ENGINEERING TRIPOS    PART IIB

---

Monday 1 May 2006    9.00 to 10.30

---

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.*

*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:* 4D10 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 $610 \times 229 \times 140$ Universal Beam, made from S355 grade steel, is 17 m long, and is simply supported at its ends. It carries a load $W$, 6 m from the left-hand end, and a load $W/3$, 6 m from the right-hand end. At the ends, and also at the loading points, the beam is free to warp, but is restrained against lateral deflection and rotation about its axis.

(a) Ignoring self-weight, determine the distribution of bending moment along the beam. [20%]

(b) Estimate the failure-load of the beam, by either lateral-torsional buckling or classical plastic collapse. Which is the more stringent? [70%]

(c) Would the effect of self-weight be significant for this beam? [10%]

2 (a) Describe briefly, without calculation, the factors which reduce the load-carrying capacity of a pin-ended column below that predicted by Euler's formula. [20%]

(b) A 350 mm × 450 mm × 10 mm rectangular hollow section is fabricated by welding S355 steel plates, to form a column of length 15 m. The end supports are such that for bending about the major axis, one end is built-in and the other is pinned; while for minor-axis bending both ends are built-in.

(i) Determine the effective cross-section of the column under axial compression alone. [30%]

(ii) Find the carrying-capacity of the column under a non-eccentric axial force. [50%]

3 The box-girder beam whose cross-section is shown in Fig. 1 is fabricated by welding from steel plates of grade S275. The main plates are 12 mm thick, and each of the longitudinal stiffeners has cross-section 100 × 10 mm. The drawing is not to scale, and the dimensions shown are in mm. Diaphragms are provided at 1.5 m centres along the beam.

At a particular cross-section the beam sustains a major-axis bending moment of 860 kNm, together with an axial compression of 950 kN.

(a) Determine, approximately, the relevant overall cross-sectional parameters for axial compression and major-axis bending of the beam, by 'smearing' the stiffeners uniformly over the width of the plates to which they are attached. [40%]

(b) Investigate the adequacy of the compressive flange as an effective column between diaphragms. [40%]

(c) Investigate the adequacy of the most heavily stressed web panel. [20%]

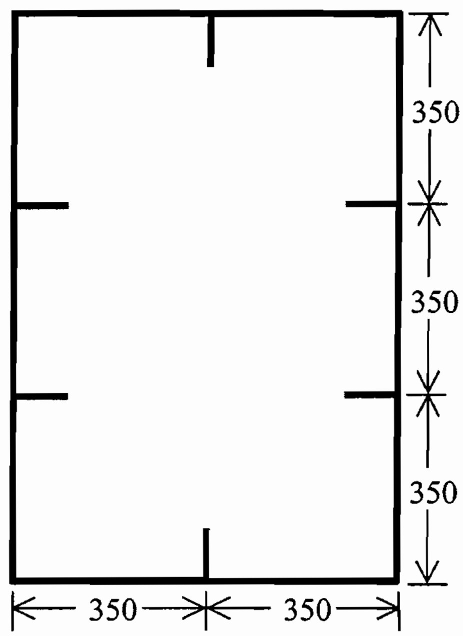



Fig. 1

(TURN OVER

4 A composite floor is to be designed to carry a uniformly-distributed imposed load of 7.0 $kNm^{-2}$ in addition to self-weight, and 0.7 $kNm^{-2}$ of permanent services. The partial safety factors for imposed and dead loads are 1.6 and 1.4, respectively.

The floor consists of a concrete slab of uniform thickness 120 mm, acting compositely with 457 × 191 × 89 Universal Beams in steel of grade S355, each of span 13 m and placed at 3.6 m spacing, and simply supported at their ends. The concrete has design strength $f_{cd}$ = 30 MPa, and its unit weight is 24 $kNm^{-3}$.

(a) Show that the floor can carry the specified loading, and by what margin. [40%]

(b) How many 65 mm × 13 mm shear studs are needed for each beam, in order to achieve full composite action? [25%]

(c) Estimate the central deflection of the beams caused by short-term action of the imposed load. [35%]

