EGT3
ENGINEERING TRIPOS PART IIB

Tuesday 19 April 2016 9.30 to 11.00

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 Sheets (9 pages)
Engineering Data Book

10 minutes reading time is allowed for this paper.

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

1 (a) Figure 1(a) shows a simply-supported 8 m beam of $533 \times 210 \times 122$ UB section in grade S275 steel. Its web is in the plane of the paper. At each end, lateral deflections and twist rotations are prevented, but the beam ends are free to warp. Secondary beams connect at midspan (as denoted by the large cross). These beams prevent lateral deflections and twist rotation where they connect. These secondary beams apply a load W to the main beam, as shown. You may assume that the yield stress is 275 MPa for all components, and you may ignore self-weight.

Determine the maximum value of W that can be safely carried. [70%]

(b) Figure 1(b) shows a 4 m high beam-column, also of $533 \times 210 \times 122$ UB section in S275 steel, which is simply supported against major and minor axis flexure at top and bottom, such that the ends are fully restrained against sway. The supports prevent twist rotation at the ends but allow warping. A major axis end-moment of 300 kNm is applied at the base of the beam-column, as shown.

Determine a *single* axial-moment interaction diagram that governs stability involving *only* minor axis flexural buckling and lateral torsional buckling. (You may assume this is a straight line). Hence determine the maximum axial load P that can be sustained simultaneously with the applied 300 kNm end-moment. [30%]

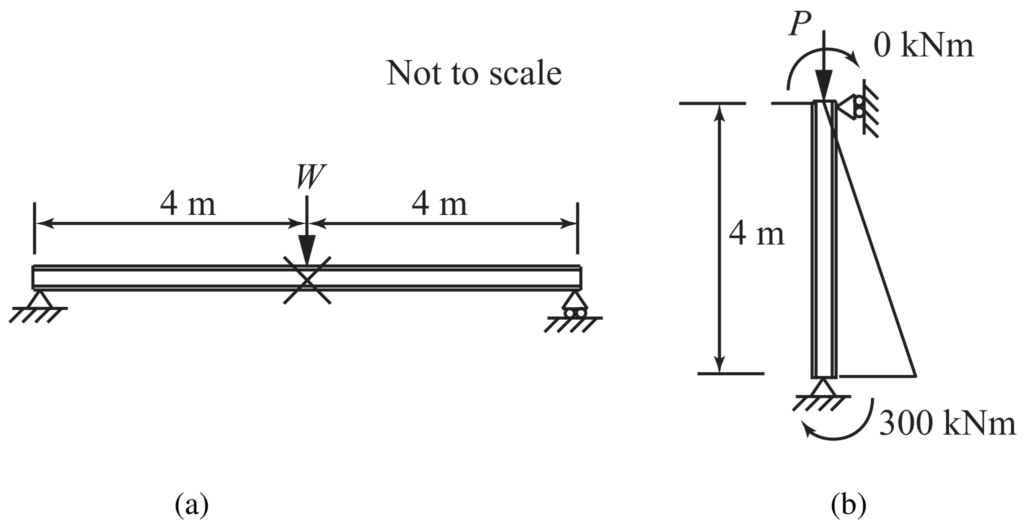


Fig. 1

2 (a) Stating any assumptions, derive the formula

$$F_j = \frac{M}{\sum_{i=1}^{N} \left(r_i^2 / r_j\right)}$$

for the shear F_j on bolt j in a group of N bolts, with all bolts acting in shear. Here M is the applied moment in the plane of the bolt group and r_i is the radial distance of the i^{th} bolt from the group centroid. [30%]

(b) Figure 2 shows a bolted connection between two steel plates. There are five bolts, each being M36 grade 8.8 and each acting in single shear. The connection is to carry a load P at an offset as shown.

