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CALCULATION SHEET

Job No.		Sheet 1 of 7	Rev A
Job Title	RFCS Stainless Steel Valorisation Project		
Subject	Design Example 12 – Design of a lipped channel in a exposed floor		
Client RFCS	Made by ER/EM	Date Feb 2006	
	Checked by HB	Date March 2006	

DESIGN EXAMPLE 12 – DESIGN OF A LIPPED CHANNEL IN AN EXPOSED FLOOR

Design a simply supported beam with a lipped channel in an exposed floor. The material is stainless steel grade 1.4401 CP500, i.e. cold worked to a yield strength of 500 N/mm^2. The beam is simply supported with a span, l of 4 m. The distance between adjacent beams is 1 m.

As the load is not applied through the shear centre of the channel, it is necessary to check the interaction between the torsional resistance of the cross-section and the lateral torsional buckling resistance of the member. However, this example only checks the lateral torsional buckling resistance of the member.

Factors

Partial factor $\gamma_{M0} = 1,1$ and $\gamma_{M1} = 1,1$ — Table 2.1

Load factor $\gamma_G = 1,35$ (permanent loads) and $\gamma_Q = 1,5$ (variable loads) — Section 2.3.2

Actions

Permanent actions (G): 2 kN/m^2

Variable actions (Q): 3 kN/m^2

Load case to be considered in the ultimate limit state:

$$q^* = \sum_j \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} = 7,2 \text{ kN/m}$$ — Eq. 2.3

Structural Analysis

Reactions at support points (Design shear force)

$$V_{Ed} = \frac{q^* \times 4}{2} = 14,4\ kN$$

Design bending moment

$$M_{Ed} = \frac{q^* \times 4^2}{8} = 14,4\ kNm$$

Material Properties

Yield strength	f_y = 500 N/mm^2	Table 3.5
Modulus of elasticity	E = 200 000 N/mm^2	Section 3.2.4
Shear modulus	G = 76900 N/mm^2	Section 3.2.4

Cross-section Properties

The influence of rounded corners on cross-section resistance may be neglected if the internal radius $r \leq 5t$ and $r \leq 0,10b_p$ and the cross section may be assumed to consist of plane elements with sharp corners. For cross-section stiffness properties the influence of rounded corners should always be taken into account. — Section 4.6.2

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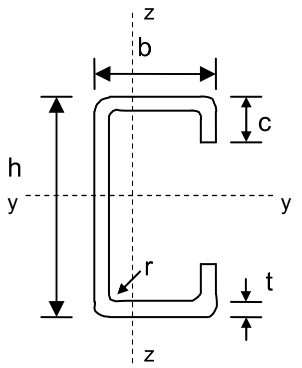

h = 160 mm
b = 125 mm
c = 30 mm
t = 5 mm
r = 5 mm

$b_p = b - t - 2g_r = 115{,}6$ mm — Figure 4.5

$g_r = r_m[\tan(\phi/2) - \sin(\phi/2)] = 2{,}2$ mm

$r_m = r + t/2 = 7{,}5$ mm

$r = 5 \text{ mm} \le 5t = 25$ mm

$r = 5 \text{ mm} \le 0{,}10 b_p = 11{,}56$ mm

The influence of rounded corners on section properties may be taken into account with sufficient accuracy by reducing the properties calculated for an otherwise similar cross-section with sharp corners, using the following approximations:

Notional flat width of the flange, $b_{p,f} = b - t - 2g_r = 115{,}61$ mm

Notional flat width of the web, $b_{p,w} = h - t - 2g_r = 150{,}61$ mm

Notional flat width of the lip, $b_{p,l} = c - t/2 - g_r = 25{,}30$ mm

$A_{g,sh} = 2162 \text{ mm}^2$

$I_{yg,sh} = 9{,}069 \times 10^6 \text{ mm}^4$

$\delta = 0{,}43 \sum_{j=1}^{n} r_j \frac{\phi_j}{90^o} / \sum_{i=1}^{m} b_{p,i} = 0{,}02$ — Eq 4.21

$A_g = A_{g,sh}(1 - \delta) = 2119 \text{ mm}^2$ — Eq 4.18

$I_g = I_{g,sh}(1 - 2\delta) = 8{,}708 \times 10^6 \text{ mm}^4$ — Eq 4.19

Classification of the cross-section — Section 4.3

Table 4.2

$\varepsilon = \left[\frac{235}{f_y}\frac{E}{210000}\right]^{0,5} = 0{,}669$

Flange: Internal compression parts. Part subjected to compression.

