Version SS/JB/KAS/04

EGT3
ENGINEERING TRIPOS PART IIB

Monday 03 May 2021 9 to 10.40

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 and at the top of each answer sheet.*

STATIONERY REQUIREMENTS
Write on single-sided paper.

SPECIAL REQUIREMENTS TO BE SUPPLIED FOR THIS EXAM
CUED approved calculator allowed.
Attachments: 4D10 Structural Steelwork Data Sheets (9 pages).
You are allowed access to the electronic version of the Engineering Data Books.

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

The time taken for scanning/uploading answers is 15 minutes.

Your script is to be uploaded as a single consolidated pdf containing all answers.

1 Consider the 254 × 146 × 37 Universal Beam shown in Fig. 1 and made of grade S355 steel. The length is $L = 10$ m and the beam is initially simply supported. The ends are free to warp but are restrained against lateral deflection and twist. A moment, M, is applied about the major axis at either end, with both moments being in the same direction, as indicated.

(a) Determine the maximum value of M that the beam can safely carry. [60%]

(b) A restraint to prevent lateral deflection and twist rotation is added at a distance $x = 3L/4$ from the left end. Calculate the factor by which M can safely increase. [40%]

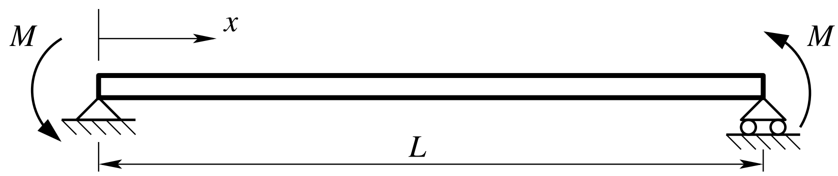

Fig. 1

2 Profiled metal sheeting is to be used to span between beams in a composite floor. Although sheeting manufacturers provide design charts, describe suitable checks that you could perform to establish that the choice of sheeting is adequate. State any assumptions and include diagrams of the associated stress blocks. [20%]

Figure 2 shows a composite floor with a span of 10 m. The steel beams are $356 \times 171 \times 51$ Universal Beams made of grade S275 steel. The beams run the full length of the slab and are simply-supported at each end. The concrete slab has a total thickness of 100 mm, which includes 50 mm troughs from steel decking. The troughs are perpendicular to the supporting steel beams, as shown. The transverse spacing between beam centres is 3 m and the decking has been properly designed to span between the beams. The concrete has a design strength, f_{cd} = 30 MPa, and density, 2400 kg m^{-3}. The floor is to support its self-weight together with a uniformly-distributed live load, w kPa, which is to be determined. Partial safety factors of 1.35 for dead loads and 1.5 for live loads are required.

(a) Assuming full composite action, determine the floor load w at the Ultimate Limit State. [40%]

(b) Propose a suitable arrangement of shear studs to achieve full composite action under the floor load just calculated. [10%]

(c) Devise and undertake a suitable serviceability check. [30%]

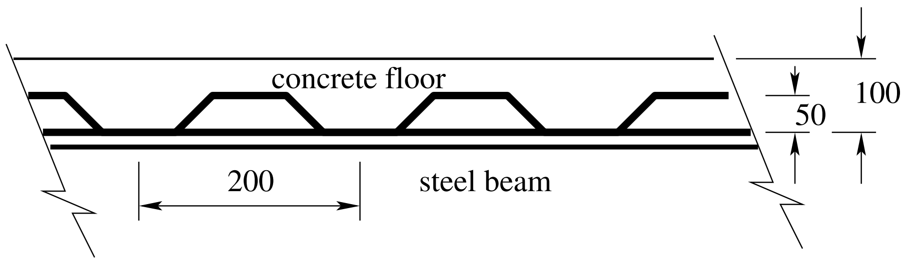

Fig. 2: all dimensions are in mm

3 A column has a length of 1 m and is pin-supported at the centroids of both end sections. Its uniform cross-section is shown in Fig. 3 and consists of an equal-leg angle with a leg length of 100 mm and a thickness of 5 mm. The column is made of S355 grade steel and is initially unloaded.

