EGT3
ENGINEERING TRIPOS PART IIB

Tuesday 30 April 2019 9.30 to 11.10

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 at the start of the exam.

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

1 (a) Figure 1 shows a subframe ABCD consisting of beams AB and CD and the column BC. These three members are rigidly connected at B and C. The beams have UB $457 \times 191 \times 98$ section of S235 steel and the column has UC $305 \times 305 \times 283$ section of S460 steel. All webs are in the plane of the diagram, and the dimensions and support conditions are as shown. All members are braced against deformations out of the plane of the diagram. The frame is braced by thin, axially-stiff, pin-ended diagonal braces AC and BD.

The hyperbolic graphs in Fig. 2 have axis coordinates k_1 and k_2 which are appropriate flexural rigidity ratios of members connecting at the column top and base. Using these graphs, determine the effective length of the column BC and hence find the value of P applied at B that will cause elastic flexural buckling of the column about its major axis. (Self-weight may be ignored). [40%]

(b) The 12 m continuous beam EFGH shown in Fig. 3 has a UB $356 \times 127 \times 39$ section in grade S275 steel. Its web is in the plane of the paper. At ends E and H, lateral deflections and twist rotations are prevented, but the sections are free to warp. Concentrated vertical loads W and $2W$ are to be applied via side beams at F and G respectively. These side beams prevent lateral deflection and twist rotation where they connect.

Ignoring self-weight, determine the value of W at the Ultimate Limit State. [60%]

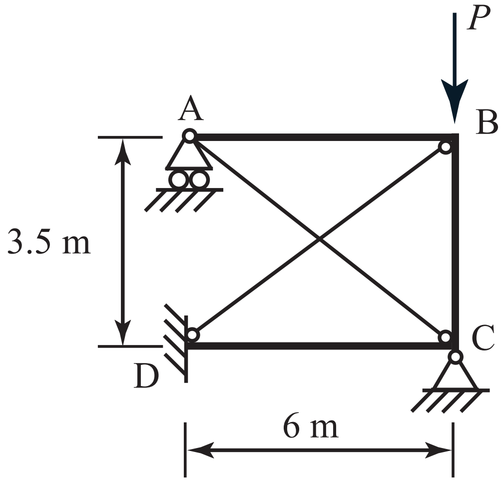
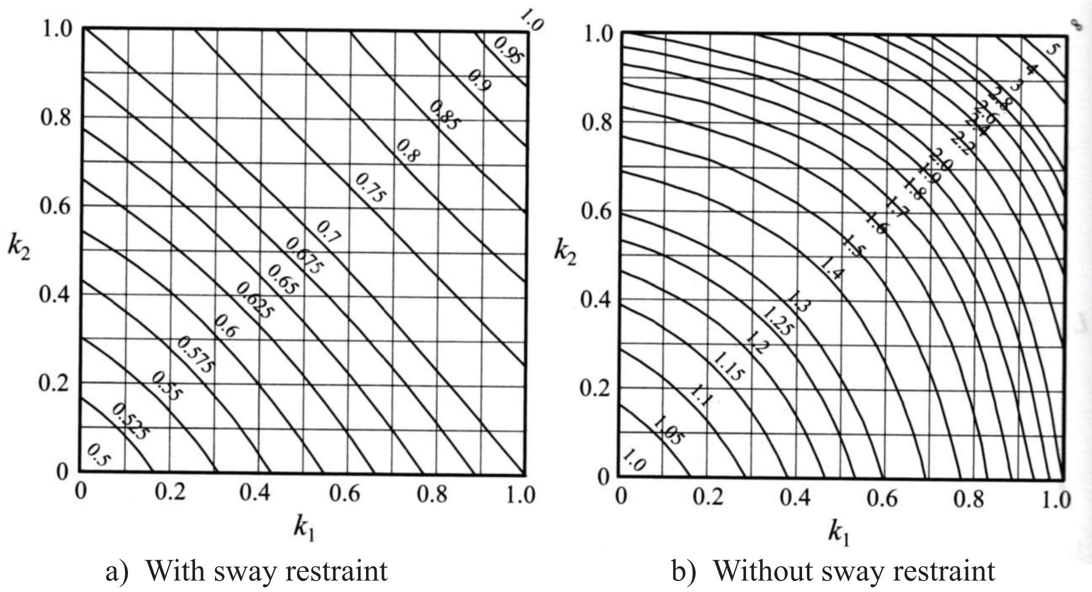
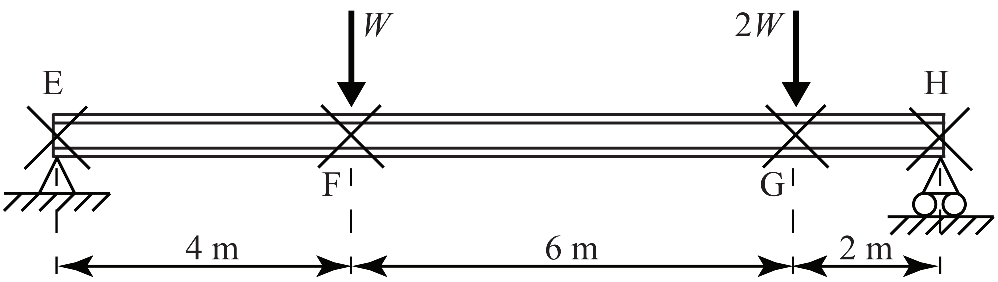