(i) Identify which bolt or bolts are critical for this load case, and determine the maximum load P that can be carried by the bolt group. [60%]

(ii) What other features of this connection detail would need to be checked? [10%]

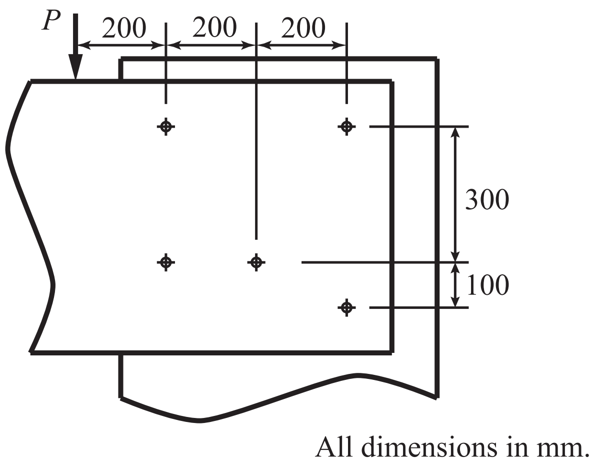


Fig. 2

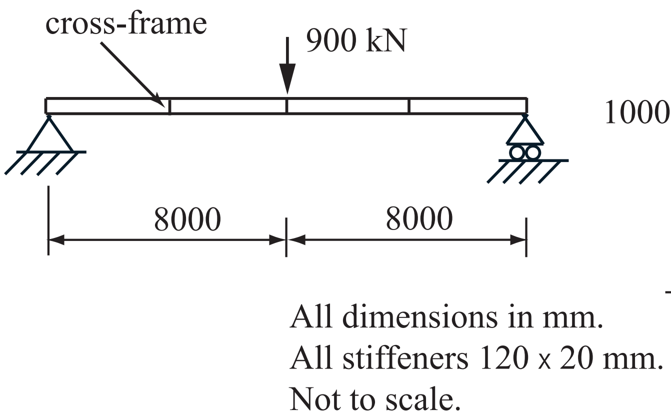
3 Figure 3(a) shows a box-girder beam simply-supported over a 16 m span. It has a cross-section as shown in Fig. 3(b), made by welding S275 steel plates of thickness 12 mm. Along its full length there are 120×20 mm longitudinal stiffeners in the top and bottom flanges as shown. There are transverse cross-frames at the supports and at 4 m intervals along the beam. The beam is fully braced against global lateral torsional buckling. It is to carry a point load of 900 kN at midspan, which includes all necessary load factors. You may ignore self-weight in all the following calculations.

(a) Determine the compactness or otherwise of the various parts of the cross-section. [10%]

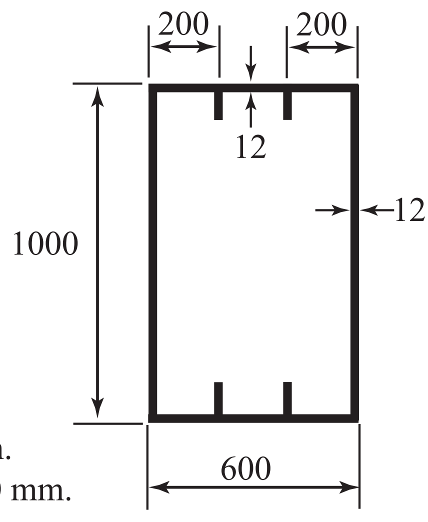
(b) Using a smeared section approximation to account for the stiffeners, estimate the major axis second moment of area. [20%]

(c) By assuming the stiffened top flange consists of pin-ended "T"-struts spanning between cross-frames, determine whether the top flange is adequately stiffened for this load case. [30%]

(d) Check the web panels for strength and stability. [40%]



(a)



(b)

Fig. 3

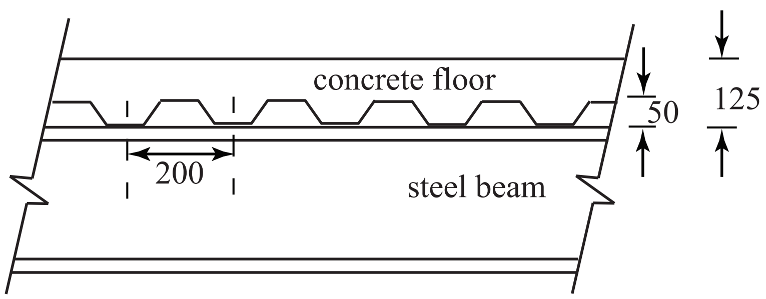
4 A composite floor is to span 10 m. It consists of a concrete slab of total thickness 125 mm, which includes 50 mm troughs from steel decking. The troughs are perpendicular to the supporting steel beams, as shown in Fig. 4. The supporting beams run the full length of the slab. They are 457 × 152 × 67 Universal Beams of S355 steel, simply supported at each end, and the transverse spacing between beam centres is 3 m. 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 (unfactored) uniformly distributed loads of 3 kN m^{-2} for permanent services and 7 kN m^{-2} for live loads. Partial safety factors of 1.35 for permanent loads and 1.5 for live loads are required.

