Version JAB/2

EGT3
ENGINEERING TRIPOS PART IIB

Tuesday 5 May 2026 14.00 to 15.40

Module 4D10

STRUCTURAL STEELWORK

*Answer not more than **three** questions.*

All questions carry the same number of marks.

*The **approximate** percentage of marks allocated to each part of a question is indicated in the right margin.*

*Write your candidate number **not** your name on the cover sheet.*

STATIONERY REQUIREMENTS
Single-sided script paper

SPECIAL REQUIREMENTS TO BE SUPPLIED FOR THIS EXAM
CUED approved calculator allowed
Attachment: 4D10 Structural Steelwork data sheet (21 pages)
Engineering Data Book

10 minutes reading time is allowed for this paper at the start of the exam.

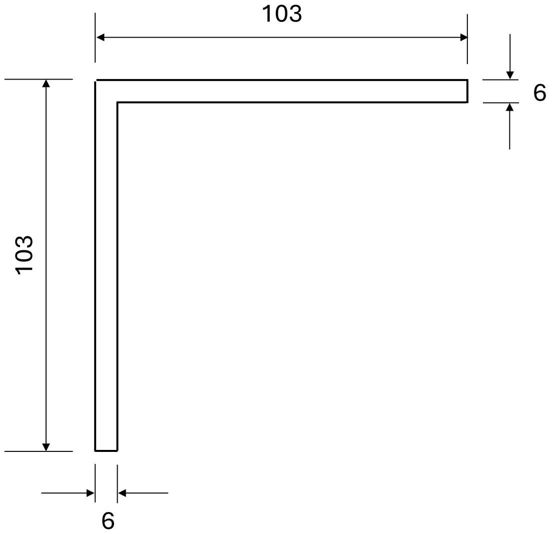
You may not start to read the questions printed on the subsequent pages of this question paper until instructed to do so.

You may not remove any stationery from the Examination Room.

1 An equal-leg angle profile is used as a 2 m long column. Its leg length is 103 mm and its thickness is 6 mm, as shown in Fig. 1. The steel grade is S355. The ends of the column are fixed against rotations about any axis, including twist.

(a) Identify all possible instabilities in the column. [20%]

(b) Determine the compressive capacity of the column. Note that columns with fixed ends do not undergo a shift of the effective centroid. [80%]



All dimensions in mm.

Figure 1

2 A column made of a linear elastic material is pin-ended and has length L, as shown in Fig. 2. The column is free of residual stresses, but has an initial geometric imperfection which is described by the following relationship:

$$v_0 = e_0 \sin\left(\frac{\pi x}{L}\right)$$

(a) Formulate a differential equation which describes the equilibrium of the column in its deformed state. [20%]

(b) Find a solution for the deflected shape $v(x)$ and the bending moments $M(x)$ along the column. [30%]

(c) Using the solution found in Part (b), derive the Perry-Robertson equations. [50%]

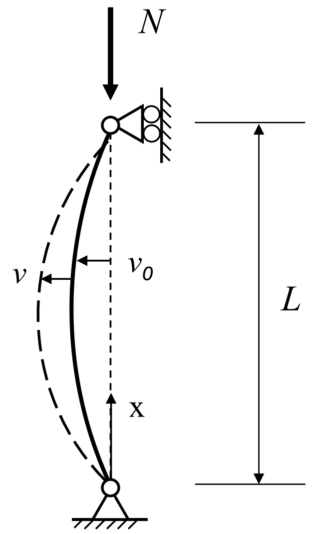


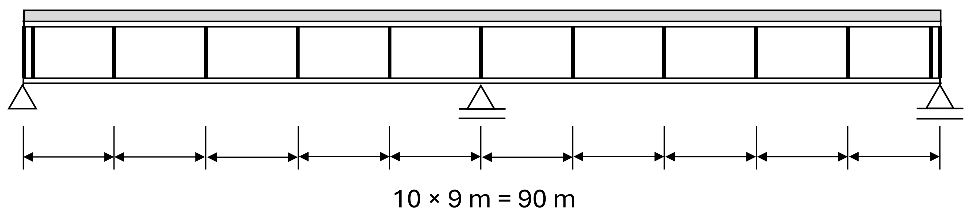
Figure 2

(TURN OVER

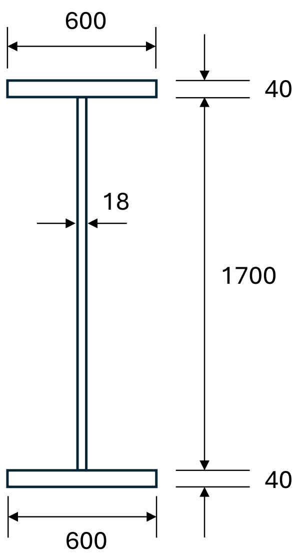
3 The steel bridge girder pictured in Fig. 3(a) is continuous over two 45 m long spans. The cross-section of the girder is shown in Fig. 3(b). Its torsion constant is $J = 2.89\times10^{7}$ mm^4, and its warping constant is $C_w = 1.09\times10^{15}$ mm^6. Note that the vertical scale in Fig. 3(a) is exaggerated to highlight the locations of the vertical stiffeners, which are indicated in thick black line. Cross-bracing to adjacent girders is provided at 9 m spacing, connected to the vertical stiffeners. The spacing between girder lines is 2.5 m. The girder acts compositely with a 250 mm thick concrete bridge deck. Traffic loads can be considered uniformly distributed, with a characteristic value of 5 kPa. The grade of steel used is S355 and the concrete class is C30/37.

(a) Check the girder at the ULS for bending at the location of the internal support. [50%]

(b) Check the girder at the ULS for bending at the critical location for sagging bending within the span. [50%]



(a)



(b) All dimensions in mm.

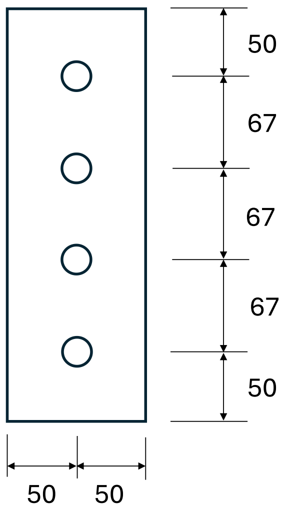
Figure 3

4 A UB 356×127×33 needs to be connected to a column, using a simple flag plate connection. The maintained design philosophy is that the connection cannot be the 'weak link', and that the connection needs to be able to transfer a force equivalent to the full shear capacity of the beam. The flag plate and the beam are made of grade S355 steel (f_u = 490 MPa). The dimensions of the flag plate, as well as the locations of the bolt holes, are shown in Fig. 4.

(a) Determine the minimum bolt size to be used in the connection, if bolts are grade 8.8. [30%]

(b) Determine the minimum thickness of the flag plate. [70%]

<u>Note:</u> The partial safety factor γ_{M2} may be taken as 1.1.



All dimensions in mm.

Figure 4

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4D10 Data Sheets

Do not use for practical design. Consult the Eurocodes instead.