Fig. 1



1.0
0.8
0.6
0.4
0.2
0
k_2
0
0.2
0.4
0.6
0.8
1.0
k_1
0.5
0.525
0.55
0.575
0.6
0.625
0.65
0.675
0.7
0.75
0.8
0.85
0.9
0.95
1.0
a) With sway restraint
1.0
0.8
0.6
0.4
0.2
0
k_2
0
0.2
0.4
0.6
0.8
1.0
k_1
1.0
1.05
1.1
1.15
1.2
1.25
1.3
1.4
1.5
1.6
1.7
1.8
1.9
2.0
2.2
2.4
2.6
2.8
3
4
5
b) Without sway restraint

Fig. 2



W
$2W$
E
H
F
G
4 m
6 m
2 m

Fig. 3

2 A 5 m beam column of UC 152 × 152 × 37 section in S275 steel is subject to an axial force N and equal major-axis end-moments M. The end moments have the same sense, as illustrated in Fig. 4. Torsional rotation is restrained at the ends of the member and at the mid-span. The sections at the member ends are free to warp. The member has simple supports with respect to flexure about both major and minor axes, and there is a brace preventing lateral defection at the mid-span.
Determine an appropriate (M, N) interaction diagram for the capacity of the beam column.

[100%]

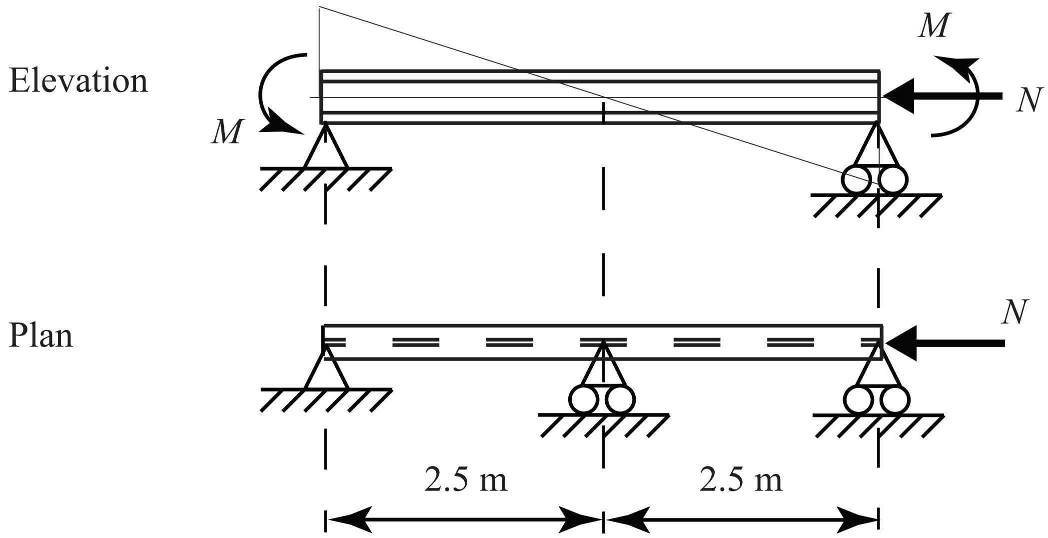


Fig. 4

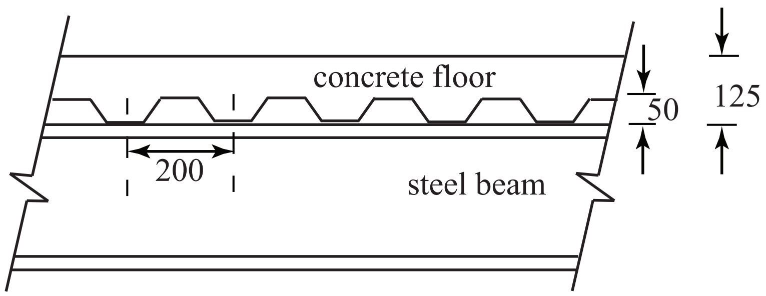
3 A composite floor is to span 12 m. It consists of a concrete slab of total thickness 125 mm, which includes 50 mm troughs from steel decking. The troughs are perpendicular to the supporting steel beams, as shown in Fig. 5. The supporting beams run the full length of the slab. They are UB 406 × 178 × 74 of S235 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 1 kN m^{-2} for permanent services and live loads of 4 kN m^{-2}. 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 has sufficient strength. [40%]

(b) Propose a suitable arrangement of shear studs to achieve full composite action. [20%]

