								
	4 Column								
	▶ Туре		Auto	Auto		Independ	Column	Solver	
	Axial force (N _{Ed})	N _{Ed}				Independ	Column	Solver	L
	Bending moment about Y-axis (M _{Edy})	M _{Edy}				Independ	Column	Solver	
	Rending moment about 7-axis (Me.)	M	1973	1973		Independ	Calumn.	Sahoar	١.

⇒ Uniaxial bending calculation



Principle

The reinforcement is designed for NEd and one bending moment MEd,y or MEd,z:

- Uniaxial around y: MEdz is ignored, the reinforcement is designed only for NEd and MEd,y
- Uniaxial around z: MEdy is ignored, the reinforcement is designed only for NEd and MEd,z

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

- If $M_{Ed,y} > M_{Ed,z} \rightarrow A_s = A_{sy}$ is designed for forces N_{Ed} and $M_{Ed,y}$
- If $M_{Ed,z} > M_{Ed,y} \rightarrow A_s = A_{sz}$ is designed for forces N_{Ed} and $M_{Ed,z}$

Example: Uniaxial bending.esa

Geometry

Column cross-section: RECT 350x350mm²

Height: 4,5 m

Concrete grade: C45/55

Concrete Setup

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

Con	cret	e sett	tings									×
Viev	NS:	Com	nplete setup 👻 View settings 🔻 Load default Find	1					l.	National a	innex:	
	Des	scrip	tion	Symbol	Value	Default	Unit	Chapter	Code	Struc	Check	7
<a< td=""><td>11></td><td></td><td>Q</td><td><all> ρ</all></td><td><all> ρ</all></td><td><all> ρ</all></td><td></td><td><all> ρ</all></td><td><all> \wp</all></td><td><a ,0<="" td=""><td><a td="" 🔎<=""><td></td></td></td></a<>	11>		Q	<all> ρ</all>	<all> ρ</all>	<all> ρ</all>		<all> ρ</all>	<all> \wp</all>	<a ,0<="" td=""><td><a td="" 🔎<=""><td></td></td>	<a td="" 🔎<=""><td></td>	
	Des	ign	defaults									
	Þ	Reir	nforcement									
	Þ	Mini	imum cover									
	Sol	vers	setting									
	Þ	Gen	eral									
		Inte	rnal forces									
		9	Shear force reduction above supports					5.2.1(8)	EN 1992	Beam,	Solver	
		1	Moment reduction above supports				1	5.3.2.2 (4)	EN 1992	Beam,	Solver	
		4	Shifting of moment curve to cover additional tensile force caused by shear				9	9.2.1.3(2)	EN 1992	Beam,	Solver	>>
		(Geometric imperfection in ULS	ei,ULS			1	5.2(2)	EN 1992	Column	Solver	1
		(Geometric imperfection in SLS	e,sls			199	5.2(3)	EN 1992	Column	Solver	
		1	Minimum eccentricity	e _{min}	In first ord.	In first	1	5.1(4)	EN 1992	Column	Solver	
		F	First order eccentricity with the equivalent moment				1	5.8.8.2(2)	EN 1992	Column	Solver	
		+ 5	Second order eccentricity	e ₂	0		1	5.8.8	EN 1992	Column	Solver III	
		4.1	Internal forces modifications									
			Limit ratio for uniaxial method	Plim	0.10	0.10	8		Independ	1D (Be	Solver	
			p Beam									
			Column									
_			Type		Auto	Auta			Independ.	Column	Saluar	

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

/s: (Con	mplete setup 👻 View settings 🔻 Load de	fault	Find						Nationa	al annex:	
Des	crip	iption	Symbol	Valu	e	Default	Unit	Chapter	Code	Structu	CheckT	ī.
>			o ⊲all> ∫) <all></all>	. p	<all></all>	Q>	<all></all>	<all> \wp</all>	<all> 🔎</all>	<all></all>	L
V	ura	ack width							-			L
⊳	Def	eflections										L
4	Det	tailing provisions		-								L
	Þ	Beam / Rib										L
	Þ	Beam slab		-								L
	4	Column		-								L
		▲ Longitudinal		-								
		Check min. bar distance				~		8.2(2)	EN 1992-1-1	Column	Solver se	
		Check max. bar distance							Independent	Column	Solver se	
		Check max. bar distance (torsion)				~		9.2.3(4)	EN 1992-1-1	Column	Solver se	
		Check min. reinforcement area						9.5.2(2)	EN 1992-1-1	Column	Solver se	
		Check max. reinforcement area						9.5.2(3)	EN 1992-1-1	Column	Solver se	
		Check min. bar diameter				~		9.5.2(1)	EN 1992-1-1	Column	Solver se	
		Check min. number of bars						9.5.2(4)	EN 1992-1-1	Column	Solver se	
		∡ Transverse										
		Check max. percentage of stirrups						6.2.3(3)	EN 1992-1-1	Column	Solver se	
		Check min. mandrel diameter						8.3(2)	EN 1992-1-1	Column	Solver se	
		Check max. longitudinal spacing						9.5.3(3)	EN 1992-1-1	Column	Solver se	
		Check min. bar diameter						9.5.3(1)	EN 1992-1-1	Column	Solver se	

Loads

Column B1:

- LC1: Permanent load > F = 500 kN; M_y = 100 kNm
- LC2: Variable load > F = 1000 kN; M_y = 100 kNm

Column B2:

- LC1: Permanent load > F = 500 kN; M_y = 100 kNm
- LC2: Variable load > F = 1000 kN; M_y = 100 kNm; M_z = 10 kNm

Combination according to the Eurocode:

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

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

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

Additional design moment in column B2 Mzd = 22,5 kNm

Results

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

Looking at the Detailed output for column B1:

 $\begin{array}{l} \textbf{Determination type of calculation} \\ \textbf{Calculation maximum bending moments around y and z axis} \\ \textbf{M}_{y,max} = -285 \text{ kNm } \textbf{M}_{z,max} = 0 \text{ kNm} \\ \textbf{Calculation maximum ratio of bending moments} \\ \textbf{p}_{M} = 0 \\ \textbf{Determination type of calculation} \\ \textbf{p}_{M} = 0 < \textbf{p}_{Mlim} = 0.1 \text{ and } |\textbf{M}_{y,max}| = 285 \text{ kNm} > |\textbf{M}_{z,max}| = 0 \text{ kNm} = > \\ = > \textbf{Uniaxial method around y axis. Moment } \textbf{M}_{z} \text{ will not take into account } (\textbf{M}_{z} = 0 \text{ kNm}). \end{array}$

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

COL	umn	B1				REC	T (350;	350)			
EC EN	1992-1-1:2	004/AC:2	2008			Secti	on 0 [dx =	0 m]			
Mem	ber length	ı	Ld =	4.5 m		Mat	erials				
Buck	ing length	у	Ly =	9.01 m		Conc	Concrete C45/55				
Buck	ing length	z	Lz =	9.01 m		Reinf	Reinforcement B 500A				
Longit	udinal r	einford	ement					Shear	reinfo	rcement	
										0 0	
φ = 1 esign o	6 mm, c =	30 mm, udinal	reinforce	ement	205 kM	Im Max =	0 kNm	n _{s.re}	_q = 2, φ _{s.rec}	_q = 8 mm, α _{s.req} = 90	
φ = 1 esign o A _{s.z.} : 1.3 A _{s.z} : 1.3	6 mm, c = • f longit 35*LC1+1.5 5*LC1+1.5	30 mm, udinal 50*LC2 : N 0*LC2 : N	reinforc N _{Ed} = -2175 N _{Ed} = -2175	ement kN, M _{Edy} kN, M _{Edy} :	= -285 kN = -285 kN	lm, M _{Edz} = m, M _{Edz} =	0 kNm 0 kNm	n _{s.re}	q = 2, Φs.red	q = 8 mm, 0 _{sreq} = 9	
φ = 1 esign o A _{s.z.} : 1.3 A _{s.z} : 1.3 equired	6 mm, c = f longit 35*LC1+1.5 5*LC1+1.5 i	: 30 mm, udinal 50*LC2 : N 0*LC2 : N	reinforc N _{Ed} = -2175 J _{Ed} = -2175	ement kN, M _{Edy} kN, M _{Edy} :	= -285 kN = -285 kN	Im, M _{Edz} = m, M _{Edz} =	0 kNm 0 kNm	n _{s.re}	q = 2, φ _{s.re} ,	g = 8 mm, o _{streg} = 9	
φ = 1 esign o A _{s.z.} : 1.3 A _{s.z} : 1.3 equired Edge	6 mm, c = f longit 85*LC1+1.5 5*LC1+1.5 f Layer	30 mm, udinal 50*LC2 : N 0*LC2 : N y [m]	reinforce N _{Ed} = -2175 J _{Ed} = -2175 z [m]	ement i kN, M _{Edy} kN, M _{Edy} A _{s.stat} [mm ²]	= -285 kN = -285 kN A _{s.det.min} [mm ²]	Im, M _{Edz} = m, M _{Edz} = A _{s.det.max} [mm ²]	0 kNm 0 kNm ΔA _{s.tor} [mm ²]	A _{s.req} [mm ²]	A _{s.req,bar} [mm ²]	q = 8 mm, α _{s.req} = 9 Reinf	
φ = 1 esign o A _{s.z+} : 1.5 A _{s.z} : 1.3 equirec Edge	6 mm, c = f longit \$5*LC1+1.5 5 *LC1+1.5 1 Layer 1	30 mm, udinal 50*LC2 : N 0*LC2 : N y [m] 0	reinforce N _{Ed} = -2175 L _{Ed} = -2175 z [m] -0.129	ement kN, M _{Edy} kN, M _{Edy} : A _{s.stat} [mm ²] 1552	= -285 kN = -285 kN A _{s.det.min} [mm ²] 0	Im, M _{Edz} = m, M _{Edz} = A _{s.det.max} [mm ²] 0	0 kNm 0 kNm ΔA _{s.tor} [mm ²] 0	A _{s.req} [mm ²] 1552	A _{s.req.bar} [mm ²] 1608	q = 8 mm, α _{s.req} = 9 Reinf 8φ16	



Looking at the Detailed output for column B2:

Determination type of calculation Calculation maximum bending moments around y and z axis
M _{y.max} = -285 kNm M _{z.max} = -22.5 kNm Calculation maximum ratio of bending moments
ρ _M = 0.0789
Determination type of calculation
ρ_M = 0.0789 < $\rho_{M,lim}$ = 0.1 and $ M_{y.max} $ = 285 kNm > $ M_{z.max} $ = 22.5 kNm =>
= > Uniaxial method around y axis. Moment M_z will not take into account (M_z = 0 kNm).

And the Standard output:

COI	umn	B2				REC	T (350;	350)		
EC EN	1992-1-1:2	004/AC:2	008			Secti	on 0 [dx =	0 m]		
Mem	iber length	1	Ld =	4.5 m		Mat	terials			
Buck	ling length	у	Ly =	9.01 m		Conc	rete	C45	/55	
Buck	ling length	z	Lz =	9.01 m		Reinf	orcement	B 50	A0(
Longi	tudinal r	einford	ement					Shear	r reinfo	rcement
	6 mm c -	30 mm						n _{s.re}	q = 2, φ _{s.rec}	_q = 8 mm, α _{s.req} = 90
φ = 1 esign c	of longit	udinal	reinforc	ement						
φ = 1 esign c A _{s.z+} : 1.3 A _{s.z} .; 1.3	of longit 35*LC1+1.5 5*LC1+1.5	udinal (50*LC2 : N 0*LC2 : N	reinforc I _{Ed} = -2175 _{Ed} = -2175	ement 5 kN, M _{Edy} : 5 kN, M _{Edy} :	= -285 kN = -285 kNi	lm, M _{Edz} = m, M _{Edz} =	0 kNm 0 kNm			
φ = 1 esign o A _{5.2+} : 1.3 A _{5.2} : 1.3 equireo Edge	of longit 35*LC1+1.5 5*LC1+1.5 d Layer	udinal (50*LC2 : N 0*LC2 : N y [m]	reinforco _{led} = -2175 _{ed} = -2175 z [m]	ement 5 kN, M _{Edy} 5 kN, M _{Edy} 5 kN, M _{Edy}	= -285 kN = -285 kN A s.det.min [mm ²]	m, M _{Edz} = m, M _{Edz} = A _{s.det.max} [mm ²]	0 kNm 0 kNm ΔA _{s.tor} [mm ²]	A _{s.req} [mm ²]	A _{s.req.bar} [mm ²]	Reinf
φ = 1 esign c A _{sz+} : 1.3 A _{sz} : 1.3 equired Edge	of longit 35*LC1+1.5 5*LC1+1.5 d Layer 1	udinal (50*LC2 : N 0*LC2 : N y [m] 0	reinforco _{Ed} = -2175 _{Ed} = -2175 z [m] -0.129	ement 5 kN, M _{Edy} 5 kN, M _{Edy} 5 kN, M _{Edy} 5 kN, M _{Edy} 1552	= -285 kN = -285 kN A s.det.min [mm ²] 0	m, M _{Edz} = m, M _{Edz} = A _{s.det.max} [mm ²] 0	0 kNm 0 kNm ΔA _{s.tor} [mm ²] 0	A _{s.req} [mm ²] 1552	A _{s.req.bar} [mm ²] 1608	Reinf 8¢16



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

You have the possibility to force the biaxial method design on column B2 using 1D member data:

_		
E	CMD	
	Name	CMD1D
	Member	B4
	Member type	Column v
>	Design defaults	
4	Solver setting	
₽	General	
	Internal forces	
	Isolated member	
	Geometric imperfection in ULS	
	Geometric imperfection in SLS	
	Minimum eccentricity	In first order ecc. 🗸 🗸
	Second order eccentricity	
	Internal forces modifications	
	4 Column	
	Туре	Biaxial 🗸
1	Axial force (N _{Ed})	Image: A start of the start
	Bending moment about Y-axis (M _{Edy})	V
	Bending moment about Z-axis (M _{Edz})	✓
	Torsional moment (T _{Ed})	
	Shear force in Y-axis (V Edy)	✓
	Shear force in Z-axis (V _{Edz})	V
₽	Design As	
Þ	Conversion to rebars	
₽	Interaction diagram	
Act	ions	
		Load default values >>>
		OK Cancel

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

Col	umn l	B2				REC	T (350;	350)		
EC EN	1992-1-1:2	004/AC:2	800			Secti	on 0 [dx =	0 m]		
Mem	ber length	1	Ld =	4.5 m		Mat	terials			
Buck	ling length	у	Ly =	9.01 m		Cond	rete	C45	/55	
Buck	ing length	z	Lz =	9.01 m		Reinf	forcement	B 50	A00	
Longit	udinal r	einforc	ement					Shear	r reinfo	rcement
φ = 1	6 mm, c =	30 mm,						n _{s.re}	q = 2, φ _{s.re}	_q = 8 mm, α _{s.req} = 90 °
esign o A _{s.z+} : 1.3 A _{s.z} .: 1.3	of longit 35*LC1+1.5 5*LC1+1.5	udinal 1 50*LC2 : N 0*LC2 : N	reinforce I _{Ed} = -2175 _{Ed} = -2175	ement kN, M _{Edy} kN, M _{Edy}	= -285 kN = -285 kN	Im, M _{Edz} = m, M _{Edz} =	-23 kNm -23 kNm			
equired	ł									
Edge	Layer	y [m]	z [m]	A _{s.stat} [mm ²]	A _{s.det.min} [mm ²]	A _{s.det.max} [mm ²]	ΔA _{s.tor} [mm ²]	A _{s.req} [mm ²]	A _{s.req.bar} [mm ²]	Reinf
1	1	0	-0.129	2046	0	0	0	2046	2212	11016

0

0

2046

2212

11¢16

⇒ Biaxial bending calculation

3

1

0

0.129

2046

0



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

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

where:

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

0

a exponent:

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

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

with linear interpolation for intermediate values

- o N_{Ed} design value of axial force
 - $N_{Rd} = A_c \cdot (f_{cd} + \mu_s \cdot f_{yd})$, design axial resistance of the section, where:
 - Ac gross area of the concrete section
 - f_{cd} design value of concrete compressive strength
 - f_{yd} design yield strength of reinforcement
 - µs mechanical reinforcement ratio in the calculation of limit slenderness obtained with an iterative calculation

(5.39)

GIRCULAR COLUMN

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

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

Example: Circular column.esa

Geometry

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

Loads

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

Concrete Setup

Geometrical imperfection and 2nd order moments are deactivated: Concrete settings > Complete Setup view :

vs: Comp	lete setup 👻 View settings 🔻 Load default 🛛 Find	1					National a	innex	
Descriptio	on	Symbol	Value	Default Ur	it Chapter	Code	Struc	Check	1
>	Q	<all> \wp</all>	<all> D</all>	<all> D</all>	<all> D</all>	<all> P</all>	<a 0<="" th=""><th><a <math="">\rho</th><th></th>	<a <math="">\rho	
Design de	efaults								
▶ Reinfe	orcement								
Minin	num cover								
Solverse	etting								
⊳ Gener	ral								
▲ Intern	nal forces								
Sh	near force reduction above supports				6.2.1(8)	EN 1992	Beam,	Solver	
Me	oment reduction above supports				5.3.2.2 (4)	EN 1992	Beam,	Solver	ł.
SH	nifting of moment curve to cover additional tensile force caused by shear		- · · · ·		9.2.1.3(2)	EN 1992	Beam,	Solver	L
Ge	eometric imperfection in ULS	e _{i,ULS}			5.2(2)	EN 1992	Column	Solver	L
Ge	eometric imperfection in SLS	e _{i,SLS}			5.2(3)	EN 1992	Column	Solver	L
Mi	inimum eccentricity	e _{min}	In first ord.	In first	6.1(4)	EN 1992	Column	Solver	L
Fi	rst order eccentricity with the equivalent moment				5.8.8.2(2)	EN 1992	Column	Solver	L
► Se	econd order eccentricity	e ₂	0		5.8.8	EN 1992	Column	Solver III	L
🖌 İn	ternal forces modifications								L
	Limit ratio for uniaxial method	Plim	0.10	0.10 -		Independ	1D (Be	Solver	L
Þ	Beam								L
	Column								L

All detailing provisions are considered.

Design defaults

The bar diameter is set to ϕ 20 mm in Reinforcement design > Design defaults > Tab Columns, or from 1D Member data if applied.

vs: Complete setup 👻 View	settings 👻 🛛	oad defau	ilt	Find					Natio	nal annex 🗾	
Description		Symbol		Value	Default	Unit	Chapter	Code	Structure	CheckType	
>	م م	<all></all>	ρ	<all></all>	<all> ₽</all>	< P	<all></all>	all> 🔎	<all> ρ</all>	<all> ₽</all>	
Design defaults											
 Reinforcement 											
> Beam / Rib											
b Beam slab											
Column					100						
Design of provided reinforcen	nent			2				Independent	Column	Design def	
Rectangular section				Column_R	Column_R			Independent	Column	Design def	
Circular				Column_C	Column_C			Independent	Column	Design def	
Oval				Column	Column_0			Independent	Column	Design def	
Other and general				Column	Column_0			Independent	Column	Design def	
 Longitudinal 											
🔺 Main (m)											
Type of cover				User	Auto		4.4.1	EN 1992-1-1	Column	Design def	
Concrete cover (c)		с		35.0	30.0	mm	4.4.1	EN 1992-1-1	Column	Design def	
Diameter		d _{s.m}		20	16.0	mm		EN 1992-1-1	Column	Design def	
Detailing (det)					21						
 Stirrups (sw) 											
Diameter		dss		8.0	8.0	mm		EN 1992-1-1	Column	Design def	
Number of cuts		ns		2.0	2.0			Independent	Column	Design def	
Plate											
> Wall / Deep beam											

Results

Go to Reinforcement design > 1D members > Reinforcement design.

Choose Standard output in the Properties window and open the Preview at the bottom of the Properties window:



In this example $A_{s,req}$ is determined by the minimum amount of reinforcement according to the detailing provision, $A_{s,det,min}$.

Since $A_{s,req} = 1257mm^2$, the software will propose 5 bars of $\phi 20mm$ (5*314mm² = 1571mm² = $A_{s,req,bar}$) which is the closest amount of bar with $A_{s,req,bar} > A_{s,req}$.

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

So, the final required reinforcement displayed on the screen is As, req, bar.

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

ews: Compl	lete setup 💌	View settings 🔻	Load default	Find	National annex:
Descriptio	on		Symbol	Value	Provided reinforcement (design)
Design de	faulte		- all p	sau p	
A Reinfo	orcement				
b Re	am / Rib				Column_Circular_Basic_AddList
b Re	am slab				Column_Circ_Empty
4 Co	lumn				Column Circ Basic Add
-	Design of provided reinfo	orcement			Column Circ Basic AddList
	Rectangular section		-	Column R.	
	Circular			Column ¥	Name Column_Circ_
	Oval			Column O	Description Empty reinfor
	Other and general			Column O	C Member type Column
2	Longitudinal				Cross-section Circular Z
	Main (m)				Mode Standard
	Type of cover			Auto	
	Diameter		d _{s.m}	16.0	1>V
	Detailing (det)				
4	Stirrups (sw)				
	Diameter		d _{ss}	8.0	
	Number of cuts		ns	2.0	
Þ Pla	ate				
⊳ Wa	all / Deep beam				
10	num cover				

Remark 2:

According to EN1992-1-1 art 9.5.2(4), there is a minimum number of bars in a circular column.

This parameter is set by default to "4" in Concrete Settings > Complete setup view.

Concre	te s	ettin	gs																_		×
Views:	Co	mpl	ete setup 🔹 👻	View settings 🔻		Load defau	ult	Find									N	latior	nal anne	ex:	
D	escr	iptic	on			Symbol		Value		Default		Unit	Chapter		Code		Struct	ure	Check	Туре	
<all></all>					ρ	<all></all>	P	<all></all>	ρ	<all></all>	ρ	< P	<all></all>	P	<all></all>	ρ	<all></all>	2	<all></all>	Q	
Þ	De	flec	tions																		
	De	tail	ling provisions					•													4
	⊳	Be	am / Rib					•													
	⊳	Be	am slab					-													
		Co	lumn					•													
			Longitudinal					•													
			Check min. bar distan	ce				~		~			8.2(2)		EN 1992-1-1		Colum	n	Solver	sett	
			Minimal bar distanc	e		^S lc,min		20		20		mm	8.2(2)		EN 1992-1-1		Colum	n	Solver	sett	
			Check max. bar distar	nce				~		~					Independer	it	Colum	n	Solver	sett	
			Maximal bar distance	ce		Slc,max		350		350		mm			Independer	it	Colum	n	Solver	sett	>>
			Check max. bar distar	nce (torsion)				~		~			9.2.3(4)		EN 1992-1-1		Colum	n	Solver	sett	
			Maximal bar distanc	ce (torsion)		Sict,max		350		350		mm	9.2.3(4)		EN 1992-1-1		Colum	n	Solver	sett	
			Check min. reinforcen	nent area				~		~			9.5.2(2)		EN 1992-1-1		Colum	n	Solver	sett	
			Check max. reinforcer	ment area				~		~			9.5.2(3)		EN 1992-1-1		Colum	n	Solver	sett	
			Check min. bar diame	eter				~		~			9.5.2(1)		EN 1992-1-1		Colum	n	Solver	sett	
			Check min. number of	fbars				~		~			9.5.2(4)		EN 1992-1-1		Colum	n	Solver	sett	
			Minimal number of	bars in circular co	ol	n _{le,min}		4.0		4.0			9.5.2(4)		EN 1992-1-1		Colum	n	Solver	sett	
			Transverse					•													
			Check max. percentag	ge of stirrups				~		~			6.2.3(3)		EN 1992-1-1		Colum	n	Solver	sett	
			Check min. mandrel d	liameter									8.3(2)		EN 1992-1-1		Colum	n	Solver	sett	

If we increase the loads: $F_z = -1250 \text{kN}$

M = 50 kNm

The results are as follows.

