EGT3
ENGINEERING TRIPOS PART IIB

Monday 30 April 2018 9.30 to 11.10

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 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.

1 (a) The column BE forms part of the subframe shown in Fig. 1. It is rigidly connected to beams ABC and DEF at the top and base as shown. The column has a UC 305 × 305 × 97 section in S460 steel and the beams are UB 457 × 191 × 67 in S235 steel. All webs are in the plane of the diagram. The dimensions and support conditions are shown. The hyperbolic graphs in Fig. 2 have axis coordinates k_1 and k_2 which are appropriate flexural rigidity ratios of members connecting at the column top and base. Using these graphs, determine the effective length of the column BE and hence the value of the applied load P that will lead to major axis flexural buckling of the column. [40%]

(b) The beam shown in Fig. 3 spans 14 m. It has a UB 254 × 146 × 31 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. Concentrated vertical loads W and $2W$ are applied via side beams as shown. These side beams prevent lateral deflection and twist rotation where they connect. You may ignore self-weight. Determine the value of W at the Ultimate Limit State. [60%]

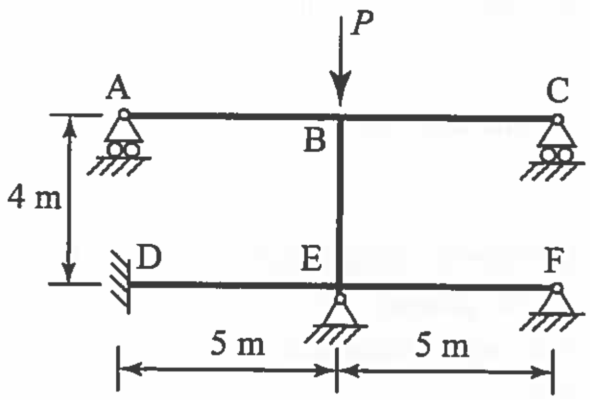

Fig. 1

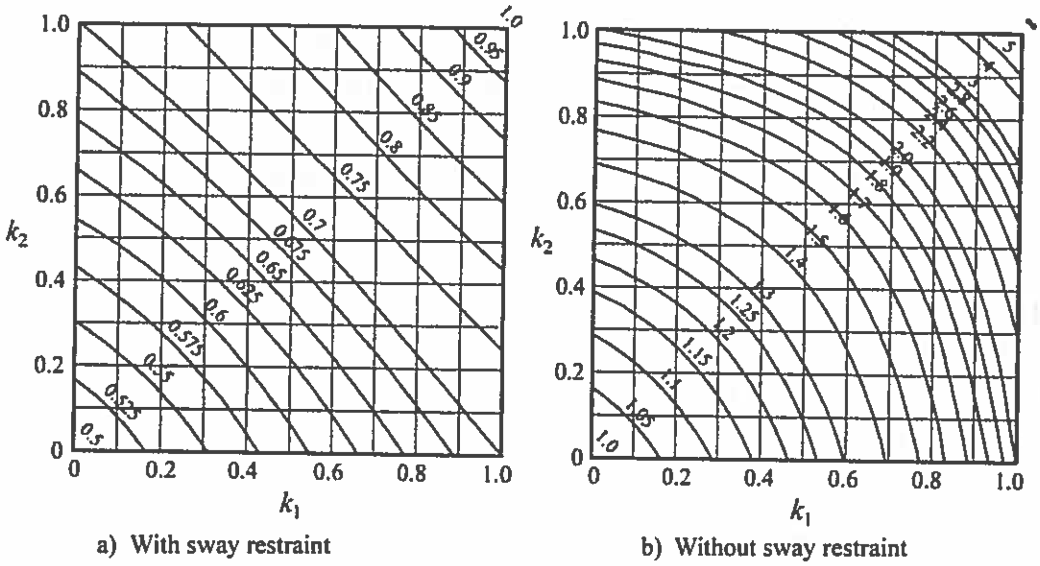

Fig. 2

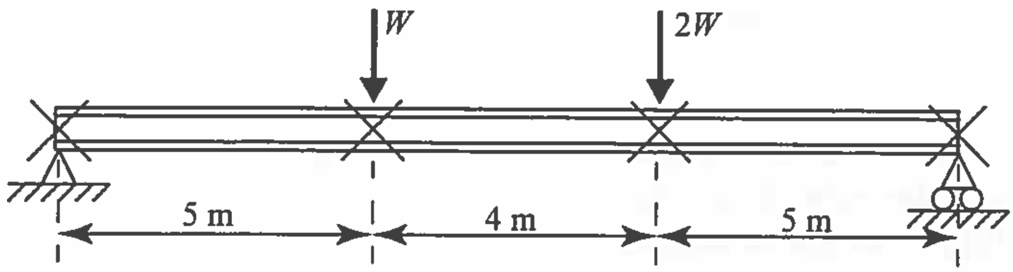

Fig. 3

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2 (a) Stating all assumptions, derive from first principles the Perry-Robertson equation for the buckling resistance of an Euler column made of steel. Hence derive the formula for the reduction factor χ that is shown beneath the graphs in Data Sheet 1. [50%]