$c = b_p = b - t - 2g_r = 115{,}6$ mm

c/t=23,12 > 30,7 ε, therefore the flanges are Class 4

Web: Internal compression parts. Part subjected to bending.

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$c = h - t - 2g_r = 150{,}6$ mm

$c/t = 30{,}12 \leq 56\,\varepsilon$, therefore the web is Class 1

Lip: Outstand flanges. Part subjected to compression, tip in compression,

$c = c - t/2 - g_r = 25{,}30$ mm

$c/t = 5{,}06 \leq 10\,\varepsilon$, therefore the lip is Class 1

Calculation of the effective section properties — Section 4.4.1

Flange effective width: Internal compression elements. Part subjected to compression. — Table 4.3

$\bar{b} = b_p = b - t - 2g_r = 115{,}6$ mm

Assuming uniform stress distribution in the compression flange:

$\psi = \dfrac{\sigma_2}{\sigma_1} = 1$ and the buckling factor $k_\sigma = 4$

$$\bar{\lambda}_p = \frac{\bar{b}/t}{28{,}4\varepsilon\sqrt{k_\sigma}} = 0{,}608$$ Eq 4.2

Cold formed internal elements: $\rho = \dfrac{0{,}772}{\bar{\lambda}_p} - \dfrac{0{,}125}{\bar{\lambda}_p^2} = 0{,}9311 < 1$

$b_{eff} = \rho\bar{b} = 107{,}64$ mm , $b_{e1} = 0{,}5b_{eff} = 53{,}82$ mm, $b_{e2} = 0{,}5b_{eff} = 53{,}82$ mm

Effects of shear lag — Section 4.4.2

Shear lag in flanges may be neglected if $b_0 < L_e/50$, where b_0 is taken as the flange outstand or half the width of an internal element and L_e is the length between points of zero bending moment.

For internal elements: $b_o = (b-t)/2 = 60$ mm

The length between points of zero bending moment: $L_e = 4000$ mm, $L_e/50 = 80$ mm

Therefore shear lag can be neglected

Flange curling — Section 4.4.3; prEN 1993-1-3, clause 5.4

$$u = 2\frac{\sigma_a^2}{E^2}\frac{b_s^4}{t^2 z} = 2{,}55\,\text{mm}$$

$b_s = 141$ mm is the distance between webs

$t = 5$mm

$z = 77{,}5$ mm is the distance of the flange under consideration from neutral axis

σ_a is mean stress in the flanges calculated with gross area ($f_y = 500$ N/mm^2 is assumed)

Flange curling can be neglected if it is less than 5% of the depth of the profile cross-section:

$u = 2{,}55$ mm $< 0{,}05h = 8$ mm, therefore flange curling can be neglected.

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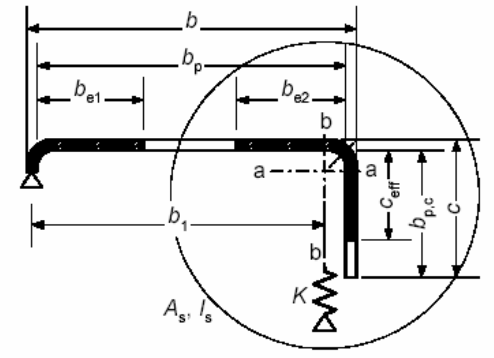
Stiffened elements. Edge stiffeners
Distortional buckling. Plane elements with edge stiffeners

Section 4.5.1 and prEN 1993-1-3, clause 5.5.3



$b/t \leq 60$
a) single edge fold

Step 1: Initial effective cross-section for the stiffener

prEN 1993-1-3, clause 5.5.3.2

For flanges (as calculated before)

$b=125$ mm
$b_p=115{,}61$ mm
$b_{eff}=107{,}65$ mm
$b_{e1}=0{,}5b_{eff}=53{,}82$ mm
$b_{e2}=0{,}5b_{eff}=53{,}82$ mm

For the lip, the effective width c_{eff} should be calculated using the corresponding buckling factor k_σ, $\bar{\lambda}_p$ and ρ expressions as follows:

$b_{p,c}=c-t/2-g_r=25{,}30$ mm

$b_p=115{,}6$ mm

$b_{p,c}/b_p=0{,}22 < 0{,}35$ then $k_\sigma=0{,}5$ — prEN 1993-1-3, Eq. 5.13b

$$\bar{\lambda}_p = \frac{\bar{b}/t}{28{,}4\varepsilon\sqrt{k_\sigma}} = 0{,}45 \qquad (\bar{b} = 30 \text{ mm})$$ Eq 4.2

