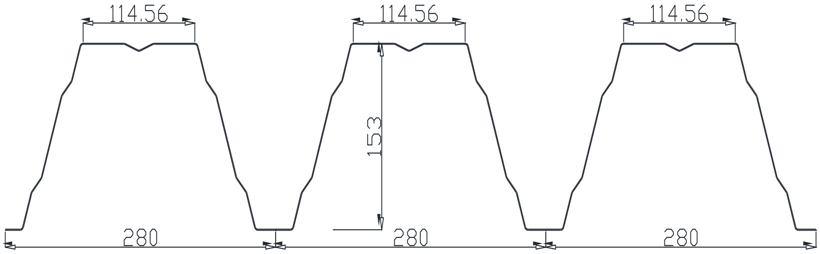
ETSEIB

1. AISI H150

Design of cold-formed steel structural members using the Direct Strength Method (AISI 100 Appendix 1)



Local Buckling (1.2.2.2)

The nominal flexural strength, M_{nl}, for local buckling should be calculated accordingly to:

$$M_{ne} := S_f \cdot F_y$$

$I_{xx} := 1172099.49\,mm^4$ values taken from CUFSM

$z_{cg} := 88.7768\,mm$ values taken from CUFSM

$$S_f := \frac{I_{xx}}{z_{cg}} = 1.32 \times 10^4 \cdot mm^3$$

$$F_y := 327.81 \frac{N}{mm^2}$$

$$M_{ne} := S_f \cdot F_y = 4.328 \times 10^6 \cdot N \cdot mm$$

$$M_{crl} := 10006848\,N \cdot mm = 6.848 \times 10^6 \cdot N \cdot mm$$

$$\lambda_l := \sqrt{\frac{M_{ne}}{M_{crl}}} = 0.795 \qquad \text{(Eq.1.2.2.7)}$$

For $\lambda_l > 0.776$

$$M_{nl} := \left[1 - 0.15\left(\frac{M_{crl}}{M_{ne}}\right)^{0.4}\right]\cdot\left(\frac{M_{crl}}{M_{ne}}\right)^{0.4}\cdot M_{ne} = 4.263\times 10^{6}\cdot N\cdot mm \qquad \text{(Eq.1.2.2.6)}$$

S_f - Gross section modulus referenced to the extreme fiber in the first yield

F_y - Yield stress

M_{crl} - Critical elastic local buckling

Distorsional Buckling (1.2.2.3)

The nominal flexural strength, M_{nd}, for distortional buckling should be calculated accordingly to:

$$M_{crd} := 10005352\,N\cdot mm = 5.352\times 10^{6}\cdot N\cdot mm$$

$$M_y := M_{ne} = 4.328\times 10^{6}\cdot N\cdot mm$$

$$\lambda_d := \sqrt{\frac{M_y}{M_{crd}}} = 0.899 \qquad \text{(Eq.1.2.2.10)}$$

For $\lambda_d > 0.673$

$$M_{nd} := \left[1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right]\cdot\left(\frac{M_{crd}}{M_y}\right)^{0.5}\cdot M_y = 3.635\times 10^{6}\cdot N\cdot mm \qquad \text{(Eq.1.2.2.9)}$$

M_{crd} - Critical elastic local buckling

The ultimate bending moment should be chosen, from the minimum between, the nominal flexural strength for local, distortional buckling:

$$M_{fin} := \min(M_{nd}, M_{nl})$$

$$M_{fin} = 3.635\times 10^{6}\cdot N\cdot mm$$

The ultimate bending moment for the whole sheet, with three webs:

$$M_{Sheet} := 3\cdot M_{fin} = 1.091\times 10^{7}\cdot N\cdot mm$$

$$M_{exp} := 14226750\,N\cdot mm$$

$$P_{\%} := M_{Sheet}\cdot\frac{100}{M_{exp}} = 76.66 \qquad \%$$

Using the member plastic moment for the following calculations:

Local Buckling (1.2.2.2)

The nominal flexural strength, M_{nl}, for local buckling should be calculated accordingly to:

$$M_{pl} := Z_f\cdot F_y$$

$$Z_f := 17886.46\,mm^3$$

$$F_y := 327.81\frac{N}{mm^2}$$

$$M_{pl} := Z_f\cdot F_y = 5.863\times 10^6\cdot N\cdot mm$$

$$M_{crl} := 10006848\,N\cdot mm = 6.848\times 10^6\cdot N\cdot mm$$

$$\lambda_l := \sqrt{\frac{M_{pl}}{M_{crl}}} = 0.925 \qquad \text{(Eq.1.2.2.7)}$$

For $\lambda_l > 0.776$

$$M_{nl} := \left[1 - 0.15\left(\frac{M_{crl}}{M_{pl}}\right)^{0.4}\right]\cdot\left(\frac{M_{crl}}{M_{pl}}\right)^{0.4}\cdot M_{pl} = 5.243\times 10^6\cdot N\cdot mm \qquad \text{(Eq.1.2.2.6)}$$

F_y - Yield stress
M_{crl} - Critical elastic local buckling
M_{pl} - Member plastic moment
Z_f - Plastic section modulus

Distorsional Buckling (1.2.2.3)

The nominal flexural strength, M_{nd}, for distortional buckling should be calculated accordingly to:

$$M_{crd} := 10005352\,N\cdot mm = 5.352\times 10^6\cdot N\cdot mm$$

$M_y := M_{pl} = 5.863\times 10^6 \cdot N\cdot mm$

$$\lambda_d := \sqrt{\frac{M_y}{M_{crd}}} = 1.047 \qquad \text{(Eq.1.2.2.10)}$$

For $\lambda_d > 0.673$

$$M_{nd} := \left[1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right]\cdot\left(\frac{M_{crd}}{M_y}\right)^{0.5}\cdot M_y = 4.424\times 10^6\cdot N\cdot mm \qquad \text{(Eq.1.2.2.9)}$$

M_{crd} - Critical elastic local buckling

M_{pl} - Member plastic moment

Z_f - Plastic section modulus

The ultimate bending moment should be chosen, from the minimum between, the nominal flexural strength for local, distortional buckling:

$M_{fin} := \min(M_{nd}, M_{nl})$

$M_{fin} = 4.424\times 10^6\cdot N\cdot mm$

The ultimate bending moment for the whole sheet, with three webs:

$M_{Sheet} := 3\cdot M_{fin} = 1.327\times 10^7\cdot N\cdot mm$

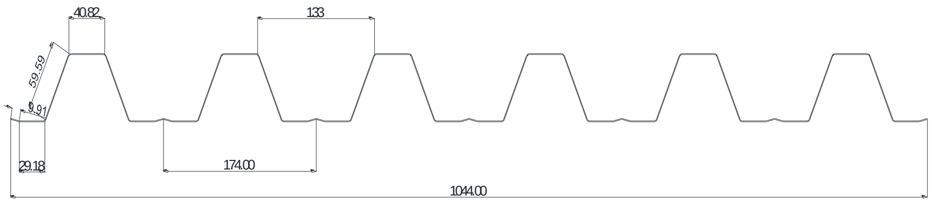
$M_{exp} := 14226750 N\cdot mm$

$$P_{\%} := M_{Sheet}\cdot\frac{100}{M_{exp}} = 93.298\ \%$$

2. AISI H 55

Design of Cold-Formed Steel Structural Members Using the Direct Strength Method (AISI-S100-Appendix 1)

Sheet N2:



Local Buckling (1.2.2.2)

The nominal flexural strength, M_{nl}, for local buckling should be calculated accordingly to:

$M_{ne} := S_f \cdot F_y$

$I_{xx} := 85282 mm^4$ value taken from CUFSM

$z_{cg} := 54 \cdot mm - 22.77 \cdot mm = 31.23 mm$ value taken from CUFSM

$$S_f := \frac{I_{xx}}{z_{cg}} = 2.731 \times 10^3 \cdot mm^3$$

$$F_y := 337.7 \frac{N}{mm^2}$$

$M_{ne} := S_f \cdot F_y = 9.222 \times 10^5 \cdot N \cdot mm$

$M_{crl} := 1000999.23 N \cdot mm = 9.992 \times 10^5 \cdot N \cdot mm$

$$\lambda_l := \sqrt{\frac{M_{ne}}{M_{crl}}} = 0.961 \qquad \text{(Eq. 1.2.2.7)}$$

For $\lambda_l > 0.776$

$$M_{nl} := \left[1 - 0.15\left(\frac{M_{crl}}{M_{ne}}\right)^{0.4}\right]\cdot\left(\frac{M_{crl}}{M_{ne}}\right)^{0.4}\cdot M_{ne} = 8.048\times 10^{5}\cdot N\cdot mm \qquad \text{(Eq. 1.2.2.6)}$$

S_f - Gross section modulus referenced to the extreme fiber in the first yield
F_y - Yield stress
M_{crl} - Critical elastic local buckling

The ultimate bending moment:

$$M_{fin} := M_{nl} = 8.048\times 10^{5}\cdot N\cdot mm$$

$$M_{Sheet} := M_{fin} = 8.048\times 10^{5}\cdot N\cdot mm$$

$M_{exp} := 782666 N\cdot mm$ for one web

$$P_{\%} := M_{Sheet}\cdot\frac{100}{M_{exp}} = 102.823 \quad \%$$

Using the member plastic moment for the following calculations:

Local Buckling (1.2.2.2)

The nominal flexural strength, M_{nl}, for local buckling should be calculated accordingly to:

$$M_{pl} := Z_f\cdot F_y$$

$$Z_f := 3488.32 mm^3$$

$$F_y := 337.7\frac{N}{mm^2}$$

$$M_{pl} := Z_f\cdot F_y = 1.178\times 10^{6}\cdot N\cdot mm$$

$$M_{crl} := 1000999.23 N\cdot mm = 9.992\times 10^{5}\cdot N\cdot mm$$

$$\lambda_l := \sqrt{\frac{M_{pl}}{M_{crl}}} = 1.086 \qquad \text{(Eq. 1.2.2.7)}$$

For $\lambda_l > 0.776$

$$M_{nl} := \left[1 - 0.15\left(\frac{M_{crl}}{M_{pl}}\right)^{0.4}\right]\cdot\left(\frac{M_{crl}}{M_{pl}}\right)^{0.4}\cdot M_{pl} = 9.48\times 10^{5}\cdot N\cdot mm \qquad \text{(Eq. 1.2.2.6)}$$

