ENGINEERING TRIPOS PART IIB

Friday 6 May 2011 2.30 to 4

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.*

Attachment: Structural Steelwork Data Sheets (9 pages).

STATIONERY REQUIREMENTS	SPECIAL REQUIREMENTS
Single-sided script paper	Engineering Data Book
Graph paper	CUED approved calculator allowed

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

1 (a) Briefly explain why defining the generalised slenderness $\bar{\lambda}$ of a beam via an equation of the form

$$\bar{\lambda} = \sqrt{\frac{\text{Plastic Moment}}{\text{Elastic Moment}}}$$

has advantages over definitions that treat beam slenderness in terms of a dimensionless effective length. [20%]

(b) Figure 1 shows a simply-supported 12 m beam ABCD braced at intermediate points B and C. It has 533 × 210 × 92 UB cross-section and is of Grade S275 steel. The web is in the plane of the diagram. Factored point loads of 60 kN and 120 kN are to be carried at B and C respectively. Lateral deflection and twist rotation are prevented at points A, B, C and D. Ignore self-weight. You may assume that all components of the section resistance are at least Class 2 (compact). Any reduction of moment capacity due to shear effects may be ignored. Determine if the member is adequate to carry the loads. [80%]

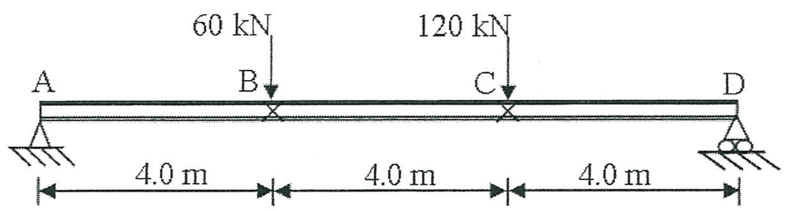


Fig. 1

2 A column has an effective length of 3 m with respect to minor axis flexural buckling. It is fully-braced against major axis flexural buckling. It has $356 \times 406 \times 235$ UC cross-section and is of Grade S275 steel. You may assume that all components of the section are at least Class 2 (compact). Any reduction of moment capacity due to shear effects may be ignored.

(a) Construct the local plastic capacity interaction diagram with axial forces (in kN) on the y-axis and major axis moments (in kNm) on the x-axis. You may use the bilinear approximation such that, were the graph suitably non-dimensional, it would pass through the point $(1, a/2)$ where a is the web fraction. [30%]

(b) Assuming that there are no applied minor axis moments and that any applied major axis end-moments are equal and opposite, construct the interaction diagram which describes the region of possible combinations of axial load and major axis end-moments for which the column is adequate against minor axis flexural buckling and lateral-torsional buckling effects. You may assume that this interaction diagram is linear between points on the M and N axes. [70%]

3 The rectangular portal frame ABCD in Fig. 2(a) is fully fixed at both feet. The frame has height L and width $3L$. The bending stiffness is uniform throughout the frame. It is initially stress-free.

A vertical load W is applied downwards to the midspan as shown in Fig. 2(a). The resulting axial force in the beam is $27W/88$ and the bending moment of largest magnitude is $39WL/88$ occurring at the midspan of the beam.

The beam has the uniform plate-girder cross-section shown in Fig. 2(b) (not drawn to scale). It has fully-effective 90×12 mm stiffeners spaced as shown. All steel is Grade S355. Cross-frames (diaphragms) are provided at 1 m centres along the beam.

If $W = 1000$kN and $L = 6$m, and assuming that the frame does not deflect out of the plane of the page and that the columns do not buckle, check:

(a) the adequacy of the top flange as an effective column between cross-frames; [50%]

(b) the adequacy of the most heavily stressed web panel in the centre of the beam. [50%]