Example: Circular column increase.esa

Overa	all De	esign	(ULS)								
Linear ca	lculation										
Combinati	ion: CO1										
Coordinate	e systen	n: Principa	al								
Extreme	1D: Globa	al									
Selection:	All										
longitud	linal roo	wirod r	aintarcomo								
Longituu		luiren 1	ennorceme	nt							
Name	dx [m]	Case	Member	A _{sz req+} [mm ²]	A _{sz req} . [mm ²]	A _{sy req+} [mm ²]	A _{sv req} . [mm²]	A _{sz req} [mm²]	A _{sv req} [mm²]	A _{s req} [mm²]	ReinfReq
Name	dx [m]	Case	Member	A _{sz req} + [mm ²] A _{sz req bar+} [mm ²]	A _{sz req} - [mm ²] A _{sz req bar-} [mm ²]	A _{sv req+} [mm ²] A _{sv req bar+} [mm ²]	A _{sy req} - [mm ²] A _{sy req bar-} [mm ²]	A _{sz req} [mm ²] A _{sz req bar} [mm ²]	A _{sv req} [mm ²] A _{sv req bar} [mm ²]	A _{s req} [mm ²] A _{s req bar} [mm ²]	ReinfReq
Name B1	dx [m] 0,000	Case CO1	Member Column	A _{sz req+} [mm ²] A _{sz req bar+} [mm ²] 1041	A _{sz req} - [mm ²] A _{sz req bar-} [mm ²] 1041	A _{sv req+} [mm ²] A _{sv req bar+} [mm ²] 1041	A _{sy req} - [mm ²] A _{sy req bar-} [mm ²] 1041	A _{sz req} [mm ²] A _{sz req bar} [mm ²] 2082	A _{sy req} [mm ²] A _{sy req bar} [mm ²] 2082	A _{s req} [mm ²] A _{s req bar} [mm ²] 4164	ReinfReq 14ф20

The corresponding bar configuration is:



2.3.2 Calculation of internal forces

DETERMINING IF MEMBER IS IN COMPRESSION

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

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

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

In Concrete settings > Complete setup view:

Concrete settings								- 0	×
Views: Complete setup View settings Load def	ault	Find					Nationa	il annex:	
Description	Symbol	Value	Default	Unit	Chapter	Code	Structu	CheckT	1
<all></all>	<all> 🔎</all>	<all> 🔎</all>	<all> 🔎</all>	< P	<all> 🔎</all>	<all></all>	<all> ₽</all>	<all> 🔎</all>	
Design defaults									
Reinforcement									
Minimum cover									
Solver setting									
✓ General									
Limit value of unity check	Lim.check	1.0	1.0			Independent	All (Bea	Solver se	
Value of unity check for not calculated unity check	Ncal.check	3.0	3.0			Independent	All (Bea	Solver se	
The coefficient for calculation effective depth of cross-sec	. Coeff _d	0.9	0.9			Independent	All (Bea	Solver se	
The coefficient for calculation inner lever arm	Co eff <mark>z</mark>	0.9	0.9			Independent	All (Bea	Solver se	
The coefficient for calculation force, where member as u	Coeff _{com}	0.1	0.1			Independent	All (Bea	Solver se	>>
Creep and shrinkage									
Age of concrete at the moment considered	t	18250.00	18250.00	day	3.1.4.B.1-2	EN 1992-1-1	All (Bea	Solver se	
Relative humidity	RH	50	50		3.1.4.B.1-2	EN 1992-1-1	All (Bea	Solver se	
Type input of creep coefficient	Type q (t,to)	Auto	Auto		3.1.4(2)	EN 1992-1-1	All (Bea	Solver se	
Age of concrete at loading	to	28.00	28.00	dav	314(2) B1	EN 1992-1-1	All (Bea	Solver se	

Condition is:

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

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

The Detailed output gives:

Compression member
Limit axial force to consider member as compression:
$N_{com} = -Coeff_{com} \cdot (f_{cd} \cdot A_c) = -0.1 \cdot (30 \cdot 10^6 \cdot 0.123) = -368 \text{ kN}$
Check condition:
N _{Ed} < N _{com} = -1100 kN < -368 kN compression member
Note: First and second order eccentricity shall be taken into account, because the member is considered as a compression member (significant normal force is presented).

CHOICE BETWEEN 1st and 2nd ORDER CALCULATION

Slenderness – Check of the criteria $\lambda < \lambda_{lim}$

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

The values for λ and λ_{im} , and the corresponding check, can be found in the main menu Deign > Concrete 1D > Slenderness for design :

RESUL	TS (1)
Name	Slenderness(Design)
SELECTION	
Type of selection	Current 🗸
Filter	No \vee
Results in sections	All \sim
RESULT CASE	
Type of load	Combinations ∨
Combination	ELU-Set B (auto) 🗸
 EXTREME 1D 	
Extreme 1D	Global 🗸
Values	$\lambda \lor$
Interval	0
 OUTPUT SETTINGS 	
Output	Brief
Print combination key	
DRAWING SETUP 1D	
ERRORS, WARNINGS AND NOTES S	ETTINGS
Run using Model Data files (Debug)	\bigcirc
ACTIONS >>>>	
Refresh	

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

sie	ndern	ess(Des	sign)					
ineai oad oord xtrei elect	r calculati case: LC1 linate sys me 1D: G tion: All	on tem: Principa lobal	91					
C	olumn I	B1			RECT (35	0; 350)		
EC E	EN 1992-1-1:2	2004/AC:2008			Section 0 [d)	(= 0 m]		
EC E	EN 1992-1-1:2 nderness	2004/AC:2008	1 [m.]	0 11	Section 0 [d	< = 0 m]		1
EC I	EN 1992-1-1:2 nderness Axis	8004/AC:2008 Braced	L _{z/y} [m]	β _{zz/yy} [-]	Section 0 [d)	x = 0 m] λ _{z/y} [-]	λ _{limz/y} [-]	$\lambda_{z/y} > \lambda_{imz/y}$

4 1st ORDER EFFECTS

1st order effects (eccentricity) are always considered.

There are 2 ways to calculate the 1st order moments and eccentricity in SCIA Engineer depending on check box **First order eccentricity with the equivalent moment** in Concrete Setup > Solver setting > Internal forces.

ews:	Complete setup View sett Load defau	lt	Find					Nationa	al annex:		
De	escription 5	Symbol	Value	Default	Unit	Chapter	Code	Struc	Check	1	Remark
all>	ρ.	<all></all>	all> 🔎	<all></all>		<all> 🔎</all>	<all> \wp</all>	<a <math="">\rho	<all> 🔎</all>		
De	esign defaults										
Þ	Reinforcement										
Þ	Minimum cover										
So	olver setting										
Þ	General										
	Internal forces										$M_{0e} = 0.6 \cdot M_{02} + 0.4 \cdot M_{01} \ge 0.4 \cdot M_{02}$
	Shear force reduction above supports					6.2.1(8)	EN 1992	Beam,	Solver s		
	Moment reduction above supports					5.3.2.2 (4)	EN 1992	Beam,	Solver s		
	Shifting of moment curve to cover additional tensi					9.2.1.3(2)	EN 1992	Beam,	Solver s	<<	
	Geometric imperfection in ULS e	I,ULS				5.2(2)	EN 1992	Column	Solver s		
	Geometric imperfection in SLS e	i,SLS				5.2(3)	EN 1992	Column	Solver s		
	Minimum eccentricity e	min	In first ord.	. In first		6.1(4)	EN 1992	Column	Solver s		The first order moment is taken into
	▶ First order eccentricity with the equivalent moment		S	~		5.8.8.2(2)	EN 1992	Column	Solver s		account as equivalent first order moment,
	Second order eccentricity e	2				5.8.8	EN 1992	Column	Solver s		this parameter is ON.
	Internal forces modifications										
₽	Design As										
Þ	Conversion to rebars										
⊳	Interaction diagram										
Þ	Shear										
Þ	Torsion										

The 2 options are:

• First order eccentricity with the equivalent moment = YES, bending moments at the ends of the column will be taken to calculate an equivalent 1st order bending moment. This leads to the same 1st order bending moment along the whole length of the member.

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

With

$$M_{0e} = (0.6 * M_{02}) + (0.6 * M_{01}) \ge 0.4 * M_{02}$$

• First order eccentricity with the equivalent moment = NO, 1st order eccentricity is calculated from bending moments in current section. As a result, bending moments in each section can be different.

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

Values of the 1st order eccentricities and moments can be viewed in Design > Concrete 1D > Internal forces for design.

Standard output gives:

Internal forces (Design)

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

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

Internal forces (FEM-based)

Extreme: ULS/1 (ULS)

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

Type of load	N [kN]	M _y [kNm]	M _z [kNm]	V _y [kN]	V _z [kN]	M _× [kNm]
Internal forces (FEM-based)	-300.0	-30.0	0.0	0.0	0.0	0.0
Content: LC1						

Slenderness

Axis	Braced	L _{z/y} [m]	β _{zz/yy} [-]	l _{0z/y} [m]	λ _{z/y} [-]	λ _{limz/y} [-]	$\lambda_{z/y} > \lambda_{limz/y}$
у-у⊥	No	4.5	2	9.01	89.2	46.5	2 nd order
z-z	No	4.5	2	9.01	89.2	46.5	2 nd order

Unfavourable direction

Second order effect and imperfections

Axis	N _{Ed} [kN]	M _{0Edy/z} [kNm]	M _{2y/z} [kNm]	M _{Edy/z} [kNm]	e _{0z/y} [mm]	e _{iz/y} [mm]	e _{0min,z/y} [mm]	e _{0Edz/y} [mm]	e _{2z/y} [mm]	e _{Edz/y} [mm]
у-у⊥	-300	-30	0	-30	100	0	0	100	0	100
z-z-	-300	0	0	0	0	0	0	0	0	0

Design forces (recalculated)

Type of load	N _{Ed}	M _{Ed,y}	M _{Ed,z}	V _{Ed,y}	V _{Ed,z}	M _{Ed,x}
	[kN]	[kNm]	[kNm]	[kN]	[kN]	[kNm]
Design forces (recalculated)	-300.0	-30.0	0.0	0.0	0.0	0.0

GEOMETRICAL IMPERFECTION (art 5.2)

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

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

iews: Complete setup 💌 View settings 💌 Load defa	ult	Fi	nd					Nationa	al annex:
Description	Symbol		Value	Default	Unit	Chapter	Code	Structu	CheckT
<all></all>		p.	<all> ρ</all>	<all></all>) <	<all> \wp</all>	<all> ρ</all>	<all> 🔎</all>	<all> ρ</all>
Design defaults									
Reinforcement									
Minimum cover									
Solver setting									
▷ General									
 Internal forces 									
Shear force reduction above supports						6.2.1(8)	EN 1992-1-1	Beam,B	Solver se
Moment reduction above supports						5.3.2.2 (4)	EN 1992-1-1	Beam,B	Solver se
Shifting of moment curve to cover additional tensile forc		1	2		100	9.2.1.3(2)	EN 1992-1-1	Beam,Ri	Solver se
Geometric imperfection in ULS	e _{i,ULS}	. 8	2			5.2(2)	EN 1992-1-1	Column	Solver se
Geometric imperfection in SLS	e _{i,SLS}					5.2(3)	EN 1992-1-1	Column	Solver se
Minimum eccentricity	e _{min}	h	n first order	In first or		6.1(4)	EN 1992-1-1	Column	Solver se
First order eccentricity with the equivalent moment		E	2			5.8.8.2(2)	EN 1992-1-1	Column	Solver se
Second order eccentricity	e ₂	8	<u>/</u>			5.8.8	EN 1992-1-1	Column	Solver se
Internal forces modifications									

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

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

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

- θ_0 basic value of inclination
- α_h reduction factor for length of column or height of structure: $\alpha_h = 2/\sqrt{l}$; $2/3 \le \alpha_h \le 1$
- $\alpha_{m,y(z)}$ reduction factor for numbers of members: $\alpha_{m,y(z)} = \sqrt{(0.5 \cdot (1 + 1/m_{y(z)}))}$
- I length of column or height of structure depending on:
 - \circ isolated member I = L, where L is the length of the member
 - not isolated member I = H, where H is the total height of building (buckling system)
- m_{y(z)} number of vertical members contributing to the total effect of the imperfection perpendicular to y(z)

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

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

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

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

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

A minimum 1st order eccentricity is also calculated according to clause 6.1(4) in EN 1992-1-1.

This can be viewed in Concrete settings > Internal forces > Use minimum value of eccentricity

Conc	rete settings									— D X
View	s: Complete setup 💌 View settings 👻	Load de	fault	Find				National	annex:	
	Description	Symbol	Value	Default	Unit	Chapter	Code	Struc	Check	Remark
<al< td=""><td> م </td><td><all></all></td><td>all></td><td>all> ,0</td><td></td><td><all> D</all></td><td><all></all></td><td><a 0<="" td=""><td><al td="" 🔎<=""><td></td></al></td></td></al<>	 م 	<all></all>	all>	all> ,0		<all> D</all>	<all></all>	<a 0<="" td=""><td><al td="" 🔎<=""><td></td></al></td>	<al td="" 🔎<=""><td></td></al>	
	Design defaults									A) No $e_0 = e_1 + e_i$
	Reinforcement									e=e_+e_
	Minimum cover									0.02
- 2	Solver setting									B) Min. ecc. to first order ecc.
	> General									e_=max(e_+e_:e_=)
	Internal forces									
	Shear force reduction above supports					6.2.1(8)	EN 1992-1.	Beam,	Solver	e-e ₀ +e ₂
	Moment reduction above supports					5.3.2.2 (4)	EN 1992-1.	Beam	Solver	C) Min. ecc. to final ecc.
	Shifting of moment curve to cover additional			2		9.2.1.3(2)	EN 1992-1.	. Beam,	Solver	0 -0 +0
	Geometric imperfection in ULS e	,ULS				5.2(2)	EN 1992-1.	. Column	Solver	««
	Geometric imperfection in SLS e	i,SLS				5.2(3)	EN 1992-1.	Column	Solver	$e = max(e_0 + e_2; e_{0min})$
	Minimum eccentricity e	min	In first ord	In first		6.1(4)	EN 1992-1.	. Column	Solver	
	First order eccentricity with the equivalent mo					5.8.8.2(2)	EN 1992-1.	Column	Solver	e =max(h/30:20mm)
	Second order eccentricity e	2	-			5.8.8	EN 1992-1.	. Column	Solver	C _{0min} =max(n) 50,20mm
	Internal forces modifications									The minimum value of the eccentricity can be set as
	Design As									follows:
	Conversion to rebars									A) Switched OFF, no minimum value is accounted for
	Interaction diagram									 B) The minimum is considered for the calculation of the first order eccentricity.
	> Shear									C) The minimum is considered for the final value of the
	> Torsion									eccentricity
						1		-		

Buckling data for I and m_y(z)

Settings for I and $m_y(z)$ for the calculation of the geometrical imperfection can be set in the properties of the columns.

Properties > System lengths and buckling settings



When opening the buckling menu, you need to define both the 'Active buckling constraints' and 'Span settings' for buckling around the local y-axis (buckling span y-y) and local z-axis (buckling span z-z).

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

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

ond ord	er effe	ct and i	mperfe	ctions						
Axis	N _{Ed} [kN]	M _{0Edy/z} [kNm]	M _{2y/z} [kNm]	M _{Edy/z} [kNm]	e _{0z/y} [mm]	e _{iz/y} [mm]	e _{0min,z/y} [mm]	e _{0Edz/y} [mm]	e _{2z/y} [mm]	e _{Edz/y} [mm]
у-у⊥	-405	-49.1	0	-49.1	100	21.2	20	121	0	121
z-z⊥	-405	8.1	0	8.1	0	0	-20	-20	0	-20

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

$$M_{0Ed,y(z)} = N_{Ed} \cdot e_{0Ed,z(y)}$$

```
e_{0Ed,z(y)} = e_{0,y(z)} + e_{i,y(z)} > e_{0,min,y(z)}
```

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

4 2nd ORDER EFFECTS

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

In SCIA Engineer the following methods are available:

- General method according to clause 5.8.2(2) based on a nonlinear calculation
- Simplified method based on nominal curvature according to clause 5.8.8
- Simplified method based on nominal stiffness according to clause 5.8.7

The simplified method is taken into account:

- For ultimate limit state
- For Member type = Column with compression according to "Determination if member is in compression"
- If option "Use second order effect" in switched ON, see Concrete settings > Internal forces.
- This option is activated by default.
- If slenderness $\lambda > \lambda_{lim}$, see chapter "Slenderness criteria"

Simplified method based on nominal stiffness

The total design moment, including second order moment, may be expressed as a magnification of the bending moments resulting from a first analysis, namely:

$$M_{Ed} = M_{0Ed} \cdot \left[1 + \frac{\beta}{\left(\frac{N_B}{N_{Ed}}\right) - 1} \right]$$

Where:

- M_{0Ed} is the first order moment
- β is a factor which depends on distribution of 1st and 2nd order moments: $\beta = \pi^2/c_0$
- c₀ is a coefficient which depends on the distribution of first order moment
- N_{Ed} is the design value of axial load
- N_B is the buckling load based on nominal stiffness
Simplified method based on nominal curvature

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

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

With:

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

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

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

Otherwise:

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

With:

- (1/r)_{z(y)}curvature around z(y), calculated according to clause 5.8.8.3
 - $c_{z(y)}$ factor depending on the curvature distribution around z(y) axis according to clause 5.8.8.2(4) $\circ = 8$, for constant 1st order bending moment (non zero) along the column and in case that
 - equivalent bending moment is taken into account ("Use equivalent first order value" ON). • = 10 otherwise.
- λ_{z(y)} slenderness
- $\lambda_{z(y),lim}$ limit slenderness
- $I_{0,z(y)}$ effective length of the column around z(y) buckling length

Effective length

The effective length, or buckling length, is by default calculated by SCIA Engineer. Be aware that formulas for automatic calculation are only valid for simple structures!

Otherwise, it is also possible to input the value of the effective length manually.

Automatic calculation of effective length

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

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

• For a non-sway structure:

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

• For a sway structure:

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

With:

- B buckling factor
- L system length
- E modulus of Young
- I moment of inertia
- C_i stiffness in node i
- M_i moment in node i
- Φ_i rotation in node i

$$\begin{split} \mathbf{x} &= \frac{4\rho_1\rho_2 + \pi^2\rho_1}{\pi^2(\rho_1 + \rho_2) + 8\rho_1\rho_2} \\ \rho_i &= \frac{c_i L}{EI} \qquad \qquad C_i = \frac{M_i}{\varphi_i} \end{split}$$

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

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

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

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

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

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

VS:	Co	mplete setup 👻 View settings 👻 Load defa	ult	F	ind						Nation	al annex	
D	escr	iption	Symbol		Value		Default	Unit	Chapter	Code	Structu	Check	y
11>		Q	<all></all>	P	<all></all>	0	<all></all>	<	<all></all>	all> D	<all></all>	all>	2
De	sig	n defaults											_
Þ	Re	inforcement											
Þ	Mi	nimum cover											
So	lversetting												
4	Ge	General											
		Limit value of unity check	Lim.chec	k –	1.0		1.0			Independent	All (Bea	Solver	se
		Value of unity check for not calculated unity check	Ncal.chee	k	3.0		3.0			Independent	All (Bea	Solver	6e
		The coefficient for calculation effective depth of cross-sec			0.9		0.9			Independent	All (Bea	Solver	se
		The coefficient for calculation inner lever arm			0.9		0.9			Independent	All (Bea	Solver	6e
		The coefficient for calculation force, where member as u	Coeff _{com}		0.1		0.1			Independent	All (Bea	Solver	se
	Þ	Creep and shrinkage											
	Þ	SLS											
	-	Default sway type											
	Е	Sway around y axis	Sway yy		Solution		~	ļ		Independent	All (Bea	Solver	se
		Sway around z axis	Sway zz		2					Independent	All (Bea	Solver	5e
Þ	In	ternal forces						1					
Þ	De	sign As											
Þ	Co	nversion to rebars											
Þ	In	teraction diagram											
Þ	Sh	ear											
Þ	То	rsion											

You can easily modify these default settings for a specific column in the project within the buckling menu. This menu can be accessed – as explained in the previous section – by navigating to the option '**System lengths and buckling settings**' within the properties of the member.

Z			
	MEMI	BER (1)	
¥	#		
	Name	B8	
	Layer	Calque1 🗸	E
	Туре	column (100) ∨	
	Analysis model	Standard \checkmark	
	FEM type	standard 🗸	
	Cross-section	CS2 - Rectangle (350; 350)	Ξ
	Alpha [deg]	0.00	
	Member system-line at	Centre \checkmark	
	ey [mm]	0.00	
	ez [mm]	0.00	
	LCS	standard \lor	
	LCS Rotation [deg]	0.00	
▼ B	UCKLING		
yste	m lengths and buckling sett	Default \vee	Ξ
	Material and no. of parts	Concrete - 1	
	Secondary member	0	

- 🗆 X
Settings Results Name BG1 Buckling span Deflection span • y.y Deflection z = y.y * z.z = z.z * Deflection y = z.z * • Active buckling constraints • Span settings Buckling length factors Settings per span for y-y axis Setary factor Sway y-y From setup * Member imperfection in 2.nd order analysis Total height Calculate * Total height 1
Save Cancel

This new setting has the name, here **BG1**, which you can attribute to others similar columns in their properties window:

			MEMB	ER (1)	5
₿	ø	\$			
			Name	B1	
			Layer	Standaard \checkmark	
			Туре	column (100) 🗸	
			Analysis model	Standard 💛	
			FEM type	standard \checkmark	
			Cross-section	CS2 - CIRC (400) 🗸	
			Alpha	0 ~	
		Memb	er system-line at	Centre \checkmark	
			ez [mm]	0.00	
			LCS	standard 🗸	
▼ B	UCKLI	NG		10000	
Syste	m leng	ths an	d buckling sett	BG1	× =
	Ν	lateria	and no. of parts	Default	
		Sec	ondary member	BG1	U

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

🎯 👁 🛱 🖓		
Concrete settings	•	
Concrete 1D	•	Internal forces for design
Concrete 2D	•	\int_{λ} Slenderness for design
👕 ULS punching		1D reinforcement design
Explode reinforcement to free bars		Edit reinforcement in section
Bill of reinforcement (1D)		Stiffness

Slenderness(Design)

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

Column B1	CIRC (400)
EC EN 1992-1-1:2004/AC:2008	Section 0 $[dx = 0 m]$

Slenderness

Axis	Braced	L _{z/y} [m]	β _{zz/yy} [-]	I _{0z/y} [m]	λ _{z/y} [-]	λ limz/y [-]	$\lambda_{z/y} > \lambda_{limz/y}$
у-у⊥	No	4.5	2	9.01	90.3	29.6	2 nd order
z-z⊥	No	4.5	1	4.5	45.1	29.6	2 nd order

Manual input of effective length

The same option – as seen for the automatic calculation – allows you to manually define the buckling length of the system. The option 'Buckling length factors' can be accessed within the section 'Span settings'. In the table 'Settings per span for y-y/z-z axis' you can insert the buckling length which needs to be taken into account.



Comparison of the two simplified methods

Whatever the reinforcement in the column, the method based on nominal curvature gives more or less the same result, while the method based on the nominal stiffness is very affected by the reinforcement of the column.

And on the other hand, the method based on the nominal stiffness is not usable anymore if the applied normal effort N_{Ed} is too close to the buckling effort N_B .

4 RECALCULATED INTERNAL FORCES

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

The design moment, M_{Ed} , is equal to $M_{Ed} = M_{0Ed} + M_{2.}$

With:

- M₂ 2nd order bending moment
- M_{0Ed} bending moment taking into account 1st order and geometrical imperfections

Example: 2nd order.esa

Geometry

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

Concrete Setup

All of the default values are kept.

This means that geometrical imperfection and 2nd order effects are taken into account.

Loads

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

Buckling data

Sway type is set by default.

Calculation of the effective length is done automatically by the software.

Slenderness criterion

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

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

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

Internal forces

Ask for M_{Ed} in Design > Concrete 1D > Internal forces for design.

