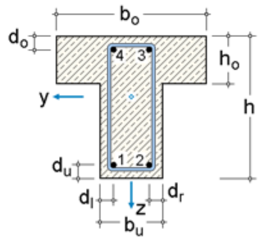
POS. 32: T-BEAM (REINFORCED CONCRETE 2-ACHS.)

bending and shear design incl. verif. of serviceability (EC 2 (1.11), NA: Deutschland)

biaxial bending with/abs. axial force (4H-BETON version: 11/2007-4l)



T-beam-cross section

h = 75.0 cm h_o = 16.0 cm

b_o = 145.0 cm b_u = 25.0 cm

edge distances of longit. reinf.

d_o = 6.0 cm d_u = 6.0 cm

d_l = 6.0 cm d_r = 6.0 cm

material

C25/30

BSt 500 (A)

γ_s = 1.15, γ_c = 1.50

exposure class X0

min./max. reinforcement

min A_s (9.2.1.1, 9.5.2), max ρ_0 = 8.00 %

reinforcement groups

Nr	rank	min A_s	max A_s
	-	cm^2	cm^2
1	x	**20.00**	100.00
2	x	**25.00**	100.00
3	x	**15.00**	100.00
4	x	**30.00**	100.00

min A_s: initial reinforcement per group
max A_s: highest reinforcement amount per group

ranking variable by load cases

stirrups: min $a_{sbü}$ = 40.00 cm^2/m

verifications in ultimate limit states are executed with stress-strain relation for concrete acc. to 3.1.7 (figure 3.3)
with fcd = αc fck / γc = 14.2 MN/m2 and reinforcement stress-strain relation acc. to 3.2.7 (fig. 3.8) with fyd = fyk / γs = 434.8 MN/m2
and ftd = ftk / γs = 456.5 MN/m2 !
verifications in serviceability limit states are executed with stress-strain relation for concrete acc. to 3.1.5 (figure 3.2)
with fc = fcm = 33.0 MN/m2 and reinforcement stress-strain relation acc. to 3.2.7 (figure 3.8) with fy = fyk, ft = 525.0 MN/m2 and εuk = 25‰ !

design calculation values and minimum reinforcement areas (EC 2, 6.1)

	γ	N_{Ed}	M_{yEd}	M_{zEd}	ε_{c2u}	ε_{s2u}	ε_{s1u}	ε_{c1u}	α_{ku}	d	z	x
	-	kN	kNm	kNm	‰	‰	‰	‰	°	cm	cm	cm
1	---	90.00	100.00	300.00	-3.50	0.52	5.92	13.65	209.40	73.7	65.4	27.4
			0.00	-20.04	-2.47	1.73	25.00	47.23	0.00	79.0	75.7	7.1

ε_{c2u}: concr. strain in state of failure (fibre 2), ε_{s1u}: reinforcement strain in state of failure (fibre 1),
α_{ku}: dir. angle of cross section principal strain, d: static height, z: lever arm of internal forces, x: height of concr. compr. zone

	A_{sb1}	A_{sb2}	A_{sb3}	A_{sb4}	note
	cm^2	cm^2	cm^2	cm^2	
1	4.14	4.14	4.14	4.14	12)
	0.22	0.22	0.22	0.22	8) 12)

8) minimum reinforcement acc. to 9.2.1.1   12) uniaxiale design calculation uneconomic

⇒ **longitudinal reinforcement:** **min A_s = 20.0/25.0/15.0/30.0 cm^2**

shear design calculation (EC 2, 6.2 + 6.3) - separated into V_{yEd} +T_{Ed} and V_{zEd} +T_{Ed}
minimum reinforcement acc. to 9.2.2(5), material quality as flexural reinf.
z = 0.9 d (10.3.4(2), d in each direction), $c_{v,D}$ = 3.0 cm, D = compression reinf.
angle of compr. strut θ_{gew} = 0°, torsion: t_{eff} = A_c/U_c > 2·min(d_o,d_u)
the minimum value of V_{Rdct} is limited acc. to design code (V_{Rdct} ≥ min V_{Rdct}).
only web design; connection of compression/tension boom has to be designed separately.

design calculation of shear force (EC 2, 6.2)

	V_{yEd}	V_{zEd}	z_y	V_{yRdct}	θ_y	V_{yRdmax}	z_z	V_{zRdct}	θ_z	V_{zRdmax}	$a_{s,büV}$	note
	kN	kN	cm	kN	°	kN	cm	kN	°	kN	cm^2/m	
1	100.00	45.00	13.0	79.43	33.8	479.16	62.1	66.63	30.2	716.75	11.86	

z: decisive inner lever arm, V_{Rdct}: design value of shear resistance without shear reinforcement
θ: angle of compr. strut, V_{Rdmax}: design value of maximal shear resistance

design calculation of torsion (EC 2, 6.3)

	T_{Ed}	V_{yEdT+V}	θ_y	V_{zEdT+V}	θ_z	T_{Rdmax}	$a_{s,büT}$	$A_{s,T}$	note
	kNm	kN	°	kN	°	kNm	cm^2/m	cm^2	
1	35.00	39.17	33.8	128.88	30.2	66.14	2.63	10.97	

stirrup reinforcement $a_{s,büT}$ per leg, longit. reinf. for torsion $A_{s,T}$ uniformly distributed along the perimeter

design calculation of shear force and torsion (EC 2, 6.3(4))

1: $(T_{Ed}/T_{Rdmax}) + (V_{yEd}/V_{yRdmax}) =$ **0.32 < 1.0**

$(T_{Ed}/T_{Rdmax}) + (V_{zEd}/V_{zRdmax}) =$ **0.32 < 1.0** ⇒ verification executed !

