EGT2
ENGINEERING TRIPOS PART IIA

Tuesday 4 May 2021   1.30 to 3.10

Module 3D3

STRUCTURAL MATERIALS AND DESIGN

*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 and at the top of each answer sheet.*

STATIONERY REQUIREMENTS
Write on single-sided paper.

SPECIAL REQUIREMENTS TO BE SUPPLIED FOR THIS EXAM
CUED approved calculator allowed.
Attachment: 3D3 Structural Materials and Design data sheet (18 pages).
You are allowed access to the electronic version of the Engineering Data Books.

10 minutes reading time is allowed for this paper at the start of the exam.

The time taken for scanning/uploading answers is 15 minutes.

Your script is to be uploaded as a single consolidated pdf containing all answers.

1 (a) With the aid of annotated sketches as appropriate, explain the role of characteristic values and partial safety factors for actions and resistances in achieving a suitably safe structural design using a limit state design approach. [30%]

(b) Actions on a structure may be permanent (i.e. they are always present) or transient (i.e. they may or may not be present at any given time). Different partial safety factors are usually used for permanent and transient actions. Figure 1 shows a continuous three-span beam over simple supports. The loadings on each span will be uniformly distributed. The beam is to be designed for a permanent load of 10 kNm^{-1} and a transient load of 20 kNm^{-1}. Suitable partial safety factors are indicated in Table 1. If the beam is to be made from a brittle material, sketch the design load cases that would lead to:

(i) Maximum sagging moment at the mid-span of BC. [10%]

(ii) Maximum sagging moment at the mid-span of CD. [15%]

(iii) Maximum hogging moment over support C. [15%]

Fig. 1

Table 1.

action		**partial safety factor**
permanent action	adverse effect	1.35
	beneficial effect	1.00
transient action	adverse effect	1.50
	beneficial effect	0.00

(c) The final design for the beam uses a ductile material. During construction, the support at D is built slightly too low, such that the unloaded beam is initially not in contact with the support. Without calculation but with the aid of annotated sketches, explain how the strength of the beam and the deflection at the mid-span of CD is affected for the case of a mid-span point load on span CD as shown in Fig. 2. [30%]

Fig. 2

2 Figure 3 shows a 12 m long reinforced concrete beam continuous over two simple supports at B and C. The beam is to be 300 mm wide and 600 mm deep and requires a cover of 45 mm for reasons of fire. The concrete strength is to be C32/40 meaning that it has a characteristic compressive cylinder strength f_{ck} = 32 MPa and a characteristic compressive cube strength f_{cu} = 40 MPa. All reinforcing steel used in the design is to be high yield deformed bar with a characteristic compressive and tensile strength f_{yk} = 500 MPa. Material partial safety factors are $\gamma_c = 1.5$ for concrete and $\gamma_s = 1.15$ for steel. Permanent loads including the self-weight of the beam can be taken as 3 kNm^{-1} and transient loads on the beam are 24 kNm^{-1}. Partial safety factors for load should be taken as 1.35 for permanent loads and as 1.50 for transient loads. A single load case of uniformly distributed load along the full length of the beam should be considered, as indicated in Fig. 3.

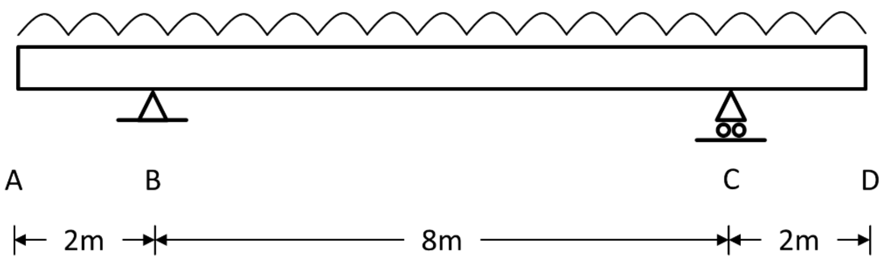

Fig. 3

(a) Sketch the shear force and bending moment diagrams for the beam, clearly identifying salient values and their locations. [20%]

(b) Design the required longitudinal reinforcement for the regions of maximum sagging and hogging moments. [30%]

(c) Determine whether transverse shear reinforcement is required in the region of maximum shear and design this reinforcement if required. [20%]

(d) Without further calculation, sketch an efficient reinforcement layout for the proposed reinforcement design. [10%]

(e) After construction, a proposal is made to reduce the tip deflection of the right-hand cantilever by providing an additional support at D. Without further calculation but with the aid of sketches, comment on the implications of this proposal and any concerns you might have as the structural engineer. [20%]

3 (a) During a design meeting, the architect you are working with shows you a sketch summarizing their preferred primary structural grid layout for a small, multi-storey office building. You are concerned that the architect's sketch does not appear to have considered stability.

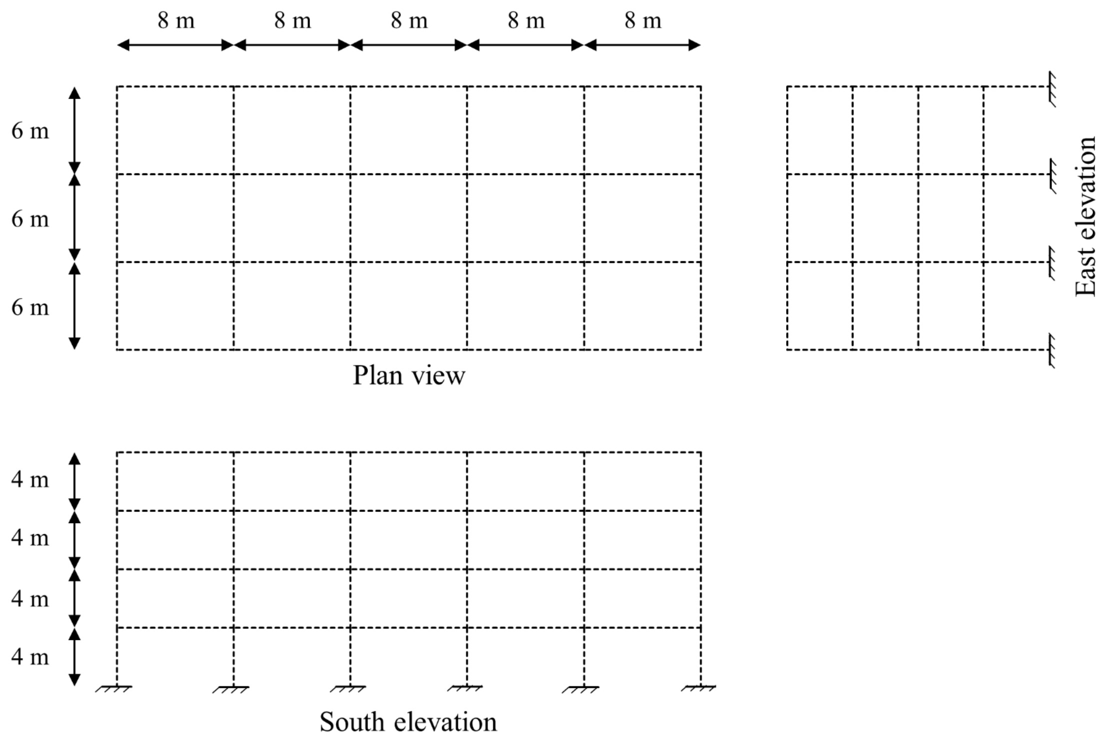

Fig. 4

(i) Sketch two suitable, efficient structural arrangements and stability systems for the building based on the architect's proposed grid – one for a structural steel frame option and one for a structural concrete frame option. Clearly indicate the restraint conditions at the ends of members (e.g., fixed, pinned) and any other design assumptions as appropriate for each option. [30%]

(ii) Comment on the reasons for your design choices in (i). [10%]

(b) The client is also attending the meeting and has a new-found interest in sustainability. During the meeting, the client asks you, "whether an engineered timber structure might reduce the environmental impacts of the building compared to a steel or concrete option?" Provide a brief but balanced answer to the client's question. [30%]

(c) The client's mind now turns to fire. Briefly explain to the client the differences between the design of a structure using steel, concrete and engineered timber from the point of view of structural fire safety. [30%]

4 A plastic tank with negligible weight, having the shape of a cube, rests on top of a steel frame. The tank is filled with 8000 litres of water. Fig. 5 shows the elevation view, which is identical in both orthogonal directions. The frame is made of S355 (hot-finished) square hollow sections, which are welded together at the connections. For the purposes of design, all connections can be considered to be hinges.

