ENGINEERING TRIPOS PART IIB

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Monday 28 April 2008 9 to 10.30

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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
Single-sided script paper

SPECIAL REQUIREMENTS
Engineering Data Book
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.**

Version 4: post read-through and final version

1 Figure 1 shows a continuous horizontal beam resting on simple supports, which is initially unstressed. A concentrated load $F$ is applied vertically to the mid-point of the left span. At all points of loading and reaction, the beam is prevented from twisting and from deflecting laterally, but is free to warp; appropriate bearing stiffeners are also provided. Self-weight is to be ignored.

(a) Show that the magnitude of the resulting bending moment over the middle support is $3FL/32$ . Hence, determine the bending moment profile along the beam, marking salient values in terms of $F$ and $L$ . [25%]

(b) The beam is a 457 × 152 × 82 grade S355 UB and $L = 8$ m . By consideration of lateral stability of the right span only, calculate the maximum value of $F$ for which the span will be safe. [40%]

(c) During construction, an error results in the right-most support not being secured to the beam. Determine the factor by which $F$ can be ultimately increased or decreased. [35%]

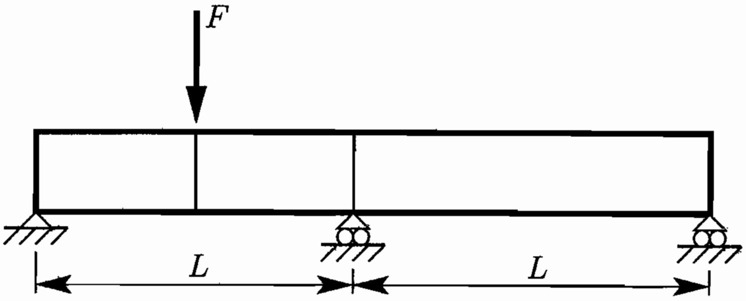


Fig. 1: not drawn to scale.

2 The vertical column AB in Fig. 2 is a welded 250 × 150 × 6 grade S275 rectangular hollow section, which is rigidly connected to the horizontal beam BCD at B. This beam has two equal length spans of 457 × 191 × 82 grade S275 UB, which are pinned together at C . Lateral deflections and twisting are prevented at A , B , C and D but which are otherwise free to warp. Two concentrated loads F are applied vertically at B and C as shown.

(a) Sketch the bending moment and shear force diagrams, marking salient values in terms of $F$ and $L$ . [20%]

(b) For $F =$ 20 kN and $L = 4$ m , determine:

(i) whether the horizontal beam is adequate; [30%]

(ii) by using the *Column Deflections Curves* approach, whether the column is adequate if it is restrained against minor axis flexural buckling. [30%]

(c) Comment on other design considerations that apply to the beam. [20%]

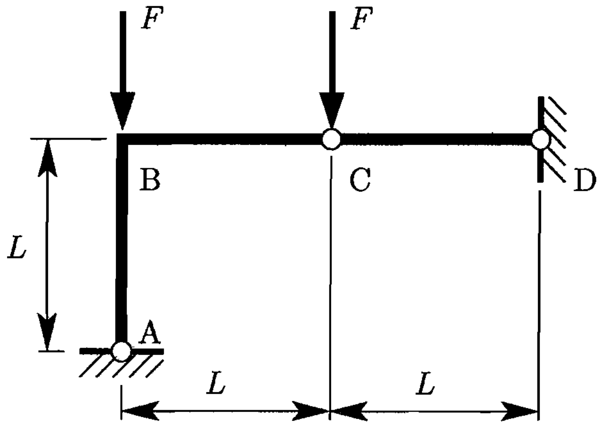


Fig. 2

3 A $300 \times 200 \times 6$ rectangular hollow section is fabricated by welding grade S355 steel to form a 20 m long column. Minor axis buckling is restrained and the support conditions for major axis bending may be regarded as being pinned-pinned.

(a) Determine the effective cross-section under an axial force alone. [20%]

(b) Calculate the maximum axial force that can be carried safely by the column. [40%]

(c) Determine the effective cross-section for bending and hence, its limiting moment capacity. Compare your result to the fully plastic moment of the complete section. [40%]

4 (a) The equivalent slenderness for a plate section is given in lectures as $(b/t)\sqrt{\sigma_Y/355}$. Identify the meaning of the symbols and explain the origin of this formula without a derivation. [30%]

(b) A novel plate-girder section whose cross-section is shown in Fig. 3 is fabricated by welding steel plates of grade S355. The longitudinal stiffeners each have cross-section 100 × 10 mm . The drawing is not to scale and dimensions are shown in mm.