(a) Determine the effective area of the column in compression. Also calculate the shift of the effective centroid caused by local buckling. [40%]

(b) Determine the effective cross-section in minor axis bending with the tips of the angle in compression. [30%]

(c) Determine the maximum compressive load which the column can sustain. (Note: take any factors accounting for the interaction of bending moment and axial force as 1.0.) [30%]

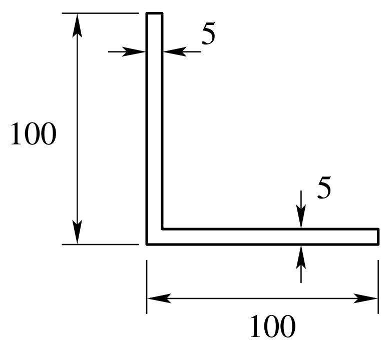

Fig. 3: all dimensions are in mm

4 The bottom chord of a truss consists of an SHS 200 × 200 × 6.3 square hollow section, made of S355 grade steel with f_u = 490 MPa. The chord carries a tensile design load of 1250 kN.

Design a bolted tension splice for the chord. [100%]

(Hint: Note that installing bolts requires access to both the head and the nut of the bolts, and that a solution with simple splice plates is not achievable in this case.)

END OF PAPER

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Data Sheets

DO NOT USE FOR ACTUAL DESIGN OF STRUCTURAL STEELWORK

DS1: Basic Buckling Resistance Curves

BS EN 1993-1-1:2005
EN 1993-1-1:2005 (E)

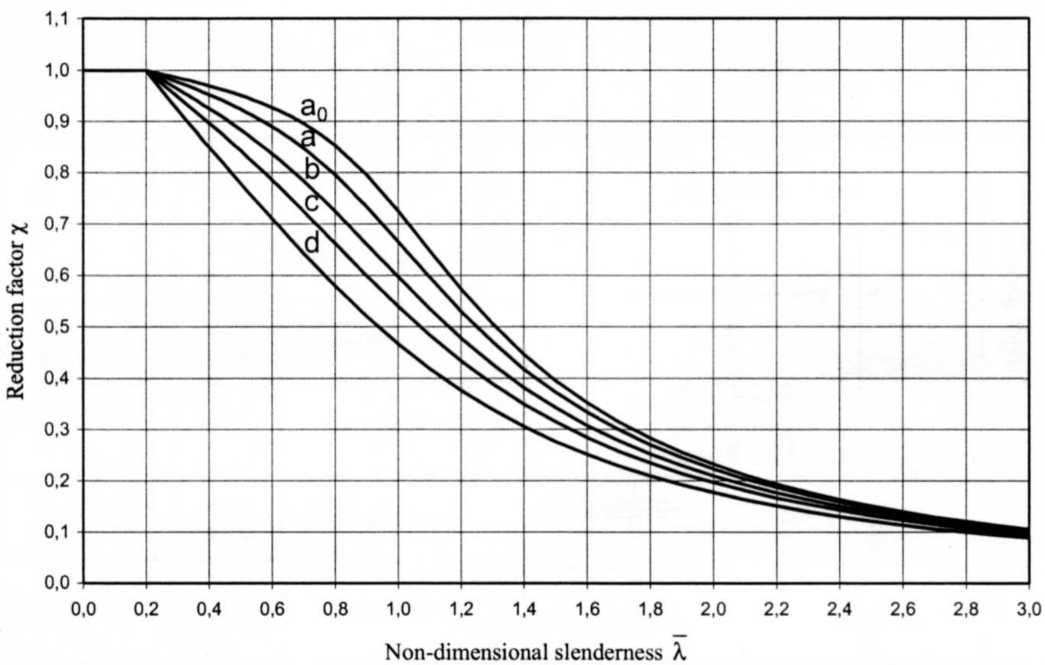


Figure 6.4: Buckling curves

The curves are defined by $\chi = \dfrac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}}$ in which $\Phi \equiv \dfrac{1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2}{2}$