(b) Show that the derivation includes an imperfection parameter $\eta = \alpha\overline{\lambda}$ that may be decomposed into three dimensionless factors which depend in turn on section properties, initial bow and slenderness. [10%]

(c) Describe the typical residual stresses that may develop in the production of a hot-rolled I-beam, and explain how the effect of these is taken into account by the Perry-Robertson formula. [10%]

(d) Describe any theoretical difficulties that may be encountered in trying to perform a similar Perry-Robertson analysis for the lateral torsional buckling resistance of an I-beam. [10%]

(e) Explain then how it is that many steel design codes can nevertheless use a Perry-Robertson formulation for lateral torsional buckling resistance. [10%]

(f) For long beams of length L, Data Sheet 3 shows that the critical moment for lateral torsional buckling scales as $1/L$. However, the buckling graphs of Data Sheet 1 were derived for the flexural buckling of Euler columns, in which case they scale as $1/L^2$ for large L. Explain then how the Data Sheet 1 curves can nevertheless be used to describe lateral torsional buckling of beams. [10%]

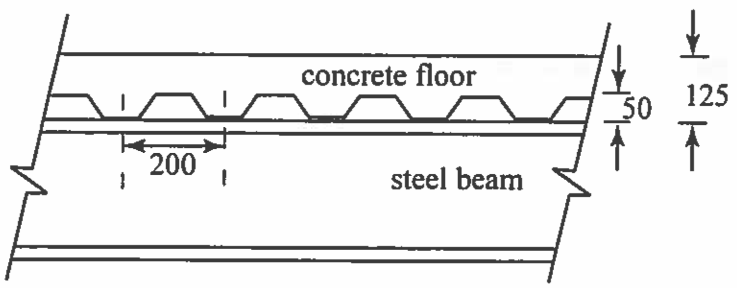
3 A composite floor is to span 10 m. It consists of a concrete slab of total thickness 125 mm, which includes 50 mm troughs from steel decking. The troughs are perpendicular to the supporting steel beams, as shown in Fig. 4. The supporting beams run the full length of the slab. They are UB 457 × 152 × 82 of S235 steel, simply supported at each end. 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 2 kN m^{-2} for permanent services and live loads of w per unit floor area. Partial safety factors of 1.35 for permanent loads and 1.5 for live loads are required.