**END OF PAPER**

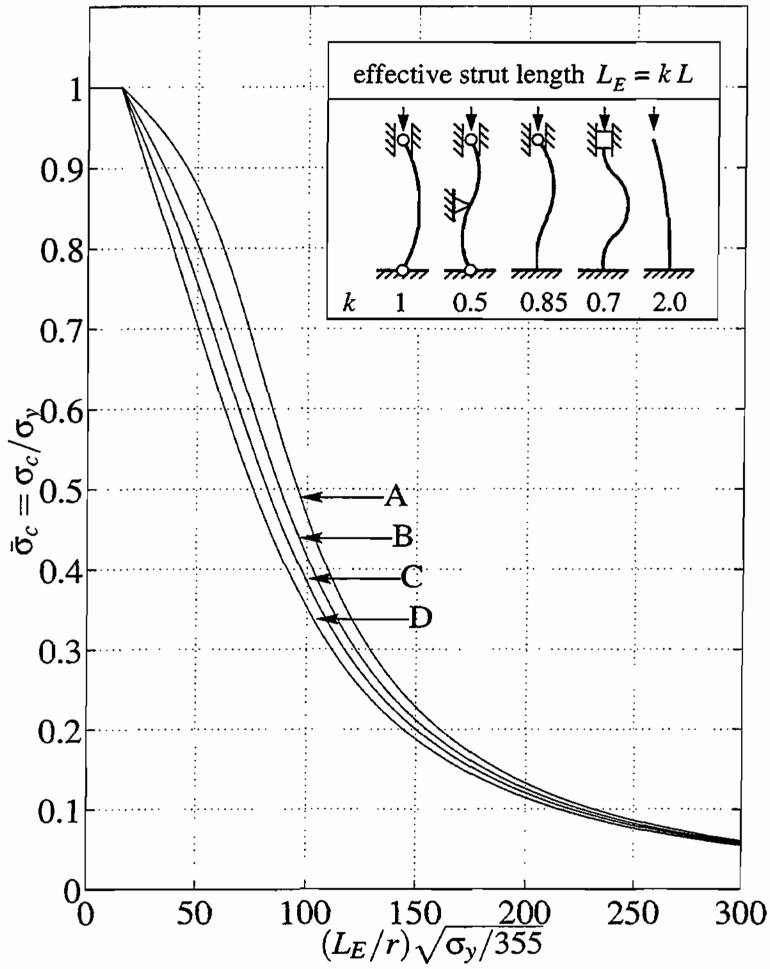
4D10 Structural Steelwork 2005/06

# Data Sheets

DO NOT USE FOR ACTUAL DESIGN OF STRUCTURAL STEELWORK

CRC, August 1, 2005

# DS1: Column Buckling Capacity $\sigma_c$

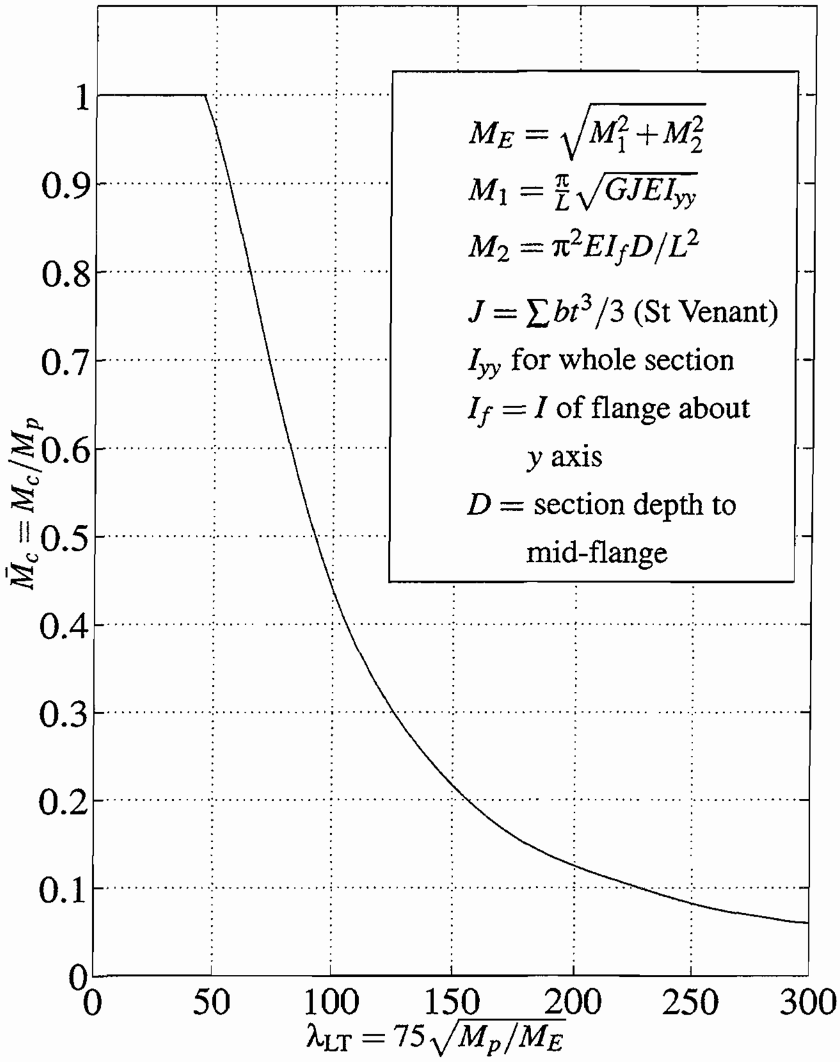



***note 1:*** $\sigma_y$ in $N/mm^2$; $r$ is the radius of gyration about centroid of cross-section; curves are selected as follows (linear interpolation used for intermediate $r/y$ values.)

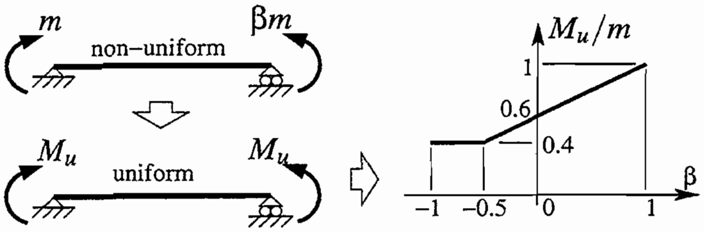
| | members fabricated by welding | all other members including stress-relieved welded members |
|---|---|---|
| $r/y \geq 0.7$ | curve B | curve A |