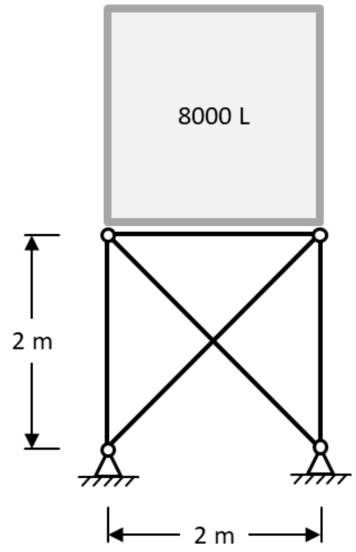

Fig. 5

The load is first transferred by contact to the horizontal upper members of the frame, which act as simply supported beams. To ensure lateral stability, the frame needs to be able to resist a lateral force equal to 2% of the vertical load. Partial safety factors for permanent load may be taken as 1.35 and for transient load may be taken as 1.50.

(a) Determine the design values of the actions that the beams, columns and bracing members that comprise the structure must resist. [20%]

Material partial safety factors are $\gamma_{M0} = 1$ for the resistance of the cross section, $\gamma_{M1} = 1$ for the resistance of the member to buckling and $\gamma_{M2} = 1.25$ for the resistance of the cross section in tension to fracture.

(b) All members have the same cross-section. Determine the member size with minimum weight so that the necessary design checks are satisfied for:

(i) the beams; [30%]

(ii) the columns; [30%]

(iii) the bracing members. [20%]

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University of Cambridge
Department of Engineering

Engineering Tripos Part IIA

Module 3D3
Structural Materials & Design

Datasheets

Michaelmas 2020

THE CUMULATIVE NORMAL DISTRIBUTION FUNCTION

$$\Phi(u) = \frac{1}{\sqrt{2\pi}}\int_{-\infty}^{u} e^{-\frac{x^2}{2}}dx \quad \text{FOR} \quad 0{\cdot}00 \le u \le 4{\cdot}99.$$

u	·00	·01	·02	·03	·04	·05	·06	·07	·08	·09
·0	·5000	·5040	·5080	·5120	·5160	·5199	·5239	·5279	·5319	·5359
·1	·5398	·5438	·5478	·5517	·5557	·5596	·5636	·5675	·5714	·5753
·2	·5793	·5832	·5871	·5910	·5948	·5987	·6026	·6064	·6103	·6141
·3	·6179	·6217	·6255	·6293	·6331	·6368	·6406	·6443	·6480	·6517
·4	·6554	·6591	·6628	·6664	·6700	·6736	·6772	·6808	·6844	·6879
·5	·6915	·6950	·6985	·7019	·7054	·7088	·7123	·7157	·7190	·7224
·6	·7257	·7291	·7324	·7357	·7389	·7422	·7454	·7486	·7517	·7549
·7	·7580	·7611	·7642	·7673	·7703	·7734	·7764	·7794	·7823	·7852
·8	·7881	·7910	·7939	·7967	·7995	·8023	·8051	·8078	·8106	·8133
·9	·8159	·8186	·8212	·8238	·8264	·8289	·8315	·8340	·8365	·8389
1·0	·8413	·8438	·8461	·8485	·8508	·8531	·8554	·8577	·8599	·8621
1·1	·8643	·8665	·8686	·8708	·8729	·8749	·8770	·8790	·8810	·8830
1·2	·8849	·8869	·8888	·8907	·8925	·8944	·8962	·8980	·8997	·90147
1·3	·90320	·90490	·90658	·90824	·90988	·91149	·91309	·91466	·91621	·91774
1·4	·91924	·92073	·92220	·92364	·92507	·92647	·92785	·92922	·93056	·93189
1·5	·93319	·93448	·93574	·93699	·93822	·93943	·94062	·94179	·94295	·94408
1·6	·94520	·94630	·94738	·94845	·94950	·95053	·95154	·95254	·95352	·95449
1·7	·95543	·95637	·95728	·95818	·95907	·95994	·96080	·96164	·96246	·96327
1·8	·96407	·96485	·96562	·96638	·96712	·96784	·96856	·96926	·96995	·97062
1·9	·97128	·97193	·97257	·97320	·97381	·97441	·97500	·97558	·97615	·97670
2·0	·97725	·97778	·97831	·97882	·97932	·97982	·98030	·98077	·98124	·98169
2·1	·98214	·98257	·98300	·98341	·98382	·98422	·98461	·98500	·98537	·98574
2·2	·98610	·98645	·98679	·98713	·98745	·98778	·98809	·98840	·98870	·98899
2·3	·98928	·98956	·98983	$\cdot 9^{2}0097$	$\cdot 9^{2}0358$	$\cdot 9^{2}0613$	$\cdot 9^{2}0863$	$\cdot 9^{2}1106$	$\cdot 9^{2}1344$	$\cdot 9^{2}1576$
2·4	$\cdot 9^{2}1802$	$\cdot 9^{2}2024$	$\cdot 9^{2}2240$	$\cdot 9^{2}2451$	$\cdot 9^{2}2656$	$\cdot 9^{2}2857$	$\cdot 9^{2}3053$	$\cdot 9^{2}3244$	$\cdot 9^{2}3431$	$\cdot 9^{2}3613$
2·5	$\cdot 9^{2}3790$	$\cdot 9^{2}3963$	$\cdot 9^{2}4132$	$\cdot 9^{2}4297$	$\cdot 9^{2}4457$	$\cdot 9^{2}4614$	$\cdot 9^{2}4766$	$\cdot 9^{2}4915$	$\cdot 9^{2}5060$	$\cdot 9^{2}5201$
2·6	$\cdot 9^{2}5339$	$\cdot 9^{2}5473$	$\cdot 9^{2}5604$	$\cdot 9^{2}5731$	$\cdot 9^{2}5855$	$\cdot 9^{2}5975$	$\cdot 9^{2}6093$	$\cdot 9^{2}6207$	$\cdot 9^{2}6319$	$\cdot 9^{2}6427$
2·7	$\cdot 9^{2}6533$	$\cdot 9^{2}6636$	$\cdot 9^{2}6736$	$\cdot 9^{2}6833$	$\cdot 9^{2}6928$	$\cdot 9^{2}7020$	$\cdot 9^{2}7110$	$\cdot 9^{2}7197$	$\cdot 9^{2}7282$	$\cdot 9^{2}7365$
2·8	$\cdot 9^{2}7445$	$\cdot 9^{2}7523$	$\cdot 9^{2}7599$	$\cdot 9^{2}7673$	$\cdot 9^{2}7744$	$\cdot 9^{2}7814$	$\cdot 9^{2}7882$	$\cdot 9^{2}7948$	$\cdot 9^{2}8012$	$\cdot 9^{2}8074$
2·9	$\cdot 9^{2}8134$	$\cdot 9^{2}8193$	$\cdot 9^{2}8250$	$\cdot 9^{2}8305$	$\cdot 9^{2}8359$	$\cdot 9^{2}8411$	$\cdot 9^{2}8462$	$\cdot 9^{2}8511$	$\cdot 9^{2}8559$	$\cdot 9^{2}8605$
3·0	$\cdot 9^{2}8650$	$\cdot 9^{2}8694$	$\cdot 9^{2}8736$	$\cdot 9^{2}8777$	$\cdot 9^{2}8817$	$\cdot 9^{2}8856$	$\cdot 9^{2}8893$	$\cdot 9^{2}8930$	$\cdot 9^{2}8965$	$\cdot 9^{2}8999$
3·1	$\cdot 9^{3}0324$	$\cdot 9^{3}0646$	$\cdot 9^{3}0957$	$\cdot 9^{3}1260$	$\cdot 9^{3}1553$	$\cdot 9^{3}1836$	$\cdot 9^{3}2112$	$\cdot 9^{3}2378$	$\cdot 9^{3}2636$	$\cdot 9^{3}2886$
3·2	$\cdot 9^{3}3129$	$\cdot 9^{3}3363$	$\cdot 9^{3}3590$	$\cdot 9^{3}3810$	$\cdot 9^{3}4024$	$\cdot 9^{3}4230$	$\cdot 9^{3}4429$	$\cdot 9^{3}4623$	$\cdot 9^{3}4810$	$\cdot 9^{3}4991$
3·3	$\cdot 9^{3}5166$	$\cdot 9^{3}5335$	$\cdot 9^{3}5499$	$\cdot 9^{3}5658$	$\cdot 9^{3}5811$	$\cdot 9^{3}5959$	$\cdot 9^{3}6103$	$\cdot 9^{3}6242$	$\cdot 9^{3}6376$	$\cdot 9^{3}6505$
3·4	$\cdot 9^{3}6631$	$\cdot 9^{3}6752$	$\cdot 9^{3}6869$	$\cdot 9^{3}6982$	$\cdot 9^{3}7091$	$\cdot 9^{3}7197$	$\cdot 9^{3}7299$	$\cdot 9^{3}7398$	$\cdot 9^{3}7493$	$\cdot 9^{3}7585$
3·5	$\cdot 9^{3}7674$	$\cdot 9^{3}7759$	$\cdot 9^{3}7842$	$\cdot 9^{3}7922$	$\cdot 9^{3}7999$	$\cdot 9^{3}8074$	$\cdot 9^{3}8146$	$\cdot 9^{3}8215$	$\cdot 9^{3}8282$	$\cdot 9^{3}8347$
3·6	$\cdot 9^{3}8409$	$\cdot 9^{3}8469$	$\cdot 9^{3}8527$	$\cdot 9^{3}8583$	$\cdot 9^{3}8637$	$\cdot 9^{3}8689$	$\cdot 9^{3}8739$	$\cdot 9^{3}8787$	$\cdot 9^{3}8834$	$\cdot 9^{3}8879$
3·7	$\cdot 9^{3}8922$	$\cdot 9^{3}8964$	$\cdot 9^{4}0039$	$\cdot 9^{4}0426$	$\cdot 9^{4}0799$	$\cdot 9^{4}1158$	$\cdot 9^{4}1504$	$\cdot 9^{4}1838$	$\cdot 9^{4}2159$	$\cdot 9^{4}2468$
3·8	$\cdot 9^{4}2765$	$\cdot 9^{4}3052$	$\cdot 9^{4}3327$	$\cdot 9^{4}3593$	$\cdot 9^{4}3848$	$\cdot 9^{4}4094$	$\cdot 9^{4}4331$	$\cdot 9^{4}4558$	$\cdot 9^{4}4777$	$\cdot 9^{4}4988$
3·9	$\cdot 9^{4}5190$	$\cdot 9^{4}5385$	$\cdot 9^{4}5573$	$\cdot 9^{4}5753$	$\cdot 9^{4}5926$	$\cdot 9^{4}6092$	$\cdot 9^{4}6253$	$\cdot 9^{4}6406$	$\cdot 9^{4}6554$	$\cdot 9^{4}6696$
4·0	$\cdot 9^{4}6833$	$\cdot 9^{4}6964$	$\cdot 9^{4}7090$	$\cdot 9^{4}7211$	$\cdot 9^{4}7327$	$\cdot 9^{4}7439$	$\cdot 9^{4}7546$	$\cdot 9^{4}7649$	$\cdot 9^{4}7748$	$\cdot 9^{4}7843$
4·1	$\cdot 9^{4}7934$	$\cdot 9^{4}8022$	$\cdot 9^{4}8106$	$\cdot 9^{4}8186$	$\cdot 9^{4}8263$	$\cdot 9^{4}8338$	$\cdot 9^{4}8409$	$\cdot 9^{4}8477$	$\cdot 9^{4}8542$	$\cdot 9^{4}8605$
4·2	$\cdot 9^{4}8665$	$\cdot 9^{4}8723$	$\cdot 9^{4}8778$	$\cdot 9^{4}8832$	$\cdot 9^{4}8882$	$\cdot 9^{4}8931$	$\cdot 9^{4}8978$	$\cdot 9^{5}0226$	$\cdot 9^{5}0655$	$\cdot 9^{5}1066$
4·3	$\cdot 9^{5}1460$	$\cdot 9^{5}1837$	$\cdot 9^{5}2199$	$\cdot 9^{5}2545$	$\cdot 9^{5}2876$	$\cdot 9^{5}3193$	$\cdot 9^{5}3497$	$\cdot 9^{5}3788$	$\cdot 9^{5}4066$	$\cdot 9^{5}4332$
4·4	$\cdot 9^{5}4587$	$\cdot 9^{5}4831$	$\cdot 9^{5}5065$	$\cdot 9^{5}5288$	$\cdot 9^{5}5502$	$\cdot 9^{5}5706$	$\cdot 9^{5}5902$	$\cdot 9^{5}6089$	$\cdot 9^{5}6268$	$\cdot 9^{5}6439$
4·5	$\cdot 9^{5}6602$	$\cdot 9^{5}6759$	$\cdot 9^{5}6908$	$\cdot 9^{5}7051$	$\cdot 9^{5}7187$	$\cdot 9^{5}7318$	$\cdot 9^{5}7442$	$\cdot 9^{5}7561$	$\cdot 9^{5}7675$	$\cdot 9^{5}7784$
4·6	$\cdot 9^{5}7888$	$\cdot 9^{5}7987$	$\cdot 9^{5}8081$	$\cdot 9^{5}8172$	$\cdot 9^{5}8258$	$\cdot 9^{5}8340$	$\cdot 9^{5}8419$	$\cdot 9^{5}8494$	$\cdot 9^{5}8566$	$\cdot 9^{5}8634$
4·7	$\cdot 9^{5}8699$	$\cdot 9^{5}8761$	$\cdot 9^{5}8821$	$\cdot 9^{5}8877$	$\cdot 9^{5}8931$	$\cdot 9^{5}8983$	$\cdot 9^{6}0320$	$\cdot 9^{6}0789$	$\cdot 9^{6}1235$	$\cdot 9^{6}1661$
4·8	$\cdot 9^{6}2067$	$\cdot 9^{6}2453$	$\cdot 9^{6}2822$	$\cdot 9^{6}3173$	$\cdot 9^{6}3508$	$\cdot 9^{6}3827$	$\cdot 9^{6}4131$	$\cdot 9^{6}4420$	$\cdot 9^{6}4696$	$\cdot 9^{6}4958$
4·9	$\cdot 9^{6}5208$	$\cdot 9^{6}5446$	$\cdot 9^{6}5673$	$\cdot 9^{6}5889$	$\cdot 9^{6}6094$	$\cdot 9^{6}6289$	$\cdot 9^{6}6475$	$\cdot 9^{6}6652$	$\cdot 9^{6}6821$	$\cdot 9^{6}6981$

