EGT2
ENGINEERING TRIPOS PART IIA

Thursday 7 May 2026   9.30 to 11.10

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.*

STATIONERY REQUIREMENTS
Single-sided script paper

SPECIAL REQUIREMENTS TO BE SUPPLIED FOR THIS EXAM
CUED approved calculator allowed
Attachment: 3D3 Structural Materials and Design data sheet (18 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.

You may not remove any stationery from the Examination Room.

1 An incomplete design for a steel framed building with concrete decking is shown in Fig. 1.

(a) With the aid of sketches where appropriate, devise a suitable structural scheme to provide overall stability to the structure. Describe the principal load paths for the complete structure for both horizontal and vertical loading. Explain any assumptions made. [30%]

(b) A colleague comments that the design might be vulnerable to disproportionate collapse. What do they mean and how might their concerns be addressed? [20%]

(c) Part of the roof will be covered by frameless glass panels spanning 4.5 m simply supported between roof beams. You consider that a uniformly distributed characteristic medium term snow load of 0.5 kN m^{-2} is likely to govern and that self-weight of the glazing can be initially estimated as 0.5 kN m^{-2}. The partial safety factor for transient loads is 1.5 and the partial safety factor for permanent loads is 1.35. Fully toughened glass is to be used, having f_{rk} = 90 N mm^{-2} and f_{gk} = 45 N mm^{-2}; material partial safety factors γ_{mV} = 1.2 and γ_{mA} = 1.8; the load duration factor k_{mod} = 0.43 for medium term loads and 0.29 for long term loads; and the stressed area factor k_A = 1.0.

(i) If single ply glazing is to be used, what minimum thickness of glass would be required for adequate flexural resistance? [30%]

(ii) For reasons of post-fracture performance, you decide that a 2-ply laminated glazing solution with a PVB interlayer is preferable to a single ply. Both layers are to be of equal thickness and the thickness of the PVB interlayer can be assumed to be negligible. Determine the required thickness of the laminated glass for adequate flexural resistance. [10%]

(iii) Briefly explain the role of the interlayer in the post-fracture performance of laminated glass and any additional design considerations that this may introduce. [10%]

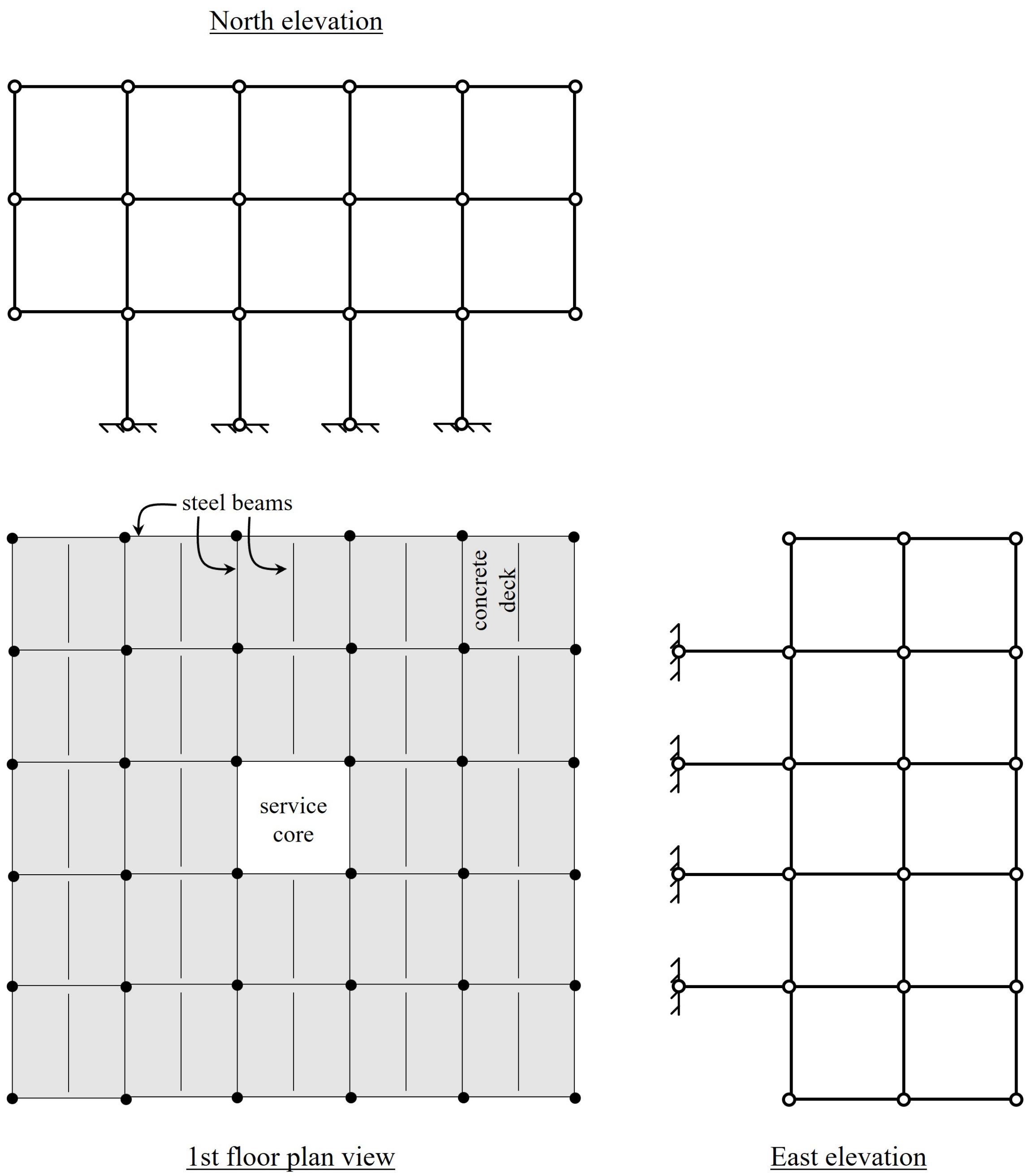
North elevation
steel beams
concrete deck
service core
1st floor plan view
East elevation