| $r/y = 0.6$ | curve C | curve B |
| $r/y = 0.5$ | curve C | curve B |
| $r/y \leq 0.45$ | curve C | curve C |
| all rolled sections with flange thickness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

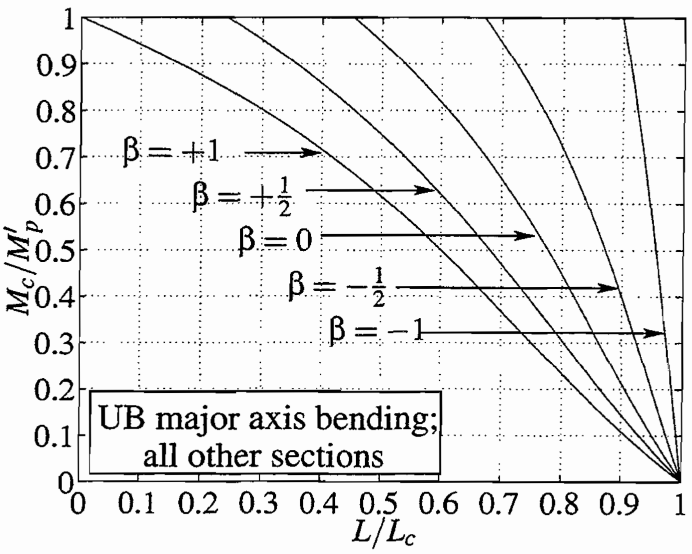


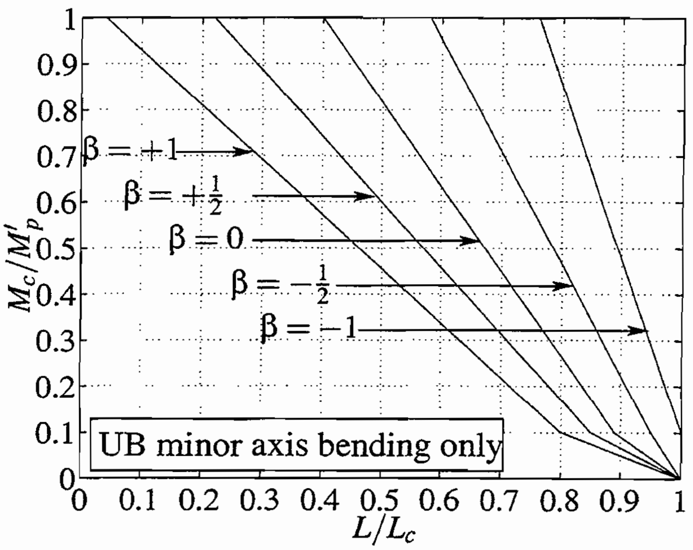

$$M_u = (0.6 + 0.4\beta)m, \quad -0.5 \le \beta \le 1;$$
$$M_u = 0.4m, \quad -1 \le \beta \le -0.5$$

***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 $V_c = A_{\text{web}}\tau_y$, $M_p$ in $\bar{M}_c$ and $\lambda_{LT}$ 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; $\beta$ 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 $\lambda_{\text{web}} \leq 56$.

