SB-Produksjon

STATICAL CALCULATIONS FOR BCC 250

CONTENT

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PART 1 – BASIC ASSUMPTIONS

1.1 GENERAL

In these calculations certain assumptions has been made about dimensions and qualities in the precast concrete elements that may not always be the case. Therefore the following calculations of anchorage of the units and the resulting reinforcement must be considered as an example to illustrate the calculation model.

In beams it must always be checked that the forces from the anchorage reinforcement can be transferred to the beam's main reinforcement. The recommended shear reinforcement (stirrups) includes all necessary stirrups in the beam end; i.e. the normal shear reinforcement in beam ends and an addition due to the cantilever action of the beam unit from the beam.

The information found here and in the memos assumes that the design of the elements and the use of the units in structural elements are carried out under the supervision of a structural engineer with knowledge about the flow of forces.

1.2 STANDARDS

The calculations are carried out according to:
Eurocode 2: Design of concrete structures. Part 1-1. General rules and rules for buildings.
Eurocode 3: Design of steel structures. Part 1-1: General rules and rules for buildings.
Eurocode 3: Design of steel structures. Part 1-8: Design of joints.
EN 10080: Steel for the reinforcement of concrete. Weldable reinforcing steel. General.
For all NPDs in the Eurocodes the recommended values are used.
These NDPs are: In EC 2: γ_c; γ_s; α_{cc}; α_{ct}; k_1; k_2 and $Ø_{m,min}$. In EC 3: γ_{M0}; γ_{M1} and γ_{M2}.

1.3 LOADS

Vertical ultimate limit state load = F_V = 250 kN
Horizontal ultimate limit state load = $F_H = 0{,}3 \times F_V$ = 75 kN
The design is carried out with a horizontal force of 30 % of the vertical force.
Recommended capacity for transfer of active forces (as wind loads) may be assumed to be 20 % of smallest vertical load present at the same time.

1.4 QUALITIES

Concrete grade C45/55: $f_{cd} = \alpha_{cc} \cdot f_{ck}/\gamma_c = 1{,}0 \cdot 45/1{,}5 = 30{,}0$ MPa
$f_{ctd} = \alpha_{ct} \cdot f_{ctk,0,05}/\gamma_c = 1{,}0 \cdot 2{,}70/1{,}5 = 1{,}80$ MPa
$f_{bd} = 2{,}25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} = 2{,}25 \cdot 1{,}0 \cdot 1{,}0 \cdot 1{,}80 = 4{,}05$ MPa

Reinforcement B500C: $f_{yd} = f_{yk}/\gamma_s = 500/1{,}15 = 435$ MPa

Tension in threaded bars: 8.8 quality steel: $f_{yd} = f_y/\gamma_{M2} = 640/1{,}25 = 512$ MPa

Nominal diameter (mm)	M10	M12	M16	M20	M24	M30	M33	M36
Equivalent diameter (mm)	8,6	10,4	14,1	17,7	21,2	26,7	29,7	32,6
Stress area (mm^2)	58	84	157	245	353	561	694	835

Structural steel S355:
Tension and compression:
$f_{yd} = f_y/\gamma_{M0} = 355/1{,}00 = 355$ MPa $< f_u/\gamma_{M2} = 510/1{,}25 = 408$ MPa
Shear: $f_{sd} = f_y/(\gamma_{M0} \cdot \sqrt{3}) = 355/(1{,}00 \cdot \sqrt{3}) = 205$ MPa
Welds: $f_{vw.d} = \dfrac{f_u}{\gamma_{M2}\sqrt{3}} \times \dfrac{1}{\beta_w} = \dfrac{510}{1{,}25 \times \sqrt{3}} \times \dfrac{1}{0{,}9} = 262$ MPa

PART 2 – ANCHORAGE OF THE UNITS

2.1 BEAM UNIT - EQUILIBRIUM

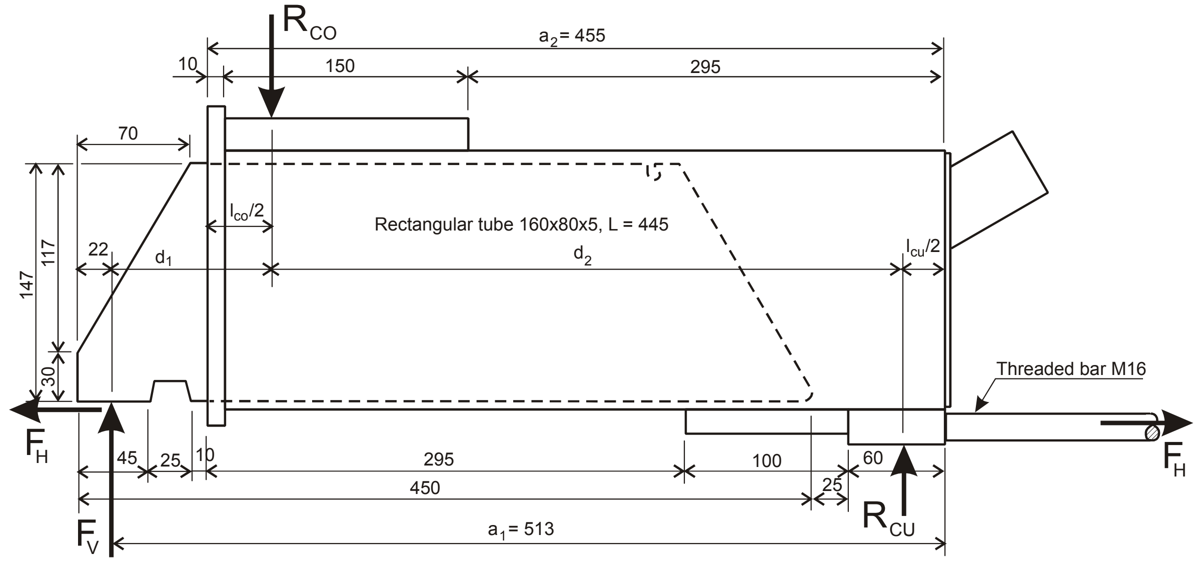

Figure 1. Designations and assumed dimensions.

Neglecting the moment created by the small vertical shift in the transfer of F_H.
$R_{CU} = F_V \times d_1/d_2$
$R_{CO} = F_V + R_{CU}$
$l_{CU} = R_{CU} / (f_{cd} \times b)$ (b is the width of the beam unit)
$l_{CO}/2$ is decided by the location of the front reinforcement. Assume 4-Ø12:
$l_{CO}/2 = 10+(8\times14+3\times8)/2 = 78$ mm
$d_1 = a_1 - a_2 + l_{CO}/2$
$d_2 = a_2 - l_{CO}/2 - l_{CU}/2$

Use a spread sheet. Assume d_1/d_2, calculate resulting d_1/d_2. Change assumed value until they are equal. (Spread sheet ”Equilibrium-BCC250-beam-unit.exc”.)