(c) Determine whether the central deflection due to short-term application of the unfactored live load exceeds span/250. [40%]



All dimensions in mm.
(Drawing not to scale).

Fig. 5

4 Figure 6(a) shows a frame of height 12 m with a 24 m span. It has the same stiffened box cross-section throughout, as shown in Fig. 6(b), made by welding S235 steel plates of thickness 10 mm. Both supports may be assumed to be pinned, and the right hand support is on a roller bearing. The structure is fully braced against global flexural buckling of the column and lateral torsional buckling of the beam. There are transverse cross-frames at each end of the beam and at 3 m intervals in between. The beam is to carry a distributed load of 80 kN m^{-1} over its full length together with a central point load of 200 kN. The self-weight of the beam is included in these loads and no further load factors need to be applied. The column may be assumed to be adequately designed, and you are only required to consider the beam.

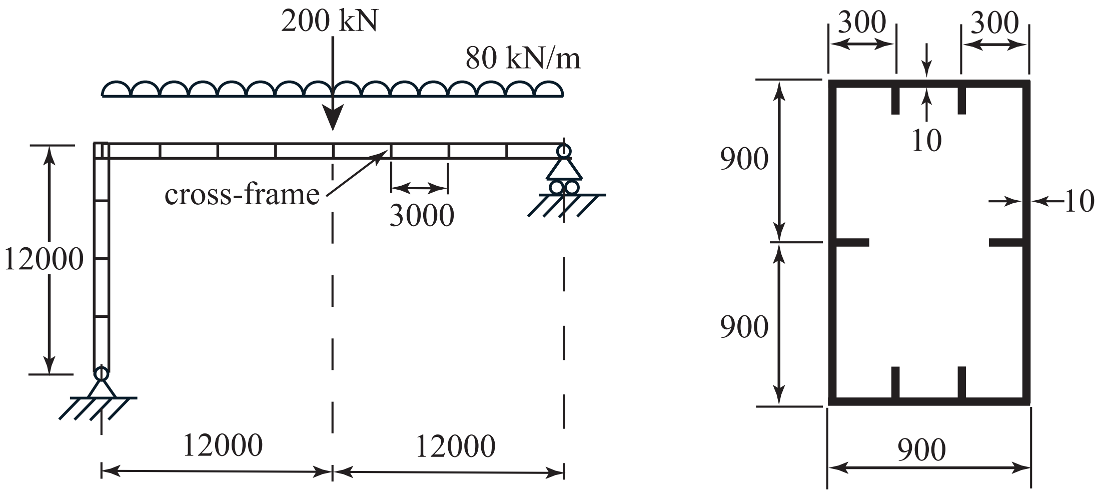
(a) Determine the compactness or otherwise of the various parts of the cross-section. [10%]

(b) Using a smeared section approximation to account for the stiffeners, estimate the major axis second moment of area. [20%]

(c) Determine whether the top flange is adequately stiffened for this load case. [20%]

(d) Check the web panels for strength and stability at critical locations. [40%]

(e) If the design is found to be inadequate in any location(s), suggest potential improvement(s). [10%]



All dimensions in mm.
All stiffeners 180 x 20 mm.
Not to scale.