(a) Assuming full composite action, determine whether the floor can carry the loads specified. [50%]

(b) Calculate the number of 100 mm × 25 mm shear studs needed for each beam to achieve full composite action. [20%]

(c) Estimate the maximum deflection due to short-term application of the unfactored live load and check the serviceability limit state, assuming a maximum allowable deflection of span/250 for this case. [30%]



All dimensions in mm.
(Drawing not to scale).

Fig. 4

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Data Sheets

DO NOT USE FOR ACTUAL DESIGN OF STRUCTURAL STEELWORK

FAM, DDS & KAS March 20, 2014

DS1: Basic Buckling Resistance Curves

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

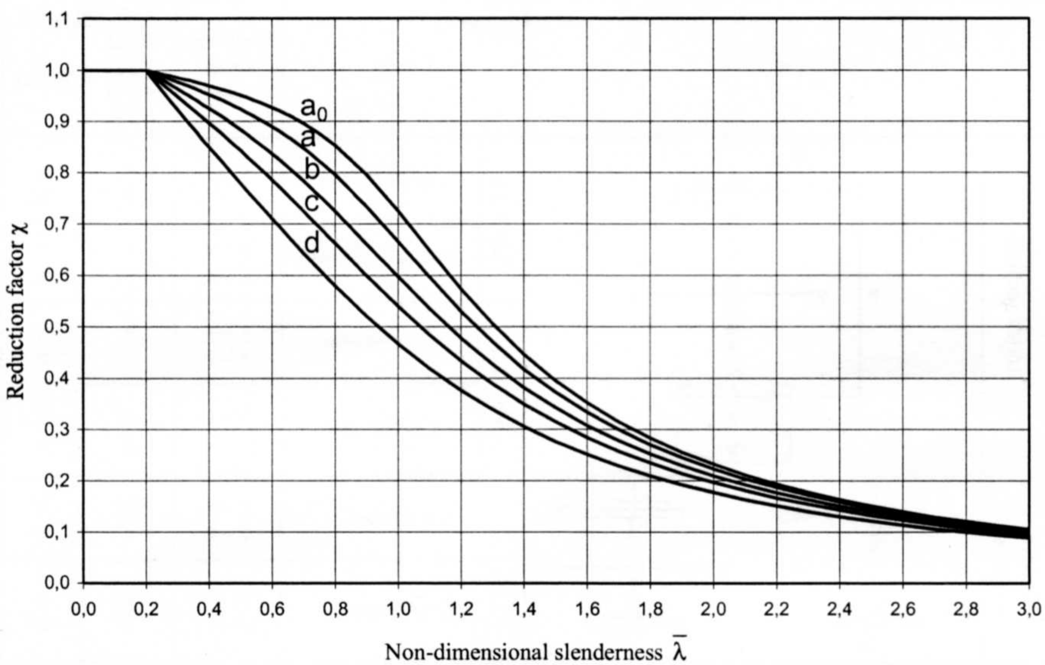


Figure 6.4: Buckling curves

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

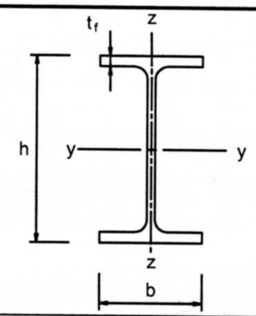
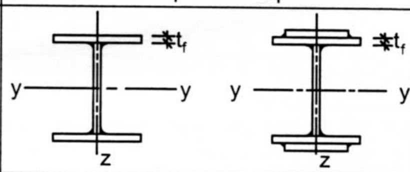
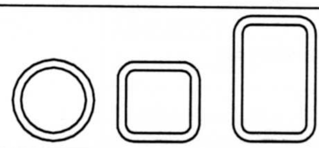
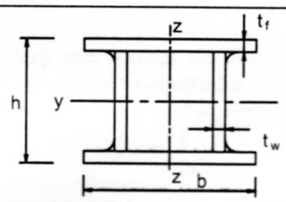
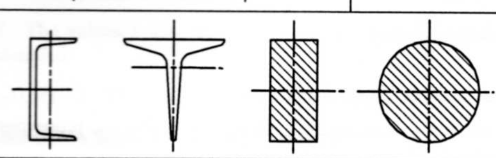
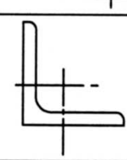
and the imperfection factor α appropriate for each curve is:

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:

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

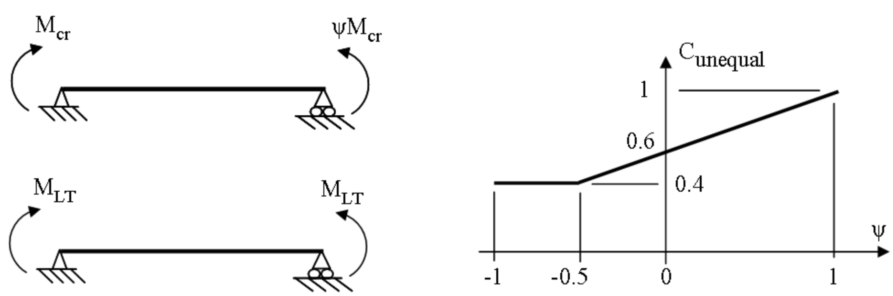
where EI, GJ and $E\Gamma$ are the minor axis flexural rigidity, the torsional rigidity and the warping rigidity respectively. (It is assumed that the supports prevent vertical, lateral and torsional deflections but do not restrain warping.)

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



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

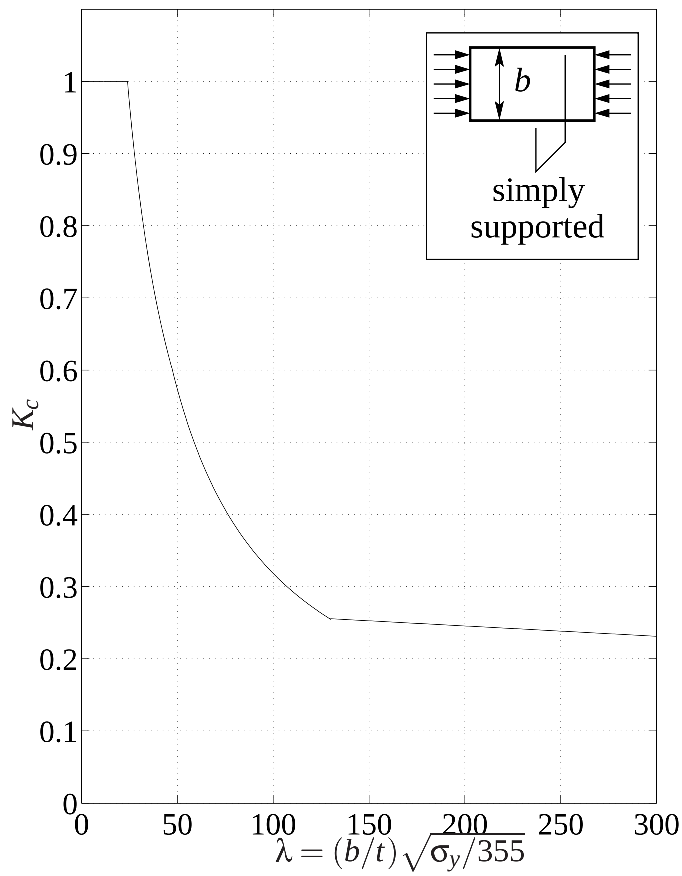
Lateral torsional buckling curve selection

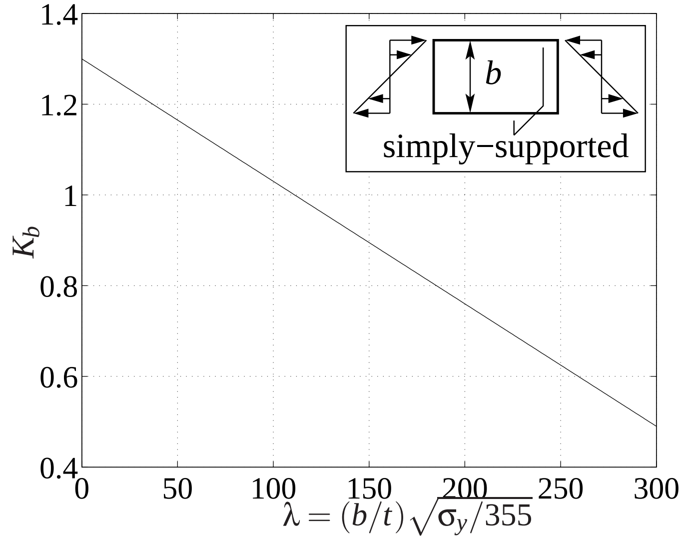
For lateral torsional buckling, the buckling resistance curves (DS1) may be used, with curves selected via the table below. Height h and width b are defined in DS2.