(a) Assuming full composite action, determine the strength of the floor, and hence determine the maximum permissible live load w. [50%]

(b) Propose a suitable arrangement of shear studs to achieve full composite action. [20%]

(c) If the maximum allowable deflection due to short-term application of the unfactored live load w is span/250, determine the value of w for which serviceability governs. [30%]



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

Fig. 4

(TURN OVER

4 Figure 5(a) shows a portal frame of height 10 m with a 20 m span. It has the same stiffened box cross-section throughout, as shown in Fig. 5(b), made by welding S235 steel plates of thickness 10 mm. Both supports may be assumed to be pinned, and one support may be considered to be on a roller. It is fully braced against global flexural buckling of the columns and lateral torsional buckling of the beam. There are transverse cross-frames at each end of the beam and at 2.5 m intervals in between. The beam is to carry a distributed load of 100 kN/m over its full length. This includes the self-weight of the beam and no further load factors need to be applied. The columns may be assumed to be adequately designed, and you are only required to consider the beam.

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

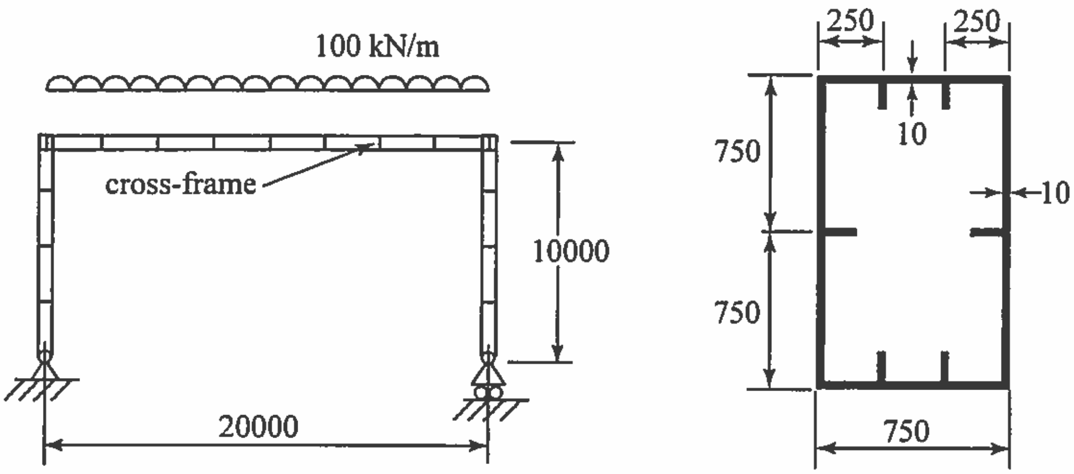
(b) Using a smeared section approximation to account for the stiffeners, estimate the major axis second moment of area. [20%]

(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. [20%]

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

(e) If the design is found to be inadequate in any location(s), suggest potential improvement(s). [10%]

(cont.



All dimensions in mm.
All stiffeners 180 x 20 mm.
Not to scale.

(a) (b)

Fig. 5

END OF PAPER

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4D10 Structural Steelwork 2014/15

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)