Example: $\Phi(3{\cdot}57) = \cdot 9^{3}8215 = 0{\cdot}9998215$.

Steel Data Sheet

(EN 1993-1-1)

Table 3.1: Nominal values of yield strength f_y and ultimate tensile strength f_u for hot rolled structural steel

Standard and steel grade	Nominal thickness of the element t [mm]			
	$t \leq 40$ mm		40 mm < $t \leq 80$ mm	
	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]
EN 10025-2				
S 235	235	360	215	360
S 275	275	430	255	410
S 355	355	AC2 490 AC2	335	470
S 450	440	550	410	550

Tension members

Yielding of the gross cross-section A_g:

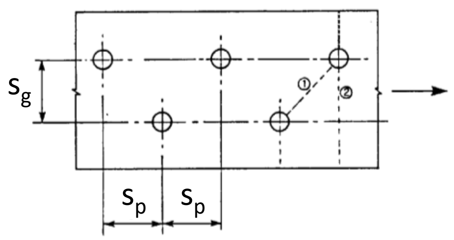
$$N_{pl,Rd} = \frac{A_g f_y}{\gamma_{M0}}$$

Fracture of the net cross-section A_n:

$$N_{u,Rd} = \frac{0.9 A_n f_u}{\gamma_{M2}}$$

Staggered bolt holes:

$$A_n = A_g - n_b d_0 t + \sum_{staggers} \frac{s_p^2 t}{4 s_g}$$



d_0 = bolt hole diameter
n_b = number of bolt lines across the member

Bolt size	**12**	**14**	**16**	**18**	**20**	**22**	**24**	**27 to 36**
Clearance (mm)	1	1	2	2	2	2	2	3