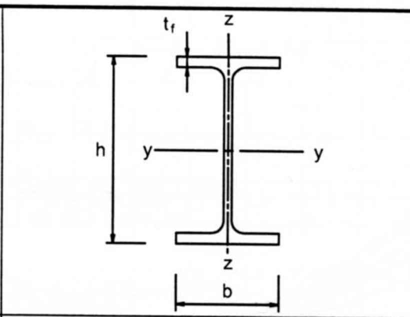
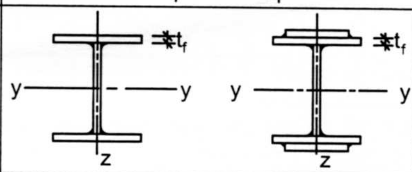
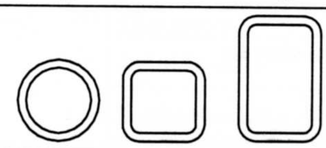
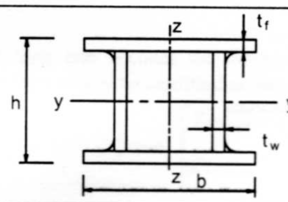
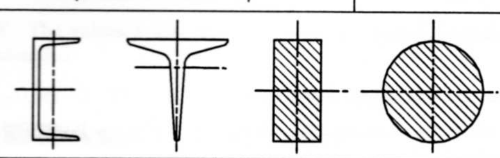
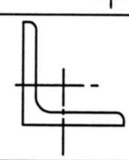
and the imperfection factor α appropriate for each curve is:

Buckling curve	a_0	a	b	c	d
Imperfection factor α	0.13	0.21	0.34	0.49	0.76

DS2: Basic Resistance Curve Selection for Flexural Buckling

BS EN 1993-1-1:2005
EN 1993-1-1:2005 (E)

Table 6.2: Selection of buckling curve for a cross-section

Cross section		Limits		Buckling about axis	Buckling curve: S 235, S 275, S 355, S 420	Buckling curve: 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

DS3: Lateral-Torsional Buckling Equations

Critical Moment

The critical magnitude of equal-and-opposite end-moments to cause elastic lateral torsional buckling of a beam is:

$$M_{LT} = \frac{\pi}{L}\sqrt{EIGJ}\sqrt{1+\frac{\pi^2}{L^2}\frac{E\Gamma}{GJ}}$$

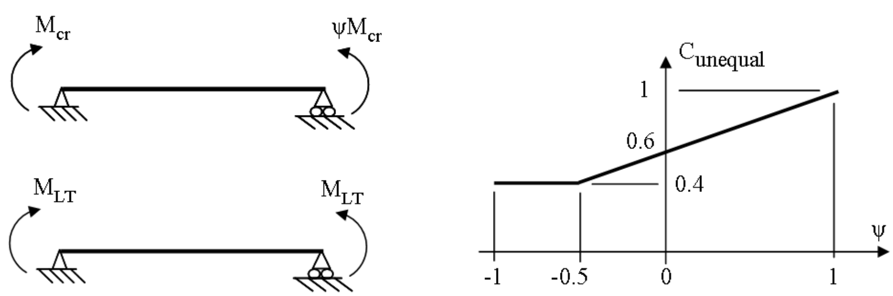
where EI, GJ and $E\Gamma$ are the minor axis flexural rigidity, the torsional rigidity and the warping rigidity respectively. (It is assumed that the supports prevent vertical, lateral and torsional deflections but do not restrain warping.)

For a doubly-symmetric I-beam

$$\Gamma \approx \frac{ID^2}{4}$$

where D is the distance between flange centroids and I is the second moment of area of the section about its minor axis.

Unequal end moments



$$M_{cr} = \frac{M_{LT}}{C_{\text{unequal}}} \quad \text{where} \quad C_{\text{unequal}} = \max(0.6+0.4\psi, 0.4)$$

Lateral torsional buckling curve selection

For lateral torsional buckling, the buckling resistance curves (DS1) may be used, with curves selected via the table below. Height h and width b are defined in DS2.