Equilibrium of BCC 250 beam unit:					
			Concrete grade = C	45	/55
			Material coefficient for concrete =	1,5	
Geometry: a_1 =	51 3	mm	f_{cd} =	30,0	MPa
a_2 =	45 5	mm			
Width of the beam unit =	80	mm	F_V =	250	kN
$l_{CO}/2$ =	78	mm			
			Assumed d_1 / d_2 =	0,3807731	
R_{CU} =	95	kN	d_1 =	136	mm
R_{CO} =	34 5	kN	d_2 =	357	mm
l_{CU} =	40	mm			
			Calculated d_1 / d_2 =	0,3807731	

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2.2 BEAM UNIT – ANCHORAGE IN FRONT

2.2.1 Check of capacity

$A_{s,reqd} = R_{CO}/f_{yd,reinf} = 345/0{,}435 = 793\ mm^2$
Assumed 4-Ø12 = 2×4×113 = 904 mm^2 – ok

2.2.2 Anchorage

In order to achieve equilibrium in the joint where the reinforcement bringing R_{CO} down is anchored, the same amount of reinforcement must be supplied horizontally in order to carry a 45° compression diagonal (strut and tie model). Select 4-Ø12 U-shaped stirrups.

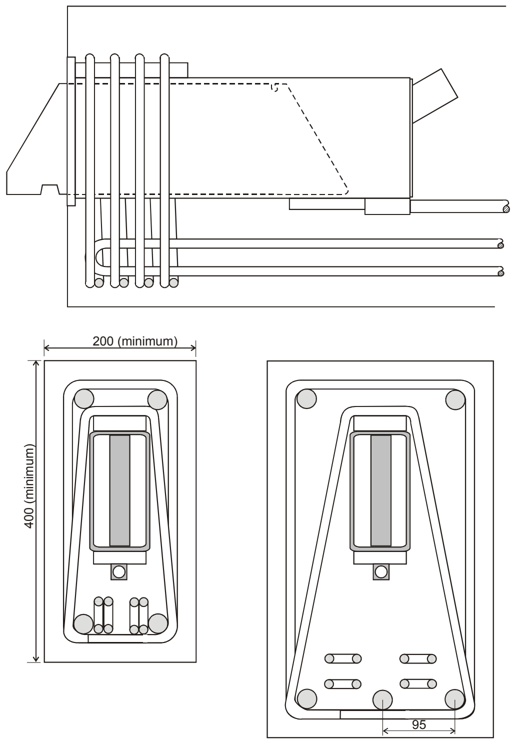

Figure 2. Anchorage of front reinforcement.

As illustrated in the left section in figure 2 it will be congested if the beam has minimum dimensions. This must be taken into account when the anchorage length of the horizontal U-shaped bars is calculated. (The main reinforcement of the beam is not shown in the side view.)

Select to check the anchorage for a situation as illustrated in the right hand section in figure 2.

According to EC2 clause 8.4.4:
$l_{bd} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,reqd} \geq l_{b,min}$

$$l_{b,reqd} = \frac{Ø}{4} \cdot \frac{\sigma_{sd}}{f_{bd}} = \frac{12}{4} \cdot \frac{345/0{,}904}{4{,}05} = 283\ mm$$

$l_{b,min} = \max(0{,}3 \cdot l_{b.reqd};\ 10 \cdot Ø;\ 100\ mm) = 120\ mm$

Anchorage in the node:
Lower bars: c = 25+15+25 = 65 mm
Upper bars: c = 65+15+15+15 = 110 mm
U-shaped anchorage: $c_d = c = 65\ mm > 3 \cdot Ø = 36\ mm$; i.e. $\alpha_1 = 0{,}7$
Concrete cover: $\alpha_2 = 1-0{,}15 \cdot (c_d-3 \cdot Ø)/Ø = 1-0{,}15 \cdot (50-3 \cdot 12)/12 = 0{,}825$
Confinement by reinforcement:
Assume transverse reinforcement in the anchorage zone to be 4-Ø12 bars (see figure 5):

$$\alpha_3 = 1-K \cdot \lambda = 1-0{,}05 \cdot \frac{113 \cdot 4-0{,}25 \cdot 113}{113} = 0{,}813$$

Confinement by welded transverse reinforcement: $\alpha_4 = 1{,}0$
Confinement by transverse pressure: $\alpha_5 = 1{,}0$

$\alpha_2 \cdot \alpha_3 \cdot \alpha_5 = 0{,}825 \cdot 0{,}813 \cdot 1{,}0 = 0{,}671 < 0{,}7$ i.e. use 0,7.

$l_{bd} = 0{,}7 \cdot 0{,}7 \cdot 1{,}0 \cdot 283 = 139\ mm$

No contribution to the anchorage is required from the main reinforcement in the beam.

Anchorage for lap splicing with the main reinforcement in the beam:
Straight bar: $\alpha_1 = 1{,}0$
Concrete cover: $c_d = \min(a/2;\ c_1;\ c) = \min(48/2;\ 25+15+25+15;\ 25+15) = 24$ mm
$\alpha_2 = 1-0{,}15\cdot(c_d-Ø)/Ø = 1-0{,}15\cdot(24-12)/12 = 0{,}85$
Confinement by reinforcement:
Assume transverse reinforcement Ø12-c/c 80 mm (see figure 5):

$$\alpha_3 = 1-K\cdot\lambda = 1-0{,}05\cdot\frac{113\cdot\frac{210}{80}-0{,}25\cdot 113}{113} = 0{,}881$$

Confinement by welded transverse reinforcement: $\alpha_4 = 1{,}0$
Confinement by transverse pressure: $\alpha_5 = 1{,}0$

$\alpha_2\cdot\alpha_3\cdot\alpha_5 = 0{,}85\cdot 0{,}881\cdot 1{,}0 = 0{,}75 > 0{,}7$

$l_{bd} = 1{,}0\cdot 0{,}75\cdot 1{,}0\cdot 283 = 212$ mm

Minimum total horizontal length of the U-shaped bars = 139+212 = 351 mm

It must always be checked that the beam's main reinforcement has sufficient anchorage at the end of the horizontal part of the front anchorage. This will probably lead to much greater lengths for the horizontal part of the front anchorage than calculated here.

It is recommended to make sure the anchorage lengths always are ample in order to avoid anchorage failures, as anchorage failures normally will be brittle failures.

