Version SDG/4

EGT3
ENGINEERING TRIPOS PART IIB

Thursday 27 April 2017 14.00 to 15.30

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 Figure 1 shows a simply-supported 15 m beam of 356 × 171 × 67 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 the third points, 5 m from each end (as denoted by the large crosses). These beams prevent lateral deflections and twist rotation where they connect. These secondary beams apply a load F to the main beam at each connection, as shown. An axial load P is also to be applied to the beam, as shown. You may assume that the yield stress is 275 MPa for all components, and you may ignore self-weight.

(a) In the absence of axial load P, determine the value of the transverse loads F at the Ultimate Limit State. [50%]

(b) In the absence of transverse loads F, determine the value of the axial load P at the Ultimate Limit State. [50%]

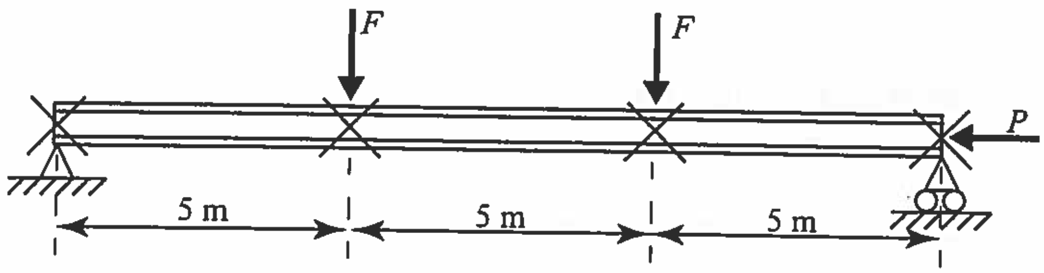


Fig. 1

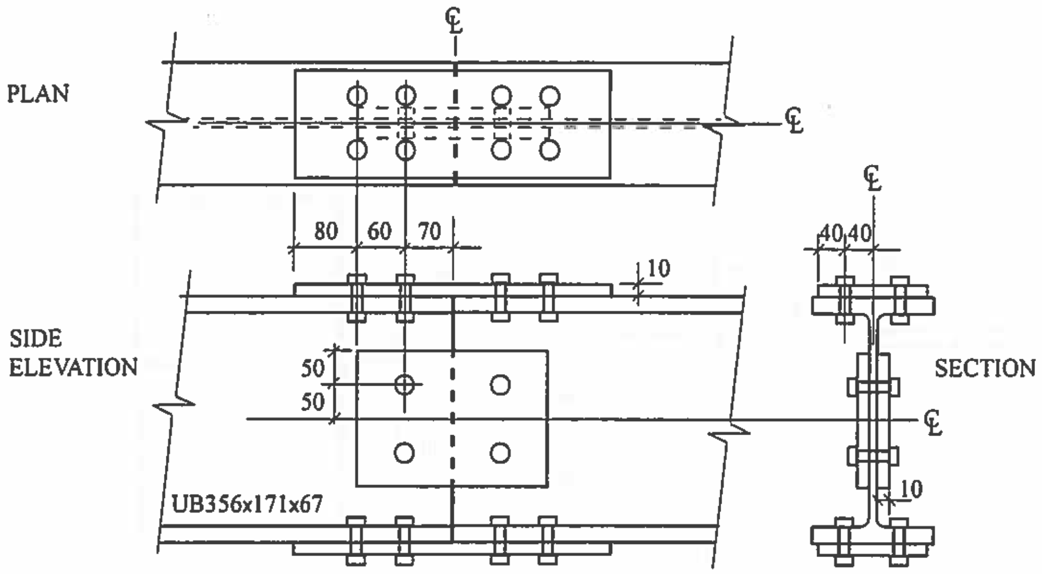
2 Figure 2 shows a splice connecting two beams of UB356 × 171 × 67 section in steel Grade S275. There are four splice plates, one at each flange and one either side of the web. All splice plates are 10 mm thick and of Grade S450 steel. All bolts are M16 of Grade 8.8. The detail is symmetric about every centreline.