(i) Determine the overall cross-sectional parameters by *smearing* the stiffeners over the width of the web to which they are connected. [20%]

(ii) Check the capacity of the web panels at a cross-section well away from any vertical stiffeners if, at the same section, the plate-girder sustains a major axis bending moment of 3000 kNm and a vertical shear force of 1000 kN . [50%]

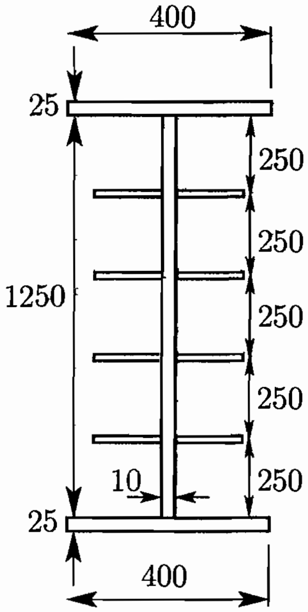


Fig. 3

**END OF PAPER**

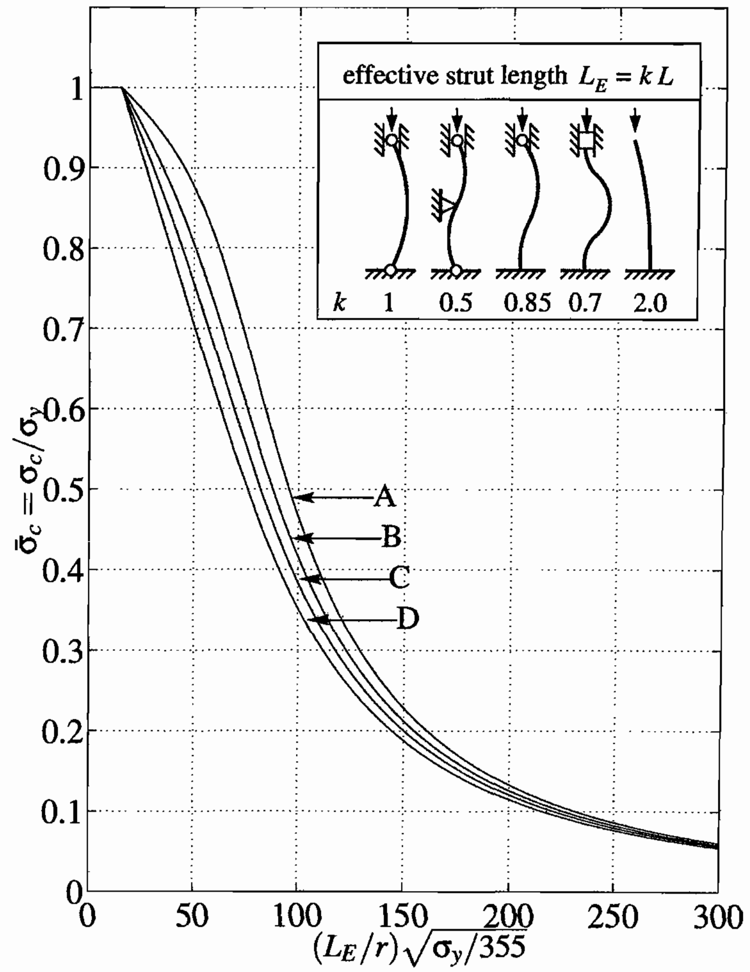
4D10 Structural Steelwork 2007/08

# Data Sheets

DO NOT USE FOR ACTUAL DESIGN OF STRUCTURAL STEELWORK

KAS, November 12, 2007