The Standard output is chosen:

Type of load	d		N [kN]	1	M _y [kNm]	M _z [kNm]	V _y [kN]	V _z [kN	א 1 [M _x kNm]
Internal force	nternal forces (FEM-based)				-40.5	0.0	0.0	0.0	C	0.0
cond ord	er effec	t and ir	nperfe M _{2w/z}	ctions MEdwa	e _{0z/v}	e _{iz/v}	e0min.z/v	e0Edz/v	e _{2z/v}	e _{Edz/v}
cond ord _{Axis}	er effec N _{Ed} [kN]	t and in M _{0Edy/z} [kNm]	nperfe M _{2y/z} [kNm]	ctions M _{Edy/z} [kNm]	e _{0z/y} [[mm]	e _{iz/y} [mm]	e _{0min,z/y} [mm]	e _{0Edz/y} [mm]	e _{2z/y} [mm]	e _{Edz/y} [mm
cond ord Axis y-y	er effec N _{Ed} [kN] -405	t and in M _{OEdy/z} [kNm] -49.1	nperfe M _{2y/z} [kNm] -73.2	Ctions M _{Edy/z} [kNm] -122	e _{0z/y} [[mm] 100	е _{в/у} [mm] 21.2	e _{Ominz/y} [mm] 20	e _{0Edz/y} [mm] 121	e _{2z/y} [mm] 181	e _{Edz/y} [mm 302
Axis	er effec N _{Ed} [kN] -405	t and in M _{0Edy/z} [kNm] -49.1	M23/z [kNm] -73.2	MEdy/z [kNm] -122	e _{0z/y} [[mm] 100	e _{iz/y} [mm] 21.2	e _{0min,z/y} [mm] 20	e _{0Edz/y} [mm] 121	e _{2z/y} [mm] 181	e _{Edz} / [mm 302

Results

The results for the reinforcement design are shown below:

Ca	se		N _{Ed} [kN]	V _{Edy} [kN]	V _{Edz} [kN]	T _{Ed} [kNm]	M _{Edy} [kNm]	M _{Edz}	1]	λ/λ _{iim} y-y⊥		λ/λ _{in} z-z⊥	i)
UL	S/1		-300.0	0.0	0.0	0.0	-30.0) ().0			-	7
UL	S/2		-405.0	0.0	0.0	0.0	-122.3	71	1.6 2	.38	2nd	2.38	2nc
ULS	5/1		LC1										
ULS	5/2		1.35*LC1										
ngi	itudinal ı	einforceme	nt										
	Basic	Additional	Deta	ailing	A _{s.ult} [mm ²]	A _{s.min} [mm ²]	A _{s,req}	A _{s,prov} [mm ²]	A _{s,max} [mm ²]	s _{min} [mm]	s _{max} [mm]	Status	
[1]	2ф16				500	61	500	402	1225	242	258	N	ot Ol
										≥37	-		
[2]	2ф16				385	61	385	402	1225	70	86		0
										≥37	-		
[3]	2ф16				500	61	500	402	1225	242	258	N	ot OI
										≥37	-		
[4]	2ф16				385	61	385	402	1225	70	86		0
	~									≥37	-		
ΣY	4ф16	105					1000	804					
~7	4016						770	804					
24	222 100 100 100												

Note that biaxial bending method was used for reinforcement calculation.

2.4 Plate design

2.4.1 Used example

📕 🔰 INPUT OF GEOMETRY

Project data: 2D environment = Plate XY

	DATA			MATERIAL		
	Name:	Example project		Concrete	C25/30	~
	Part:	ACT Reinforce Concre	ete	Reinforcement mate	B 500A	×
	Description:	Plate design		Steel Masonry		
11	Author:	SCIA Engineer		Aluminium		
	Date:	03. 09. 2021		Steel fibre concrete		
	[- Other		
	Structure:	🕐 Plate XY	~	CODE		
	Destauration			National Code:		
	environment	🤞 default	v	EC - EN		*
	Model:	関 One	*	National annex:		
		64hitve	rsion info	Standard EN		×

The Reinforcement material (e.g. B500A) chosen in the Project data window, will define the steel quality used for the theoretical reinforcement design.



Properties of the slab and the line supports:

⇒ Load cases & Load groups



🕹 LOADS

Load Case	Action type	Load Group	Relation	EC1-Load type
Self-weight	Permanent	LG1	/	/
Walls	Permanent	LG1	/	/
Service load	Variable	LG2	Standard	Cat B: Offices

Load cases			×	Load groups	Load groups				
et -: 🗹 🕩 🖬 🖷	•	📄 🖸 🛛 All	• T	et -1 🗹 🕩 🗟	s 🖉 🖬 😭				
LC1 - Self weight		Name	LC1	LG2	1	Name LG2			
LC2 - Walls		Description	Selfweight			Relation Standard	*		
LC3 - Service load		Action type	Permanent	•		Load Variable			
		Load group	LG1 ¥	•		Structure Building			
		Load type	Selfweight			Load type Cat B: Offices	*		
		Direction	-Z	•					
New Insert Edit	Delete		Close	New Insert Ed	lit Delete		ОК		

⇒ Load combinations

Type EN-ULS (STR/GEO) Set B

Type EN-SLS Quasi Permanent

Combinations				X
et -1 🖸 🕪 🔒	• • •	Input combinations		
ULS-Set B (auto)		Name	SLS-Quasi (auto)	
SLS-Char (auto)		Description		
SLS-Quasi (auto)		Туре	EN-SLS Quasi-permanent	
		Updated automatically	~	
		Structure	Building	
		Active coefficients		
	Contents	of combination		
		LC1 - Self weight [-]	1.000	
		LC2 - Walls [-]	1.000	
		LC3 - Service load [-]	1.000	
	Actions			
			Explode to envelopes	; >>>
			Explode to linear	>>>
			Show Decomposed EN combinations	, >>>
New Insert Edi	t Delete			Close

⇒ Result classes

All ULS+SLS

Result classes		×
et -1 🖸 🕩 🕇	🕈 🗢 🗖 All	~ T
AII ULS		Name All ULS+SLS
All SLS		Description
All ULS+SLS	 List 	
		ULS-SetB (auto) - EN-ULS (STR/GEO) SetB
		SLS-Char (auto) - EN-SLS Characteristic
		SLS-Quasi (auto) - EN-SLS Quasi-permanent
New Insert	Edit Delete	Close

🔸 🔰 FINITE ELEMENT MESH

⇒ Introduction

2 types of finite elements are implemented in SCIA Engineer:

- The **Mindlin element** including shear force deformation, which is the standard in SCIA Engineer. The Mindlin theory is valid for the calculation of both thin and thick plates.
- The **Kirchhoff element** without shear force deformation, which can be used to calculate and design only thin plates.

The element type used for the current calculation is defined in the Tools menu > Calculation & Mesh > Solver Settings:

Solver setup			
	Name	SolverSetup1	
	Specify load cases for linear calculation		
Advanced solver settings			
▲ General			
	Neglect shear force deformation (Ay, Az >> A)		
	Bending theory of plate/shell analysis	Mindlin	~
	Type of solver	Direct	٣
	Number of sections on average member	10	
	Warning when maximal translation is greater than [mm]	1000.0	
	Warning when maximal rotation is greater than [mrad]	100.0	
	Coefficient for reinforcement	1	
Effective width of plate ribs			
	Number of thicknesses of rib plate	20	
Detection of adjacent beam / edge			
	Parallelism tolerance [deg]	10.00	

⇒ Mesh generation

Via the tools menu \rightarrow Calculation & mesh \rightarrow Generate mesh

⇒ Graphical display of the mesh

Set view settings for all entities, via right mouse click in screen or more options > View settings for all entities



- Structure tab → Mesh → Draw mesh
- Labels tab \rightarrow Mesh \rightarrow Display label
- ⇒ Mesh refinement

Via the tools menu \rightarrow Calculation & mesh \rightarrow Mesh settings

Average size of 2D (mesh) elements is by default = 1 m



OR

The mesh size can be changed in the FE analysis window before running the calculation.

FE analysis		×
Calculations	Mesh setup	
	Average number of 1D mesh element 1	
Linear analysis	Average size of 1D mesh element on (0.200	
Loud cases. 5	Average size of 2D mesh element [m] 1.000	
Other processes	Connect members/nodes 🗹	
Test input of data	Setup for connection of structural en	•
	Advanced mesh settings	
Save project after analysis	 Solver setup 	
	Specify load cases for linear calculati	
	Advanced solver settings	
Calculate		

'Basic rule' for the size of 2D mesh elements: take 1 to 2 times the thickness of the plates in the project. For this example, take a mesh size of 0,25 m.



2.4.2 Results for the linear calculation

SPECIFICATION OF RESULTS

After running the linear calculation, go to the Results menu \rightarrow 2D members \rightarrow 2D Internal Forces.

Specify the desired result in the Properties panel:

.TS (1)	2	
2D internal forces		
All \sim		
No 🗸		
Combinations V		
ULS-Set B (auto) \vee		
Absolute extreme $ \smallsetminus $		
0		
In nodes avg. on macro 🗸		
LCS mesh element \vee	_	
Global 🗸	_	
Basic magnitudes ∨		
m_x ~		
\bigcirc		
\odot		
SETTINGS		
	_	
nation key		
	TS (1) 2D internal forces All ~ No ~ Combinations ~ ULS-Set B (auto) ~ Absolute extreme ~ O In nodes avg. on macro ~ LCS mesh element ~ Global ~ Basic magnitudes ~ m_x ~ SETTINGS SETTINGS	

System:

- LCS mesh element: according to the local axes of the individual mesh elements
- LCS Member 2D: according to the LCS of the 2D member (<u>Pay attention</u> when working with shell elements!)

Location: 4 different ways to ask for the results, see chapter Results

Type forces: Basic, Principal or Design magnitudes, see Annex 1

After making changes in the Properties panel, you always have to execute the 'Refresh' action.

TYPES OF RESULTS

⇒ Basic magnitudes

Combination = ULS; Type forces = Basic magnitudes; Envelope = Minimum; Values = m_x



These are the characteristic values coming from de FE-analysis in the center of the plate.

⇒ Elementary design magnitudes





The convention for the sign of the design moments has been changed since the v17 post-processor. Now a moment is positive when it causes a tensile force on the bottom of the plate and negative when it causes tensile force at the top of the plate.

In the v16 post-processor a design moment is positive when you should reinforce for this moment. This means that for a positive value for m_xD+ there is a tensile force at the top of the plate and that for a positive value for m_xD- there is a tensile force at the bottom of the plate.

The available values are mxD, myD and mcD, where 'D' stands for design. The '+' and '-' respectively stand for the values at the positive and negative side of the local z axis of the 2D member.

So for instance the value mxD+ is the moment that will be used for the design of the upper reinforcement in the local x-direction of the 2D member.

The calculation of design moments for *plates* and *shells* according to the EC2 algorithm follows the chart from CSN P ENV 1992-1-1, Annex 2, paragraph A2.8.



What happens, is that for the 3 characteristic (bending and torsion) moments an equivalent set of 3 design moments is calculated:

mx		mxD
my	≈	myD
mxy		mcD

It is clear that mxD and myD are the moments to be used for the reinforcement design in the respective direction. The quantity mcD is the design moment that has to be taken by the concrete. The Eurocode does not mention any check for this value, but it is however available in SCIA Engineer for the reason of completeness.

The calculation of design forces for *walls* according to the EC2 algorithm follows the chart from CSN P ENV 1992-1-1, Annex 2, paragraph A2.9.



Analogously, if membrane effects are present, for the 3 characteristic membrane forces an equivalent set of 3 design forces is calculated:

nx		nxD
ny	*	nyD
nxy		ncD

Here, the quantity ncD does have a clear meaning: it is the compression force that has to be taken by the concrete compression struts. Therefore, to make sure that concrete crushing will not occur, the value ncD should be checked to be \leq fcd.

<u>Attention</u>: These design magnitudes are not the ones used by SCIA Engineer for the reinforcement design in the Concrete menu. A much more refined transformation procedure is implemented there to calculate the design magnitudes from the basic magnitudes.

⇒ Principal magnitudes

Results menu \rightarrow 2D members \rightarrow 2D stresses/strain

Combination = ULS; Type forces = Principal stress; Envelope = Maximum; Values = σ 1+



'1' and '2' refer to the principal directions, calculated based on Mohr's circle.

The first direction is the direction of maximum tension (or minimum compression). The second direction is the direction of maximum compression (or minimum tension).

Keep in mind that the most economic reinforcement paths are the ones that follow the trajectories of the principal directions!

Left Comparison Mindlin ⇔ Kirchhoff

⇒ Shear force vx

Combination = ULS; Type forces = Basic magnitudes; Envelope = Maximum; Values = v_x



⇒ Torsion moment mxy



Combination = ULS; Type forces = Basic magnitudes; Envelope = Maximum; Values = m_xy

Conclusion: Kirchhoff gives the expected shear force values, Mindlin gives the expected torsion moments.

2.4.3 Concrete setups

GENERAL SETUPS

⇒ Setup 1: National Determined parameters

File \rightarrow Project settings \rightarrow National annex [...] \rightarrow EN 1992-1-1 [...] OR

Click on the flag at the top right of SCIA Engineer \rightarrow Manage annexes \rightarrow EN 1992-1-1 [...]

Type of values	E Standard EN	Name	Standard EN		
NA building 🗸	Concrete	 Concrete 			
Type of functionality	General	▲ General			
Type of functionality Hollow core beams Prestressing	General Concrete Non-prestressed reinforcement Prestressed reinforcement Durability and concrete cover ULS General Detailing provisions Common detailing provisions 2D structures and slabs Punching	 General Concrete National annex EN_1992_1_1 Y_{SH}-partial factor for shrinkage acti- Value [-] Y_C - partial factor for design values (-) f_{ck,max} - maximum value of the char Value [MPa] a_{cc} - coeff. taking account of long ter Value [-] a_{cc} - coeff. taking account of long ter Value [-] k_{1,red} - coeff. for calculation of ratio Value [-] k_{2,red} - coeff. for calculation of ratio Value [-] k_{3,red} - coeff. for calculation of ratio Value [-] 	1.00 1.50/1.20 90.00 1.00 1.00 0.44 Formula 0.54		
		k _{5,red} - coeff. for calculation of ratio Value [-]	0.70		
		4 k _{6,red} - coeff. for calculation of ratio Value []	0.90		
		value [-]		10	

⇒ Setup 2: Concrete settings

Concrete menu \rightarrow Concrete settings

Complete setup	It Find						Nation	al annex:	
Description	Symbol	Value	Default	Unit	Chapter	Code	Struct	CheckT	Remark
م II>	<all> \wp</all>	<all></all>	o ⊲all> 🔎)	<all> \wp</all>	<all> \wp</all>	<all> 🔎</all>	<all> 🔎</all>	
Design defaults				1	-				
Reinforcement									
Minimum cover									
Solver setting									
∡ General									
Limit value of unity check	Lim.check	1.0	1.0			Independe	All (Bea	Solver se	
Value of unity check for not calculated unity check	Ncal.check	3.0	3.0			Independe	All (Bea	Solver se	
The coefficient for calculation effective depth of cros	. Coeff _d	0.9	0.9			Independe	All (Bea	Solver se	
The coefficient for calculation inner lever arm	Coeffz	0.9	0.9		1	Independe	All (Bea	Solver se	
The coefficient for calculation force, where member	Coeff _{com}	0.1	0.1			Independe	All (Bea	Solver se	
 Creep and shrinkage 									<<
Age of concrete at the moment considered	t	18250.00	18250.00	day	3.1.4.B.1-2	EN 1992-1-1	All (Bea	Solver se	
Relative humidity	RH	50	50	%	3.1.4.B.1-2	EN 1992-1-1	All (Bea	Solver se	
Type input of creep coefficient	Type φ(t,to)	Auto	Auto		3.1.4(2)	EN 1992-1-1	All (Bea	Solver se	
Age of concrete at loading	t ₀	28.00	28.00	day	3.1.4(2),B1	EN 1992-1-1	All (Bea	Solver se	
Consider drying and autogenous shrinkage	Type ε _{cs} (t,ts	Auto	Auto		3.1.4(6)	EN 1992-1-1	All (Bea	Solver se	
Age of concrete at the beginning of drying shri	t _s	7.00	7.00	day	3.1.4(6),B2	EN 1992-1-1	All (Bea	Solver se	
▲ SLS									
Use effective modulus of concrete					7.1(2)	EN 1992-1-1	All (Bea	Solver se	
Default sway type				1					
Sway around y axis	Sway yy	2				Independe	All (Bea	Solver se	
Sway around z axis	Sway zz	2				Independe	All (Bea	Solver se	
Internal forces									

All of the adjustments made in one of the two general setups are valid for the whole project.

HEMBER DATA

It is possible to **overwrite** the data from the general setups per 2D member, namely by means of Member data. This is automatically defined on any 2D member (same is true for 1D members).

2D MEMBER (1) > 0	CONCRETE 2D DATA (1)	F
≠ 🗲 💶 🖾		
Name	CMD2D	
2D member	\$1	
Member type	Plate	\sim
REINFORCEMENT PLATE LONGITUDINAL		
Design of provided reinforcement		
Design template of provided reinforc	Plate	~ =
Material	B 500B	~ =
UPPER (Z+)		
Type of cover	Auto	~
Type of first layer	Principal	\sim
Diameter of first layer [mm]	10,0	\sim
Angle of first layer direction [deg]	0,00	
Type of second layer	Principal	~
Diameter of second layer [mm]	10,0	\sim
Angle of second layer direction [deg]	90,00	
I THE REAL PROPERTY AND ADDRESS OF THE REAL PROPERTY ADDRESS OF THE REAL PROPER		

2.4.4 ULS design

REINFORCEMENT DESIGN

⇒ Internal forces

Design menu \rightarrow Concrete 2D \rightarrow Internal forces

Basic (centroid) : the values shown here are exactly the same as in the Results menu; they are calculated by the FEM solver.

Design (centroid) : the values shown here are different from those in the Results menu.

- The design magnitudes in the **Results** menu are calculated by the **FEM** solver according to some simple formulas specified in EC-ENV.
- The design magnitudes in the **Concrete** menu are calculated by the **NEDIM** solver, where a much finer transformation procedure is implemented, based on the theory of Baumann. These are the values that will be used for the SCIA Engineer reinforcement design.

Theory of Baumann:

1) Calculation of the lever arm.

The lever arm is necessary for the calculation of surface forces. Value z will be calculated in the direction of the angle of the first principal moment. The forces will be recalculated and a cross-section set will be created in this direction. The reinforcement will be designed for these recalculated forces and from the designed reinforcement the inner lever arm will be calculated.

Principal stresses and directions at both surfaces

$$\sigma_{I_{-}} = 0.48 \text{ MPa } \sigma_{II_{-}} = 0.11 \text{ MPa } -> \alpha_{z_{-}} = -5.86 = -5.86^{\circ}$$

 $\sigma_{I_{+}} = -0.11 \text{ MPa } \sigma_{II_{+}} = -0.48 \text{ MPa } -> \alpha_{z_{+}} = -5.86^{\circ}$
 $-> \text{ direction for calculation inner lever arm}$
 $\alpha_{z} = -5.86$
Recalculated forces to direction of inner lever arm
 $n_{z} = 0.0 \text{ m}_{z} = 4970.4$
 $f_{cd} = \frac{\alpha_{cc} \cdot f_{ck}}{\gamma_{c}} = \frac{1 \cdot 20 \cdot 10^{6}}{1.5} = 13.33 \text{ MPa}$
 $d = 210 \text{ mm}$
 $\eta = 1 - 0.5 \cdot \frac{\varepsilon_{c2}}{\varepsilon_{cu2}} = 1 - 0.5 \cdot \frac{0.0018}{0.0035} = 0.75$
 $\beta = 1 - \frac{\frac{\varepsilon_{cu2}^{2}}{2} - \frac{\varepsilon_{c2}^{2}}{6}}{\varepsilon_{cu2}^{2} - \frac{\varepsilon_{c2}^{2}}{2}} = 1 - \frac{0.0035^{2}}{2} - \frac{0.0035 \cdot 0.0018}{2} = 0.389$
 $\xi_{bal} = \frac{\varepsilon_{cu2}}{\varepsilon_{cu2} + \frac{f_{yk}}{\gamma_{S} \cdot E_{s}}} = \frac{0.0035}{0.0035 + \frac{500}{1.15 \cdot 200000}} = 0.617$
 $x_{bal} = \xi_{bal} \cdot d = 0.617 \cdot 210 = 0.13$
 $n_{cbal} = -\xi_{bal} \cdot d \cdot b \cdot \eta \cdot f_{cd} = -0.617 \cdot 210 \cdot 1000 \cdot 0.75 \cdot 13.33 = -1295 \text{ kN/m}$
 $n_{z} = 0 \text{ kN/m} > n_{c,bal} = -1295 \text{ kN/m} = > \text{ predominant tension}$
 $x = \frac{d}{2 \cdot \beta} \cdot \left(1 - \sqrt{1 - 4 \cdot \beta \cdot \frac{abs(m_{z}) - n_{z} \cdot (d - 0.5 \cdot h)}{100 \cdot 0.21^{2} \cdot 0.75 \cdot 13.33}}\right) = 2 \text{ mm}$
 $z = d - \beta \cdot x = 210 - 0.389 \cdot 2 = 209 \text{ mm}$
 $z_{+} = 124 \text{ mm}$
 $z_{+} 85 \text{ mm}$

If value z cannot be calculated it will be calculated according to formula: z = 0,9 * d

2) Calculation of normal forces at the surfaces of 2D element.

The inputted internal forces will be recalculated to both surfaces according the following formulas:

Lower surface

$$n_{x-} = \frac{n_x}{2} + \frac{m_x}{z} = \frac{0}{2} + \frac{4.93}{0.209} = 23.6 \text{ kN/m}$$

$$n_{y-} = \frac{n_y}{2} + \frac{m_y}{z} = \frac{0}{2} + \frac{1.22}{0.209} = 5.8 \text{ kN/m}$$

$$n_{xy-} = \frac{n_{xy}}{2} + \frac{m_{xy}}{z} = \frac{0}{2} + \frac{-0.385}{0.209} = -1.8 \text{ kN/m}$$
Upper surface

$$n_{x+} = \frac{n_x}{2} - \frac{m_x}{z} = \frac{0}{2} - \frac{4.93}{0.209} = -23.6 \text{ kN/m}$$

$$n_{y+} = \frac{n_y}{2} - \frac{m_y}{z} = \frac{0}{2} - \frac{1.22}{0.209} = -5.8 \text{ kN/m}$$

$$n_{xy+} = \frac{n_{xy}}{2} - \frac{m_{xy}}{z} = \frac{0}{2} - \frac{-0.385}{0.209} = 1.8 \text{ kN/m}$$

3) Calculation of principal forces at surfaces of 2D element.

The principal forces at both surfaces and the direction of the first principal force will be calculated according to the following formulas:

$$\begin{split} & \frac{\text{Lower surface}}{\text{Principal forces at lower surface:}} \\ & n_{I-} = \frac{n_{x-} + n_{y-}}{2} + \frac{1}{2} \cdot \sqrt{\left(n_{x-} - n_{y-}\right)^2 + 4 \cdot n_{xy-}^2} \\ & = \frac{23.6 + 5.8}{2} + \frac{1}{2} \cdot \sqrt{\left(23.6 - 5.8\right)^2 + 4 \cdot -1.8}^2} = 23.8 \text{ kN/m} \\ & n_{II-} = \frac{n_{x-} + n_{y-}}{2} - \frac{1}{2} \cdot \sqrt{\left(n_{x-} - n_{y-}\right)^2 + 4 \cdot n_{xy-}^2} \\ & = \frac{23.6 + 5.8}{2} - \frac{1}{2} \cdot \sqrt{\left(23.6 - 5.8\right)^2 + 4 \cdot -1.8}^2} = 5.7 \text{ kN/m} \\ & \text{Direction of principal forces:} \\ & \alpha_{I-} = 0.5 \cdot \text{ArcTg}\left(\frac{2 \cdot n_{xy-}}{n_{x-} - n_{y-}}\right) = 0.5 \cdot \text{ArcTg}\left(\frac{2 \cdot -1.8}{23.6 - 5.8}\right) = -6 \text{ }^\circ \end{split}$$

Upper surface

Principal forces at upper surface:

$$n_{1+} = \frac{n_{x+} + n_{y+}}{2} + \frac{1}{2} \cdot \sqrt{\left(n_{x+} - n_{y+}\right)^{2} + 4 \cdot n_{xy+}^{2}}$$

$$= \frac{-23.6 + -5.8}{2} + \frac{1}{2} \cdot \sqrt{\left(-23.6 - -5.8\right)^{2} + 4 \cdot 1.8^{2}} = -5.7 \text{ kN/m}$$

$$n_{11+} = \frac{n_{x+} + n_{y+}}{2} - \frac{1}{2} \cdot \sqrt{\left(n_{x+} - n_{y+}\right)^{2} + 4 \cdot n_{xy+}^{2}}$$

$$= \frac{-23.6 + -5.8}{2} - \frac{1}{2} \cdot \sqrt{\left(-23.6 - -5.8\right)^{2} + 4 \cdot 1.8^{2}} = -23.8 \text{ kN/m}$$
Direction of principal forces:

$$\alpha_{1+} = 0.5 \cdot \text{ArcTg}\left(\frac{2 \cdot n_{xy+}}{n_{x+} - n_{y+}}\right) - 90 = 0.5 \cdot \text{ArcTg}\left(\frac{2 \cdot 1.8}{-23.6 - -5.8}\right) - 90 = -96^{\circ}$$

4) Recalculation of principal forces at both surfaces to inputted directions.