You may assume that the load capacity in bearing of a plate adjacent to a bolt is $2\sigma_y dt$, where d is the bolt diameter, t is the plate thickness and σ_y is the yield stress of the plate material.

At each stage of the following checks, state clearly what part of the connection you are checking and what assumptions you are making.

(a) In the absence of moment at the central interface, determine the Ultimate Limit State vertical shear capacity S of the splice. [30%]

(b) In the absence of vertical shear at the central interface, determine the Ultimate Limit State major-axis moment capacity M of the splice. [70%]



Not to scale. All dimensions in mm.

Fig. 2

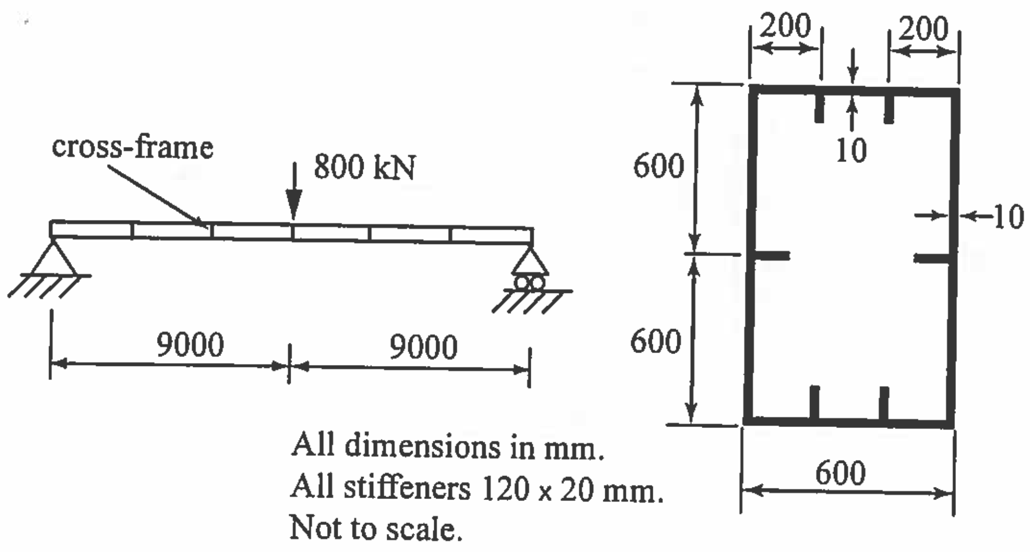
3 Figure 3(a) shows a box-girder beam simply-supported over an 18 m span. It has a cross-section as shown in Fig. 3(b), made by welding S450 steel plates of thickness 10 mm. Along its full length there are 120 × 20 mm longitudinal stiffeners in the top and bottom flanges, and in the webs, as shown. There are transverse cross-frames at the supports and at 3 m intervals along the beam. The beam is fully braced against global lateral torsional buckling. It is to carry a point load of 800 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

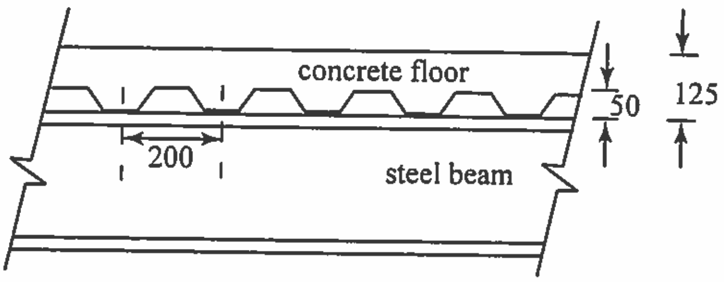
4 A composite floor is to span 12 m. It consists of a concrete slab of total thickness 125 mm, which includes 50 mm troughs from steel decking. The troughs are perpendicular to the supporting steel beams, as shown in Fig. 4. The supporting beams run the full length of the slab. They are 533 × 210 × 82 Universal Beams of S355 steel, simply supported at each end. The transverse spacing between beam centres is 3.5 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