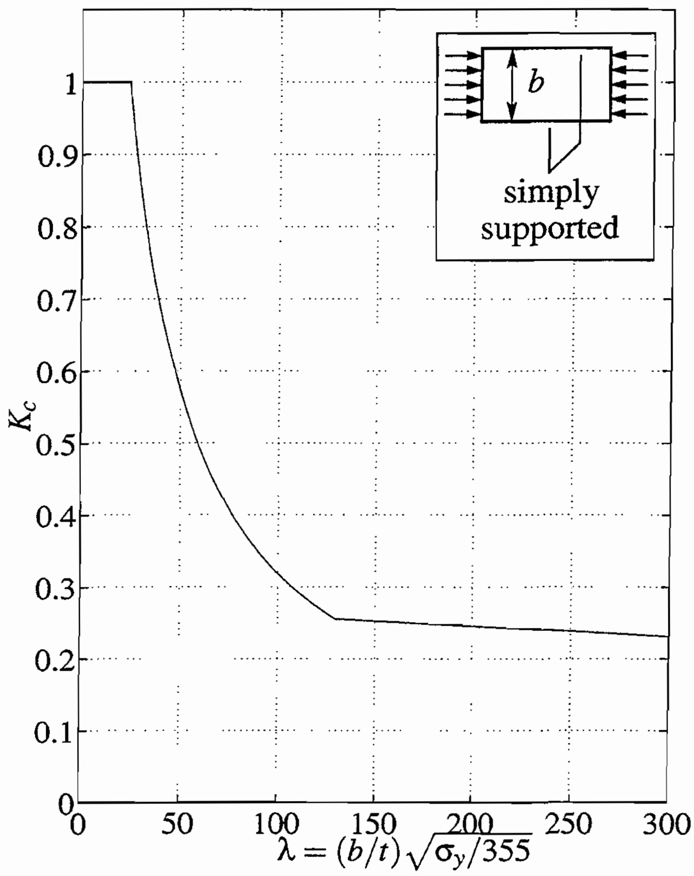
b. Interaction Equations.

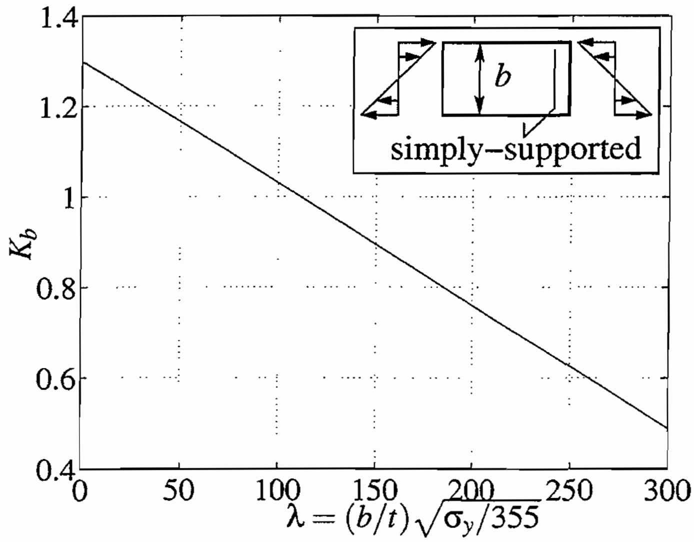
for strength: $$\frac{P}{P_p} + \frac{M_{\max}}{M_p} \leq 1 \quad \text{(or find } M'_p \text{ directly)}$$