Fig. 1

2 A simply supported 9 m long reinforced concrete beam of prismatic rectangular cross section is shown in Fig. 2. Concrete cover requirements for durability are 35 mm. The beam is to achieve a fire resistance rating of 2 hours, requiring a minimum axis distance of 65 mm and a minimum breadth of 200 mm. The concrete is to have a characteristic compressive cube strength $f_{cu} = 40$ MPa and a compressive cylinder strength $f_{ck} = 32$ MPa. Steel reinforcement is to have a characteristic yield strength $f_y = 500$ MPa. The material partial safety factor for concrete $\gamma_c = 1.50$ and for reinforcing steel $\gamma_s = 1.15$. The partial safety factor for transient load is 1.50 and for permanent load is 1.35.

The beam is to be initially designed to support a uniformly distributed characteristic permanent load of 15 kN m^{-1} and a uniformly distributed characteristic transient load of 12 kN m^{-1} along the full length of the beam. The permanent load includes the self-weight of the beam.

(a) Determine a suitable effective depth and breadth for the beam. [10%]

(b) Design the required longitudinal reinforcement for the beam at the most critical location for bending. [35%]

(c) Determine whether transverse shear reinforcement is required at the most critical location for shear and design this reinforcement if required. [35%]

(d) Without further calculation, sketch an efficient reinforcement layout for the proposed reinforcement design. Clearly indicate the required height of the concrete cross-section to accommodate this layout. [10%]

(e) Suggest some approaches that a structural designer might take to reduce the embodied carbon of the beam. [10%]

characteristic permanent load = 15 kN m^{-1}
characteristic transient load = 12 kN m^{-1}

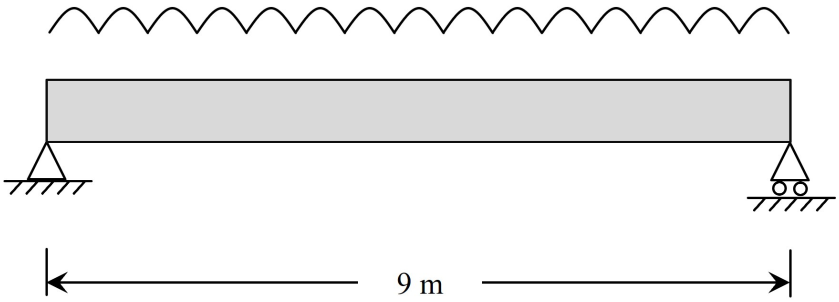

Fig. 2

3 A timber beam is to span simply supported across two long masonry walls as shown in Fig. 3. The beam is to be designed to resist a uniformly distributed permanent vertical load of 4 kN m^{-1} and a medium-term transient vertical load of 5 kN m^{-1}.

The timber beam is to have a breadth of 150 mm. The timber is to be grade C24 softwood. The beam can be considered as service class 1. The beam will be fully laterally restrained along its length. The material partial safety factor for solid timber is 1.3. The size effect factor k_h can be taken as 1.0, the cracking factor k_{cr} can be taken as 0.67, the load sharing factor k_{ls} can be taken as 1.0, and the bearing factor $k_{c,90}$ can be taken as 1.0.

The masonry walls are 150 mm thick and 3 m high. The masonry has a characteristic strength of $f_k = 2$ MPa and a material partial safety factor $\gamma_m = 3.0$.