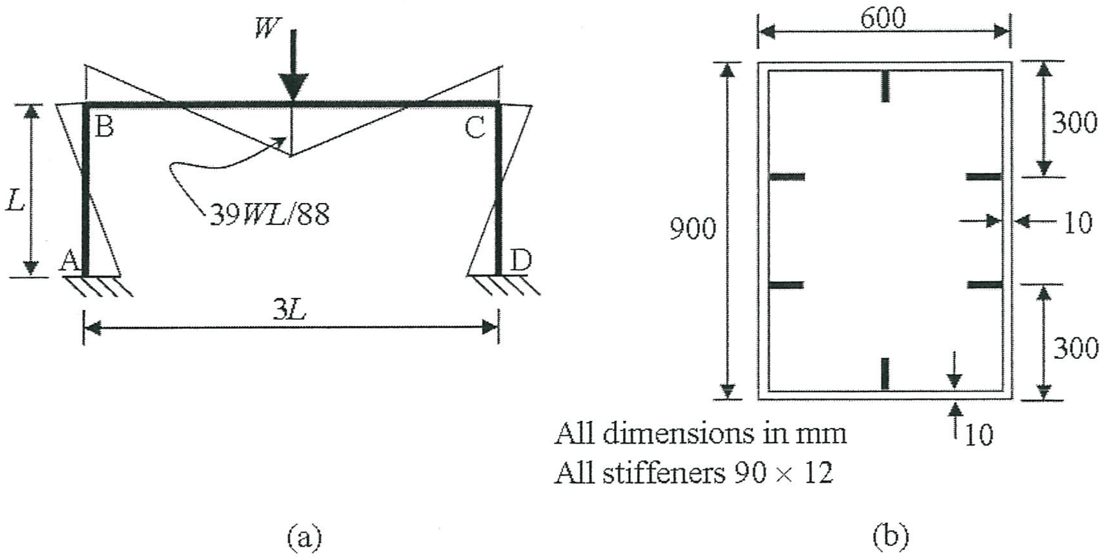


Fig. 2

4 A composite floor consists of a concrete slab of maximum thickness 100 mm cast onto profiled steel decking, as shown in Fig. 3. Note that the diagram is not drawn to scale. The floor carries an imposed load of 5.5 kPa and a load from permanent services of 3.0 kPa, with corresponding load factors of 1.5 and 1.35 respectively. The concrete has a density of 2400 kg.m^{-3} and a design strength $f_{cd} = 30$ MPa. The slab is supported by $406 \times 178 \times 60$ UBs of Grade S275. These are at 2.8 m centres and orthogonal to the deck troughs. The beams are 10 m long. They are simply supported at their ends and the slab does not extend past the ends of the beams.

(a) Show that, if there is full composite action, the floor can carry the specified permanent load, imposed load and self-weight. [50%]

(b) Calculate the total number of 65 mm $\times$ 13 mm shear studs needed for each UB to achieve full composite action. [20%]

(c) Estimate the central deflection induced by short-term application of the imposed load. [30%]