DS1: Load combinations and partial safety factors

Persistent and transient design situations	Permanent actions		Leading variable action	Accompanying variable actions[a]	
	Unfavourable	Favourable		Main (if any)	Others
(Eq. 6.10)	$1{,}35G_{\mathrm{kj,sup}}$	$1{,}00G_{\mathrm{kj,inf}}$	$1{,}5Q_{\mathrm{k,1}}$		$1{,}5\psi_{0,1}Q_{\mathrm{k,i}}$

Table NA.A1.1 — Values of Ψ factors for buildings

Action	Ψ_0	Ψ_1	Ψ_2
Imposed loads in buildings, category (see EN 1991-1.1)			
Category A: domestic, residential areas	0,7	0,5	0,3
Category B: office areas	0,7	0,5	0,3
Category C: congregation areas	0,7	0,7	0,6
Category D: shopping areas	0,7	0,7	0,6
Category E: storage areas	1,0	0,9	0,8
Category F: traffic area, vehicle weight $\leq$ 30 kN	0,7	0,7	0,6
Category G: traffic area, 30 kN < vehicle weight $\leq$ 160 kN	0,7	0,5	0,3
Category H: roofs[a]	0,7	0	0
Snow loads on buildings (see EN 1991-3)			
— for sites located at altitude H > 1 000 m a.s.l.	0,70	0,50	0,20
— for sites located at altitude H $\leq$ 1 000 m a.s.l.	0,50	0,20	0
Wind loads on buildings (see EN 1991-1-4)	0,5	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0

[a] See also EN 1991-1-1: Clause 3.3.2 (1)

Partial safety factors on resistances:

- Resistance of cross-section: γ_{M0} = 1.0
- Resistance of member to buckling: γ_{M1} = 1.0
- Resistance against fracture: γ_{M2} = 1.1

DS2: Members in tension, compression and bending

	EN 1993-1-1	
Steel grade	Thickness range (mm)	Yield strength, f_y (N/mm^2)
S235	$t \leq 40$	235
	$40 < t \leq 80$	215
S275	$t \leq 40$	275
	$40 < t \leq 80$	255
S355	$t \leq 40$	355
	$40 < t \leq 80$	335

Members in tension

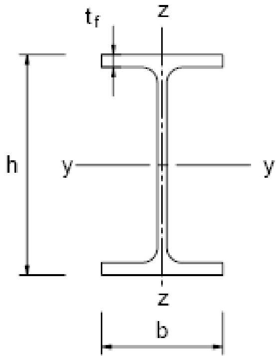
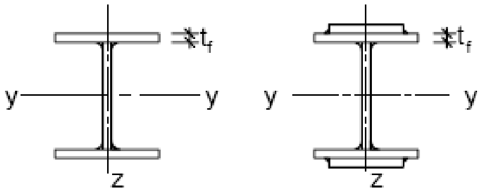
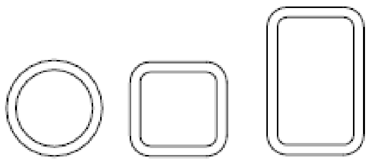
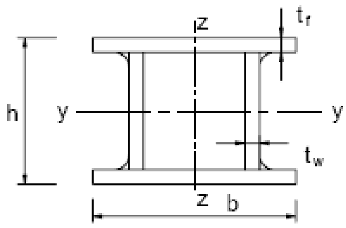
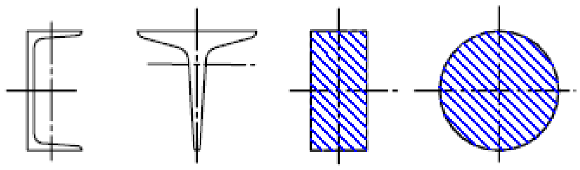
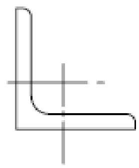
Shear lag effect in eccentric connections: reduction factor β

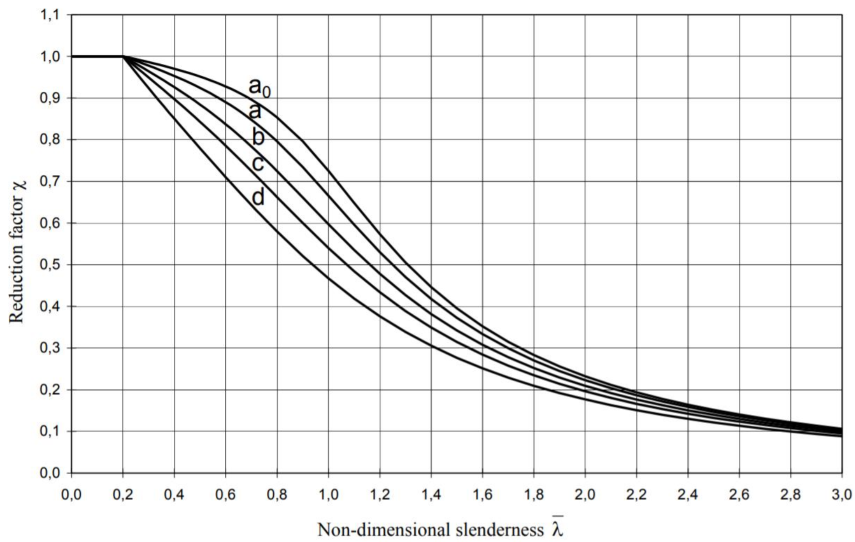
Pitch	p_1	$\leq 2{,}5\ d_o$	$\geq 5{,}0\ d_o$
2 bolts	β_2	0,4	0,7
3 bolts or more	β_3	0,5	0,7

Columns

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \qquad \phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right]$$

Buckling curve	a_0	a	b	c	d
Imperfection factor α	0,13	0,21	0,34	0,49	0,76

Cross section		Limits		Buckling about axis	Buckling curve	
					S 235 S 275 S 355 S 420	S 460
Rolled sections		$h/b > 1,2$	$t_f \le 40$ mm	y – y z – z	a b	a_0 a_0
			40 mm < $t_f \le 100$	y – y z – z	b c	a a
		$h/b \le 1,2$	$t_f \le 100$ mm	y – y z – z	b c	a a
			$t_f > 100$ mm	y – y z – z	d d	c c
Welded I-sections		$t_f \le 40$ mm		y – y z – z	b c	b c
		$t_f > 40$ mm		y – y z – z	c d	c d
Hollow sections		hot finished		any	a	a_0
		cold formed		any	c	c
Welded box sections		generally (except as below)		any	b	b
		thick welds: $a > 0,5t_f$ $b/t_f < 30$ $h/t_w < 30$		any	c	c
U-, T- and solid sections				any	c	c
L-sections				any	b	b



Equations of column stability:

$$EI_x v'' + P(v - x_0\phi) = 0$$

$$EI_y u'' + P(u + y_0\phi) = 0$$

$$EC_w\phi'''' - \left(GJ - P\frac{I_0}{A}\right)\phi'' + Py_0u'' - Px_0v'' = 0$$

Uncoupled solutions:

$$P_x = \pi^2\frac{EI_x}{L_{cr,x}^2} \qquad P_y = \pi^2\frac{EI_y}{L_{cr,y}^2} \qquad P_\phi = \frac{A}{I_0}\left(GJ + \pi^2\frac{EC_w}{L_{cr,\phi}^2}\right)$$

$$I_0 = I_x + I_y + A(x_0^2 + y_0^2)$$

For single-symmetric cross-sections:

$$P_{y,\phi} = \frac{1}{2\left(1 - {Ay_0^2}/{I_0}\right)}\left(P_y + P_\phi - \sqrt{\left(P_y - P_\phi\right)^2 + 4\frac{Ay_0^2}{I_0}P_yP_\phi}\right)$$

For asymmetric cross-sections:

$$\frac{(I_x + I_x)}{I_0}P^3 + \left[\frac{A}{I_0}\left(P_xy_0^2 + P_yx_0^2\right) - (P_x + P_x + P_x)\right]P^2 + \left(P_xP_y + P_xP_\phi + P_yP_\phi\right)P - P_xP_yP_\phi = 0$$