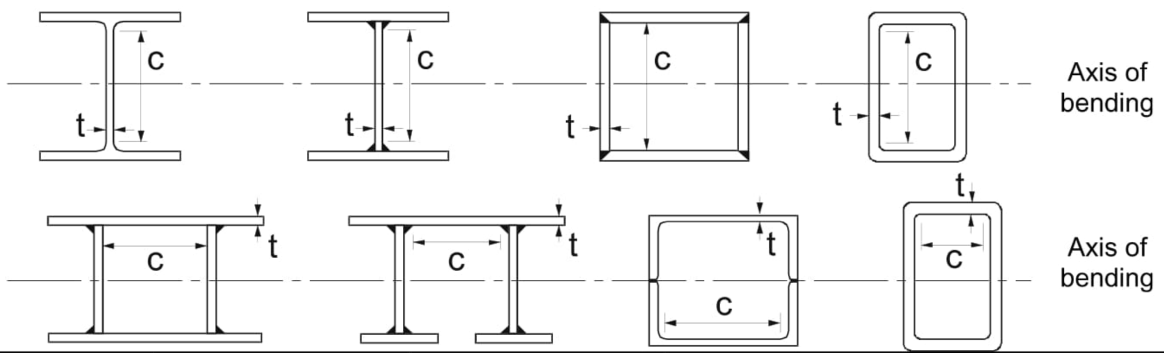
	Limits	Curve
Rolled I-sections	$h/b \leq 2$	a
	$h/b > 2$	b
Welded I-sections	$h/b \leq 2$	c
	$h/b > 2$	d
Other	-	d

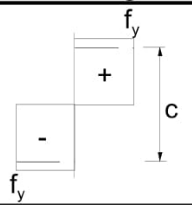
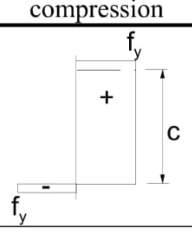
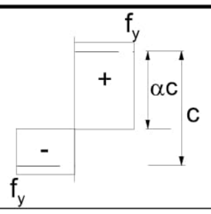
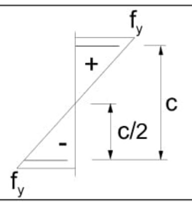
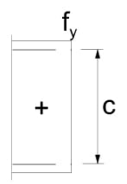
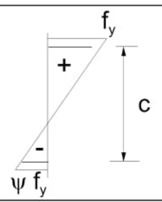
DS4: Thin-walled Structures

Cross-sectional classification

(EN 1993-1-1; Table 5.2)

Internal compression parts

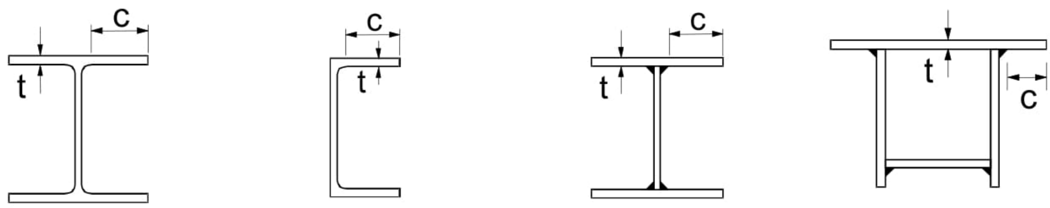


Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \le 72\varepsilon$	$c/t \le 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \frac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t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

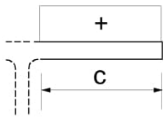
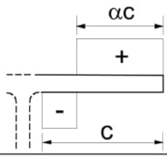
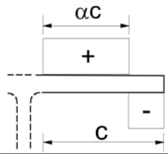
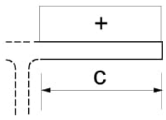
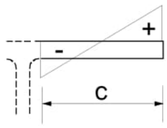
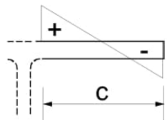
$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

*) $\psi \le -1$ applies where either the compression stress $\sigma \le f_y$ 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	Part subject to bending and 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.

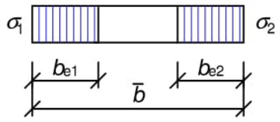
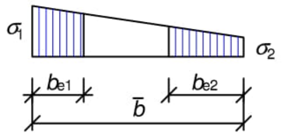
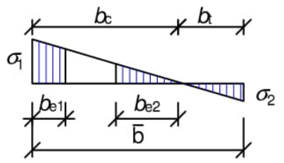
- For plates in uniform longitudinal compression:

  $K = 4$ for internal elements.

  $K = 0.43$ for outstand elements.