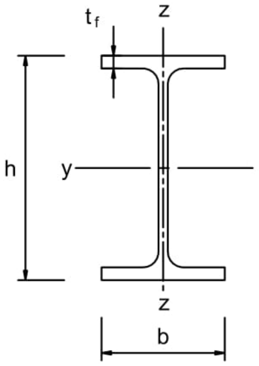
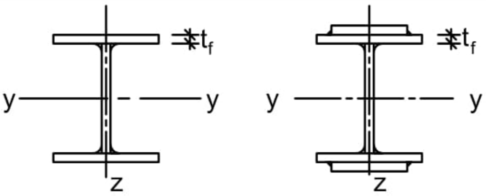
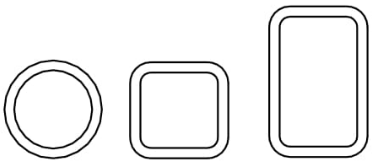
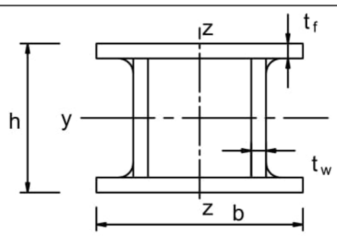
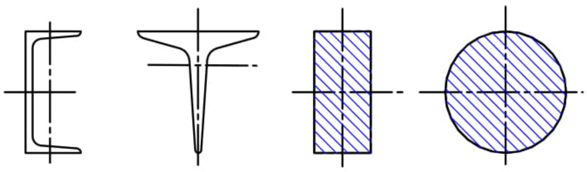
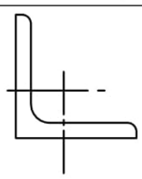
Reduction factor for shear lag in eccentrically connected angles:

Pitch	p_1	$\leq 2{,}5\ d_o$	$\geq 5{,}0\ d_o$
2 bolts	β_2	0,4	0,7
3 bolts or more	β_3	0,5	0,7

Column 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
			$40 \text{ mm} < t_f \leq 100$	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

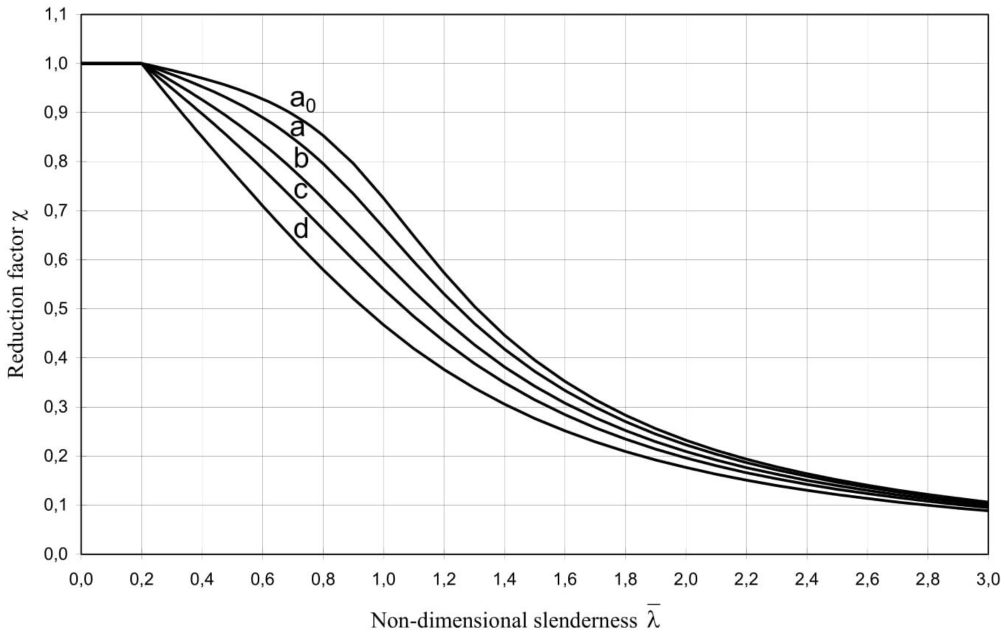


Figure 6.4: Buckling curves

6.3.1.2 Buckling curves

(1) For axial compression in members the value of χ for the appropriate non-dimensional slenderness $\bar{\lambda}$ should be determined from the relevant buckling curve according to:

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \quad \text{but } \chi \leq 1{,}0 \tag{6.49}$$

where $\Phi = 0{,}5\left[1 + \alpha\left(\bar{\lambda} - 0{,}2\right) + \bar{\lambda}^2\right]$

$\bar{\lambda} = \sqrt{\dfrac{Af_y}{N_{cr}}}$ for Class 1, 2 and 3 cross-sections

$\bar{\lambda} = \sqrt{\dfrac{A_{eff} f_y}{N_{cr}}}$ for Class 4 cross-sections

α is an imperfection factor

N_{cr} is the elastic critical force for the relevant buckling mode based on the gross cross sectional properties.

(2) The imperfection factor α corresponding to the appropriate buckling curve should be obtained from Table 6.1 and Table 6.2.

Table 6.1: Imperfection factors for buckling curves

Buckling curve	a_0	a	b	c	d
Imperfection factor α	0,13	0,21	0,34	0,49	0,76

Local buckling

$$\sigma_{cr} = K\frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2$$

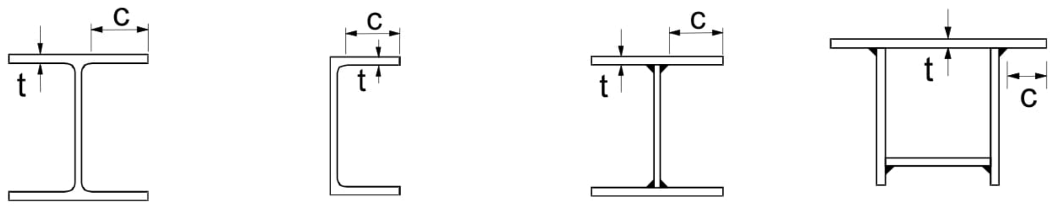
where b is the width of the plate and t is its thickness.

For plates in uniform longitudinal compression:

$K = 4$ for internal elements.

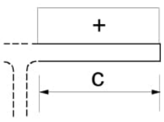
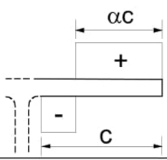
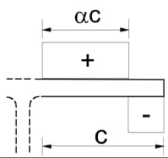
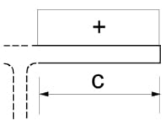
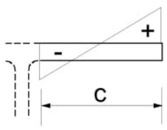
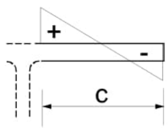
$K = 0.43$ for outstand elements.

Outstand flanges



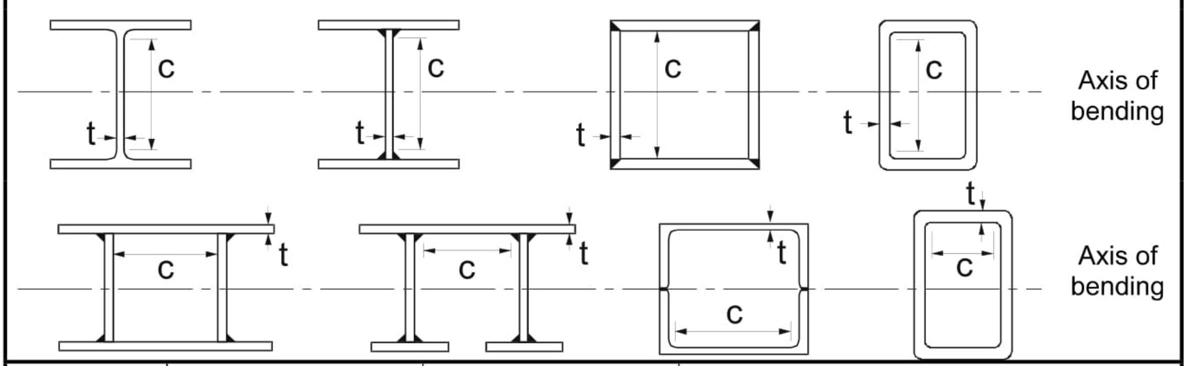
Rolled sections

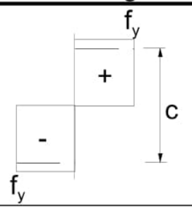
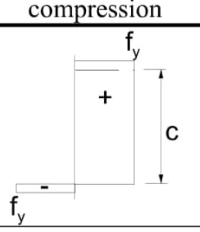
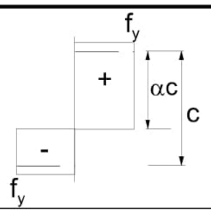
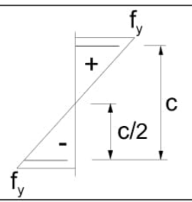
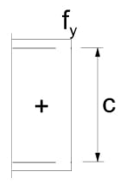
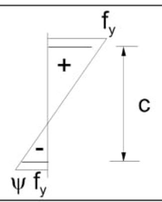
Welded sections

