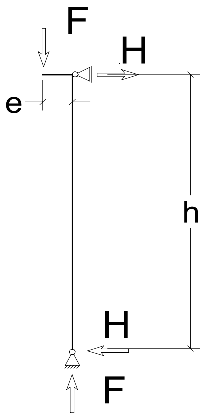
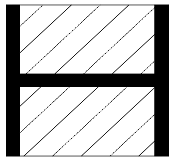
Euro-Code 4

Columns

Concrete encasement composite column with eccentric force application:





System:

Height h =	8,00 m		
Eccentricity e =	0,15 m		
Beam type =	SEL("steel/profils"; Name;)	=	HEB
Nominal height NH =	SEL("steel/"type; NH;)	=	340
A_a =	TAB("steel/"type; A; NH=NH)	=	171,00 cm²
h_p =	TAB("steel/"type; h; NH=NH)	=	340,00 mm
s =	TAB("steel/"type; s; NH=NH)	=	12,00 mm
t =	TAB("steel/"type; t; NH=NH)	=	21,50 mm
b =	TAB("steel/"type; b; NH=NH)	=	300,00 mm
I_{ya} =	TAB("steel/"type; I_y ; NH=NH)	=	36660,00 cm^4
W_{pa} =	1.14*TAB("steel/"type; W_y ; NH=NH)	=	2462,40 cm³
W_{pa} =			2408,00 cm³

Materials:

Concrete =	SEL("concrete/EC"; Name;)	=	C30/37
Steel =	SEL("steel/EC"; NameEN;)	=	S235
E_{cm} =	TAB("concrete/EC"; E_{cm}; Name=Concrete)	=	32000,00 N/mm²
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)	=	30,00 N/mm²
α =	0,85		
E_a =	TAB("steel/EC"; E; NameEN=Steel)	=	210000,00 N/mm²
f_{yk} =	TAB("steel/EC"; fy; NameEN=Steel)	=	235,00 N/mm²

Partial safety factors:

Dead load γ_G =	1,35
Imposed Load γ_Q =	1,50
Concrete γ_c =	1,50
Construction steel γ_a =	1,10
Profile steel sheeting γ_{ap} =	1,10
Longitudinal shear γ_{vs} =	1,25
Elastic modulus γ =	1,35

Load:

g_k = 450,00 kN
q_k = 1000,00 kN

Calculation:

N_{Sd} =	$g_k * \gamma_G + q_k * \gamma_Q$	=	2107,50 kN
M_{Sd} =	$N_{Sd} * e$	=	316,13 kNm
V_{Sd} =	M_{Sd} / h	=	39,52 kN
f_{cd} =	$\alpha * f_{ck} / \gamma_c$	=	17,00 N/mm²
f_{yd} =	f_{yk} / γ_a	=	213,64 N/mm²
E_{cd} =	E_{cm} / γ	=	23703,70 N/mm²
A_c =	$(h_p * b) /100 - A_a$	=	849,00 cm²
I_{yc} =	$(h_p^3 * b / 120000) - I_{ya}$	=	61600,00 cm^4

Local buckling of parts of the steel section:

ε =	$\sqrt{(235 / f_{yk})}$	=	1,00
$(b / t) / (\varepsilon * 44)$		=	<u>**0,32 < 1**</u>

Load-bearing capacity of the column under centric pressure with buckling risk:

$N_{pl,Rd}$ = $(A_a * f_{yd} + A_c * f_{cd}) /10$ = 5096,54 kN

Structural verification procudure applicable?

δ =	$A_a * f_{yd} / (10 * N_{pl,Rd})$	=	0,72
$0.2 / \delta$		=	<u>**0,28 < 1**</u>
$\delta / 0.9$		=	<u>**0,80 < 1**</u>

⇒ Structural verification procudure applicable.

On the effect of lateral shear:

A_v =	$(1.04 * h_p * s) / 100$	=	42,43 cm²
$V_{pl,Rd}$ =	$A_v * f_{yd} / (\sqrt{(3)} * 10)$	=	523,35 kN
$V_{Sd} / V_{pl,Rd}$		=	<u>**0,08 < 0,5**</u>

⇒ The lateral shear force has no effect on the load-bearing capacity.

Cross section bearing capacity under pressure and single-axis bending:

Plastic limit moment at point D:

W_{pc} =	$b * h_p^2 / 4000 - W_{pa}$	=	6262,00 cm³
$M_{max,Rd}$ =	$(W_{pa} * f_{yd} + 1/2 * W_{pc} * f_{cd}) / 10^3$	=	567,67 kNm
N_D =	$1/20 * A_c * f_{cd}$	=	721,65 kN

Plastic limit moment in points B and C:

$N_{pm,Rd}$ =	$(A_c * f_{cd}) / 10$	=	1443,30 kN
h_N =	$N_{pm,Rd} / (0.2 * b/10 * f_{cd} + 2 * s/100 * (2 * f_{yd} - f_{cd}))$	=	7,20 cm
$h_N / ((b / 2 - t) / 10)$		=	<u>**0,56 < 1**</u>

⇒ Plastic neutral axis is in the rib!

