



**The Steel  
Construction  
Institute**

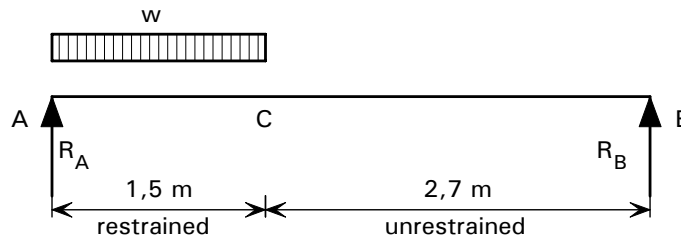
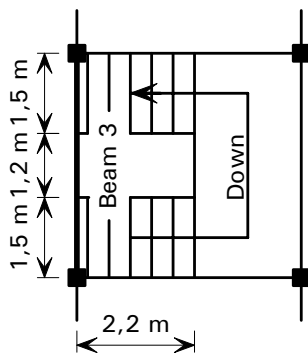
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**CALCULATION SHEET**

Job No.	OSM 466	Sheet	1 of 8	Rev	B
Job Title	ECSC Stainless Steel Valorisation Project				
Subject	Design Example 9 – Beam with unrestrained compression flange				
Client ECSC	Made by	SMH	Date	Aug 2001	
	Checked by	NRB	Date	Dec 2001	
	Revised by	MEB	Date	April 2006	

**DESIGN EXAMPLE 9 - BEAM WITH UNRESTRAINED COMPRESSION FLANGE**

Design a staircase support beam. The beam is a single section channel, simply supported between columns. The flight of stairs lands between A and C and provides restraint to the top flange of this part of the beam. The top flange is unrestrained between B and C. The overall span of the beam is taken as 4,2 m.



**Actions**

Assuming the beam carries the load from the first run of stairs to the landing only:

Permanent actions (*G*): Load on stairs  $1,0 \text{ kN/m}^2 = (1,0 \times 2,2) = 2,2 \text{ kN/m}$   
Self weight of beam  $0,13 \text{ kN/m}$

Variable actions (*Q*): Load on stairs  $4 \text{ kN/m}^2 = (4,0 \times 2,2) = 8,8 \text{ kN/m}$

Load case to be considered (ultimate limit state):

$$\sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \psi_{0,i} Q_{k,i}$$

Eqn. 2.3

As there is only one variable action ( $Q_{k,1}$ ) the last term in the above expression does not need to be considered in this example.

$$\gamma_{G,j} = 1,35 \text{ (unfavourable effects)}$$

$$\gamma_{Q,1} = 1,5$$

Section 2.3.2

Factored actions:

Permanent action: Load on stairs =  $1,35 \times 2,2 = 2,97 \text{ kN/m}$

Self weight of beam =  $1,35 \times 0,13 = 0,17 \text{ kN/m}$

Variable action Load on stairs =  $1,5 \times 8,8 = 13,2 \text{ kN/m}$

**Structural analysis**

Reactions at support points

$$R_A + R_B = (2,97 + 13,2) \times 1,5 + 0,17 \times 4,2 = 24,97 \text{ kN}$$



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Taking moments about *A*

$$R_B = \frac{1,5 \times 16,17 \times 0,75 + 0,17 \times 4,2 \times (4,2/2)}{4,2} = 4,69 \text{ kN}$$

$$\Rightarrow R_A = 24,97 - 4,69 = 20,28 \text{ kN}$$

Maximum bending moment occurs at a distance  $1,5 \left( 1 - \frac{1,5}{2 \times 4,2} \right) = 1,23 \text{ m}$  from *A*

$$M_{\max} = 20,28 \times 1,23 - 16,17 \times \frac{1,23^2}{2} - 0,17 \times \frac{1,23^2}{2} = 12,58 \text{ kNm}$$

Maximum shear occurs at *A*

$$F_{Sd} = 20,28 \text{ kN}$$

**Material properties**

Use material grade 1.4401

0,2% proof stress = 220 N/mm<sup>2</sup>

Take  $f_y$  as the 0,2% proof stress = 220 N/mm<sup>2</sup>

$E = 200\,000 \text{ N/mm}^2$  and  $G = 76\,900 \text{ N/mm}^2$

Table 3.1  
Section 3.2.4  
Section 3.2.4

Try a 200 × 75 channel section, thickness = 5 mm

**Cross section properties**

$$I_y = 9,456 \times 10^6 \text{ mm}^4 \quad W_{el,y} = 94,56 \times 10^3 \text{ mm}^3$$