S_f - Gross section modulus referenced to the extreme fiber in the first yield

F_y - Yield stress

M_{crl} - Critical elastic local buckling

M_{pl} - Member plastic moment

Z_f - Plastic section modulus

The ultimate bending moment:

$$M_{fin} := M_{nl} = 9.48\times 10^{5}\cdot N\cdot mm$$

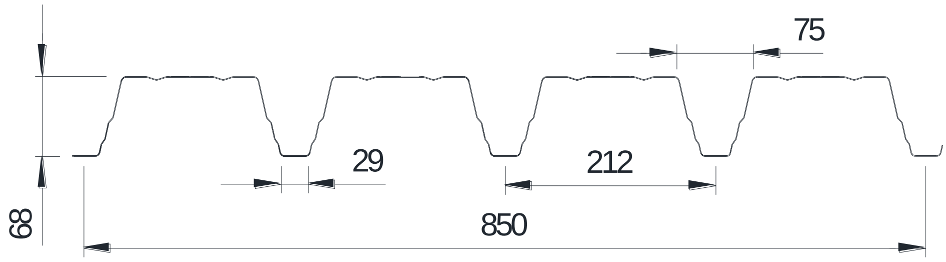
$$M_{Sheet} := M_{fin} = 9.48\times 10^{5}\cdot N\cdot mm$$

$M_{exp} := 782666 N\cdot mm$ for one web

$$P_{\%} := M_{Sheet}\cdot\frac{100}{M_{exp}} = 121.13 \quad \%$$

3. AISI H 68

Design of Cold-Formed Steel Structural Members Using the Direct Strength Method (AISI-S100-Appendix 1)



Local Buckling (1.2.2.2)

The nominal flexural strength, M_{nl}, for local buckling should be calculated accordingly to:

$$M_{ne} := S_f \cdot F_y$$

$I_{xx} := 167464.16\,mm^4$ value taken from CUFSM

$z_{cg} := 44.89\,mm$ value taken from CUFSM

$$S_f := \frac{I_{xx}}{z_{cg}} = 3.731 \times 10^3 \cdot mm^3$$

$$F_y := 280 \cdot \frac{N}{mm^2}$$

$$M_{ne} := S_f \cdot F_y = 1.045 \times 10^6 \cdot N \cdot mm$$

$$M_{crl} := 1000\,3057 \cdot N \cdot mm = 3.057 \times 10^6 \cdot N \cdot mm$$

$$\lambda_l := \sqrt{\frac{M_{ne}}{M_{crl}}} = 0.585 \qquad \text{(Eq. 1.2.2.7)}$$

For $\lambda_l < 0.776$

$$M_{nl} := M_{ne} = 1.045\times 10^{6}\cdot N\cdot mm \qquad \text{(Eq. 1.2.2.5)}$$

S_f - Gross section modulus referenced to the extreme fiber in the first yield
F_y - Yield stress
M_{crl} - Critical elastic local buckling

Distorsional Buckling (1.2.2.3)

The nominal flexural strength, M_{nd}, for distortional buckling should be calculated accordingly to:

$$M_{crd} := 1000834.9 N\cdot mm = 8.349\times 10^{5}\cdot N\cdot mm$$

$$M_y := M_{ne} = 1.045\times 10^{6}\cdot N\cdot mm$$

$$\lambda_d := \sqrt{\frac{M_y}{M_{crd}}} = 1.119 \qquad \text{(Eq. 1.2.2.10)}$$

For $\lambda_d > 0.673$ (Eq. 1.2.2.8)

$$M_{nd} := \left[1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right]\cdot\left(\frac{M_{crd}}{M_y}\right)^{0.5}\cdot M_y = 7.502\times 10^{5}\cdot N\cdot mm \qquad \text{(Eq. 1.2.2.9)}$$

M_{crd} - Critical elastic local buckling

The ultimate bending moment should be chosen, from the minimum between, the nominal flexural strength for local, distortional buckling:

$$M_{fin} := \min\left(M_{nd}, M_{nl}\right)$$

$$M_{fin} = 7.502\times 10^{5}\cdot N\cdot mm$$

The ultimate bending moment for the whole sheet, with three webs:

$$M_{Sheet} := 4\cdot M_{fin} = 3.001\times 10^{6}\cdot N\cdot mm$$

$$M_{exp} := 5116800 N\cdot mm$$

$$P_{\%} := M_{Sheet}\cdot\frac{100}{M_{exp}} = 58.645 \quad \%$$

Using the member plastic moment of the section in the following calculations:

Local Buckling (1.2.2.2)

The nominal flexural strength, M_{nl}, for local buckling should be calculated accordingly to:

$$M_{pl} := Z_f \cdot F_y$$

$$Z_f := 6059.43\,mm^3$$

$$F_y := 280 \cdot \frac{N}{mm^2}$$

$$M_{pl} := Z_f \cdot F_y = 1.697 \times 10^6 \cdot N \cdot mm$$

$$M_{crl} := 1000 \cdot 3057 \cdot N \cdot mm = 3.057 \times 10^6 \cdot N \cdot mm$$

$$\lambda_l := \sqrt{\frac{M_{pl}}{M_{crl}}} = 0.745 \quad \text{(Eq. 1.2.2.7)}$$

$$\text{For} \quad \lambda_l > 0.776 \quad \text{(Eq. 1.2.2.5)}$$

$$M_{nl} := \left[1 - 0.15\left(\frac{M_{crl}}{M_{pl}}\right)^{0.4}\right] \cdot \left(\frac{M_{crl}}{M_{pl}}\right)^{0.4} \cdot M_{pl} = 1.74 \times 10^6 \cdot N \cdot mm \quad \text{(Eq. 1.2.2.6)}$$

F_y - Yield stress
M_{crl} - Critical elastic local buckling
M_{pl} - Member plastic moment
Z_f - Plastic section modulus

Distorsional Buckling (1.2.2.3)

The nominal flexural strength, M_{nd}, for distortional buckling should be calculated accordingly to:

$$M_{crd} := 1000 \cdot 834.9 \cdot N \cdot mm = 8.349 \times 10^5 \cdot N \cdot mm$$

$$M_y := M_{pl} = 1.697 \times 10^6 \cdot N \cdot mm$$

$$\lambda_d := \sqrt{\frac{M_y}{M_{crd}}} = 1.426 \quad \text{(Eq. 1.2.2.10)}$$

For $\lambda_d > 0.673$ (Eq. 1.2.2.8)

$$M_{nd} := \left[1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right]\cdot\left(\frac{M_{crd}}{M_y}\right)^{0.5}\cdot M_y = 1.007\times 10^6\cdot N\cdot mm \qquad \text{(Eq. 1.2.2.9)}$$

M_{crd} - Critical elastic local buckling

M_{pl} - Member plastic moment

Z_f - Plastic section modulus

The ultimate bending moment should be chosen, from the minimum between, the nominal flexural strength for local, distortional buckling:

$$M_{fin} := \min(M_{nd}, M_{nl})$$

$$M_{fin} = 1.007\times 10^6\cdot N\cdot mm$$

The ultimate bending moment for the whole sheet, with three webs:

$$M_{Sheet} := 4\cdot M_{fin} = 4.026\times 10^6\cdot N\cdot mm$$

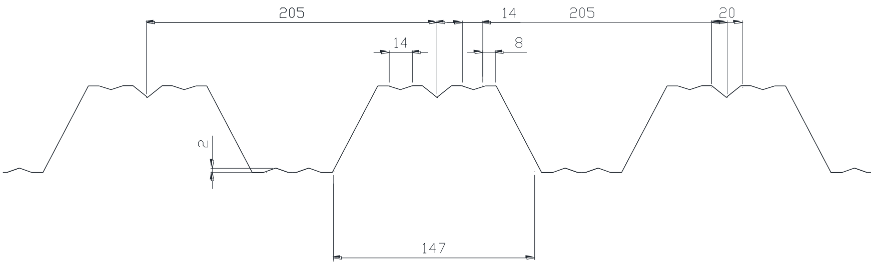
$$M_{exp} := 5116800 N\cdot mm$$

$$P_{\%} := M_{Sheet}\cdot\frac{100}{M_{exp}} = 78.682 \quad \%$$

4. AISI EUROCOL 60

Design of Cold-Formed Steel Structural Members Using the Direct Strength Method (AISI-S100-Appendix 1)

Sheet N4:



Local Buckling (1.2.2.2)

The nominal flexural strength, M_{nl}, for local buckling should be calculated accordingly to:

$$M_{ne} := S_f \cdot F_y$$

$I_{xx} := 122532.92mm^4$ value taken from CUFSM

$z_{cg} := 32.80mm$ value taken from CUFSM

$$S_f := \frac{I_{xx}}{z_{cg}} = 3.736 \times 10^3 \cdot mm^3$$

$$F_y := 369 \cdot \frac{N}{mm^2}$$

$$M_{ne} := S_f \cdot F_y = 1.378 \times 10^6 \cdot N \cdot mm$$

$$M_{crl} := 1000 5214 N \cdot mm = 5.214 \times 10^6 \cdot N \cdot mm$$

$$\lambda_l := \sqrt{\frac{M_{ne}}{M_{crl}}} = 0.514$$

For $\lambda_l < 0.776$ (Eq. 1.2.2.7)

$$M_{nl} := M_{ne} = 1.378\times 10^{6}\cdot N\cdot mm$$ (Eq. 1.2.2.5)

S_f - Gross section modulus referenced to the extreme fiber in the first yield

F_y - Yield stress

M_{crl} - Critical elastic local buckling

Distorsional Buckling (1.2.2.3)

The nominal flexural strength, M_{nd}, for distortional buckling should be calculated accordingly to:

$$M_{crd} := 1000\,2411\cdot N\cdot mm = 2.411\times 10^{6}\cdot N\cdot mm$$

$$M_y := M_{ne} = 1.378\times 10^{6}\cdot N\cdot mm$$

$$\lambda_d := \sqrt{\frac{M_y}{M_{crd}}} = 0.756$$ (Eq. 1.2.2.10)

For $\lambda_d > 0.673$

$$M_{nd} := \left[1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right]\cdot\left(\frac{M_{crd}}{M_y}\right)^{0.5}\cdot M_y = 1.293\times 10^{6}\cdot N\cdot mm$$ (Eq. 1.2.2.9)

M_{crd} - Critical elastic local buckling

The ultimate bending moment should be chosen, from the minimum between, the nominal flexural strength for local, distortional buckling:

$$M_{fin} := \min\left(M_{nd}, M_{nl}\right)$$

$$M_{fin} = 1.293\times 10^{6}\cdot N\cdot mm$$

The ultimate bending moment for the whole sheet, with three webs:

$$M_{Sheet} := 3\cdot M_{fin} = 3.878\times 10^{6}\cdot N\cdot mm$$

$$M_{exp} := 4180000\,N\cdot mm$$

$$P_{\%} := M_{Sheet} \cdot \frac{100}{M_{exp}} = 92.773 \quad \%$$

Using the member plastic moment of the section in the following calculations:

Local Buckling (1.2.2.2)

The nominal flexural strength, M_{nl}, for local buckling should be calculated accordingly to:

$$M_{pl} := Z_f \cdot F_y$$

$$Z_f := 4697.9 mm^3$$

$$F_y := 369 \frac{N}{mm^2}$$

$$M_{pl} := Z_f \cdot F_y = 1.734 \times 10^6 \cdot N \cdot mm$$

$$M_{crl} := 1000 5214 N \cdot mm = 5.214 \times 10^6 \cdot N \cdot mm$$

$$\lambda_l := \sqrt{\frac{M_{pl}}{M_{crl}}} = 0.577 \qquad \text{(Eq. 1.2.2.7)}$$

For $\lambda_l < 0.776$

$$M_{nl} := M_{pl} = 1.734 \times 10^6 \cdot N \cdot mm \qquad \text{(Eq. 1.2.2.5)}$$

F_y - Yield stress
M_{crl} - Critical elastic local buckling
M_{pl} - Member plastic moment
Z_f - Plastic section modulus

Distorsional Buckling (1.2.2.3)

The nominal flexural strength, M_{nd}, for distortional buckling should be calculated accordingly to:

$$M_{crd} := 1000 2411 N \cdot mm = 2.411 \times 10^6 \cdot N \cdot mm$$

$$M_y := M_{pl} = 1.734 \times 10^6 \cdot N \cdot mm$$

$$\lambda_d := \sqrt{\frac{M_y}{M_{crd}}} = 0.848$$

For $\lambda_d > 0.673$ (Eq. 1.2.2.10)

$$M_{nd} := \left[1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right]\cdot\left(\frac{M_{crd}}{M_y}\right)^{0.5}\cdot M_y = 1.514\times 10^{6}\cdot N\cdot mm$$ (Eq. 1.2.2.9)

M_{crd} - Critical elastic local buckling

M_{pl} - Member plastic moment

Z_f - Plastic section modulus

The ultimate bending moment should be chosen, from the minimum between, the nominal flexural strength for local, distortional buckling:

$$M_{fin} := \min(M_{nd}, M_{nl})$$

$$M_{fin} = 1.514\times 10^{6}\cdot N\cdot mm$$

The ultimate bending moment for the whole sheet, with three webs:

$$M_{Sheet} := 3\cdot M_{fin} = 4.542\times 10^{6}\cdot N\cdot mm$$

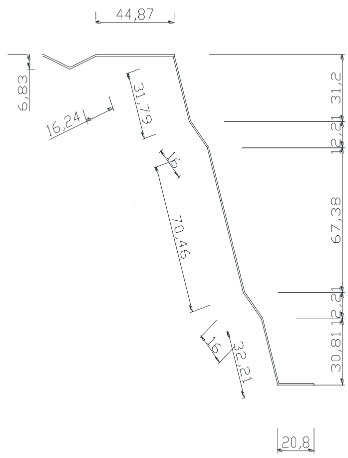
$$M_{exp} := 4180000 N\cdot mm$$

$$P_{\%} := M_{Sheet}\cdot\frac{100}{M_{exp}} = 108.658 \quad \%$$

5. EN-1993 H150

Sheeting with stiffeners in flange and web

Bending moment capacity of a sheeting with stiffeners in flange and web.



Cross section of the section to be calculated.

Values of symbols

$$E := 210000 \frac{N}{mm^2}$$

$$f_y := 327.81 \frac{N}{mm^2}$$

$$f_{yb} := f_y$$

$$t := 0.79 \cdot mm$$

$$r := 3 \cdot mm$$

$$b_p := 44.87 \cdot mm$$

$$\frac{r}{t} = 3.797 \qquad \frac{r}{t} < 5 = 1$$

$$\frac{r}{b_p} = 0.067 \qquad \frac{r}{b_p} < 0.10 = 1$$

Acording to clause 5.1(3) the influence of rounded corners may be neglected

Top flange

Effective width of part of the top flange (see 4.4 of EN 1993-1-5)

$$b_1 := \frac{b_p}{t} = 56.797$$

$$\varepsilon := \sqrt{\frac{235}{327.81}} = 0.847$$

With k.σ assumed:4

$$k_\sigma := 4$$

$$\lambda_p := \frac{b_1}{28.4 \cdot \varepsilon \cdot \sqrt{k_\sigma}} = 1.181$$

$\psi := 1$ for $k_\sigma := 4$

$$\rho := \frac{\lambda_p - 0.055(3 + \psi)}{\lambda_p^{\,2}} = 0.689 \qquad (4.2)$$

$$b_{e1} := 0.5 \cdot 0.689 \cdot 44.87 \cdot mm = 15.458mm$$

$$b_{e2} := b_{e1} = 15.458mm$$

Intermediate stiffener (see clause 5.5.3.4.2)

For caculations of As:0.5be=15.458mm

For calculation of Is:15t=15*0.79=11.85mm

	I	z	Iz	Iz^2	h	$\frac{Ih^2}{12}$
	23.7	0	0	0	0	0
	32.48	3.02	98.08	296.23	6.83	126.26
Sum	56.18		98.08	296.23		

$$z := \frac{98.08 \cdot mm^2}{56.18 \cdot mm} = 1.746 \cdot mm$$

$$A_{s1} := 2 \cdot 15.458mm + 32.48mm = 63.396mm \qquad \frac{A_s}{t}$$

$$b_p := 44.87 \cdot mm$$

$$b_s := 2 \cdot 16.24mm = 32.48mm$$

$I_s := 170.36\text{mm}^4$ value taken from CUFSM

$$I_{s1} := \frac{I_s}{t} = 215.646\text{mm}^3$$

The values of lb and kwo may be determined from the following:
-for a compression flange with one intermediate stiffener:

$$l_b := 3.07\cdot\sqrt[4]{I_s\cdot b_p^{\,2}\cdot\frac{(2\cdot b_p + 3\cdot b_s)}{t^3}} \quad (5.25)$$

$$k_{wo} := \sqrt{\frac{s_w + 2\cdot b_d}{s_w + 0.5\cdot b_d}} \quad (5.26)$$

$$l_b := 3.07\cdot\left[215.646\text{mm}^3\cdot 0.79\cdot\text{mm}\cdot 44.87^2\cdot\text{mm}^2\cdot\frac{(2\cdot 44.87\cdot\text{mm} + 3\cdot 32.48\cdot\text{mm})}{0.79^3\cdot\text{mm}^3}\right]^{\frac{1}{4}} = 327.947\text{mm}$$

$$s_w := \sqrt{60^2\text{mm}^2 + 153^2\cdot\text{mm}^2} = 164.344\text{mm}$$

if $\dfrac{l_b}{s_w} = 1.995 < 2$

$$b_d := 2\cdot b_p + b_s = 122.22\text{mm}$$

$$k_{wo} := \sqrt{\frac{s_w + 2\cdot b_d}{s_w + 0.5\cdot b_d}} = 1.347$$

$$k_w := k_{wo} - (k_{wo} - 1)\cdot\left[\frac{2\cdot l_b}{s_w} - \left(\frac{l_b}{s_w}\right)^2\right]$$

$$k_w = 1.343$$

For one central flange stiffener, the elastic critical buckling stress **σ**cr.s should be obtained from:

$$\sigma_{cr.s} := \frac{(4.2\cdot k_w\cdot E)}{A_{s1}\cdot t}\cdot\sqrt{\frac{I_{s1}\cdot t^3\cdot t}{4\cdot b_p^{\,2}\cdot(2\cdot b_p + 3\cdot b_s)}} = 176.603\frac{\text{N}}{\text{mm}^2} \quad (5.22)$$

Determination of the position of centroidal axis when the web and the stiffeners in the top flange are fully effective

For symbols see figure 5.14 of EN 1993-1-3:

l	z	l*z
30.358	0	0
16.24	3.02	49.0448
31.79	15.6	495.924
16	37.3	596.8
70.46	77.1	5432.466
16	116.89	1870.24
32.21	138.4	4457.864
20.8	153	3182.4
233.858		16084.74

$$\Sigma l := 233.858\,mm$$

$$\Sigma lz := 16084.74\,mm^2$$

$$e_c := \frac{\Sigma lz}{\Sigma l} = 68.78\,mm$$

$e_c = 0.069\,m$ -the bending moment capacity is governed by the compression stress at the top flange

WEB

Effective widths in the web (see clause 5.5.3.4.3)

For symbols see figure 5.14 of EN 1993-1-3

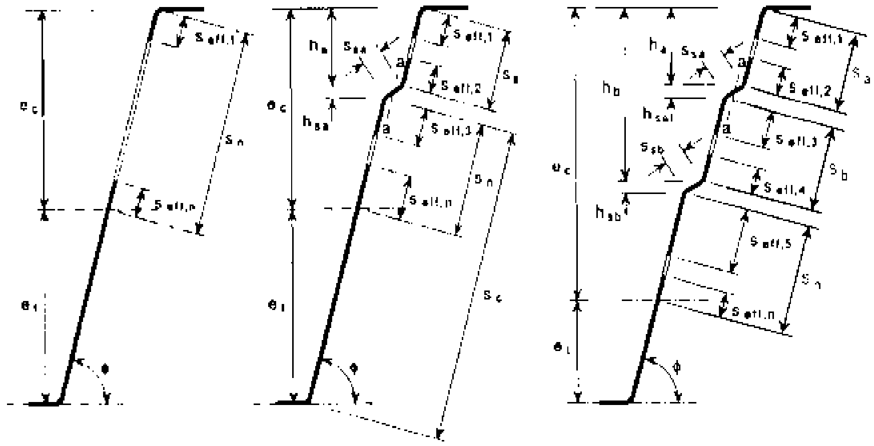
Initially the location of the centroidal axis should be based on the effective cross-section of the flange but the gross cross-sections of the webs. In this case the basic effective width Seff0 should be obtained from:

$$\sigma_{com.Ed} := 256.72\frac{N}{mm^2} \qquad \gamma_{M0} := 1$$

$$S_{eff.0} := 0.76 \cdot t \cdot \sqrt{\frac{E}{\gamma_{M0} \cdot \sigma_{com.Ed}}} = 17.172\,mm \qquad (5.32)$$