The recalculation of the principal forces to the inputted direction will be done separately for both surfaces by using Baumann's transformation formula.

 $\frac{\text{Lower surface}}{\text{Angles for Bauman's transformation formula}} \\ \alpha_{1-} = \alpha_{\text{inp,1-}} - \alpha_{I-} = 0 - -6 = 6^{\circ} \\ \alpha_{2-} = \alpha_{\text{inp,2-}} - \alpha_{I-} = 90 - -6 = 96^{\circ} \\ \alpha_{3-} = \alpha_{\text{con-}} - \alpha_{I-} = 45 - -6 = 51^{\circ} \\ \text{Recalculated dimensional forces at lower surface (acc. to Baumann)} \\ n_{\text{Eds1-}} = \frac{n_{I-} \cdot \sin(\alpha_{2-}) \cdot \sin(\alpha_{3-}) + n_{II-} \cdot \cos(\alpha_{2-}) \cdot \cos(\alpha_{3-})}{\sin(\alpha_{2-} - \alpha_{1-}) \cdot \sin(\alpha_{3-} - \alpha_{1-})} \\ = \frac{23.8 \cdot \sin(96) \cdot \sin(51) + 5.7 \cdot \cos(96) \cdot \cos(51)}{\sin(96 - 6) \cdot \sin(51 - 6)} = 25.4 \text{ kN/m} \\ n_{\text{Eds2-}} = \frac{n_{I-} \cdot \sin(\alpha_{3-}) \cdot \sin(\alpha_{1-}) + n_{II-} \cdot \cos(\alpha_{3-}) \cdot \cos(\alpha_{1-})}{\sin(\alpha_{3-} - \alpha_{2-}) \cdot \sin(\alpha_{1-} - \alpha_{2-})} \\ = \frac{23.8 \cdot \sin(51) \cdot \sin(6) + 5.7 \cdot \cos(51) \cdot \cos(6)}{\sin(51 - 96) \cdot \sin(6 - 96)} = 7.7 \text{ kN/m} \\ n_{\text{Eds3-}} = \frac{n_{I-} \cdot \sin(\alpha_{1-}) \cdot \sin(\alpha_{2-}) + n_{II-} \cdot \cos(\alpha_{1-}) \cdot \cos(\alpha_{2-})}{\sin(\alpha_{1-} - \alpha_{3-}) \cdot \sin(\alpha_{2-} - \alpha_{3-})} \\ = \frac{23.8 \cdot \sin(6) \cdot \sin(96) + 5.7 \cdot \cos(6) \cdot \cos(96)}{\sin(6 - 51) \cdot \sin(96 - 51)} = -3.7 \text{ kN/m}$

 $\begin{array}{l} \underline{\text{Upper surface}}\\ \text{Angles for Baumann's transformation formula}\\ \alpha_{1+} = \alpha_{\text{inp},1+} - \alpha_{1+} = 0 - .96 = 96 \ ^{\circ}\\ \alpha_{2+} = \alpha_{\text{inp},2+} - \alpha_{1+} = 90 - .96 = 186 \ ^{\circ}\\ \alpha_{3+} = \alpha_{\text{con+}} - \alpha_{1+} = 135 - .96 = 231 \ ^{\circ} \end{array}$

 $\begin{aligned} \text{Recalculated dimensional forces at upper surface (acc. to Baumann)} \\ n_{Eds1+} &= \frac{n_{I+} \cdot \sin(\alpha_{2+}) \cdot \sin(\alpha_{3+}) + n_{II+} \cdot \cos(\alpha_{2+}) \cdot \cos(\alpha_{3+})}{\sin(\alpha_{2+} - \alpha_{1+}) \cdot \sin(\alpha_{3+} - \alpha_{1+})} \\ &= \frac{-5.7 \cdot \sin(186) \cdot \sin(231) + -23.8 \cdot \cos(186) \cdot \cos(231)}{\sin(186 - 96) \cdot \sin(231 - 96)} = -21.7 \text{ kN/m} \\ n_{Eds2+} &= \frac{n_{I+} \cdot \sin(\alpha_{3+}) \cdot \sin(\alpha_{1+}) + n_{II+} \cdot \cos(\alpha_{3+}) \cdot \cos(\alpha_{1+})}{\sin(\alpha_{3+} - \alpha_{2+}) \cdot \sin(\alpha_{1+} - \alpha_{2+})} \\ &= \frac{-5.7 \cdot \sin(231) \cdot \sin(96) + -23.8 \cdot \cos(231) \cdot \cos(96)}{\sin(231 - 186) \cdot \sin(96 - 186)} = -4.0 \text{ kN/m} \\ n_{Eds3+} &= \frac{n_{I+} \cdot \sin(\alpha_{1+}) \cdot \sin(\alpha_{2+}) + n_{II+} \cdot \cos(\alpha_{1+}) \cdot \cos(\alpha_{2+})}{\sin(\alpha_{1+} - \alpha_{3+}) \cdot \sin(\alpha_{2+} - \alpha_{3+})} \\ &= \frac{-5.7 \cdot \sin(96) \cdot \sin(186) + -23.8 \cdot \cos(96) \cdot \cos(186)}{\sin(96 - 231) \cdot \sin(186 - 231)} = -3.7 \text{ kN/m} \end{aligned}$

5) Calculation of virtual forces at both surfaces to inputted directions.

The virtual forces are necessary to convert the pressure/tensile forces at the surface back to the center of the plate. The virtual force represents the equivalent force at the other side of the plate.





6) Recalculation of forces at surfaces to center of gravity of cross-section.

Using the transformed dimensional forces and virtual forces the internal forces at the center of the plate can be calculated.

Lower surface
Dimensional forces of lower surface transformed to centroid
$n_{Ed1-} = n_{Eds1-} + n_{Edsvirt1+} = 25.4 + -21.7 = 3.7 \text{ kN/m}$
$m_{Ed1-} = n_{Eds1-} \cdot z_{-} - n_{Edsvirt1+} \cdot z_{+} = 25.4 \cdot 8521.7 \cdot 124 = 4.9 \text{ kNm/m}$
$n_{Ed2-} = n_{Eds2-} + n_{Edsvirt2+} = 7.7 + -4.0 = 3.7 \text{ kN/m}$
$m_{Ed2-} = n_{Eds2-} \cdot z_{-} - n_{Edsvirt2+} \cdot z_{+} = 7.7 \cdot 854.0 \cdot 124 = 1.2 \text{ kNm/m}$
$n_{Ed3-} = n_{Eds3-} + n_{Edsvirt3+} = -3.7 + -3.7 = -7.4 \text{ kN/m}$
$m_{Ed3-} = n_{Eds3-} \cdot z_{-} - n_{Edsvirt3+} \cdot z_{+} = -3.7 \cdot 853.7 \cdot 124 = 0.1 \text{ kNm/m}$
Upper surface
Dimensional forces of upper surface transformed to centroid
$n_{Ed1+} = n_{Eds1+} + n_{Edsvirt1-} = -21.7 + 25.4 = 3.7 \text{ kN/m}$
$m_{Ed1+} = -n_{Eds1+} \cdot z_{+} + n_{Edsvirt1-} \cdot z_{-} = -21.7 \cdot 124 + 25.4 \cdot 85 = 4.9 \text{ kNm/m}$
$n_{Ed2+} = n_{Eds2+} + n_{Edsvirt2-} = -4.0 + 7.7 = 3.7 \text{ kN/m}$
$m_{Ed2+} = -n_{Eds2+} \cdot z_{+} + n_{Edsvirt2-} \cdot z_{-} = -4.0 \cdot 124 + 7.7 \cdot 85 = 1.2 \text{ kNm/m}$
$n_{Ed3+} = n_{Eds3+} + n_{Edsvirt3-} = -3.7 + -3.7 = -7.4 \text{ kN/m}$
$m_{Ed3+} = -n_{Eds3+} \cdot z_{+} + n_{Edsvirt3-} \cdot z_{-} = -3.7 \cdot 124 + -3.7 \cdot 85 = 0.1 \text{ kNm/m}$

The available values are: mEd,1+, mEd,2+, mEd,c+, mEd,1-, mEd,2-, mEd,c-, nEd,1+, nEd,2+, nEd,c+, nEd,1-, nEd,2-, nEd,c- and vEd. "+" and "-" stand for the design values at respectively the positive and the negative side of the local z-axis of the 2D member. "1" and "2" stand for the reinforcement directions, which are by default respectively the local x- and y- direction of the 2D member. (mEd,c+ and mEd,c- are the design moments that would have to be taken by the concrete, but they have no real significance for the reinforcement design.)

Combination = ULS; Type values = Design internal forces; Value = mEd,1+



Compare the result for this value mEd,1+ (Concrete menu) with the result for the equivalent value mxD+ (Result menu) shown on page 120.

Despite the different transformation procedures, the general image of the results will be similar for *orthogonal* reinforcement directions (acc. to the local x and y axes). The largest difference is caused by the shift rule that is only taken into account in the design magnitudes calculated by the NEDIM solver (values mEd,1 and mEd,2).

The <u>shift rule</u> takes into account the additional tensile force caused by the shear force by shifting the moment line by a distance a_i . a_i is determined as in the image below.



The shift rule is taken into account in the default concrete settings. You can deactivate this option in the concrete settings.

oncrete settings									- 0
ews: Complete setup 👻 View sett 🔻 Load default Find						N	Natio	onal annex:	
Description	Symbol	Value	Default	Unit	Chapter	Code	Structure	CheckTy	Remark
<all></all>	<all></all>	₽ <all></all>	P <all></all>	Q ~ Q	<all></all>	all>	all>	<all> D</all>	
Design defaults									
Reinforcement									
Minimum cover									
 Solver setting 									
▷ General									
▲ Internal forces									1D: $a_i = \operatorname{Coeff}_{e} \cdot d \cdot (\cot \theta - \cot \alpha) / 2$ 2D: $a_i = d$
Shear force reduction above supports					6.2.1(8)	EN 1992-1-1	Beam,Be.	. Solver set	
Moment reduction above supports		<u>.</u>			5.3.2.2 (4)	EN 1992-1-1	Beam,Be	. Solver set	₩ ₩
Shifting of moment curve to cover additional tensile force caused by shear					9.2.1.3(2)	EN 1992-1-1	Beam,Ri	Solver set	
Geometric imperfection in ULS	ei,ULS				5.2(2)	FN 1992-1-1	Column	Solver set	
Geometric imperfection in SLS	e _{i,SLS}				5.2(3)	EN 1992-1-1	Column	Solver set	<<
Minimum eccentricity	e _{min}	In first or	der In first or		6.1(4)	EN 1992-1-1	Column	Solver set	If the check box is ON, the additional tensi
First order eccentricity with the equivalent moment		S			5.8.8.2(2)	EN 1992-1-1	Column	Solver set	force caused by the shear force is taken
Second order eccentricity	e2				5.8.8	EN 1992-1-1	Column	Solver set	into account using the shift rule
Internal forces modifications									
Design As									
Conversion to rebars									
Interaction diagram									
▶ Shear									
Torsion									
Punching									
Stress limitations									
Cracking forces									

If we uncheck this option the general image of mEd,1+ is closer to the one obtained for mxD+ (page 120).



⇒ Provided reinforcement

Before calculating the theoretical reinforcement it is possible to add a template of reinforcement to your plate(s). This template can be used to:

- Compare the template with the calculated theoretical reinforcement. By doing this it is easy to see where this basic template is not sufficient.
- Perform the punching design, Crack width check and the code dependent deflections.

The reinforcement added by the template is called **Provided reinforcement**.

To add **Provided reinforcement** go to Concrete menu → Concrete settings → Design defaults



Click on the 3 dots next to the 'Design template of provided reinforcement'. This opens a window with all the default templates.



You can select one of these templates, make a new one or edit one of the existing templates. Select the first template and click 'Edit'.

Member Plate, Shell(Plate) Image: Cross-section Englisit En	Provided rein	nforcement (design) edit	t - Plate_B	asic_AddList_All								- 0) >	<
x x	Member Cross-sectic Mode	Plate, Shell(Plate) Standard	* *		Longitudinal	sic	By Dian	neti 👻						
Layer Diameter Spacing Area Type Diameter Spacing Area (+) (1) <						Bas	sic reinforce	ment		Additional	reinforcement			
x [m] [m] [mm]					Layer	Diameter	Spacing	Area	Туре	Diameter	Spacing	Are	a	
(+) (1) (2) (1) ([mm]	[mm]	[mm^2/m]	-	[mm]	[mm]	[mm^2/r	n]	
(+) (1) (2) 393 Listby spacing 10.0 0;100;150;200 0;785;524;333 [1-1] 10.0 150 524 Listby spacing 10.0 0;100;150;200 0;785;524;333 [2-1] 10.0 150 524 Listby spacing 10.0 0;100;150;200 0;785;524;333 [2-1] 10.0 150 524 Listby spacing 10.0 0;100;150;200 0;785;524;333		∆z			[1+]	10.0	200	393	List by spacing	10.0	0;100;150;200	0;785;524	4;393	
(+) (1) (2) (-) (-) (-) (-) (-) (-) (-) (-) (-) (-					[2+]	10.0	200	393	List by spacing	10.0	0;100;150;200	0;785;524	4;393	
(+) (-) x 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0					[1-]	10.0	150	524	Listby spacing	10.0	0;100;150;200	0;785;524	4;393	
					[2-]	10.0	150	524	List by spacing	10.0	0;100;150;200	0;785;524	4;393	
	(-) ×		000	2)										

In this window the reinforcement can be defined. There are 2 types of reinforcement in templates:

- Basic reinforcement: This type of reinforcement is added over the entire plate.
- Additional reinforcement: This type of reinforcement is only added in zones where, according to the calculated theoretical reinforcement, extra reinforcement is needed. You can define a single diameter and spacing as extra reinforcement. Or a list of reinforcement with either various diameters or various spacings.

<u>Note:</u> In the design defaults you can change the reinforcement directions. These directions are respected by as well the provided as the theoretical required reinforcement.

Com	nplet	e setup	✓ View sett Load default Find									Nation	al annex: 🧃
Descri	iptio	ı			Symbol		Value	Default	Uni	Chapter	Code	Struct	CheckT
				P	<all></all>	P	<all></all>	<all></all>	P	<all> \wp</all>	<all> D</all>	<all> D</all>	<all> D</all>
esign	n def	aults											
A Re	info	cement											
Þ	Bea	m / Rib											
₽	Bea	m slab											
Þ	Col	umn											
	Pla	te											
		Longitudii	nal										
		Design of provided reinforcement									Independe	Plate,S	Design d
		Desig	m template of provided reinforcement				Plate_Basic_Ad	Plate_Ba			Independe	Plate,S	Design d
		 Upper 	(z+)										
		Тур	pe of cover		Type _{C+}		Auto	Auto		4.4.1	EN 1992-1-1	Plate,S	Design d
		Dia	ameter of first layer		d _{s1+}		10.0	10.0	mm		EN 1992-1-1	Plate,S	Design d
		An	gle of first layer direction		α ₁₊		0.00	0.00	deg		EN 1992-1-1	Plate,S	Design d
		Dia	ameter of second layer		d _{s2+}		10.0	10.0	mm		EN 1992-1-1	Plate,S	Design d
		An	gle of second layer direction		α ₂₊		90.00	90.00	deg		EN 1992-1-1	Plate,S	Design d
		Lower	(z-)										
		Тур	be of cover		Type _c .		Auto	Auto		4.4.1	EN 1992-1-1	Plate,S	Design d
		Dia	ameter of first layer		d _{s1-}		10.0	10.0	mm		EN 1992-1-1	Plate,S	Design d
		An	gle of first layer direction		α ₁₋		0.00	0.00	deg		EN 1992-1-1	Plate,S	Design d
		Dia	ameter of second layer		d _{s2-}		10.0	10.0	mm		EN 1992-1-1	Plate,S	Design d
		An	gle of second layer direction		a2-		90.00	90.00	deg		EN 1992-1-1	Plate,S	Design d

⇒ Theoretical reinforcement

Concrete Menu→ ULS & SLS 2D Reinforcement design

In the menu Reinforcement design (ULS) you have 5 types of values:

• **Required:** These values represent the theoretical reinforcement calculated by SCIA Engineer. This takes into account the detailing provisions.

Pla	ite, Shell(Plate)		-
	Longitudinal		-
	Check min. ratio of principal reinforcement		
	Type of the minimum tension principal reinforcement f		Auto
	Type of the minimum tension principal reinforcement f		Auto
	Check max. ratio of principal reinforcement		
	Check min. transverse ratio of secondary reinforcement		
	Check min. bar distance		
	Minimal bar distance sl	lp.min	20
	Check max.spacing of principal longitudinal reinforcement		
	Check max.spacing of secondary longitudinal reinforcem		
	Shear		-
	Check min. ratio of shear reinforcement		
	Check min. thickness of member with shear reinforcement		2
	Min. thickness of member with shear reinforcement h	⁾ min	200
	Check max. spacing of shear links		2
	Max. spacing of shear links C	Coeff _{smax.p.s}	0.8



As,req1+: Theoretical required reinforcement on the top side of the plate (positive z direction) in the first reinforcement direction. Taking into account the detailing provisions.

• **Required – Statically:** These values represent the theoretical reinforcement calculated by SCIA Engineer **without** the detailing provisions taken into account.



As,ult1+: Theoretical required reinforcement on the top side of the plate (positive z direction) in the first reinforcement direction. **Without** taking into account the detailing provisions.

• **Required – Not covered:** These values show if there is extra reinforcement needed on top of the provided reinforcement. Areas where this value is 0 are areas where no extra reinforcement is needed (compared to the provided reinforcement). Areas where these values are not 0 are areas where the provided reinforcement is not sufficient.



 Δ As,req1+: Theoretical additional required reinforcement on top of the provided reinforcement on the top side of the plate (positive z direction) in the first reinforcement direction.

• **Provided:** These values show you the provided reinforcement defined in the templates.



As,prov,1+ or N ϕ ,prov,1+: Provided reinforcement on the plate in mm²/m or as the amount of reinforcement respectively. If elements are red the additional reinforcement in the template is not sufficient.

- **Provided Utilization:** Unity checks where provided reinforcement is compared to the required reinforcement. This will give you an idea of the efficiency of the reinforcement.
- ⇒ Calculation of longitudinal reinforcement

The theoretical longitudinal reinforcement is calculated out of the design internal forces.



⇒ Calculation of shear reinforcement

Before calculating the shear reinforcement two checks are done:

• V_{Ed} ≤ V_{Rd,max}: The design internal forces on the plate should be lower or equal to the maximum shear resistance of the plate.

$$v_{Rd,max} = \frac{\alpha_{cw} \cdot b_{w} \cdot z \cdot v_{1} \cdot f_{cd}}{\left(\cot g\left(\theta\right) + tg\left(\theta\right)\right)}$$

• V_{Ed} < V_{Rdc}: If V_{Ed} is smaller than V_{Rdc} no shear reinforcement is required. If this is not the case punching shear reinforcement will be automatically calculated by SCIA Engineer.



Check shear capacity (without shear reinforcement)

 $\label{eq:keylength} \begin{array}{l} \underline{Check \; v_{Rd,max}} \\ v_{Ed} = 82.3 \; kN/m \, \le \, v_{Rd,max} \; = 878 \; kN/m \; \mbox{(OK)} \\ \underline{Check \; v_{Rdc}} \\ v_{Ed} = 82.3 \; kN/m \; <= v_{Rdc} = 112 \; kN/m \; \mbox{(OK, no shear reinforcement is required)} \end{array}$

When $V_{Ed} > V_{Rd,max}$ the following error appears in the output of the reinforcement design.

	Punching shear resistance at the column	Increase the column size or change plate
Warning	perimeter (vRd,max) is not sufficient acc. to	properties (use higher grade of concrete
	§6.4.3(2).	material or increase the thickness).

This error message is found at locations with high peak values for the shear stress. Most of the time these peak values are singularities, and do not occur in reality. You have roughly 2 options: you can just ignore the peaks or average them, for example by means of Averaging strips.

Practical reinforcement design

Next to theoretical required and provided reinforcement you have also practical or **User** reinforcement. This type of reinforcement can be added to the plate via the Concrete menu \rightarrow 2D Reinforcement.

Reinforcement 2D		3
	Name RR1	
	2D member Slab1	
111111 //	A Reinforcement	
	Type Bars	
HH	Material B 500A	۷.,
	Surface Upper	
	Number of directions 2	
	Direction closest to surface 1	
7	Angle of first direction [deg] 0.00	
	41	
cl	Diameter (dl) [mm] 10.0	
s s du	Concrete cover (cl.cu) [mm] 30	
+ + + >	Bar distance (sl) [mm] 200	
	Offset [mm] 0	
	Reinf, area [mm^2/m] 393	
	4 2	
	Diameter (dl) [mm] 10.0	
	Concrete cover (cl.cu) [mm] 40	
	Bar distance (sl) [mm] 200	
	Offset [mm] 0	
	Reinf area (mm^2/m) 393	
	Total weight [kg] 58.26	
	4 Geometry	
	Geometry defined by Polygon	,
	Actions	
	1	oad from setup >>>
		OK Cancel
		Cancer

This reinforcement is to be added separately at the upper and lower side, and in the different reinforcement directions.



<u>Note:</u> You can add multiple layers of practical reinforcement on the same area. The reinforcement added to this area is the sum of all these layers.

Combination Provided reinforcement and user reinforcement

After running the reinforcement design, it might be possible the provided reinforcement is insufficient in certain areas. This means you should introduce some additional reinforcement. In this case you can apply two different workflows:

- (a) Define all the reinforcement as practical reinforcement;
- (b) Combine the provided reinforcement and the practical reinforcement which will only be defined in the areas where it is necessary to define additional reinforcement.

This principle will be explained by using the following example for the ULS reinforcement design in **direction 1 or the local x-direction**. Within the design defaults, you can define a template for the provided reinforcement which can be used within the actual design. In this case the basic reinforcement will be set to **Ø10 à 150** and the addition reinforcement will be set to zero.



When running the ULS design for the value **As_prov,1-**, it can be seen the provided reinforcement of **Ø10** à **150** will be insufficient to withstand the acting loads. This indicates the application of additional reinforcement will be necessary.



When generating the value **As_add,req,1-**, you can see the exact amount of reinforcement in mm2/m which needs to be added on top of the provided reinforcement. In this case an additional reinforcement of **578 mm2/m** will be necessary. This value can be translated into the configuration of **Ø10 à 100** as practical reinforcement.



This value can be translated into the configuration of **Ø10 à 100** as practical reinforcement. Since there is no required additional reinforcement in direction 2, only one direction of reinforcement will be added to the 2D member by using practical reinforcement as defined within the previous section.

a statistic contraction CO		
	News PRo	
/	Name RR3	
hun l	2D member stabi	
	A Reinforcement	
	Type Bars	
 /	Material B 500A	Υ.
	Surface Lower	
	Number of directions 1	
	Angle of first direction [deg] 0.00	
	Diameter (dl) [mm] 10.0	
• • • •	Concrete cover (cl,cu) [mm] 40	
Cl	Offset [mm] 0	
sl sl ødi	Bar distance (sl) [mm] 100	
	Reinf. area [mm^2/m] 785	
	Total weight [kg] 359.44	
	▲ Geometry	
	Geometry defined by Polygon	
	Actions	
		Load from setup >>>
		OK Calleet

When generating the results once more for the value **As_prov,1-** and activating the option '**Consider user reinforcement**', it can be seen the user defined reinforcement of **Ø10** à **100** is added on top of the basic reinforcement of **Ø10** à **150** which is defined within the design defaults.