# DS1: Column Buckling Capacity $\sigma_c$



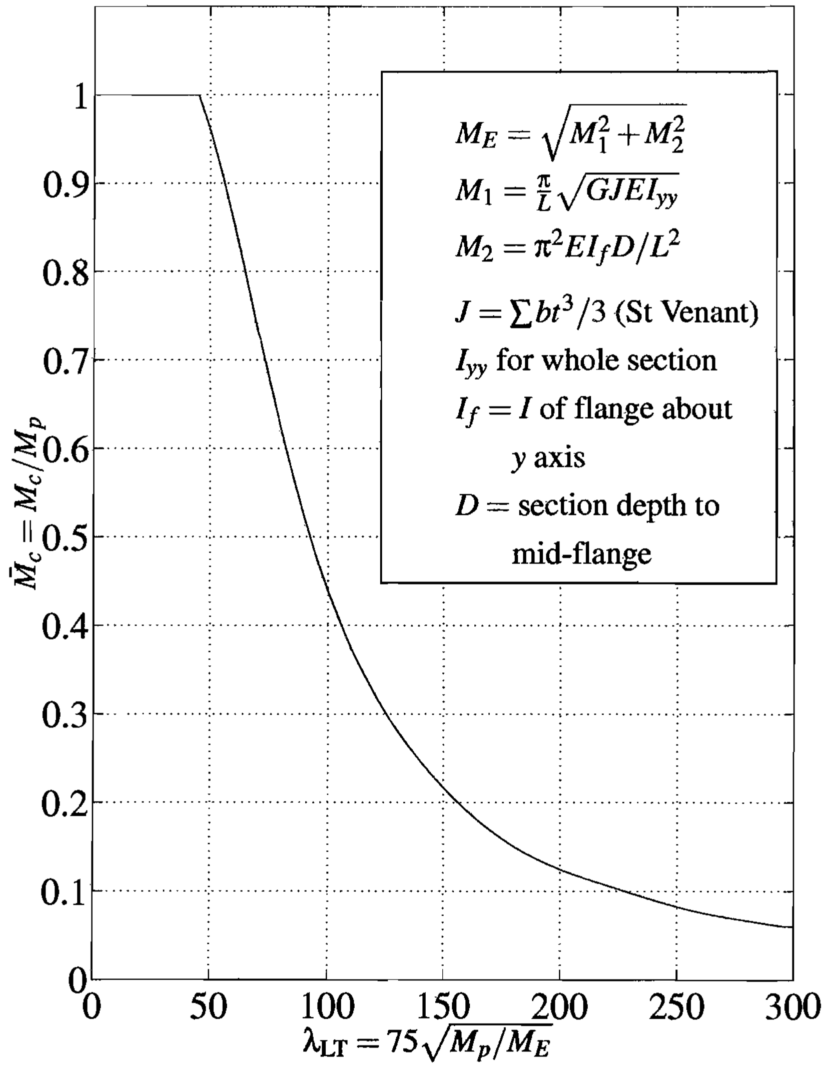
***note 1:*** $\sigma_y$ in $\mathrm{N/mm^2}$; $r$ is the radius of gyration about centroid of cross-section; curves are selected as follows (linear interpolation used for intermediate $r/y$ values.)

| | 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 |

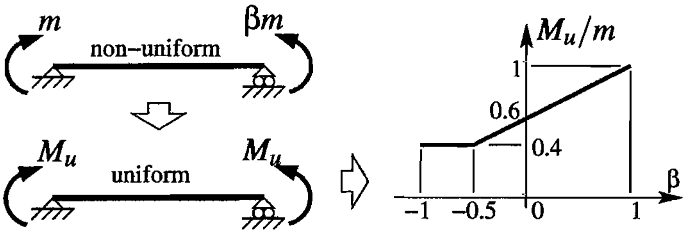
***note 2:*** $y$ is extreme fibre distance from centroid for the same axis as $r$.

***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 $\beta$



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

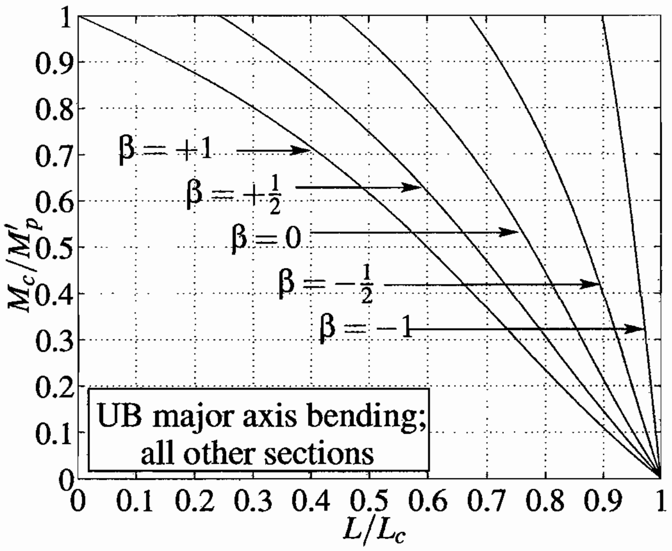
***note 2:*** for stability, $M_u < M_c$.