W_{pan} =	$s/10 * h_N^2$	=	62,21 cm³
W_{pcn} =	$(b * h_N^2) / 10 - W_{pan}$	=	1492,99 cm³
$M_{N,Rd}$ =	$(W_{pan} * f_{yd} + 1/2 * W_{pcn} * f_{cd})/10^3$	=	25,98 kNm

The plastic limit moment in points B and C is:

$M_{pl,Rd}$ =	$M_{max,Rd} - M_{N,Rd}$	=	541,69 kNm
N_B =			0,00 kN
N_C =	$2 * N_D$	=	1443,30 kN

Load-bearing capacity of a composite column under pressure and single-axis bending:

$N_{pl,Rk}$ = $(A_a * f_{yk} + A_c * \alpha * f_{ck})/10$ = 6183,45 kN

Effective elastic bending rigidity:

EI_e = $(E_a * I_{ya} + 0.8 * E_{cd} * I_{yc})/10^5$ = 88667,18 kN/m²

Reference degree of slenderness:

N_{cr} = $\pi^2 * EI_e / h^2$ = 13673,57 kN

λ' = $\sqrt{(N_{pl,Rk} / N_{cr})}$ = 0,67

⇒ Long-term behaviour (creep and shrinkage) may be disregarded.

Strut curve b:

Type = SEL("comp/buck"; Desc;) = concrete-encased profile strong axis

line = TAB("comp/buck"; line; Desc=Type) = b

κ = TAB("comp/buck"; κ ; line=line; λlat=λ') = 0,800

κ_d = $N_{Sd} / N_{pl,Rd}$ = 0,414

Proportion of edge moments r = 0,00

κ_N = $\kappa * ((1 - r) / 4)$ = 0,200

The resulting values for point C are:

κ_c = $N_C / N_{pl,Rd}$ = 0,283

μ_c = 1,00

μ_k = $(1 - \kappa) / (1 - \kappa_c) * \mu_c$ = 0,279

μ_d = $(1 - \kappa_d) / (1 - \kappa) * \mu_k$ = 0,817

The length μ is:

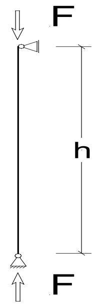
μ = $\mu_d - \mu_k * (\kappa_d - \kappa_N) / (\kappa - \kappa_N)$ = 0,717

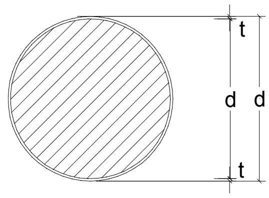
Structural verification of load-bearing capacity:

$N_{Sd} / (\kappa * N_{pl,Rd})$ = **0,52 < 1**

$M_{Sd} / (0.9 * \mu * M_{pl,Rd})$ = **0,90 < 1**

Concrete-filled hollow section:





System:

Column height h =	7,00 m
Outside pipe diameter d =	273,00 mm
Pipe thickness t =	6,30 mm

Load:

g_k =	500,00 kN
q_k =	600,00 kN

Materials:

Concrete =	SEL("concrete/EC"; Name;)	=	C30/37
Steel =	SEL("steel/EC"; NameEN;)	=	S355
E_{cm} =	TAB("concrete/EC"; E_{cm}; Name=Concrete)	=	32000,00 N/mm²
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)|	=	30,00 N/mm²
E_a =	TAB("steel/EC"; E; NameEN=Steel)	=	210000,00 N/mm²
f_{yk} =	TAB("steel/EC"; fy; NameEN=Steel)	=	355,00 N/mm²
α =	1,00		

Partial safety factors:

Dead load γ_G =	1,35
Imposed Load γ_Q =	1,50
Concrete γ_c =	1,50
Construction steel γ_a =	1,10
Profile steel sheeting γ_{ap} =	1,10
Longitudinal shear γ_{vs} =	1,25
Concrete elastic modulus γ =	1,35

Calculation:

f_{yd} =	f_{yk} / γ_a	=	322,73 N/mm²
f_{cd} =	$\alpha * f_{ck} / \gamma_c$	=	20,00 N/mm²
N_{sd} =	$\gamma_G * g_k + \gamma_Q * q_k$	=	1575,00 kN
f_{yd} =	f_{yk} / γ_a	=	322,73 N/mm²
A_a =	π / 400 * (d² - (d - 2 * t)²)	=	52,79 cm²
I_a =	π / 640000 * (d^4 - (d - 2 * t)^4)	=	4695,82 cm^4
E_{cd} =	E_{cm} / γ	=	23703,70 N/mm²
A_c =	π * (d - 2 * t)² /400	=	532,56 cm²
I_c =	π / 640000 * (d - 2 * t)^4	=	22570,10 cm^4

Local buckling of parts of the steel section:

$\varepsilon = \sqrt{(235 / f_{yk})}$ = 0,81

$(d/t) / (90 * \varepsilon^2)$ = **0,73 < 1**

Structural verification of the column's load-bearing capacity:

$N_{pl,Rd} = (A_a * f_{yd} + A_c * f_{cd}) /10$ = 2768,81 kN

Structural verification procudure applicable?

$\delta = A_a * f_{yk} / \gamma_a / (N_{pl,Rd} * 10)$ = 0,62

$0.2 / \delta$ = **0,32 < 1**

$\delta / 0.9$ = **0,69 < 1**

⇒Structural verification procudure applicable

$N_{pl,Rk} = (A_a * f_{yk} + A_c * \alpha * f_{ck})/10$ = 3471,72 kN

Effective elastic bending rigidity:

$EI_e = (E_a * I_a + 0{,}8 * E_{cd} * I_c)/10^5$ = 14141,18 kNm²

Reference degree of slenderness:

$N_{cr} = \pi^2 * EI_e / h^2$ = 2848,32 kN

$\lambda' = \sqrt{(N_{pl,Rk} / N_{cr})}$ = 1,10

$\lambda'_g = 0.8 / (1 - \delta)$ = 2,11

λ' / λ'_g = 0,52 < 1

⇒ Long-term behaviour (creep and shrinkage) may be disregarded.
Concrete-filled hollow section, therefore strut curve a

Type = SEL("comp/buck"; Desc;) = concrete-filled hollow section

line = TAB("comp/buck"; line; Desc=Type) = a

κ = TAB("comp/buck"; κ ; line=line; λlat=λ') = 0,596

$N_{Rd} = \kappa * N_{pl,Rd}$ = 1650,21 kN

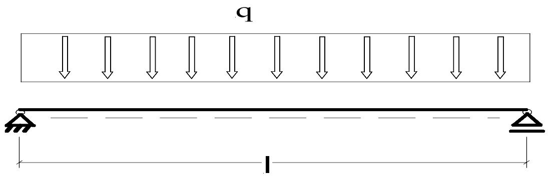
Structural verification:

N_{sd} / N_{Rd} = **0,95 < 1**

The ultimate load bearing capacity is almost completely exploited.