The applied values are visible within the preview of the reinforcement design.

	Basic	c Additional		α	A _{s,min}	A _{s.ult}	ΔA _{s.serv}	A _{s.serv} A _{s.reg}	A _{s.prov}	A _{s.max}	Smin(cl)	s _{max}	Status		
		User	Auto	[°]	[mm ²]	[mm ²]	[mm ²]	[mm ²]	[mm ²]	[mm ²]	[mm]	[mm]			
[2+]	φ10/150			90.0	277	53		277	524	10000	58	60	ОК		
8 194								0.11%	0.21%		≥37	≤400			
[1-]	φ10/150	φ 10/100		0.0	291 1102	291	0.0 291	1102	1000	1102	1309	10000	55	60	ОК
	-							0.44%	0.52%		≥37	≤400			
[2-]	φ10/150			90.0	277	70		277	524	10000	58	60	ок		
					ADDATE:			0.11%	0.21%		≥37	≤400			

The option '**Consider user reinforcement**' is also accessible within all the reinforcement checks – crack width, punching and CDD. This allows you to easily check the reinforcement introduced by both the template and the practical bars.

2.4.5 SLS Design of 2D members – Crack width and stress limitation

Next to the ULS design of 2D members the Eurocode defines some restrictions related to SLS design as well, more specifically the crack width and the limitation of the tensile stress in the reinforcement. Due to these SLS conditions you might need to increase the amount of reinforcement which should be sufficient to withstand the acting ULS forces. The total amount of reinforcement to fulfil the conditions for both the ULS and SLS design can be calculated within SCIA as well as the increment of statically required reinforcement.

The principle of this design method will be explained by the following example of a 2D plate. On this member CMD will be applied in which the crack width in the first direction at the bottom surface will be limited to **0,100 mm**. The tensile stress in the reinforcement can be limited both within the design defaults and the CMD. In this example the limit will be set to **150 MPa**.

CMD			5
▲ Design As			1
▲ Plate, Wall	, Shell(Plate), Shell(Wall), Deep Beam		
Coefficient fo	r increasing the statically required area of reinforceme 0.00		
Coefficient fo	r increasing the statically required area of reinforceme 0.00		
▲ Interaction	diagram		
	Interaction diagram method NRdMRd		*
▲ Shear			
	Type calculation/input of angle of compression strut User(angle)		*
	Angle of compression strut [deg] 40.00		
	Cotangent angle of compression strut 1.19175359259	421	
Stress limit	ations		
	Stress limit in the reinforcement User input		¥
	Limit stress in reinforcement [MPa] 150.0		
Cracking for	rces		
	Type of strength for calculation of cracking force f_{ctm}		*
	Value of strength for calculation cracking force f_{ct,eff}		*
Crack width			
	Type of maximal crack width User-defined for	or different surfaces	*
	User defined crack width for upper surface [mm] 0.100		
	User defined crack width for lower surface [mm] 0.100		
Chapter : 7.3.1(5) Code : EN 1992-1- Remark : User de	-1 fined crack width for lower surface w-		OK Cancel
	▲ Stress limitations		
	Indirect load (imposed deformation)	1771	
	Stress limit in the reinforcement	Auto	Auto
	Cracking forces	Auto	
	Crack width	Yield strength	
	Deflections	User input	

Since this design method is applicable for the ULS and SLS, it is important to select a result class which contains both ULS and SLS combination.

RESU	JLTS (1)					
Name	Reinforcement design (ULS+SLS					
 SELECTION 						
Type of selection	All \checkmark					
Filter	No 🗸					
 RESULT CASE 						
Type of load	Classes 🗸					
Class	All ULS+SLS \lor					
Envelope (for 2D drawing)	Absolute extreme V					

The first step of the design procedure consists of the determination of **As_req** for the ULS state for each direction and each surface. During this step SCIA will determine two values, more specifically:

(a) As_ult: the statically required reinforcement to withstand the ULS acting forces;

(b) **As_req:** the required reinforcement including the detailing provisions from the EN.

When looking at the given example, it can be seen the required reinforcement As_req,1- is equal to 1614 mm2/m. The statically required reinforcement As_ult,1- is equal to 1102 mm2/m. This value is a bit lower since it does not contain the increment of longitudinal reinforcement due to the SLS design.



After the calculation of **As_ult** you can choose to integrate the SLS restriction and you have three possibilities:

- Combination of the ULS and SLS design based on cracks.
- Combination of the ULS and SLS design based on stress limitation.
- Combination of the ULS and SLS design based on cracks and stress limitation.

This can be defined within the properties of the reinforcement design.


After activating these settings the increment of longitudinal reinforcement can be generated, in this case the value Δ **As_serv,1-**. SCIA will determine the principal forces **mEd,ch** and **mEd,QP** in order to calculate the appearance of cracks based on the designed ULS reinforcement **As_ult**. Next to the principal forces it is also necessary to calculate the amount of reinforcement in the direction of the principal forces.

Within the following step, SCIA will determine the maximum allowable crack width based on chapter 7.3.4 from EN 1992-1-1:2004 and compare it to the defined limit as shown below.

Principal stress σ _l [-]=-4.58°	
m _{Ed,char} = 65 kNm/m n _{Ed,char} = 0 kN/m	
$m_{Ed,qp} = 47 \text{ kNm/m} n_{Ed,qp} = 0 \text{ kN/m}$	
Recalculation of required areas to direction of principal stress	
$A_{s,ult,\sigma} = A_{s,ult,1-} \cdot \cos\left(\Delta\alpha_{1-}\right)^2 + A_{s,ult,2-} \cdot \cos\left(\Delta\alpha_{2-}\right)^2$	
$= 1102 \cdot \cos(-5)^2 + 277 \cdot \cos(-95)^2 = 1097 \text{ mm}^2$	
$A_{s,serv,\sigma} = A_{s,ult,\sigma} + \Delta A_{s,serv,1-} \cdot \cos(\Delta \alpha_{1-})^2 + \Delta A_{s,serv,2-} \cdot \cos(\Delta \alpha_{2-})^2$	
$= 1097 + 511 \cdot \cos\left(-5\right)^2 + 0 \cdot \cos\left(-95\right)^2 = 1605 \text{ mm}^2$	
Check of cracks occuring	(§7.1(2))
f _{ct,eff} = 2.6 MPa	
σ_{ct} = 5.716 MPa > σ_{cr} = 2.6 MPa => cracks appear	
Check of reinforcement stress limitation	(§7.2(5))
σ _s = 149.3 MPa ≤ σ _{s,lim} = 150 MPa	
Effective tension area	(§7.3.2(3))
$h_{c,eff} = 64.4 \text{ mm} \Rightarrow A_{s,eff} = 1605 \text{ mm}^2 (\rho_{p,eff} = 2.49 \%)$	
Calculation of crack width	(§7.3.4)
$s_{r,max} = k_3 \cdot c + \frac{k_1 \cdot k_2 \cdot k_4 \cdot \phi_{eq}}{\rho_{p,eff}} = 3.4 \cdot 0.03 + \frac{0.8 \cdot 0.5 \cdot 0.425 \cdot 0.01}{0.0249} = 170 \text{ mm}$	(7.11)

$$\epsilon_{sm} \epsilon_{cm} = \max\left(\frac{\sigma_{s} - k_{t} \cdot \left(\frac{f_{ct,eff}}{\rho_{p,eff}}\right) \cdot \left(1 + \alpha_{e} \cdot \rho_{p,eff}\right)}{E_{s}}, \frac{0.6 \cdot \sigma_{s}}{E_{s}}\right)$$

$$= \max\left(\frac{149.3 - 0.46 \cdot \left(\frac{2.6}{0.0249}\right) \cdot \left(1 + 6.35 \cdot 0.0249\right)}{200000}, \frac{0.6 \cdot 149.3}{200000}\right) = 0.468 \%$$

$$w_{k} = s_{r,max} \cdot \epsilon_{sm} \epsilon_{cm} = 170 \text{ mm} \cdot 0.468 \% = 0.0797 \text{ mm}$$
Check of crack width
$$w_{k} = 0.0797 \text{ mm} \le w_{max} = 0.1 \text{ mm}$$

If the cracks are within the limit, then **As_ult** is sufficient to fulfill the restrictions for both ULS and SLS. If not, then SCIA will start the iteration process to increase the **As,ult** by an extra amount of reinforcement to ensure the crack width is within the allowable limits. When looking at the table below it can be seen an additional amount of **1166 mm2/m** for the first direction at the bottom of the member should be added to the reinforcement **As_ult,1-**.

	Basic	Addi	itional	α	A _{s,min}	A _{s,ult}	ΔA _{s,serv}	A _{s,req}	A _{s.prov}	A _{s,max}	Smin(cl)	Smax	Status
		User	Auto	[°]	[mm ²]	[mm ²]	[mm ²]	[mm ²]	[mm ²]	[mm ²]	[mm]	[mm]	
[2+]	φ10/150			90.0	277	53	0	277	524	10000	58	60	OK
								0.11%	0.21%		≥37	≤400	
[1-]	φ10/150	φ10/100		0.0	291	1102	511	1613	1310	10000	55	60	Not O
							1.0056230127	0.65%	0.52%		≥37	≤400	
[2-]	φ10/150		222	90.0	277	70	0	277	524	10000	58	60	OK
								0.11%	0.21%		≥37	≤400	

When looking at the output for Δ **As_serv,1-** a value of **562 mm²/m** can be generated.



If this value of $\Delta As_serv,1$ will be added to the value of $As_ult,1$ -, it will result in the value $As_req,1$ -. In short the following summary can be created:

- **As_req,i,+/-:** Required reinforcement area for ULS and SLS including detailing provisions for the particular direction (1,2) and surface (+,-).
- As_ult,i,+/-: Statically required reinforcement based on ULS for particular direction (1,2) and surface (+,-).
- ΔAs_serv,i,+/-: Increment of statically required reinforcement based on SLS for particular direction (1,2) and surface (+,-).

The same procedure can be applied for the limitation of tensile stress within the reinforcement. In this case SCIA will determine the amount of reinforcement for the ULS and use this reinforcement to calculate the actual stresses in the reinforcement. This value will then be compared to the defined allowable limit. The limit can be defined in both the design defaults and CMD. You have three possibilities to define the limit of the stresses:

- Auto: based on definition in the national annexes 7.2(5).
- Yield Strength: the limit is determined based on fyk (characteristic yield strength of reinforcement)
- User input: the limit must be decided by the user.

This can be checked within the output, in this case the user defined value of **150 MPa** can be seen.



As previously mentioned when the SLS restrictions are not fulfilled an increment must be calculated **serv_coeff** will be calculated depending on the following conditions:

- In case of crack width only: serv_{coeff}=w_{k,coeff}= (w_k / w_{k,max})^{0,5}+0,01
- In case of reinforcement stress only: serv_{coeff}=s_{s,coeff}= (s_s / s_{s,lim})+0,005
- In case of reinforcement stress only: serv_{coeff} = max(s_{s,coeff};w_{k,coeff})

When the statically reinforcement is designed based on ULS +SLS, the verification of the detailing provisions must be done. The same procedure and warnings as used for ULS design will be applied for ULS+SLS design, only one step further. The final reinforcement area As_req for direction (1,2) and surface (+,-) will be determined by the following formula, taking into account the minimal and maximal areas from detailing provisions:

 $A_{s,req,1,2,\pm} = min (max(A_{s,ult,1,2,\pm}; A_{s,serv,1,2,\pm}; A_{s,min}); A_{s,max})$

2.4.6 Crack control

INPUT DATA FOR CRACK CONTROL

⇒ Maximum crack width

The values of the maximum crack width (w_{max}) are national determined parameters, dependent on the chosen exposure class. Therefore, this value can be found in the setup for National Determined Parameters, via the File menu \rightarrow Project settings \rightarrow National annex [...] \rightarrow EN 1992-1-1 [...].

LI Concrete setup		
 Type of values NA building Type of functionality Hollow core beams Prestressing 	CC-EN Concrete Concrete Concrete Non-prestressed reinforcement Prestressed reinforcement ULS General Octaling provisions Concrete Octaling provisions Detailing provisions Punching	Name EC-EN Concrete General ULS SIS General National annex Value [-] 3.40 Value [-] 3.40 Value [-] 3.40 Value [-] 0.42 Value [-] 0.42 Value [-] 0.42 Detailing provisions
Select all Unselect all	Refresh	Load default NA parameters OK Ca

⇒ Type of used reinforcement

You can perform the Crack width check for all three types of reinforcement (Required, provided and user reinforcement). The crack width check is performed on a Quasi-permanent SLS combination.

If the type of reinforcement used for the crack width check is either the provided or required reinforcement an ULS combination should be chosen as well. This is necessary because the required/provided reinforcement is calculated based on an ULS combination. After this reinforcement is calculated it can be used to perform the crack width check. All this is done automatically and can be set in the properties window of the crack width check.



Required/provided reinforcement



⇒ Theoretical background

Crack appearance

If condition below is satisfied no cracks will appear in the concrete.

$$\sigma_{ct,max\pm} \leq f_{ct,eff}$$

With:

- $\sigma_{ct,max\pm} = \frac{n_{i\pm}}{A_{i,i\pm}} + \frac{m_{i\pm}}{I_{i,i\pm}} \cdot z_{t,max,i\pm} = normal \text{ concrete stress on un-cracked section at the most tensioned fiber of concrete cross-section}$
- $f_{ct,eff}$ = mean value of the tensile strength of the concrete effective at the time

Calculation of crack width

$$w = \epsilon_{sm_cm} . s_{r,max}$$

With:

$$\begin{aligned} \bullet \quad (\epsilon_{sm} - \epsilon_{cm})_{i\pm} &= \max \left[\frac{\sigma_{s,i\pm} - k_t \cdot \frac{f_{ct,eff}}{\rho_{p,eff,i\pm}} (1 + \alpha_{e,i\pm} \cdot \rho_{p,eff,i\pm})}{E_{s,i\pm}}; 0,6 \cdot \frac{\sigma_{s,i\pm}}{E_{s,i\pm}} \right] \\ \bullet \quad s_{r,max,i\pm} &= \begin{cases} \min \left(k_3 c_{i\pm} + \frac{k_{1,i\pm} k_{2,i\pm} k_4 d_{s,i\pm}}{\rho_{p,eff,i\pm}}; 1,3 \cdot (h - x_{i\pm}) \right) & \text{if } s_{s,i\pm} < 5 (c_{i\pm} + 0,5 d_{s,i\pm}) \\ 1,3 \cdot (h - x_{i\pm}) & \text{f } s_{s,i\pm} \ge 5 (c_{i\pm} + 0,5 d_{s,i\pm}) \end{cases} \end{aligned}$$

4 RESULTS FOR REQUIRED THEORITICAL REINFORCEMENT

Desing menu \rightarrow Concrete 2D \rightarrow SLS crack width

Crack width w+

Combination = SLS; Type of reinforcement = Required; Value = w+



Crack width w-

Combination = SLS; Type of reinforcement = Required; Value = w-



Unity check

Combination = SLS; Type of reinforcement = Required; Value = UC



A green value stands for a unity check ≤ 1 ($w_{calc} \leq w_{max}$), a grey value stands for unity check ≤ 0.25 and a red value means that w_{max} is exceeded.

2.5 Punching

2.5.1 Theoretical background

🖶 General

Punching shear can result from a concentrated load or reaction acting on a relatively small area, called the loaded area Aload of a slab or a foundation.

The most common situations where punching shear has to be considered is the region immediately surrounding a column in a flat ceiling plate or where column is supported on foundation plate.

The following problem types can be distinguished: interior, edge and corner columns.

Design of punching shear reinforcement is based on clause 6.4 of EN 1992-1-1: 2004 / A1:2014 + National Annexes.

The verification reveals either that the load-bearing capacity of the reinforced concrete is sufficiently high, or that punching shear reinforcement must be designed and installed. If the verification limits are exceeded, the verification result is marked as not permissible. In this case, you must change the model parameters or select a suitable design alternative.

The verification of punching failure at the ultimate limit state can be resumed as follows:

- Check of the shear resistance at the face of the column noted u₀, and at the basic control perimeter named u₁.
- If shear reinforcement is required, a further perimeter u_{out,ef} should be found where shear reinforcement is no longer required.

Those control perimeters are shown in the following pictures:



Load distribution and basic control perimeter

⇒ Basic control perimeter u1

The basic control perimeter u₁ is taken at a distance 2d from the loaded area, where d is the effective depth.



In case the loaded area is close to an edge or a corner:



In case there is openings near the loaded area, they are dealt with according to clause 6.4.2(3).

If the shortest distance between the perimeter of the loaded area and the edge of the opening does not exceed 6d (see figure), part of the control perimeter contained between two tangents drawn to the outline of the opening from the center of the loaded area is ineffective.



In SCIA Engineer, openings on 2D members are automatically considered according to the previous criteria.

⇒ Effective depth d_{eff}

The effective depth of the slab is assumed constant and is calculated acc. to formula 6.32 of EN1992-1-1:

$$d_{eff} = \frac{(d_y + d_z)}{2}$$

where d_y and d_z are the effective depths of the reinforcement in two orthogonal directions.

Punching shear calculation

The punching shear calculation is done according to EN1992-1-1 art.6.4.3.

First the design shear resistances along the control sections are calculated:

- v_{Rd,c} design value of the shear resistance of a slab *without* punching shear reinforcement along the control section considered
- v_{Rd,cs} design value of the punching shear resistance of a slab *with* punching shear reinforcement along the control section considered
- v_{Rd,max} design value of the *maximum* punching shear resistance along the control section considered

Then the following checks should be performed.

⇒ Check at the column perimeter u₀

At the column perimeter u_{o} , or at the perimeter of the loaded area, the maximum punching shear stress should not be exceeded.

$$N_{Ed0} \le V_{Rd,max}$$

With :

• v_{Ed0} design shear stress at the column perimeter u_0

- V_{Rd,max} = 0.5 * v * f_{cd}
- v = 0.6*(1 fck/250)

⇒ Check at the basic perimeter u₁

At the basic control perimeter u1:

- If $v_{Ed} \le v_{Rd,c}$ Punching reinforcement is not needed
- If $v_{Ed} > v_{Rd,c}$ Punching reinforcement is needed

The punching shear resistance of a plate V_{Rd,c} is calculated according to formula 6.47, EN1992-1-1:

$$v_{Rd,c} = C_{Rd,c} \cdot k \cdot (100 \cdot \rho_l \cdot f_{ck})^{1/3} + k_1 \sigma_{cp} \ge (v_{min} + k_1 \sigma_{cp})$$

With:

- ρ₁ average reinforcement ratio in specific distance around column
- f_{ck} characteristic concrete compressive strength in MPa
- $v_{min} = 0.035 \cdot k^{3/2} \cdot f_{ck}^{1/2}$

•
$$C_{\text{Rd,c}} = \frac{0.18}{\gamma_c}$$

•
$$k = 1 + \sqrt{\frac{200}{d}} \le 2,0$$

• d in mm

The maximum shear stress v_{Ed} is calculated for considered control perimeter u_i according to clause 6.4.3(1) as follows:

$$v_{Ed} = \beta . \frac{V_{Ed}}{u_i d}$$

The β -factor is to consider the non-uniform load transfer (due to unbalanced bending moment). If the load transfer is non-uniform, local peak loading should be compensated by help of this β -factor.

In case that lateral stability of the structure does not depend on frame action between the slabs and the columns, and where the adjacent spans do not differ in length by more than 25%, approximate values for β may be used according to clause 6.4.3(6).

In SCIA Engineer, you must decide whether these approximate values can be used, because the program cannot check the preconditions described above.

By default, the recommended approximated values are:



Those values might be different according to the National Annexes and can be viewed in the National Annexes setup:

Type of members	Standard EN	Ø General		,
1D 🔽	E- Concrete	Punching		
2D 🔽	Concrete	 National annex 		
Type of values	- Non-prestressed reinforcement	▲ C _{Rd,o}		_
NA building	Durability and concrete cover	Value [-	j 0.18	
Type of functionality		k ₁ - factor considering effects of axial load	1	
Hollow core beams 🗹	General	Value [-] 0.10	
Prestressing 🔽	E-SLS	v _{min} - min. value of shear resistance		
	General	Formul	Formula	
	Prestressing Allowable stress	VRd,max - design value of max. shear resist	2	
	Stress limitation during tensioning	Formul	a Formula	
		⁴ β _{int} - coeff. to increase shear stress around	(
	- Common detailing provisions	Value [] 1.15	
	Columns	⁴ β _{edge} - coeff. to increase shear stress arou	r	
	- 2D structures and slabs	Value [] 1.40	
	Punching	⁴ β _{cor} - coeff. to increase shear stress aroun	d	
		Value [1.50	
		* k _{max} - factor limiting shear capacity of ap	P	
		Value [] 1.50	
		* k _{out} - factor defining placement of last per	1	
		Value [] 1.50	
		SLS		
		Allowable stress		
		Detailing provisions		
	< >			
Calact all Upralact all	Distant			A

Otherwise, as described in art 6.4.3, the β -factor can be calculated by the following general formula:

$$\beta = 1 + \sqrt{\left(k_y.\frac{M_{Ed,y}}{V_{Ed}}.\frac{u_1}{W_{1y}}\right)^2 + \left(k_z.\frac{M_{Ed,z}}{V_{Ed}}.\frac{u_1}{W_{1z}}\right)^2}$$

Calculation of β -factor with general formula can be set in Concrete setup > Punching:

Concret	e se	ettin	gs										- 0		×
Views:	Co	mpl	lete setup 💌 View settings 💌 Load defa	ult	ł	Find						Nation	al annex:	0	
De	scri	iptic	on	Symbol		Value	Default		Unit	Chapter	Code	Structu	CheckT		
<all></all>			Q	<all></all>	ρ	<all></all>	<all></all>	2	<	<all> 🔎</all>	<all></all>	<all> \wp</all>	<all></all>		
V	De	aigi	000					-						1	
⊳	Co	onve	ersion to rebars												
⊳	Int	tera	action diagram												
⊳	Sh	ear													
⊳	То	rsio	on												
	Pu	Inch	ning												
		Sh	ear stress calculation		_										
		►	Type of Beta factor	Type <mark>β</mark>	Г	Approximat 🔺	Approxi	m		6.4.3(3-6)	EN 1992-1-1	Plate	Solver se.		
			Reduction of shear stress by soil pressure			Approximate				6.4.4(2)	EN 1992-1-1	Plate	Solver se.		>>
		Co	ontrol perimeter			Formula (DIN E	N)								
			Distance of control perimeter for ceiling plate	coeff ku1.e	eil	2.00	2.00		-	6.4.2(1)	EN 1992-1-1	Plate	Solver se.		
			Distance of control perimeter for foundation plate	coeff ku1.f	oun	2.00	1.00		-	6.4.2(1)	EN 1992-1-1	Plate	Solver se.		
			Distance from column face to consider openings	coeff k _{ope}	n	6.00	6.00		-	6.4.2(3)	EN 1992-1-1	Plate	Solver se.		
			Distance from column face to collect input data abo	coeff k _{rein}	f	3.00	3.00		-	6.4.4(1)	EN 1992-1-1	Plate	Solver se.		
⊳	Ste	ress	limitations											11	
⊳	Cra	ack	ing forces					_							
⊳	Cra	ack	width												
Þ	De	flec	tions												
Þ	De	tail	ling provisions					_						11	
			-				1					0	к	Can	cel

⇒ Design of punching reinforcement if required

In case that v_{Ed} > $v_{Rd,c}$, punching reinforcement should be designed.

