UNIVERSITY OF CAMBRIDGE

ENGINEERING TRIPOS PART IIB

Monday 26 April 2004 9 to 10.30

Module 4D10

STRUCTURAL STEELWORK

*Answer not more than **three** questions.*

All questions carry the same number of marks.

*The **approximate** number of marks allocated to each part of a question is indicated in the right margin.*

Unless otherwise indicated, in all questions the given loads are already factored and no partial material factors need to be applied, and self-weight can be ignored.

Attachment (i) 4D10 data sheets (9 pages)

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.

(TURN OVER

1 A composite floor consists of a concrete slab, of maximum thickness 100 mm, cast onto a profiled steel decking, as shown in Fig. 1: note that the diagram is not drawn to scale. The floor carries 5.5 kN/m^2 of imposed load and 3.0 kN/m^2 of permanent services, and the load factors for design are 1.6 and 1.4, respectively. The concrete has a design strength, $f_{cd} = 30$ N/mm^2, and a density of 2400 kg/m^3. The slab is connected to 406 $\times$ 178 $\times$ 60 grade S275 UB's placed orthogonally to the deck troughs and at 2.8 m centres. Overall, the floor is simply supported over a 10 m span between the ends of beams.

(a) Show that 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 a full composite action. [20%]

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

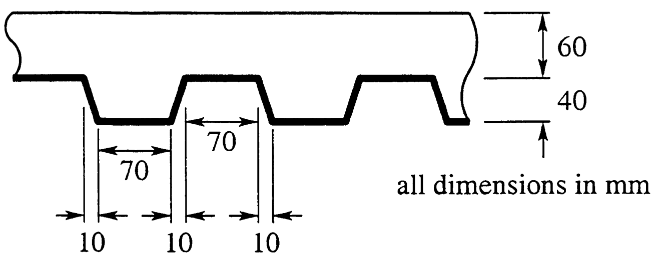


Fig. 1

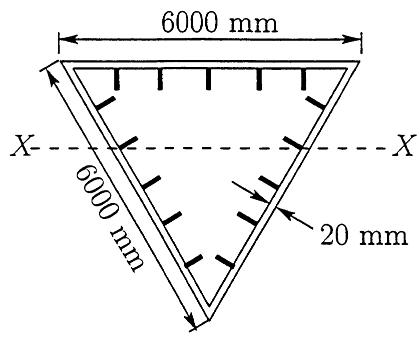
2 A novel beam has the plate-girder cross-section with fully effective 150 mm $\times$ 20 mm stiffeners shown in Fig. 2, which is not drawn to scale. All plates have the same thickness and are fabricated from grade S355 steel. Cross-frames are provided at 5 m centres along the beam. The cross-section carries a shear force, S, vertically downwards, an axially compressive force, P, and a bending moment, M, about $X - X$, the neutral axis, which induces compressive stresses in the top flange. The beam is initially free of stresses, and does not buckle globally. At two locations along the beam, 1 and 2, the stress resultants are given by

location	S [kN]	M [kN m]	P [kN]
1	10000	60000	0
2	0	20000	30000

For each location, check:

(a) the adequacy of the top flange as an effective column between cross-frames; [40%]

(b) the adequacy of the most heavily stressed web panel. [60%]



all stiffeners
150 mm $\times$ 20 mm
placed at 1 m centres

Fig. 2

3 The uniform beam in Fig. 3(a) is fully built-in at A and is simply-supported at B, a distance of $3L$ from A. The beam is initially free of stresses before a vertical force, F, is applied to the tip.