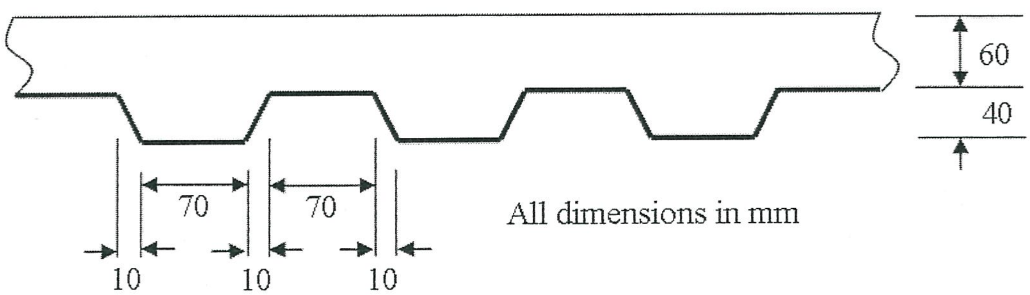


Fig. 3

END OF PAPER

4D10 Structural Steelwork 2010/11

Data Sheets

DO NOT USE FOR ACTUAL DESIGN OF STRUCTURAL STEELWORK

FAM, DDS & KAS April 14, 2011

DS1: Basic Buckling Resistance Curves

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

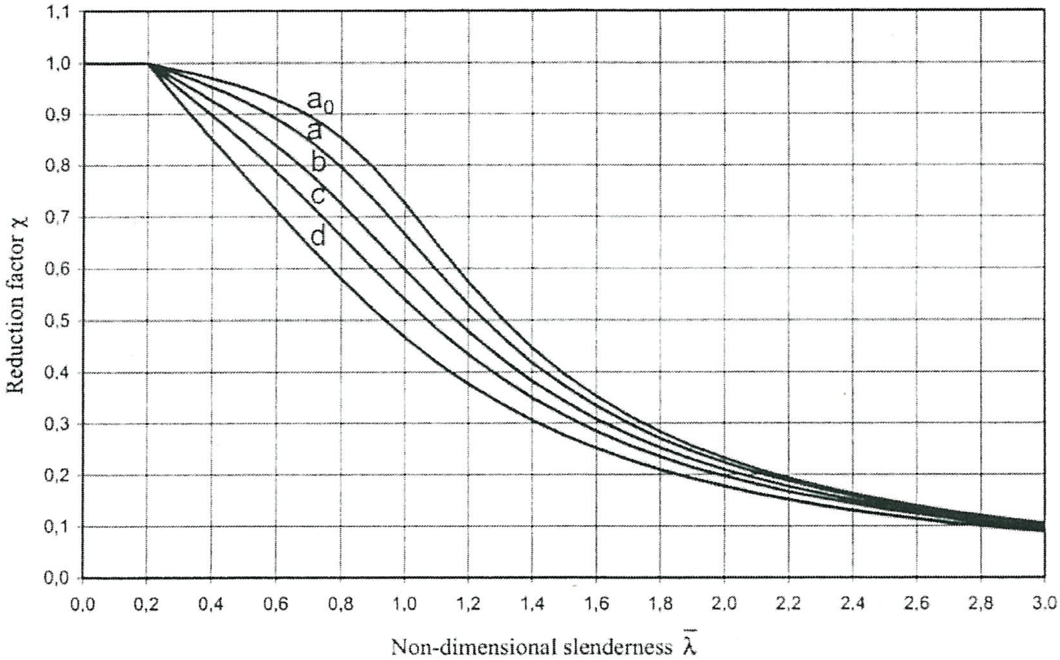


Figure 6.4: Buckling curves

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

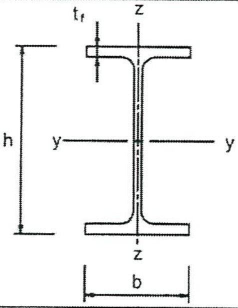
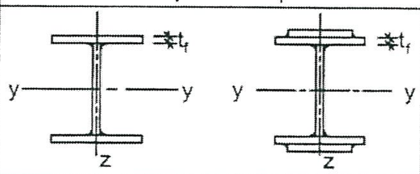
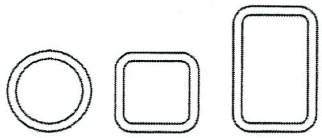
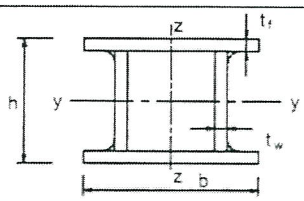
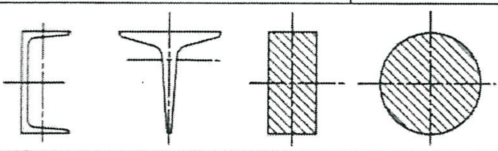
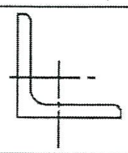
and the imperfection factor α appropriate for each curve is:

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:

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

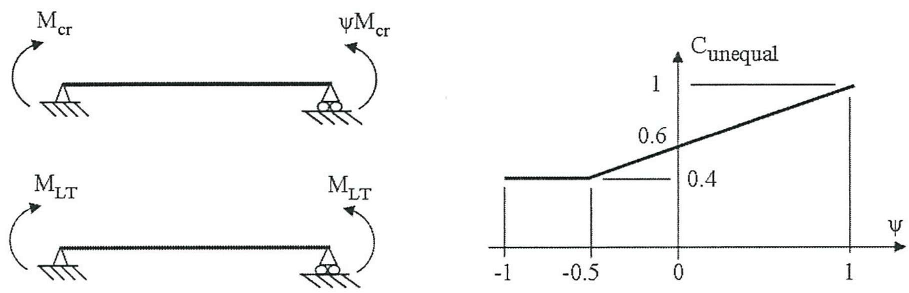
where EI, GJ and $E\Gamma$ are the minor axis flexural rigidity, the torsional rigidity and the warping rigidity respectively. (It is assumed that the supports prevent vertical, lateral and torsional deflections but do not restrain warping.)