$$I_z = 0,850 \times 10^6 \text{ mm}^4 \quad W_{pl,y} = 112,9 \times 10^3 \text{ mm}^3$$

$$I_w = 5085 \times 10^6 \text{ mm}^4 \quad A_g = 1650 \text{ mm}^2$$

$$I_t = 1,372 \times 10^4 \text{ mm}^4$$

**Classification of the cross-section**

$$\varepsilon = 1,01$$

Assume conservatively that  $c = h - 2t = 200 - 10 = 190 \text{ mm}$  for web

Web subject to bending:  $\frac{c}{t} = \frac{190}{5} = 38$

For Class 1,  $\frac{c}{t} \leq 56\varepsilon$ , therefore web is Class 1

Outstand flange subject to compression:  $\frac{c}{t} = \frac{75}{5} = 15$

Table 4.2

Table 4.2

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For Class 3,  $\frac{c}{t} \leq 11,9\varepsilon = 12,0$ , therefore outstand flange is Class 4

Therefore, overall classification of cross-section is Class 4

**Calculation of effective section properties**

Calculate reduction factor  $\rho$  for cold formed outstand elements

$$\rho = \frac{1}{\lambda_p} - \frac{0,231}{\lambda_p^2} \quad \text{but } \leq 1 \quad \text{Eq. 4.1b}$$

$$\bar{\lambda}_p = \frac{\bar{b}/t}{28,4\varepsilon\sqrt{k_\sigma}} \quad \text{where } \bar{b} = c = 75\text{mm} \quad \text{Eq. 4.2}$$

Assuming uniform stress distribution within the compression flange, Table 4.4

$$\psi = \frac{\sigma_2}{\sigma_1} = 1$$

$$\Rightarrow k_\sigma = 0,43 \quad \text{Table 4.4}$$

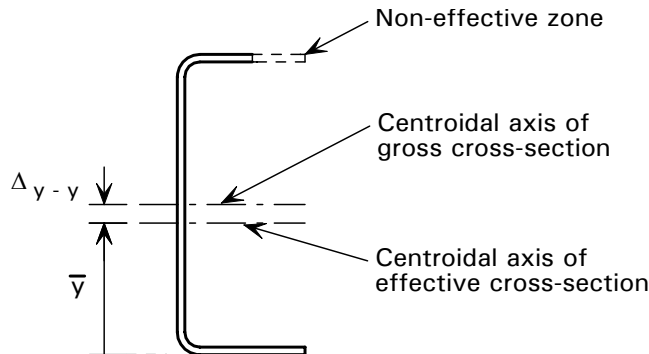
$$\bar{\lambda}_p = \frac{75/5}{28,4 \times 1,01 \times \sqrt{0,43}} = 0,797$$

$$\rho = \frac{1}{0,797} - \frac{0,231}{0,797^2} = 0,891$$

$$c_{\text{eff}} = 0,891 \times 75 = 66,8 \text{ mm} \quad \text{Table 4.4}$$

$$A_{\text{eff}} = A_g - (1 - \rho)ct = 1650 - (1 - 0,891) \times 75 \times 5 = 1609 \text{ mm}^2$$

Calculate shift of neutral axis of section under bending



$$\bar{y} = \frac{A_g \times \frac{h}{2} - (1 - \rho) \times c \times t \times \left( h - \frac{t}{2} \right)}{A_{\text{eff}}} = \frac{1650 \times \frac{200}{2} - (1 - 0,891) \times 75 \times 5 \times \left( 200 - \frac{5}{2} \right)}{1609}$$

$$\bar{y} = 97,53 \text{ mm}$$



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$$\text{Shift of neutral axis position, } \Delta_{y-y} = \frac{h}{2} - \bar{y} = \frac{200}{2} - 97,53 = 2,47 \text{ mm}$$

Calculate  $I_{\text{eff},y}$

$$I_{\text{eff},y} = \left( I_y - \frac{(1-\rho)ct^3}{12} - (1-\rho)ct \left( \frac{h}{2} - \frac{t}{2} \right)^2 - A_{\text{eff}} A_{y-y}^2 \right)$$

$$I_{\text{eff},y} = 9,456 \times 10^6 - \frac{(1-0,891) \times 75 \times 5^3}{12} - (1-0,891) \times 75 \times 5 \times (100 - 2,5)^2 - 1609 \times 2,47^2$$

$$= 9,06 \times 10^6 \text{ mm}^4$$

$$W_{\text{eff},y} = \frac{I_{\text{eff},y}}{\frac{h}{2} + \Delta_{y-y}} = \frac{9,06 \times 10^6}{\frac{200}{2} + 2,47} = 88,4 \times 10^3 \text{ mm}^3$$