(a) (b)

Fig. 6

END OF PAPER

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4D10, Structural Steelwork, May 2019 Numerical answers

1 a) 2.9 m, 193.7 MN b) 12.6 kN

4 b) 35×10^9 mm^4

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)

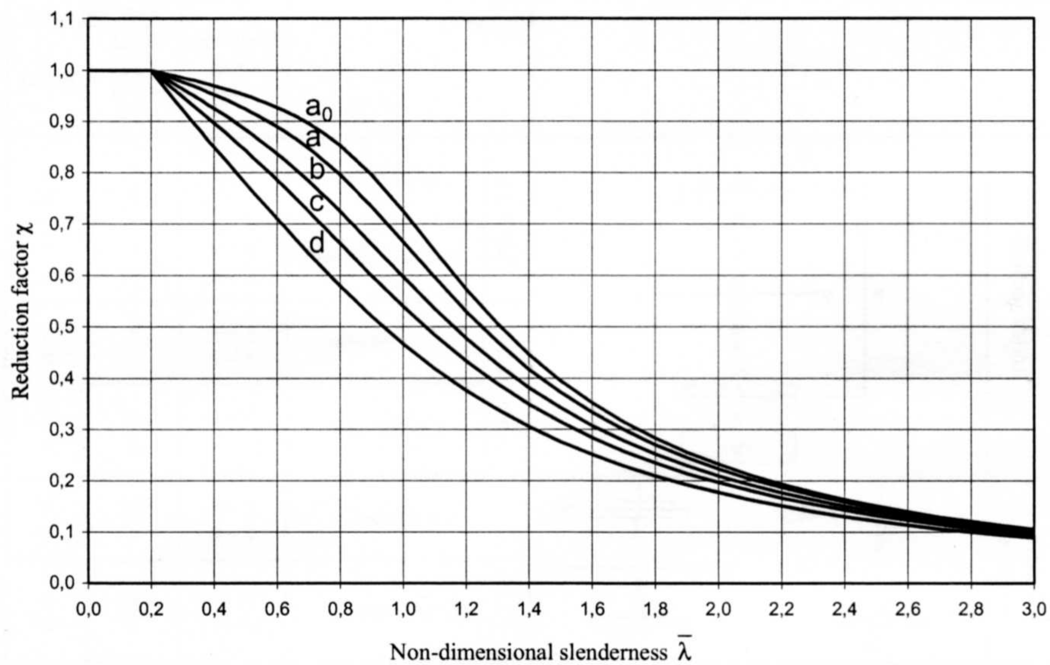


Figure 6.4: Buckling curves

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

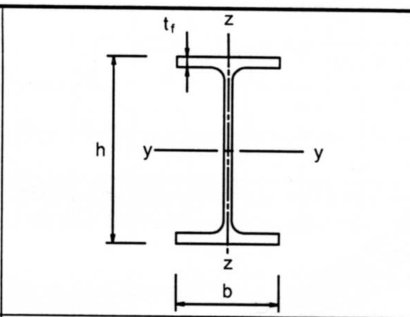
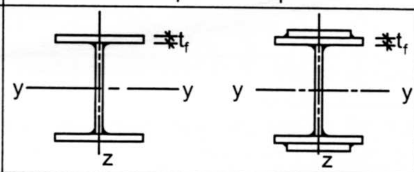
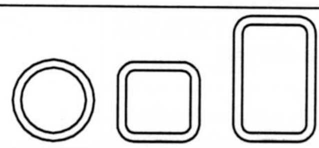
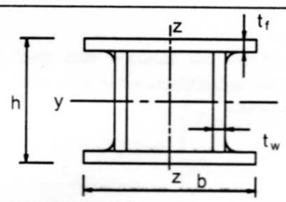
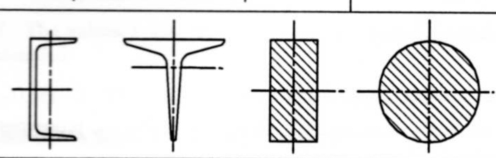
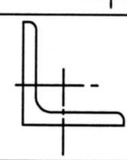
and the imperfection factor α appropriate for each curve is:

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

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

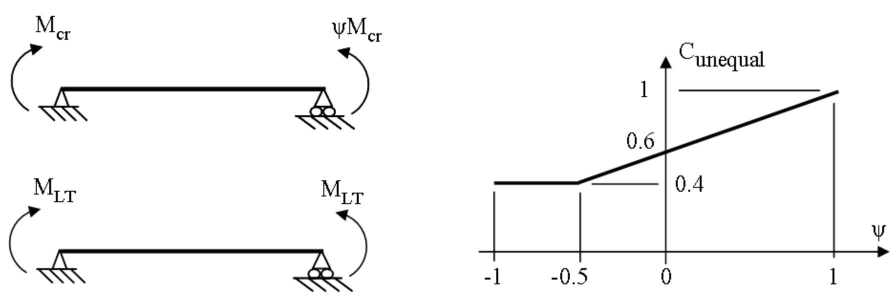
where EI, GJ and $E\Gamma$ are the minor axis flexural rigidity, the torsional rigidity and the warping rigidity respectively. (It is assumed that the supports prevent vertical, lateral and torsional deflections but do not restrain warping.)

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



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

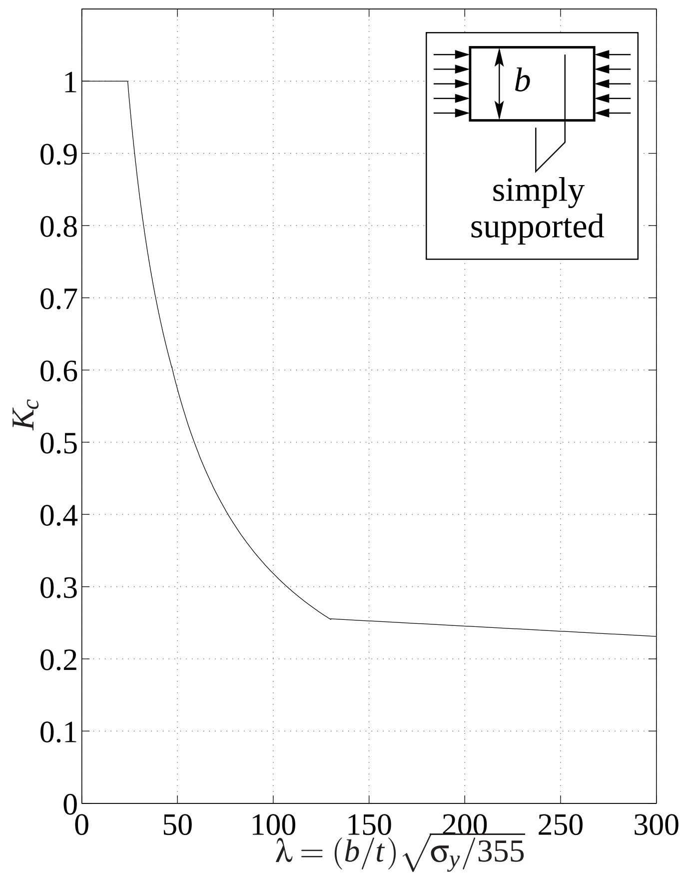
Lateral torsional buckling curve selection

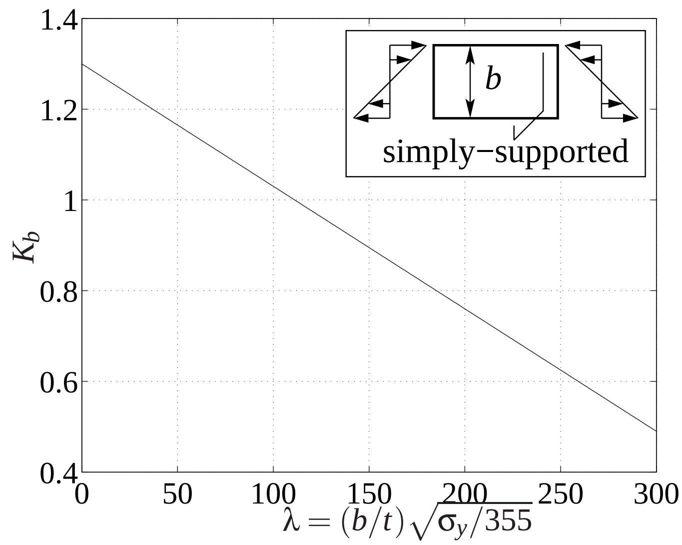
For lateral torsional buckling, the buckling resistance curves (DS1) may be used, with curves selected via the table below. Height h and width b are defined in DS2.