Torsion

Closed sections: $J = 4\dfrac{A_e^2}{\oint \frac{ds}{t}}$ (A_e = enclosed area; t = thickness; s = coordinate along the centreline)

Open sections: $J = \sum_i \dfrac{b_i t_i^3}{3}$ (b_i= width of plate i; t_i = thickness of plate i)

$C_w = I_w = \int_A \omega_n^2 \, dA$ (warping constant)

For I-beams: $C_w = I_y d^2/4$ (I_y= second moment of area about the minor axis; d = distance between the centrelines of the flanges)

$\omega_n = \omega - \omega_0$ (normalized sectorial coordinate)

$$\omega_0 = \frac{1}{A}\int_A \omega \, dA$$

$$\left.\begin{aligned} \int_A \omega_n x \, dA = 0 \\ \int_A \omega_n y \, dA = 0 \end{aligned}\right\} \qquad \left.\begin{aligned} x_0 = \frac{1}{I_x}\int_A \omega_c y \, dA \\ y_0 = \frac{1}{I_y}\int_A \omega_c x \, dA \end{aligned}\right\}$$ (shear centre coordinates)

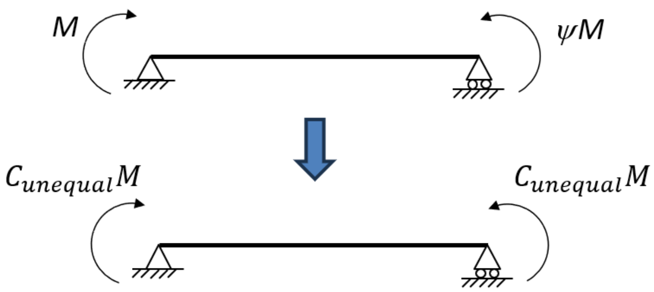
$$T = GJ\phi' - EC_w\phi'''$$

Beams

$$M_{cr,0} = \frac{\pi^2 EI_z}{L_{cr}^2}\sqrt{\frac{C_w}{I_z} + \frac{L_{cr}^2}{\pi^2}\frac{GJ}{EI_z}}$$

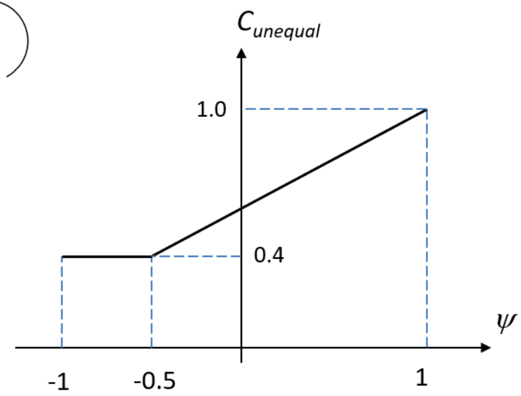
	Limits	Curve
Rolled I-sections	$h/b \leq 2$	a
	$h/b > 2$	b
Welded I-sections	$h/b \leq 2$	c
	$h/b > 2$	d
Other	-	d

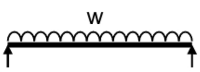
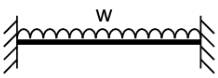
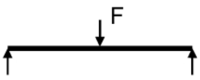
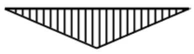
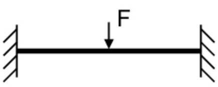
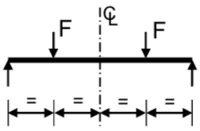
$$M_{cr} = C_1 M_{cr,0}$$



$$C_{unequal} = \max(0.6 + 0.4\psi, 0.4)$$

$$C_1 = \frac{1}{C_{unequal}}$$



Loading and support conditions	Bending moment diagram	Value of C_1
		1.132
		1.285
		1.365
		1.565
		1.046

Shear

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

For rolled I-beams: $A_v = A - 2bt_f + (t_w + 2r)t_f$ but $\geq h_w t_w$

(b = flange width; t_f = flange thickness; t_w = web thickness; h_w = web height; r = web-flange transition radius)

For other I beams: $A_v = h_w t_w$

Beam-Columns

$$\frac{N_{\mathrm{Ed}}}{\frac{\chi_{\mathrm{y}} N_{\mathrm{Rk}}}{\gamma_{\mathrm{M1}}}} + k_{\mathrm{yy}} \frac{M_{\mathrm{y,Ed}} + \Delta M_{\mathrm{y,Ed}}}{\chi_{\mathrm{LT}} \frac{M_{\mathrm{y,Rk}}}{\gamma_{\mathrm{M1}}}} + k_{\mathrm{yz}} \frac{M_{\mathrm{z,Ed}} + \Delta M_{\mathrm{z,Ed}}}{\frac{M_{\mathrm{z,Rk}}}{\gamma_{\mathrm{M1}}}} \leq 1,0$$

$$\frac{N_{\mathrm{Ed}}}{\frac{\chi_{\mathrm{z}} N_{\mathrm{Rk}}}{\gamma_{\mathrm{M1}}}} + k_{\mathrm{zy}} \frac{M_{\mathrm{y,Ed}} + \Delta M_{\mathrm{y,Ed}}}{\chi_{\mathrm{LT}} \frac{M_{\mathrm{y,Rk}}}{\gamma_{\mathrm{M1}}}} + k_{\mathrm{zz}} \frac{M_{\mathrm{z,Ed}} + \Delta M_{\mathrm{z,Ed}}}{\frac{M_{\mathrm{z,Rk}}}{\gamma_{\mathrm{M1}}}} \leq 1,0$$

Table 8.7 — Interaction factors k_{yy} and k_{yz}

Plastic cross-sectional properties Class 1, Class 2, Class 3 (with W_{ep} for Class 3, according to Annex B)	**Elastic cross-sectional properties Class 3 with W_{el}, Class 4**
For $\bar{\lambda}_{\mathrm{y}} < 1,0$: $k_{\mathrm{yy}} = C_{\mathrm{my}} \left[1 + \left(\bar{\lambda}_{\mathrm{y}} - 0,2\right) n_{\mathrm{y}}\right]$	For $\bar{\lambda}_{\mathrm{y}} < 1,0$: $k_{\mathrm{yy}} = C_{\mathrm{my}} \left(1 + 0,6\, \bar{\lambda}_{\mathrm{y}}\, n_{\mathrm{y}}\right)$
For $\bar{\lambda}_{\mathrm{y}} \geq 1,0$: $k_{\mathrm{yy}} = C_{\mathrm{my}} \left(1 + 0,8\, n_{\mathrm{y}}\right)$	For $\bar{\lambda}_{\mathrm{y}} \geq 1,0$: $k_{\mathrm{yy}} = C_{\mathrm{my}} \left(1 + 0,6\, n_{\mathrm{y}}\right)$
k_{yz} = 0,6 k_{zz}, see Table 8.8	k_{yz} = k_{zz}, see Table 8.8
NOTE 1 See (9) for n_{y}. NOTE 2 See (10) and Table 8.9 for C_{my}.	

$$n_{\mathrm{y}} = \frac{N_{\mathrm{Ed}}}{\chi_{\mathrm{y}}\, N_{\mathrm{Rk}} / \gamma_{\mathrm{M1}}}$$

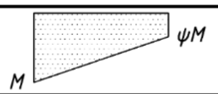
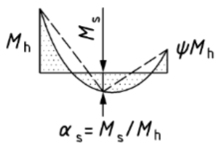
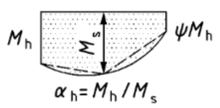
$$n_{\mathrm{z}} = \frac{N_{\mathrm{Ed}}}{\chi_{\mathrm{z}}\, N_{\mathrm{Rk}} / \gamma_{\mathrm{M1}}}$$