Class	Part subject to compression	Part subject to bending and compression	
		Tip in compression	Tip in tension
Stress distribution in parts (compression positive)			
1	$c/t \le 9\varepsilon$	$c/t \le \frac{9\varepsilon}{\alpha}$	$c/t \le \frac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	$c/t \le 10\varepsilon$	$c/t \le \frac{10\varepsilon}{\alpha}$	$c/t \le \frac{10\varepsilon}{\alpha\sqrt{\alpha}}$
Stress distribution in parts (compression positive)			
3	$c/t \le 14\varepsilon$	$c/t \le 21\varepsilon\sqrt{k_\sigma}$ For k_σ see EN 1993-1-5	

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

Internal compression parts



Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \leq 72\varepsilon$	$c/t \leq 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \leq \dfrac{396\varepsilon}{13\alpha - 1}$ when $\alpha \leq 0{,}5$: $c/t \leq \dfrac{36\varepsilon}{\alpha}$
2	$c/t \leq 83\varepsilon$	$c/t \leq 38\varepsilon$	when $\alpha > 0{,}5$: $c/t \leq \dfrac{456\varepsilon}{13\alpha - 1}$ when $\alpha \leq 0{,}5$: $c/t \leq \dfrac{41{,}5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)			
3	$c/t \leq 124\varepsilon$	$c/t \leq 42\varepsilon$	when $\psi > -1$: $c/t \leq \dfrac{42\varepsilon}{0{,}67 + 0{,}33\psi}$ when $\psi \leq -1^{*)}$: $c/t \leq 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

*) $\psi \leq -1$ applies where either the compression stress $\sigma \leq f_y$ or the tensile strain $\varepsilon_y > f_y/E$

Beams

Elastic lateral-torsional buckling moment of a beam with doubly symmetric cross-section:

$$M_{cr,0} = \frac{\pi^2 E I_z}{L_{cr}^2}\left[\frac{I_w}{I_z} + \frac{L_{cr}^2 G I_T}{\pi^2 E I_z}\right]^{0.5}$$

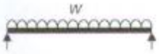
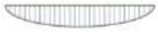
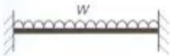
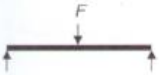
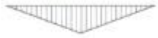
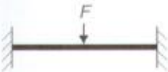
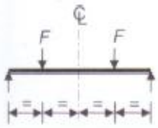
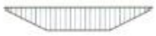
where:

I_T = torsional constant
I_w = warping constant (= $d^2 I_{yy}/4$ for I-beams, with d the distance between the centerlines of the flanges)
I_z = second moment of area about the minor axis
G = shear modulus
L_{cr} = unrestrained length for lateral-torsional buckling

In the case of non-uniform bending:

$$M_{cr} = C_1 M_{cr,0}$$

Loading and support conditions	Bending moment diagram	Value of C_1
	$\psi = +1$ 	1.000
	$\psi = +0.75$ 	1.141
	$\psi = +0.5$ 	1.323
	$\psi = +0.25$ 	1.563
	$\psi = 0$ 	1.879
	$\psi = -0.25$ 	2.281
	$\psi = -0.5$ 	2.704
	$\psi = -0.75$ 	2.729
	$\psi = -1$ 	2.752

Loading and support conditions	Bending moment diagram	Value of C_1
		1.132
		1.285
		1.365
		1.565
		1.046

(EN 1993-1-1)

6.3.2.2 Lateral torsional buckling curves – General case

(1) Unless otherwise specified, see 6.3.2.3, for bending members of constant cross-section, the value of χ_{LT} for the appropriate non-dimensional slenderness $\bar{\lambda}_{LT}$, should be determined from:

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \quad \text{but } \chi_{LT} \leq 1{,}0 \qquad (6.56)$$

where $\Phi_{LT} = 0{,}5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0{,}2\right) + \bar{\lambda}_{LT}^2\right]$

α_{LT} is an imperfection factor

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}}$$

M_{cr} is the elastic critical moment for lateral-torsional buckling

(2) M_{cr} is based on gross cross sectional properties and takes into account the loading conditions, the real moment distribution and the lateral restraints.

NOTE The imperfection factor α_{LT} corresponding to the appropriate buckling curve may be obtained from the National Annex. The recommended values α_{LT} are given in Table 6.3.

Table 6.3: Recommended values for imperfection factors for lateral torsional buckling curves

Buckling curve	a	b	c	d
Imperfection factor α_{LT}	0,21	0,34	0,49	0,76

The recommendations for buckling curves are given in Table 6.4.

Table 6.4: Recommended values for lateral torsional buckling curves for cross-sections using equation (6.56)

Cross-section	Limits	Buckling curve
Rolled I-sections	h/b ≤ 2 h/b > 2	**a** **b**
Welded I-sections	h/b ≤ 2 h/b > 2	**c** **d**
Other cross-sections	-	**d**

$$M_{b,Rd} = \chi_{LT} \; W_y \frac{f_y}{\gamma_{M1}}$$

Interaction between moment and shear in the cross-section:

$$f_{yr} = (1-\rho) f_y \qquad \rho = \left(\frac{2V_{Ed}}{V_{pl,Rd}} - 1\right)^2 \qquad \text{(for } V_{Ed} > 0.5_{Vpl,Rd}\text{)}$$

$$M_{y,V,Rd} = \left[W_{pl,y} - \frac{\rho {A_W}^2}{4t_w}\right]\frac{f_y}{\gamma_{M0}} \le M_{y,c,Rd} \qquad \text{where } A_w = h_w t_w$$

Shear

$$V_{pl,Rd} = A_v \frac{(f_y/\sqrt{3})}{\gamma_{M0}}$$

$$A_v = A - 2bt_f + (t_w + 2r)t_f \qquad \text{but} \quad \ge \; h_w t_w$$

where:
b = flange width
t_f = flange thickness
t_w = web thickness
h_w = web height
r = transition radius between web and flange

Shear buckling:

$$\tau_{cr} = K \frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2$$

$$K = 5.34 + \frac{4}{(a/b)^2} \qquad \text{if} \;\; a > b$$

$$K = 5.34 + \frac{4}{(b/a)^2} \qquad \text{if} \;\; b > a$$

Shear buckling needs to be checked if: $\frac{h_w}{t_w} \ge 72\varepsilon$

where h_w is the web height, t_w is the web thickness and $\varepsilon = \sqrt{235/f_y}$ (with f_y in MPa).

$$V_{b,Rd} = \chi_w \frac{(f_y/\sqrt{3})h_w t_w}{\gamma_{M1}} \qquad \lambda_w = 0.76\sqrt{\frac{f_y}{\tau_{cr}}}$$

Table 5.1: Contribution from the web χ_w to shear buckling resistance

	Rigid end post	Non-rigid end post
$\bar{\lambda}_w < 0{,}83/\eta$	η	η
$0{,}83/\eta \le \bar{\lambda}_w < 1{,}08$	$0{,}83/\bar{\lambda}_w$	$0{,}83/\bar{\lambda}_w$
$\bar{\lambda}_w \ge 1{,}08$	$1{,}37/(0{,}7+\bar{\lambda}_w)$	$0{,}83/\bar{\lambda}_w$

Web crippling:

$$\bar{\lambda}_F = \sqrt{\frac{F_y}{F_{cr}}} = \sqrt{\frac{l_y t_w f_{yw}}{F_{cr}}} \qquad F_{cr} = 0.9 k_F E \frac{t_w^3}{h_w}$$

$$\chi_F = \frac{0.5}{\bar{\lambda}_F} \le 1.0 \qquad F_{Rd} = \chi_F \frac{l_y t_w f_{yw}}{\gamma_{M1}}$$