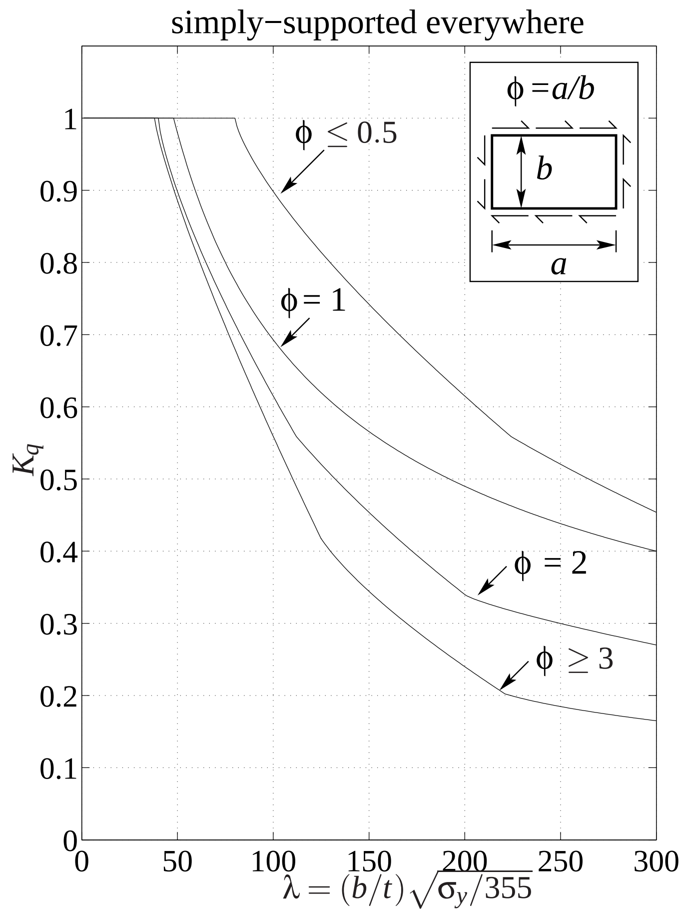
	Limits	Curve
Rolled I-sections	$h/b \leq 2$	a
	$h/b > 2$	b
Welded I-sections	$h/b \leq 2$	c
	$h/b > 2$	d
Other	-	d

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.

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

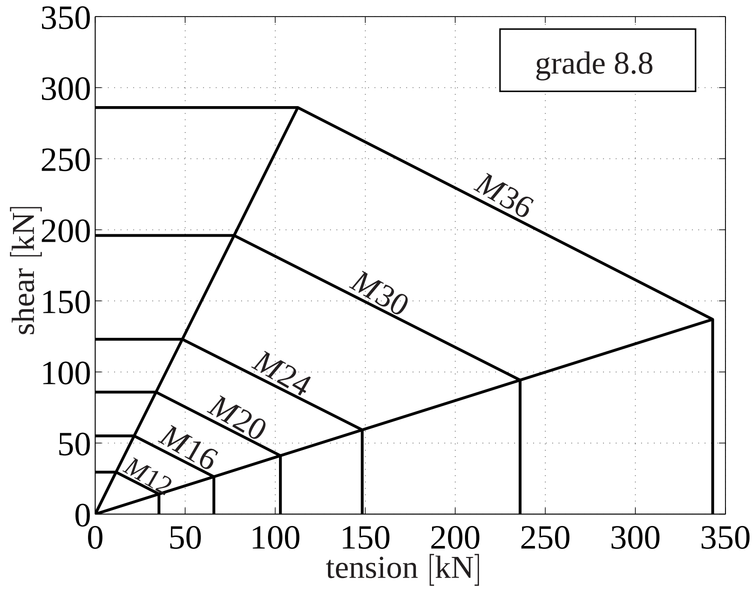
note 3: effective width, b_e, of compression flange with stiffener spacing, b, is $K_c b$.