Table 8.8 — Interaction factors k_{zy} and k_{zz}

	Type of section	Plastic cross-sectional properties Class 1, Class 2, Class 3 (with W_{ep} for Class 3, according to Annex B)	Elastic cross-sectional properties Class 3 with W_{el}, Class 4
k_{zy}	Not susceptible to lateral-torsional buckling	$k_{zy} = 0{,}6\ k_{yy}$ See Table 8.7.	$k_{zy} = 0{,}8\ k_{yy}$ See Table 8.7.
	Susceptible to lateral-torsional buckling	For $\bar{\lambda}_z < 1{,}0$: $k_{zy} = 1 - \dfrac{0{,}1\bar{\lambda}_z\, n_z}{C_{mLT} - 0{,}25}$ but $k_{zy} \le 0{,}6 + \bar{\lambda}_z$ for $\bar{\lambda}_z < 0{,}4$ For $\bar{\lambda}_z \ge 1{,}0$: $k_{zy} = 1 - \dfrac{0{,}1 n_z}{C_{mLT} - 0{,}25}$	For $\bar{\lambda}_z < 1{,}0$: $k_{zy} = 1 - \dfrac{0{,}05\bar{\lambda}_z\, n_z}{C_{mLT} - 0{,}25}$ For $\bar{\lambda}_z \ge 1{,}0$: $k_{zy} = 1 - \dfrac{0{,}05 n_z}{C_{mLT} - 0{,}25}$
k_{zz}	I-sections	For $\bar{\lambda}_z < 1{,}0$: $k_{zz} = C_{mz}\left[1 + \left(2\bar{\lambda}_z - 0{,}6\right)n_z\right]$ For $\bar{\lambda}_z \ge 1{,}0$: $k_{zz} = C_{mz}\left(1 + 1{,}4 n_z\right)$	For $\bar{\lambda}_z < 1{,}0$: $k_{zz} = C_{mz}\left(1 + 0{,}6\bar{\lambda}_z\, n_z\right)$ For $\bar{\lambda}_z \ge 1{,}0$: $k_{zz} = C_{mz}\left(1 + 0{,}6 n_z\right)$
	Rectangular, circular or elliptical hollow sections	For $\bar{\lambda}_z < 1{,}0$: $k_{zz} = C_{mz}\left[1 + \left(\bar{\lambda}_z - 0{,}2\right)n_z\right]$ For $\bar{\lambda}_z \ge 1{,}0$: $k_{zz} = C_{mz}\left[1 + 0{,}8 n_z\right]$	

NOTE 1 See (9) for n_z.

NOTE 2 See (10) and Table 8.9 for C_{my} and C_{mLT}.

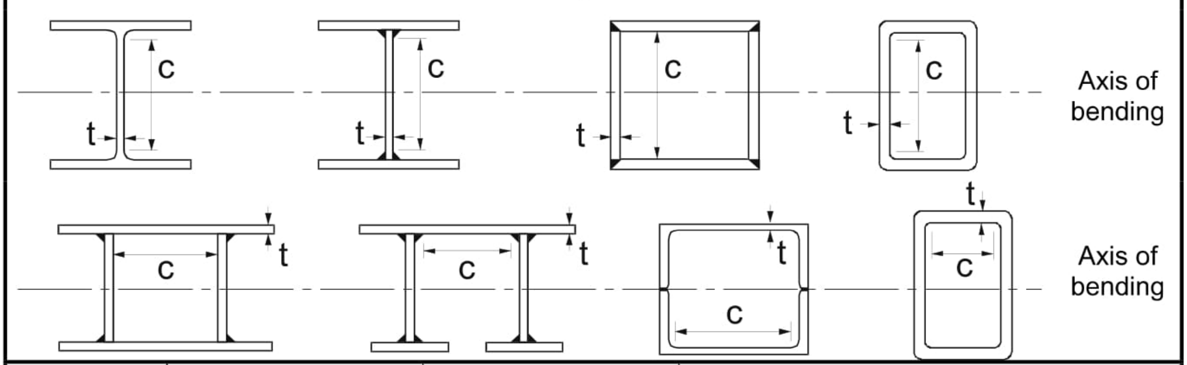
Moment diagram	Range		C_{my} and C_{mz} and C_{mLT}	
	$-1 \le \psi \le 1$		$0{,}6 + 0{,}4\psi \ge 0{,}4$	
Members with transverse loading			**Uniform loading**	**Concentrated load**
	$0 \le \alpha_s \le 1$	$-1 \le \psi \le 1$	$0{,}2 + 0{,}8\alpha_s \ge 0{,}4$	$0{,}2 + 0{,}8\alpha_s \ge 0{,}4$
	$-1 \le \alpha_s < 0$	$0 \le \psi \le 1$	$0{,}1 - 0{,}8\alpha_s \ge 0{,}4$	$-0{,}8\alpha_s \ge 0{,}4$
		$-1 \le \psi < 0$	$0{,}1(1-\psi) - 0{,}8\alpha_s \ge 0{,}4$	$-0{,}2\psi - 0{,}8\alpha_s \ge 0{,}4$
	$0 \le \alpha_h \le 1$	$-1 \le \psi \le 1$	$0{,}95 + 0{,}05\alpha_h$	$0{,}90 + 0{,}10\alpha_h$
	$-1 \le \alpha_h < 0$	$0 \le \psi \le 1$	$0{,}95 + 0{,}05\alpha_h$	$0{,}90 + 0{,}10\alpha_h$
		$-1 \le \psi < 0$	$0{,}95 + 0{,}05\alpha_h\left(1 + 2\psi\right)$	$0{,}90 + 0{,}10\alpha_h\left(1 + 2\psi\right)$

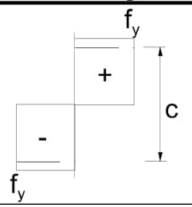
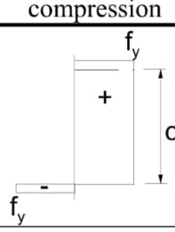
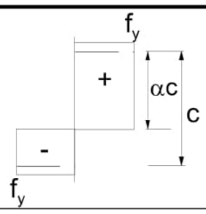
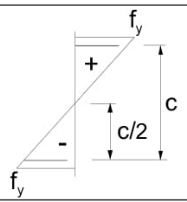
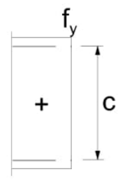
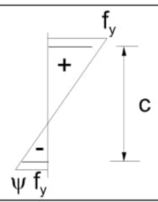
DS3: Thin-walled Structures

Cross-sectional classification

(EN 1993-1-1; Table 5.2)

Internal compression parts

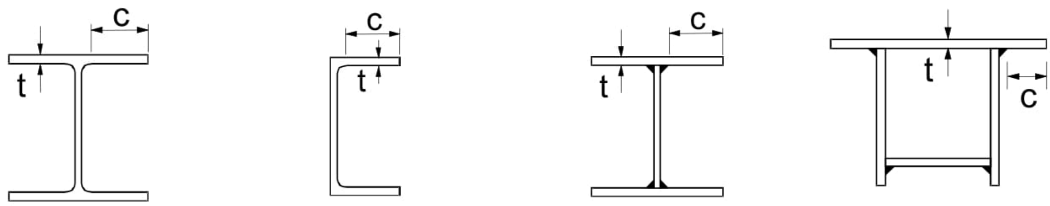


Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \leq 72\varepsilon$	$c/t \leq 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \leq \dfrac{396\varepsilon}{13\alpha - 1}$ when $\alpha \leq 0{,}5$: $c/t \leq \dfrac{36\varepsilon}{\alpha}$
2	$c/t \leq 83\varepsilon$	$c/t \leq 38\varepsilon$	when $\alpha > 0{,}5$: $c/t \leq \dfrac{456\varepsilon}{13\alpha - 1}$ when $\alpha \leq 0{,}5$: $c/t \leq \dfrac{41{,}5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)			
3	$c/t \leq 124\varepsilon$	$c/t \leq 42\varepsilon$	when $\psi > -1$: $c/t \leq \dfrac{42\varepsilon}{0{,}67 + 0{,}33\psi}$ when $\psi \leq -1^{*)}$: $c/t \leq 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