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



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

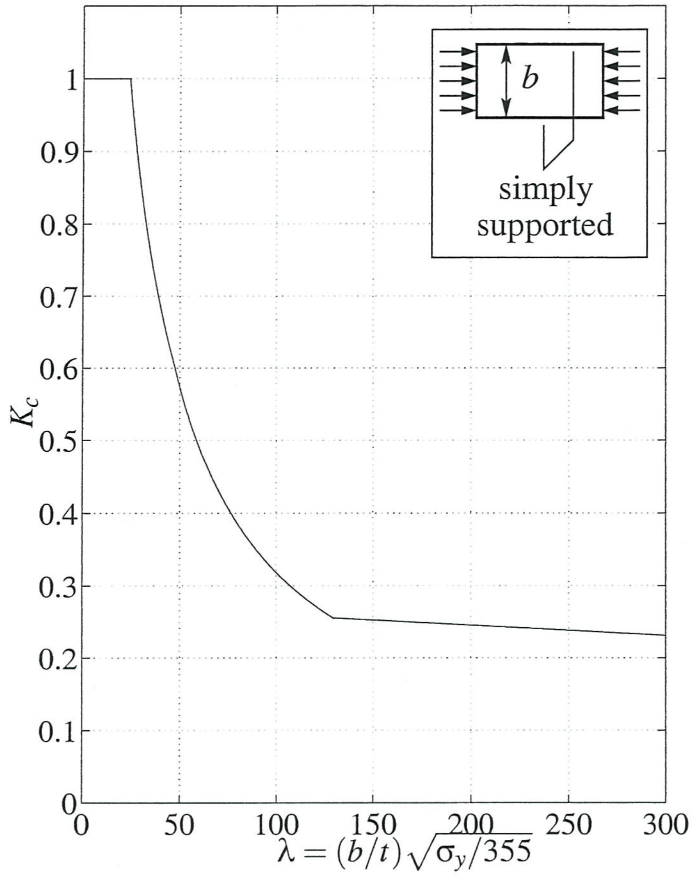
Lateral torsional buckling curve selection

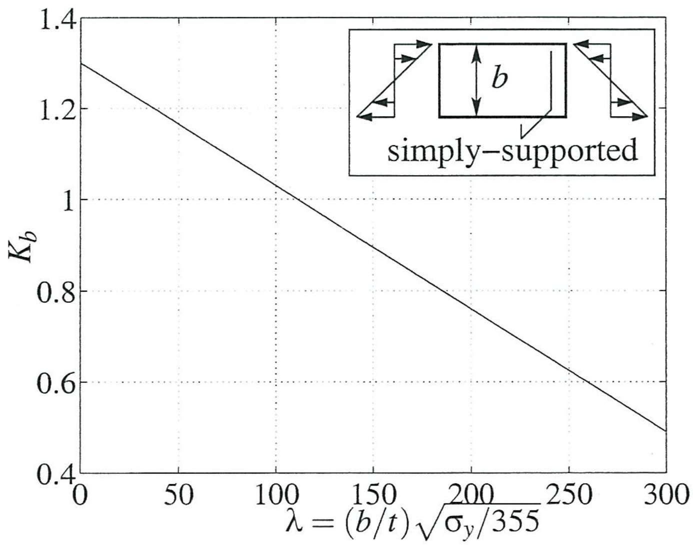
For lateral torsional buckling, the buckling resistance curves (DS1) may be used, with curves selected via the table below. Height h and width b are defined in DS2.