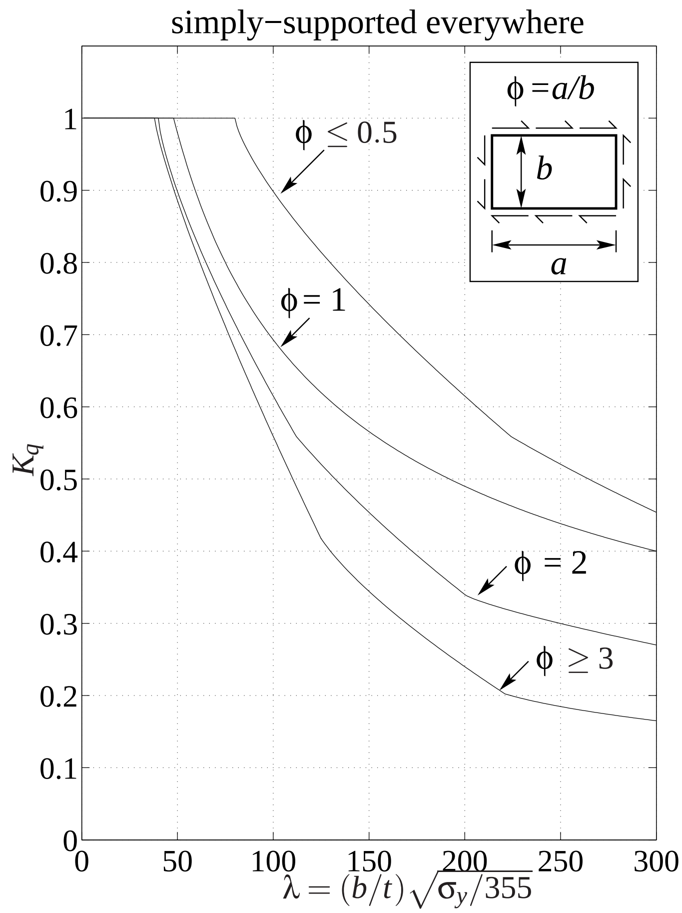
	Limits	Curve
Rolled I-sections	$h/b \leq 2$	a
	$h/b > 2$	b
Welded I-sections	$h/b \leq 2$	c
	$h/b > 2$	d
Other	-	d

DS4: Panel Strength and Plate Compactness

a. Panel strength: use the following three figures in the expressions overleaf.







$$\text{panel stability:}\quad \frac{\sigma_c}{\sigma_{cc}} + \left(\frac{\sigma_b}{\sigma_{bc}}\right)^2 + \left(\frac{\tau}{\tau_c}\right)^2 \leq 1$$

note 1: $\sigma_{cc} = K_c\sigma_y$; $\sigma_{bc} = K_b\sigma_y$; $\tau_c = K_q\tau_y$ $(K_q\sigma_y/\sqrt{3})$.

note 2: τ is the shear stress on the panel, σ_c is the average compressive stress and σ_b is the maximum bending stress.