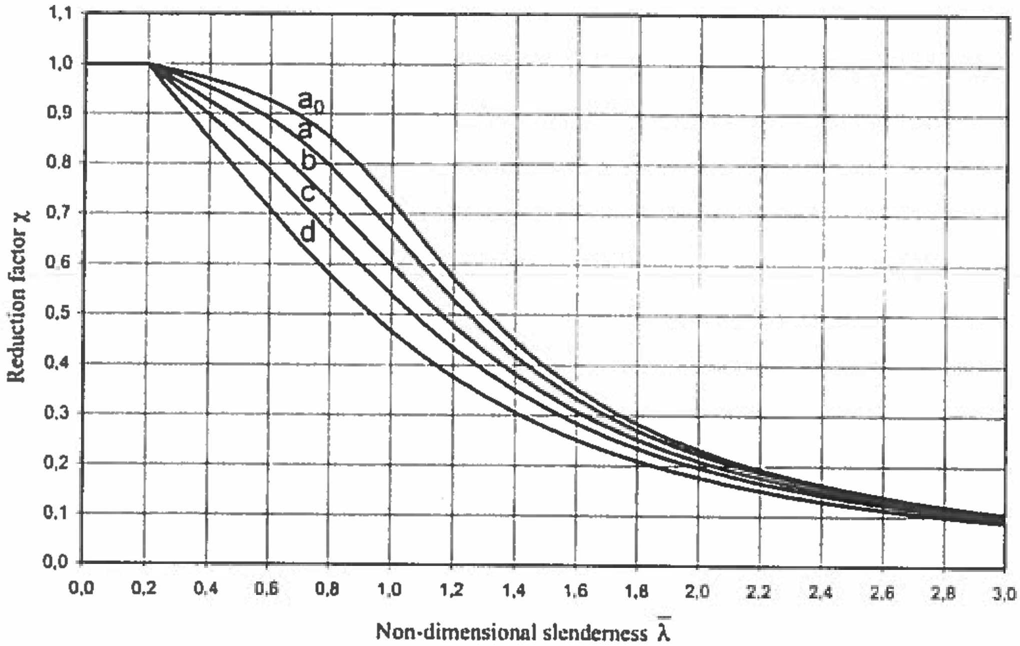


Figure 6.4: Buckling curves

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

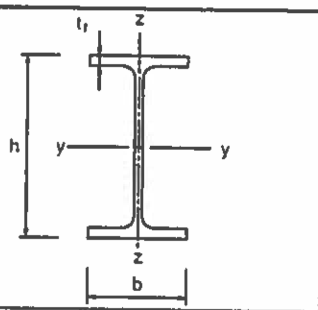
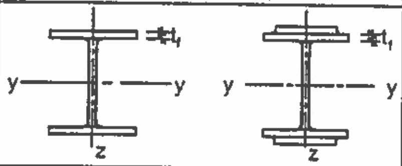
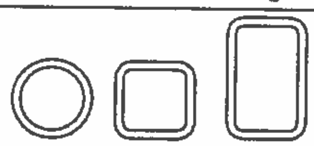
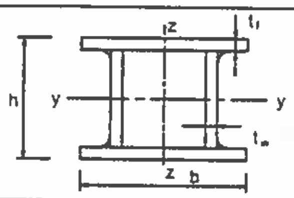
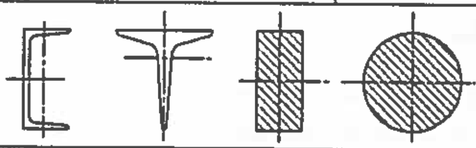
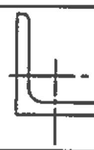
and the imperfection factor α appropriate for each curve is:

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
			$40 \text{ mm} < t_f \le 100$	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:

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

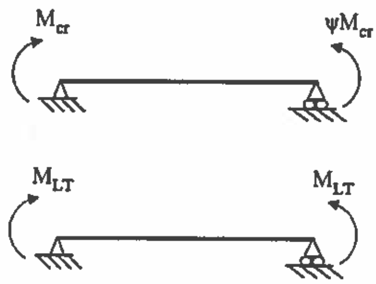
where EI, GJ and $E\Gamma$ are the minor axis flexural rigidity, the torsional rigidity and the warping rigidity respectively. (It is assumed that the supports prevent vertical, lateral and torsional deflections but do not restrain warping.)