Compressive strut:
According to EC2, clause 6.5.2 and 6.5.4.c):
□ = 1-45/250 = 0,82
$\sigma_{Rd,max} = k_3\cdot□\cdot f_{cd} = 0{,}75\cdot 0{,}82\cdot 30 = 18{,}5$ MPa
$F_{cd} = R_{CO}\cdot\sqrt{2} = 345\cdot\sqrt{2} = 488$ kN
Required concrete area = $488\cdot 10^3/18{,}5 = 26373$ mm^2
Height of compressive strut is at least equal to the anchorage length·√2
= 139·√2 = 197 mm
Required beam width = (26373/197)+80 = 214 mm
← width of BCC unit

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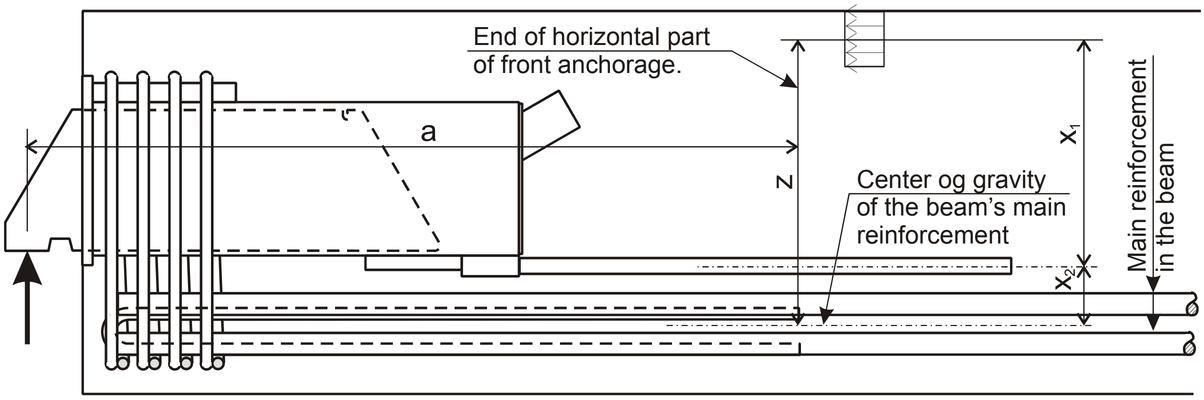

Figure 3. Anchorage of the beam's main reinfocement.

The force is approximately [$F_V \times \frac{a}{z} + \frac{F_V}{2} + F_H \frac{x_1}{z}$]. Additional force in the beam's top reinforcement is about [$F_H \cdot \frac{x_2}{z}$], but this force leads to a reduction in the compressive stresses and may be neglected. If there is a large distance between the horizontal part of the front anchorage and the beam's main reinforcement, the length of the horizontal part of the front anchorage must be increased with this distance. (This distance is approximately zero in the figure.)

2.3 BEAM UNIT – ANCHORAGE OF HORIZONTAL FORCES
2.3.1 Check of capacity
F_H = 75 kN (see clause 1.3)
$A_{s,reqd}$ = 75/0,512 = 146 mm^2 (see clause 1.4)
M16 threaded bar (157 mm^2) is assumed - ok

2.3.2 Anchorage length
$l_{bd} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,reqd} \geq l_{b,min}$
No information exists on the bond characteristics of threaded bars. Assume η_1 to be 0,7.

f_{bd} = 2,25·0,7·1,0·1,80 = 2,84 MPa; $l_{b,reqd} = \frac{Ø}{4} \cdot \frac{\sigma_{sd}}{f_{bd}} = \frac{16}{4} \cdot \frac{75/0,157}{2,84} = 673$ mm

$l_{b,min}$ = max(0,3·$l_{b.reqd}$; 10·Ø; 100 mm) = 202 mm
Straight bar: α_1 = 1,0
Concrete cover: c_d = min(a/2; c_1; c) = min(50/2; 100; 100) = 25 mm
α_2 = 1−0,15·(c_d−Ø)/Ø = 1−0,15·(25-16)/16 = 0,916
Confinement by reinforcement: Assume transverse reinforcement Ø12-c/c150 mm:

$$\alpha_3 = 1 - K \cdot \lambda = 1 - 0{,}05 \cdot \frac{113 \cdot \frac{300}{150} - 0{,}25 \cdot 113}{113} = 0{,}91$$

Confinement by welded transverse reinforcement: α_4 = 1,0
Confinement by transverse pressure: α_5 = 1,0

l_{bd} = 1,0·0,916·0,91·1,0·1,0·673 = 560 mm

Recommended length 666 mm (18 pieces from a 12 m bar).

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2.4 COLUMN UNIT – ANCHORAGE IN THE COLUMN

2.4.1 Check of capacity

$M = F_H \times (15+22/2)+F_V \times [(70/2)-35] =$
$= 250 \times (0,3 \times 26+0) = 1950$ kNmm
$S = 1950/245 = 7,96$ kN
$A_s = 7,96/0,512 = 16$ mm^2
M10 at the top is ok (see table in clause 1.4)

$F_H - S = 0,3 \times 250 - 8 = 67$ kN
$A_s = 67/0,512 = 131$ mm^2
M16 in the bottom is ok

2.4.2 Anchorage of the bolts

See separate calculations: "Design of anchorages used in the BCC units."
See also Memo 27.

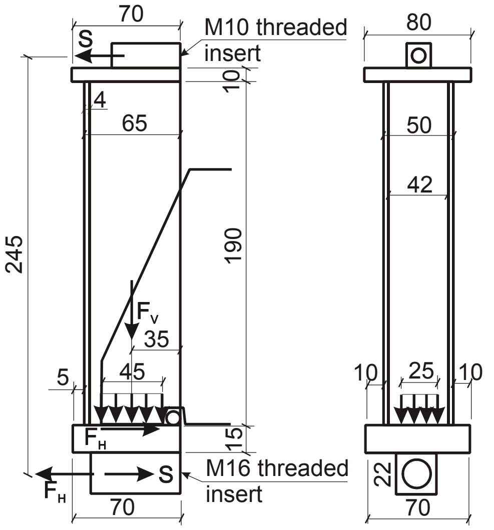

Figure 4. Assumed dimensions. Equilibrium of the column unit.

PART 3 – REINFORCEMENT

3.1 BEAM – REINFORCEMENT IN THE REAR

$A_{s,reqd} = R_{CU}/f_{sd} = 95/0,435 = 218$ mm^2,
corresponding to $218/(2 \times 113) = 0,96$; i.e. 1 – Ø 12 stirrup.

3.2 BEAM – SHEAR STIRRUPS

Use a strut-and-tie model with compressive diagonals at 45°.
The shear force within the central part of the beam unit is assumed to be $R_{CO} = 345$ kN.
Assume z = 300 mm and stirrup diameter Ø12.
$s = A_{sw} \cdot z \cdot f_{ywd} \cdot \cot \theta / V$
$s = 2 \times 113 \times 435 \times 300 \times \cot 45^{o} / 345 \times 10^3 = 85$ mm
Select Ø12 c/c 80 mm. This reinforcement should be brought approximately 200 mm past the end of the beam unit in order to absorb any splitting effects from the anchorage of the threaded bar.

3.3 BEAM – CHECK OF SHEAR COMPRESSION

Assume beam width of 200 mm and beam depth of 450 mm. (See also clause 2.2.) This means that $z_{min} \approx 450–50–100 = 300$ mm with one layer of reinforcement in the top of the beam and three layers in the bottom.
The shear reinforcement is calculated based on 45° compressive field. The check of shear compression will be carried out under the same assumption:
$V_{Rd,max} = \alpha_{CW} \cdot b_w \cdot z \cdot \upsilon_1 \cdot f_{cd}/(\cot \theta + \tan \theta)$
$V_{Rd,max} = \{1,0 \cdot (200-80) \cdot 300 \cdot 0,6 \cdot [1-(45/250)] \cdot 30,0/(\cot 45^o + \tan 45^o)\} \cdot 10^{-3}$
$V_{ccd} = 531$ MPa > 345 MPa - OK

3.4 COLUMN – SPLITTING STRESS

Splitting stress under the column unit according to EN 1992-1-1, clause 6.5.3:

$$T = \frac{1}{4} \cdot \frac{300-70}{300} \cdot F_V = 0,192 \cdot 250 = 48 \text{ kN}$$

$A_S = 48/0,435 = 110$ mm
2-Ø10 stirrups ($2 \times 78,5$ mm^2) are sufficient.