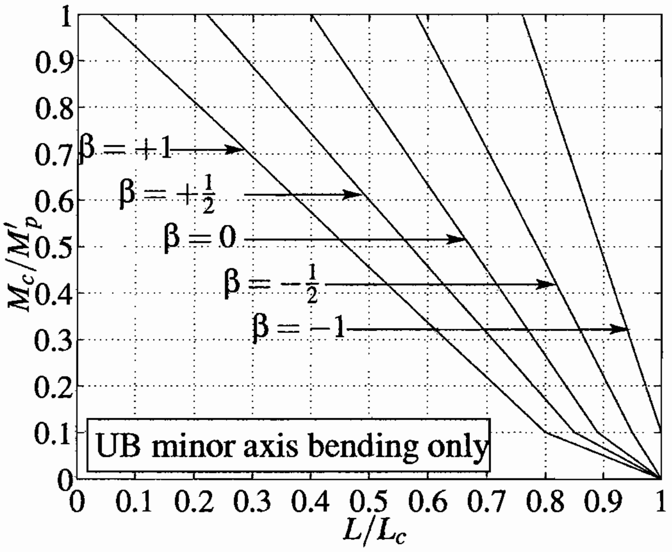
***note 3:*** for strength, $m < M_p$

***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 $\lambda_{\text{web}} \leq 56$.

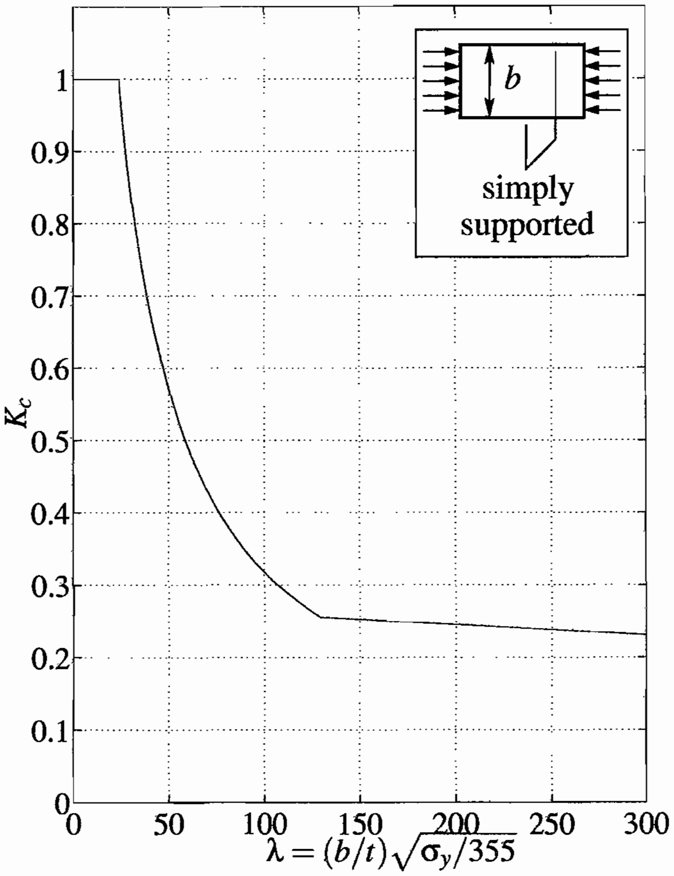
b. Interaction Equations.

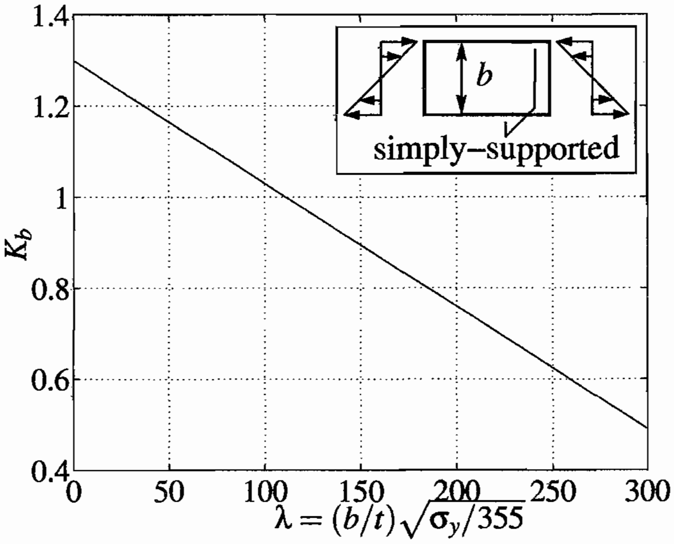
for strength: $$\frac{P}{P_p} + \frac{M_{\max}}{M_p} \leq 1 \quad \text{(or find } M_p' \text{ directly)}$$