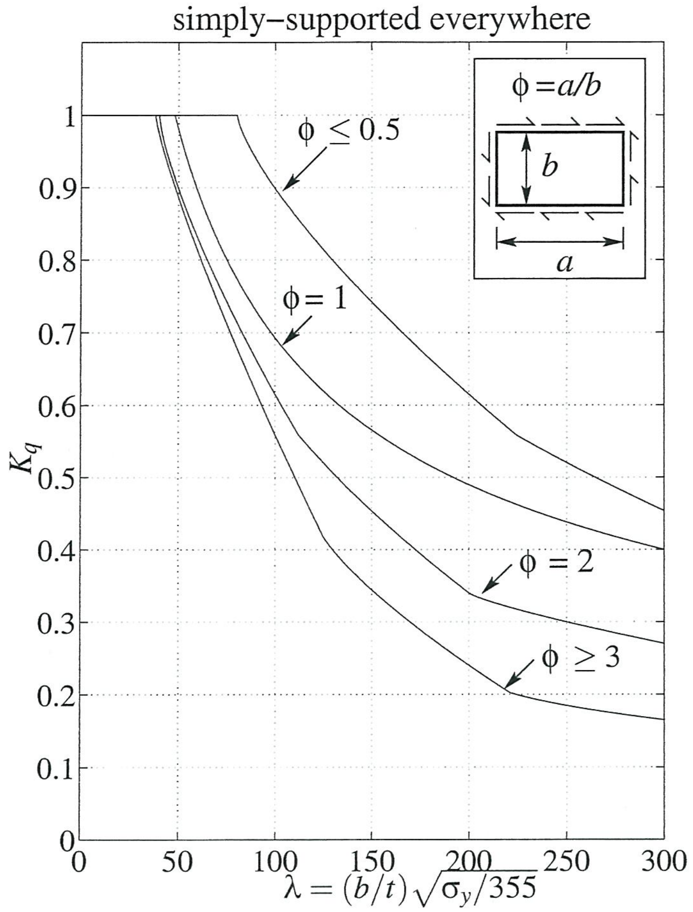
	Limits	Curve
Rolled I-sections	$h/b \leq 2$	a
	$h/b > 2$	b
Welded I-sections	$h/b \leq 2$	c
	$h/b > 2$	d
Other	-	d

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.

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

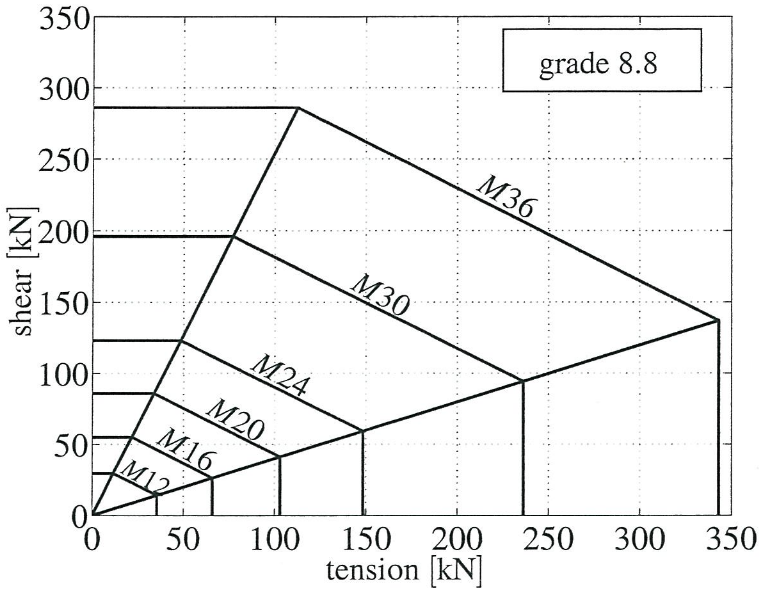
note 3: effective width, b_e, of compression flange with stiffener spacing, b, is $K_c b$.