(a) Show that the magnitudes of the resulting bending moment and shear force at A are $FL/2$ and $F/2$, respectively. Hence, determine the bending moment profile along the beam in terms of F and L. [30%]

(b) The beam is a 203 × 102 × 23 grade S275 UB, and $L = 2$ m. At B and the tip, the beam is restrained against both lateral deflection and twist, but is free to warp. By consideration of stability, calculate the maximum value of F for which the beam will be *everywhere* safe. [40%]

(c) The beam at A is fully welded to a rigid end-plate, which has two sets of three bolt holes above and below the axis of bending, $X - X$, at the indicated distances in Fig. 3(b). It is proposed to use identical grade 4.6 bolts, in order to connect the end-plate to ground. Determine the smallest bolt size that safely transmits the stress resultants at A to ground under the previous value of F. [30%]

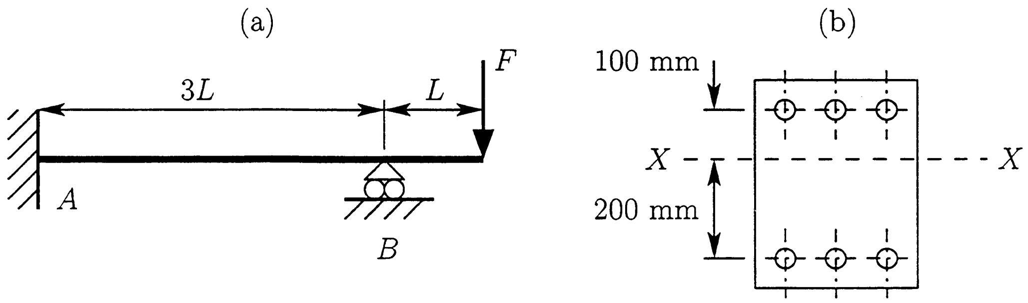


Fig. 3

4 A $300 \times 400 \times 8$ rectangular hollow section is fabricated by welding grade S355 steel to form a column of length 12 m. The support conditions are arranged so that the ends of column behave as (i) pinned for major axis bending and as (ii) built-in for minor axis bending.

(a) Determine the effective cross-section under an axial force alone. [30%]

(b) Show that the column can safely carry an non-eccentric axial force of 1000 kN. [60%]

(c) Determine the factor by which the load in part (b) may be increased before the beam fails. [10%]

END OF PAPER

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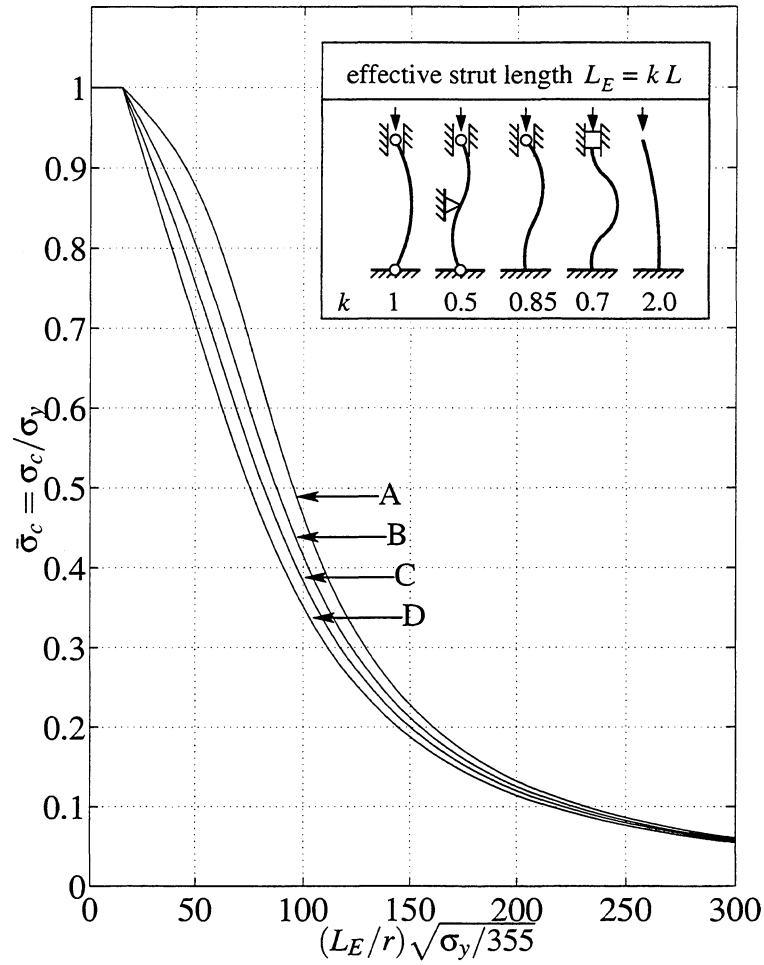
4D10 Structural Steelwork 2003/04