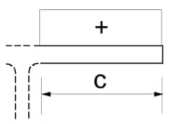
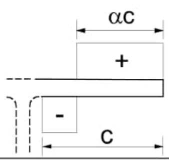
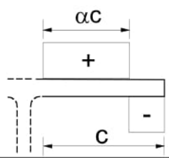
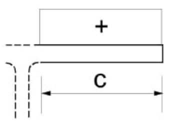
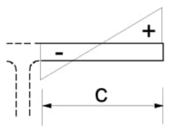
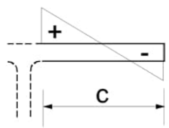
$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

*) $\psi \leq -1$ applies where either the compression stress $\sigma \leq f_y$ or the tensile strain $\varepsilon_y > f_y/E$

Outstand flanges



Rolled sections Welded sections

Class	Part subject to compression	Part subject to bending and compression	
		Tip in compression	Tip in tension
Stress distribution in parts (compression positive)			
1	$c/t \leq 9\varepsilon$	$c/t \leq \frac{9\varepsilon}{\alpha}$	$c/t \leq \frac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	$c/t \leq 10\varepsilon$	$c/t \leq \frac{10\varepsilon}{\alpha}$	$c/t \leq \frac{10\varepsilon}{\alpha\sqrt{\alpha}}$
Stress distribution in parts (compression positive)			
3	$c/t \leq 14\varepsilon$	$c/t \leq 21\varepsilon\sqrt{k_\sigma}$ For k_σ see EN 1993-1-5	

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

Local buckling of plates

$$\sigma_{cr} = K \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{t}{b}\right)^2$$

where b is the width of the plate and t is its thickness.

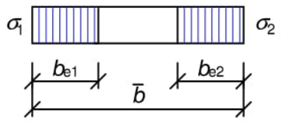
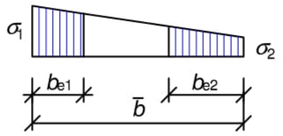
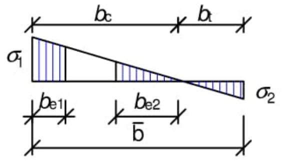
- For plates in uniform longitudinal compression:

  K = 4 for internal elements.

  K = 0.43 for outstand elements.

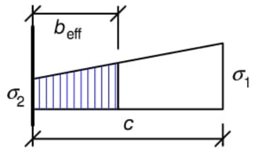
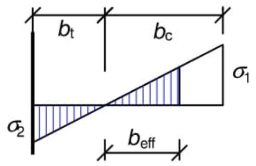
- For plates under in-plane bending (EN 1993-1-5): $K = k_\sigma$ (see table below)

Table 4.1: Internal compression elements

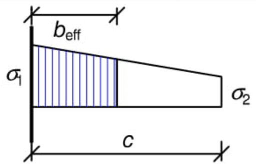
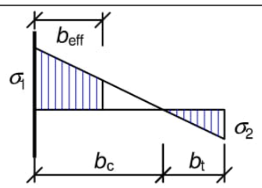
Stress distribution (compression positive)	Effective^p width b_{eff}
	$\underline{\psi = 1}$: $b_{eff} = \rho\ \bar{b}$ $b_{e1} = 0{,}5\ b_{eff}$ $\quad b_{e2} = 0{,}5\ b_{eff}$
	$\underline{1 > \psi \geq 0}$: $b_{eff} = \rho\ \bar{b}$ $b_{e1} = \dfrac{2}{5-\psi} b_{eff}$ $\quad b_{e2} = b_{eff} - b_{e1}$
	$\underline{\psi < 0}$: $b_{eff} = \rho\ b_c = \rho\ \bar{b} / (1-\psi)$ $b_{e1} = 0{,}4\ b_{eff}$ $\quad b_{e2} = 0{,}6\ b_{eff}$

$\psi = \sigma_2/\sigma_1$	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	$5{,}98\ (1 - \psi)^2$

Table 4.2: Outstand compression elements

Stress distribution (compression positive)	Effective^p width b_{eff}
	$\underline{1 > \psi \geq 0}$: $b_{eff} = \rho\ c$
	$\underline{\psi < 0}$: $b_{eff} = \rho\ b_c = \rho\ c / (1-\psi)$

$\psi = \sigma_2/\sigma_1$	1	0	-1	$1 \geq \psi \geq -3$
Buckling factor k_σ	0,43	0,57	0,85	$0{,}57 - 0{,}21\psi + 0{,}07\psi^2$

Stress distribution (compression positive)	Effective^p width b_{eff}
	$\underline{1 > \psi \geq 0}$: $b_{eff} = \rho\ c$
	$\underline{\psi < 0}$: $b_{eff} = \rho\ b_c = \rho\ c / (1-\psi)$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1
Buckling factor k_σ	0,43	$0{,}578 / (\psi + 0{,}34)$	1,70	$1{,}7 - 5\psi + 17{,}1\psi^2$	23,8

- For plates in shear:

$$\tau_{cr} = K \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{t}{b}\right)^2$$

$K = 5.34 + \frac{4}{(a/b)^2}$ with a > b

Effective widths

(EN 1993-1-5; Clause 4.4)

$$A_{c,eff} = \rho\, A_c \qquad (4.1)$$

where ρ is the reduction factor for plate buckling.

(2) The reduction factor ρ may be taken as follows:

– internal compression elements:

$\rho = 1{,}0$ for AC1 $\bar{\lambda}_p \leq 0{,}5 + \sqrt{0.085 - 0.055\psi}$ AC1

$\rho = \dfrac{\bar{\lambda}_p - 0{,}055\,(3+\psi)}{\bar{\lambda}_p^2} \leq 1{,}0$ for AC1 $\bar{\lambda}_p > 0{,}5 + \sqrt{0.085 - 0.055\psi}$, AC1 AC1 *text deleted* AC1 (4.2)

– outstand compression elements:

$\rho = 1{,}0$ for $\bar{\lambda}_p \leq 0{,}748$

$\rho = \dfrac{\bar{\lambda}_p - 0{,}188}{\bar{\lambda}_p^2} \leq 1{,}0$ for $\bar{\lambda}_p > 0{,}748$ (4.3)

where $\bar{\lambda}_p = \sqrt{\dfrac{f_y}{\sigma_{cr}}} = \dfrac{\bar{b}/t}{28{,}4\,\varepsilon\,\sqrt{k_\sigma}}$

Shear buckling

Shear buckling needs to be checked if: $\frac{h_w}{t_w} \geq 72\varepsilon$

where h_w is the web height, t_w is the web thickness and $\varepsilon = \sqrt{235/f_y}$ (with f_y in MPa).

$$V_{b,Rd} = \chi_w \frac{(f_y/\sqrt{3})\, h_w t_w}{\gamma_{M1}}$$

$$\lambda_w = 0.76 \sqrt{\frac{f_y}{\tau_{cr}}}$$

Table 5.1: Contribution from the web χ_w to shear buckling resistance

	Rigid end post	Non-rigid end post
$\bar{\lambda}_w < 0{,}83/\eta$	η	η
$0{,}83/\eta \leq \bar{\lambda}_w < 1{,}08$	$0{,}83/\bar{\lambda}_w$	$0{,}83/\bar{\lambda}_w$
$\bar{\lambda}_w \geq 1{,}08$	$1{,}37/(0{,}7+\bar{\lambda}_w)$	$0{,}83/\bar{\lambda}_w$

Flange induced buckling

(EN 1993-1-5; Clause 8)

(1) To prevent the compression flange buckling in the plane of the web, the following criterion should be met:

$$\frac{h_w}{t_w} \leq k \frac{E}{f_{yf}} \sqrt{\frac{A_w}{A_{fc}}} \tag{8.1}$$

where A_w is the cross section area of the web;

A_{fc} is the effective cross section area of the compression flange;

h_w is the depth of the web;

t_w is the thickness of the web.