If punching reinforcement is required, the outer control perimeter u_{out} beyond which the reinforcement is no longer needed is calculated acc. to clause 6.4.5(4):

$$u_{out,ef} = \frac{\beta. V_{Ed}}{v_{Rd,c}. d}$$

Calculation of the required punching reinforcement

In SCIA Engineer, the shear reinforcement is designed using the following assumptions:

- the distribution of the shear links is considered as radial only
- only vertical shear links are supported
- the shape of reinforcement perimeters around the column is the same as for the shape of the basic control perimeter

The required area A_{sw,req} of one perimeter of shear reinforcement around the column assumed as radially distributed vertical shear links is calculated as:

$$A_{sw,req} = \frac{(v_{Ed,u1} - 0.75 \cdot v_{Rd,c}) \cdot u_1 \cdot s_r}{1.5 \cdot f_{vwd,ef}}$$

fywd,ef is the effective design strength of the punching reinforcement according to formula:

 $f_{ywd,ef} = 200 + 0.25 \, \cdot \, d_{eff} \leq f_{ywd}$

Detailing provisions for the punching reinforcement

The required area might be adjusted to fulfil detailing provision rules acc. to clause 9.4.3(1), so that number of shear links n_s per each reinforcement perimeter is:

$$n_{s} = \max \{ \frac{4 \cdot A_{sw,req}}{\pi \cdot d_{s}^{2}}; \frac{u_{1,last}}{s_{t,max,u1}}; \frac{u_{s,last}}{s_{t,max,out}} \}$$

- d_s diameter of shear link
- $\frac{u_{1,last}}{s_{t,max,u1}}$ condition of maximum allowed tangential spacing of links of reinforcement perimeters placed within the basic control perimeter (u_{1,last} is length of last perimeter of shear reinforcement there)
- $\frac{u_{s,last}}{s_{t,max,out}}$ condition of maximum allowed tangential spacing of links of reinforcement perimeters placed outside the basic control perimeter (u_{s,last} is length of last perimeter of shear reinforcement there)



In SCIA Engineer, limitation of spacing $s_{t,max,u1}$ and $s_{t,max,out}$ are set in Concrete setup > Detailing provisions > Punching:

vs:	Cor	nplete setup 👻 View settings 💌 Load defa	ult I	Find					Nationa	al annex: 🔣	2
De	scri	ption	Symbol	Value	Default	Unit	Chapter	Code	Structu	CheckT	Ľ.
l >		Q	<all> ₽</all>	<all> \wp</all>	<all> 🔎</all>	<	<all> \wp</all>	<all> \wp</all>	<all> 🔎</all>	<all> 🔎</all>	
Þ	Cra	ackwidth									
⊳	De	flections									
4	De	tailing provisions		8							
	⊳	Beam / Rib		-							
	⊳	Beam slab		-							
	₽	Column		-							
	⊳	Plate, Shell(Plate)		-							
	Þ	Wall, Shell(Wall)									п.
	⊳	Deep beam		-							
	4	Punching									
		Check min. shear reinforcement		~	Image: A start and a start		9.4.3(2)	EN 1992-1-1	Plate	Solver se	
		Check distance of the first perimeter of shear links		~	Z		9.4.3(1,4)	EN 1992-1-1	Plate	Solver se	
		Min. distance from column face	coeff s _{0,min}	0.30	0.30		9.4.3(1)	EN 1992-1-1	Plate	Solver se	
		Max. distance from column face	coeff s _{0,max}	0.50	0.50		9.4.3(4)	EN 1992-1-1	Plate	Solver se	
		Check max. radial spacing of shear links		~	Z		9.4.3(1)	EN 1992-1-1	Plate	Solver se	
		Max. spacing of shear links	coeff s _{r,max}	0.75	0.75	-	9.4.3(1)	EN 1992-1-1	Plate	Solver se	
		Check max. tangential spacing of shear links					9.4.3(1)	EN 1992-1-1	Plate	Solver se	
		Max. tangential spacing within the first control peri	coeff s _{t,max,u}	1.50	1.50	-	9.4.3(1)	EN 1992-1-1	Plate	Solver se	
		Max. tangential spacing outside the first control per	coeff s _{t,max,o}	2.00	2.00	-	9.4.3(1)	EN 1992-1-1	Plate	Solver se	
		Check minimum number of perimeters of shear links					9.4.3(1)	EN 1992-1-1	Plate	Solver se	
		Min. number of perimeters of shear links	Dear min	2	2		9.4.3(1)	EN 1992-1-1	Plate	Solver se	

The last condition, which must be fulfilled acc. to clause 9.4.3(2) is minimum reinforcement area of single shear link A_{sw1,min} according to formula (9.11):

$$A_{sw1,min} = \frac{0.08 \cdot \sqrt{f_{ck} / f_{ywk}} \cdot s_r \cdot s_t}{1.5}$$

With:

- sr spacing of shear links in the radial direction
- st spacing of shear links in the tangential direction

The final designed area of each perimeter of shear reinforcement around the column is:

$$A_{sw} = \frac{n_s * \pi * d_s^2}{4} \ge n_s * A_{sw1,min}$$

The required number of shear reinforcement perimeters around columns, n_{per} , is determined based on clause 6.4.5(4), which specifies that the outermost perimeter of shear reinforcement, $a_{s, last} = s_0 + s_r * n_{per}$, should be placed at a distance not greater than $k_{out} * d_{eff}$ within u_{out} . The following formula for n_{per} is derived:

$$n_{per} = \left[\frac{a_{out} - s_0 - k_{out} * d_{eff}}{s_r} + 1\right] \ge n_{per,min}$$

With:

- K_{out} Coefficient to determine the maximum distance of last perimeter from u_{out}. Default value is 1,5. This is a National Annexes parameter.
- N_{per,min} Minimum number of reinforcement perimeters around column required acc. to clause 9.4.3(1). Default value is 2 in Concrete settings > Complete setup view > Detailing provisions > Punching.
- A_{out} Distance of the outer perimeter u_{out}.

The total amount of shear reinforcement Asw,tot around the column is then calculated as :

 $A_{sw,tot} = n_{per} * A_{sw}$

2.5.2 Punching design

Configuration

The punching check in SCIA Engineer is only available when a real column or a nodal support have been connected to a plate. No punching check can be performed for a point load or a little surface load applied to the plate.

SCIA Engineer supports circular and rectangular cross sections only for the punching check.

The column position with regard to the edges of the plate and the openings is recognize. Also, for the punching check, all edges and angles of the plate are taken as straight... so if they are not in your model, the program makes an approximation.

SCIA Engineer doesn't support all punching cases of column-plate connection. The list of all current limitations can be found in our webhelp. Each unsupported configuration is mentioned in the list of Errors/warning/notes of the report in the punching check report.

Summary													
Name	Case	Punching case	Punching shape	UCvrd, [-]	max	UCvRd,c [-]	re	Shear inforcemen perimeters	t UCvr	td,cs]	UCAsw,det [-]	UC [-] Check	Errors, warnings, notes
N61	ULS/1	N/A	N/A	3	3.00	3.00	N//	4	-		-	3.00 NOT OK	W6/131
N63	ULS/1	N/A	N/A	3	3.00	3.00	N//	A	-		-	3.00 NOT OK	W6/124
Concrete	e												
Name	Case	Punching case	Punching shape	V _{Ed} [kN] β [-]	Mea [kN Mea [kN	_{d,y} Pla m] l _{d,z} [m m]	nte 1 m]	Material f _{cd} [MPa]	d _{eff} [mm] βι [%]	U0 [m] U1 [m]	VEd,u0] [MPa] VEd,u1] [MPa]	V _{Rd,max} [MPa] V _{Rd,c} [MPa]	UC _{vRd,max} [-] UC _{vRd,c} [-]
N61	ULS/1	N/A	N/A	-	-	N/A		N/A	-	-	-	-	3.00
N63	ULS/1	N/A	N/A	-	-	N/A -		N/A -	-	-	-	-	3.00 3.00 3.00
E/W/N W6/131 W6/124	Prese N61 N63	ent on memb	ers										
E/W/N W6/131	Node o	cannot be calc n has not supp	Descriptio culated for pur ported type of	n nching. T f cross-se	The co ection	onnected			S	Solut	ion		
W6/124	Node of column	cannot be calc n goes throug	ulated for pu h the plate.	nching. T	The co	onnected	Sp ab	olit the colum ove and belo	n in the r w the pl	node ate.	to get a sep	arate colur	nn

Ghoice of reinforcement

The punching design will check if the longitudinal reinforcement As in the plate is sufficient to resist to the shear force around a column-plate or nodal support-plate connection.

In SCIA Engineer you can choose between 3 types of reinforcement for the punching check/design:

- As, required calculated by the software for a specific load combination
- As,provided user set in Reinforcement design > Design defaults
- As,user practical reinforcement inputted by user manually in 2D Reinforcement

The choice between As, required, As, provided or As, user is done in the Properties panel for Punching design:





Punching check

Studied example: punching.esa

Geometry

Concrete class C30/37 Reinforcement class B500B Plate thickness 200 mm Column cross-section 10 x R 300x300 mm² and 6 x circular C400 mm² Plate and columns are connected to each other by means of the action Connect members/nodes.

Loads

*Load cases

- SW: Self weight
- DL: Dead Load = Surface load -1 kN/m² + Line force on edges -1 kN/m
- LL: Live Load = Surface load -1 kN/m²
- LL1: Additional case for further study= -25 kN/m², to be explained later

*Combinations

- ULS (Type EN ULS (STR/GEO Set B)) = SW, DL, LL
- SLS (Type EN SLS Quasi Permanent) = SW, DL, LL



Work method

The Punching Design command can be selected in the main menu "Design":



The command is available, when EC – EN national code is selected in Project data and the linear or non-linear static analysis is done for the model containing 2D members from concrete material. Once the command is selected, appropriate parameters are listed and can be adjusted in property window with following options.

RESUL	TS (1)	F
Name	Pons ontwerp	
SELECTION		
Type of selection	All \sim	
Filter	No \vee	
RESULT CASE		
Type of load	Combinations \checkmark	
Combination	ULS \lor	
REINFORCEMENT		
Type of reinforcement	Required \lor	
LIMIT STATE CONDITION		
Design ULS		
Averaging of peak		
Location	In nodes avg. \lor	
System	LCS mesh element \vee	
EXTREME		
Extreme	Node 💛	
Values	$\rm uc \sim$	
OUTPUT SETTINGS		
Output	Brief \sim	
Print explanation of symbols	Ø	
Print combination key		
F ERRORS, WARNINGS AND NOTES	ETTINGS	
how Information about warning	\square	
Show errors	None 🗸	
Show warnings	None 🖂	
Show notes	None 🗸	
ACTIONS >>>		
Refresh		
New combination from Combin	ation key	

Set the type of Selection to ALL, the Type of load to Combination ULS and the type of Reinforcement to Required then click "Refresh"

You will notice that the UC for every node will be displayed along with the control parameter in colour. In total there are 3 colours (Green, blue and red).

- Green: Shear capacity <u>without</u> reinforcement is sufficient ($UC_{vRd,c} \le 1.0$ and $UC_{vRd,max} \le 1.0$)
- Blue: Shear capacity with shear reinforcement is sufficient ($UC_{vRd,c} > 1.0$ but $UC_{vRd,cs} \le 1.0$)
- Red: Plate is not designable by application of reinforcement or maximum shear capacity of concrete adjacent to the column is not sufficient (UCvRd,cs > 1.0 or UCvRd,max > 1.0)



Presentation of results as a numerical output is possible via Preview and / or Table results. For the Punching Design, there is available two types of output:

• Brief - contains just a summary table with basic results

Punch Linear ca Combinat Extreme: Selection: Summar	lculation ion: ULS Node All Y	lesign							
Name	Case	Punching	Punching	UC _{vRd,max}	UC _{vRd,c}	Shear	UC vRd,cs	UC _{Asw,det}	UC
		case	shape	[-]	[-]	reinforcement	[-1]	[-]	[-]
						perimeters			Check
N15	ULS/1	Corner	Rectangle	0.82	0.96	not required	-	-	0.96
		column	(300;300)						OK
N20	ULS/1	Corner	Rectangle	0.86	1.01	3x 9Ø8(radial)	0.68	1.00	1.00
		column	(300;300)			80+2x80=240			OK, BUT
N53	ULS/1	Internal	Circle (400)	0.37	1.07	3x 12Ø8(radial)	0.72	1.00	1.00
		column				80+2x80=240			OK, BUT
N55	ULS/1	Internal	Circle (400)	0.12	0.37	not required	-	-	0.37
		column							OK
N57	ULS/1	Internal	Circle (400)	0.37	1.07	3x 12Ø8(radial)	0.72	1.00	1.00
		column				80+2x80=240			OK, BUT
N59	ULS/1	Internal	Circle (400)	0.36	1.06	3x 12Ø8(radial)	0.71	1.00	1.00
		column				80+2x80=240			OK, BUT
N61	ULS/1	Internal	Circle (400)	0.17	0.52	not required	-	-	0.52
		column							OK
N63	ULS/1	Internal	Circle (400)	0.37	1.08	3x 12Ø8(radial)	0.72	1.00	1.00
		column				80+2x80=240			OK, BUT
N88	ULS/1	Edge column	Rectangle	0.43	0.98	not required	-	-	0.98
			(300;300)						OK
N90	ULS/1	Edge column	Rectangle	0.43	0.98	not required	-	-	0.98
			(300;300)						OK
N95	ULS/1	Corner	Rectangle	0.21	0.44	not required	-	-	0.44
		column	(300;300)						OK, BUT
NIOT	1100/4	Index ashima	D	0 40	0.07	the set of a section of			0.07

Standard - contains the same summary table as in Brief output supplemented by additional tables
providing further semi-results

⇒ Shear capacity without reinforcement is sufficient

Select Node N61 and change the type of selection to current.

A brief output will show:

Punch Linear ca Combinat Extreme: Selection: Summar	lculation ion: ULS Node N61 y	design							
Name	Case	Punching case	Punching shape	UC _{vRd,max} [-]	UC _{vRd,c} [-]	Shear reinforcement perimeters	UC _{vRd,cs} [-]	UC Asw,det [-]	UC [-] Check
N61	ULS/1	Internal column	Circle (400)	0.17	0.52	not required	-	-	0.52 OK
Name ULS/1	1.35*S	Combination W + 1.35*DL	key + 1.50*LL						

We can see that the UC < 1, let's look at the standard output for this node:

Puncl Linear ca Combinat Extreme: Selection: Summar	hing d alculation tion: ULS Node No1 Y	lesign										
Name	Case	Punching case	Punching shape	UC _{vRd,ma} [-]	× UC _{vR}	i,c rei	Shear nforcement erimeters	UC _{vRd,cs} [-]	UC _{As}	w,det [] [Ch	UC [-] heck	
N61	ULS/1	Internal column	Circle (400)	0.1	.7 0.	52 not	required	-	-	0.5 OK	2	
Concret	e											
Name	Case	Punching case β [-]	Punching shape	V _{Ed} [kN] ΔV _{Ed} [kN]	M _{Ed,y} [kNm] M _{Ed,z} [kNm]	Plate h [mm]	Material f _{cd} [MPa]	d _{eff} [mm] ρι [%]	u₀ [m] u₁ [m]	V _{Ed,u0} [MPa] V _{Ed,u1} [MPa]	V _{Rd,max} [MPa] V _{Rd,c} [MPa]	UC vRd,max [-] UC vRd,c [-]
N61	ULS/1	Internal column 1.15	Circle (400)	128.46 0.00	0.09 13.98	Ceiling 200.00	C30/37 20.00	160.00 0.17	1.257 3.267	0.73 0.28	4.22 0.55	0.17 0.52

We can see that $V_{Ed,u1} = 0.28MPa < V_{Rd,c} = 0.55MPa$ so the shear capacity without reinforcement is sufficient. The control parameter is displayed in Green colour.

⇒ Shear capacity with reinforcement is sufficient

Let us look now at the standard output for node N59:

Punch Linear cal Combinati Extreme: Selection: Summary	culation ion: ULS Node N59 y	lesign														
Name	Case	Punching case	Punct shaj	ning pe	UC _{vRd,ma} [-]	× UC _{VR} [-]	td,c	reinf	Shear forcer rimete	ment ers	UC,	vRd,cs -]	UC As	w,det -]	UC [-] Check	
N59	ULS/1	Internal column	Circle (400)	0.3	6 1	.06	3x 12) 80+2>	Ø8(rac <80=2	dial) 40		0.71		1.00 1	L.00 DK, BUT	
Concrete	2															
Name	Case	Punching case β [-]	Punch shap	ing De	V _{Ed} [kN] ΔV _{Ed} [kN]	M _{Ed,y} [kNm] M _{Ed,z} [kNm]	P [1	late h nm]	Mate fa [MI	erial ∞d Pa]	de [mn ρι [%	ff n] 	u₀ [m] u₁ [m]	V _{Ed,u0} [MPa V _{Ed,u1} [MPa) V _{Rd,max}] [MPa] . V _{Rd,c}] [MPa]	UC _{vRd,max} [-] UC _{vRd,c} [-]
N59	ULS/1	Internal column 1.15	Circle ('	400)	265.21 0.00	26.85 6.10	Cei 20	iling 0.00	C30/3 20.00	37	160 0).00).37	1.257 3.267	1.5 0.5	52 4.22 58 0.55	0.36 1.06
Reinforc	ement															
Name	Case	Shear reinforcen perimete	nent rs	U _{out} [m] a _{out} mm]	St,u1 [mm] St,out [mm]	Contro (distan	l pei ce/c	rimete capaci	ers ty)	Mater fywd, [MPa	ial _{ef} a]	A _{sw,} [mn A _{sw1} [mn	.req n ²] .,min n ²]	A _{sw} [mm ²] A _{sw,tot} [mm ²]	V _{Rd,cs} [MPa] kmaxVRd,c [MPa]	UC _{vRd,cs} [-] UCAsw,det [-]
N59	ULS/1	3x 12Ø8(rad 80+2x80=24	ial) 0	3.472 354	230 230	320/71%				B 500B 290.0			103 11	603 1810	1.42 0.82	0.71 1.00

We can see here that $V_{Ed,u1} = 0.58$ MPa < $V_{Rd,c} = 0.55$ MPa and the UC_{vRd,c} = 1.06 > 1.

So shear reinforcement needs to be designed. The final value is $A_{sw,tot} = 1810 mm^2$ which take into account detailing provisions.

The control parameter is displayed in blue colour.

You can also show the Asw,tot graphically:



⇒ Use of provided reinforcement

Let's add some provided reinforcement to the plate.

In the Concrete settings, go to the Design Defaults view:

Concrete settings														×
Views: Design defaults View settings	l defau	ult	Fi	nd							Natio	nal a	nnex: 🏹	
Description		Symbol		Value		Default	Unit	Chapter		Code	Struc	ture	CheckTy	
<all></all>	P	<all></all>	ρ	<all></all>	ρ	<all></all>	<p< td=""><td><all></all></td><td>ρ</td><td><all></all></td><td><all></all></td><td>ρ</td><td>Design ($imes$</td><td></td></p<>	<all></all>	ρ	<all></all>	<all></all>	ρ	Design ($ imes$	
 Design defaults 														
 Reinforcement 														
Beam / Rib														
Beam slab														
▷ Column														
▷ Plate														
> Wall / Deep beam														
Minimum cover														
														>>

Activate the provided template for the plates:

Description	Symbol	Value	Default	Unit	
	Q <all> Q</all>	<all></all>	<all> D</all>		
Design defaults			1	-	Plate_Basic_Lower
A Reinforcement				-	Plate_Basic_Both
Beam / Rib				1	Plate_Basic_Add_L
▶ Beam slab				-	Plate_Basic_Add
▶ Column				1	Plate_Basic_AddLi
⊿ Plate					Name Plate Bar
▲ Longitudinal					Description Only basi
Design of provided reinforcement			2		Markarkar Blate Shi
Design template of provided reinforcement		Plate_Basic_Bev	Plate_Ba		Member typi Prate, Site
▲ Upper (z+)					
Type of cover	Type _{o+}	Auto	Auto		Mode Standard (+)
Diameter of first layer	d _{s1+}	10	10	mm	
Angle of first layer direction	a.1+	0.00	0.00	deg	
Diameter of second layer	d ₅₂₊	10	10	mm	
Angle of second layer direction	α ₂₊	90.00	90.00	deg	
 Lower (z-) 					
Type of cover	Type _{o-}	Auto	Auto		
Diameter of first layer	d _{s1-}	10	10	mm	× OO
Angle of first laver direction		0.00.	0.00	don	

Here you can choose between the different templates.

You can give a basic provided reinforcement without any additional reinforcement or allow SCIA Engineer to calculate additional reinforcement when needed.

For this example, we will define the basic reinforcement without additional reinforcement and we will use diameter 16 mm with a spacing of 150 mm.



Now look at the standard output for node N59. With the required reinforced we needed additional shear reinforcement but with the provided reinforcement set above no need for shear reinforcement:

Punch Linear cal Combinati Extreme: Selection: Summar	ing c Iculation ion: ULS Node N59 y	lesign										
Name	Case	Punching case	Punching shape	UC _{vRd,ma} [-]	x UC _{vR}	i,c rein pe	Shear forcement rimeters	UC _{vRd,cs} [-]	UC _A ₅ [-	_{w,det} (] [Ch	JC -] eck	
N59	ULS/1	Internal column	Circle (400)	0.3	6 0.	82 not re	equired	-	-	0.8 OK	2	
Concrete	2											
Name	Case	Punching case β [-]	Punching shape	V _{Ed} [kN] ΔV _{Ed} [kN]	M _{Ed,y} [kNm] M _{Ed,z} [kNm]	Plate h [mm]	Material f₀d [MPa]	d _{eff} [mm] ρι [%]	uo [m] u1 [m]	V _{Ed,u0} [MPa] V _{Ed,u1} [MPa]	V _{Rd,max} [MPa] V _{Rd,c} [MPa]	UC _{vRd,max} [-] UC _{vRd,c} [-]
N59	ULS/1	Internal column 1.15	Circle (400)	265.21 0.00	26.85 6.10	Ceiling 200.00	C30/37 20.00	160.00 0.84	1.257 3.267	1.52 0.58	4.22 0.71	0.36 0.82

We can see that $V_{Ed,u1} = 0.58$ MPa < $V_{Rd,c} = 0.71$ MPa so the shear capacity without reinforcement is sufficient. The control parameter is now displayed in Green colour instead of blue.

⇒ Unity check is not ok: control perimeter is red

Change the "Type of Result" to Load Case LL1 and display the result for node N59:



Control perimeter is now displayed in red and the UC = 1,44 > 1.



Take a look at the Standard Output:

We can also show the errors and warning in the output by checking this option in the properties panel:



2.6 Code dependent deflection (CDD)

2.6.1 Intro

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

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

To be able to use this method in SCIA Engineer, the following settings should be set beforehand: 1. Use the post processing environment '**default**' in the Project settings:

	DATA			MATERIAL		
	DATA			Concrete	~	
	Name:	-		Material	C30/37	v
11	Part:	-		Reinforcement mate	B 500A	¥
	Description:	-		Steel		
21	bescription	-		Masonry		
1	Author:	User		Aluminium		
	Date:	30. 08. 2021		Steel fibre concrete		
				- Other		
	Structure:	🜓 Frame XZ	*	CODE		
	Destarossing			National Code:		
/ III	environment	🤞 default		EC - EN		•
Li cata	Model:	関 One	~	National annex:		
		Edbituor	cion into	Standard EN		¥

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



2.6.2 Types of combination for CDD

The combinations used for the CDD calculation can either be automatically generated or inserted by the user.

Automatic Creation of combinations for CDD

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

- Combination for calculation of total deflection
 - Generated directly from the user choice of combination in the CDD check, properties panel:

	RESU	LTS (1)
	Name	Code dependent deflection
ECTION		
Type of	selection	All \sim
	Filter	No \checkmark
Automatic com	bination	
ULT CASE FOR DE	FLECTION	
Тур	e of load	Combinations \vee
Com	bination	SLS-Char (auto) 🗸
Envelope (for 2D	drawing)	Absolute extreme 🗸
Type of reinfo	orcement	User 🗸

 Combination for calculation of immediate deflection Uses the generated combination for total deflection and removes variable load cases with duration type Medium, Short or Instantaneous.