(a) Determine a suitable depth for the beam based on deflection at the serviceability limit state (SLS). The deflection limit can be taken as span/360. For long term deflection, the creep factor k_{def} can be taken as 0.6 and the partial safety factor for the transient load can be taken as 0.3. [20%]

(b) At the ultimate limit state (ULS), the partial safety factor for permanent load is 1.35 and for transient load is 1.5.

(i) Determine whether the flexural resistance of the beam is adequate to resist the design loads at the critical section for bending. [20%]

(ii) Determine whether the shear resistance of the beam is adequate to resist the design loads at the critical section for shear. [10%]

(iii) Determine whether the bearing resistance of the beam is adequate to resist the design load at the supports. [10%]

(iv) Determine whether the bearing and buckling resistance of the masonry walls are adequate to resist the design load from the beam at the supports. [40%]

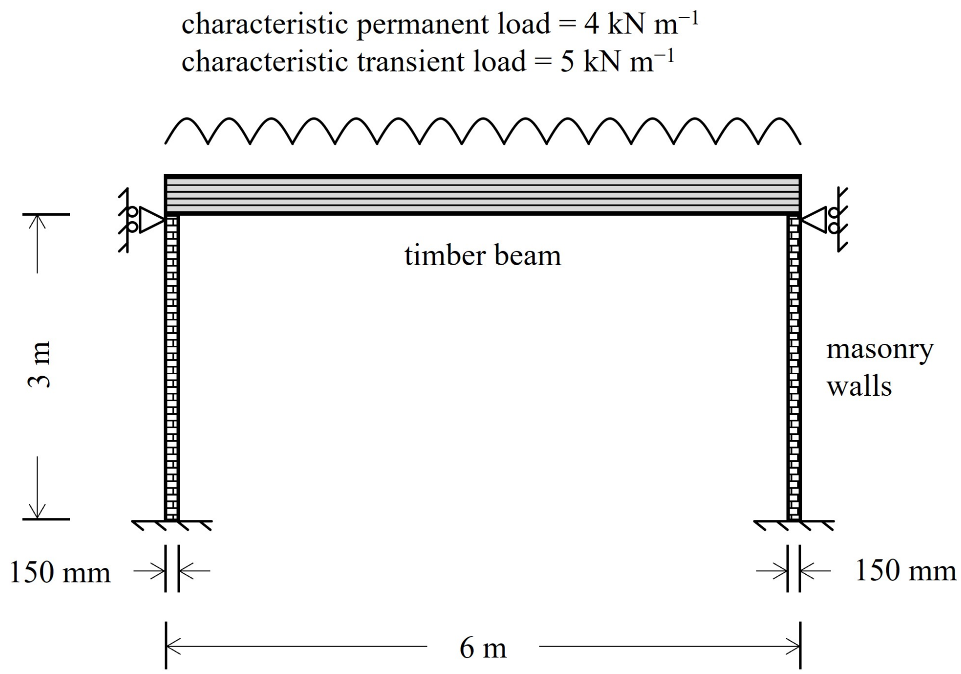
characteristic permanent load = 4 kN m^{-1}
characteristic transient load = 5 kN m^{-1}
timber beam
3 m
masonry
walls
150 mm
150 mm
6 m

Fig. 3

4 The steel bridge girder pictured in Fig. 4 is continuous over two 45 m long spans. The cross-section dimensions of the girder are shown in Fig. 5. Note that the vertical scale in Fig. 4 is exaggerated to highlight the locations of the vertical stiffeners, which are indicated in thick black line. Cross-bracing to adjacent girders is provided at 9 m spacing, connected to the vertical stiffeners. The girder supports a concrete bridge deck, which results in a characteristic load of 15 kN m^{-1} onto the girder. The characteristic traffic loading is 12.5 kN m^{-1}. Both loads can be considered uniformly distributed along the length of the girder. The grade of steel used is S355. The partial safety factor for permanent load is 1.35, and for transient load is 1.5. The partial safety factors for the resistances may be taken as 1.0.

(a) Check the girder at the ULS for bending at the location of the internal support. The torsion constant of the girder section $I_T = 2.89 \times 10^7$ mm^4, and the warping constant $I_w = 1.09 \times 10^{15}$ mm^6. [55%]

(b) Check the girder at the ULS for shear at the location of the internal support. [25%]

(c) Explain the specific purpose of the stiffeners indicated by a, b and c in Fig. 4. [20%]