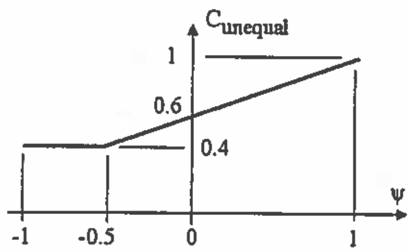
For a doubly-symmetric I-beam

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

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





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

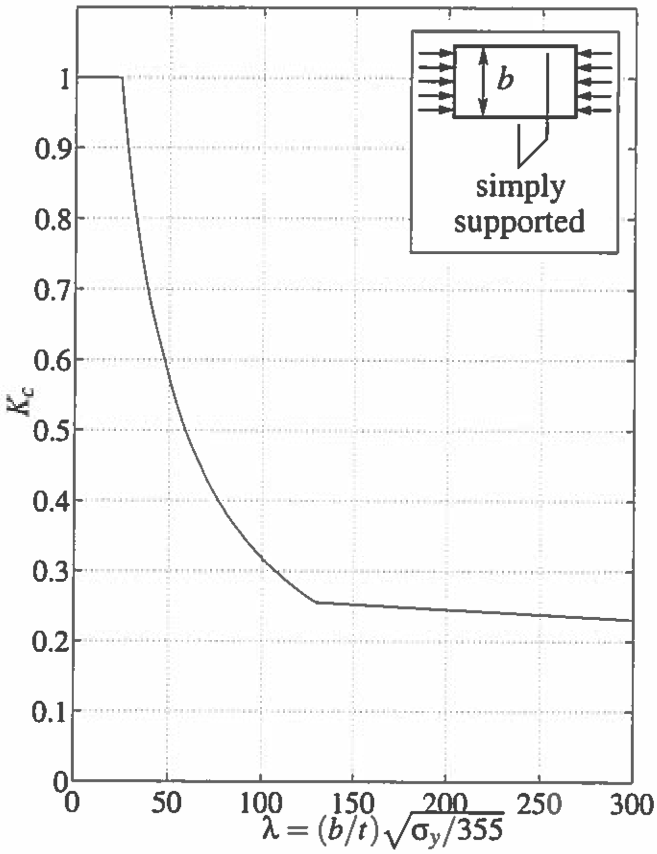
Lateral torsional buckling curve selection

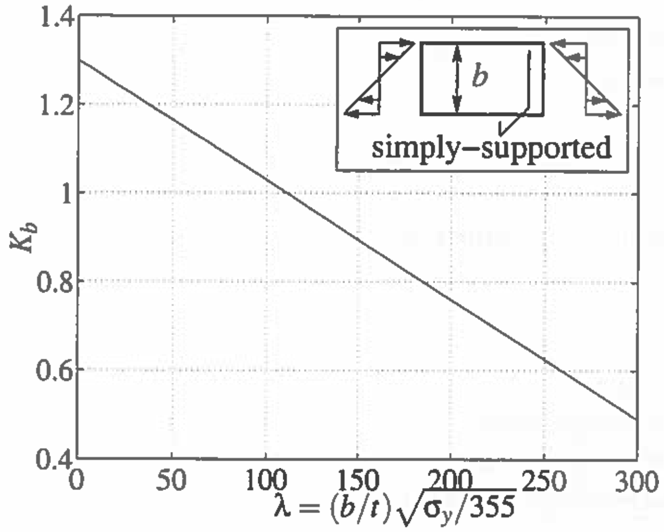
For lateral torsional buckling, the buckling resistance curves (DS1) may be used, with curves selected via the table below. Height h and width b are defined in DS2.

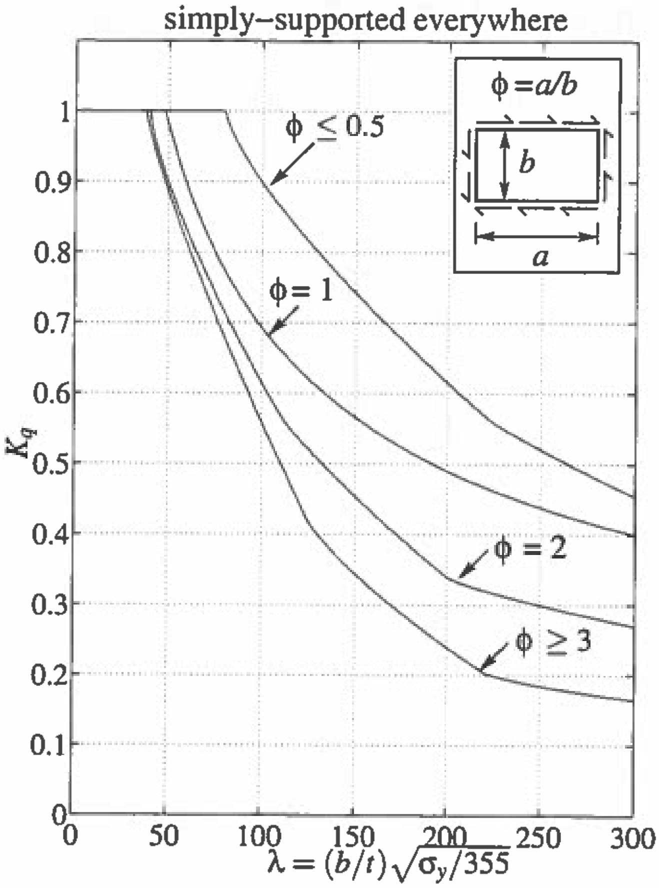
	Limits	Curve
Rolled I-sections	$h/b \leq 2$	a
	$h/b > 2$	b
Welded I-sections	$h/b \leq 2$	c
	$h/b > 2$	d
Other	-	d