The value of the factor k should be taken as follows:

- plastic rotation utilized $k = 0{,}3$
- plastic moment resistance utilized $k = 0{,}4$
- elastic moment resistance utilized $k = 0{,}55$

Stiffener buckling

(EN 1993-1-5; Clause 4.5.3)

$$\alpha_c = \alpha + \frac{0.09}{i/e}$$

where:

α = 0.34 for closed section stiffeners

α = 0.49 for open section stiffeners

i = the radius of gyration of the effective column

e = max (e_1, e_2)

e_1 = the distance between the centroid of the stiffener and the centroid of the effective column

e_2 = the distance between the centre line of the stiffened plate and the centroid of the effective column

Moment-shear-axial force interaction

(EN 1993-1-3; Clause 8)

(1) For cross-sections subject to the combined action of an axial force N_{Ed}, a bending moment M_{Ed} and a shear force V_{Ed} no reduction due to shear force need not be done provided that $V_{Ed} \leq 0{,}5\ V_{w,Rd}$. If the shear force is larger than half of the shear force resistance then following equations should be satisfied:

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} + \left(1 - \frac{M_{f,Rd}}{M_{pl,Rd}}\right)\left(\frac{2V_{Ed}}{V_{w,Rd}} - 1\right)^2 \leq 1{,}0 \qquad \ldots(6.27)$$

where:

- N_{Rd} is the design resistance of a cross-section for uniform tension or compression given in 6.1.2 or 6.1.3;
- $M_{y,Rd}$ is the design moment resistance of the cross-section given in 6.1.4;
- $V_{w,Rd}$ is the design shear resistance of the web given in 6.1.5(1);
- $M_{f,Rd}$ is the moment of resistance of a cross-section consisting of the effective area of flanges only, see EN 1993-1-5;
- $M_{pl,Rd}$ is the plastic moment of resistance of the cross-section, see EN 1993-1-5.

DS4: Connections

Bolt size	Tensile Area
-	mm^2
M10	58.0
M12	84.3
M14	115
M16	157
M18	192
M20	245
M22	303
M24	353
M27	459
M30	561

Shear capacity of a bolt (single shear):

$$F_{V,Rd} = 0.6Af_{ub}/\gamma_{M2}$$

Tensile capacity of a bolt:

$$F_{\mathrm{t},Rd} = 0.9A_s f_{ub}/\ \gamma_{M2}$$

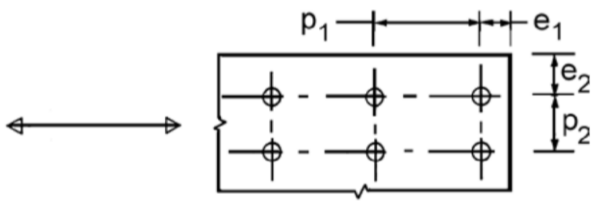
Bolt in tension and shear:

$$\frac{F_{V,Ed}}{F_{V,Rd}} + \frac{F_{t,Ed}}{1.4F_{t,Rd}} \leq 1.0$$

Table 18 — Values of the nominal minimum preloading force $F_{\mathrm{p,C}}$ in [kN]

Property class	Bolt diameter in mm									
	12	**14**	**16**	**18**	**20**	**22**	**24**	**27**	**30**	**36**
8.8	47	65	88	108	137	170	198	257	314	458
10.9	59	81	110	134	172	212	247	321	393	572

Minimum end and edge distances, and bolt spacings:



$e_1 \geq 1.2\ d_0$

$e_2 \geq 1.2\ d_0$

$p_1 \geq 2.2\ d_0$

$p_2 \geq 2.4\ d_0$

Bolt tear-out:

$$F_{1,Rd} = 2at\frac{f_y}{\sqrt{3}}/\gamma_{M0}$$

t = ply thickness

A_{nt} = area in tension

A_{nV} = area in shear

Block tear-out in tension:

$$F_{eff,1,Rd} = \frac{f_u A_{nt}}{\gamma_{M2}} + \frac{f_y}{\sqrt{3}}\frac{A_{nV}}{\gamma_{M0}}$$

Block tear-out in shear:

$$F_{eff,2,Rd} = 0.5\frac{f_u A_{nt}}{\gamma_{M2}} + \frac{f_y}{\sqrt{3}}\frac{A_{nV}}{\gamma_{M0}}$$

Bolt bearing:

$$F_{b,Rd} = \frac{k_1 \propto_b f_u d t}{\gamma_{M2}}$$

where α_b is the smallest of α_d ; $\frac{f_{ub}}{f_u}$ or 1,0;

in the direction of load transfer:

- for end bolts: $\alpha_d = \frac{e_1}{3d_0}$; for inner bolts: $\alpha_d = \frac{p_1}{3d_0} - \frac{1}{4}$

perpendicular to the direction of load transfer:

AC2 - for edge bolts: k_1 is the smallest of $2{,}8\frac{e_2}{d_0} - 1{,}7$, $1{,}4\frac{p_2}{d_0} - 1{,}7$ and 2,5 AC2

- for inner bolts: k_1 is the smallest of $1{,}4\frac{p_2}{d_0} - 1{,}7$ or 2,5

Punching shear:

$$B_{p,Rd} = 0.6\pi d_m t_p f_u / \gamma_{M2}$$

t_p = thickness of the plate under the head/nut

d_m = average diameter of the head/nut

Bolt slip load:

$$F_{s,Rd} = \frac{n\mu F_{p,C}}{\gamma_{M3,ser}}$$

n = number of friction planes

$F_{p,C}$ = bolt pre-load

μ = friction coefficient (see below)

$\gamma_{M3,ser}$ = 1.10

Classifications that may be assumed for friction surfaces

Surface treatment	Class	Slip factor (μ)
Surfaces blasted with shot or grit with loose rust removed, not pitted	A	0.50
Surfaces hot-dip galvanized and flash (sweep) blasted and with alkali-zinc silicate paint with a nominal thickness of 60 μm	B	0.40
Surfaces blasted with shot or grit; a) coated with alkali-zinc silicate paint with a nominal thickness of 60 μm^{+} b) thermally sprayed with aluminium or zinc or a combination of both to a nominal thickness not exceeding 80 μm	B	0.40
Surfaces hot-dip galvanized and flash (sweep) blasted	C	0.35
Surfaces cleaned by wire brush or flame cleaning, with loose rust removed	C	0.30
Surfaces as rolled	D	0.20

Reduction factor for long bolted connections ($L_j > 15d$, where d is the bolt diameter):

$$\beta_{L,f} = 1 - \frac{L_j - 15d}{200d} \qquad 0.75 \leq \beta_{L,f} \leq 1.0$$