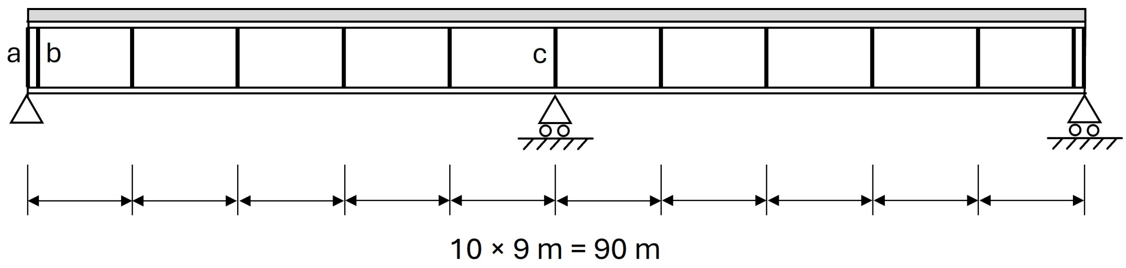

Fig. 4

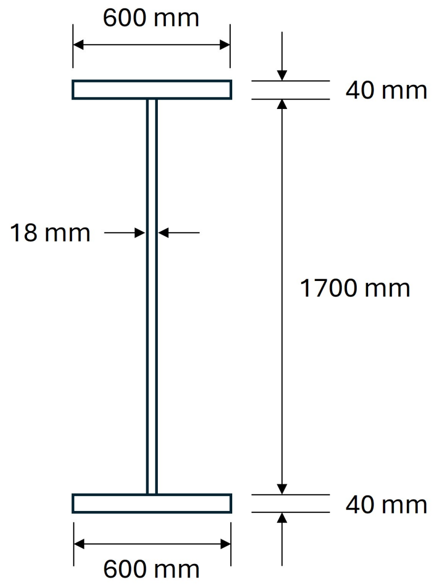

Fig. 5

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University of Cambridge
Department of Engineering

Engineering Tripos Part IIA

Module 3D3
Structural Materials & Design

Datasheets

Michaelmas 2025

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	·9^{2}0097	·9^{2}0358	·9^{2}0613	·9^{2}0863	·9^{2}1106	·9^{2}1344	·9^{2}1576
2·4	·9^{2}1802	·9^{2}2024	·9^{2}2240	·9^{2}2451	·9^{2}2656	·9^{2}2857	·9^{2}3053	·9^{2}3244	·9^{2}3431	·9^{2}3613
2·5	·9^{2}3790	·9^{2}3963	·9^{2}4132	·9^{2}4297	·9^{2}4457	·9^{2}4614	·9^{2}4766	·9^{2}4915	·9^{2}5060	·9^{2}5201
2·6	·9^{2}5339	·9^{2}5473	·9^{2}5604	·9^{2}5731	·9^{2}5855	·9^{2}5975	·9^{2}6093	·9^{2}6207	·9^{2}6319	·9^{2}6427
2·7	·9^{2}6533	·9^{2}6636	·9^{2}6736	·9^{2}6833	·9^{2}6928	·9^{2}7020	·9^{2}7110	·9^{2}7197	·9^{2}7282	·9^{2}7365
2·8	·9^{2}7445	·9^{2}7523	·9^{2}7599	·9^{2}7673	·9^{2}7744	·9^{2}7814	·9^{2}7882	·9^{2}7948	·9^{2}8012	·9^{2}8074
2·9	·9^{2}8134	·9^{2}8193	·9^{2}8250	·9^{2}8305	·9^{2}8359	·9^{2}8411	·9^{2}8462	·9^{2}8511	·9^{2}8559	·9^{2}8605
3·0	·9^{2}8650	·9^{2}8694	·9^{2}8736	·9^{2}8777	·9^{2}8817	·9^{2}8856	·9^{2}8893	·9^{2}8930	·9^{2}8965	·9^{2}8999
3·1	·9^{3}0324	·9^{3}0646	·9^{3}0957	·9^{3}1260	·9^{3}1553	·9^{3}1836	·9^{3}2112	·9^{3}2378	·9^{3}2636	·9^{3}2886
3·2	·9^{3}3129	·9^{3}3363	·9^{3}3590	·9^{3}3810	·9^{3}4024	·9^{3}4230	·9^{3}4429	·9^{3}4623	·9^{3}4810	·9^{3}4991
3·3	·9^{3}5166	·9^{3}5335	·9^{3}5499	·9^{3}5658	·9^{3}5811	·9^{3}5959	·9^{3}6103	·9^{3}6242	·9^{3}6376	·9^{3}6505
3·4	·9^{3}6631	·9^{3}6752	·9^{3}6869	·9^{3}6982	·9^{3}7091	·9^{3}7197	·9^{3}7299	·9^{3}7398	·9^{3}7493	·9^{3}7585
3·5	·9^{3}7674	·9^{3}7759	·9^{3}7842	·9^{3}7922	·9^{3}7999	·9^{3}8074	·9^{3}8146	·9^{3}8215	·9^{3}8282	·9^{3}8347
3·6	·9^{3}8409	·9^{3}8469	·9^{3}8527	·9^{3}8583	·9^{3}8637	·9^{3}8689	·9^{3}8739	·9^{3}8787	·9^{3}8834	·9^{3}8879
3·7	·9^{3}8922	·9^{3}8964	·9^{4}0039	·9^{4}0426	·9^{4}0799	·9^{4}1158	·9^{4}1504	·9^{4}1838	·9^{4}2159	·9^{4}2468
3·8	·9^{4}2765	·9^{4}3052	·9^{4}3327	·9^{4}3593	·9^{4}3848	·9^{4}4094	·9^{4}4331	·9^{4}4558	·9^{4}4777	·9^{4}4988
3·9	·9^{4}5190	·9^{4}5385	·9^{4}5573	·9^{4}5753	·9^{4}5926	·9^{4}6092	·9^{4}6253	·9^{4}6406	·9^{4}6554	·9^{4}6696
4·0	·9^{4}6833	·9^{4}6964	·9^{4}7090	·9^{4}7211	·9^{4}7327	·9^{4}7439	·9^{4}7546	·9^{4}7649	·9^{4}7748	·9^{4}7843
4·1	·9^{4}7934	·9^{4}8022	·9^{4}8106	·9^{4}8186	·9^{4}8263	·9^{4}8338	·9^{4}8409	·9^{4}8477	·9^{4}8542	·9^{4}8605
4·2	·9^{4}8665	·9^{4}8723	·9^{4}8778	·9^{4}8832	·9^{4}8882	·9^{4}8931	·9^{4}8978	·9^{5}0226	·9^{5}0655	·9^{5}1066
4·3	·9^{5}1460	·9^{5}1837	·9^{5}2199	·9^{5}2545	·9^{5}2876	·9^{5}3193	·9^{5}3497	·9^{5}3788	·9^{5}4066	·9^{5}4332
4·4	·9^{5}4587	·9^{5}4831	·9^{5}5065	·9^{5}5288	·9^{5}5502	·9^{5}5706	·9^{5}5902	·9^{5}6089	·9^{5}6268	·9^{5}6439
4·5	·9^{5}6602	·9^{5}6759	·9^{5}6908	·9^{5}7051	·9^{5}7187	·9^{5}7318	·9^{5}7442	·9^{5}7561	·9^{5}7675	·9^{5}7784
4·6	·9^{5}7888	·9^{5}7987	·9^{5}8081	·9^{5}8172	·9^{5}8258	·9^{5}8340	·9^{5}8419	·9^{5}8494	·9^{5}8566	·9^{5}8634
4·7	·9^{5}8699	·9^{5}8761	·9^{5}8821	·9^{5}8877	·9^{5}8931	·9^{5}8983	·9^{6}0320	·9^{6}0789	·9^{6}1235	·9^{6}1661
4·8	·9^{6}2067	·9^{6}2453	·9^{6}2822	·9^{6}3173	·9^{6}3508	·9^{6}3827	·9^{6}4131	·9^{6}4420	·9^{6}4696	·9^{6}4958
4·9	·9^{6}5208	·9^{6}5446	·9^{6}5673	·9^{6}5889	·9^{6}6094	·9^{6}6289	·9^{6}6475	·9^{6}6652	·9^{6}6821	·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:

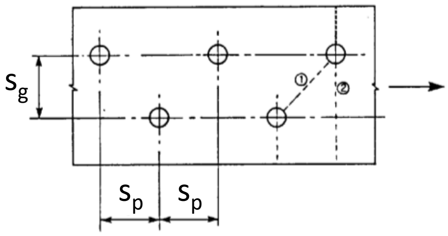
$$N_{pl,Rd} = \frac{A_g f_y}{\gamma_{M0}}$$

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

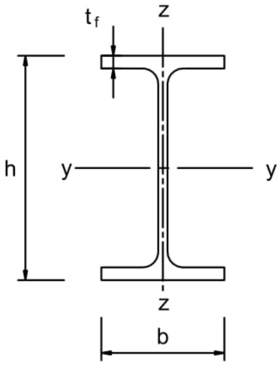
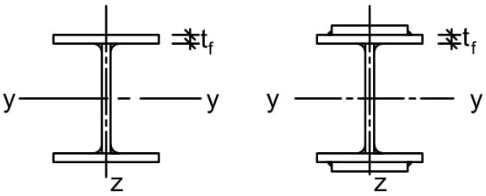
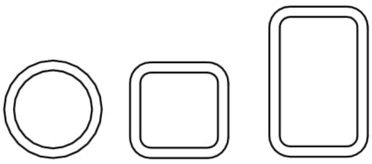
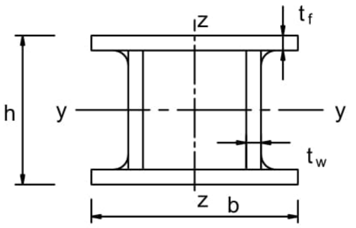
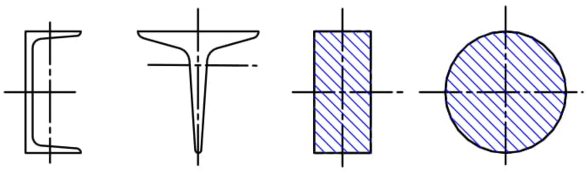
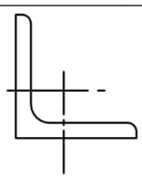
Reduction factor for shear lag in eccentrically connected angles:

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

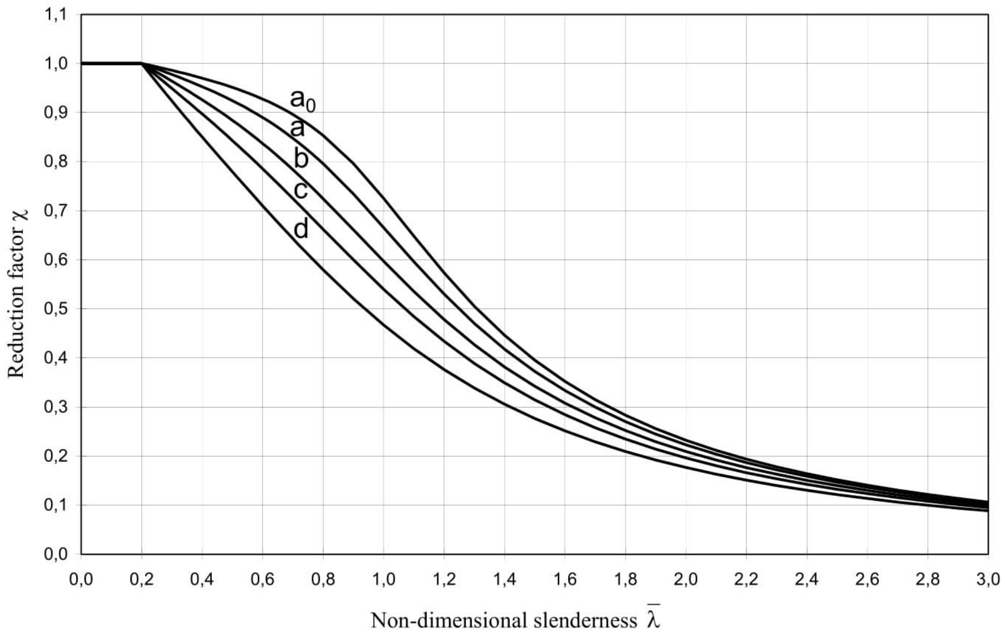


Figure 6.4: Buckling curves

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:

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

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

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

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

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

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

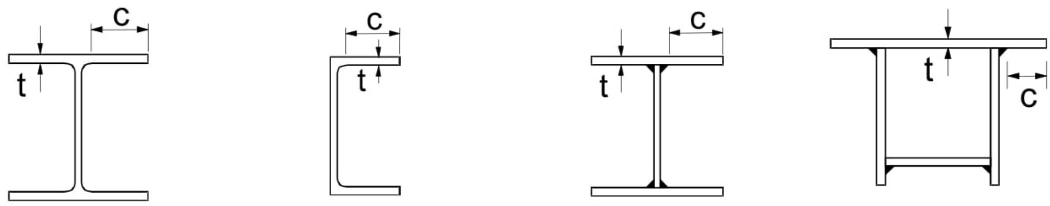
where b is the width of the plate and t is its thickness.

For plates in uniform longitudinal compression:

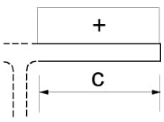
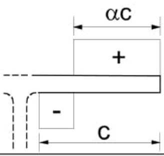
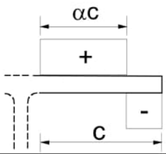
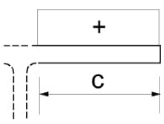
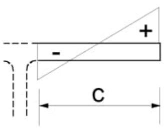
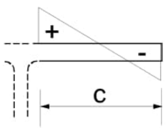
$K = 4$ for internal elements.

$K = 0.43$ for outstand elements.