⇒ **shear reinforcement: min $a_{s,büV}$ = 40.00 cm^2/m**

(incl. initial reinf.)

torsion: **min $a_{s,büT}$ = 2.63 cm^2/m** **(1-shear)**

Σ (2-shear) **min $a_{s,bü}$ = 45.26 cm^2/m**

torsion: **min $A_{s,T}$ = 11.0 cm^2** (uniformly distributed along the perimeter)

crack control (EC 2, 7.3: 7.3.2 minimum reinforcement, 7.3.3 without direct calculation)

cracking in bending restraint (self induced)

factor for the concrete hardening process $k_{z,t}$ = 1.00

axial force in the centre of gravity at formation of first crack N_{cr} = 0.00 kN

crack width w_k = 0.30 mm

crack forces and moments: N_r = 75.00 kN M_{yr} = 100.00 kNm M_{zr} = 165.00 kNm

reinforcement (initial state): A_s = 20.00/25.00/15.00/30.00 cm^2

Nr	d_s	k_c	k	$A_{s,min}$	d_{sgr}	σ_s	ΔA_{sr}
	mm	-	-	cm^2	mm	N/mm^2	cm^2
1	20	0.40	0.80	1.79	60.0	32.2	0.00
2	20	0.40	0.80	1.79	60.0	49.2	0.00
3	20	0.40	0.80	1.79	60.0	48.3	0.00
4	20	0.40	0.80	1.79	60.0	31.3	0.00

concr. tens. str. (restr.) $f_{ct,eff}$ = 2.56 N/mm^2 concr. tens. strength (load) $f_{ct,eff} = f_{ctm}$ = 2.56 N/mm^2

k_c: coeff. - stress distribution, k: coeff. - concr. tens. stress, As,min: min. reinf. from restraint

d_s: selected bar diameter, d_{sgr}: existing bar diameter

σ_s: steel tensile stress, ΔA_{sr}: reinf. increase from load and restraint

⇒ **no additional anti-crack reinforcement !**

fatigue design (EC 2, 6.8.5 + 6.8.7(1))

for steel: $U_{s1} = \gamma_{F,fat}\ \gamma_{Ed,fat}\ \Delta\sigma_{s,equ} \le U_{s2} = \Delta\sigma_{Rsk}(N^*)/\gamma_{s,fat}$ = 152.17 N/mm^2

damage equivalent stress range $\Delta\sigma_{s,equ} = \sigma_{s,0} - \sigma_{s,U}$

partial safety factors $\gamma_{F,fat}$ = 1.00, $\gamma_{Ed,fat}$ = 1.00, $\gamma_{s,fat} = \gamma_s$ = 1.15

allowable stress range $\Delta\sigma_{Rsk}(N^*)$ = 175.0 N/mm^2

shear force : $\Delta\sigma_{Rskv}(N^*)$ = 107.0 N/mm^2 ⇒ $U_{s2v} = \Delta\sigma_{Rskv}(N^*)/\gamma_{s,fat}$ = 93.04 N/mm^2

for conc.: $U_{c1} = |\sigma_{cd,max,equ}|/f_{cd,fat} + 0.43\ \text{sqrt}(1 - \sigma_{cd,min,equ}/\sigma_{cd,max,equ}) \le 1.0$

design value of compression strength $f_{cd,fat}$ = 15.00 N/mm^2 at t_0 = 28 d

material safety $\gamma_{c,fat} = \gamma_c$ = 1.50

load: N_{s1} = 25.00 kN M_{ys1} = 55.00 kNm M_{zs1} = 135.00 kNm V_{ys1} = 100.00 kN V_{zs1} = 80.00 kN

N_{s2} = 35.00 kN M_{ys2} = 75.00 kNm M_{zs2} = 80.00 kNm V_{ys2} = 80.00 kN V_{zs2} = 125.00 kN

reinforcement (initial state): A_s = 20.00/25.00/15.00/30.00 cm^2 $a_{s,büV}$ = 40.00 cm^2/m

fatigue design for steel:

Nr	$\sigma_{s,0}$	$\sigma_{s,U}$	$\Delta\sigma_{s,equ}$	U_{s1}	$\Delta A_{s,fat}$
	N/mm^2	N/mm^2	N/mm^2	N/mm^2	cm^2
1	25.81	14.54	11.27	11.27	0.00
2	31.96	29.10	2.86	2.86	0.00
3	43.16	13.70	29.46	29.46	0.00
4	28.59	7.55	21.04	21.04	0.00

reinforcement (shear force):