Reduction factor for shear lag in eccentrically connected angles (EN 1993-1-8 Clause 3.10.3):

Pitch	p_1	$\leq 2{,}5\ d_o$	$\geq 5{,}0\ d_o$
2 bolts	β_2	0,4	0,7
3 bolts or more	β_3	0,5	0,7

Bolt group subject to moment:

$$F_i = kr_i \qquad\qquad k = \frac{M}{\sum r_i^2}$$

M = applied moment

r_i = distance from the bolt to the centre of rotation of the bolt group

F_i = bolt shear force

Design of welds:

$$\sqrt{\sigma_x^2 + 3\left(\tau_y^2 + \tau_z^2\right)} \leq \frac{f_u}{\beta_w \gamma_{M2}} \qquad \text{and} \qquad \sigma_x \leq 0.9\frac{f_u}{\gamma_{M2}}$$

f_u = ultimate tensile strength of the weaker connected part

β_w = 0.9 for S355; 0.85 for S275

Reduction factor for long welds:

- if $l_w \geq 150a$: $\quad \beta_{Lw,1} = 1.2 - \dfrac{0.2 l_w}{150a} \qquad \beta_{Lw,1} \leq 1.0$
- if l_w ≥1.7 m in stiffeners of plate girders: $\quad \beta_{Lw,2} = 1.1 - \dfrac{l_w}{17}$ (l_w in m)

$$0.6 \leq \beta_{Lw,2} \leq 1.0$$

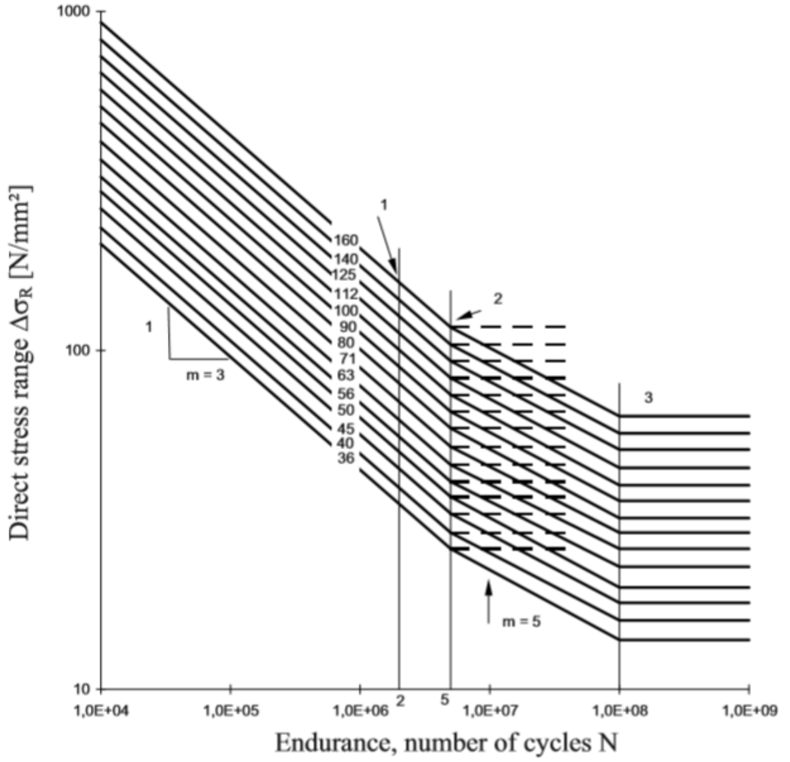
where a is the throat thickness of the weld.

S-N curves: $N_r(\Delta\sigma_r)^m = K$

N_r = number of cycles causing failure

$\Delta\sigma_r$ = amplitude of the stress cycle

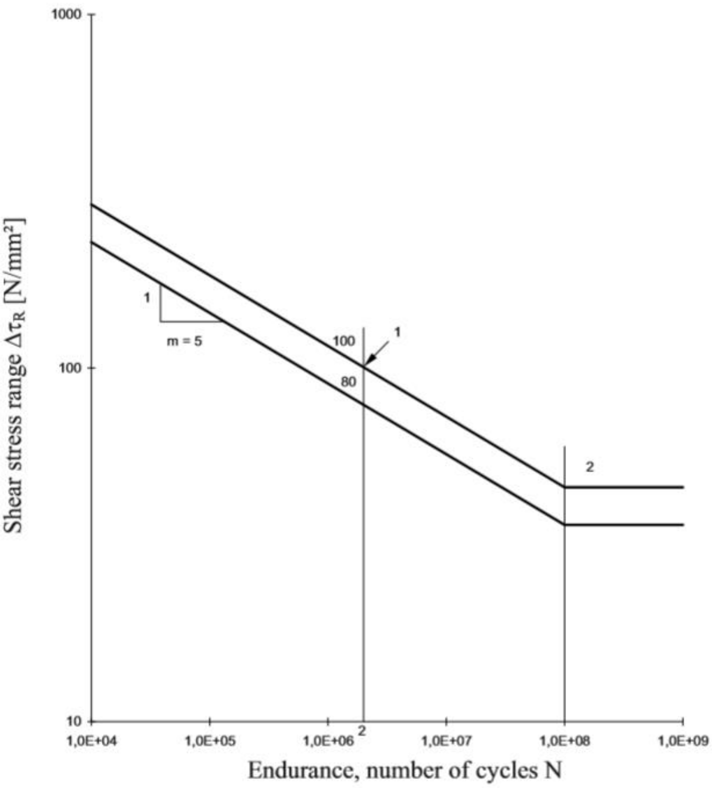
K, m = constants



1 Detail category $\Delta\sigma_C$

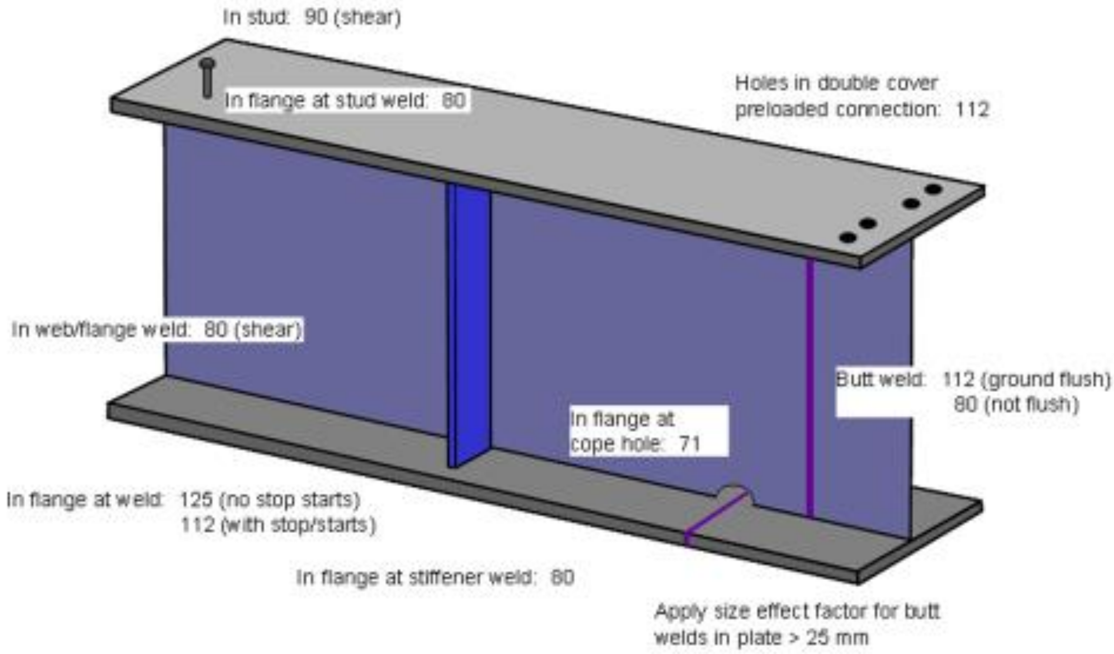
2 Constant amplitude fatigue limit $\Delta\sigma_D$

3 Cut-off limit $\Delta\sigma_L$



1 Detail category $\Delta\tau_C$

2 Cut-off limit $\Delta\tau_L$



Typical fatigue details in plate girders.