$\sigma_{com.Ed}$ is the stress in the compression flange when the cross-section resistance is reached

$$t := 0.79\,mm$$



$h_a := 31.2 \cdot mm$

$h_{sa} := 12.21 \cdot mm$

$$S_{eff.1} := S_{eff.0} = 17.172\,mm \qquad (5.33a)$$

$$S_{eff.2} := \left(1 + 0.5 \cdot \frac{h_a}{e_c}\right) S_{eff.0} = 21.067\,mm \qquad (5.33b)$$

$$S_{eff.3} := S_{eff.0} \cdot \left[1 + 0.5 \cdot \frac{\left(h_a + h_{sa}\right)}{e_c}\right] = 22.591\,mm \qquad (5.33c)$$

$$S_{eff.n} := 1.5 \cdot S_{eff.0} = 25.758\,mm \qquad (5.33f)$$

If S.eff1+S.eff.2 > S.a the whole of S.a is effective, so revise as follows:

$S_a := 32 \cdot mm$

$$S_{eff.1} := \frac{S_a}{2 + 0.5 \cdot \frac{h_a}{e_c}} = 14.37 \cdot mm \qquad (5.35a)$$

$$S_{eff.2} := S_a \cdot \frac{\left(1 + 0.5 \cdot \frac{h_a}{e_c}\right)}{2 + 0.5 \cdot \frac{h_a}{e_c}} = 17.63 \cdot mm \qquad (5.35b)$$

If S.eff3+S.eff.n > S.n the whole of S.n is effective, so revise as follows:

$$S_n := 27.175mm$$

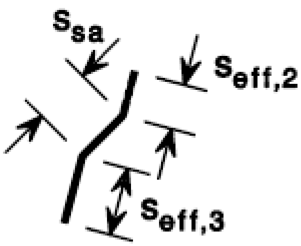
$$S_{eff.3} := S_n \cdot \frac{1 + 0.5 \cdot \frac{\left(h_a + h_{sa}\right)}{e_c}}{2.5 + 0.5 \cdot \frac{\left(h_a + h_{sa}\right)}{e_c}} = 12.697mm \tag{5.36a}$$

$$S_{eff.n} := \frac{1.5 \cdot S_n}{2.5 + 0.5 \cdot \frac{\left(h_a + h_{sa}\right)}{e_c}} = 14.478mm \tag{5.36b}$$

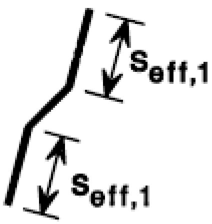
Web stiffener (see clause 5.5.3.4.3)

Cross section for determination of A.s (see figure 5.15)

Cross section for determination of I.s (see figure 5.15)



Cross-section for determining A_s



Cross-section for determining I_s

$$S_{sa} := 16 \cdot mm$$

$$A_{sa1} := S_{sa} + S_{eff.2} + S_{eff.3} = 46.327\cdot mm$$

$$I_{s1} := 16\cdot mm\cdot\frac{10.34^2\cdot mm^2}{12} + \left(2\cdot S_{eff.1}\right)\cdot\left(\frac{10.34\,mm}{2}\right)^2 = 910.76\,mm^3$$

For a single stiffener, the elastic critical buckling stress σ.cr.sa should be determined using;

$$S_c := 115\cdot mm$$

$$s_1 := 0.9\cdot\left(S_a + S_{sa} + S_c\right) = 146.7\cdot mm \qquad (5.39b)$$

$$s_2 := s_1 - S_a - 0.5\cdot S_{sa} = 106.7\cdot mm \qquad (5.39d)$$

$$k_f := 1$$

$$\sigma_{cr.sa} := \frac{1.05\cdot k_f\cdot E\cdot\sqrt{I_{s1}\cdot t\cdot t^3\cdot s_1}}{A_{sa1}\cdot t\cdot s_2\cdot\left(s_1 - s_2\right)} = 322.027\frac{N}{mm^2} \qquad (5.39a)$$

Interaction of flange and web stiffener should be taken into account according clause: 5.5.3.4.4

$$h_{ha} := 12.21\cdot mm$$

$$\beta_s := 1 - \frac{\left(h_a + 0.5\cdot h_{ha}\right)}{e_c} = 0.458$$

for a profile in bending

$$\sigma_{cr.mod} := \frac{\sigma_{cr.s}}{\sqrt[4]{1 + \left(\beta_s\cdot\frac{\sigma_{cr.s}}{\sigma_{cr.sa}}\right)^4}} = 176.428\frac{N}{mm^2} \qquad (5.42)$$

Reduced thickness ratio for flange and web stiffener according to clause 5.5.3.1.(7)

$$\lambda_d := \sqrt{\frac{f_{yb}}{\sigma_{cr.s}}} = 1.362 \qquad (5.12d)$$

$$\chi_d := 1.47 - 0.723\lambda_d = 0.485 \qquad (5.12b)$$

$$\frac{t_{red}}{t} := 0.485$$

Element	l	t.red/t	A/t	z	A*z/t	A*z*z/t	h	A*h*h/12t
1	16.24	0.485	7.8764	3.02	23.78673	71.83592	6.04	23.94530619
2	15.458	0.485	7.49713	0	0	0	0	0
3	15.458	1	15.458	0	0	0	0	0
4	14.37	1	14.37	6.88	98.8656	680.1953	13.8	228.0519
5	17.63	0.485	8.55055	22.26	190.3352	4236.863	17	205.9257458
6	16	0.485	7.76	37.3	289.448	10796.41	12.21	96.407718
7	12.697	0.485	6.158045	49.07	302.1753	14827.74	12.14	75.63085074
9	57.763	1	57.763	82.68	4775.845	394866.9	55.09	14608.78305
10	16	1	16	116.89	1870.24	218612.4	12.21	198.7788
11	32.21	1	32.21	138.4	4457.864	616968.4	30.415	2483.048031
12	20.08	1	20.08	153	3072.24	470052.7	153	39171.06
			193.7231		15080.8	1731113		57091.6314

$$e_c := \frac{15080.8\,mm^2}{193.72\,mm} = 77.848\,mm$$

$$\frac{I}{t} := 1731113\,mm^3 + 57091.63\,mm^3 - e_c^{\,2} \cdot 193.72\,mm = 6.142 \times 10^5 \cdot mm^3$$

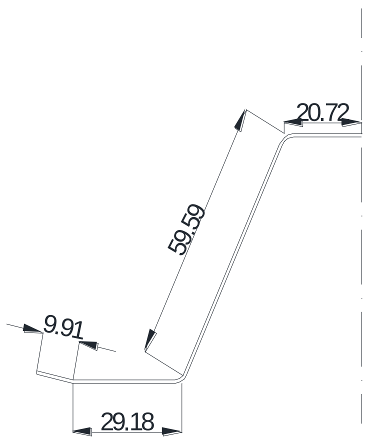
$$M_{crd} := 614201 \cdot mm^3 \cdot \frac{0.79 \cdot mm}{e_c} \cdot 327 \cdot \frac{N}{mm^2} \cdot \frac{2}{280 \cdot mm} = 1.456 \times 10^4\,N$$

$$M_{sheet} := M_{crd} \cdot 873 \cdot mm = 12.709\,kN \cdot m$$

6. EN-1993 H 55

Sheeting with stiffenersin flange and web

Bending moment capacity of a sheeting with stiffeners in flange and web.



Values of symbols

$E := 210000 \cdot \frac{N}{mm^2}$

$f_y := 337 \cdot \frac{N}{mm^2}$

$f_{yb} := f_y$

$t := 0.77 \cdot mm$

$r := 2 \cdot mm$

$b_p := 20.72 \cdot mm$

$\frac{r}{t} = 2.597$ $\qquad \frac{r}{t} < 5 = 1$

$\frac{r}{b_p} = 0.097$ $\qquad \frac{r}{b_p} < 0.10 = 1$

Acording to clause 5.1(3) the influence of rounded corners may be neglected

Top flange

Effective width of part of the top flange(see 4.4 of EN 1993-1-5)

$$b_1 := \frac{b_p}{t} = 26.909$$

$$\varepsilon := \sqrt{\frac{235}{337}} = 0.835$$

With k.σ assumed:4

$$k_\sigma := 4$$

$$\lambda_p := \frac{b_1}{28.4\varepsilon \cdot \sqrt{k_\sigma}} = 0.567$$

$\psi := 1$ for $k_\sigma := 4$

$$\rho := \frac{\lambda_p - 0.055(3 + \psi)}{\lambda_p^{\,2}} = 1.079 \qquad (4.2)$$

$$b_{e1} := 0.5 \cdot 1.079 \cdot 20.72 \cdot mm = 11.178\,mm$$

$$b_{e2} := b_{e1} = 11.178\,mm$$

$$A_s := 41.44mm \cdot 0.77mm \cdot 2 = 63.818\,mm^2$$

$$I_s := \frac{44.14mm \cdot 0.77^3 \cdot mm^3}{12} = 1.679\,mm^4$$

$$b_d := 2 \cdot b_p$$

$$l_b := 3.07 \cdot \sqrt[4]{I_s \cdot b_p^{\,2} \cdot \frac{(2 \cdot b_p)}{t^3}} \qquad (5.25)$$

$$s_w := \sqrt{27.39^2 \cdot mm^2 + 55^2 \cdot mm^2}$$

$$k_{wo} := \sqrt{\frac{s_w + 2 \cdot b_d}{s_w + 0.5 b_d}} \qquad (5.26)$$

if $\dfrac{l_b}{s_w} = 0.799$ $\quad < 2$

$$k_{wo} := \sqrt{\frac{s_w + 2 \cdot b_d}{s_w + 0.5 b_d}} = 1.325$$

$$k_w := k_{wo} - (k_{wo} - 1) \cdot \left[\frac{2 \cdot l_b}{s_w} - \left(\frac{l_b}{s_w} \right)^2 \right]$$

$$k_w = 1.013$$

The critical buckling stress should be obtained from:

$$\sigma_{cr.s} := \frac{(4.2 k_w \cdot E)}{A_s} \cdot \sqrt{\frac{I_s \cdot t^3}{4 \cdot b_p{}^2 \cdot (2 \cdot b_p)}} = 45.958 \frac{N}{mm^2} \qquad (5.22)$$

Determination of the position of centroidal axis when the web and the stiffeners in the top flange are fully effective

For symbols see figure 5.14 of EN 1993-1-3:

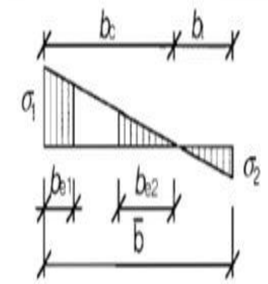
l	z	l*z
22.34	0	0
59.59	27.5	1638.725
29.18	55	1604.9
9.91	54	535.14
121.02		3778.765

$$\Sigma l := 121.02 mm$$

$$\Sigma lz := 3778.76 mm^2$$

$$e_c := \frac{\Sigma lz}{\Sigma l} = 31.224 mm$$

Effective width of part of the web(see 4.4 of EN 1993-1-5)



$\psi < 0$:

$b_{eff} = \rho\, b_c = \rho\, \bar{b} / (1-\psi)$

$b_{e1} = 0{,}4\, b_{eff}$ $b_{e2} = 0{,}6\, b_{eff}$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1	AC1 $-1 > \psi \geq -3$ AC1
Buckling factor k_σ	4,0	$8{,}2 / (1{,}05 + \psi)$	7,81	$7{,}81 - 6{,}29\psi + 9{,}78\psi^2$	23,9	$5{,}98\,(1-\psi)^2$

$\psi < 0$

$\psi := -1$

$k_\sigma := 23.9$

$b := e_c \cdot 2$

$t := 0.77\,mm$

$\lambda_p := \dfrac{\frac{b}{t}}{28.4\varepsilon \cdot \sqrt{k_\sigma}} = 0.7$ $\qquad b_c := e_c$

$\lambda_p < 0.5 + \sqrt{0.085 - 0.055\psi}$ (4.3)

$\rho := 1$

$b_{eff} := \rho \cdot b_c = 0.031\,m$

$b_{e1} := 0.4 \cdot b_{eff} = 12.49\,mm$ (5.34a)

$b_{e2} := 0.6 \cdot b_{eff} = 18.735\,mm$ (5.34b)

Element	l	t.red/t	A/t	z	A*z/t	A*z*z/t	h	A*h*h/12t
1	11.17	1	11.17	0	0	0	0	0
2	12.49	1	12.49	5.565	69.50685	386.8056	11.13	128.9352068
3	18.73	1	18.73	19.465	364.5795	7096.539	16.67	433.7382581
4	30.49	1	30.49	41.285	1258.78	51968.72	26.97	1848.153637
5	29.18	1	29.18	55	1604.9	88269.5	0	0
6	9.91	1	9.91	54	535.14	28897.56	2	3.303333333
			111.97		3832.906	176619.1		2285.195228

$$e_c := \frac{3832.9mm^2}{121.97mm} = 31.425mm$$

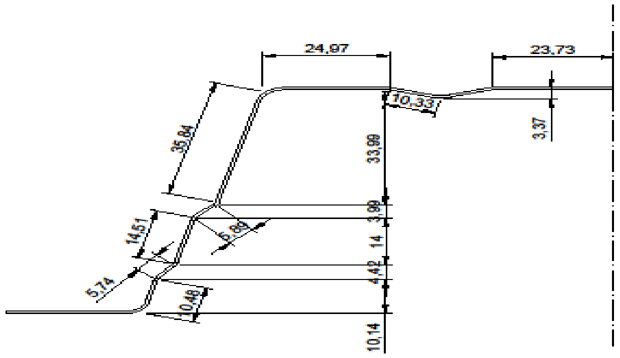
$$I_t := 176619mm^3 + 2285.19mm^3 - e_c^{\,2}\cdot 121.97mm = 5.846\times 10^4\cdot mm^3$$

$$M_{crd} := 58460mm^3\cdot\frac{(0.77\cdot mm)}{e_c}\cdot 337\cdot\frac{N}{mm^2}\cdot\frac{2}{174\cdot mm}$$

$$M_{sheet} := M_{crd}\cdot 1044mm = 7.632\,kN\cdot m$$

7. EN-1993 H 68
Sheeting with stiffeners in flange and web.

Bending moment capacity of a sheeting with stiffeners in flange and web.



3.5.1 Values of symbols

$$E := 210000 \frac{N}{mm^2}$$

$$f_y := 280 \cdot \frac{N}{mm^2}$$

$$f_{yb} := f_y$$

$$t := 0.85mm$$

$$r := 4.98mm$$

$$b_p := 24.9 \cdot mm$$

$$\frac{r}{t} = 5.859 \qquad \frac{r}{t} < 5 = 0$$

$$\frac{r}{b_p} = 0.2 \qquad \frac{r}{b_p} < 0.10 = 0$$

Because the conditions are not achieved, the rounded corners may not be neglected.EC-3 (5.1.)

$$A_g := A_{gsh} \cdot (1 - \delta) \qquad (5.1a)$$

$$I_g := I_{gsh} \cdot (1 - 2 \cdot \delta) \qquad (5.1b)$$

$$\delta := 0.43 \frac{\sum r_j \cdot \frac{\phi_j}{90}}{\sum b_{pi}} \qquad (5.1d)$$

Where :

r_j -is the internal radius of curved element j

ϕ_j -is the angle between two plane elements

b_{pi} -is the notional flat width of plane element i for cross-section with sharp corners

$$\Sigma b_{pi} := (13.61 + 10.48 + 5.74 + 14.51 + 12.78 + 23.76 + 24.97 + 10.33 + 10.33 + 23.73) \cdot 2 = 300.48$$

$$\Sigma b_{pi} := 300.48mm$$

$$\Sigma r_j \cdot \frac{\phi_j}{90} := 2 \cdot \left(3.99 \frac{105}{90} + 4 \cdot 1.99 \frac{155}{90} + 4.98 \frac{105}{90}\right) = 48.348$$

$$\delta := 0.43 \frac{48.348}{300.48} = 0.069$$

$A_{gsh} := 264.35mm^2$ -value taken from CUFSM

$I_{gsh} := 167464.16mm^4$ -value taken from CUFSM

$A_g := A_{gsh} \cdot (1 - \delta) = 246.06mm^2$

$I_g := I_{gsh} \cdot (1 - 2 \cdot \delta) = 1.443 \times 10^5 \cdot mm^4$

Top flange

Effective width of part of the top flange(see 4.4 of EN 1993-1-5)

$$b_1 := \frac{b_p}{t} = 29.294$$

$$\varepsilon := \sqrt{\frac{235}{280}} = 0.916$$

With k.σ assumed:4

$k_\sigma := 4$

$$\lambda_p := \frac{b_1}{28.4 \varepsilon \cdot \sqrt{k_\sigma}} = 0.563$$

$\psi := 1$ for $k_\sigma := 4$

$$\rho := 1 = 1 \qquad (4.2)$$

$b_{e1} := 0.5 \cdot 1 \cdot 24.97 \cdot mm = 12.485mm$

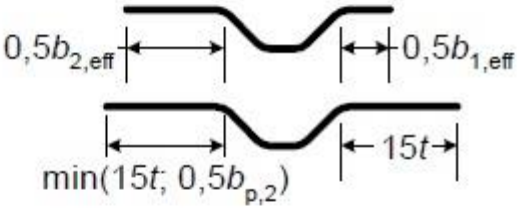
$b_{e2} := b_{e1} = 12.485mm$

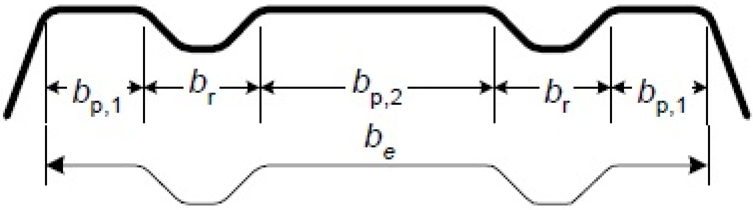
Intermediate stiffener (see clause 5.5.3.4.2)

For calculations of As:0.5be=12.485mm

For calculation of Is:15t=15*0.85=12.75mm

For two simetrically placed flange stiffeners, the elastic critical buckling stress $\sigma_{cr.s}$ should be obtained with:





$b_{p1} := 24.97 \cdot mm$

$b_{p2} := 47.46 mm$

$b_s := 20.66 mm$

$b_r := 19.77 \cdot mm$

$b_e := 2 \cdot b_{p1} + b_{p2} + 2 \cdot b_s = 138.72 mm$

$b_1 := b_{p1} + 0.5 \cdot b_r$

b_{p1} -is the notional flat width of an outer plane element, as shown in figure 5.13
b_{p2} -is the notional flat width of the central plane element, as shown in figure 5.13
b_r -is the overall width of a stiffener, see figure 5.13

The values of l_b and k_{wo} may be determined from the following:
- for a compression flange with two intermediate stiffeners

$$l_b := 3.65 \sqrt[4]{I_s \cdot b_1^{\,2} \cdot \left(\frac{3 \cdot b_e - 4 \cdot b_1}{t^3}\right)} \quad (5.27)$$

$$k_{wo} := \sqrt{\frac{(2 \cdot b_e + s_w) \cdot (3 \cdot b_e - 4 \cdot b_1)}{b_1 \cdot (4 \cdot b_e - 6 \cdot b_1) + s_w \cdot (3 \cdot b_e - 4 \cdot b_1)}} \quad (5.28)$$

l	z	l*z	l*z*z	h	l*h*h/12
25.5	0	0	0	0	0
47.46	0	0	0	0	0
20.66	1.68	34.7088	58.31078	3.37	19.5528
20.66	1.68	34.7088	58.31078	3.37	19.5528
114.28		69.4176	116.6216		39.10559

$$z := \frac{69.41}{114.28} = 0.607$$

$$A_s := b_e \cdot t = 117.912mm^2$$

$$I_s := 0.85mm\left(116.62mm^3 + 39.10mm^3 - 114.28mm0.607^2 \cdot mm^2\right) = 96.572mm^4$$

$$s_w := \left(68^2 \cdot mm^2 + 23.06^2 \cdot mm^2\right)^{\frac{1}{2}} = 71.804mm$$

$$l_b := 3.65\sqrt[4]{I_s \cdot b_1^{\,2} \cdot \left(\frac{3 \cdot b_e - 4.b_1}{t^3}\right)} = 311.237mm$$

$$k_{wo} := \sqrt{\frac{\left(2 \cdot b_e + s_w\right) \cdot \left(3 \cdot b_e - 4 \cdot b_1\right)}{b_1 \cdot \left(4 \cdot b_e - 6 \cdot b_1\right) + s_w \cdot \left(3 \cdot b_e - 4 \cdot b_1\right)}} = 1.74$$