Floors

Composite beams - Ultimate load bearing capacity:



System:

Beam length L = 12,00 m
Beam distance s = 3,60 m

Materials:

Beam type =	SEL("steel/profils"; Name;)	=	IPE
Nominal height NH =	SEL("steel/"type; NH;)	=	450
A =	TAB("steel/"type; A; NH=NH)	=	98,80 cm²
h =	TAB("steel/"type; h; NH=NH)/10	=	45,00 mm
I_y =	TAB("steel/"type; I_y ; NH=NH)	=	33740,00 cm4

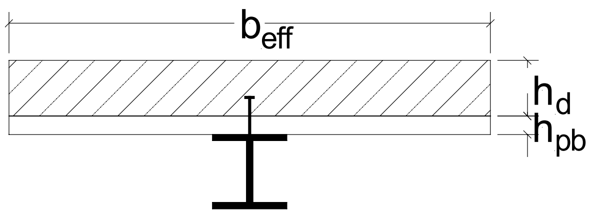
Shear connectors ∅ 22

h_b = 100,00 mm
d = 22,00 mm
Distance of shear connectors e_L = 15,00 cm

Concrete =	SEL("concrete/EC"; Name;)	=	C25/30
Steel =	SEL("steel/EC"; NameEN;)	=	S355

Load:

g_{1k} = 15,28 kN/m
g_{2k} = 7,74 kN/m (after removal of temporary props)
q_k = 18,00 kN/m



Floor thickness h_d = 16,00 cm
Thickness of profile steel sheeting h_{pb} = 5,10 cm

Material properties:

E_{cm} =	TAB("concrete/EC"; E_{cm}; Name=Concrete)	=	30500,00 N/mm²
E_a =	TAB("steel/EC"; E; NameEN=Steel)	=	210000,00 N/mm²

Cross section load bearing capacity at full shear connection:

b_{eff} =	2 * L / 8	=	3,00 m
b_{eff} =	MIN(b_{eff} ;s)	=	**3,00 m**

Working state analysis of composite beams:

Form factors:

Short term load at point of time t =0:

$n_0 =$ E_a / E_{cm} = 6,89

Effective component thickness:

Beam width in air u = 360,00 cm

$h_o =$ $2 * h_d * u / u$ = 32,00 cm

Final creep value t_0 =14 days

according to EC2 Appendix1 φ = 2,70

constant continuous load $\psi_{A,B}$ = 1,10

Shrinkage $\psi_{A,S}$ = 0,55

Creep under continuous load $n\varphi$ = $n_0 * (1+ \psi_{A,B} * \varphi)$ = 27,35

Shrinkage n_S = $n_0 * (1+ \psi_{A,S} * \varphi)$ = 17,12

$A_c =$ $100 * b_{eff} * (h_d - h_{pb})$ = 3270,00 cm²

$I_c =$ $100 * b_{eff} * (h_d - h_{pb})^3 / 12$ = 32375,72 cm²m²

$z'_{s0} =$ $(A*(h/2+h_d)+A_c/n_0*(h_d-h_{pb})/2)/(A+A_c/n_0)$ = 11,14 cm

$(I_y+I_c/n_0)/10000+(A*(h/2+h_d)^2+A_c/n_0*((h_d-h_{pb})/2)^2)/10000$ = 19,90 cm²m²

$((z'_{s0}/100)^2*(A+A_c/n_0)*(-1))$ = -7,12 cm²m²

I_{i0}= <u>12,78 cm²m²</u>

$z'_{s\varphi} =$ $(A*(h/2+h_d)+A_c/n_\varphi*(h_d-h_{pb})/2)/(A+A_c/n_\varphi)$ = 20,40 cm

$(I_y+I_c/n\varphi)/10000+(A*(h/2+h_d)^2+A_c/n\varphi*((h_d-h_{pb})/2)^2)/10000$ = 18,49 cm²m²

$((z'_s\varphi/100)^2*(A+A_c/n\varphi)*(-1))$ = -9,09 cm²m²

$I_{i\varphi}$= <u>9,40 cm²m²</u>

$z'_{sS} =$ $(A*(h/2+h_d)+A_c/n_S*(h_d-h_{pb})/2)/(A+A_c/n_S)$ = 16,72 cm

$(I_y+I_c/n_S)/10000+(A*(h/2+h_d)^2+A_c/n_S*((h_d-h_{pb})/2)^2)/10000$ = 18,78 cm²m²

$((z'_{sS}/100)^2*(A+A_c/n_S)*(-1))$ = -8,10 cm²m²

I_{iS}= <u>10,68 cm²m²</u>

Calculation of midspan deflection:

Release of temporary prop at point of time t =0

$B =$ $g_{1k} * 1.25 * L/2$ = 114,60 kN

$f_{B0} =$ $100 * B * L^3 / (48 * E_a/10 * I_{i0})$ = 1,54 cm

Assumption: 40% of the live load as a permanent load at point of time t =0:

Ratio= 0,40

$f_{g2,0} =$ $100 * 5/384 *(g_{2k} + Ratio* q_k) * L^4 / (E_a/10 * I_{i0})$ = 1,50 cm

from imposed load (short term ratio):

Ratio2 = 1 - Ratio = 0,60

$f_q =$ $100 * 5/384 * Ratio2 * q_k * L^4 / (E_a/10 * I_{i0})$ = 1,09 cm

From removal of temporary prop at point of time t =∞:

$f_B = \quad f_{B0} * I_{i0} / I_{i\varphi} \quad = \quad 2{,}09$ cm

From $g_{k2} + q_{permanent}$ at point of time t =∞:

$f_{g2} = \quad f_{g2,0} * I_{i0} / I_{i\varphi} \quad = \quad 2{,}04$ cm

From shrinkage at point of time t =∞:

Final shrinkage value ε_{cS} = $325*10^{-6}$

$N_{sch} = \quad (A_c * E_a / n_S * \varepsilon_{cS}) / 10 \quad = 1303{,}61$ kN

$z_{sch} = \quad z'_{sS} - (h_d - h_{pb})/2 \quad = \quad 11{,}27$ cm

$M_{sch} = \quad N_{sch} * z_{sch} / 100 \quad = \quad 146{,}92$ kN

$f_{sch} = \quad 100 * 1/8 * M_{sch} * L^2 / (E_a/10 * I_{iS}) \quad = \quad 1{,}18$ cm

Maximum deflection:

$max\_f = \quad f_B + f_{g2} + f_{sch} + f_q \quad = \quad 6{,}40$ cm

Recommended camber:

$f_0 = \quad f_{B0} + f_{g2,0} \quad = \quad 3{,}04$ cm

Maximum deflection in final state:

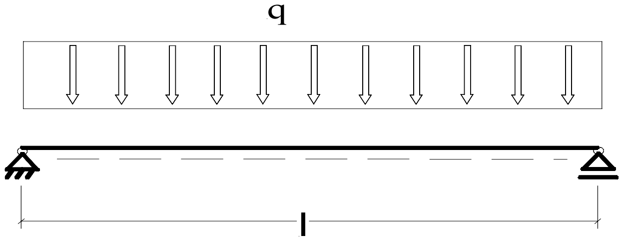
$f = \quad max\_f - f_0 \quad = \quad 3{,}36$ cm

Structural verification:

f/100 / (L / 300) = **0,84 < 1**

The deflection in the working state meets the required standard!

Composite floor in the final state:



System:

Steel profile with lugs; values according to manufacturer's specifications (approval)

m =	166,00
k =	0,15
Sheet thickness t =	0,86 mm
e =	17,00 mm
Beam length L =	4,80 m
Stability f_{yp} =	350,00 N/mm²
Cross section area A_p =	1562,00 N/mm²
$\tau_{u,Rd}$ =	280,00 kN/m
minimum width of concrete ribs b_0 =	750,00 mm/m

Concrete:

Floor thickness h_t = 14,00 cm

Concrete =	SEL("concrete/EC"; Name;)	=	C25/30
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)	=	25,00 N/mm²
f_{ctk005} =	TAB("concrete/EC"; fctk05; Name=Concrete)|	=	1,80 N/mm²

Influences:

Final state:

Permanent load of composite floor G_1 =	3,30 kN/m
Permanent load of design loads G_2 =	1,20 kN/m
Imposed load Q =	5,00 kN/m

Partial safety factors:

Dead load γ_G =	1,35
Imposed Load γ_Q =	1,50
Concrete γ_c =	1,50
Profile steel sheeting γ_{ap} =	1,10
Longitudinal shear γ_{vs} =	1,25

Structural verification of the composite floor in the final state:

b = 1000,00 mm

Deflection:

M_{Sd} =	$(\gamma_G * (G_1 + G_2) + \gamma_Q * Q) * L^2 / 8$	=	39,10 kNm/m

Design resistance:

N_{cf} =	$(A_p * f_{yp} / \gamma_{ap})/1000$	=	497,00 kN
α =			0,85
χ =	$1000 * N_{cf} / (b * (\alpha * f_{ck}/\gamma_c))$	=	35,08
d_p =	$10*h_t - e$	=	123,00 mm
$M_{p,Rd}$ =	$N_{cf} * (d_p - 0.5 * \chi) / 1000$	=	52,41 kNm/m

Structural verification:

$M_{Sd} / M_{p,Rd}$ = **<u>0,75 < 1</u>**

Longitudinal shear, m + k - method:

Design shear force at the footing point:

V_{Sd} =	(γ_G*(G_1+G_2) + γ_Q * Q) * L / 2	=	32,58 kN/m

Longitudinal shear resistance:

Shear length L_s =	1000 * L / 4	=	1200,00 mm
$V_{l,Rd}$ =	(b * d_p * (m * A_p / (b * L_s) + k) / γ_{vs})/1000	=	36,02 kN/m

Structural verification:

V_{Sd} / $V_{l,Rd}$ = **<u>0,90 < 1</u>**

Longitudinal shear, partial shear connection (EC4, Appendix E):

Design value of the bond strength according to manufacturer's information:

Shear length at full shear connection η =			1,00
L_{sf} =	N_{cf} / (b/1000 * $\tau_{u,Rd}$)	=	1,77 m