Cold formed outstand elements: $\rho = \frac{1}{\bar{\lambda}_p} - \frac{0{,}231}{\bar{\lambda}_p^2} = 1{,}08 > 1$ then $\rho=1$ — Eq 4.1b

$c_{eff}=\rho b_{p,c}=25{,}30$ mm — prEN 1993-1-3, Eq. 5.13a

Step 2: Reduction factor for distortional buckling

Calculation of geometric properties of effective edge stiffener section

$b_{e2}=53{,}82$ mm

$c_{eff}=25{,}30$ mm

$A_s=(b_{e2}+c_{eff})t=395{,}64$ mm^2

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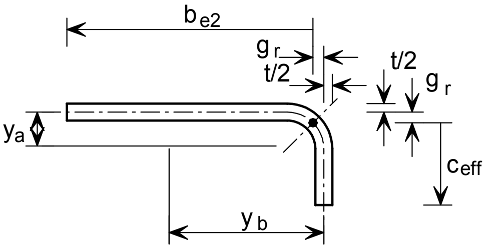

prEN 1993-1-3, Fig. 5.9

y_a=4,01 mm
y_b=18,27 mm

I_s=21211,8 mm^4

Calculation of linear spring stiffness

$$K_1 = \frac{Et^3}{4(1-\upsilon^2)}\frac{1}{b_1^2 h_w + b_1^3 + 0{,}5 b_1 b_2 h_w k_f} = 2{,}487\ \text{N/mm}^2$$

prEN 1993-1-3, Eq. 5.10b

b_1=b-y_b-t/2=104,23 mm (the distance from the web-to-flange junction to the gravity center of the effective area of the edge stiffener, including the efficient part of the flange b_{e2})

k_f=0 (flange 2 is in tension)

h_w=150 mm is the web depth

Elastic critical buckling stress for the effective stiffener section

$$\sigma_{cr,s} = \frac{2\sqrt{KEI_s}}{A_s} = 519{,}195\ \text{N/mm}^2$$

prEN 1993-1-3, Eq. 5.15

Reduction factor χ_d for distortional buckling

$$\bar{\lambda}_d = \sqrt{f_{yb}/\sigma_{cr,s}} = 0{,}98$$

$0{,}65 < \bar{\lambda}_d < 1{,}38$ then $\chi_d = 1{,}47 - 0{,}723\bar{\lambda}_d = 0{,}76$

prEN 1993-1-3, Eq. 5.12d

Reduced area and thickness of effective stiffener section

$$A_{s,red} = \chi_d A_s \frac{f_{yb}/\gamma_{M0}}{\sigma_{com,Ed}} = 300{,}88\ \text{mm}^2$$

prEN 1993-1-3, Eq. 5.17

t_{red}=t$A_{s,red}$/A_s=3,8 mm

Calculation of effective section properties with distortional buckling effect

$A_{eff,sh}$=2028 mm^2

$$\delta = 0{,}43\sum_{j=1}^{n} r_j \frac{\phi_j}{90^o} / \sum_{i=1}^{m} b_{p,i} = 0{,}02$$

Eq 4.21

A_{eff} = $A_{eff,sh}$ (1 - δ) = 1987 mm^2

Eq 4.18

$z_G = 68{,}98$ mm (distance from the bottom fibre to the neutral axis)

$I_{y,eff,sh} = 8{,}274\times10^6$ mm^4

$I_{y,eff} = I_{y,eff,sh}\,(1 - 2\delta) = 7{,}943\times10^6$ mm^4 — Eq 4.19

$W_{y,eff,\,sup} = 92{,}34\times10^3$ mm^3

$W_{y,eff,\,inf} = 115{,}2\times10^3$ mm^3

Resistance of cross-section — Section 4.7

Cross-section subject to bending moment — Section 4.7.4

$M_{c,Rd} = W_{y,eff,min} f_y / \gamma_{M0} = 41{,}97$ kNm for Class 4 cross-section — Eq. 4.29