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3.5 CONCLUSION – REINFORCEMENT IN THE BEAM

For show the beam end reinforcement more clearly, the beam's main reinforcement is not show in the side view. Between the shown stirrups in each end of the beam a normal calculation of the shear reinforcement must be carried out. The main reinforcement in the beam must of course also be calculated.

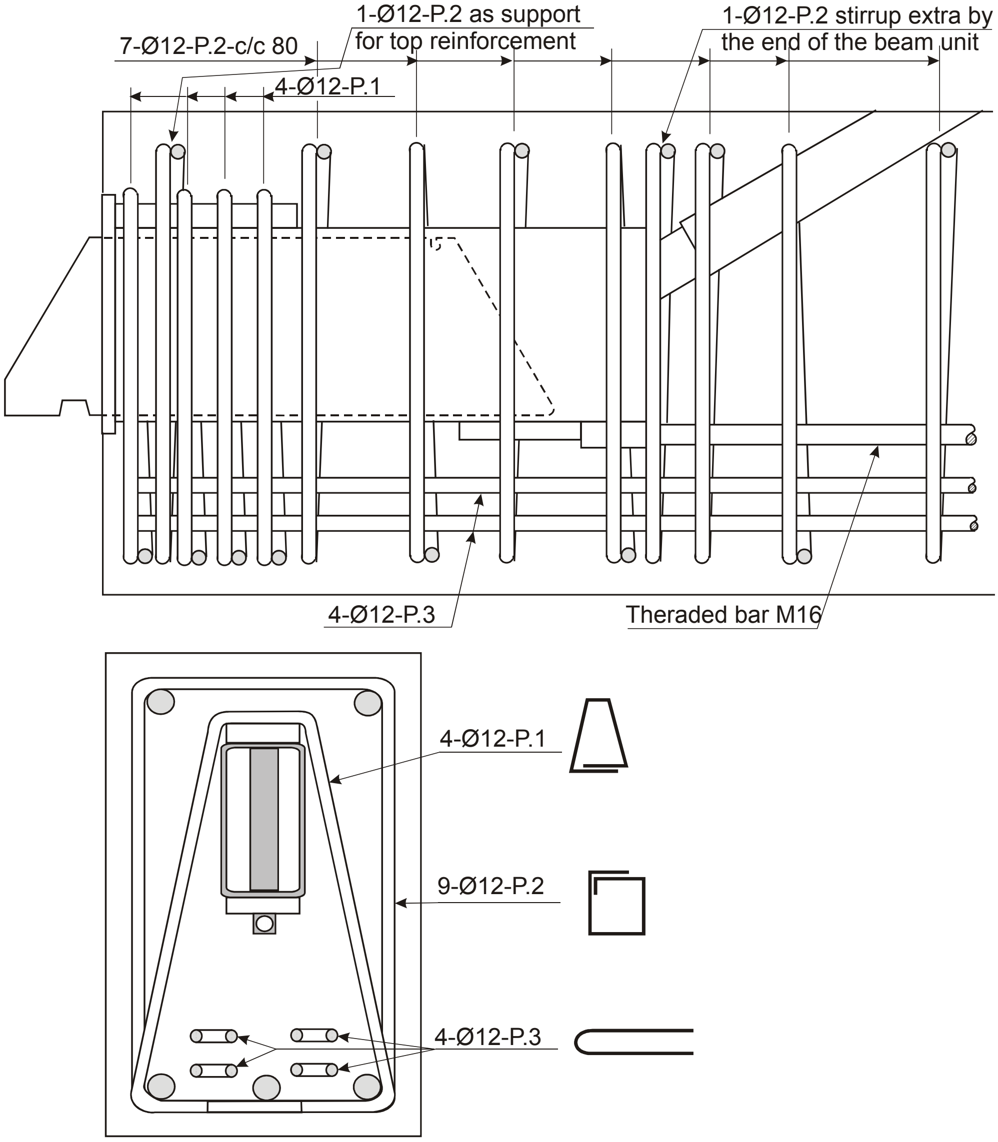

Figure 5. Reinforcement in the beam end.

At the end of the horizontal part of the front anchorage (P.3) it must be checked that the beam's main reinforcement has sufficient anchorage. See clause 2.2.2.

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PART 4 – DESIGN OF STEEL UNITS

4.1 EQUILIBRIUM OF KNIFE

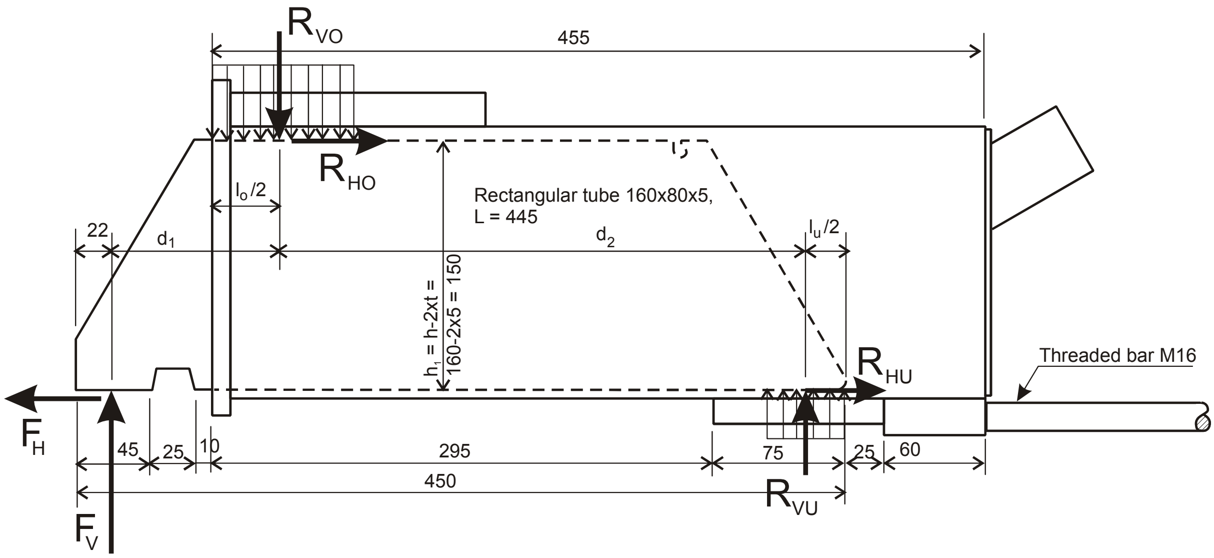

Figure 6. Equilibrium of knife.