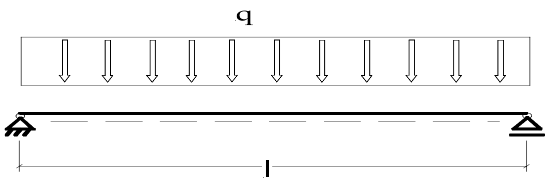
Design resistance:

τ_{Rd} =	0.09 * $(f_{ck})^{(1/3)}$	=	0,26 N/mm²
k_v =	1.6 - d_p /1000	=	1,48 > 1
A_p =	b_0 * t	=	645,00 mm²
ρ =	A_p / (b_0 * d_p)	=	0,007
$V_{v,Rd}$ =	(b_0 * d_p * τ_{Rd} * k_v * (1.2 + 40 * ρ))/1000	=	52,54 kN/m

Structural verification:

V_{Sd} / $V_{v,Rd}$ = **<u>0,62 < 1</u>**

Composite beams - Ultimate load bearing capacity:



System:

Beam length L =	12,00 m
Number of temporary props a =	1,00
Beam distance s =	3,60 m
Flange width b_0 =	12,60 cm

Materials:

Concrete =	SEL("concrete/EC"; Name;)	=	C25/30
Steel =	SEL("steel/EC"; NameEN;)	=	S355
Beam type =	SEL("steel/profils"; Name;)	=	IPE
Nominal height NH =	SEL("steel/"type; NH;)	=	400
A =	TAB("steel/"type; A; NH=NH)	=	84,50 cm²
h =	TAB("steel/"type; h; NH=NH)/10	=	40,00 cm
t =	TAB("steel/"type; s; NH=NH)/10	=	0,86 cm
I_y =	TAB("steel/"type; I_y ; NH=NH)	=	23130,00 cm^4
M_{pl} =	TAB("steel/"type; M_{plyd}; NH=NH)	=	289,00 kNm
M_{pl} =	IF(Steel="S355";1,5;1)*M_{pl}	=	433,50 kNm

Shear connectors∅ 22

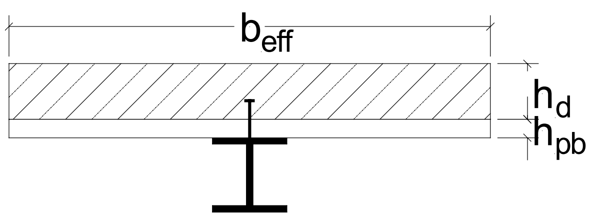
h_b =	100,00 mm
d =	19,00 mm
Distance of shear connectors e_L =	15,00 cm
Ultimate strength f_u =	450,00 N/mm²

Load:

g_k =	14,50 kN/m
q_k =	18,00 kN/m

Partial safety factors:

Dead load γ_G =	1,35
Imposed Load γ_Q =	1,50
Concrete γ_c =	1,50
Construction steel γ_a =	1,10
Profile steel sheeting γ_{ap} =	1,10
Longitudinal shear γ_{vs} =	1,25



Floor thickness h_d =	16,00 cm
Thickness of profile steel sheeting h_{pb} =	5,10 cm
Cross-sectional area of plate A_p =	15,62 kN/m

Material properties:

E_{cm} =	TAB("concrete/EC"; E_{cm}; Name=Concrete)	=	30500,00 N/mm²
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)	=	25,00 N/mm²
f_{ctk005} =	TAB("concrete/EC"; fctk05; Name=Concrete)	=	1,80 N/mm²
f_{cd} =	f_{ck} / γ_c	=	16,67 N/mm²
E_a =	TAB("steel/EC"; E; NameEN=Steel)	=	210000,00 N/mm²
f_{yk} =	TAB("steel/EC"; fy; NameEN=Steel)	=	355,00 N/mm²
f_{yd} =	f_{yk} / γ_a	=	322,73 N/mm²

Holorib sheeting in accordance with building regulations approval:

f_{yp} = 280,00 N/mm²

Conformation of fitness for purpose:

Stress resultants in ultimate state analysis of fitness for purpose:

r_d =	$\gamma_G * g_k + \gamma_Q * q_k$	=	46,58 kN/m
M_{Sd} =	r_d * L² / 8	=	838,44 kNm
V_{Sd} =	r_d *L / 2	=	279,48 kN

Cross section load bearing capacity with full shear connection:

b_{eff} =	2 * L / 8	=	3,00 m
b_{eff} =	MIN(b_{eff} ; s)	=	**<u>3,00 m</u>**
A_v =	1.04 * h * t	=	35,78 cm²
$N_{pl,a,Rd}$ =	A * f_{yd} / 10	=	2727,07 kN
N_{cd} =	0.85 * f_{cd} * 100 * b_{eff} * (h_d - h_{pb})	=	46334,26 kN
α_c =	0,85		
z_{pl} =	$N_{pl,a,Rd}$ / (10 * α_c * f_{cd} * b_{eff})	=	6,42 cm

⇒ The plastic neutral axis is in the concrete chord above the profile steel sheeting.