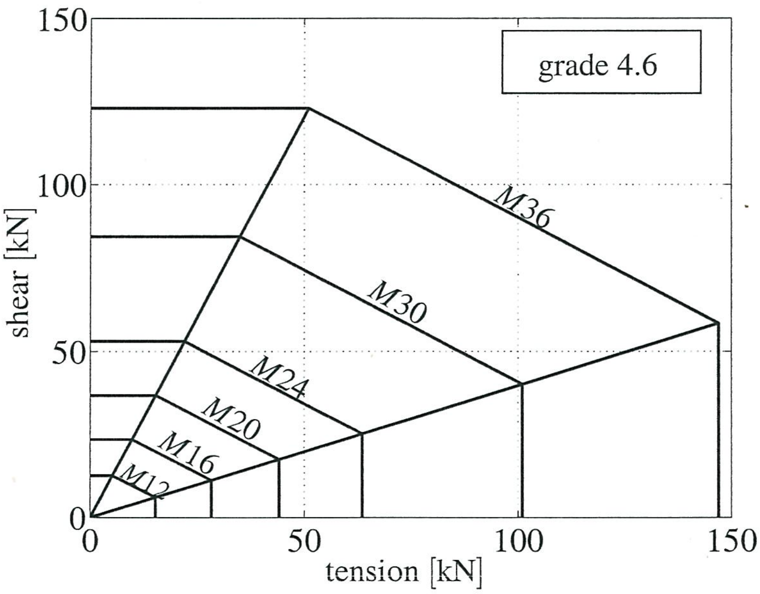
b. Plate compactness.

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.





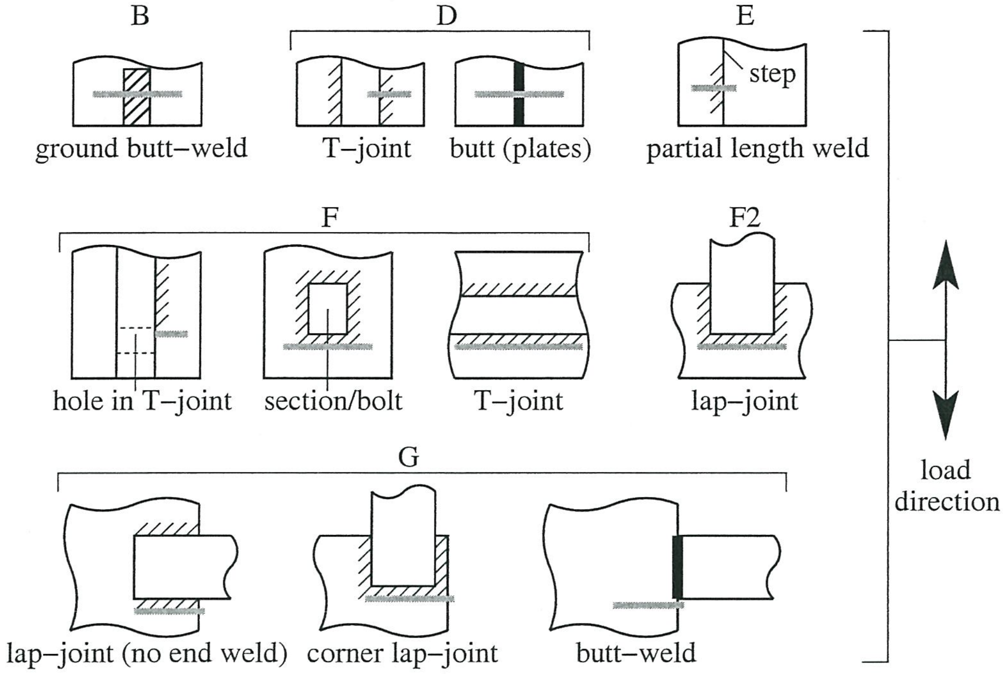
b. Bolt placement.

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

$$N\sigma_r^m = K_2 \quad (\sigma_r \text{ in MPa})$$

where the constants m and K_2 take different values for each class of weld from the following table.

detail class	m	K_2	σ_o [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: the number of repetitions of each stress range, σ_r, less than σ_o —the non-propagating stress—, should be reduced by a factor $(\sigma_r/\sigma_o)^2$.

note 3: 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.

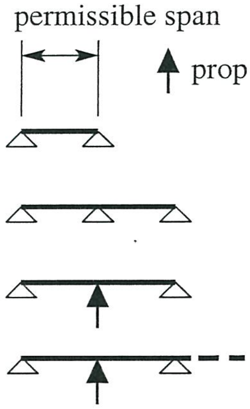
Young's modulus for grade 30 concrete, E_c, depends on duration of loading as:

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.