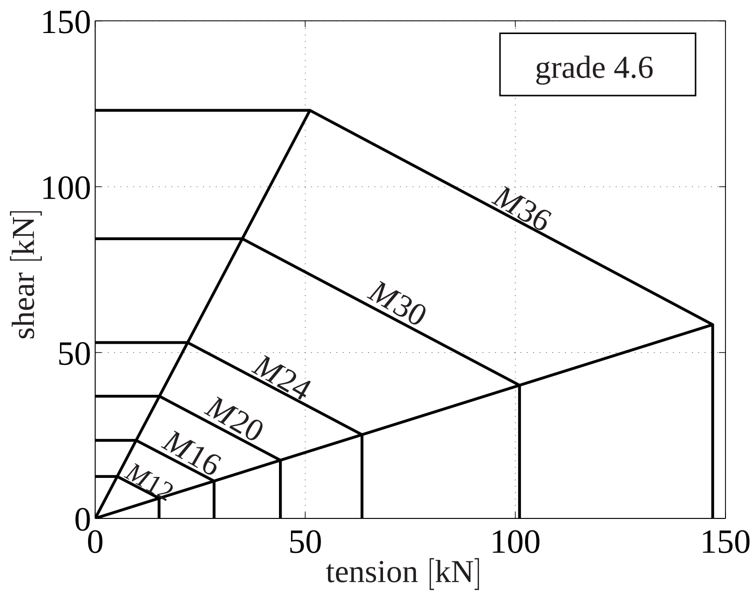
b. Plate compactness.

member and action	compact if $\lambda\ (= (b/t)\sqrt{\sigma_y/355})$
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.





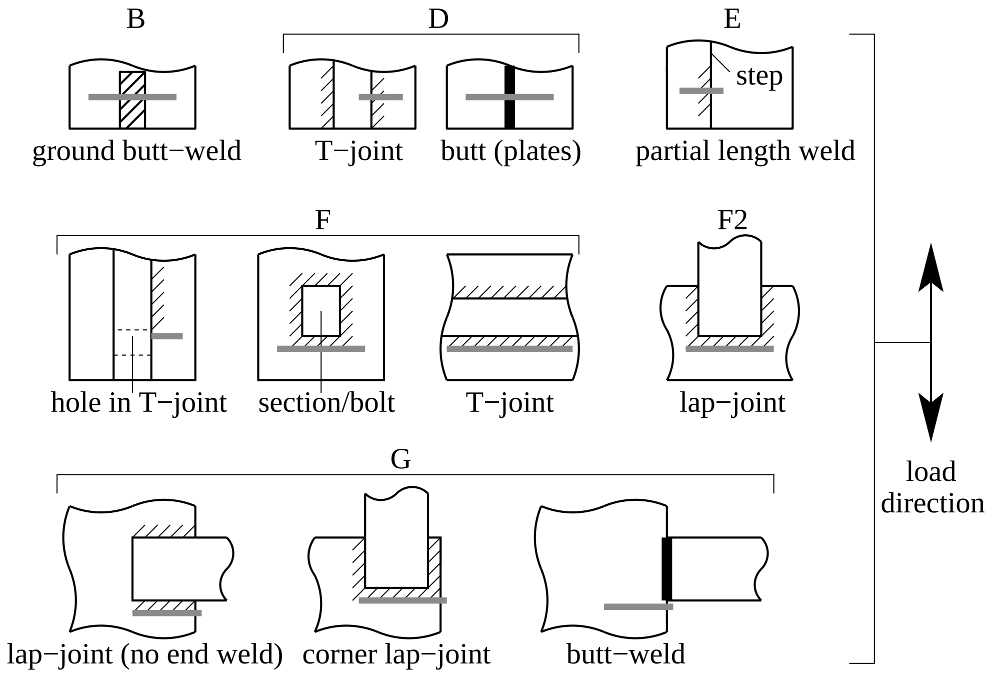
b. Bolt placement.

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 $\times\ \tau_y$.

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, 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.

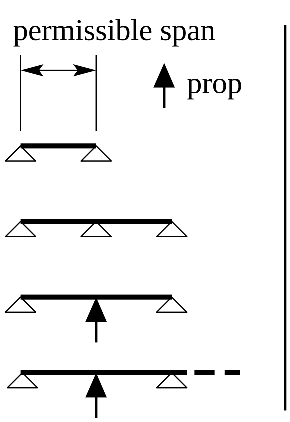
Young's modulus for grade 30 concrete, E_c, depends on duration of loading as:

$$\begin{aligned} \text{short term:} \quad & E_c = 28 \text{ GPa} \\ \text{long term:} \quad & E_c = 14 \text{ GPa} \end{aligned}$$

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.