Outstand flanges

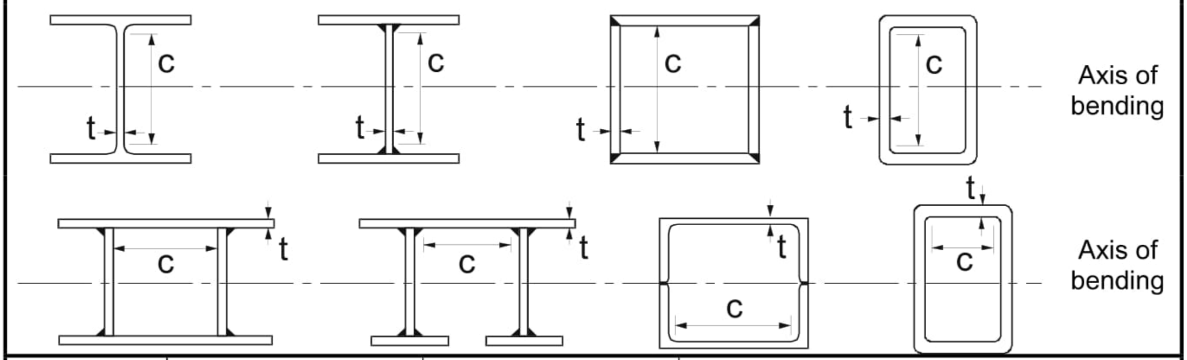


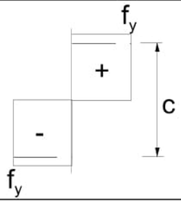
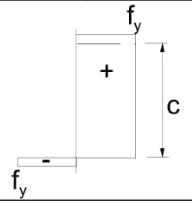
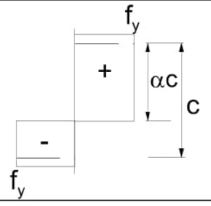
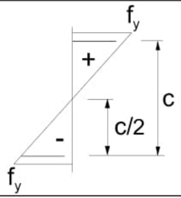
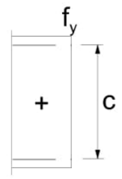
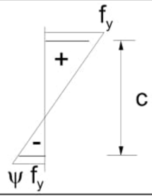
Rolled sections Welded sections

Class	Part subject to compression	Part subject to bending and compression: Tip in compression	Part subject to bending and compression: Tip in tension
Stress distribution in parts (compression positive)			
1	$c/t \le 9\varepsilon$	$c/t \le \dfrac{9\varepsilon}{\alpha}$	$c/t \le \dfrac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	$c/t \le 10\varepsilon$	$c/t \le \dfrac{10\varepsilon}{\alpha}$	$c/t \le \dfrac{10\varepsilon}{\alpha\sqrt{\alpha}}$
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	$c/t \le 72\varepsilon$	$c/t \le 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \dfrac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \dfrac{36\varepsilon}{\alpha}$
2	$c/t \le 83\varepsilon$	$c/t \le 38\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \dfrac{456\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \dfrac{41{,}5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)			
3	$c/t \le 124\varepsilon$	$c/t \le 42\varepsilon$	when $\psi > -1$: $c/t \le \dfrac{42\varepsilon}{0{,}67 + 0{,}33\psi}$ when $\psi \le -1$*): $c/t \le 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

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

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

Beams

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

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

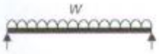
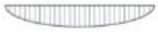
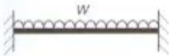
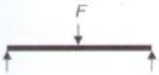
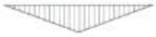
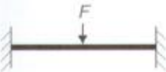
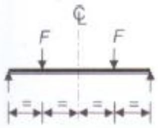
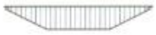
where:

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:

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

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

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

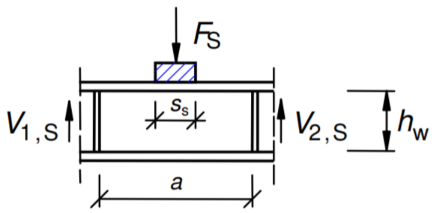
with: $\ell_e = \dfrac{k_F E t_w^2}{2 f_{yw} h_w} \le s_s + c$

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

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

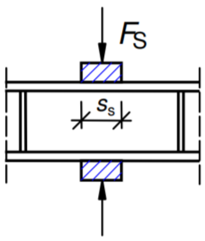
$$m_2 = 0 \quad if\ \bar{\lambda}_F \le 0{,}5$$

(IOF)



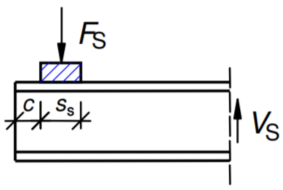
$$k_F = 6 + 2\left(\frac{h_w}{a}\right)^2$$

(ITF)



$$k_F = 3{,}5 + 2\left(\frac{h_w}{a}\right)^2$$

(EOF)



$$k_F = 2 + 6\left(\frac{s_s + c}{h_w}\right) \le 6$$

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 $\|\|$	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 $\|\|$	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 $\|\|$	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	2	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

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

Bearing – Design bearing stress

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

<u>Stability</u> – 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)"