Distance to centre of gravity:

a =	h / 2 + h_d - z_{pl} / 2	=	32,79 cm
⇒ $M_{pl,Rd}$ =	$N_{pl,a,Rd}$ * a / 100	=	894,21 kNm
$V_{pl,Rd}$ =	A_v * f_{yk} / (10 * √(3) * γ_a)	=	666,68 kN

Structural verifications:

$M_{Sd} / M_{pl,Rd}$	=	**<u>0,94 < 1</u>**
$V_{Sd} / V_{pl,Rd}$	=	**<u>0,42 < 1</u>**

Longitudinal shear load capacity and shear connection:

h_b / d = 5,26 > 4 -> shear connectors are ductile

α =	IF(h_b /d>4; 1; 0.2 * ((h_b/d)+1))	=	1,00

Marginal force of a shear connector:

P_{Rd1} =	(0.8 * f_u * π * d² / (4 * γ_{vs}))/1000	=	81,66 kN
P_{Rd2} =	(0,29 * α * d² * √(f_{ck} * E_{cm}) /γ_{vs})/1000	=	73,13 kN
P_{Rd} =	MIN(P_{Rd1} ; P_{Rd2})	=	73,13 kN

single-row:

N_r = 1,00 (one row of shear connectors)

k_{t1} = (0.7 / √(N_r)) * (b_0 / h_{pb}) * ((h_b /10) / h_{pb}- 1) = 1,66

k_{t2} = 0,85

k_t = MIN (k_{t1} ; k_{t2}) = 0,85

P_{Rd1} = k_t * P_{Rd} = 62,16 kN

double-row:

N_r = 2,00 (two rows of shear connectors)

k_{t1} = (0.7 / √(N_r)) * (b_0 / h_{pb}) * ((h_b /10) / h_{pb}- 1) = 1,17

k_{t2} = 0,70

k_t = MIN (k_{t1} ; k_{t2}) = 0,70

P_{Rd2} = k_t * P_{Rd} = 51,19 kN

Possible shear connector configurations:

7 pairs of shear connectors at the end of the beam Anzp = 7,00

33 individual shear connectors in the span Anze = 33,00

aufn_V = Anze * P_{Rd1} + 2 * Anzp * P_{Rd2} = 2767,94 kN

Structural verification:

$N_{pl,a,Rd}$ / aufn_V = **0,99 < 1**

Structural verification against diagonal strut failure according to NAD for Germany:

A_{cv} = 100 * (h_d - h_{pb}) = 1090,00 N/m

Normal concrete η = 1,00

$v_{Rd,2}$ = 0.2 * A_{cv} * η * f_{ck} / (γ_c *10) = 363,33 kN/m

v_{Sd} = P_{Rd1} / (e_L / 100) = 414,40 kN/m

$v_{Sd,li}$ = v_{Sd} *1/2 = 207,20 kN/m

$v_{Sd,li}$ / $v_{Rd,2}$ = **0,57 < 1**

This calculation disregards the input from profile steel sheeting!

Cross reinforcement necessary:

NAD τ_{Rd} = 0.09 * $f_{ck}^{(1/3)}$ = 0,26 N/mm²

v_{pd} = A_p * f_{yp} /(γ_{ap} * 10) = 397,60 kN/m

$v_{Rd,3}$ = 2.5 * A_{cv} * η * (τ_{Rd} / 10) + v_{pd} = 468,45 kN/m

Structural verification of diagonal strut:

$v_{Sd,li}$ / $v_{Rd,3}$ = **0,44 < 1**

Working state analysis of composite beams:

Form factors:

Short term load at point of time t =0:

n_0 = E_a / E_{cm} = 6,89

Effective component thickness:

Beam width in air u = 360,00 cm

h_o = 2 * h_d *u / u = 32,00 cm

Final creep value t_0 =14 days

according to EC2 Appendix1 φ =			2,70
Creep under continuous load $n\varphi$ =	$n_0 * 3$	=	20,67
Shrinkage and creep n_S =	$n_0 * 2$	=	13,78
A_c =	$100 * b_{eff} * (h_d - h_{pb})$	=	3270,00 cm²
I_c =	$100 * b_{eff} * (h_d - h_{pb})^3 / 12$	=	32375,72 cm²m²

z'_{s0} = $(A*(h/2+h_d)+A_c/n_0*(h_d-h_{pb})/2)/(A+A_c/n_0)$ = 10,07 cm

$(I_y+I_c/n_0)/10000+(A*(h/2+h_d)^2+A_c/n_0*((h_d-h_{pb})/2)^2)/10000$ = 15,14 cm²m²
$((z'_{s0}/100)^2*(A+A_c/n_0)*(-1))$ = -5,67 cm²m²

I_{i0}= 9,47 cm²m²

$z'_{s\varphi}$ = $(A*(h/2+h_d)+A_c/n_\varphi*(h_d-h_{pb})/2)/(A+A_c/n_\varphi)$ = 16,09 cm

$(I_y+I_c/n\varphi)/10000+(A*(h/2+h_d)^2+A_c/n\varphi*((h_d-h_{pb})/2)^2)/10000$ = 13,89 cm²m²
$((z'_s\varphi/100)^2*(A+A_c/n\varphi)*(-1))$ = -6,28 cm²m²

$I_{i\varphi}$= 7,61 cm²m²

z'_{sS} = $(A*(h/2+h_d)+A_c/n_S*(h_d-h_{pb})/2)/(A+A_c/n_S)$ = 13,47 cm

$(I_y+I_c/n_S)/10000+(A*(h/2+h_d)^2+A_c/n_S*((h_d-h_{pb})/2)^2)/10000$ = 14,20 cm²m²
$((z'_{sS}/100)^2*(A+A_c/n_S)*(-1))$ = -5,84 cm²m²

I_{iS}= 8,36 cm²m²

Calculation of midspan deflection:

Release of temporary prop at point of time t =0

B =	$g_k * 1.25 * L/2$	=	108,75 kN
f_{B0} =	$100 * B * L^3 / (48 * E_a/10 * I_{i0})$	=	1,97 cm

Assumption: 40% of the live load as a permanent load at point of time t =0:

Ratio=			0,40
$f_{g2,0}$ =	$100 * 5/384 * Ratio* q_k * L^4 / (E_a/10 * I_{i0})$	=	0,98 cm
f_0 =	**$f_{B0} + f_{g2,0}$**	=	2,95 cm

from imposed load (short term ratio):