$$\text{panel local strength:}\quad \sigma \leq \sqrt{\sigma_y^2 - 3\tau^2}$$

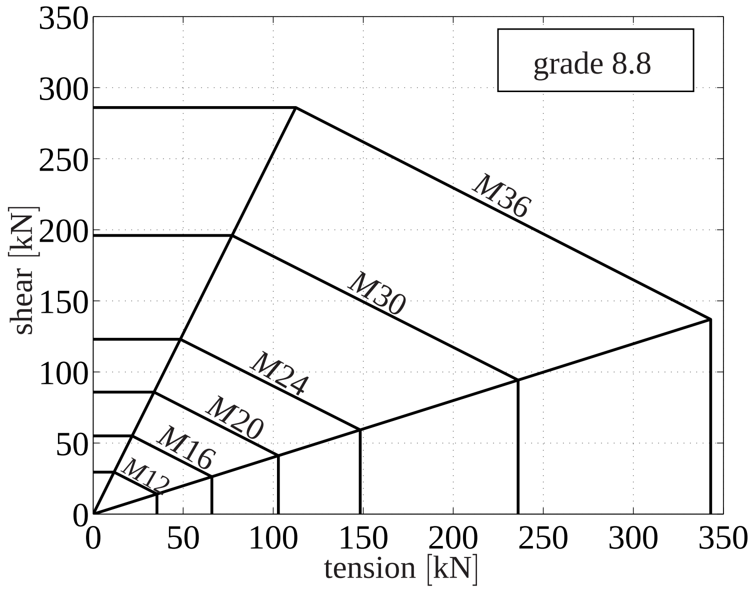
note 3: effective width, b_e, of compression flange with stiffener spacing, b, is $K_c b$.

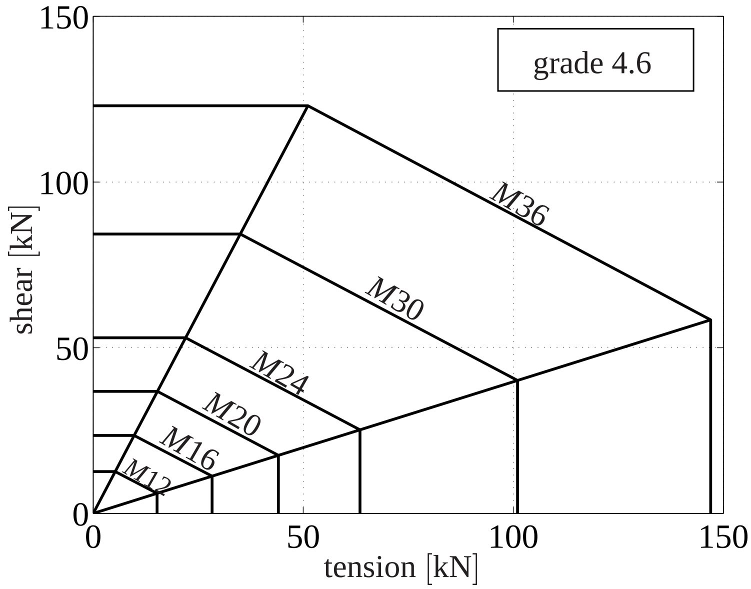
b. Plate compactness.

member and action	compact if λ $(= (b/t)\sqrt{\sigma_y/355})$
internal plate in compression	≤ 24
external plate in compression	≤ 8
internal plate in bending (no axial load)	≤ 56

DS5: Connector Capacity and Fatigue Life

a. Bolt strength in combined tension and shear.





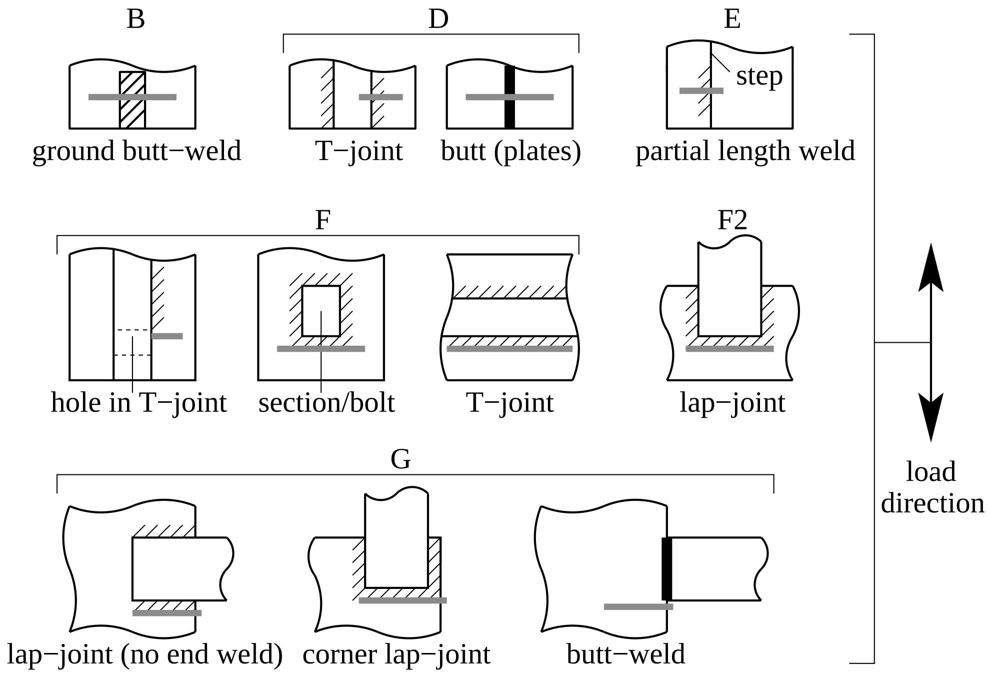
b. Bolt placement.