The value of kw may be calcuated from the compression flange buckling wavelength as follows:

$$\frac{l_b}{s_w} = 4.335 \qquad \frac{l_b}{s_w} > 2 \tag{5.24a}$$

$$k_w := k_{wo}$$

$$\sigma_{cr.s} := \frac{4.2 k_w \cdot E}{A_s} \cdot \sqrt{\frac{I_s \cdot t^3}{8 \cdot b_1^{\,2} \cdot \left(3 \cdot b_e - 4 \cdot b_1\right)}} = 61.118\frac{N}{mm^2} \tag{5.23a}$$

Determination of the position of centroidal axis when the web and the stiffeners in the top flange are fully effective

For symbols see figure 5.14 of EN 1993-1-3:

l	z	l*z
25.23	0	0
10.33	1.68	17.3544
10.33	1.68	17.3544
25.23	0	0
10.33	1.68	17.3544
36.54	17.44	637.2576
5.89	36.88	217.2232
14.51	45.88	665.7188
5.74	55.09	316.2166
10.48	62.93	659.5064
14.5	68	986
10.33	1.68	17.3544
179.44		3551.34

$$\Sigma l := 179.44mm$$

$$\Sigma lz := 3551.34mm^2$$

$$e_c := \frac{\Sigma lz}{\Sigma l} = 19.791mm$$

$e_c = 0.02\text{m}$ -the bending moment capacity is governed by the compression stress at the top flange

If the webs are unstiffened, the reduction factor χ_d should be obtained directly from $\sigma_{cr.s}$ using the method given in 5.5.3.1(7)

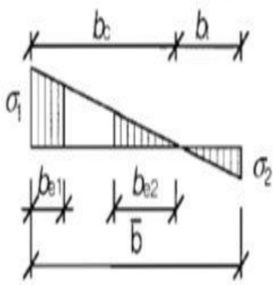
$$\lambda_d := \sqrt{\frac{f_{yb}}{\sigma_{cr.s}}} = 2.14 \qquad \text{if} \qquad \lambda_d > 1.38 \qquad \text{then} \qquad (5.12d)$$

$$\chi_d := \frac{0.66}{\lambda_d} = 0.308 \qquad (5.12c)$$

$$t_{red} := 0.308t = 0.262\text{mm}$$

$$\frac{t_{red}}{t} := 0.308$$

Effective width of part of the web(see 4.4 of EN 1993-1-5)



$\psi < 0$:

$b_{eff} = \rho\, b_c = \rho\, \bar{b} / (1-\psi)$

$b_{e1} = 0{,}4\, b_{eff} \qquad b_{e2} = 0{,}6\, b_{eff}$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1	$-1 > \psi \geq -3$
Buckling factor k_σ	4,0	$8{,}2 / (1{,}05 + \psi)$	7,81	$7{,}81 - 6{,}29\psi + 9{,}78\psi^2$	23,9	$5{,}98\,(1-\psi)^2$

Table 4.1 Internal compression element

$\psi < 0$

$\psi := -1$

$k_\sigma := 23.9$

$b := e_c \cdot 2$

$t := 0.85mm$

$$\lambda_p := \frac{\frac{b}{t}}{28.4\varepsilon \cdot \sqrt{k_\sigma}} = 0.366 \qquad b_c := e_c$$

$$\lambda_p < 0.5 + \sqrt{0.085 - 0.055\psi} \qquad (4.3)$$

$\rho := 1$

$b_{eff} := \rho \cdot b_c = 0.02m$

$$b_{e1} := 0.4 \cdot b_{eff} = 7.916mm \qquad (5.34a)$$

$$b_{e2} := 0.6 \cdot b_{eff} = 11.875mm \qquad (5.34b)$$

Element	l	t.red/t	A/t	z	A*z/t	A*z*z/t	h	A*h*h/12t
1	12.485	0.308	3.84538	0	0	0	0	0
2	12.485	0.308	3.84538	0	0	0	0	0
3	12.75	0.308	3.927	0	0	0	0	0
4	10.33	0.308	3.18164	1.685	5.361063	9.033392	3.37	3.01113061
5	10.33	0.308	3.18164	1.685	5.361063	9.033392	3.37	3.01113061
6	7.91	1	7.91	4.16	32.9056	136.8873	8.32	45.62909867
7	11.88	1	11.88	14.055	166.9734	2346.811	11.47	130.245291
9	16.05	1	16.05	27.515	441.6158	12151.06	15.45	319.2645938
10	5.89	1	5.89	37.88	223.1132	8451.528	3.99	7.81411575
11	14.51	1	14.51	46.88	680.2288	31889.13	14	236.9966667
12	5.74	1	5.74	56.09	321.9566	18058.55	4.42	9.344911333
13	10.48	1	10.48	63.37	664.1176	42085.13	10.14	89.795784
14	14.5	1	14.5	68	986	67048	0	0
			104.941		3527.633	182185.2		845.1127224

$$e_c := \frac{3527.633mm^2}{104.941mm} = 33.615mm$$

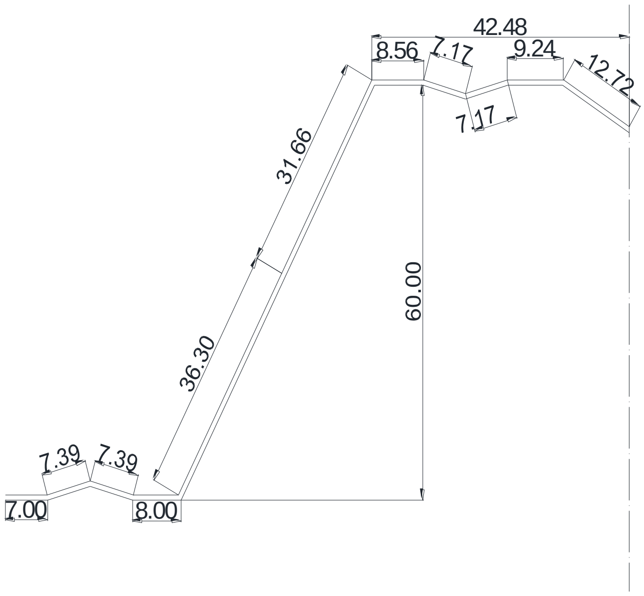
$$\frac{I}{t} := 182185.2\,mm^3 + 845.112\,mm^3 - e_c{}^2 \cdot 104.941\,mm = 6.445\times 10^4 \cdot mm^3$$

$$M_{crd} := 64450\,mm^3 \cdot \frac{(0.85\,mm)}{e_c} \cdot 280 \cdot \frac{N}{mm^2} \cdot \frac{2}{212 \cdot mm} = 4.305\times 10^3\,N$$

$$M_{sheet} := M_{crd} \cdot 850 \cdot mm = 3.659\,kN \cdot m$$

8. EN-1993 EUROCOL 60

Sheeting with stiffeners in flange and web



Values of symbols

$$E := 210000 \frac{N}{mm^2}$$

$$f_y := 369 \cdot \frac{N}{mm^2}$$

$$f_{yb} := f_y$$

$$t := 0.75 mm$$

$$r := 2 \cdot mm$$

$$b_p := 42.48 mm$$

$$\frac{r}{t} = 2.667 \qquad \frac{r}{t} < 5 = 1$$

$$\frac{r}{b_p} = 0.047 \qquad \frac{r}{b_p} < 0.10 = 1$$

Acording to clause 5.1(3) the influence of rounded corners may be neglected

Top flange

Effective width of part of the top flange(see 4.4 of EN 1993-1-5)

$$b_1 := \frac{b_p}{t} = 56.64$$

$$\varepsilon := \sqrt{\frac{235}{369}} = 0.798$$

With k.σ assumed:4

$$k_\sigma := 4$$

$$\lambda_p := \frac{b_1}{28.4\varepsilon \cdot \sqrt{k_\sigma}} = 1.25$$

$\psi := 1$ for $k_\sigma := 4$

$$\rho := \frac{\lambda_p - 0.055(3+\psi)}{\lambda_p^{\ 2}} = 0.659 \qquad (4.2)$$

$$b_{e1} := 0.5 \cdot 0.659 \cdot 42.48\,mm = 13.997 \cdot mm$$

$$b_{e2} := b_{e1} = 13.997 \cdot mm$$

$$b_1 := 8.56\,mm$$

$b_{e1} > b_1$ -conclusion, b_1 going to be fully active

Intermediate stiffeners (see clause 5.5.3.4.2)

For multiple stiffened flane (three or more stiffeners) the effective area of the entire flange is

$A_{eff} := \rho \cdot b_e \cdot t$ where ρ is the reduction factor according to EN 1993-1-5

$$\sigma_{cr.s} := 1.8E \cdot \sqrt{\frac{I_s \cdot t}{b_o^{\ 2} \cdot b_e^{\ 3}}} + 3.6\frac{E \cdot t^2}{b_o^{\ 2}} \qquad (5.23c)$$

where:

I_s - is the sum of the second moment of area of the stiffeners about the centroidal axis
b_o - is the width of the flanes as shown in 5.13
b_e - is the developed width of the flange as shown in figure 5.13

ETSEIB

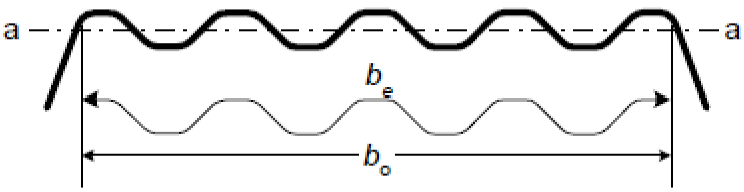


Figure 5.11 Compression flange with multiple stiffeners

$b_o := 84 \cdot mm$

$b_e := 89.6 \cdot mm$

l	z	l*z	l*z*z	h	l*h*h/12
8.56	0	0	0	0	0
7.17	1.285	9.21345	11.83928	2.57	3.946428
7.17	1.285	9.21345	11.83928	2.57	3.946428
9.24	0	0	0	0	0
12.72	3.825	48.654	186.1016	7.65	62.03385
9.24	0	0	0	0	0
7.17	1.285	9.21345	11.83928	2.57	3.946428
7.17	1.285	9.21345	11.83928	2.57	3.946428
8.56	0	0	0	0	0
77		85.5078	233.4587		77.81956

$A_s := b_e \cdot t = 67.2\,mm^2$

$\psi := \frac{\sigma_2}{\sigma_1} = \blacksquare \cdot 1$

$\psi := 1$

$k_\sigma := 4$

$b_1 := \frac{b_o}{t} = 112$

$\lambda_p := \frac{b_1}{28.4\varepsilon \cdot \sqrt{k_\sigma}} = 2.471$

$$\rho := \frac{\lambda_p - 0.055(3+\psi)}{\lambda_p^{\,2}} \qquad (4.2)$$

$$\rho = 0.369$$

$$A_{eff} := \rho \cdot b_e \cdot t = 24.775\,mm^2 \qquad (5.23b)$$

$$I_s := 192.82\,mm^4$$

$$\sigma_{cr.s} := 1.8E \cdot \sqrt{\frac{I_s \cdot t}{b_o^{\,2} \cdot b_e^{\,3}}} + 3.6\frac{E \cdot t^2}{b_o^{\,2}} = 124.073\frac{N}{mm^2} \qquad (5.23c)$$

Determination of the position of centroidal axis when the web and the stiffeners in the top and the flange are fully effective.