Design bending moment $M_{Ed} = 14{,}4$ kNm

Cross-section moment resistance is Ok

Cross-section subject to shear — Section 4.7.5

$V_{pl,Rd} = A_v\left(f_y/\sqrt{3}\right)/\gamma_{M0} = 209{,}95$ kN — Eq. 4.30

$A_v = 800$ mm^2 is the shear area

Design shear force $V_{Ed} = 14{,}4$ kN

Cross-section shear resistance is Ok

Cross-section subjected to combination of loads — Section 4.7.6

$V_{Ed} = 14{,}4$ kN $> 0{,}5V_{pl,Rd} = 104{,}97$ kN

There is no interaction between bending moment and shear force

Flexural members — Section 5.4

Lateral-torsional buckling — Section 5.4.2

$M_{b,Rd} = \chi_{LT} W_{y,eff,sup} f_y / \gamma_{M1}$ for Class 4 cross-section — Eq 5.8

$$\chi_{LT} = \frac{1}{\varphi_{LT} + \left[\varphi_{LT}^2 - \bar{\lambda}_{LT}^2\right]^{0,5}} \leq 1$$ — Eq 5.9

$$\varphi_{LT} = 0{,}5\left(1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0{,}4\right) + \bar{\lambda}_{LT}^2\right)$$ — Eq 5.10

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{y,eff} f_y}{M_{cr}}}$$ — Eq 5.11

$\alpha_{LT} = 0{,}34$ for cold formed sections

Determination of the elastic critical moment for lateral-torsional buckling

$$M_{cr} = C_1\frac{\pi^2 EI_z}{(k_z L)^2}\left(\left[\left(\frac{k_z}{k_w}\right)^2\frac{I_w}{I_z} + \frac{(k_z L)^2 GI_t}{\pi^2 EI_z} + \left(C_2 z_g - C_3 z_j\right)^2\right]^{1/2} - \left(C_2 z_g - C_3 z_j\right)\right)$$ — Appendix B, Section B.1

For simply supported beams with uniform distributed load: $C_1=1,12$, $C_2=0,45$ and $C_3=0,525$.

Assuming normal conditions of restraint at each end: $k_z=k_w=1$

$z_j=0$ for equal flanged section

$z_g = z_a - z_s = h/2 = 80$ mm

z_a is the co-ordinate of point load application

z_s is the co-ordinate of the shear centre

$y_G=45,34$ mm (distance from the central axis of the web to the gravity centre)

$I_{z,sh}=4,274\times10^6$ mm^4

$I_{t,sh}=18,02\times10^3$ mm^4

$I_{w,sh}=23,19\times10^9$ mm^6

$I_z=I_{z,sh}\ (1 - 2\delta) = 4,103\times10^6$ mm^4

$I_t= I_{t,sh}\ (1 - 2\delta) =17,30\times10^3$ mm^4

$I_w= I_{w,sh}\ (1 - 4\delta) =21,33\times10^9$ mm^6

Note: The expression used to determine the warping torsion is obtained from Wei-Wen You, "Cold-Formed Steel Design", Appendix B-Torsion

$$\text{Then, } M_{cr}=C_1\frac{\pi^2 EI_z}{(k_zL)^2}\left(\left[\left(\frac{k_z}{k_w}\right)^2\frac{I_w}{I_z}+\frac{(k_zL)^2GI_t}{\pi^2EI_z}+\left(C_2z_g\right)^2\right]^{1/2}-\left(C_2z_g\right)\right)=33,74\text{ kNm}$$

$$\bar{\lambda}_{LT}=\sqrt{\frac{W_{y,eff,sup}f_y}{M_{cr}}}=1,17 \qquad (W_{y,eff,\,sup}=92,39\times10^3\text{ mm}^3\text{, compression flange})$$

$$\varphi_{LT}=0,5\left(1+\alpha_{LT}\left(\bar{\lambda}_{LT}-0,4\right)+\bar{\lambda}_{LT}^{\,2}\right)=1,315$$

$$\chi_{LT}=\frac{1}{\varphi_{LT}+\left[\varphi_{LT}^{\,2}-\bar{\lambda}_{LT}^{\,2}\right]^{0,5}}=0,522$$

$$M_{b,Rd}=\chi_{LT}W_{y,eff,sup}f_y/\gamma_{M1}=21,91\text{ kNm}$$

Design moment $M_{Ed}=14,4$ kNm, therefore lateral torsional buckling resistance Ok

Note: As the load is not applied through the shear centre of the channel, it is also necessary to check the interaction between the torsional resistance of the cross-section and the lateral torsional buckling resistance of the member.

Shear buckling resistance Section 5.4.3

The shear buckling resistance only requires checking when $h_w/t \geq 52\varepsilon/\eta$ for an unstiffened web.

The recommended value for $\eta = 1,20$

$h_w/t = 28$, $52\varepsilon/\eta = 28,99$, therefore no further check required.

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