$F_V \times d_1 + F_H \times h_1 = R_{VU} \times d_2 + R_{HU} \times h_1$, dvs. $F_V \times d_1 + F_H \times 150 = R_{VU} \times d_2 + R_{HU} \times 150$
$F_V + R_{VU} = R_{VO}$; $F_H = R_{HO} + R_{HU}$
For R_{VU} is f_{yd} reduced with about 25% to 266 MPa due to the rounding of the tip of the knife.
$l_o = R_{VO} / (f_{yd} \times t_k) = R_{VO} / (0,355 \times 25) = 0,1127 \times R_{VO}$
$l_u = R_{VU} / (f_{yd} \times t_k) = R_{VU} / (0,355 \times 0,75 \times 25) = 0,1502 \times R_{VU}$
$d_1 = (45-22) + 25 + 10 + l_o/2 = 58 + l_o/2$
$d_2 = (295+75) - l_o/2 - l_u/2 = 370 - l_o/2 - l_u/2$
$R_{HU} = 0,3 \times R_{VU}$
$R_{HO} = F_H - R_{HU}$
$F_V \times d_1 + 0,3 \times F_V \times 150 = R_{VU} \times d_2 + 0,3 \times R_{VU} \times 150$
$F_V \times (d_1+45) = R_{VU} \times (d_2+45)$
$R_{VU} = F_V \times (d_1+45) / (d_2+45)$
$R_{VO} = F_V + F_V \times (d_1+45)/(d_2+45) = F_V \times [1 + (d_1+45)/(d_2+45)]$
Use a spread sheet. Assume $(d_1+45)/(d_2+45)$, calculate resulting $(d_1+45)/(d_2+45)$.
Change assumed value until they are the same.
(Spread sheet ”Equilibrium-BCC250-knife.exc”.)

F_V =	250	kN	F_H =	75	kN
Assumed $(d_1+45)/(d_2+45)$ =	0,3109002				
R_{VU} =	78	kN	R_{HU} =	23	kN
R_{VO} =	328	kN	R_{HO} =	52	kN
l_o =	37	mm	d_1 =	76	mm
l_u =	12	mm	d_2 =	346	mm
Calculated $(d_1+45)/(d_2+45)$ =	0,3109002				

4.2 DESIGN OF KNIFE

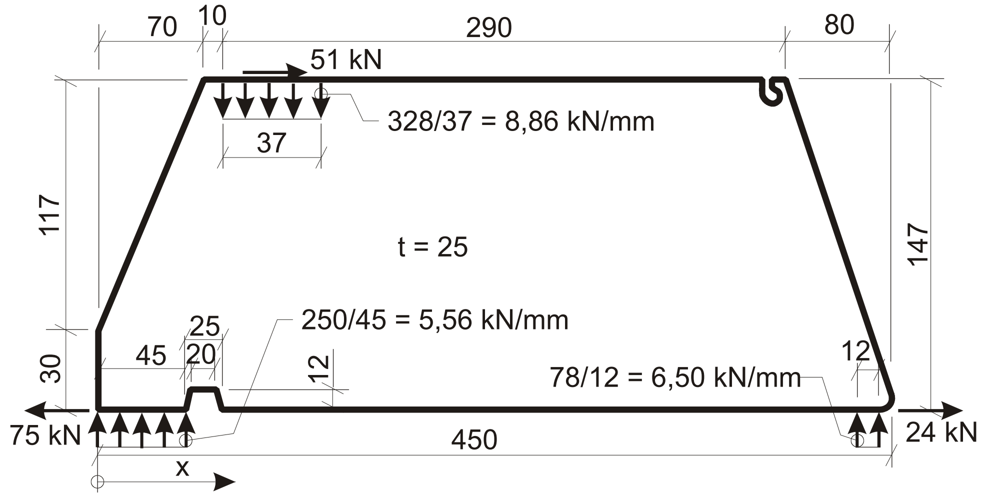

Figure 7. Forces on the knife.

<u>$0 \le x \le 45$:</u>
h = 30+117x/70 = 30+1,67x
V = 5,56x; N = 75x/45 = 1,667x

$$M = 5{,}56\frac{x^2}{2} + \frac{75}{45}x\frac{30+1{,}67x}{2}$$

$$M = 2{,}78x^2 + 0{,}83x(30+1{,}67x)$$

<u>$45 \le x \le 70$:</u>
h = 30+1,67x-12 = 18+1,67x
V = 250; N = 75
M = 250(x–22)+75[12+(18+1,67x)/2]
M = 312x–3925

<u>$70 \le x \le 80$:</u>
h = 147
V = 250; N = 75
M = 250(x–22)+75×147/2
M = 250x+13

<u>$80 \le x \le 117$:</u>
h = 147
V = 250–8,86(x–80); N = 75 –51(x–80)/37
M = $250x+13–8{,}86(x–80)^2/2+(51/37)(x–80)(147/2)$
M = $250x+13–4{,}43(x–80)^2+101(x–80)$

<u>$117 \le x \le 370$:</u>
h = 147
V = 250–328 = –78 kN; N = 75–51 = 24
M = 250x+13–328(x–98,5)+51×147/2 = 36070–78x

$W = 25\times h^2/6 = 4{,}17\times h^2$; $A = 25\times h$
Design plastic shear resistance = $V_{pl,Rd}$:
$V_{pl,Rd} = A_v\cdot(f_y/\sqrt{3})/\gamma_{M0} = h\times 25\times(0{,}355/\sqrt{3})/1{,}00 = 5{,}12\times h$;
For $V/V_{pl,Rd} \le 0{,}5$ the effect of shear on the moment resistance may be neglected.
$\sigma_M = N/A + M/W$

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x (mm)	h (mm)	V (kN)	N (kN)	M (kNmm)	$V_{pl,Rd}$ (kN)	A (mm^2)	$V/V_{pl,Rd}$	W (mm^3)	σ_M (MPa)
0	30	0	0	0	154	750	0,00	3750	0
22	67	122	37	2563	342	1669	0,36	18559	160
45	105	250	75	9552	538	2629	0,46	46069	236
45	93	250	75	10115	477	2329	0,52	36154	312
55	110	250	75	13235	562	2746	0,44	50279	291
65	127	250	75	16355	648	3164	0,39	66729	269
70	135	250	75	17915	691	3373	0,36	75825	259
70	147	250	75	17513	753	3675	0,33	90038	215
75	147	250	75	18763	753	3675	0,33	90038	229
80	147	250	75	20013	753	3675	0,33	90038	243
80	147	250	75	20013	753	3675	0,33	90038	243
100	147	73	47	25261	753	3675	0,10	90038	293
110	147	-16	34	26556	753	3675	-0,02	90038	304
117	147	-78	24	26935	753	3675	-0,10	90038	306
117	147	-78	24	26944	753	3675	-0,10	90038	306
150	147	-78	24	24370	753	3675	-0,10	90038	277
200	147	-78	24	20470	753	3675	-0,10	90038	234
370	147	-78	24	7210	753	3675	-0,10	90038	87
							< 0,5 - ok		< 355 - ok

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4.3 BEAM UNIT – TRANSVERSE BENDING OF TOP FLANGE OF RECTANGULAR TUBE