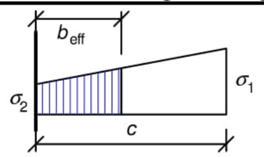
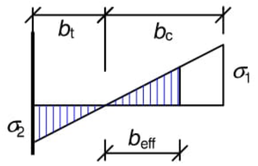
- For plates under in-plane bending (EN 1993-1-5): $K = k_\sigma$

Table 4.1: Internal compression elements

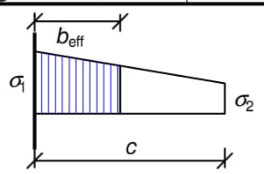
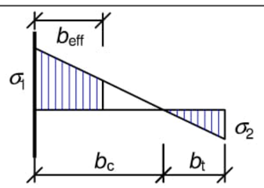
Stress distribution (compression positive)	Effective^{p} width b_{eff}
	$\underline{\psi = 1}$: $b_{eff} = \rho\ \bar{b}$ $b_{e1} = 0,5\ b_{eff}$ $b_{e2} = 0,5\ b_{eff}$
	$\underline{1 > \psi \geq 0}$: $b_{eff} = \rho\ \bar{b}$ $b_{e1} = \frac{2}{5-\psi} b_{eff}$ $b_{e2} = b_{eff} - b_{e1}$
	$\underline{\psi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

$\psi = \sigma_2/\sigma_1$	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:

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

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

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

Effective widths

(EN 1993-1-5; Clause 4.4)

$$A_{c,eff} = \rho A_c \tag{4.1}$$

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

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

– outstand compression elements:

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

$$\rho = \frac{\bar{\lambda}_p - 0{,}188}{\bar{\lambda}_p^2} \leq 1{,}0 \quad \text{for } \bar{\lambda}_p > 0{,}748 \tag{4.3}$$

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

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
$0{,}83/\eta \le \bar{\lambda}_w < 1{,}08$	$0{,}83/\bar{\lambda}_w$	$0{,}83/\bar{\lambda}_w$
$\bar{\lambda}_w \ge 1{,}08$	$1{,}37/(0{,}7+\bar{\lambda}_w)$	$0{,}83/\bar{\lambda}_w$

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:

$$\frac{h_w}{t_w} \le k\,\frac{E}{f_{yf}}\sqrt{\frac{A_w}{A_{fc}}} \qquad (8.1)$$

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:

$\alpha = 0.34$ for closed section stiffeners

$\alpha = 0.49$ 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.

DS5: 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:

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

Tensile capacity of a bolt:

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

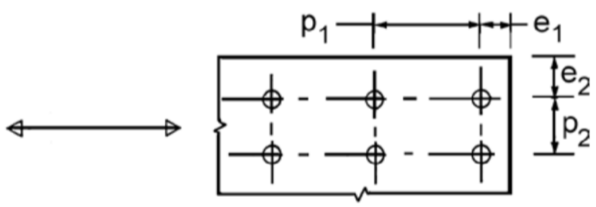
Bolt in tension and shear:

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

Table 18 — Values of the nominal minimum preloading force $F_{p,C}$ 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:

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

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:

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

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

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

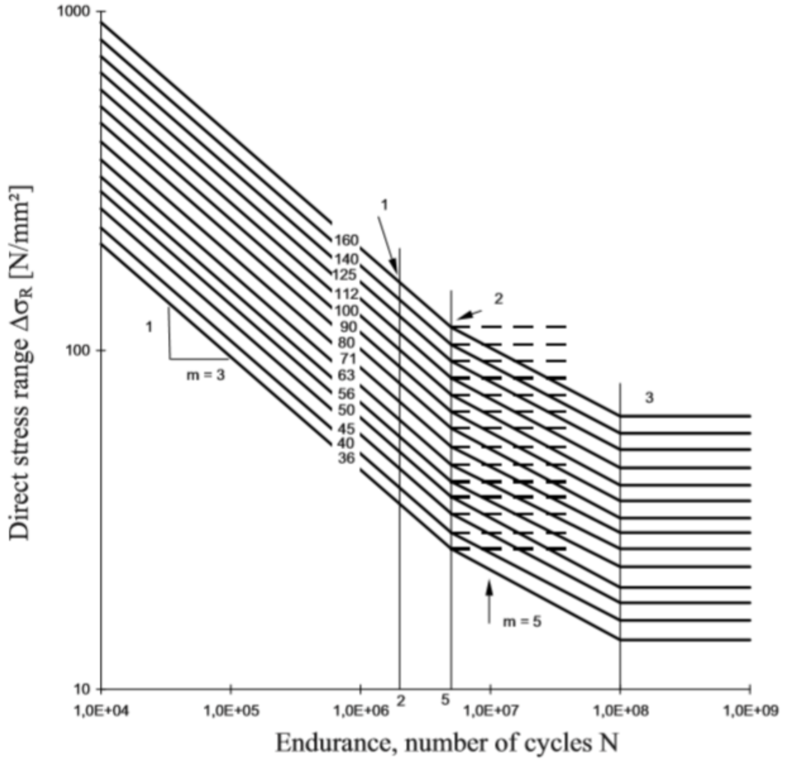
where a is the throat thickness of the weld.