Data Sheets

DO NOT USE FOR ACTUAL DESIGN OF STRUCTURAL STEELWORK

KAS, December 1, 2003

DS1: Column Buckling Capacity σ_c

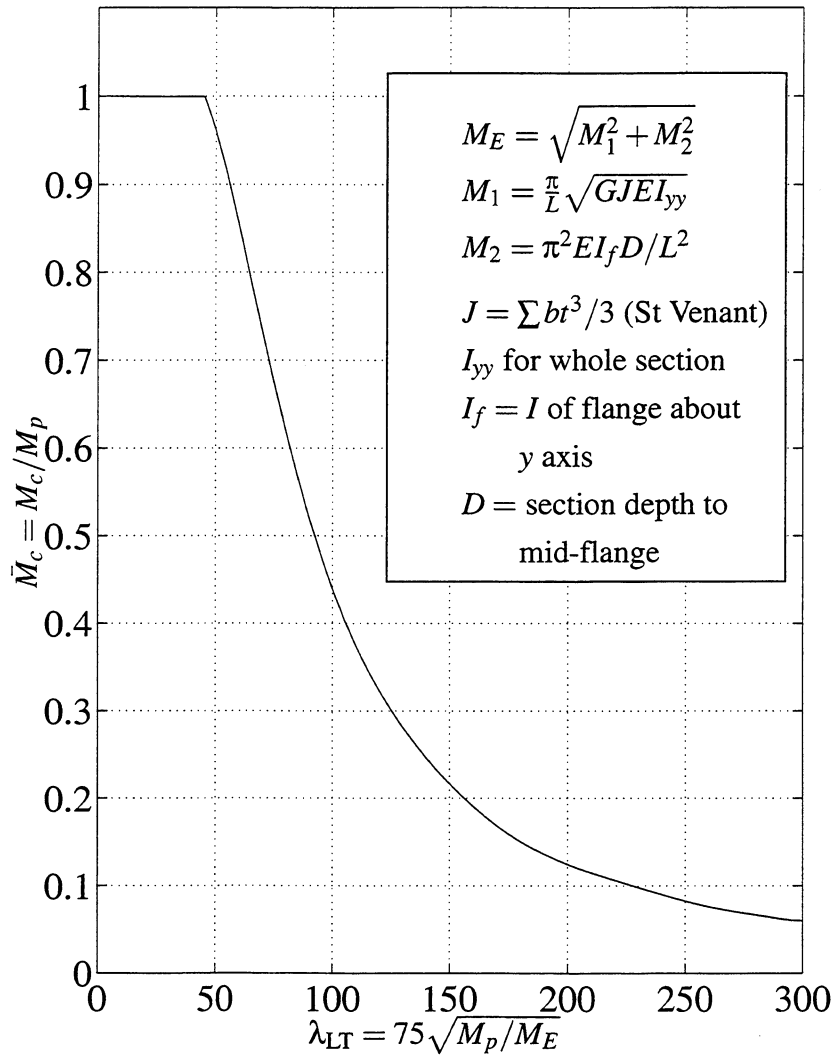


note 1: σ_y in N/mm^2; r is the radius of gyration about centroid of cross-section; curves are selected as follows (linear interpolation used for intermediate r/y values.)