END OF PAPER

THIS PAGE IS BLANK

ENGINEERING TRIPOS PART IIB 2017

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) 38.3 kN
1b) 831 kN

2a) 94 kN
2b) 80 kN

3a) Compactness: flange, 20.3; web, 65.3; stiffeners, 6.8
3b) 10.24 X 10^9 mm^4
3c) N_design = 1010 kN vs N_actual = 928 kN, thus adequate
3d) Kc = 0.42, Kb = 1.1, Kq = 0.73; stability margin = 0.6, stiffness margin = 0.43, thus panel adequate

4a) M_design = 1322 kNm vs M_actual = 1137 kNm, thus adequate
4b) 50 studs minimum, but 60 in total if placed in each trough
4c) 21.7 mm (< span/250 = 12 m/250 = 48 mm)

F.A.M/K.A.S. May 2012

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)

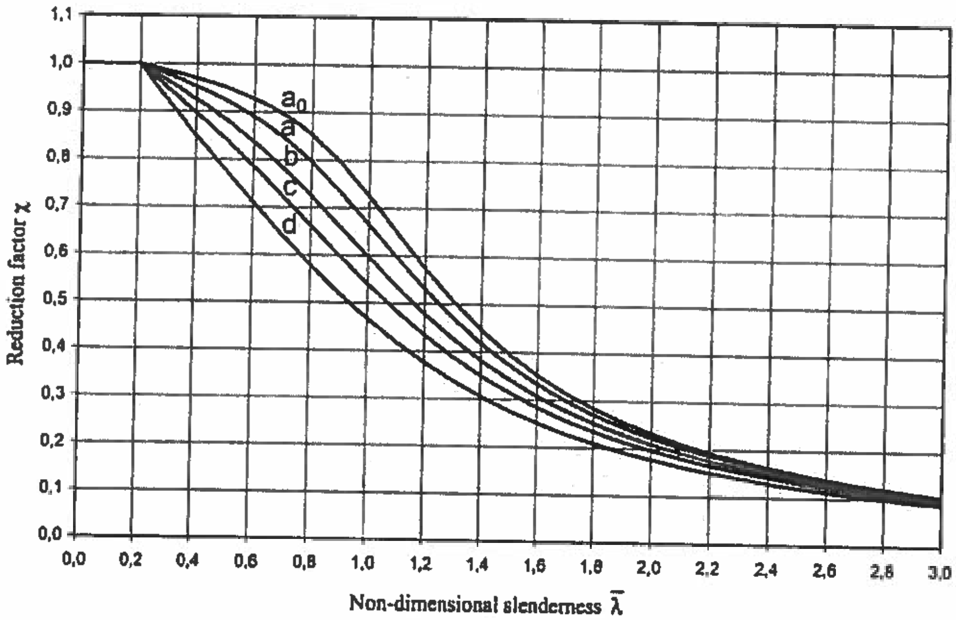


Figure 6.4: Buckling curves

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

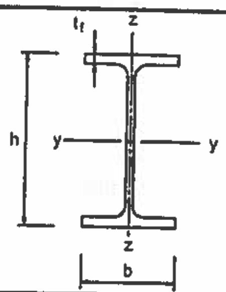
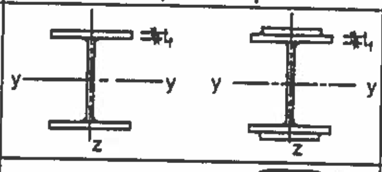
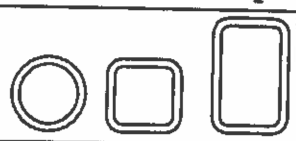
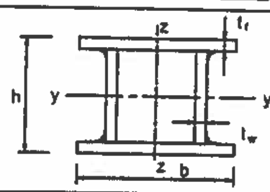
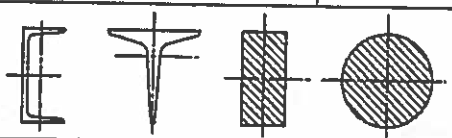
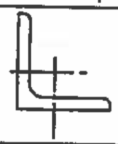
and the imperfection factor α appropriate for each curve is:

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		$h/b > 1{,}2$	$t_f \leq 40$ mm	y – y	a	a_0
				z – z	b	a_0
			40 mm $< t_f \leq 100$	y – y	b	a
				z – z	c	a
		$h/b \leq 1{,}2$	$t_f \leq 100$ mm	y – y	b	a
				z – z	c	a
			$t_f > 100$ mm	y – y	d	c
				z – z	d	c
Welded I-sections			$t_f \leq 40$ mm	y – y	b	b
				z – z	c	c
			$t_f > 40$ mm	y – y	c	c
				z – z	d	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:

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

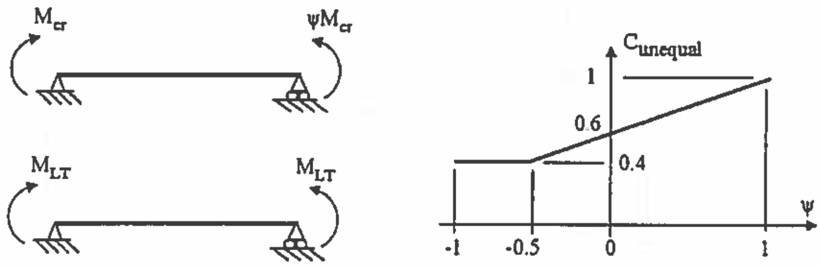
where EI, GJ and $E\Gamma$ are the minor axis flexural rigidity, the torsional rigidity and the warping rigidity respectively. (It is assumed that the supports prevent vertical, lateral and torsional deflections but do not restrain warping.)

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



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

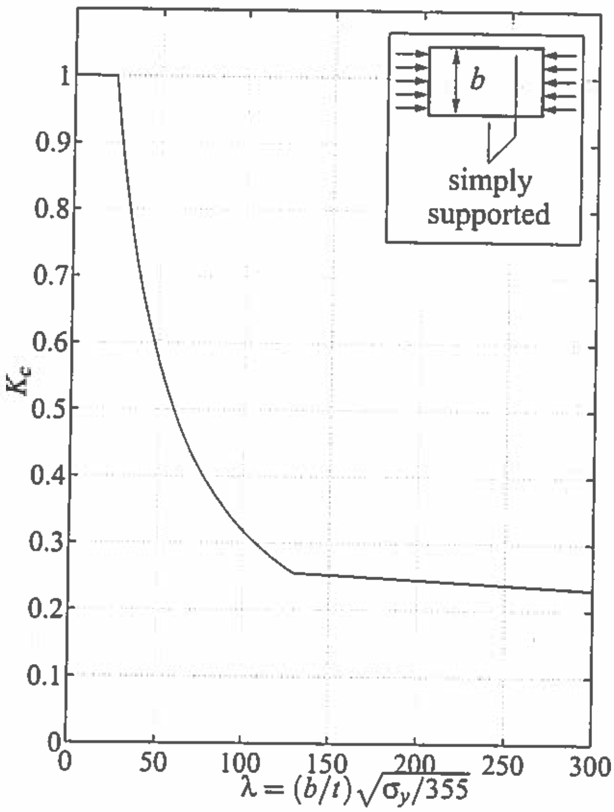
Lateral torsional buckling curve selection