IOF/ITF: $\ell_y = s_s + 2t_f\left(1+\sqrt{m_1+m_2}\right) \le a$

$$m_1 = \frac{f_{yf} b_f}{f_{yw} t_w}$$

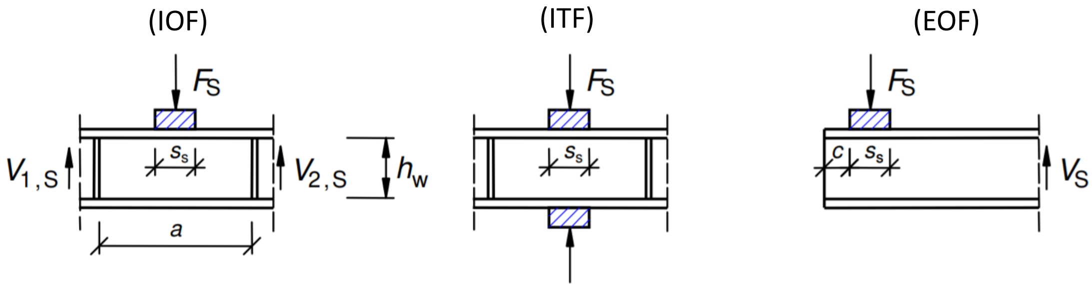
EOF:

$$\ell_y = \min\begin{cases} \ell_e + t_f\sqrt{\dfrac{m_1}{2}+\left(\dfrac{\ell_e}{t_f}\right)^2+m_2} \\ \ell_e + t_f\sqrt{m_1+m_2} \end{cases}$$

$$m_2 = 0{,}02\left(\frac{h_w}{t_f}\right)^2 \quad \text{if } \bar{\lambda}_F > 0{,}5$$

$$m_2 = 0 \quad \text{if } \bar{\lambda}_F \le 0{,}5$$

with: $\ell_e = \dfrac{k_F E t_w^2}{2 f_{yw} h_w} \le s_s + c$



$$k_F = 6 + 2\left(\frac{h_w}{a}\right)^2 \qquad k_F = 3{,}5 + 2\left(\frac{h_w}{a}\right)^2 \qquad k_F = 2 + 6\left(\frac{s_s+c}{h_w}\right) \le 6$$

Deflections:

Vertical deflection	
Cantilevers	Length/180
Beams carrying plaster or other brittle finish	Span/360
Other beams (except purlins and sheeting rails)	Span/200
Purlins and sheeting rails	To suit the characteristics of particular cladding

3D3 – Structural Materials and Design – Timber Datasheet

			C14	C16	C18	C22	C24	C27	C40
$f_{m,k}$	bending	MPa	14	16	18	22	24	27	40
$f_{t,0,k}$	tens $\vert\vert$	MPa	8	10	11	13	14	16	24
$f_{t,90,k}$	tens $\perp$	MPa	0.3	0.3	0.3	0.3	0.4	0.4	0.4
$f_{c,0,k}$	comp $\vert\vert$	MPa	16	17	18	20	21	22	26
$f_{c,90,k}$	comp $\perp$	MPa	4.3	4.6	4.8	5.1	5.3	5.6	6.3
$f_{v,k}$	shear	MPa	1.7	1.8	2.0	2.4	2.5	2.8	3.8
$E_{0,mean}$	tens mod $\vert\vert$	GPa	7	8	9	10	11	12	14
$E_{0,05}$	tens mod	GPa	4.7	5.4	6	6.7	7.4	8	9.4
$E_{90,mean}$	tens mod $\perp$	GPa	0.23	0.27	0.3	0.33	0.37	0.4	0.47
G_{mean}	shear mod	GPa	0.44	0.50	0.56	0.63	0.69	0.75	0.88
ρ_k	density	kg/m^3	290	310	320	340	350	370	420
ρ_{mean}	density	kg/m^3	350	370	380	410	420	450	500

Table 11.2 Selected strength classes - characteristic values according to EN 338 [11.3]– Coniferous Species and Poplar (Table 1)

Table 3.1.7 Values of k_{mod}

Material/ load-duration class	Service class 1	Service class 2	Service class 3
Solid and glued laminated timber and plywood			
Permanent	0.60	0.60	0.50
Long-term	0.70	0.70	0.55
Medium-term	0.80	0.80	0.65
Short-term	0.90	0.90	0.70
Instantaneous	1.10	1.10	0.90

Selected Modification Factors for Service Class and Duration of Load [11.2]

[11.2] DD ENV 1995-1-1 :1994 Eurocode 5: Design of timber structures – Part 1.1 General rules and rules for buildings
[11.3] BS EN 338:1995 Structural Timber – Strength classes

Flexure - Design bending strength

$$f_{m,d} = k_{mod} k_h k_{crit} k_{ls} f_{m,k} / \gamma_m$$

Shear – Design shear stress

$$f_{v,d} = k_{mod} k_{ls} f_{v,k} / \gamma_m$$

Bearing – Design bearing stress

$$f_{c,90,d} = k_{ls} k_{c,90} k_{mod} f_{c,90,k} / \gamma_m$$

Stability – Relative slenderness for bending

$\lambda_{rel,m} = \sqrt{f_{m,k} / \sigma_{m,crit}}$

"For beams with an initial lateral deviation from straightness within the limits defined in chapter 7, k_{crit} may be determined from (5.2.2 c-e)"

$$k_{crit} = \begin{cases} 1 & \text{for} \quad \lambda_{rel,m} \leq 0.75 & (5.2.2c) \\ 1.56 - 0.75\lambda_{rel,m} & \text{for} \quad 0.75 < \lambda_{rel,m} \leq 1.4 & (5.2.2d) \\ 1/\lambda_{rel,m}^2 & \text{for} \quad 1.4 < \lambda_{rel,m} & (5.2.2e) \end{cases}$$

Extract from [11.2] - k_{crit}

Joints

For bolts and for nails *with* predrilled holes, the characteristic embedding strength $f_{h,0,k}$ is:

$f_{h,0,k} = 0.082(1 - 0.01d)\rho_k$ N/mm²

For bolts up to 30 mm diameter at an angle α to the grain:

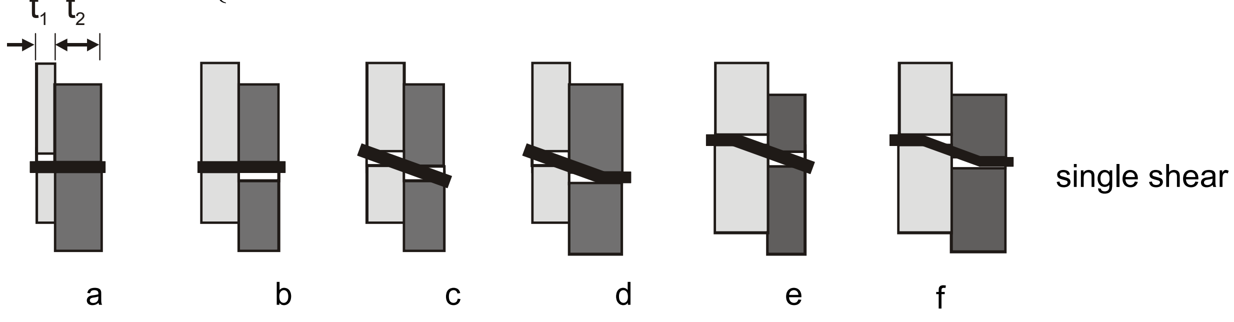
$$f_{h,\alpha,k} = \frac{f_{h,0,k}}{k_{90}\sin^2\alpha + \cos^2\alpha} \qquad \begin{array}{ll} \text{for softwood} & k_{90} = 1.35 + 0.015d \\ \text{for hardwood} & k_{90} = 0.90 + 0.015d \end{array}$$

Design yield moment for round steel bolts: $M_{y,d} = (0.8 f_{u,k} d^3)/(6\gamma_m)$

Design embedding strength e.g. for material 1: $f_{h,1,d} = (k_{mod,1} f_{h,1,k})/\gamma_m$

Design load-carrying capacities for fasteners in single shear

$$R_d = \min.\begin{cases} f_{h,1,d}\, t_1\, d & (6.2.1a) \\ f_{h,1,d}\, t_2\, d\beta & (6.2.1b) \\ \dfrac{f_{h,1,d}\, t_1 d}{1+\beta}\left[\sqrt{\beta + 2\beta^2\left[1 + \dfrac{t_2}{t_1} + \left(\dfrac{t_2}{t_1}\right)^2\right] + \beta^3\left(\dfrac{t_2}{t_1}\right)^2} - \beta\left(1 + \dfrac{t_2}{t_1}\right)\right] & (6.2.1c) \\ 1.1\dfrac{f_{h,1,d}\, t_1\, d}{2+\beta}\left[\sqrt{2\beta(1+\beta) + \dfrac{4\beta(2+\beta)M_{y,d}}{f_{h,1,d}\, d\, t_1^2}} - \beta\right] & (6.2.1d) \\ 1.1\dfrac{f_{h,1,d}\, t_2\, d}{1+2\beta}\left[\sqrt{2\beta^2(1+\beta) + \dfrac{4\beta(1+2\beta)M_{y,d}}{f_{h,1,d}\, d\, t_2^2}} - \beta\right] & (6.2.1e) \\ 1.1\sqrt{\dfrac{2\beta}{1+\beta}}\sqrt{2M_{y,d} f_{h,1,d}\, d} & (6.2.1f) \end{cases}$$