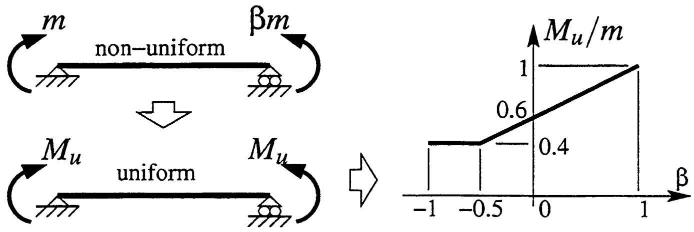
	members fabricated by welding	all other members including stress-relieved welded members
$r/y \geq 0.7$	curve B	curve A
$r/y = 0.6$	curve C	curve B
$r/y = 0.5$	curve C	curve B
$r/y \leq 0.45$	curve C	curve C
all rolled sections with flange thickness > 40 *mm*	curve D	curve D
hot-finished hollow sections	curve A	curve A

note 2: y is extreme fibre distance from centroid for the same axis as r.
note 3: intermediate bracing stiffness $> 16P_E/L$ for buckling force $P_c = 4P_E$ (pin-ends only).

DS2: Lateral Torsional Buckling Uniform Moment Capacity M_c



note 1: for non-uniform end moments in the ratio of β

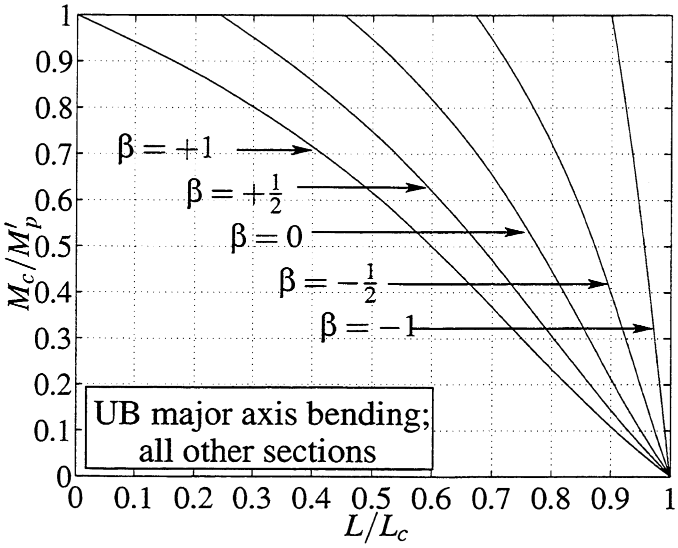


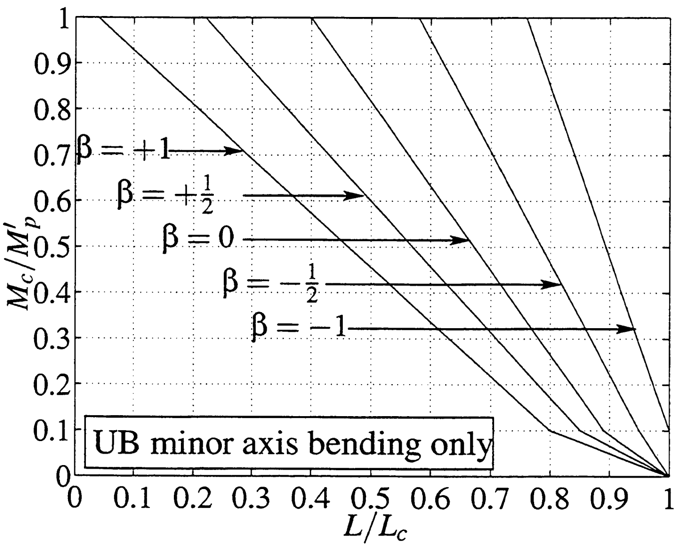
$$
\begin{aligned}
M_u &= (0.6 + 0.4\beta)m, \quad -0.5 \le \beta \le 1; \\
M_u &= 0.4m, \quad -1 \le \beta \le -0.5
\end{aligned}
$$

note 2: for stability, $M_u < M_c$.
note 3: for strength, $m < M_p$
note 4: if the shear force, V, is larger than $V_c/2$, where $V_c = A_{\mathrm{web}}\tau_y$, M_p in $\bar{M}_c$ and λ_{LT} is replaced by M_y, equal to $Z_e\sigma_y$.