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.

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

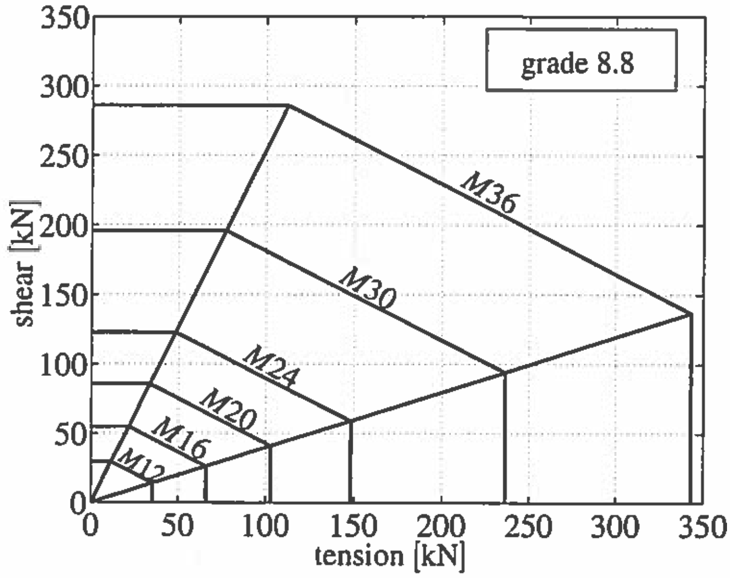
note 3: effective width, b_e, of compression flange with stiffener spacing, b, is $K_c b$.

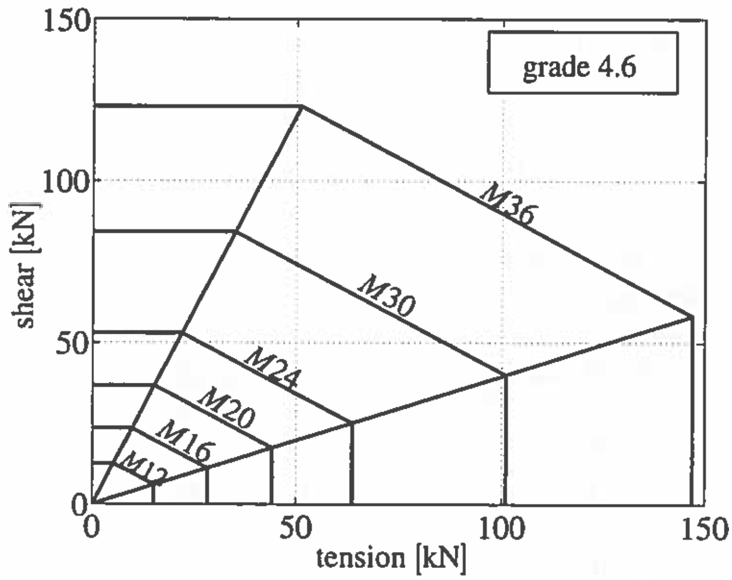
b. Plate compactness.