Ratio2 =	1 - Ratio	=	0,60
f_q =	$100 * 5/384 * Ratio2 * q_k * L^4 / (E_a/10 * Ii0)$	=	1,47 cm

From removal of temporary prop at point of time t =∞:

f_B =	$f_{B0} * I_{i0} / I_i\varphi$	=	2,45 cm

From g_{k2} + $q_{permanent}$ at point of time t =∞:

f_{g2} = $f_{g2,0}$ * I_{i0} / $I_{i\varphi}$ = 1,22 cm

f_d = $f_B + f_{g2}$ = 3,67 cm

Creep under continuous load only:

f_k = $f_d - f_0$ = 0,72 cm

From shrinkage at point of time t =∞:

Final shrinkage value ε_{cS} = $325*10^{-6}$

N_{sch} = $(A_c * E_a / n_S * \varepsilon_{cS}) / 10$ = 1619,58 kN

z_{sch} = $z'_{sS} - (h_d - h_{pb})/2$ = 8,02 cm

M_{sch} = $N_{sch} * z_{sch} / 100$ = 129,89 kN

f_{sch} = $100 * 1/8 * M_{sch} * L^2 / (E_a/10 * I_{iS})$ = 1,33 cm

Maximum deflection:

max_f = $f_B + f_{g2} + f_{sch} + f_q$ = 6,47 cm

Recommended camber:

f_0 = $f_{B0} + f_{g2,0}$ = 2,95 cm

Maximum deflection in final state:

f = max_f - f_0 = 3,52 cm

Structural verification:

f/100 / (L / 250) = **0,73 < 1**

The deflection in the working state meets the required standard!

Vibration behaviour:

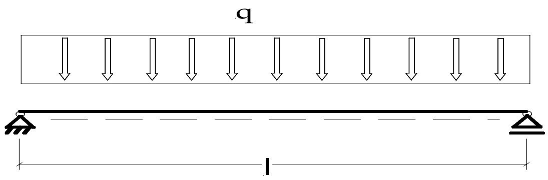
ω = $10 * \pi^2 / L^2 * \sqrt{(E_a * I_{i0} / (g_k * 98.1))}$ = 25,63 (1/s)

f = $\omega / (2 * \pi)$ = 4,08 Hz

The natural frequency should be no less than 3 Hz.

For sports or dance halls, it should be no less than 5 Hz.

POS.: Composite beams - Ultimate load bearing capacity:



System:

Beam length L = 12,00 m

Number of temporary props a = 1,00

Beam distance s = 3,60 m

Flange width b_0 = 12,60 cm

Materials:

Concrete =	SEL("concrete/EC"; Name;)	=	C25/30
Steel =	SEL("steel/EC"; NameEN;)	=	S355
Beam type =	SEL("steel/profils"; Name;)	=	IPE
Nominal height NH =	SEL("steel/"type; NH;)	=	450
A =	TAB("steel/"type; A; NH=NH)	=	98,80 cm²
h =	TAB("steel/"type; h; NH=NH)/10	=	45,00 cm
t =	TAB("steel/"type; s; NH=NH)/10	=	0,94 cm
I_y =	TAB("steel/"type; I_y ; NH=NH)	=	33740,00 cm4
M_{pl} =	TAB("steel/"type; M_{plyd} ; NH=NH)	=	373,00 kNm
M_{pl} =	IF(Steel="S355";1,5;1)*M_{pl}	=	559,50 kNm

Shear connectors∅ 22

h_b = 100,00 mm

d = 22,00 mm

Distance of shear connectors e_L = 15,00 cm

Ultimate strength f_u = 450,00 N/mm²

Load:

g_{k1} = 15,28 kN/m

g_{k2} = 7,74 kN/m (after removal of temporary props)

q_k = 18,00 kN/m

Partial safety factors:

Dead load γ_G = 1,35

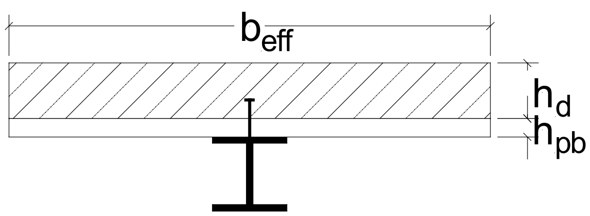
Imposed Load γ_Q = 1,50

Concrete γ_c = 1,50

Construction steel γ_a = 1,10

Profile steel sheeting γ_{ap} = 1,10

Longitudinal shear γ_{vs} = 1,25



Floor thickness h_d = 16,00 cm

Thickness of profile steel sheeting h_{pb} = 5,10 cm

Cross-sectional area of plate A_p = 15,62 kN/m

Material properties:

E_{cm} = TAB("concrete/EC"; E_{cm}; Name=Concrete) = 30500,00 N/mm²
f_{ck} = TAB("concrete/EC"; fck; Name=Concrete) = 25,00 N/mm²
f_{ctk005} = TAB("concrete/EC"; fctk05; Name=Concrete) = 1,80 N/mm²
f_{cd} = f_{ck} / γ_c = 16,67 N/mm²

E_a = TAB("steel/EC"; E; NameEN=Steel) = 210000,00 N/mm²
f_{yk} = TAB("steel/EC"; fy; NameEN=Steel) = 355,00 N/mm²
f_{yd} = f_{yk} / γ_a = 322,73 N/mm²
Holorib sheeting in accordance with building regulations approval:
f_{yp} = 280,00 N/mm²