DS3: Beam Columns; Limiting M_c Under Axial Load, P

a. Column Deflection Curves.





note 1: M'_p is the reduced plastic moment; β as in DS2; L_c is the length of a pin-ended column buckling under P alone (found with DS1); only use CDC method if $\lambda_{\text{web}} \leq 56$.

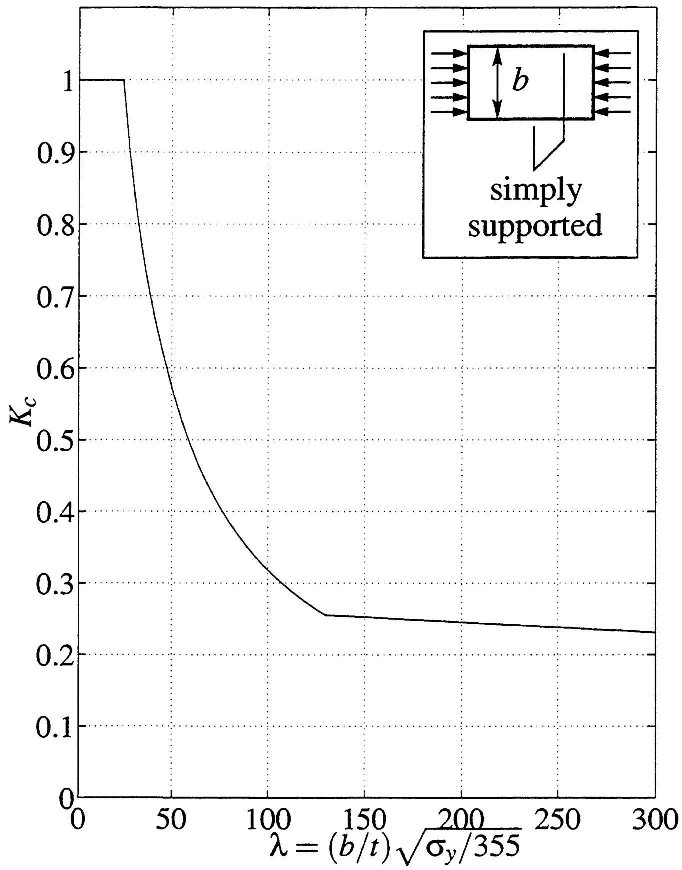
b. Interaction Equations.

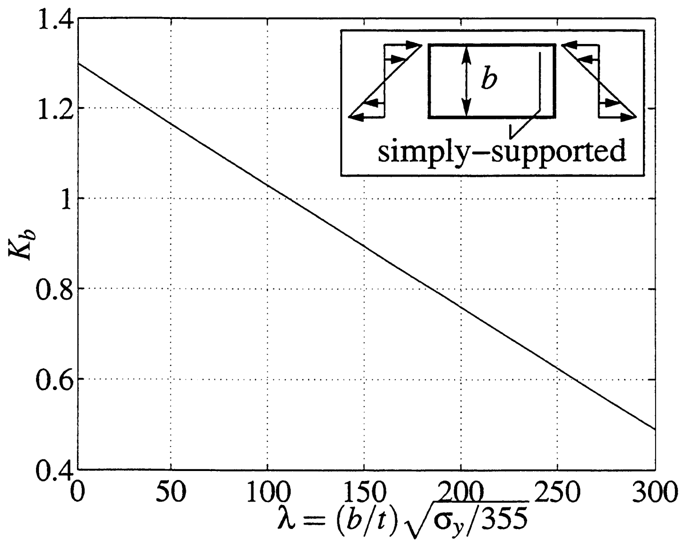
for strength: $\dfrac{P}{P_p} + \dfrac{M_{\max}}{M_p} \leq 1$ (or find M'_p directly)