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

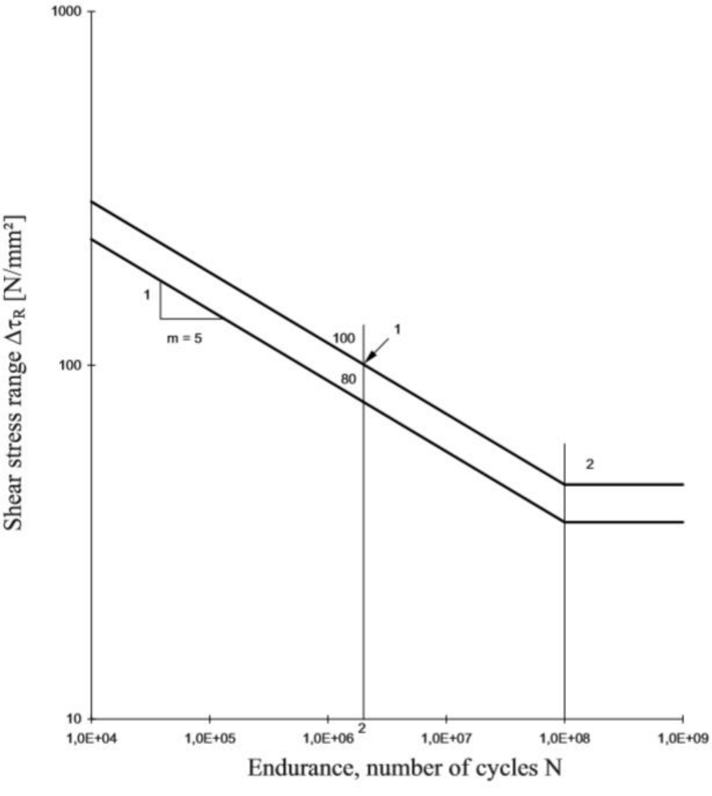
K, m = constants



1 *Detail category $\Delta\sigma_C$*

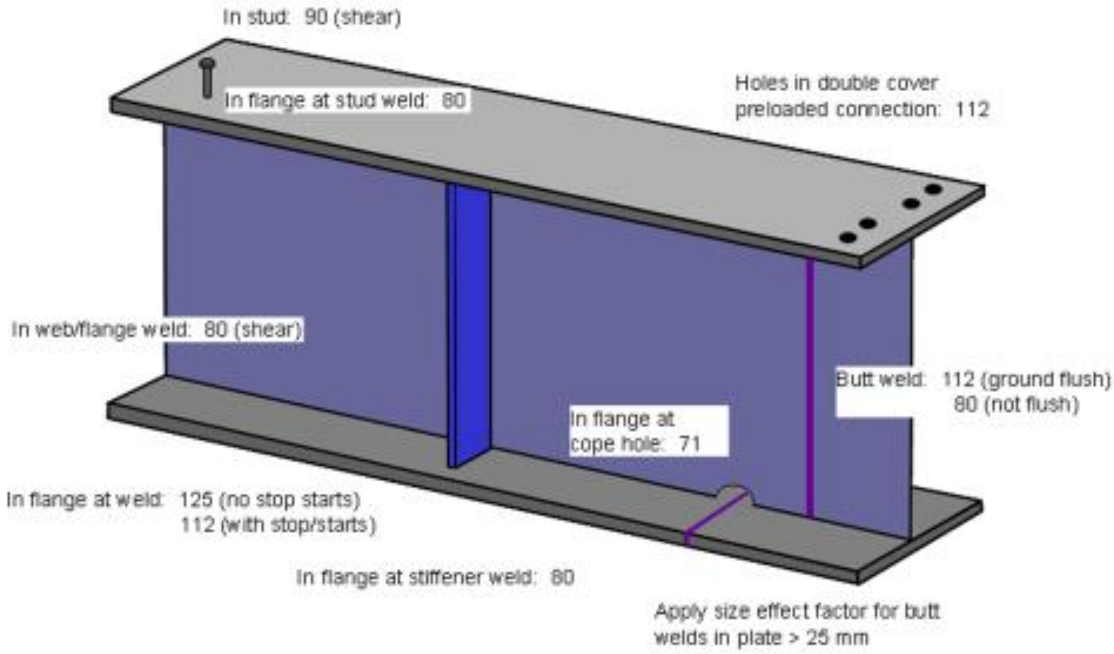
2 *Constant amplitude fatigue limit $\Delta\sigma_D$*