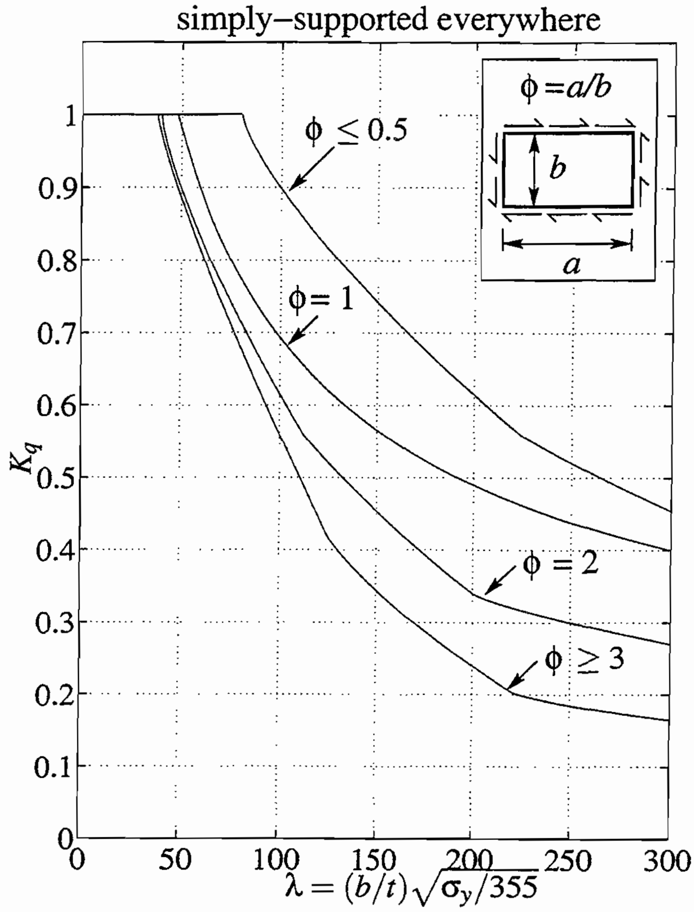
for stability: $$\frac{P}{P_c} + \frac{M_u}{M_c} \leq 1 \quad (P_c \text{ from DS1, } M_u \text{ and } M_c \text{ via DS2: all } \textbf{notes} \text{ apply})$$

# 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:*** $\tau$ is the shear stress on the panel, $\sigma_c$ is the average compressive stress and $\sigma_b$ is the maximum bending stress.

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

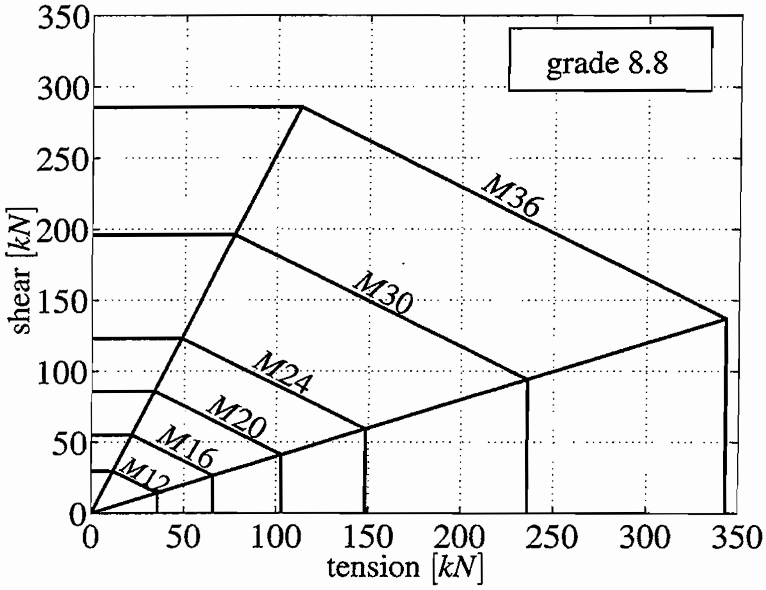
***note 3:*** effective width, $b_e$, of compression flange with stiffener spacing, $b$, is $K_cb$.

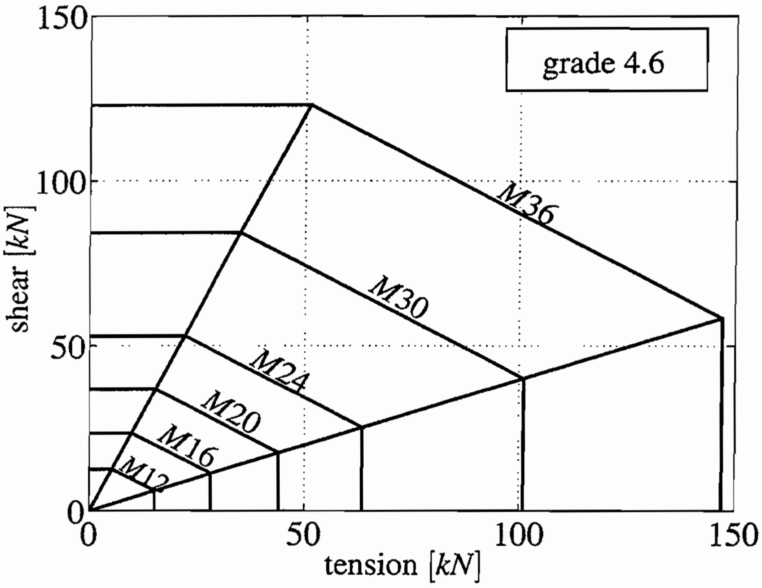
b. Plate compactness.