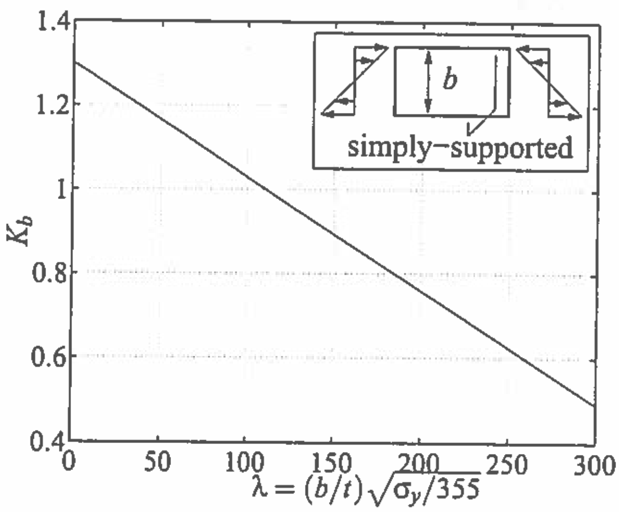
For lateral torsional buckling, the buckling resistance curves (DS1) may be used, with curves selected via the table below. Height h and width b are defined in DS2.

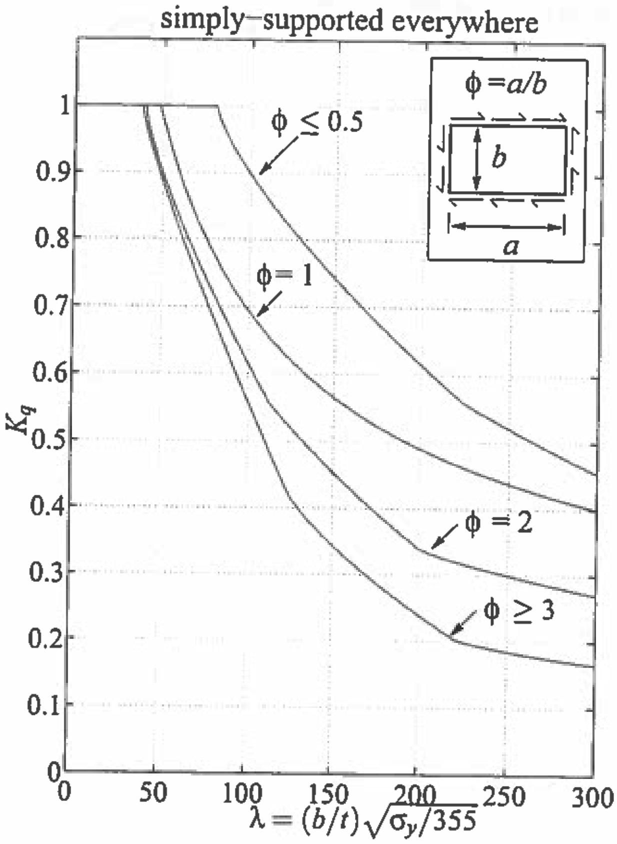
	Limits	Curve
Rolled I-sections	$h/b \leq 2$	a
	$h/b > 2$	b
Welded I-sections	$h/b \leq 2$	c
	$h/b > 2$	d
Other	-	d

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.

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

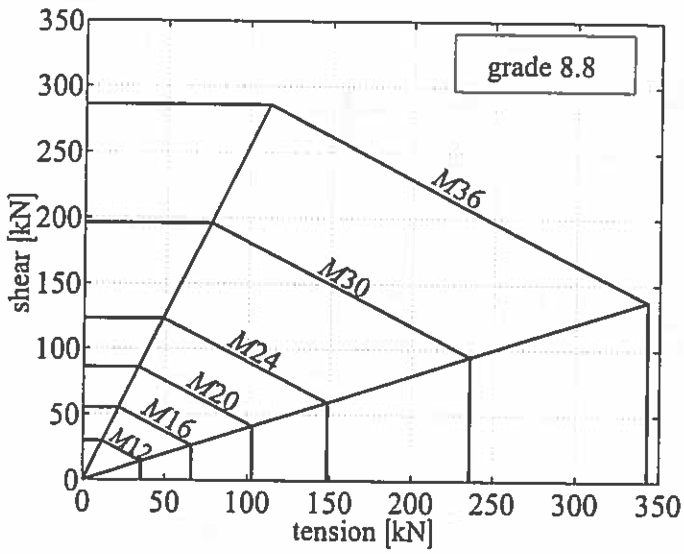
note 3: effective width, b_e, of compression flange with stiffener spacing, b, is $K_c b$.