$\Delta\sigma_{sv,equy}$ = 111.03 - 88.82 = 22.21 N/mm^2

U_{s1vy} = 22.21 < U_{s2v} = 93.04

$\Delta\sigma_{sv,equz}$ = 29.05 - 18.59 = 10.46 N/mm^2

U_{s1vz} = 10.46 < U_{s2v} = 93.04

concrete fatigue design:

$\sigma_{cd,min,equ}$ = 3.19 N/mm^2

$\sigma_{cd,max,equ}$ = 7.07 N/mm^2

U_{c1} = 0.79 < 1.00 ⇒ verification executed !

verif. of compression strut for decisive $V_{s1,2}$:

$\sigma_{cdv,min,equ}$ = 2.46 N/mm^2

$\sigma_{cdv,max,equ}$ = 3.08 N/mm^2

U_{c1v} = 0.30 < 0.61 ⇒ verification executed !

⇒ **no additional fatigue reinforcement !**

limitation of steel tension and concrete compression stresses (EC 2, 7.2)
permitted tensile stress of reinf. $\sigma_s = 0.80 \cdot f_{yk} = 400.0$ N/mm²
permitted concrete compression stress $\sigma_c = 0.60 \cdot f_{ck} = -15.0$ N/mm²
stress forces and moments: $N_\sigma = 0.00$ kN $M_{y\sigma} = 100.00$ kNm $M_{z\sigma} = 300.00$ kNm
reinforcement (initial state): A_s = 20.00/25.00/15.00/30.00 cm²

maximal reinforcement tensile stresses

Nr	σ_{0s} N/mm²	σ_s N/mm²	$\Delta A_{s\sigma}$ cm²
1	21.0	19.5	1.13
2	54.0	51.5	1.33
3	95.9	91.6	0.96
4	62.9	59.5	1.54

σ_{0s}: initial state, σ_s: end state
$\Delta A_{s\sigma}$: reinforcement increase from steel and concrete design

minimal concrete compression stress
initial state:
$\sigma_{0c} = -15.2$ N/mm²
end state:
$\sigma_c = -15.0$ N/mm² > -15.0

⇒ incl. stress reinforcement: min A_s = 21.1/26.3/16.0/31.5 cm²

total reinforc.: total A_s = 21.1/26.3/16.0/31.5 cm²
total $a_{s,büV}$ = 40.00 cm²/m
total $a_{s,büT}$ = 2.63 cm²/m, $A_{s,T}$ = 11.0 cm²
degree of utilization: U = 0.56

additional reinforcement: ΔA_s = 1.1/1.3/1.0/1.5 cm²

selected:
longitudinal, E1: 1 Ø 25 = 4.9 cm² < 21.1 cm²
E2: 1 Ø 25 = 4.9 cm² < 26.3 cm²
E3: 1 Ø 25 = 4.9 cm² < 16.0 cm²
E4: 1 Ø 25 = 4.9 cm² < 31.5 cm²
stirrups, 2-shear: Ø 8 / 15 cm = 6.70 cm²/m < 45.26 cm²/m

cross-section data
gross area of concrete: A_c = 37.9 dm² second moment of areae: I_{cys} = 174.5 dm⁴, I_{czs} = 414.2 dm⁴
centroid coordinates (from centre of upper edge): y_s = 0.0 cm, z_s = 22.6 cm
total area of longitudinal reinforcement: Σ(min A_s) = 95.0 cm² ⇒ ρ_s = 2.50% < 8.00%

material properties for design calculation

concrete	f_{ck} MN/m²	α -	ε_{c2} ‰	ε_{c2u} ‰	n_c -	E_{cm} MN/m²	f_{ctm} MN/m²
C25/30	25.0	0.850	-2.00	-3.50	2.00	31475.8	2.565

design value of compression strength $f_{cd} = \alpha_c f_{ck} / \gamma_c$
strain at reaching the maximum strength ε_{c2}, ult. compr. strain ε_{c2u}
concr. comp. stress $\sigma_c = f_{cd} (1-(1-\varepsilon_c/\varepsilon_{c2})^n)$ for $0 \geq \varepsilon_c > \varepsilon_{c2}$ and $\sigma_c = f_{cd}$ for $\varepsilon_{c2} \geq \varepsilon_c > \varepsilon_{c2u}$
modulus of elasticity E_{cm}, mean value of axial tensil strength f_{ctm}

reinforcem.	f_{yk} MN/m²	f_{tk} MN/m²	ε_{su} %	E_s MN/m²
BSt 500 (A)	500.0	525.0	25.00	200000.0

design yield strength $f_{yd} = f_{yk} / \gamma_s$
design tensile strength $f_{td} = f_{tk} / \gamma_s$
ult. tensile strain ε_{su}, modulus of elasticity E_s