Palmgren-Miner rule: $$\frac{n_1}{N_1}+\frac{n_2}{N_2}+\frac{n_3}{N_3}+\cdots\frac{n_i}{N_i}+\cdots\leq 1$$

n_i = number of applied cycles with amplitude $\Delta\sigma_i$

N_i = number of cycles with amplitude $\Delta\sigma_i$ causing failure

DS5: Composite beams

Headed shear studs:

- standard sizes are 13 mm, 16 mm, 19 mm, 22 mm, 25 mm.
- shear capacity is the lesser of:

$$P_{Rd} = \frac{0.8 f_u (\pi d^2/4)}{\gamma_V}$$

$$P_{Rd} = \frac{0.29 d^2 \, (f_{ck} E_{cm})^{0.5}}{\gamma_V}$$

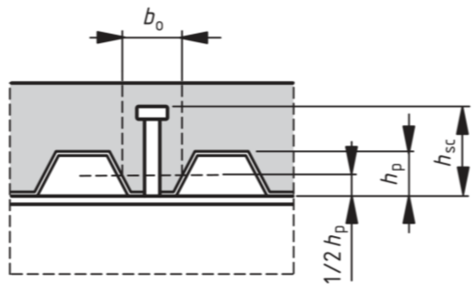
where:

f_u = ultimate tensile strength of the steel
d = diameter of the stud
f_{ck} = concrete compressive strength
E_{cm} = elastic modulus of the concrete
γ_V = 1.25

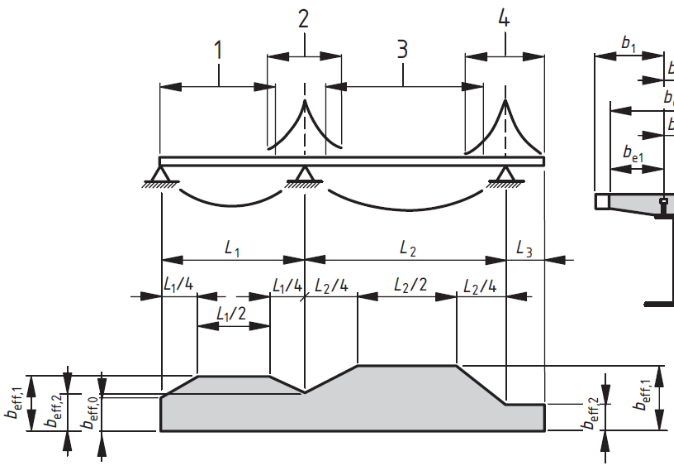
- Reduction factor for studs on steel decking:

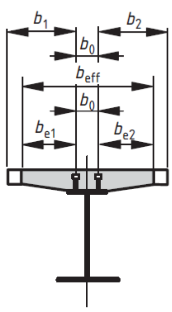
$$k_t = \frac{0.7}{\sqrt{n_r}} \frac{b_0}{h_p} \left(\frac{h_{sc}}{h_p} - 1 \right)$$

n_r = number of studs per rib



Effective width of the concrete slab:





$$b_{eff} = L_e/4 + b_0$$

Key

1 $L_e = 0{,}85 L_1$ for $b_{eff,1}$
2 $L_e = 0{,}25(L_1 + L_2)$ for $b_{eff,2}$
3 $L_e = 0{,}70 L_2$ for $b_{eff,1}$
4 $L_e = 2 L_3$ for $b_{eff,2}$

Modular ratio: $n = 2n_0$ $n_0 = E_a/E_{cm}$

where E_a is the elastic modulus of steel and E_{cm} is the elastic modulus of the concrete:

Strength classes for concrete															Analytical relation / Explanation
f_{ck} (MPa)	12	16	20	25	30	35	40	45	50	55	60	70	80	90	
$f_{ck,cube}$ (MPa)	15	20	25	30	37	45	50	55	60	67	75	85	95	105	
f_{cm} (MPa)	20	24	28	33	38	43	48	53	58	63	68	78	88	98	$f_{cm} = f_{ck}+8$(MPa)
f_{ctm} (MPa)	1,6	1,9	2,2	2,6	2,9	3,2	3,5	3,8	4,1	4,2	4,4	4,6	4,8	5,0	$f_{ctm}=0,30\times f_{ck}^{(2/3)} \leq$C50/60 $f_{ctm}=2,12\cdot\ln(1+(f_{cm}/10))$ > C50/60
$f_{ctk,0,05}$ (MPa)	1,1	1,3	1,5	1,8	2,0	2,2	2,5	2,7	2,9	3,0	3,1	3,2	3,4	3,5	$f_{ctk;0,05} = 0,7\times f_{ctm}$ 5% fractile
$f_{ctk,0,95}$ (MPa)	2,0	2,5	2,9	3,3	3,8	4,2	4,6	4,9	5,3	5,5	5,7	6,0	6,3	6,6	$f_{ctk;0,95} = 1,3\times f_{ctm}$ 95% fractile
E_{cm} (GPa)	27	29	30	31	33	34	35	36	37	38	39	41	42	44	$E_{cm} = 22[(f_{cm})/10]^{0,3}$ (f_{cm} in MPa)
ε_{c1} (‰)	1,8	1,9	2,0	2,1	2,2	2,25	2,3	2,4	2,45	2,5	2,6	2,7	2,8	2,8	see Figure 3.2 $\varepsilon_{c1}(^0/_{00}) = 0,7\, f_{cm}^{0,31} < 2.8$
ε_{cu1} (‰)					3,5					3,2	3,0	2,8	2,8	2,8	see Figure 3.2 for $f_{ck} \geq 50$ Mpa $\varepsilon_{cu1}(^0/_{00})=2,8+27[(98-f_{cm})/100]^4$
ε_{c2} (‰)					2,0					2,2	2,3	2,4	2, 5	2,6	see Figure 3.3 for $f_{ck} \geq 50$ Mpa $\varepsilon_{c2}(^0/_{00})=2,0+0,085(f_{ck}-50)^{0,53}$
ε_{cu2} (‰)					3,5					3,1	2,9	2,7	2,6	2,6	see Figure 3.3 for $f_{ck} \geq 50$ Mpa $\varepsilon_{cu2}(^0/_{00})=2,6+35[(90-f_{ck})/100]^4$
n					2,0					1,75	1,6	1,45	1,4	1,4	for $f_{ck} \geq 50$ Mpa $n=1,4+23,4[(90- f_{ck})/100]^4$
ε_{c3} (‰)					1,75					1,8	1,9	2,0	2,2	2,3	see Figure 3.4 for $f_{ck} \geq 50$ Mpa $\varepsilon_{c3}(^0/_{00})=1,75+0,55[(f_{ck}-50)/40]$
ε_{cu3} (‰)					3,5					3,1	2,9	2,7	2,6	2,6	see Figure 3.4 for $f_{ck} \geq 50$ Mpa $\varepsilon_{cu3}(^0/_{00})=2,6+35[(90-f_{ck})/100]^4$