**Shear lag**

Shear lag may be neglected provided that  $b_0 \leq L_e/50$  for outstand elements

$L_e$  = distance between points of zero moment = 4200 mm

$L_e/50 = 84 \text{ mm}$ ,  $b_0 = 75 \text{ mm}$ , therefore shear lag can be neglected

**Flange curling**

$$u = \frac{2\sigma_a^2 b_s^4}{E^2 t^2 z}$$

$\sigma_a$  = average longitudinal stress in flange = 220 N/mm<sup>2</sup> (maximum possible value)

$b_s = (75 - 5) = 70 \text{ mm}$

$z = (100 - 2,5) = 97,5 \text{ mm}$

$$\therefore u = \frac{2 \times 220^2 \times 70^4}{200000^2 \times 5^2 \times 97,5} = 0,024 \text{ mm}$$

Flange curling can be neglected if  $u < 0,05 \times 200 = 10 \text{ mm}$

Therefore flange curling is negligible

**Partial safety factors**

The following partial safety factors are used throughout the design example:

$$\gamma_{M0} = 1,1 \text{ and } \gamma_{M1} = 1,1$$

Section 4.4.2

Section 4.4.3  
prEN 1993-1-3:2004  
Clause 5.4(2)  
Eq. 5.3a

prEN 1993-1-3:2004  
Clause 5.4(1)

Table 2.1



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**Moment resistance of cross-section**

For a class 4 cross section

$$M_{c,Rd} = W_{eff,min} f_y / \gamma_{M0}$$

$$M_{c,Rd} = \frac{88,4 \times 10^3 \times 220}{1,1 \times 10^6} = 17,7 \text{ kNm}$$

Design moment = 12,58 kNm, ∴ cross-section moment resistance is OK

Section 4.7.4

Eq. 4.29

**Cross-section resistance to shear**

$$V_{pl,Rd} = A_v (f_y / \sqrt{3}) / \gamma_{M0}$$

$$A_v = h \times t = 200 \times 5 = 1000 \text{ mm}^2$$

$$V_{Rd} = \frac{1000 \times 220}{\sqrt{3} \times 1,1 \times 1000} = 115,5 \text{ kN}$$

Design shear force = 20,28 kN, therefore shear resistance of cross-section is OK

Check that shear resistance is not limited by shear buckling

Assume that  $h_w = h - 2t = 200 - 10 = 190 \text{ mm}$

$$\frac{h_w}{t} = \frac{190}{5} = 38, \text{ shear buckling resistance needs to be checked if } \frac{h_w}{t} \geq \frac{52\varepsilon}{\eta} = 43,2\varepsilon$$

∴ Shear resistance is not limited by shear buckling.

Section 4.7.5

Eq. 4.30

Section 5.4.3

**Resistance to lateral torsional buckling**

Compression flange of beam is laterally unrestrained between B and C. Check this portion of beam for lateral torsional buckling.

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

$$W_{eff,y} = 88,4 \times 10^3 \text{ mm}^3$$

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

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

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}}$$

Determine the elastic critical moment ( $M_{cr}$ )

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

Section 5.4.2

Eq. 5.8

Eq. 5.9

Eq. 5.10

Eq. 5.11

Appendix B

Section B.1



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$C$  is simply supported, while  $B$  approaches full fixity. Assume most conservative case:  $k_z = k_w = 1,0$ .

$C_1, C_2$  and  $C_3$  are determined from consideration of bending moment diagram and end conditions.

From bending moment diagram,  $\psi = 0$

$\Rightarrow C_1 = 1,77, C_2 = 0$  and  $C_3 = 1,00$

$z_j = 0$  for a cross-section with equal flanges

$$M_{cr} = 1,77 \times \frac{\pi^2 \times 200000 \times 0,850 \times 10^6}{(1,00 \times 2700)^2} \times \left( \left[ \left( \frac{1,00}{1,00} \right)^2 \frac{5085 \times 10^6}{0,850 \times 10^6} + \frac{(1,00 \times 2700)^2 \times 76900 \times 1,372 \times 10^4}{\pi^2 \times 200000 \times 0,850 \times 10^6} \right]^{0,5} \right)$$

$$M_{cr} = 41,9 \text{ kNm}$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{88,4 \times 10^3 \times 220}{41,9 \times 10^6}} = 0,68$$

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

$$\varphi = 0,5(1 + 0,34(0,68 - 0,4) + 0,68^2) = 0,779$$

$$\chi_{LT} = \frac{1}{0,779 + [0,779^2 - 0,68^2]^{0,5}} = 0,863$$

$$M_{b,Rd} = 0,863 \times 88,4 \times 10^3 \times 220 \times 10^{-6} / 1,1 = 15,3 \text{ kNm}$$

From bending moment diagram, maximum moment in unrestrained portion of beam = 12,0 kNm

Thus member has adequate resistance to lateral torsional buckling.