3D3 – Structural Materials and Design – Masonry Datasheet

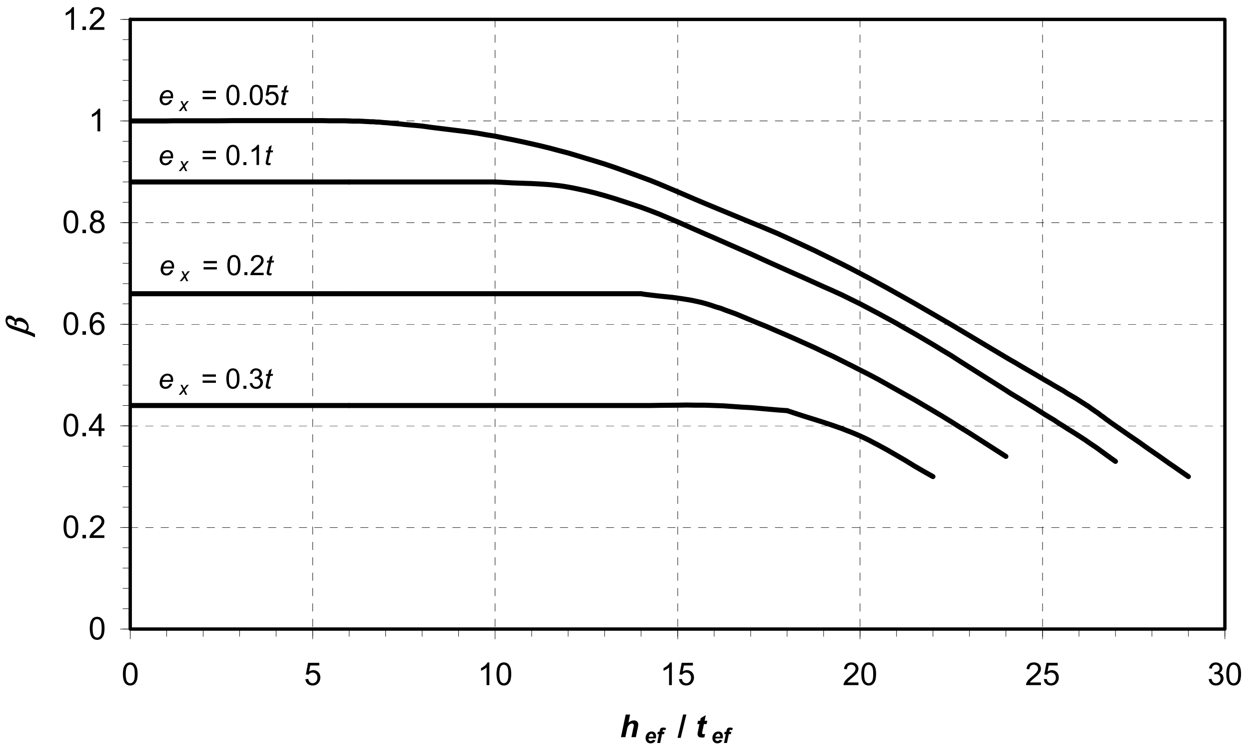
Bearing or crushing resistance per unit length

$$P_b = \frac{f_k t}{\gamma_m}$$

Buckling resistance per unit length

$$P_b = \frac{\beta f_k t}{\gamma_m}$$

Graph for capacity reduction factor β



Flexural resistance per unit length

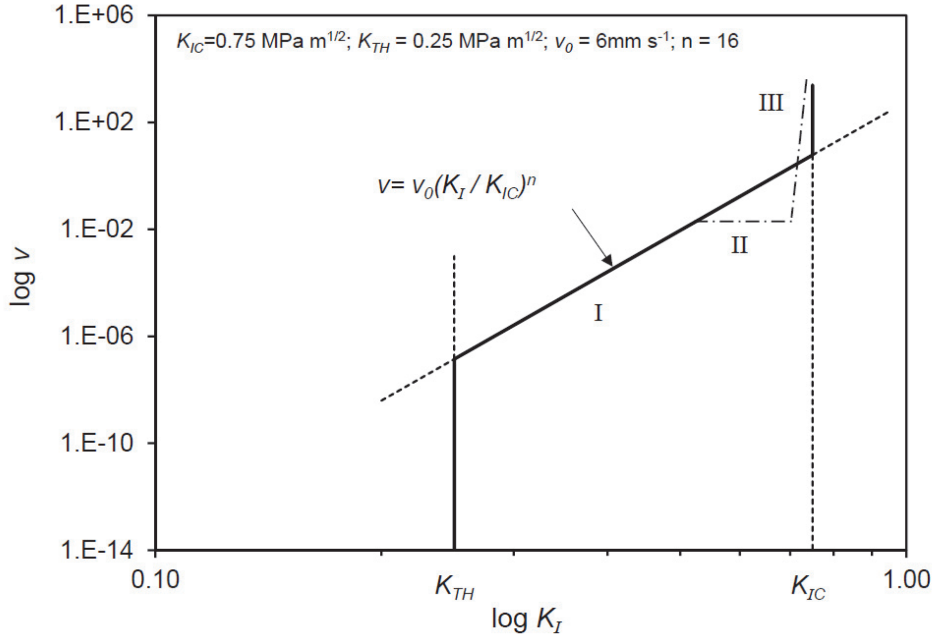
$$M = \frac{f_{kx} Z}{\gamma_m}$$

3D3 – Structural Materials and Design – Glass Datasheet

Explicit relationship between the flaw opening stress history and the initial flaw size:

$$\int_0^{t_f} \sigma^n(t)dt \approx \frac{2}{(n-2)\,v_0\,K_{IC}^{-n}\left(Y\sqrt{\pi}\right)^n a_i^{(n-2)/2}}$$

Idealised v–K relationship:



2-parameter Weibull distribution:

$$P_f = 1 - \exp\left[-kA\left(\sigma_f - f_{rk}\right)^m\right]$$

Stressed surface area factor (uniform stress):

$$\frac{\sigma_f}{\sigma_{A0}} = \left(\frac{A_0}{A_f}\right)^{1/m} = k_A$$

Load duration factor (constant stress history):

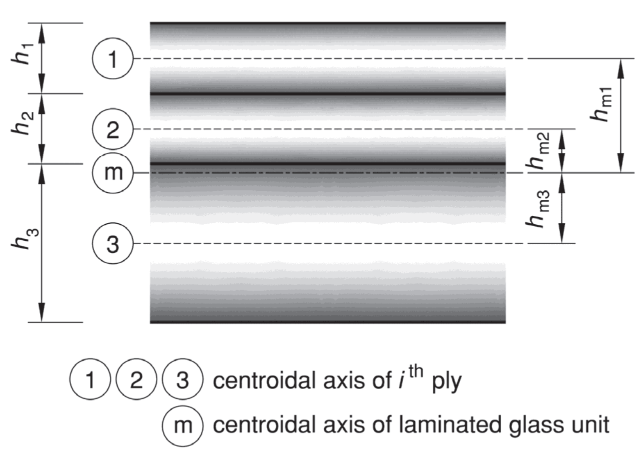
$$\frac{\sigma_f}{\sigma_{t0}} = \left(\frac{t_0}{t_f}\right)^{1/n} = k_{\text{mod}}$$

Laminated glass equivalent thickness for bending deflection:

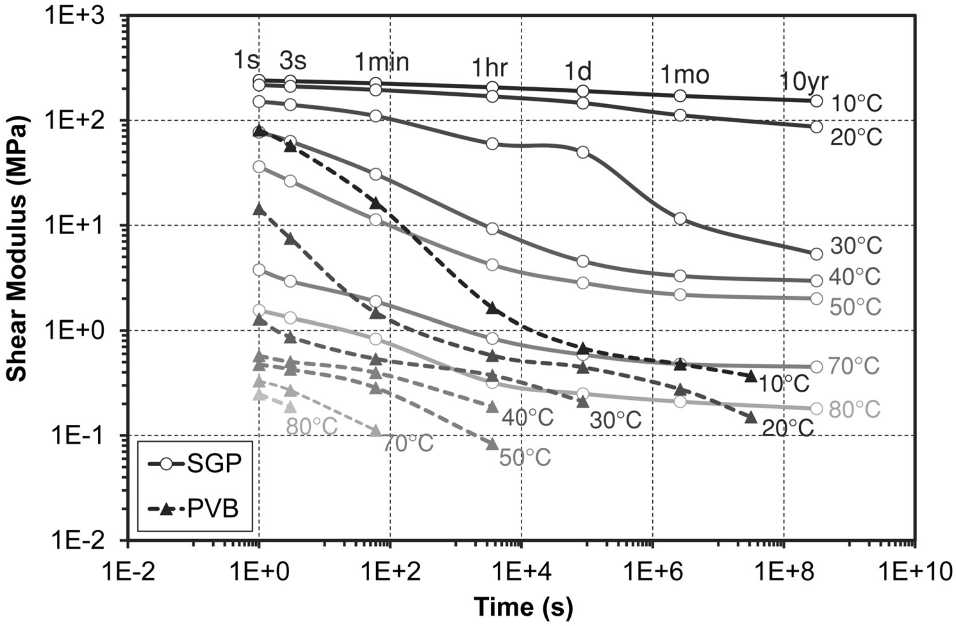
$$h_{eq,\delta} = \sqrt[3]{(1-\varpi)\sum_i h_i^{\,3} + \varpi\left(\sum_i h_i\right)^3}$$