For symbols see figure 5.14 of EN 1993-1-3:

l	z	l*z
12.72	3.825	48.654
9.24	0	0
7.17	1.375	9.85875
7.17	1.375	9.85875
8.56	0	0
67.96	33.98	2309.281
8	60	480
7.39	58.55	432.6845
7.39	58.55	432.6845
7	60	420
142.6		4143.021

$$\Sigma l := 142.6\,mm$$

$$\Sigma lz := 4143.73\,mm^2$$

$$e_c := \frac{\Sigma lz}{\Sigma l} = 29.058\,mm$$

$$e_c = 0.029\,m$$

If the webs are unstiffened, the reduction factor X_d should be obtained directly from $\sigma_{cr.s}$ using the method given in 5.5.3.1(7)

$$\lambda_d := \sqrt{\frac{f_{yb}}{\sigma_{cr.s}}} \qquad \text{if} \qquad \lambda_d > 1.38 \qquad \text{then} \qquad (5.12d)$$

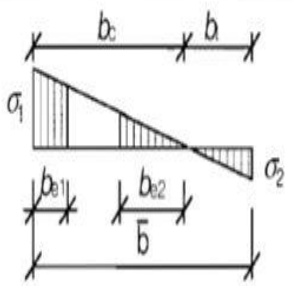
$\lambda_d := 1.72$

$$\chi_d := \frac{0.66}{\lambda_d} = 0.384 \qquad (5.12c)$$

$t_{red} := 0.383 t = 0.287 mm$

$$\frac{t_{red}}{t} := 0.383$$

Effective width of part of the web(see 4.4 of EN 1993-1-5)



$\underline{\psi < 0:}$

$b_{eff} = \rho\, b_c = \rho\, \bar{b} / (1-\psi)$

$b_{e1} = 0{,}4\, b_{eff}$ $\qquad b_{e2} = 0{,}6\, b_{eff}$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1	AC1 $-1 > \psi \geq -3$ AC1
Buckling factor k_σ	4,0	$8{,}2 / (1{,}05 + \psi)$	7,81	$7{,}81 - 6{,}29\psi + 9{,}78\psi^2$	23,9	$5{,}98\,(1-\psi)^2$

$\psi < 0$

$\psi := -1$

$k_\sigma := 23.9$

$b := e_c \cdot 2$

$t := 0.75 mm$

$$\lambda_p := \frac{\frac{b}{t}}{28.4\varepsilon \cdot \sqrt{k_\sigma}} = 0.699 \qquad b_c := e_c$$

$$\lambda_p < 0.5 + \sqrt{0.085 - 0.055\psi} \qquad (4.3)$$

$\rho := 1$

$b_{eff} := \rho \cdot b_c = 0.029\,m$

$b_{e1} := 0.4 \cdot b_{eff} = 11.623\,mm$ (5.34a)

$b_{e2} := 0.6 \cdot b_{eff} = 17.435\,mm$ (5.34b)

Element	l	t.red/t	A/t	z	A*z/t	A*z*z/t	h	A*h*h/12t
1	12.72	0.383	4.87176	3.825	18.63448	71.27689	7.65	23.75896455
2	9.24	1	9.24	0	0	0	0	0
3	7.17	0.383	2.74611	1.375	3.775901	5.191864	2.75	1.730621406
4	7.17	0.383	2.74611	1.375	3.775901	5.191864	2.75	1.730621406
5	4.28	1	4.28	0	0	0	0	0
6	11.623	1	11.623	5.1	59.2773	302.3142	10.2	100.77141
7	17.435	1	17.435	13.975	243.6541	3405.066	17.75	457.7595573
8	36.3	1	36.3	44.37	1610.631	71463.7	32.8	3254.416
9	8	1	8	60	480	28800	0	0
10	7.39	1	7.39	58.55	432.6845	25333.68	2.9	5.179158333
11	7.39	1	7.39	58.55	432.6845	25333.68	2.9	5.179158333
12	7	1	7	60	420	25200	0	0
13	4.28	0.383	1.63924	0	0	0	0	0
			120.6612		3705.118	179920.1		3850.525491

$$e_c := \frac{3705.118\,mm^2}{120.66\,mm} = 30.707\,mm$$

$$\frac{I}{t} := 179920.1\,mm^3 + 3850.52\,mm^3 - e_c^{\,2} \cdot 120.66\,mm = 7 \times 10^4 \cdot mm^3$$

$$M_{crd} := 70049\,mm^3 \cdot \frac{(0.75\,mm)}{e_c} \cdot 369 \cdot \frac{N}{mm^2} \cdot \frac{2}{205\,mm} = 6.159 \times 10^3\,N$$

$$M_{sheet} := M_{crd} \cdot 615\,mm = 3.788\,kN \cdot m$$

$$M_{experiment} := 4.18\,kN \cdot m$$

$$Results := \frac{M_{sheet} \cdot 100}{M_{experiment}} = 90.62 \quad \%$$

9. Macro H 150

```
FINI

/CLEAR

/PREP7

ET,1,281,KOP3

R,1,0.79

MP,EX,1,2.1E5

MP,NUXY,1,0.3


KEYPTS

K,1,0,147

K,2,15,153

K,3,57.2801,153

K,4,60.1490,150.8771

K,5,68.8559,122.3982

K,6,79.1995,110.1913

K,7,99.8005,42.8087

K,8,110.1441,30.6017

K,9,118.8510,2.1229

K,10,121.7199,0

K,11,140,0



K,1001,57.28,150
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K,1006,121.7199,3

L,1,2
L,2,3

LARC,3,4,1001,3.8223

L,4,5
L,5,6
L,6,7
L,7,8
L,8,9

LARC,9,10,1006,3.8223

L,10,11

K,100,140,0,270
K,101,140,0,480
K,102,140,0,1020
K,103,140,0,1230
K,104,140,0,1420
K,105,140,0,1500

L,11,100
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L,100,101
L,101,102
L,102,103
L,103,104
L,104,105

ADRAG,1,2,3,4,5,6,,11
ADRAG,7,8,9,10,,,,,11



ADRAG,17,20,22,24,26,28,,12
ADRAG,30,33,35,37,,,,,12

ADRAG,39,42,44,46,48,50,,13
ADRAG,52,55,57,59,,,,,13

ADRAG,61,64,66,68,70,72,,14
ADRAG,74,77,79,81,,,,,14



ADRAG,83,86,88,90,92,94,,15
ADRAG,96,99,101,103,,,,,15

ADRAG,105,108,110,112,114,116,,16
ADRAG,118,121,123,125,,,,,16
```

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NUMMRG,ALL

MP,EX,2,2.1E5
MP,NUXY,2,0.3

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MSHKEY,1
ESIZE,5
AESIZE,5
AMESH,ALL

ASEL,S,AREA,,11,20
MAT,2
MSHKEY,1
ESIZE,15
AESIZE,15
AMESH,ALL

ASEL,S,AREA,,21,30
MAT,1
```

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MSHKEY,1
ESIZE,15
AESIZE,15
AMESH,ALL

ASEL,S,AREA,,31,40
MAT,2
MSHKEY,1
ESIZE,15
AESIZE,15
AMESH,ALL

ASEL,S,AREA,,41,50
MAT,1
MSHKEY,1
ESIZE,30
AESIZE,30
AMESH,ALL

ASEL,S,AREA,,51,60
MAT,2
MSHKEY,1
ESIZE,30
AESIZE,30
AMESH,ALL
```

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ALLSEL,ALL
EPLOT

NUMMRG,NODES
NUMMRG,ELEM
NUMMRG,KP
NUMMRG,TYPE
NUMMRG,REAL
NUMMRG,CP
NUMMRG,CE

SELTOL,0.01
NSEL,S,LOC,z,0
D,ALL,UZ,0
D,ALL,ROTX,0
D,ALL,ROTY,0
ALLSEL,ALL
EPLOT

NSEL,S,LOC,Y,0
NSEL,R,LOC,Z,1500
D,ALL,UY,0
ALLSEL,ALL
EPLOT
```

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NSEL,S,LOC,X,140
NSEL,R,LOC,Y,0
D,ALL,UX,0
D,ALL,ROTY,0
D,ALL,ROTZ,0

NSEL,S,LOC,X,0
NSEL,R,LOC,Y,147
D,ALL,UX,0
D,ALL,ROTY,0
D,ALL,ROTZ,0
ALLSEL,ALL
EPLOT

ASEL,S,AREA,,20
ASEL,A,AREA,,40
NSLA,S,1
NPLOT
F,ALL,FY,-15
ALLSEL,ALL
EPLOT
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NUMMRG,NODES
NUMMRG,ELEM
NUMMRG,KP
NUMMRG,TYPE
NUMMRG,REAL
NUMMRG,CP
NUMMRG,CE

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TB,BISO,1,1
TBTEMP,0
TBDATA,1,327,2100

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ALLSEL,ALL
EPLOT
ANTYPE,STATIC,NEW
NLGEOM,ON
NSUBST,50,
ARCLEN,ON,2,0.001
OUTRES,ALL,ALL
SOLVE
FINISH
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10. Macro H 55

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FINI

/CLEAR

/PREP7

ET,1,281,KOP3

R,1,0.777

MP,EX,1,2.1E5

MP,NUXY,1,0.3


KEYPTS

K,1,0,52

K,2,20.6616,52

K,3,48.7653,-2

K,4,77.2985,-2

K,5,87,0


L,1,2

L,2,3

L,3,4

L,4,5




K,100,87,0,325

K,101,87,0,425
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K,102,87,0,1075
K,103,87,0,1175
K,104,87,0,1450
K,105,87,0,1500

L,5,100
L,100,101
L,101,102
L,102,103
L,103,104
L,104,105

ADRAG,1,2,3,4,,,,,5
ADRAG,11,14,16,18,,,,,6

ADRAG,20,23,25,27,,,,,7
ADRAG,29,32,34,36,,,,,8

ADRAG,38,41,43,45,,,,,9
ADRAG,47,50,52,54,,,,,10

ASEL,ALL
/VIEW,1,1,1,1
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NUMMRG,ALL


MP,EX,2,2.1E5
MP,NUXY,2,0.3


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MSHKEY,1
ESIZE,5
AESIZE,5
AMESH,ALL


ASEL,S,AREA,,5,8
MAT,2
MSHKEY,1
ESIZE,15
AESIZE,15
AMESH,ALL


ASEL,S,AREA,,9,12
MAT,1
MSHKEY,1
ESIZE,15
AESIZE,15
AMESH,ALL
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ASEL,S,AREA,,13,16
MAT,2
MSHKEY,1
ESIZE,15
AESIZE,15
AMESH,ALL

ASEL,S,AREA,,17,20
MAT,1
MSHKEY,1
ESIZE,30
AESIZE,30
AMESH,ALL

ASEL,S,AREA,,21,24
MAT,2
MSHKEY,1
ESIZE,30
AESIZE,30
AMESH,ALL

ALLSEL,ALL
EPLOT
```

```
NUMMRG,NODES
NUMMRG,ELEM
NUMMRG,KP
NUMMRG,TYPE
NUMMRG,REAL
NUMMRG,CP
NUMMRG,CE

SELTOL,0.01
NSEL,S,LOC,z,0
D,ALL,UZ,0
D,ALL,ROTX,0
D,ALL,ROTY,0
ALLSEL,ALL
EPLOT

NSEL,S,LOC,Y,-2
NSEL,R,LOC,Z,1500
D,ALL,UY,0
ALLSEL,ALL
EPLOT

NSEL,S,LOC,Y,52
NSEL,R,LOC,Z,1500
D,ALL,UY,0
```