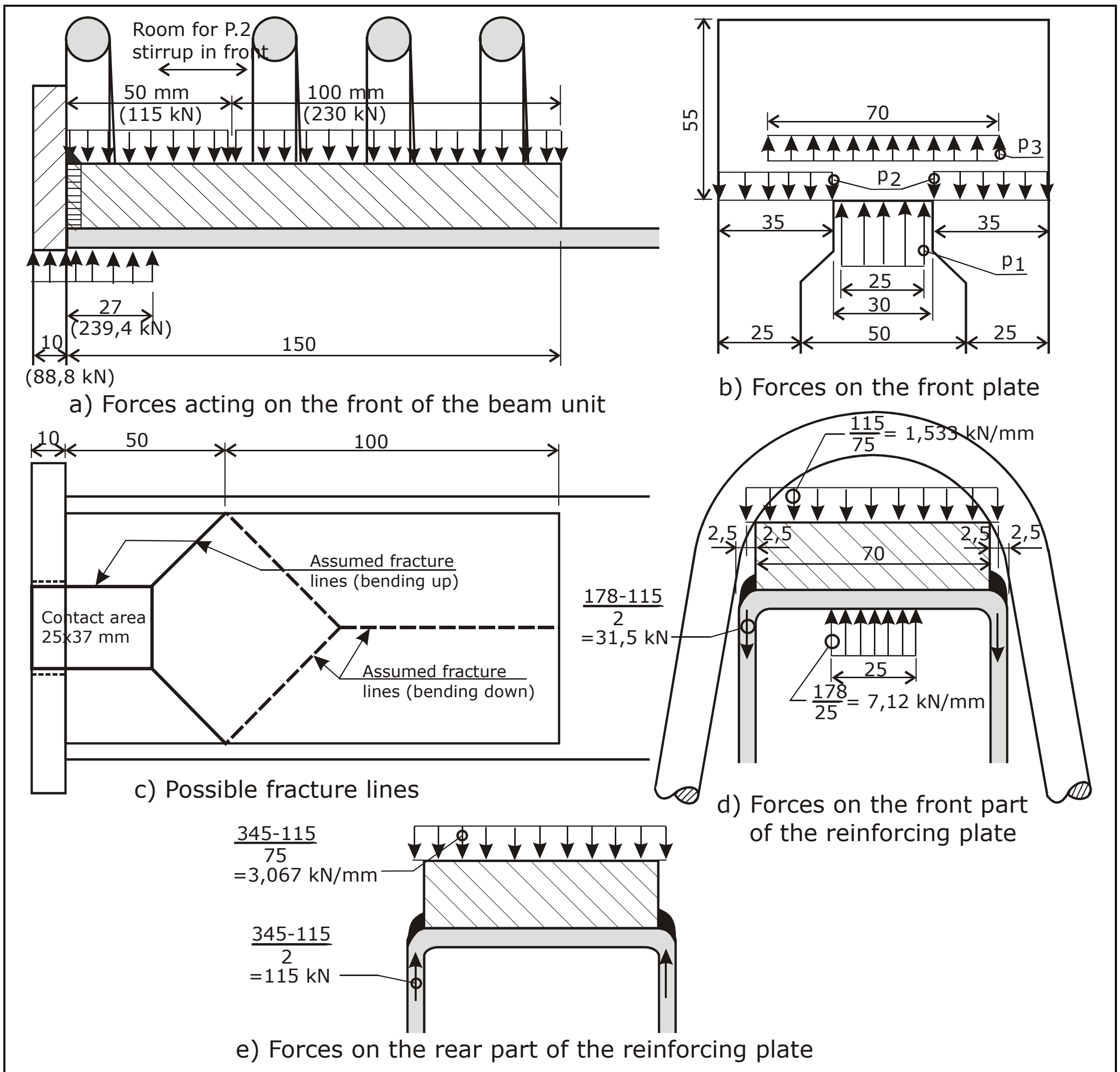

Figure 8. Reinforcing of top flange.

The knife will come to rest partly against the front plate and partly against the rectangular tube (figure 8a).

The front plate with a thickness of 10 mm can absorb (328/37)·10 = 88,6 kN of R_{VO}. The remaining 328-88,6 = 239,4 kN will be carried by 37-10 = 27 mm of the rectangular steel tube (based on l_o in spread sheet in clause 4.1). Some of this will be transferred back to the front plate through the weld between the front plate and the reinforcing plate. Considering the upper 55 mm of the front plate as a beam, the forces on the front plate will be as shown in figure 8b.

Based on the stress capacity of the smallest section of the front plate:

$p_1 = 0{,}355 \cdot 10 = 3{,}55$ kN/mm

Contact area between the knife and the front plate:

$p_2 = 10 \cdot 25 \cdot 0{,}355 = 88{,}8$ kN

/35 = 2,536 kN/mm

Remaining capacity of the front plate:
$p_3 = 2{,}536 \cdot 35 \cdot 2 - 3{,}55 \cdot 25 = 88{,}8$ kN
$/70 = 1{,}268$ kN/mm
Moment at the center line of the "beam":

$$M = 2{,}536 \cdot 35 \cdot \frac{100-35}{2} - 3{,}55 \cdot \left(\frac{25}{2}\right)^2 \cdot \frac{1}{2} - 1{,}268 \cdot \left(\frac{70}{2}\right)^2 \cdot \frac{1}{2} = 1831 kNmm$$

$W_{req'd} = 1831/0{,}355 = 5157$ mm^3

$W_{el} = 10 \cdot 55^2/6 = 5042$ mm^3
$W_{pl} = 10 \cdot 55^2/4 = 7563$ mm^3 – OK

Required weld size between front plate and reinforcing plate = 1,268/0,262 = 4,8 mm
Select 5 mm throat value.

The forces must be transferred from the front plate to the tube through welds on the vertical sides of the tube:
Required throat value = $88{,}8 \cdot 10^3/[262 \cdot (160-10)] = 2{,}3$ mm
Select 4 mm throat value.
The slight moment created in the front plate by the distance between the welds and the center of gravity of the "beam's" support reactions are taken by the front plate in diaphragm action.

Conservatively assume that only 150 kN is carried by the front plate, instead of $p_1+p_3 = 3{,}55 \cdot 25+88{,}8 = 177{,}5$ kN, as calculated here.
The remaining load, R_{VO}-150 = 328-150 = 178 kN, must be carried by the front part of the reinforcing plate. As a simplified model it is assumed that this is carried by 50 mm of the reinforcing plate as a one-way slab supported on the tube walls. Possible fracture lines are shown in figure 8c.

This force is counteracted by the uniformly distributed reaction from the concrete on the reinforcing plate. Since only 1/3 of the reinforcing plate is considered, the force must be 1/3 of R_{CO} = 345/3 = 115 kN. Forces are shown in figure 8d.
Maximum moment:

$$M = 31{,}5 \cdot \frac{75}{2} - 7{,}12 \cdot \left(\frac{25}{2}\right)^2 \cdot \frac{1}{2} + 1{,}533 \cdot \left(\frac{75}{2}\right)^2 \cdot \frac{1}{2} = 1703 kNmm$$

$W_{req'd} = 1703/0{,}355 = 4797$ mm^3

$W_{el} = 50 \cdot (20^2+5^2)/6 = 3542$ mm^3
$W_{pl} = 50 \cdot (20^2+5^2)/4 = 5313$ mm^3 – OK

Weld between reinforcing plate and tube = $31{,}5 \cdot 10^3/(262 \cdot 50) = 2{,}4$ mm
Select 4 mm throat value.