edge and end distances:	$\geq 2.5\phi$
spacing between bolt axes:	$\leq 32t$ and $\geq 2.5\phi$

note 1: ϕ is the bolt hole diameter; t is the total thickness of joint plates.

c. Weld capacity. Shear force transmitted across weld $\leq$ throat area $\times$ τ_y.

d. Weld classification. Plan-views of typical crack locations, which are shown in grey for clarity. Where a crack is shown to overlap with a step or T-joint edge, it has become vertical.



e. Weld fatigue life. The number of repetitions, N, to failure under stress amplitude, σ_r, is

$$\begin{aligned} N\sigma_r^m &= K_2 \qquad \text{for } \sigma_r > \sigma_0 \\ N\sigma_r^{m+2} &= K_2\sigma_0^2 \quad \text{for } \sigma_r < \sigma_0 \end{aligned}$$

where σ_r and σ_0 are in MPa, and the constants m and K_2 take different values for each class of weld from the following table.

detail class	m	K_2	σ_0 [MPa]
G	3	0.25×10^{12}	29
F2	3	0.43×10^{12}	35
F	3	0.63×10^{12}	40
E	3	1.04×10^{12}	47
D	3	1.52×10^{12}	53
B	4	1.01×10^{15}	100

note 2: for complex variations, use Miner's Law

$$\frac{n_1}{N_1} + \frac{n_2}{N_2} + \cdots + \frac{n_i}{N_i} + \cdots \leq 1$$

n_i is the number of *applied* cycles under σ_{ri}; N_i is the *total* number of possible cycles under σ_{ri}. Each σ_{ri} is given by application of the *Reservoir Method* described in the notes.

DS6: Composite Construction

a. Headed shear stud capacity.

headed studs		f_{cd} [MPa]			
diameter [mm]	height [mm]	20	30	40	50
		stud shear strength [kN]			
25	100	139	154	168	183
19	100	90	100	109	119
13	65	42	47	52	57

note 1: for sheeting ribs orthogonal to the supporting beam, single studs have full strength but paired studs each have 80% strength.

b. Transformed section data.

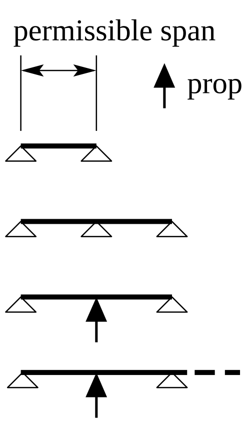
Young's modulus for grade 30 concrete, E_c, depends on duration of loading as:

short term: $E_c = 28$ GPa
long term: $E_c = 14$ GPa

Effective width of slab, b_e, is equal to $0.25 \times$ span but less than b, the beam spacing.

The maximum deflection must be less than the total span/250.

c. Profiled decking capacity.



		$t = 0.9$ mm			$t = 1.2$ mm		
support condition	total slab depth [mm]	imposed loading [kN/m^2]					
		2.5	5	7.5	2.5	5	7.5
		permissible spans [m]					
single span (no props)	100	2.3	2.3	2.3	2.8	2.8	2.8
	150	2.0	2.0	2.0	2.4	2.4	2.4
multiple span (no props)	100	2.3	2.3	2.3	2.7	2.7	2.7
	150	2.0	2.0	2.0	2.4	2.4	2.4
single span (one prop)	100	4.5	3.9	3.3	5.1	4.1	3.6
	150	4.0	4.0	4.0	4.7	4.7	3.7
multiple span (with props)	100	4.6	4.0	3.4	5.1	4.1	3.6
	150	4.1	4.1	4.1	4.8	4.8	4.8

note 2: table above only applies to 50 mm deep troughs; thickness of sheeting is t.