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ALLSEL,ALL
EPLOT

NSEL,S,LOC,X,87
NSEL,R,LOC,Y,0
D,ALL,UX,0
D,ALL,ROTY,0
D,ALL,ROTZ,0

NSEL,S,LOC,X,0
NSEL,R,LOC,Y,52
D,ALL,UX,0
D,ALL,ROTY,0
D,ALL,ROTZ,0
ALLSEL,ALL
EPLOT

ASEL,S,AREA,,7
ASEL,A,AREA,,15
NSLA,S,1
NPLOT
F,ALL,FY,-10
ALLSEL,ALL
EPLOT
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TB,BISO,1,1
TBTEMP,0
TBDATA,1,337.7,2100

/SOLU
ALLSEL,ALL
EPLOT
ANTYPE,STATIC,NEW
NLGEOM,ON
NSUBST,50,
ARCLEN,ON,2,0.001
OUTRES,ALL,ALL
SOLVE
FINISH
```

11. Macro H 68

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FINI
/CLEAR
/PREP7
ET,1,281,KOP3
R,1,0.85
MP,EX,1,2.1E5
MP,NUXY,1,0.3
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K,1,0,68

K,2,23.64,68

K,3,33.61,64.46

K,4,43.58,68

K,5,67.93,68

K,6,77.19,32.53

K,7,81.54,28.55

K,8,85.2,14.54

K,9,88.84,10.13

K,10,91.53,0

K,11,106,0


L,1,2

L,2,3

L,3,4

L,4,5

L,5,6

L,6,7

L,7,8

L,8,9

L,9,10

L,10,11


K,100,106,0,270
```

```
K,101,106,0,480

K,102,106,0,1020

K,103,106,0,1230

K,104,106,0,1395

K,105,106,0,1500


L,11,100

L,100,101

L,101,102

L,102,103

L,103,104

L,104,105


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ADRAG,7,8,9,10,,,,,11



ADRAG,17,20,22,24,26,28,,12

ADRAG,30,33,35,37,,,,,12


ADRAG,39,42,44,46,48,50,,13

ADRAG,52,55,57,59,,,,,13


ADRAG,61,64,66,68,70,72,,14

ADRAG,74,77,79,81,,,,,14
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ADRAG,83,86,88,90,92,94,,15
ADRAG,96,99,101,103,,,,,15

ADRAG,105,108,110,112,114,116,,16
ADRAG,118,121,123,125,,,,,16


ASEL,ALL
/VIEW,1,1,1,1
NUMMRG,ALL

MP,EX,2,2.1E5
MP,NUXY,2,0.3

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MAT,1
MSHKEY,1
ESIZE,5
AESIZE,5
AMESH,ALL

ASEL,S,AREA,,11,20
MAT,2
```

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MSHKEY,1
ESIZE,15
AESIZE,15
AMESH,ALL

ASEL,S,AREA,,21,30
MAT,1
MSHKEY,1
ESIZE,15
AESIZE,15
AMESH,ALL

ASEL,S,AREA,,31,40
MAT,2
MSHKEY,1
ESIZE,15
AESIZE,15
AMESH,ALL

ASEL,S,AREA,,41,50
MAT,1
MSHKEY,1
ESIZE,30
AESIZE,30
AMESH,ALL
```

```
ASEL,S,AREA,,51,60
MAT,2
MSHKEY,1
ESIZE,30
AESIZE,30
AMESH,ALL

ALLSEL,ALL
EPLOT

NUMMRG,NODES
NUMMRG,ELEM
NUMMRG,KP
NUMMRG,TYPE
NUMMRG,REAL
NUMMRG,CP
NUMMRG,CE

SELTOL,0.01
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D,ALL,UZ,0
D,ALL,ROTX,0
D,ALL,ROTY,0
ALLSEL,ALL
```

```
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NSEL,R,LOC,Z,1500
D,ALL,UY,0
ALLSEL,ALL
EPLOT

NSEL,S,LOC,X,106
NSEL,R,LOC,Y,0
D,ALL,UX,0
D,ALL,ROTY,0
D,ALL,ROTZ,0

NSEL,S,LOC,X,0
NSEL,R,LOC,Y,68
D,ALL,UX,0
D,ALL,ROTY,0
D,ALL,ROTZ,0
ALLSEL,ALL
EPLOT

ASEL,S,AREA,,20
ASEL,A,AREA,,40
NSLA,S,1
```

```
NPLOT
F,ALL,FY,-15
ALLSEL,ALL
EPLOT

NUMMRG,NODES
NUMMRG,ELEM
NUMMRG,KP
NUMMRG,TYPE
NUMMRG,REAL
NUMMRG,CP
NUMMRG,CE

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UPGEOM,7.08,1,1,file,RST
TB,BISO,1,1
TBTEMP,0
TBDATA,1,280,2100

/SOLU
ALLSEL,ALL
EPLOT
ANTYPE,STATIC,NEW
NLGEOM,ON
NSUBST,50,
```

ARCLEN,ON,2,0.001

OUTRES,ALL,ALL

SOLVE

FINISH

12. Macro EUROCOL 60

FINI

/CLEAR

/PREP7

ET,1,281,KOP3

R,1,0.75

MP,EX,1,2.1E5

MP,NUXY,1,0.3

KEYPTS

K,1,0,53.37

K,2,10,60

K,3,19.59,60

K,4,26.59,58

K,5,33.59,60

K,6,41.59,60

K,7,73.5,0

K,8,81.5,0

K,9,88.5,2

K,10,95.5,0

K,11,102.5,0

L,1,2

L,2,3

L,3,4

L,4,5

L,5,6

L,6,7

L,7,8

L,8,9

L,9,10

L,10,11

K,100,102.5,0,325

K,101,102.5,0,425

K,102,102.5,0,1075

K,103,102.5,0,1175

K,104,102.5,0,1450

K,105,102.5,0,1500

L,11,100

L,100,101

L,101,102

L,102,103

L,103,104

L,104,105

```
ADRAG,1,2,3,4,5,6,,11
ADRAG,7,8,9,10,,,,,11



ADRAG,17,20,22,24,26,28,,12
ADRAG,30,33,35,37,,,,,12

ADRAG,39,42,44,46,48,50,,13
ADRAG,52,55,57,59,,,,,13

ADRAG,61,64,66,68,70,72,,14
ADRAG,74,77,79,81,,,,,14



ADRAG,83,86,88,90,92,94,,15
ADRAG,96,99,101,103,,,,,15

ADRAG,105,108,110,112,114,116,,16
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MAT,1
MSHKEY,1
ESIZE,5
AESIZE,5
AMESH,ALL

ASEL,S,AREA,,11,20
MAT,2
MSHKEY,1
ESIZE,15
AESIZE,15
AMESH,ALL

ASEL,S,AREA,,21,30
MAT,1
MSHKEY,1
ESIZE,15
AESIZE,15
AMESH,ALL
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ASEL,S,AREA,,31,40
MAT,2
MSHKEY,1
ESIZE,15
AESIZE,15
AMESH,ALL

ASEL,S,AREA,,41,50
MAT,1
MSHKEY,1
ESIZE,30
AESIZE,30
AMESH,ALL

ASEL,S,AREA,,51,60
MAT,2
MSHKEY,1
ESIZE,30
AESIZE,30
AMESH,ALL

ALLSEL,ALL
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NUMMRG,ELEM
NUMMRG,KP
NUMMRG,TYPE
NUMMRG,REAL
NUMMRG,CP
NUMMRG,CE

SELTOL,0.01
NSEL,S,LOC,z,0
D,ALL,UZ,0
D,ALL,ROTX,0
D,ALL,ROTY,0
ALLSEL,ALL
EPLOT

NSEL,S,LOC,Y,0
NSEL,R,LOC,Z,1500
D,ALL,UY,0
ALLSEL,ALL
EPLOT

NSEL,S,LOC,X,102.5
NSEL,R,LOC,Y,0
D,ALL,UX,0
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D,ALL,ROTY,0

D,ALL,ROTZ,0

NSEL,S,LOC,X,0

NSEL,R,LOC,Y,53.37

D,ALL,UX,0

D,ALL,ROTY,0

D,ALL,ROTZ,0

ALLSEL,ALL

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ASEL,S,AREA,,20

ASEL,A,AREA,,40

NSLA,S,1

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ALLSEL,ALL

EPLOT

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NSLA,S,1

NPLOT

F,ALL,FY,-15

ALLSEL,ALL

```
EPLOT



NUMMRG,NODES
NUMMRG,ELEM
NUMMRG,KP
NUMMRG,TYPE
NUMMRG,REAL
NUMMRG,CP
NUMMRG,CE

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TB,BISO,1,1
TBTEMP,0
TBDATA,1,369,2100

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ALLSEL,ALL
EPLOT
ANTYPE,STATIC,NEW
NLGEOM,ON
NSUBST,50,
ARCLEN,ON,2,0.001
OUTRES,ALL,ALL
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SOLVE

FINISH