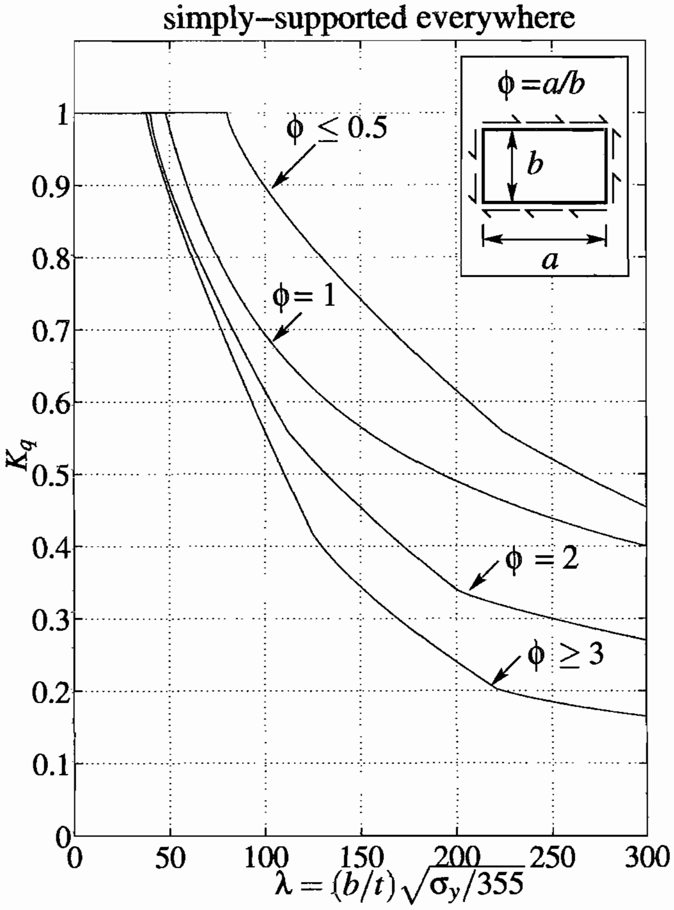
for stability: $$\frac{P}{P_c} + \frac{M_u}{M_c} \leq 1 \quad (P_c \text{ from DS1, } M_u \text{ and } M_c \text{ via DS2: all } \textbf{notes} \text{ apply})$$

# DS4: Panel Strength and Plate Compactness

a. Panel strength: use the following three figures in the expressions overleaf.







$$\text{panel stability:}\quad \frac{\sigma_c}{\sigma_{cc}} + \left(\frac{\sigma_b}{\sigma_{bc}}\right)^2 + \left(\frac{\tau}{\tau_c}\right)^2 \le 1$$

***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.

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

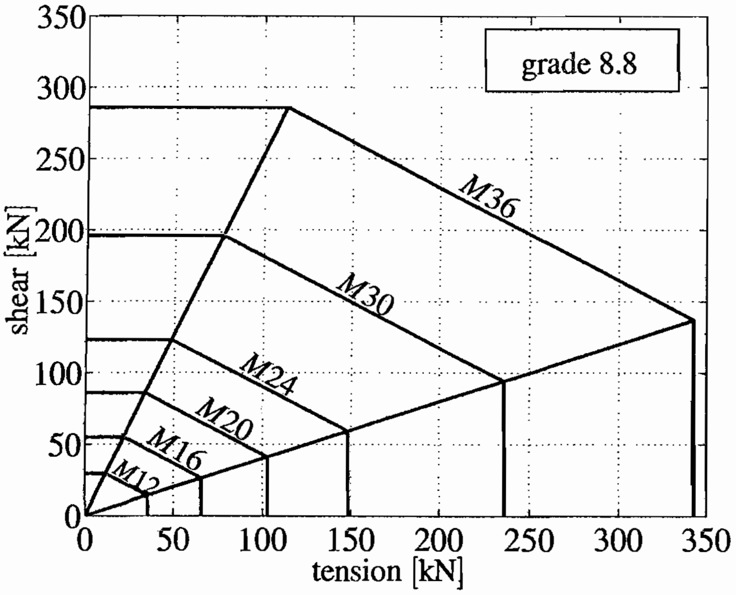
***note 3:*** effective width, $b_e$, of compression flange with stiffener spacing, $b$, is $K_c b$.