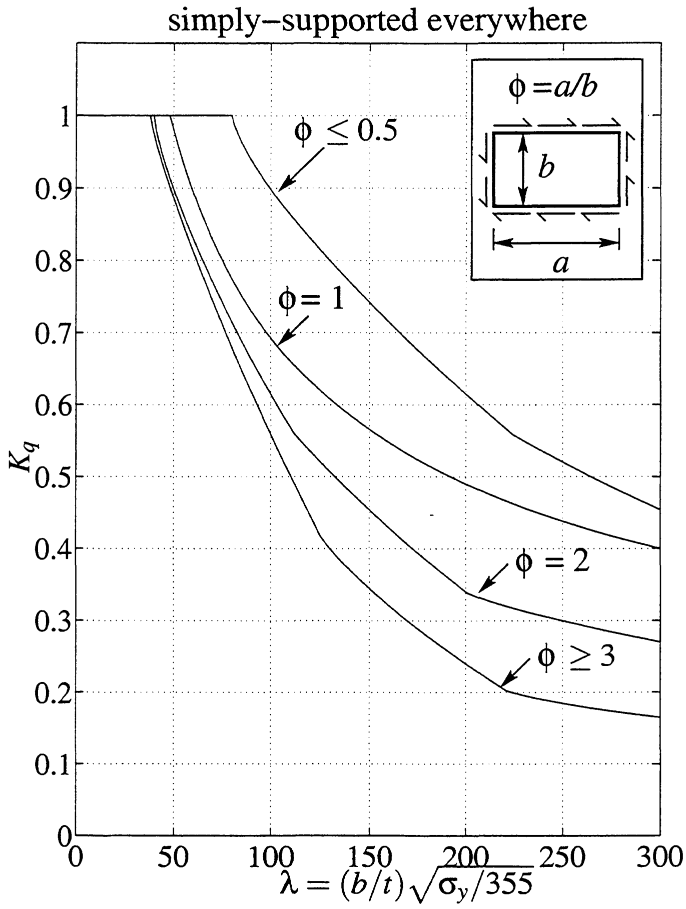
for stability: $\dfrac{P}{P_c} + \dfrac{M_u}{M_c} \leq 1$ (P_c from DS1, M_u and M_c via DS2: all **notes** apply)

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.

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

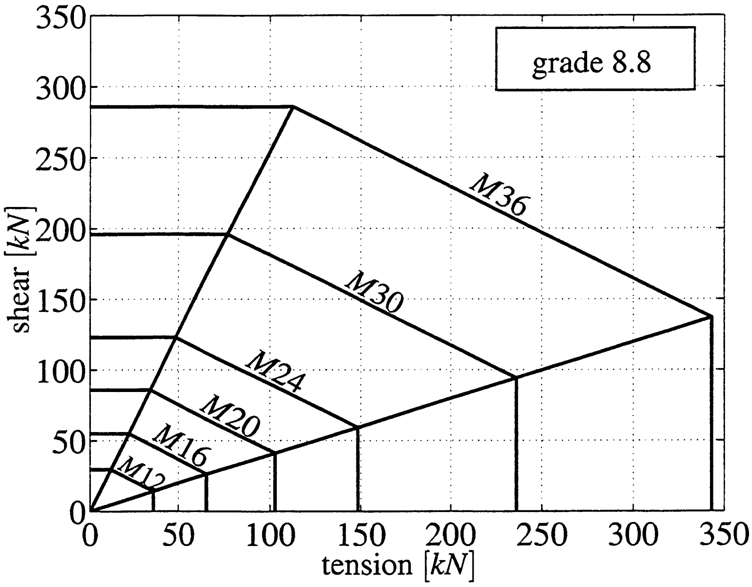
note 3: effective width, b_e, of compression flange with stiffener spacing, b, is $K_c b$.