Duration type is defined in the Load cases properties:

Load cases			×
et -: 🖸 🕩 🖬 ۹	🖌 🐟 🗖 🕞 🖸 All		• T
LC1 - SW	Name	LC3	
LC2 - per	Description	var	
LC3 - var	Action type	Variable	¥
	Load group	LG2	×
	Load type	Static	*
	Specification	Standard	*
	Duration	Short	^
	Master load case 3D Wind	Long Medium	
		Short Instantaneous	

 Combination for calculation of deflection due to creep Uses the generated combination for total deflection and multiplies variable load cases by a coefficient defined in Concrete settings > Deflections:

ws: Complete setup View sett • Load de	fault	Find					Nationa	al annex:
Description	Symbol	Value	Default	Unit	Chapter	Code	Struct	Check
all>	<all> 🔎</all>	<all> D</all>	<all> D</all>		<all> ρ</all>	<all> \wp</all>	<all> D</all>	<all> \wp</all>
Design defaults								
Reinforcement								
Minimum cover								
Solver setting								
▶ General								
Internal forces								
Design As								
Conversion to rebars								
Interaction diagram								
▷ Shear								
Torsion								
Stress limitations								
Cracking forces								
▷ Crack width								
 Deflections 								
Coefficient for increasing the amount of reinforc	Coeffreinf	1.0	1.0			Independ	All (Be	Solver s
Maximal total deflection L/x; x =	x _{tot}	250.0	250.0		7.4.1(4)	EN 1992-1-1	1D (Be	Solver s
Maximal additional deflection L/x; x =	Xadd	500.0	500.0		7.4.1(5)	EN 1992-1-1	1D (Be	Solver s
Type of variable load coefficient for the automati		Use Psi2 f 🔺	U e Psi2			Independ	All (Be	Solver s
Detailing provisions	100	Use Psi2 fact	or					

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

Manual input of combinations for CDD

It is possible for you to introduce your own combinations for calculation of immediate deflection and deflection due to creep.

In order to introduce these manual combinations, the option "Automatic combination" must be unchecked in the CDD check, properties panel.

Two new sections ("Result case: Creep deflection" and "Result case: Immediate deflection") appear in the properties window where you can choose the combinations for creep and immediate deflections.

These combinations have to be linear combinations, it means that creep and immediate deflection will be the same for all sub-combinations generated from combination for total deflection.



The combination for the calculation of the total deflection remains generated directly from your choice of combination in the CDD check in the properties panel.

2.6.3 Type of reinforcement

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

RESU	ILTS (1)	\bowtie
Name	Code dependent deflection	
▼ SELECTION		
Type of selection	All \checkmark	
Filter	No \vee	
Automatic combination		
▼ RESULT CASE FOR DEFLECTION	1	
Type of load	Combinations \lor	
Combination	SLS-Char (auto) 🗸	
Envelope (for 2D drawing)	Absolute extreme \vee	
Type of reinforcement	User	\checkmark
▼ EXTREME 1D	Required	
Extreme 1D	Provided	
Results in sections	All V	
Direction (local)	z (1D/2D) 🗸	-
Values	$\rm uc \sim$	
Output	Brief 🗸	
Print combination key		_
Print explanation of symbols	0	

2.6.4 Calculation of stiffness for 1D elements

Members which are not expected to be loaded above the level which would cause the tensile strength of the concrete to be exceeded anywhere within the member should be considered to be uncracked. Members which are expected to crack, but may not be fully cracked, will behave in a manner intermediate between the uncracked and fully cracked conditions. New stiffness (stiffness with taking into account cracking) is calculated in center of each 1D element.

Two types of stiffness are calculated:

- Short-term stiffness is calculated using 28 days modulus of elasticity E_c = E_{cm}, it follows that value of stiffness is loaded directly from properties of the concrete material
- Long-term stiffness is calculated using effective E modulus based on creep coefficient for acting load, it follows E_c = E_{c,eff} = E_{cm}/(1+φ).

Calculation effective modulus of elasticity is based on equation 5.27 in EN 1992-1-1, but instead of effective creep coefficient ϕ_{ef} , only creep coefficient ϕ is used

The following procedure is used for the calculation of stiffnesses:

- 1) The transformed cross-section characteristics of uncracked section (Ai, Ii, ti...) are calculated
- 2) The stiffnesses of the uncracked cross-section ((Eiy)ı,(Eiz)ı, (EA)ı) to the center of the uncracked transformed cross-section are calculated.
- The maximum value of tensile stress of the uncracked cross-section (value σ_{ct,res}) for respective characteristic combination (N_{char,res},M_{char,res,y}, M_{char,res,z}) is calculated
- 4) The maximum value of tensile stress of uncracked cross-section (value σ_{ct,imm}) for immediate characteristic combination (N_{char,im},M_{char,im,y}, M_{char,im,z}) is calculated
- 5) Compare σ_{ct} with $\sigma_{ct,imm}$
 - If $\sigma_{ct} \ge \sigma_{ct,imm}$, the respective characteristic combination will be used for calculation, $N_{char}=N_{char,res}, M_{char,res,y}, M_{char,z}=M_{char,res,z}, \sigma_{ct}=\sigma_{ct,res}$
 - If σ_{ct}≤σ_{ct,imm}, the immediate characteristic combination will be used N_{char}=N_{char,im},M_{char,y} =M_{char,im,y}, M_{char,z}=M_{charim,z},σ_{ct}=σ_{ct,im}
- 6) Compare σ_{ct} with σ_{cr}
 - If $\sigma_{ct} \leq \sigma_{cr}$, the cross-section is uncracked:
 - o bending stiffness around y-axis $EI_y = (Eiy)_1$
 - o bending stiffness around z axis $EI_z = (Eiy)_I$
 - \circ axial stiffness EA = EA_I,
 - If $\sigma_{ct} \ge \sigma_{cr}$, the cross-section is cracked and average stiffness will be calculated.
 - 1. The transformed Css characteristics of the cracked section (Air, Iir, tir...) is calculated.
 - 2. The stiffnesses of the fully cracked cross-section ((Eiy)II, (Eiz)II, (EA)II) to center of cracked transformed cross-section is calculated
 - 3. The stress in the tensile reinforcement of the fully cracked cross-section (value σ_{sr}) for characteristic combination (N_{char}, M_{char}, M_{char}, j is calculated.
 - 4. The stress in the tensile reinforcement of the fully cracked cross-section (value σ_s)for respective combination (N,M_y,M_z) is calculated.
 - 5. The distribution coefficient ζ according equation 7.19 in EN 1992-1-1 is calculated

$$\zeta = 1 - \beta \left(\frac{\sigma_{\rm sr}}{\sigma_{\rm s}} \right)$$

where β is a coefficient taking account the influence of the duration of the loading or of repeated loading on the average strain (β =1 for calculation of short-term stiffness, β =0,5 for calculation of long-term stiffness)

- 6. The average value of the stiffnesses based on equation 7.18 in EN 1992-1-1 is calculated
 - bending stiffness around y-axis (Eiy) = $1/[\zeta/(Eiy)_{II} + (1-\zeta)/(Eiy)_{I}]$
 - bending stiffness around z-axis (Eiz) = $1/[\zeta/(Eiz)_{II} + (1-\zeta)/(Eiz)_{II}]$
 - axial stiffness (EA) = $1/[(\zeta/(EA)_{II} + (1-\zeta)/(EA)_{I}],$

Stiffness is recalculated to principal axis for unsymmetrical cross-section

7) The five types of stiffnesses are calculated for each 1D element and each dangerous combination:

Type of stiffness	Respective combination
Short-term stiffness for immediate deflection	Immediate
Short-term stiffness for short-term deflection	Total
Short-term stiffness for creep deflection	Creep
Long-term stiffness for creep deflection	Creep
Long-term stiffness for shrinkage deflection	Total

- 8) The following stiffnesses are changes in stiffness matrix for 1D elements:
 - EA_x =EA
 - $GA_y=GA_z = G \cdot EA_x/(1,2 \cdot E_c)$
 - Ely =Eiy
 - El_z =Eiz
 - $GI_x=0,5\cdot(1-\mu)\cdot(EI_y)EI_z)^{0.5}$

Where:

- G is shear modulus of the concrete calculated according to formula $G = 0.5 \times Ec/(1+m)$
- m is Poisson coefficient of the concrete loaded from material properties of the concrete

Eccentricity of stiffnesses (distance between center of gravity of concrete cross-section and center of gravity of cracked transformed cross-section) is not taken into account in current version

Calculation of curvature, strain and stiffness caused by shrinkage of a 1D element

Calculation of shrinkage forces

The forces caused by shrinkage are calculated according to formulas below. The forces are calculated for both states: uncracked and cracked cross-section.

- $N_{shr} = -\varepsilon_{cs}(t,t_s) \cdot Coef_{Reinf} \sum (E_{si} \cdot A_{si})$
- Mshr,y = Nshr•eshr,z
- $M_{shr,z} = N_{shr} \cdot e_{shr,y}$

Where :

- $e_{shr,y} = \sum (E_{si} \cdot A_{si}) / \sum (E_{si} \cdot A_{si} \cdot y_{si}) t_{iy}$
- $e_{shr,z} = \sum (E_{si} \cdot A_{si}) / \sum (E_{si} \cdot A_{si} \cdot z_{si}) t_{iz}$
- $\varepsilon_{cs}(t,t_s)$ total shrinkage strain
- Coefreinf coefficient increasing amount of reinforcement
- Esi modulus of elasticity of i-th bar of reinforcement
- Asi area of reinforcement of i-th bar of reinforcement
- y_{si} position of i-th bar of reinforcement from center of gravity of cross-section in y-direction
- z_{si} position of i-th bar of reinforcement from center of gravity of cross-section in z-direction
- t_{iy} distance between center of gravity of transformed uncracked/cracked cross-section and center of gravity of concrete cross-section in y-direction
- t_{iz} distance between center of gravity of transformed uncracked/cracked cross-section and center of gravity of concrete cross-section in z-direction

Shrinkage deflection	(long-term stiffness)
----------------------	-----------------------

	N [kN]	My [kNm]	M _z [kNm]
Combination: CO2/1_tot	0.00	159.75	0.00
Characteristic combination (char): CO2/1_tot_char	0.00	159.75	0.00

Forces caused by shrinkage: N_{shr} = 140.74 kN, M_{shr,y} = 24.56 kNm, M_{shr,z} = 0.00 kNm

Cross-section characteristics

Type of	ty	tz	Α	l,	l _z	xi	A,
component	[mm]	[mm]	[mm ²]	[mm ⁴]	[mm ⁴]	[mm]	[mm ²]
Linear	0.0	0.0	320000	9.02·10 ⁹	17.6·10 ⁹	218.0	-
Uncracked	0.0	-19.7	356135	12.1.10 ⁹	19.7·10 ⁹	232.2	1407
Cracked	0.0	73.4	175211	5.31·10 ⁹	13.7·10 ⁹	139.1	1407

Cracking forces

N _{er}	M _{y,cr}	M _{z,cr}	σ _{ct}	σ _{cr}	Cracked section	σ _s ,	σ <u>;</u>	β	ζ	E _c
[kN]	[kNm]	[kNm]	[MPa]	[MPa]		[MPa]	[MPa]	[-]	[-]	[GPa]
0.00	73.03	0.00	4.87	2.20	YES	144.6	316.4	0.5	0.896	30.0

Stiffness calculation

Axial stiffness EA: EA_I = 9600.00 MN EA_{II} = 9600.00 MN

$EA = \frac{1}{\frac{\zeta}{EA_{\parallel}} + \frac{1-\zeta}{EA_{\parallel}}} = \frac{1}{\frac{0.896}{9600.00} + \frac{1-0.896}{9600.00}} = 9600.00 \text{ MN}$	(7.18)
Bending stiffness El _y : El _{y,l} = 672.92 MN·m ² El _{y,l} = 193.17 MN·m ²	
$EI_{y} = \frac{1}{\frac{\zeta}{EI_{y,i}} + \frac{1-\zeta}{EI_{y,i}}} = \frac{1}{\frac{0.896}{193.17} + \frac{1-0.896}{672.92}} = 208.71 \text{ MN·m}^{2}$	(7.18)
Bending stiffness El _y : El _{z,l} = 527.00 MN·m ² El _{z,ll} = 527.00 MN·m ²	
$FI = \frac{1}{1} = \frac{1}{1} = 527.00 \text{ MN/m}^2$	(7 18)

Calculation of strain and curvature caused by shrinkage

Strain and curvature caused by shrinkage are calculated for each 1D element and these values are calculated for both states (uncracked and cracked cross-section).

Calculation of strain caused by shrinkage:

• $\varepsilon_x = -\varepsilon_{cs}(t,t_s) \cdot \text{Coef}_{\text{Reinf}} \cdot \sum (E_{si} \cdot A_{si}) / (E_{ceff} \cdot A_i)$

Calculation of curvature around y and z axis caused by shrinkage

- $(1/r_y) = -\varepsilon_{cs}(t,t_s) \cdot \text{Coef}_{\text{Reinf}} \cdot \sum (E_{si} \cdot A_{si} \cdot (t_{iz} z_{si})) / (E_{ceff} \cdot I_{iy})$
- $(1/r_z) = -\varepsilon_{cs}(t,t_s) \cdot \text{Coef}_{\text{Reinf}} \cdot \sum (E_{si} \cdot A_{si} \cdot (t_{iy} y_{si})) / (E_{ceff} \cdot I_{iz})$

Where:

- $\epsilon_{cs}(t,t_s)$. total shrinkage strain
- Coefreinf coefficient increasing amount of reinforcement
- Esi is modulus of elasticity of i-th bar of reinforcement
- Asi is area of reinforcement of i-th bar of reinforcement
- y_{si} position of i-th bar of reinforcement from center of gravity of cross-section in y-direction
- z_{si} position of i-th bar of reinforcement from center of gravity of cross-section in z-direction
- t_{iy} distance between the center of gravity of transformed uncracked/cracked cross-section and center of gravity of concrete cross-section in y-direction
- tiz distance between the center of gravity of transformed uncracked/cracked cross-section and center of gravity of concrete cross-section in z-direction
- E_{ceff} effective modulus of elasticity of the concrete calculated according to formula $E_c = E_{c,eff} = E_{cm}/(1+\phi)$.
- E_{cm} secant modulus of elasticity of concrete
- φ creep coefficient
- Ai transformed area of uncracked/cracked cross-section
- I_{iy} transformed second moment of area around y-axis of uncracked/cracked cross-section calculated to center of gravity of transformed uncracked/cracked cross-section
- I_{iz} transformed second moment of area around z axis of uncracked/cracked cross-section calculated to center of gravity of transformed uncracked/cracked cross-section

Calculation of stiffnesses for shrinkage

The stiffness of uncracked/cracked cross-section for shrinkage is calculated from strain and curvatures caused by shrinkage by using total level of load (total load combination)

- bending stiffness around y-axis $EI_y = M_{tot,y}/(1/r_y)$
- bending stiffness around z axis El_z = M_{tot,z}/(1/r_z)
- axial stiffness $EA = N_{tot}/\epsilon_x$

2.6.5 Calculation of stiffness for 2D elements

The following procedure is used for the calculation of stiffness of 2D elements:

1) The principal stresses of 2D element for both surfaces is calculated

$$\sigma_{1\overline{+}} = \frac{\sigma_{x\overline{+}} + \sigma_{y\overline{+}}}{2} + \frac{1}{2} \sqrt{\left(\sigma_{x\overline{+}} - \sigma_{y\overline{+}}\right)^2 + 4 \cdot \sigma_{xy,\overline{+}}}$$
$$\sigma_{2\overline{+}} = \frac{\sigma_{x\overline{+}} + \sigma_{y\overline{+}}}{2} - \frac{1}{2} \sqrt{\left(\sigma_{x\overline{+}} - \sigma_{y\overline{+}}\right)^2 + 4 \cdot \sigma_{xy,\overline{+}}}$$

2) The angle of principal stresses at both surfaces is calculated

$$\alpha_{\sigma 1 \overline{+}} = 0.5 \cdot \tan^{-1} \left(\frac{2 \cdot \sigma_{xy\overline{+}}}{\sigma_{x\overline{+}} - \sigma_{y\overline{+}}} \right)$$

3) The final value of the principal stress is calculated

 $\alpha = \alpha_{\sigma_{1+}}$ if $\sigma_{1+} \ge \sigma_{1-}$ $\alpha = \alpha_{\sigma_{1-}}$ otherwise

4) The internal forces are recalculated to the direction of the principal stresses α $m(\alpha) = m_x \cdot \cos^2(\alpha) + m_y \cdot \sin^2(\alpha) + m_{xy} \cdot \sin(2 \cdot \alpha)$

 $n(\alpha) = n_x \cdot \cos^2(\alpha) + n_y \cdot \sin^2(\alpha) + n_{xy} \cdot \sin(2 \cdot \alpha)$

where $n_x, n_y, n_{xy}, m_x, m_y, m_{xy}$ are 2D forces in center of 2D element

5) The area of reinforcement is recalculated to the direction of the principal stress α A_s(α) = A_s·cos²(α-α_s) where A_s α is area and angle of longitudinal rainforcement.

where A_{s}, α_{s} is area and angle of longitudinal reinforcement

- The non-linear stiffness in the first principal direction is calculated according to the procedure as for 1D element
 - for rectangular cross-section (b =1m, h = thickness of 2D element in center of gravity)
 - for internal forces N = n(α), M_y= m(α) and M_z=0 according procedure as for 1D element
- 7) The non-linear stiffness in the second principal direction is calculated according to the procedure as for 1D element
 - for rectangular cross-section (b =1m, h = thickness of 2D element in center of gravity)
 - for internal forces N = $n(\alpha+90)$, M_y= $m(\alpha+90)$ and M_z=0 according procedure as for 1D element
- 8) The stiffness for shrinkage deflection is calculated in both directions of principal axes as explained in the next section.
- 9) The five types of stiffnesses are calculated for each 2D element and each dangerous combination:

Type of stiffness	Respective combination	Direction of principal stress			
Short-term stiffness for immediate deflection	Immediate	First (EA ₁ ,Ely ₁ ,Elz ₁)			
	Infinediate	Second (EA ₂ ,EIy ₂ ,EIz ₂)			
Short-term stiffness for short-term deflection	Total	First (EA1,Ely1,Elz1)			
	br short-term deflection Total Second (EA2,Ely2,Elz2) First (EA4,Ely2,Elz2)				
Short term stiffness for crean deflection	Croop	First (EA ₁ ,Ely ₁ ,Elz ₁)			
Short-term sumess for creep denection	Cleep	Second (EA ₂ ,EIy ₂ ,EIz ₂)			
Long term stiffness for groop deflection	Croop	First (EA1,Ely1,Elz1)			
Long-term sumess for creep denection	Cleep	Second (EA ₂ ,EIy ₂ ,EIz ₂)			
Long torm stiffnoss for shrinkage deflection	Total	First (EA ₁ ,Ely ₁ ,Elz ₁)			
		Second (EA ₂ ,Ely ₂ ,Elz ₂)			

- 10) The following stiffnesses are changes in stiffness matrix for 2D elements:
 - D11 = Ely₁
 - D22 = Ely₂
 - $D33 = 0.5 \cdot (1 \mu) \cdot (D11 \cdot D22)^{0.5}$
 - D44 = G·h/1.2
 - D55 = G·h/1.2
 - $D12 = \mu \cdot (D11 \cdot D22)^{0.5}$
 - d11 = EA₁
 - d22 = EA₂
 - d33 = G·h
 - $d12 = \mu \cdot (d11 \cdot d22)^{0.5}$
 - Where:
 - G is shear modulus of the concrete calculated according to formula $G = 0.5 \cdot E_c/(1+\mu)$
 - µ is Poisson coefficient of the concrete loaded from material properties of the concrete

Eccentricity of stiffnesses (distance between center of gravity of concrete cross-section and center of gravity of cracked transformed cross-section) is not taken into account in current version

Calculation of curvature, strain and stiffness caused by shrinkage of a 2D element

Calculation of shrinkage forces

The forces are calculated in the center of gravity of each element and they are calculated in two directions:

- The first one is the direction of principal stress $\boldsymbol{\alpha}$
- The second one is the direction of principal stress α +90°

The forces caused by shrinkage for first/second direction are calculated according to formulas below. The forces are calculated for both states: uncracked and cracked cross-section.

- $n_{shr} = -\epsilon_{cs}(t,t_s) \cdot Coef_{Reinf} \sum (E_{si} \cdot A_{si(\alpha)})$
- Mshr = Nshr•eshr,z

Where:

- $e_{shr,z} = \sum (E_{si} \cdot A_{si(\alpha)}) / \sum (E_{si} \cdot A_{si(\alpha)} \cdot z_{si}) t_{iz(\alpha)}$
- $\epsilon_{cs}(t,t_s)$ total shrinkage strain
- Coefreinf coefficient increasing amount of reinforcement
- E_{si} is modulus of elasticity of i-th bar of reinforcement
- A_{si(α)} is area of reinforcement of i-th bar of reinforcement in first (angle α)/second direction (angle α+90°) of principal stress
- z_{si} position of i-th bar of reinforcement from center of gravity of cross-section in z-direction
- t_{iz(α)} distance between center of gravity of transformed uncracked/cracked cross-section and centre of gravity of concrete cross-section in z-direction and in first (angle α)/second direction (angle α+90°) of principal stress

Calculation of strain and curvature caused by shrinkage

Strain and curvature caused by shrinkage are calculated for each 2D elements and these values are calculated for both states (uncracked and cracked cross-section). The values are calculated in both directions of principal stresses.

Calculation of strain caused by shrinkage:

• $\epsilon_x = -\epsilon_{cs(t,ts)} \cdot \text{CoefReinf} \cdot \sum (E_{si} \cdot A_{si(\alpha)}) / (E_{ceff} \cdot A_{i(\alpha)})$

Calculation of curvature around y and z axis caused by shrinkage:

• $(1/r) = -\epsilon_{cs}(t,t_s) \cdot Coef_{Reinf} \cdot \sum (E_{si} \cdot A_{si(\alpha)} \cdot (t_{iz(\alpha)} - z_{si}))/(E_{ceff} \cdot I_{iy(\alpha)})$

Where:

- $\epsilon_{cs}(t,t_s)$ total shrinkage strain
- Coefreinf coefficient increasing amount of reinforcement
- Esi is modulus of elasticity of i-th bar of reinforcement
- A_{si(α)} is area of reinforcement of i-th bar of reinforcement in first (angle α)/second direction (angle α+90°) of principal stress
- zsi position of i-th bar of reinforcement from center of gravity of cross-section in z-direction
- t_{iz(α)} distance between centre of gravity of transformed uncracked/cracked cross-section and centre of gravity of concrete cross-section in z-direction and in first (angle α)/second direction (angle α+90°)of principal stress
- E_{ceff} effective modulus of elasticity of the concrete calculated according formula $E_c = E_{c,eff} = E_{cm}/(1+\phi)$.
- E_{cm} secant modulus of elasticity of concrete
- φ creep coefficient
- A_{i(α)} transformed area of uncracked/cracked cross-section in the first (angle α)/second direction (angle α+90°) of principal stress
- l_{iy(α)} transformed second moment of area around y axis of uncracked/cracked cross-section calculated to centre of gravity transformed uncracked/cracked cross-section in the first (angle α)/second direction (angle α+90°) of principal stress

Calculation of stiffnesses for shrinkage

The stiffness of uncracked/cracked cross-section for shrinkage is calculated from strain and curvatures caused by shrinkage by using total level of load (total load combination)

- bending stiffness in direction of first principal axis $Ely_1 = m_{tot(\alpha)}/(1/r)_1$
- bending stiffness in direction of second principal axsi $Ely_2 = m_{tot(\alpha+90)}/(1/r)_2$
- axial stiffness in direction of first principal axis $EA_1 = n_{tot(\alpha)}/\epsilon_{x,1}$
- axial stiffness in direction of second principal axis EA₂ = n_{tot(α+90)}/ε_{x,2}

Where:

- n_{tot(α)}, n_{tot(α+90} are axial forces from total combination in 2D element recalculated to direction of first and second principal axis
- m_{tot(α)}, m_{tot(α+90} are bending moments from total combination in 2D element recalculated to direction
 of first and second principal axis
- εx,1(2) is strain caused by shrinkage calculated in direction of first (second) principal axis
- (1/r)₁₍₂₎ is curvature caused by shrinkage calculated in direction of first (second) principal axis

Deflection for shrinkage is calculated in FEM analysis for total combination, therefore the stiffness are calculated with using internal forces for total combination

2.6.6 Parameters for the calculation of shrinkage strain

The total shrinkage strain is composed of two components, the drying shrinkage strain and the autogenous shrinkage strain. The drying shrinkage strain develops slowly, since it is a function of the migration of the water through the hardened concrete. The autogenous shrinkage strain develops during hardening of the concrete.