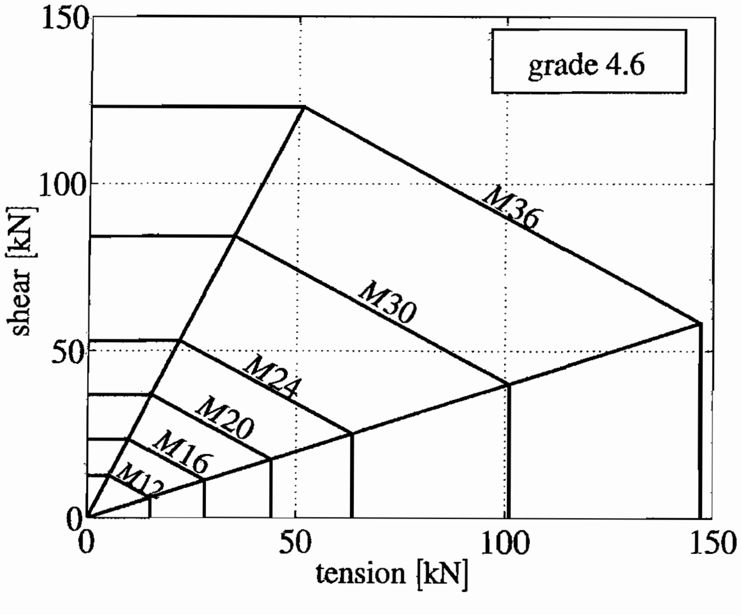
b. Plate compactness.

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

# DS5: Connector Capacity and Fatigue Life

a. Bolt strength in combined tension and shear.





b. Bolt placement.

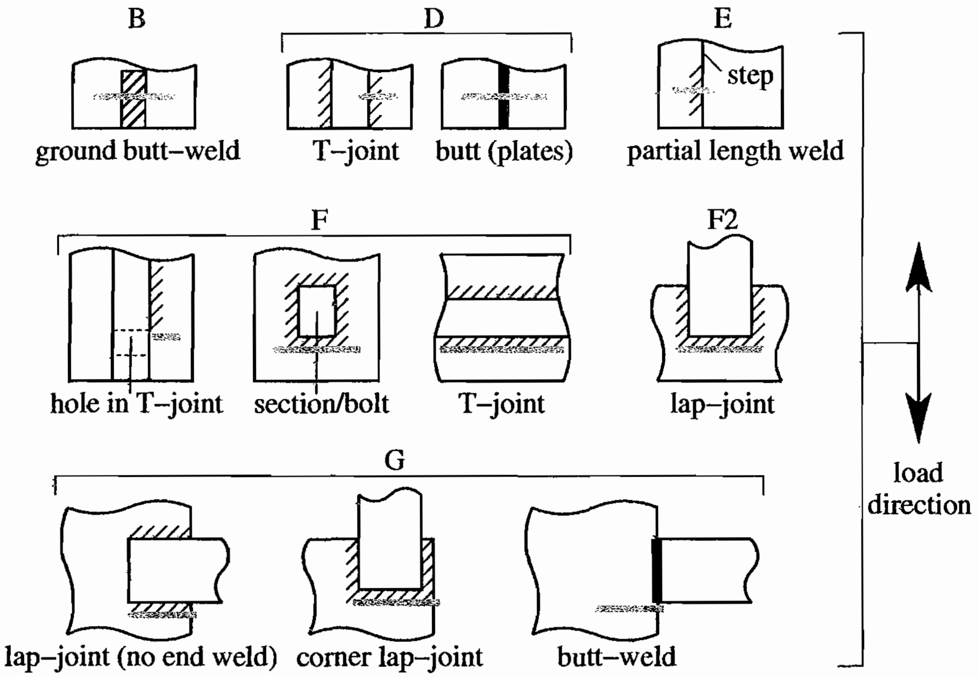
edge and end distances: $\geq 2.5\phi$

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.

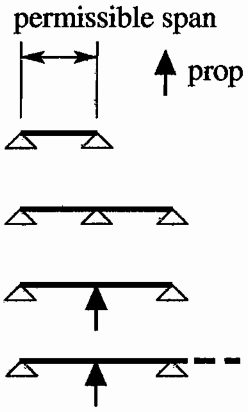
Young's modulus for grade 30 concrete, $E_c$, depends on duration of loading as:

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

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 $t$.