Laminated glass equivalent thickness for bending stress:

$$h_{eq,\sigma} = \sqrt{\frac{\left(h_{eq,\delta}\right)^3}{\left(h_i + 2\varpi h_{m,i}\right)}}$$



G(t) of PVB and SGP interlayers:



Glass design strength:

$$f_{gd} = \frac{k_{\text{mod}}\, k_A\, f_{gk}}{\gamma_{mA}} + \frac{f_{rk}}{\gamma_{mV}}$$

Stress-history (load duration) interaction equation:

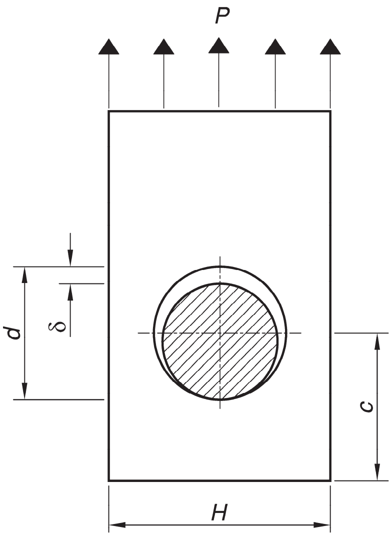
$$\frac{\sigma_{1,S}}{f_{gd,S}} + \frac{\sigma_{1,M}}{f_{gd,M}} + \frac{\sigma_{1,L}}{f_{gd,L}} \leq 1$$

Empirical stress concentration for bolted connections:

$$K_t = 1.5 + 1.25\left(\frac{H}{d} - 1\right) - 0.0675\left(\frac{H}{d} - 1\right)^2$$

where

$$K_t = \frac{\sigma_{\max}(H-d)t}{P}$$



3D3 – Structural Materials and Design – Concrete Datasheet (pg 1 of 2)

Table 1.1 Span versus depth ratio

Structural system	Span/effective depth ratio	
	EC2*	
	high	light
1. Simply supported beam, one-way or two-way spanning simply supported slab	14	20
2. End span of continuous beam or one-way continuous slab or two-way spanning slab continuous over one long side	18	26
3. Interior span of beam or one-way or two-way spanning slab	20	30
4. Slab supported on columns without beams (flat slab), based on longer span	17	24
5. Cantilever	6	8

highly stressed $\rho = 1.5\%$ and lightly stressed $\rho = 0.5\%$ (slabs are normally assumed to be lightly stressed) *Table 7.4N, NA.5 [1.2]

Table 1.2 Minimum member sizes and cover (to main reinforcement) for initial design of continuous members

Member	Fire resistance	Minimum dimension, mm		
		4 hours	2 hours	1 hour
Columns fully exposed to fire	width	450	300	200
Beams	width cover	240 70	200 50	200 45
Slabs with plain soffit	thickness cover	170 45	125 35	100 35

Extracts from Table 4.1 [1.1]

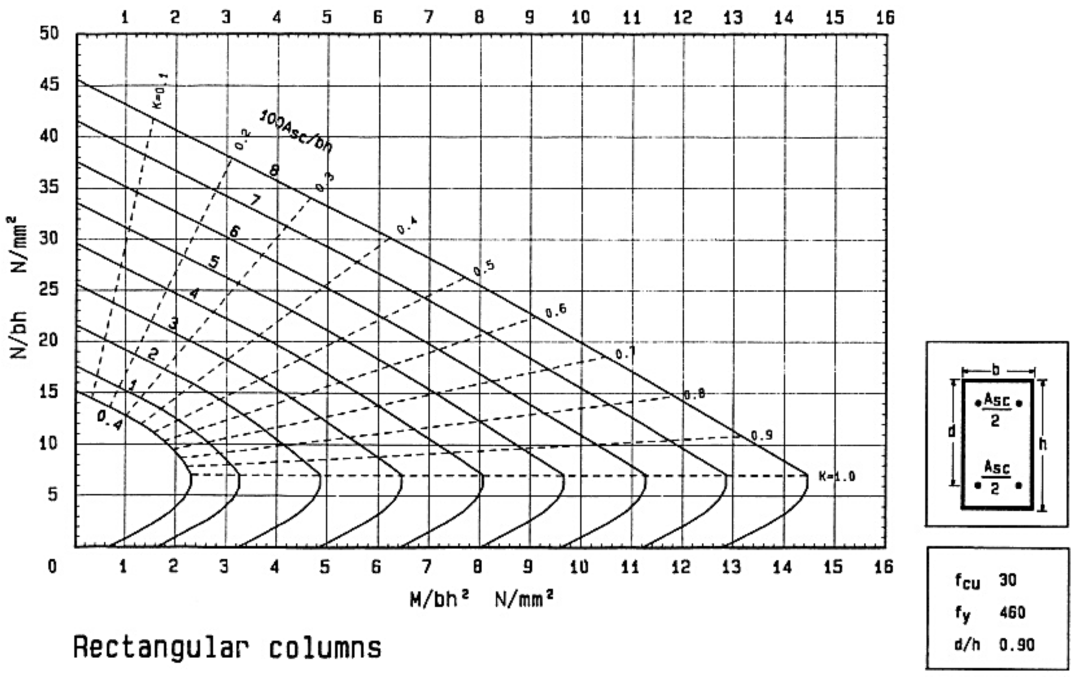


Fig 1.1 Interaction diagram from [1.3]

[1.1] Manual for the design of reinforced concrete building structures to EC2, IStructE, ICE, March 2000 - FM 507
[1.2] Eurocode 2: Design of concrete structures, EN 1992-1-1:2004, UK National Annex –NA to BS EN 1992-1-1:2004
[1.3] Structural design. Extracts from British Standards for Students of Structural design. PP7312:2002, BSi

3D3 – Structural Materials and Design – Concrete Datasheet (pg 2 of 2)

Flexure

Under-reinforced – singly reinforced

$$M_u = \frac{A_s f_y d(1-0.5x/d)}{\gamma_s}$$

$$\frac{x}{d} = \frac{\gamma_c A_s f_y}{\gamma_s 0.6 f_{cu} bd}$$

if x/d =0.5

$$M_u = 0.225 f_{cu} bd^2 / \gamma_c$$

Balanced section

$$\rho_b = \frac{A_s}{bd} = \frac{\gamma_s 0.6 f_{cu}}{\gamma_c f_y} \cdot \frac{\varepsilon_{cu}}{\varepsilon_y + \varepsilon_{cu}}$$

Shear

Without internal stirrups

$$V_{Rd,c} = [\frac{0.18}{\gamma_c} k(100\rho_1 f_{ck})^{1/3}] b_w d \geq (0.035 k^{3/2} f_{ck}^{1/2}) b_w d$$

where: f_{ck} is the characteristic concrete compressive cylinder strength (MPa).

$k = 1 + \sqrt{200/d} \leq 2.0$ (d in mm)

$\rho_l = A_s/b_w d \leq 0.02$

With internal stirrups

- Concrete resistance

$$V_{Rd,\max} = f_{c,\max}(b_w 0.9d)/(\cot\theta + \tan\theta)$$

where: $f_{c,\max} = 0.6(1 - f_{ck}/250) f_{cd}$

- Shear stirrup resistance

$$V_{Rd,s} = A_{sw} f_y (0.9d)(\cot\theta)/(s\gamma_s)$$

Columns – axial loading only

$$\sigma_u = 0.6\frac{f_{cu}}{\gamma_c} + \rho_c \frac{f_y}{\gamma_s}$$

Standard steel diameters (in mm)

6, 8, 10, 12, 16, 20, 25, 32 and 40