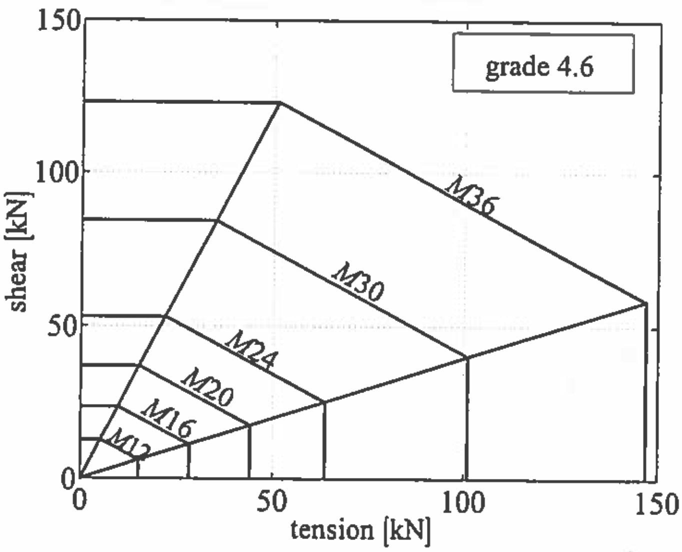
b. Plate compactness.

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.





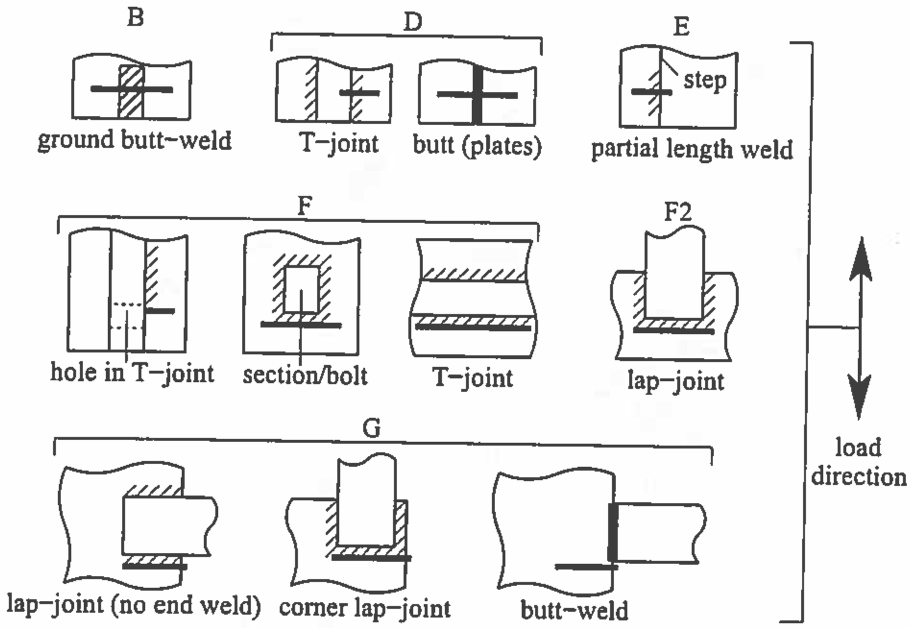
b. Bolt placement.

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

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

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.

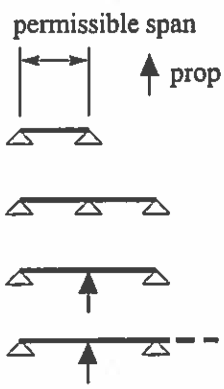
Young's modulus for grade 30 concrete, E_c, depends on duration of loading as:

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

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

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

c. Profiled decking capacity.



support condition	total slab depth [mm]	$t = 0.9$ mm			$t = 1.2$ 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.