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

DS6: Composite beams

Headed shear studs:

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

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

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

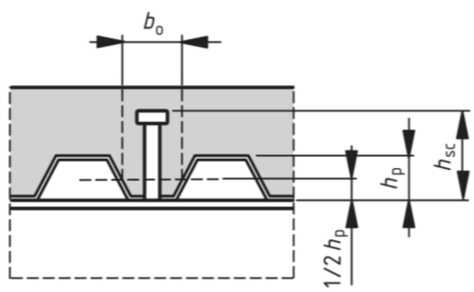
where:

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

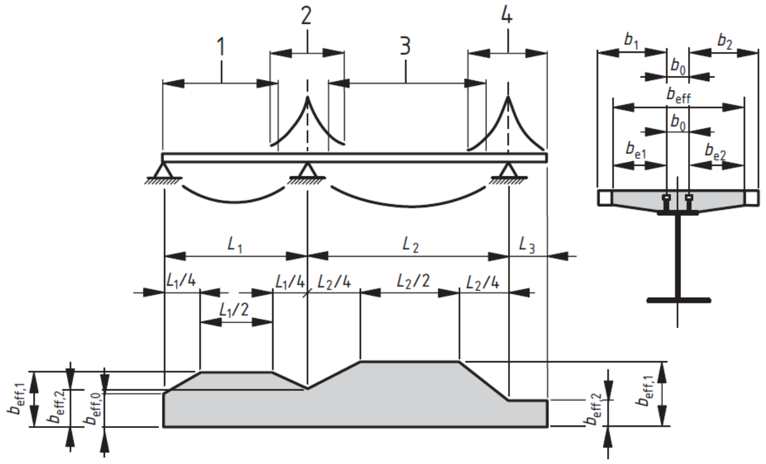
- Reduction factor for studs on steel decking:

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

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: $n = 2n_0 \qquad n_0 = E_a/E_{cm}$

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	$f_{ctm}=0{,}30\times f_{ck}^{(2/3)}$ ≤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 ε_{c1} (‰) $= 0{,}7\, f_{cm}^{0,31} < 2.8$
ε_{cu1} (‰)	3,5									3,2	3,0	2,8	2,8	2,8	see Figure 3.2 for $f_{ck} \geq 50$ Mpa ε_{cu1}(‰)$=2{,}8+27[(98-f_{cm})/100]^4$
ε_{c2} (‰)	2,0									2,2	2,3	2,4	2, 5	2,6	see Figure 3.3 for $f_{ck} \geq 50$ Mpa ε_{c2}(‰)$=2{,}0+0{,}085(f_{ck}-50)^{0,53}$
ε_{cu2} (‰)	3,5									3,1	2,9	2,7	2,6	2,6	see Figure 3.3 for $f_{ck} \geq 50$ Mpa ε_{cu2}(‰)$=2{,}6+35[(90-f_{ck})/100]^4$
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 ε_{c3}(‰)$=1{,}75+0{,}55[(f_{ck}-50)/40]$
ε_{cu3} (‰)	3,5									3,1	2,9	2,7	2,6	2,6	see Figure 3.4 for $f_{ck} \geq 50$ Mpa ε_{cu3}(‰)$=2{,}6+35[(90-f_{ck})/100]^4$