The rear of the reinforcing plate must carry the compressive stress caused by R_{CO}. If the grouting of the tube is neglected (see figure 8e):

Maximum moment:

$$M = 115 \cdot \frac{75}{2} - 3{,}067 \cdot \left(\frac{75}{2}\right)^2 \cdot \frac{1}{2} = 2156 kNmm$$

$W_{req'd}$ = 2156/0,355 = 6077 mm^3

W_{el} = 100·(20^2+5^2)/6 = 7084 mm^3
W_{pl} = 100·(20^2+5^2)/4 = 10626 mm^3 ⊢ OK

The welds are executed with reserve capacity to compensate for possible deviating force distribution.

4.4 BEAM UNIT – TRANSVERSE BENDING OF BOTTOM FLANGE OF RECTANGULAR TUBE

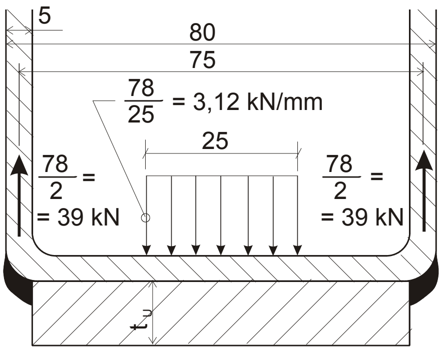

Figure 9. Reinforcing of bottom flange.

M = 39×75/2 – 3,12×(25/2)^2/2
M = 1219 kNmm
Req'd. W = 1219/0,355 = 3433 mm^3

Try 100 mm width of plate (in the longitudinal direction of the beam unit):
100×(5^2+t_u^2)/6 = 3433
t_u = 13,4 mm
Select t_u = 15 mm

The weld:
Req'd. a = 39×10^3/(262×100) = 1,5 mm
Select weld with throat measure a = 4 mm.

4.5 COLUMN UNIT - EQUILIBRIUM

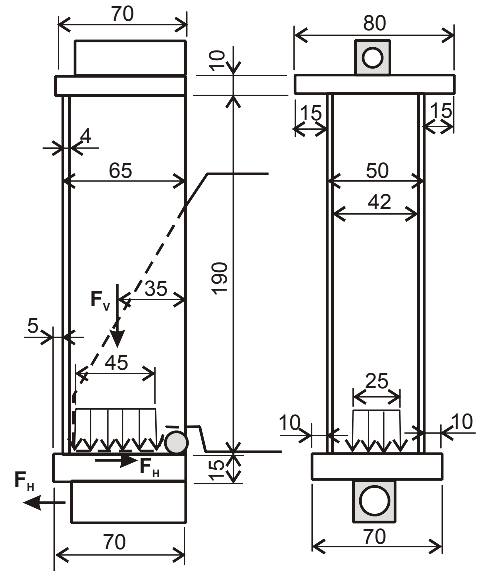

Figure 10. Assumed geometry of column unit.

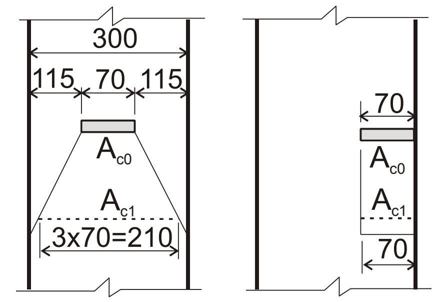

Figure 11. Load distribution in column.

Assume bottom plate to be 70×70 mm.
A_{c0} = 70×70 = 4900 mm^2

Partially loaded area:
Assume smallest relevant column dimension to be 300×300 mm.
A_{c1} = 210×70 = 14700 mm^2

$$f'_{cd} = 30{,}0 \cdot \sqrt{\frac{14700}{4900}} = 51{,}9 < 3{,}0 \cdot 30{,}0 = 90 \text{ MPa}$$

Assume top plate to be 70×80 mm.
Assume concrete compressive stress to be the same on bottom plate and cantilevering part of top plate:

Area of bottom plate = 70·70 = 4900 mm^2
Area of top plate = 2·70·15+50·5 = <u>2350 mm^2</u>
= 7250 mm^2
Concrete compressive stress = 250/7,25 = 34,5 MPa

$$y = \frac{2\times 15\times 70\times \frac{70}{2} + 5\times 50\times (65+\frac{5}{2})}{2\times 15\times 70 + 5\times 50} = 38 \text{ mm}$$

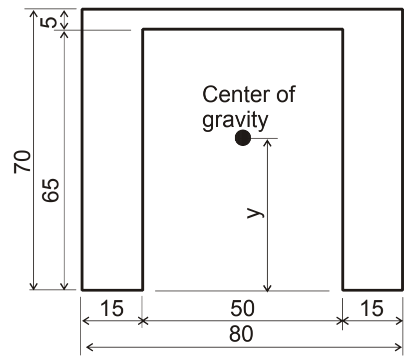

Figure 12. Cantilevering part of top plate.

Bottom plate carries 4900·34,5 = 169 kN
Top plate carries 2350·34,5 = 81 kN

4.6 COLUMN UNIT- BEDNING OF PLATES

4.6.1 Bottom plate

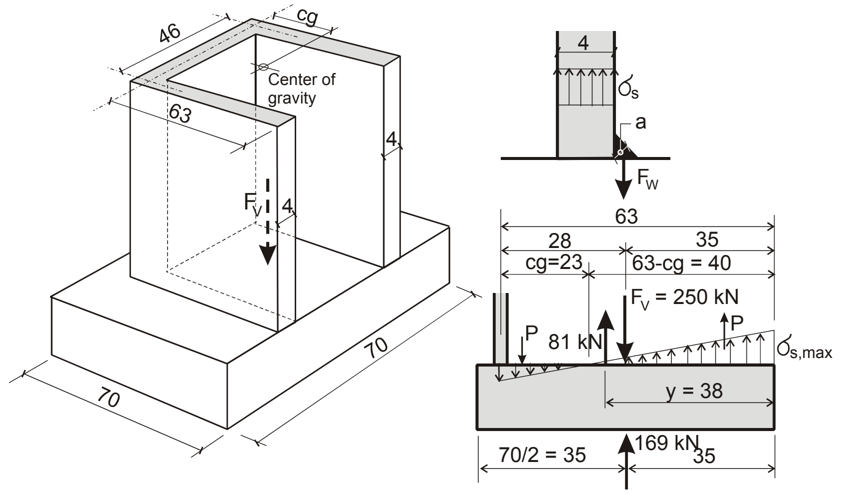

Figure 13. Equilibrium of bottom plate.