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

# DS5 Connector Capacity and Fatigue Life

a. Bolt strength in combined tension and shear.



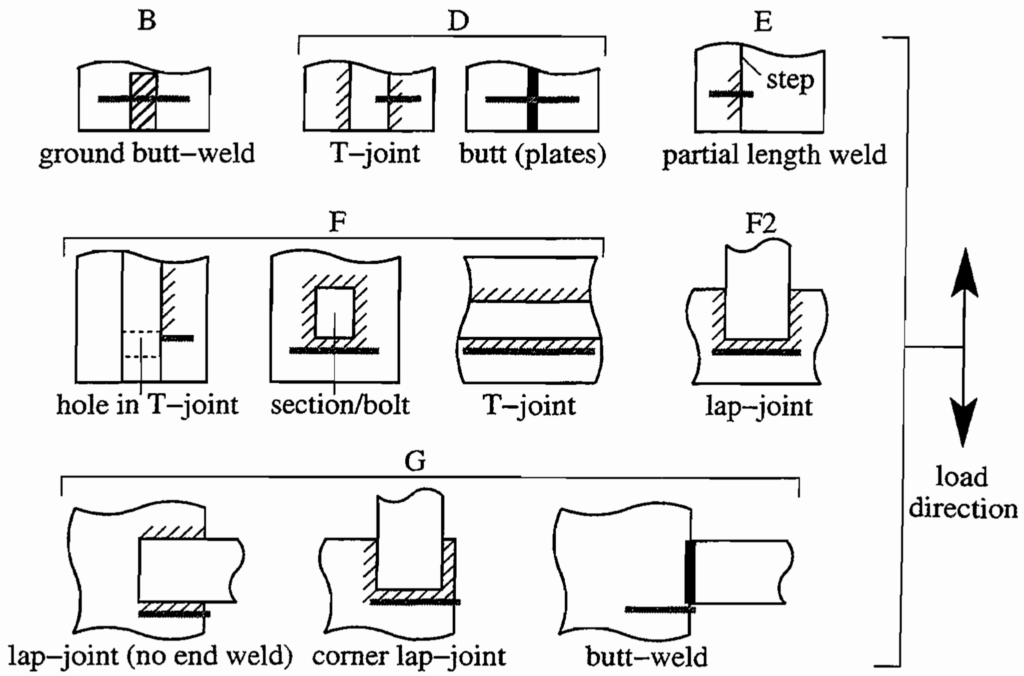

b. Bolt placement.

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

***note 1:*** $\phi$ 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, $\sigma_r$, 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 | $m$ | $K_2$ | $\sigma_o$ $[N/mm^2]$ |
|---|---|---|---|
| G | 3 | $0.25 \times 10^{12}$ | 29 |
| F2 | 3 | $0.43 \times 10^{12}$ | 35 |
| F | 3 | $0.63 \times 10^{12}$ | 40 |
| E | 3 | $1.04 \times 10^{12}$ | 47 |
| D | 3 | $1.52 \times 10^{12}$ | 53 |
| B | 4 | $1.01 \times 10^{15}$ | 100 |

***note 2:*** the number of repetitions of each stress range, $\sigma_r$, less than $\sigma_o$ —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 $\sigma_{ri}$; $N_i$ is the *total* number of possible cycles under $\sigma_{ri}$. Each $\sigma_{ri}$ 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.

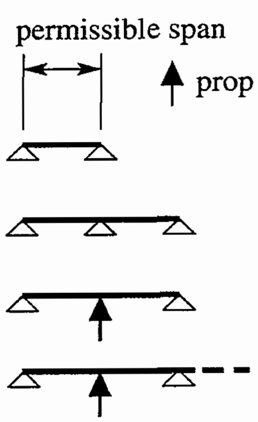
Young's modulus for grade 30 concrete, $E_c$, depends on duration of loading as:

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.




| | | $t = 0.9\ mm$ | | | $t = 1.2\ mm$ | | |
|---|---|---|---|---|---|---|---|
| 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 $t$.