Conformation of fitness for purpose:

Stress resultants in ultimate state analysis of fitness for purpose:
r_d = γ_G * (g_{k1} + g_{k2}) + γ_Q * q_k = 58,08 kN/m
M_{Sd} = r_d * L² / 8 = 1045,44 kNm
V_{Sd} = r_d *L / 2 = 348,48 kN

Cross section load bearing capacity with full shear connection:
b_{eff} = 2 * L / 8 = 3,00 m
b_{eff} = MIN(b_{eff} ; s) = **3,00 m**

A_v = 1.04 * h * t = 43,99 cm²

$N_{pl,a,Rd}$ = A * f_{yd} / 10 = 3188,57 kN
N_{cd} = 0.85 * f_{cd} * 10 * b_{eff} * (h_d - h_{pb}) = 4633,43 kN
α_c = 0,85

z_{pl} = $N_{pl,a,Rd}$ / (10 * α_c * f_{cd} * b_{eff}) = 7,50 cm
⇒ The plastic neutral axis is in the concrete chord above the profile steel sheeting.

Distance to centre of gravity:
a = h / 2 + h_d - z_{pl} / 2 = 34,75 cm

⇒ $M_{pl,Rd}$ = $N_{pl,a,Rd}$ * a / 100 = 1108,03 kNm

$V_{pl,Rd}$ = A_v * f_{yk} / (10 * √(3) * γ_a) = 819,65 kN

Structural verifications:

M_{Sd} / $M_{pl,Rd}$ = **0,94 < 1**
V_{Sd} / $V_{pl,Rd}$ = **0,43 < 1**

Longitudinal shear capacity and shear connection:

h_b / d = 4,55 > 4 -> shear connectors are ductile
α = IF(h_b /d>4; 1; 0.2 * ((h_b/d)+1)) = 1,00

Marginal force of a shear connector:
P_{Rd1} = (0.8 * f_u * π * d² / (4 * γ_{vs}))/1000 = 109,48 kN
P_{Rd2} = (0,29 * α * d² * √(f_{ck} * E_{cm}) /γ_{vs})/1000 = 98,05 kN
P_{Rd} = MIN(P_{Rd1} ; P_{Rd2}) = 98,05 kN

Number of rows of shear connectors N_r = 1,00

$k_{t1} = (0.7 / \sqrt{(N_r)}) * (b_0 / h_{pb}) * ((h_b / 10) / h_{pb} - 1)$ = 1,66

$k_{t2} =$ 0,75

$k_t = \text{MIN}(k_{t1} ; k_{t2})$ = 0,75

$P_{Rd} = k_t * P_{Rd}$ = 73,54 kN

Required number of shear connectors for full shear connection:

erf_n = $N_{pl,a,Rd} / P_{Rd} + 0.49$ = 44 Dübel

vorh_n = $100 * L/2/e_L + 0.49$ = 40 Dübel

because: erf_n / vorh_n = 1,10 is a partial connection

The maximum transferrable concrete compression force is:

$N_c =$ vorh_n * P_{Rd} = 2941,60 kN

Degree of shear connection $\eta = N_c / N_{pl,a,Rd}$ = 0,92 < 1

Reduced plastic bending moment, calculated with linear interpolation:

$M_{Rd} = M_{pl} + \eta * (M_{pl,Rd} - M_{pl})$ = 1064,15 kNm

Structural verification:

M_{Sd} / M_{Rd} = **0,98 < 1**

Note: No reduction in interaction, because maximum moment and maximum lateral shear force do not occur in the same place.

Shear connector configurations permissible?

a) Cross section class 1 or 2 $\Rightarrow$ OK

b) erf_η = $0.25 + 0.03 * L$ = 0,61

erf_η/η = **0,66 < 1**

c)

$M_{pl,Rd} / (2.5 * M_{pl})$ = **0,79 < 1**

All conditions a-c have to be met!

Connection of lateral concrete chord:

Lateral shear::

$v_{Sd} = P_{Rd} / (e_L / 100)$ = 490,27 kN/m

Relevant:

$v_{Sd,li} = v_{Sd} * 1/2$ = 245,13 kN/m

$A_{cv} = (h_d - h_{pb}) * 100$ = 1090,00 cm²/m

Normal concrete η = 1,00

$v_{Rd,2} = 0.2 * A_{cv} * \eta * f_{ck} / (\gamma_c * 10)$ = 363,33 kN/m

Structural verification against diagonal strut failure:

$v_{Sd,li} / v_{Rd,2}$ = **0,67 < 1**

This calculation disregards the input from profile steel sheeting!

EC4 $\tau_{Rd1} = 0.25 * f_{ctk005} / \gamma_c$ = 0,30 MN/mm²

NAD $\tau_{Rd2} = 0.09 * f_{ck}^{(1/3)}$ = 0,26 MN/mm²

$\tau_{Rd} = \text{MIN}(\tau_{Rd1} ; \tau_{Rd2})$ = 0,26 MN/mm²

$v_{pd} = A_p * f_{yp} / (\gamma_{ap} * 10)$ = 397,60 kN/m

$v_{Rd,3} = 2.5 * A_{cv} * \eta * (\tau_{Rd} / 10) + v_{pd}$ = 468,45 kN/m

Structural verification of diagonal strut:

$\nu_{Sd,li}$ / $\nu_{Rd,3}$ = **0,52 < 1**

The cross reinforcement has been ignored in this calculation.
In the construction, provision should be made for 0.2% or the relevant cross section area A_{cv} that may also be taken into account
as part of the bending reinforcement to absorb the negative moment (moment at support) of the transversal continuous composite floor.