$cg = 63^2/(2\times63+46)$
$cg = 23$ mm

Equilibrium: $81\times(38-35) = P\times\{[28-(23/3)]+[35-(40/3)]\}$

$P = 5{,}8$ kN

$40\times\sigma_{s,max}\times(1/2)\times2\times4 = 5{,}8\times10^3$

$\sigma_{s,max} = 36$ MPa << 355 MPa – OK

$F_{w,Ed,max} = 36\times4 = 145$ N/mm

$a = 145/262 = 0{,}6$ mm; Select $a = 4$ mm

Design bottom plate for transverse bending for the average compressive stress (see figure 13).
Consider a section 1 mm wide:
$\sigma_s = \sigma_{s,max}/2 = 36/2 = 18$ MPa
Force in the weld in the section = $18\times4 = 72$ N

$0 \le x \le 8$: $M = M_1 = 34{,}5\cdot x^2/2$
$8 \le x \le 22{,}5$: $M = M_1+72\cdot(x-8) = M_1+M_2$
$22{,}5 \le x \le 35$:
$M = M_1+M_2-222\cdot(x-22{,}5)/2 = M_1+M_2-M_3$

$W = M/355 = t^2/6$; dvs. $t = \sqrt{(M/59{,}17)}$

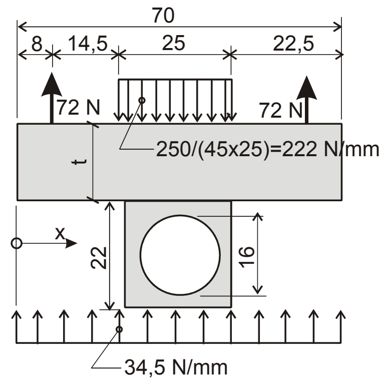

Figure 14. Forces on the bottom plate.

Use a spread sheet: "BCC250-col-unit-bot-pl.exc".

x (mm)	M_1 (Nmm)	M_2 (Nmm)	M_3 (Nmm)	M (Nmm)	t (mm)
0,0	0			0	0
12,0	2484	288		2772	7
22,5	8733	1044	0	9777	13
25,0	10781	1224	694	11312	14
27,5	13045	1404	2775	11674	14
30,0	15525	1584	6244	10865	14
32,5	18220	1764	11100	8884	12
35,0	21131	1944	17344	5732	10

Select t = 15 mm

4.6.2 Top plate

$M = 34{,}5 \times 13^2/2 = 2915$ Nmm

$W_{reqd} = 2915/355 = 8{,}21$ mm^3

$t = \sqrt{(8{,}21 \times 6)} = 7{,}0$ mm
Select t = 10 mm

The weld:
$F_w = 34{,}5 \times 15 = 518$ N/mm

$a = 518/262 = 2{,}0$ mm
Select a = 4 mm

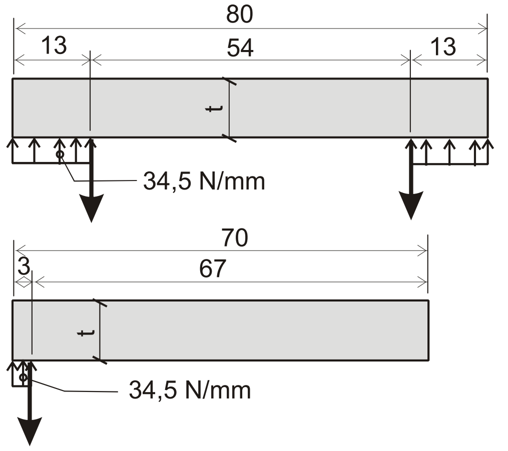

Figure 15. Forces on the top plate.

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4.7 CONCLUSION – STEEL UNITS

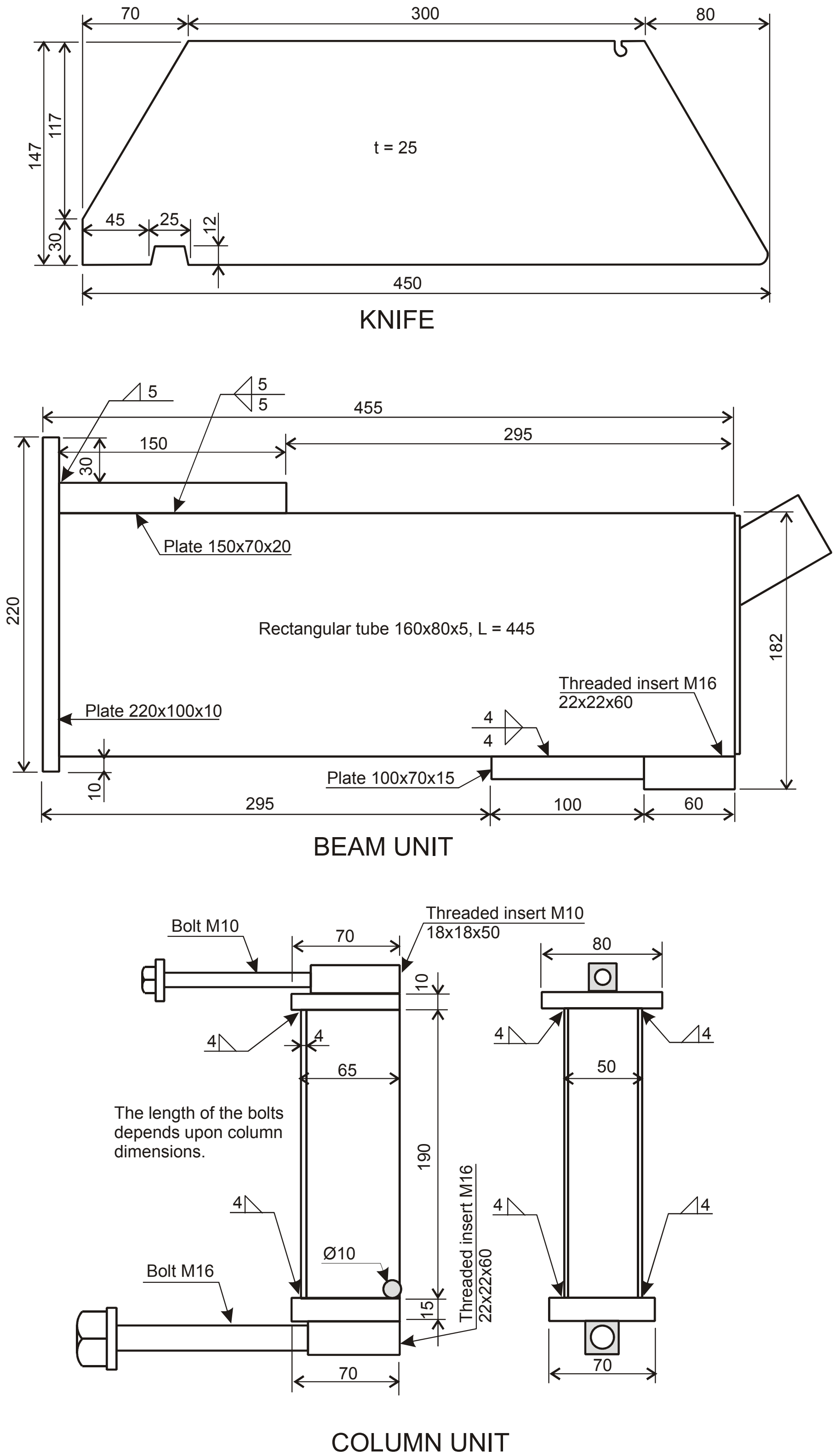

Figure 16. Steel units.