**Deflection**

Load case (serviceability limit state):  $\sum_{j \geq 1} G_{k,j} + Q_{k,1} + \sum_{i \geq 1} \psi_{0,i} Q_{k,i}$

As there is only one variable action ( $Q_{k,1}$ ) the last term in the above expression does not need to be considered in this example.

Secant modulus is used for deflection calculations - thus it is necessary to find the maximum stress due to unfactored permanent and variable actions.

Table B.1

Eq. 5.11

Section 5.4.2

Section 5.4.6

Eq. 2.8



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The secant modulus  $E_S = \left( \frac{E_{S1} + E_{S2}}{2} \right)$ ,

Where

$$E_{s,i} = \frac{E}{1 + 0,002 \frac{E}{\sigma_{i,Ed,ser}} \left( \frac{\sigma_{i,Ed,ser}}{f_y} \right)^n} \text{ and } i = 1,2$$

From structural analysis calculations the following were found:

- Maximum moment due to permanent actions = 1,90 kNm
- Maximum moment due to imposed actions = 6,68 kNm
- Total moment due to unfactored actions = 8,58 kNm

Section is Class 4, therefore  $W_{eff}$  is used in the calculations for maximum stress in the member.

Assume, conservatively that the stress in the tension and compression flange are approximately equal, i.e.  $E_{S1} = E_{S2}$ .

The following constants are used to determine the secant moduli:

For grade 1.4401 stainless steel,  $n$  (longitudinal direction) = 7,0

$$\text{Serviceability design stress, } \sigma_{i,Ed,ser} = \frac{M_{max}}{W_{eff,y}} = \frac{8,58 \times 10^6}{88,4 \times 10^3} = 97,1 \text{ N/mm}^2$$

$$E_{s,i} = \frac{200000}{1 + 0,002 \times \frac{200000}{97,1} \times \left( \frac{97,1}{220} \right)^7} = 197\,348 \text{ N/mm}^2$$

Maximum deflection due to patch loading occurs at a distance of approximately 1,9 m from support A.

Deflection at a distance  $x$  from support A due to patch load extending a distance  $a$  from support A is given by the following formulae:

$$\text{When } x \geq a \quad \delta = \frac{waL^4}{24aE_S I} n^2 \left[ 2m^3 - 6m^2 + m(4+n^2) - n^2 \right]$$

Where  $m = x/L$  and  $n = a/L$

When  $x = 1,9$  m, and  $a = 1,5$  m:  $m = 1,9/4,2 = 0,452$ ,  $n = 1,5/4,2 = 0,357$

Patch load (permanent+variable unfactored actions)  $w = 11,0$  kN/m

Uniform load (permanent action)  $w = 0,128$  kN/m

Deflection due to patch loads at a distance of 1,9 m from support A,  $\delta_1$

$$\begin{aligned} \delta_1 &= \frac{11000 \times 1,5 \times 4200^4}{24 \times 1500 \times 197348 \times 9,06 \times 10^6} \times \\ &\quad 0,357^2 \left[ 2 \times 0,452^3 - 6 \times 0,452^2 + 0,452(4 + 0,357^2) - 0,357^2 \right] \\ &= 7,09 \text{ mm} \end{aligned}$$

Appendix C

Table C.1

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Deflection at midspan due to self weight of beam,  $\delta_2$

$$\delta_2 = \frac{5}{384} \times \frac{(w \times L)L^3}{E_s I} = \frac{5}{384} \times \frac{(0,128 \times 10^3 \times 4,2) \times 4200^3}{197348 \times 9,06 \times 10^6} = 0,29 \text{ mm}$$

Total deflection  $\approx \delta_1 + \delta_2 = 7,09 + 0,29 = 7,38 \text{ mm}$

$$\delta_{\text{limiting}} = \frac{\text{span}}{250} = \frac{4200}{250} = 16,8 \text{ mm}$$

Therefore deflection is acceptably small.