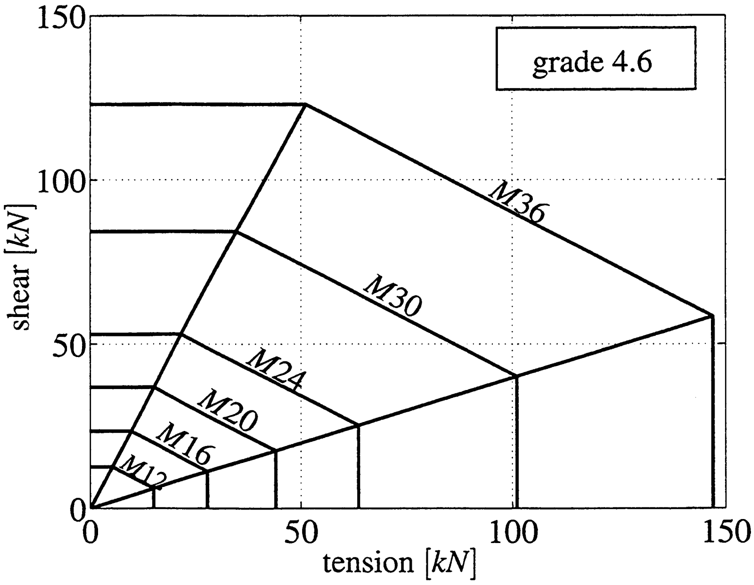
b. Plate compactness.

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.





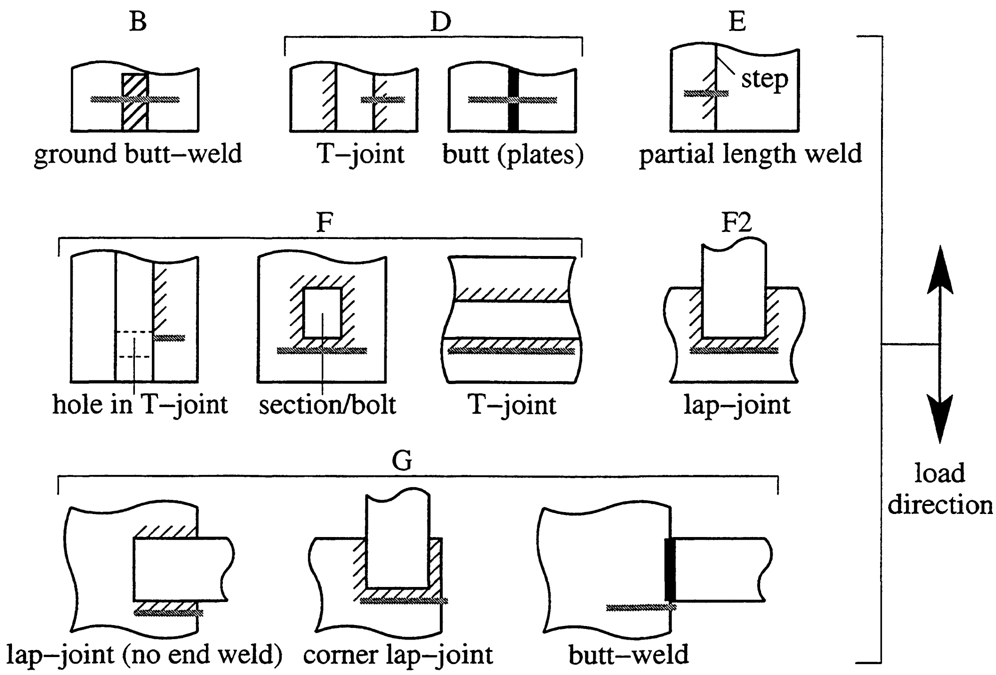
b. Bolt placement.

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

$$N\sigma_r^m = K_2 \quad (\sigma_r \text{ in } N/mm^2)$$

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 $[N/mm^2]$
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} $[N/mm^2]$			
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.

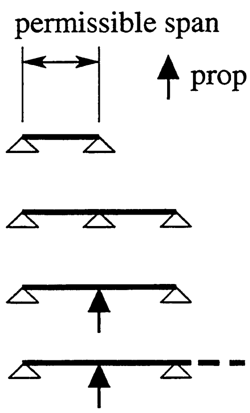
Young's modulus for grade 30 concrete, E_c, depends on duration of loading as:

short term: $E_c = 28\ kN/mm^2$
long term: $E_c = 14\ kN/mm^2$

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.