There are three options for calculation/input of total shrinkage strain that can be selected in the concrete settings menu:

- No (Consider drying and autogenous shrinkage= No): shrinkage will not be taken into account in CDD calculation
- Automatic calculation (Consider drying and autogenous shrinkage = Auto), where shrinkage strain is calculated according to EN 1992-1-1, chapter 3.1.4(6) for following input parameters:
 - o Relative humidity
 - o Age of concrete at beginning of drying shrinkage
 - o Age of concrete at moment considered

Except of these input parameters, automatic calculation of shrinkage strain depends on material properties (mean compressive strength of concrete f_{cm} , characteristic compressive cylinder strength f_{ck} , type of cement), cross-section parameters (cross-sectional area A_c and the perimeter of the member in contact with the atmosphere u)

• User input (Consider drying and autogenous shrinkage = User value) and you can input directly value of total shrinkage strain

escri	iptic	on	Symbol	Value	Default	Unit	Chapter	Code	Struct	Check
م <۱۱>				<all> \wp</all>	<all> \wp</all>		<all> \wp</all>	<all> \wp</all>	<all> D</all>	<all> 🔎</all>
esigr	n de	faults				1				
olver	set	tting								
4 Ge	ner	ral								
	Lir	mit value of unity check	Lim.check	1.0	1.0			Independe	All (Bea	Solver s
Value of unity check for not calculated unity check		Ncal.check	3.0	3.0			Independe	All (Bea	Solver s	
The coefficient for calculation effective depth of cros		Coeff _d	0.9	0.9			Independe	All (Bea	Solver s	
	Th	e coefficient for calculation inner lever arm	Coeffz	0.9	0.9			Independe	All (Bea	Solver s
	Th	e coefficient for calculation force, where member	Coeff _{com}	0.1	0.1			Independe	All (Bea	Solver s
	Cr	eep and shrinkage								
		Age of concrete at the moment considered	t	1825.00	18250.00	day	3.1.4.B.1-2	EN 1992-1-1	All (Bea	Solver s
		Relative humidity	RH	50	50	%	3.1.4.B.1-2	EN 1992-1-1	All (Bea	Solver s
		Type input of creep coefficient	Type <mark>(</mark> t,to)	Auto	Auto		3.1.4(2)	EN 1992-1-1	All (Bea	Solver s
		Age of concrete at loading	t _o	28.00	28.00	day	3.1.4(2),B1	EN 1992-1-1	All (Bea	Solver s
	Þ	Consider drying and autogenous shrinkage	Type ε _{cs} (t,ts	Auto 🔺	uto		3.1.4(6)	EN 1992-1-1	All (Bea	Solver s
		Age of concrete at the beginning of drying shrin	t _s	No	.00	day	3.1.4(6),B2	EN 1992-1-1	All (Bea	Solver s
	C1			Auto						

2.6.7 Calculation of deflection

The following deflections are calculated in the CDD check:

- δ_{lin} linear (elastic) deflection, calculated for the total combination and for linear stiffness.
- Δ_{imm} immediate deflection, the deflection after applying permanent and long-term variable loads which means calculated for short-term stiffness and immediate combination
- δ_{short} short-term deflection, the deflection which considers cracking of cross-section calculated for short-term stiffness and total combination
- δ_{creep} creep deflection, calculated as the difference between deflection calculated for long-term and short-term stiffness for the creep combination. $\Delta_{creep} = \delta_{creep, long} \delta_{creep, short}$
- δ_{shr} deflection caused by drying and autogenous shrinkage. The long-term stiffness is calculated from strain and curvature caused by shrinkage using total combination.
- δ_{add} additional deflection, the deflection after applying a variable load and considering creep calculated as the difference between total and immediate deflection. $\Delta_{add} = \delta_{tot} \delta_{imm}$
- δ_{tot} total deflection, the deflection which considers creep and cracking calculated as the sum of short-term deflection and deflection caused by creep. $\Delta_{tot} = \delta_{short} + \delta_{creep}$



All those values can be displayed on the screen:



Chapter 3: Modification of results

3.1 Location

During a calculation in SCIA Engineer, the node deformations and the reactions are calculated exactly (by means of the displacement method). The stresses and internal forces are derived from these magnitudes by means of the assumed basic functions and are therefore in the Finite Elements Method always less accurate.

The Finite Elements Mesh in SCIA Engineer exists of linear 3- and/or 4-angular elements. Per mesh element 3 or 4 results are calculated, one in each node. When asking the results on 2D members, the option 'Location' in the Properties window gives the possibility to display these results in 4 ways.

3.1.1 In nodes, no average

All of the values of the results are taken into account, there is no averaging. In each node are therefore the 4 values of the adjacent mesh elements shown. If these 4 results differ a lot from each other, it is an indication that the chosen mesh size is too large.

This display of results therefore gives a good idea of the discretisation error in the calculation model.



3.1.2 In centers

Per finite element, the mean value of the results in the nodes of that element is calculated. Since there is only 1 result per element, the display of isobands becomes a mosaic. The course over a section is a curve with a constant step per mesh element.



3.1.3 In nodes, average

The values of the results of adjacent finite elements are averaged in the common node. Because of this, the graphical display is a smooth course of isobands.

In certain cases, it is not permissible to average the values of the results in the common node:

- At the transition between 2D members (plates, walls, shells) with different local axes.

- If a result is really discontinuous, like the shear force at the place of a line support in a plate. The peaks will disappear completely by the averaging of positive and negative shear forces.





3.1.4 In nodes, average on macro

The values of the results are averaged per node *only* over mesh elements which belong to the same 2D member and which have the same directions of their local axes. This resolves the problems mentioned at the option 'In nodes, average'.





3.1.5 Accuracy of the results

If the results according to the 4 locations differ a lot, then the results are inaccurate and the mesh has to be refined. A basic rule for a good size of the mesh elements, is to take 1 to 2 times the thickness of the plate.

3.2 Averaging strip

An averaging strip averages peak values over a zone. You can find the averaging strip in the Input Panel in the "Result tools" category :

	All workstations 🗸
Res AVERAGING STRIP	All tags
r* 🕼 🖾 🖾	
RC	
RS Name RS1	
RS Name RS1 Type Strip	
RS Name RS1 Type Strip Width [m] 1.000	

Type: a point or a strip can be chosen.

Dimensions: here the dimensions of the point/strip need be set.

Direction:



Longitudinal means that the averaging is done in the longitudinal direction of the strip. In the example above this is the y-direction. This means that the averaging is done for my. The values my are averaged in the x-direction.



Perpendicular means that the averaging is done perpendicular to the longitudinal direction of the strip. In the example above this is the x-direction. This means that the averaging is done for mx. The values mx are averaged in the y-direction.



3) Direction = Both

Both means that the averaging is done in both directions of the averaging strip. This means the values are averaged for mx as well as my in the direction perpendicular to mx and my.

To activate the averaging strip, the option 'Averaging of peak' needs to be checked in the properties panel.



As an example, we will apply averaging strips to the model of the chapter "2D concrete members" for the value Asw,req.



• A_{sw,req} without Averaging of peaks

A_{sw,req} with averaging of peaks



3.3 Rib

A rib can be added to a plate in the Input Panel in the "1D Members" category:

INPUT PANEL	ullet All workstations $$
1D Membe RIB	🥔 All tags 🗸
🗲 — 🕴 🔽 🤜 🖘 🖬	

But also in the Input Panel in the "2D Members" category:

Ξ	INPUT PANEL						All workstations 🗸						
=	2D MERIBBED SLAB						🥔 All tags 🗸						
	W	40	=	-		P		4					4
			Read-model										

3.3.1 Results in ribs

When a rib is added to the model there will be an option rib available in the result properties of 1D and 2D members. This option has an influence on what results you view.


Link between the internal forces calculated for the entire T-section, and for the beam and slab separately

When calculating the internal forces in a rib, the substitute T-section is used to calculate the results. The web of this T-section is formed by the rib-beam itself, the flange of the T-section is made with the effective width of the slab. The effective width of the slab has to be used to determine the internal forces of the slab that have to be added to the internal forces calculated in the rib itself.

т	the heart of the entire substitute T-section
T1	the heart of the left part of the effective width
T2	the heart of the right part of the effective width
Т3	the heart of the original rib
L	eft part Right part +T1 +T2 +T +T +T3

The coordinates of the hearts are used as lever arms in Y and Z direction:

Lever Arm Z1 = T1z - Tz	Lever Arm Y1 = T1y - Ty	
Lever Arm Z2 = T2z - Tz	Lever Arm Y2 = T2y - Ty	
Lever Arm Z3 = T3z - Tz	Lever Arm Y3 = T3y - Ty	
Lever Arm Z = Tz - 0z	Lever Arm Y = Ty - 0y	

The final internal forces in the rib can be calculated with the formulas below:

- N = N beam + N slab, left + N slab, right
- $V_y = V_y$ beam + V_y slab, left + V_y slab, right
- $V_z = V_z$ beam + V_z slab, left + V_z slab, right
- $M_x = M_x$ beam + M_x slab, left + M_x slab, right
- M_y = M_y beam + M_y slab, left + M_y slab right + N slab, left * (Lever Arm Z₁)
 N slab, right * (Lever Arm Z₂) + N beam * Lever Arm Z₃;
- M_z = M_z beam + M_z slab, left + M_z slab, right + N slab, left * (Lever Arm Y₁)
 N slab, right * (Lever Arm Y₂) + N beam * Lever Arm Y₃;

Why is there an axial force in the rib?

SCIA Engineer integrates the ribs as eccentric beams attached to slabs. The eccentricity is calculated from the half of the slab thickness and half of the height of the cross section of the beam.



During the input of the cross section of the beam, the height of the cross section is defined as a distance between the bottom of the slab and the bottom of the beam. In the picture, the height is marked as "H".

Due to the shift of the neutral axis, the internal forces in the whole system change. In a simple system subject to a bending moment only, we get a structure with an internal bending moment as well as axial force.

Usually, if the beam is below the slab, we get compression in the slab and tension in the beam.

The eccentric beam causes axial forces in the slab. This results from the deformation of the whole slab+beam system. The picture shows the horizontal deformation "ux" to explain graphically the behaviour of the system. This system is composed of two beams of a rectangular cross section connected by rigid links. The horizontal displacement of the support is free to prevent the constraint.





The horizontal deformation in a side view:



If we look at the beginning of the beam, we can see compression in the slab and tension in the beam:



Of course, the whole system must be in equilibrium and the total axial force equal to the sum of the axial force in the slab and in the beam must be zero.



In our model, we have only one beam and all the internal forces of the top part are integrated in the axial force in the rib. Practically, the effective width of the slab is smaller than the whole width of the slab. Only exceptionally are the ribs arranged in such a way that there are no gaps in between the effective widths and all internal forces in the slab can be summed up into the rib. This happens if the distance between the ribs is smaller or equal to the effective width of the slab calculated from the national code.

Behaviour of a rib in a wide slab

Now we can investigate a system where the width of the slab is greater than the effective width of the slab. The equilibrium condition must be fulfilled. If we integrate all the axial forces in the whole slab and the beam, we - of course - get a zero result.

Distribution of the axial force in the slab. This is independent on the defined effective width of the slab. Only the stiffness of the slab and beam is responsible for the shape of the distribution of internal forces.



This is a section across the middle of the slab showing the distribution of the axial force.



We can integrate the axial force in the section across the whole width of the slab. We get 439kN.



Compared to the axial force in the beam, which is 435kN. We see the whole system is in equilibrium. The small difference results from the size of the finite elements.



Comparison of different effective widths

However, if we extend the effective width of the slab to the whole width of the slab, we neglect the distribution of the internal forces over the slab and the concentration over the beam. (In fact, there are two limit values: the minimum effective width is equal to the width of the beam and the maximum one is equal to the whole width of the slab.)

The internal forces in the slab are excluded from the slab and integrated into a new virtual T section. This virtual section consists of the effective slab width and the beam.

Distribution of the axial force in the slab. We can see that the distribution is equal to the one in the pictures above where the effective width of the slab was defined according to the code.



In the picture we can see the axial force after the forces within the effective width of the slab were excluded from the slab. In SCIA Engineer you can achieve this using the checkbox "RIB" in the results.



These axial forces within the effective width of the slab can be integrated.



We get axial force equal to 56kN, which is in the slab. The total axial force in the slab was 435kN. Therefore, in the part outside the effective width we have axial force 435 - 56 = 379kN.



In the beam, we still have the same 445kN. (The difference to the previous pictures results from the changed size of the 2D finite elements).



If we create the sum of the integrated axial force in the slab and in the beam, we must get 445 - 57 = 388kN.



Look what happens if we increase the effective width of the slab to 1500mm. This results from the following formula: $2 * (0,1 * L) + b_w = 2*0,6+0,3$



As we can see, the axial force in the slab is still the same. It must be, because the effective width of the slab has no influence on the distribution of the axial force in the finite element calculation. It only affects the split of the forces after the calculation between the slab and the virtual T section.

The area of the effective width of the slab will be removed from the slab and the forces will be integrated into the T section. The internal forces outside of the slab will remain in the slab.



These internal forces will be moved to the T section.



If we integrate the axial forces, we get 234kN.



In the rectangular section below the slab we get the original 445kN.



If we reduce this axial force of the beam by 234kN, which is the sum of the axial forces from the effective width of the slab, we get 211kN



The axial force outside the effective width remains in the slab.



If we integrate the forces (left and right) outside the effective width, we get axial force equal to 210kN, which is in equilibrium with the tension in the rib as a T Section.



3.3.2 Stiffness of ribs in CDD calculation

The calculation of the stiffness of the rib depends on the checkbox "Rib".

Check box is OFF

The stiffness of the beam and the plate will be calculated separately. If there is 1D reinforcement in part of the slab it is not taken into account for the calculation of the stiffness of the plate.

Check box is ON

- 1) Equilibrium for the final cross-section is calculated for each dangerous combination and each type of stiffness.
- 2) The stiffness of the rib, only taking into account the rib cross-section, is calculated with the height of compression zone from equilibrium on the whole (final) section. Stiffnesses are calculated to the centre of gravity of the transformed final cross-section.



- 3) Stiffness of the 2D element outside of the effective width is calculated by the standard procedure. Stiffness of the 2D element inside effective width is calculated in two directions: direction of the rib (α_{rib}) and direction perpendicular to the rib. (α_{rib} + 90)
- 4) The stiffness perpendicular to the rib is calculated by the standard procedure.

- 5) The stiffness in the direction of the rib is calculated according to following procedure:
 - The 1D reinforcement which is designed or inputted in part of the slab of the final cross-section is taken into account for the calculation of the stiffness of the 2D element. This reinforcement is transformed to 2D reinforcement and is added to the standard 2D reinforcement.
 - Uncracked stiffnesses (EA_I, Ely_{,I}, Elz_{,I}) will be calculated for the whole thickness of the 2D element with standard 2D reinforcement (required/provided/user) and with transformed reinforcement from the 1D member. The stiffness is calculated to the transformed centre of gravity of the uncracked section.
 - Cracked stiffness is calculated in case that (σ_{ct} <= σ_{cr}). The stiffness (EA_{II}, Ely,_{II}, Elz,_{II}) will be calculated taking into account parameters from the calculation of the 1D element which is nearest to centre of gravity of the 2D element. The height of compression zone is calculated according to formula:

$$x_{s} = \frac{A_{cc} - A_{cc,Rib}}{b_{eff}}$$

Where:

- \circ A_{cc} compressive area of whole cross-section for cracked CSS
- o Acc, RIB compressive area of part of cross-section (rib cross-section) for cracked CSS
- \circ b_{eff} effective width of the slab for check
- σct is maximum tensile strength calculated for final cross-section (rib cross-section + part of the slab) and for characteristic combination
- The stiffness is calculated to the transformed centre of gravity of the cracked section.
- The average stiffness will be calculated from the cracked and the uncracked stiffness using the distribution coefficient, which is calculated from the stresses calculated for the whole cross-section of the 1D element which is nearest to centre of gravity of the 2D element.
 - bending stiffness around y-axis (Ely) = $1/[\zeta/(Ely)_{\parallel} + (1-\zeta)/(Ely)_{\parallel}]$
 - bending stiffness around z-axis (Elz) = $1/[\zeta/(Elz)_{11} + (1-\zeta)/(Elz)_{11}]$
 - axial stiffness (EA) = $1/[(\zeta/(EA)_{II} + (1-\zeta)/(EA)_{I}]$

3.4 Orthotropy

In engineering practice, you may often come across a situation when the slab (or wall) to be designed has different characteristics (stiffness) in the longitudinal and transverse direction and thus, shows different behaviour in these two directions. Such a behaviour may result from the geometry (e.g. ribbed slabs) or from physical assumptions for a particular situation, for example, when determining deformations in a cracked plate or when excluding vertical members from a horizontal stiffening system (e.g. masonry walls).

Whenever you need to adjust the finite element model accordingly to reflect such a behaviour in SCIA Engineer, the orthotropic properties can be used. These orthotropic properties can be defined in two ways.

Orthotropy in the properties of a 2D member

2D MEMBER (1)				5	
Δ	1	₿	4		
1			Name	D1	
Layer			Layer	Calque1 🗸	÷
			Element type	Standard 🗸	
Element behaviour			lement behaviour	Standard FEM 🗸	
	Type Shape			plate (90) 🗸	
				Flat	
			Material	C25/30 🗸	Ŧ
	1		FEM model	Isotropic 🖂	1777
		FEM	I nonlinear model	Isotropic Orthotropic	
			Thickness type	constant 🗸	
			Thickness [mm]	200.00	
	N	lembe	r system-plane at	Centre 🗸	

Property modifier



The difference is in the data you need to enter. In the orthotropy, the stiffnesses are defined directly, while in the property modifier, a factor is specified by which the isotropic stiffnesses are multiplied.

The property modifier is a bit more flexible because it does not depend directly on the properties of the modified part. If you want to enter an uniaxially stretched plate, then you can do that for a 20cm thick plate and also for a 30cm thick plate using the same values. The orthotropic properties require that you define separate properties for each of the plates (20cm and 30cm one).

On the other hand, also the orthotropy has its advantages. It can be parameterized, and the program includes a set of generators to help you with the input.

However, it is important to understand individual orthotropic parameters. The stiffnesses are defined with parameters starting with a "D" or "d". The property modifiers ask for the following parameters for a shell element:



The parameters beginning with "D" represent plate stiffnesses. The parameters starting with "d" are membrane stiffnesses. The direction is derived from the direction of the local coordinate system.

- D11: Flexural stiffness in the "x" direction (bending)
- D22: Flexural stiffness in the "y" direction
- D12: Mixed stiffness of D11 and D22 (transverse contraction)
- D33: Torsional stiffness
- D44: Shear flexural stiffness in the "x" direction
- D55: Shear flexural stiffness in the "y" direction
- d11: Normal membrane stiffness in the "x" direction (stretching)
- d22: Normal membrane stiffness in the "y" direction
- d12: Mixed stiffness of "d11" and "d22" (transversal contraction)
- d33: Shear membrane stiffness

$$\begin{bmatrix} M_{x} \\ M_{y} \\ M_{xy} \\ T_{x} \\ T_{y} \end{bmatrix} = \begin{bmatrix} D_{11} & D_{12} & 0 \\ D_{21} & D_{22} & 0 \\ 0 & 0 & D_{33} \\ 0 & 0 & D_{33} \\ 0 & 0 & D_{55} \end{bmatrix} \begin{bmatrix} w_{xx} \\ w_{yy} \\ 2w_{xy} \\ w_{x} + \phi_{y} \\ w_{y} - \phi \end{bmatrix}$$



In case of a simple, isotropic plate, the stiffness can be expressed using the following formulas:

Plate direction	Membrane stiffness
Plate direction $D_{11} = D_{22} = \frac{E \cdot h^3}{12(1 - \nu^2)}$ $D_{12} = \nu \cdot \sqrt{D_{11} \cdot D_{22}}$ $D_{33} = G \cdot \frac{h^3}{12}$ $G = \frac{E}{2 \cdot (1 + \nu)}$	Membrane stiffness $d_{11} = d_{22} = \frac{E \cdot h}{1 - \nu^2}$ $d_{12} = \nu \cdot \sqrt{d_{11} \cdot d_{22}}$ $d_{33} = \frac{1}{2} \cdot (1 - \nu) \cdot \sqrt{d_{11} \cdot d_{22}}$
$D_{44} = D_{55} = G \cdot h$	

How to model a one-way slab in SCIA Engineer

A one-way slab is a slab that bears the load in one direction mainly. It can be a slab supported on two edges only or a slab supported on four edges for which the bigger span length L_y is at least twice the smaller span L_x . The design of a one-way slab will lead to reinforcement mainly in the bearing direction.

In a Finite Element software like SCIA Engineer, when the slab is supported on its four edges, the software will by default consider it as a two-way slab. Since there is no predefined main direction for the bearing of the load, the bending stiffness of the slab will participate in both x and y directions. In SCIA Engineer you can easily define a one-way-bearing slab.



Figure 1: Bending moments in a two-way slab (on the left) and one-way slab (on the right)

In SCIA Engineer, the input of a one-way slab can be done with orthotropy properties. Two types can be used and are explained below.

One-way slab using "two heights" orthotropy type

The example is made of a slab supported by beams and columns. In the slab properties, change the FEM model to "Orthotropy", edit the orthotropy property and select the type of orthotropy "two heights". The input data are the thickness of the plate for the calculation of the flexural stiffness in the x-direction, h_1 , and the y-direction, h_2 . For a slab bearing mainly in the x direction (smaller span length in the example), h_1 should be kept equal to the actual plate thickness (180 mm) and h_2 (thickness in y-direction) should be reduced.

2D MEMBER (1)		
1 5 4 🗄 🏢		
Name	D1	
Layer	Calque1 🗸 📑	
Element type	Standard V	
Element behaviour	Standard FEM 🗸	Name 011
Туре	plate (90) 🗸	Type of orthotropy Two_n
Shape	Flat	A Elevure
Material	C20/25 V	Effective height, h1 (x) [mm] 180
FEM model	Orthotropic 🗸	Effective height, h2 (y) [mm] 1
FEM nonlinear model	none 🗸	Torsion reduction coeff 1
Orthotropy	0T1 V	Shear reduction coeff 1.2
Member system-plane at	Centre V	D11 [MNm] 1.5188
Eccentricity z [mm]	0.00	D22 [MNm] 2.6042
LCC have	Chardend e.c.	D12 [MNm] 1.2578
LCS type		D33 [MNm] 2.5156
Swap orientation	C)	D44 [MN/m] 1.8750
LCS angle [deg]	0.00	D55 [MN/m] 1.0417

Figure 2: Parameters for a one-way slab using "two heights" orthotropy type

There is no specific rule regarding the value of h_2 . With smaller values of h_2 , the results will be close to the following load distribution:



Figure 3: Bending moment in the supporting beams of a one-way slab (on the left) and of a one-way load panel (on the right)

The resulting moment mx in the slab is then close to a 1m-wide simply supported beam:

$$m_x = \frac{q * L_x^2}{8} = \frac{3 * 5^2}{8} = 9,4$$
kNm/ml



Figure 4: bending moment mx in a one-way slab

One-way slab using "one direction" orthotropy type

This type of orthotropy requires three input parameters and can also be used for modelling hollow core slabs: the equivalent beam cross-section CSS, the spacing L used for the calculation of the flexural bending stiffness in direction 1 (or x) and the concrete topping height h used for the calculation of the flexural bending stiffness in direction 2 (or y). To model a one-way slab, a small value of h can be used. However, keep in mind, that h is also used for the calculation of the self-weight of the slab.

For the equivalent cross-section, a slab-equivalent shape is used: "thickness of the slab" x "width of the beam", i.e. 180 x 1000mm. For the spacing parameter, as the slab is plain, the same value as for the width of the beam is used, i.e. 1000mm.



Figure 5: Parameters for a one-way slab using "one direction" orthotropy type



Figure 6: Bending moment in the supporting beams and in the one way slab using the type "one direction"

For small values of h₂ or h, both types give the same results for the bending moment in the bearing direction and the load transferred to the supported beams.

There are still some differences between both orthotropy types. First, using "one direction" type leads to higher values of bending moment on the secondary beams (parallel to the bearing direction). This is due to the torsional moment component of the plate (D33) that is different between both types. Secondly, with "one direction" orthotropy type, the self-weight of the slab is calculated based on the concrete topping thickness h only. The total height of the slab is thus not accounted for and you have to add the missing part of the self-weight manually in a permanent load case.