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.





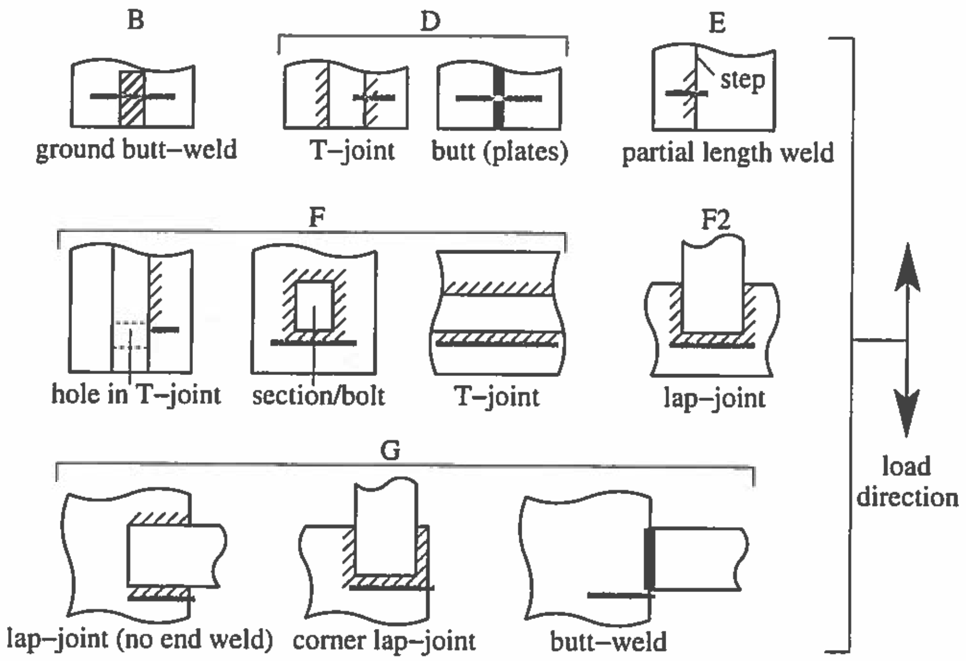
b. Bolt placement.

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 $\times\ \tau_y$.

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

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

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.

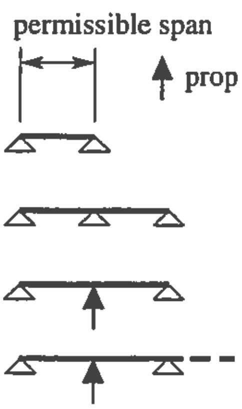
Young's modulus for grade 30 concrete, E_c, depends on duration of loading as:

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

Effective width of slab, b_e, is equal to 0.25 × 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.

ENGINEERING TRIPOS PART IIB 2018

4D10 STRUCTURAL STEELWORK

Where the datasheets have been used to read factors in the following, there may be some margin of error either way in the numerical answers.

1a) (k1, k2) = (0.39, 0.35), effective length factor = 1.28; critical load = 17.6 MN
1b) critical middle span with M_max = 71.33 kNm, W_max = 8.69 kN

3a) x_p = 54.8 mm; M_design = 813 kNm; unfactored live load = 10.2 kN/m^2
3b) 25 X 100 mm studs in pairs in each trough
3c) unfactored live load = 24 kN/m^2

4a) compactness: flange, 18.7; web, 59.3; stiffeners, 7.3
4b) 21.51 X 10^9 mm^4
4c) N_design = 1247 kN vs N_actual = 1064 kN, thus adequate
4d) critical panel in bending (no shear) Kc = 0.5, Kb = 1.15, Kq = 0; stability margin = 0.84, strength margin = 0.74, thus adequate. Shear stress at ends = 33.3 MPa, less than tau_Y = 136MPa, thus adequate in strength.

F.A.M/K.A.S. May 2018