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

Extract from [11.2] - k_{crit}

<u>Joints</u>

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/mm^2

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

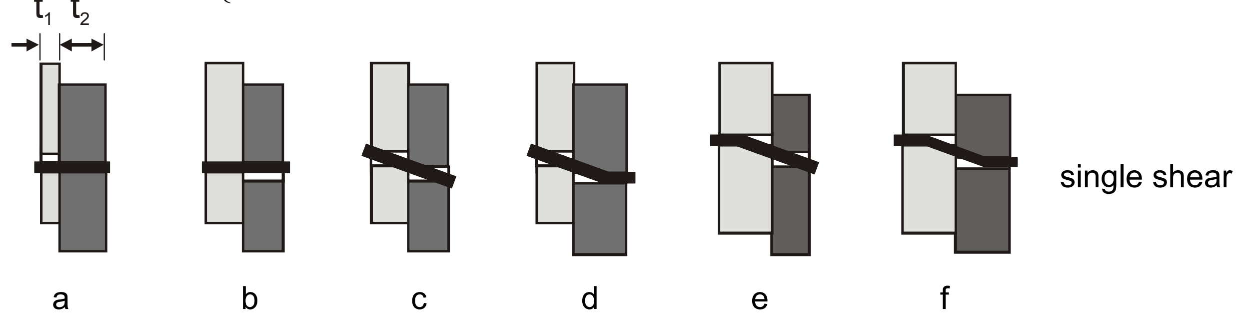
$$f_{h,\alpha,k} = \frac{f_{h,0,k}}{k_{90}\sin^2\alpha + \cos^2\alpha} \qquad \begin{array}{ll} \text{for softwood} & k_{90} = 1.35 + 0.015d \\ \text{for hardwood} & k_{90} = 0.90 + 0.015d \end{array}$$

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

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



Extract from [11.2] – Timber-to-timber and panel-to-timber joints

3D3 – Structural Materials and Design – Masonry Datasheet

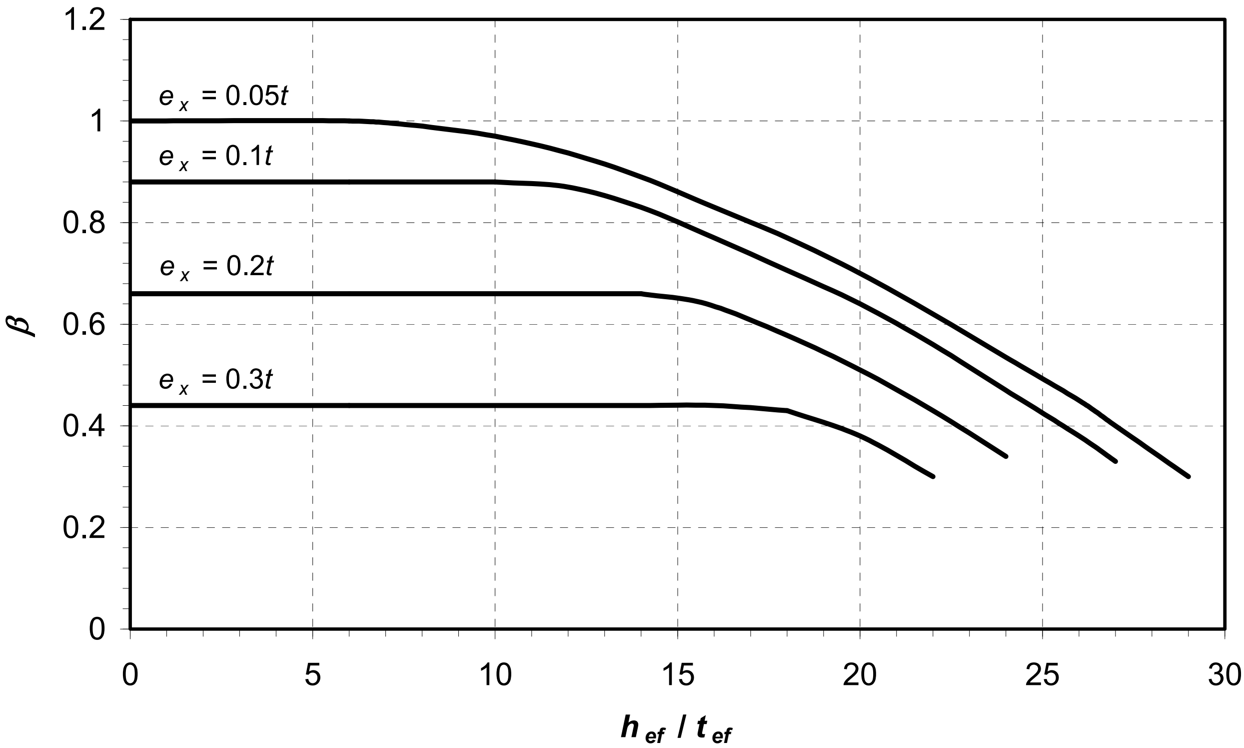
Bearing or crushing resistance per unit length

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

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

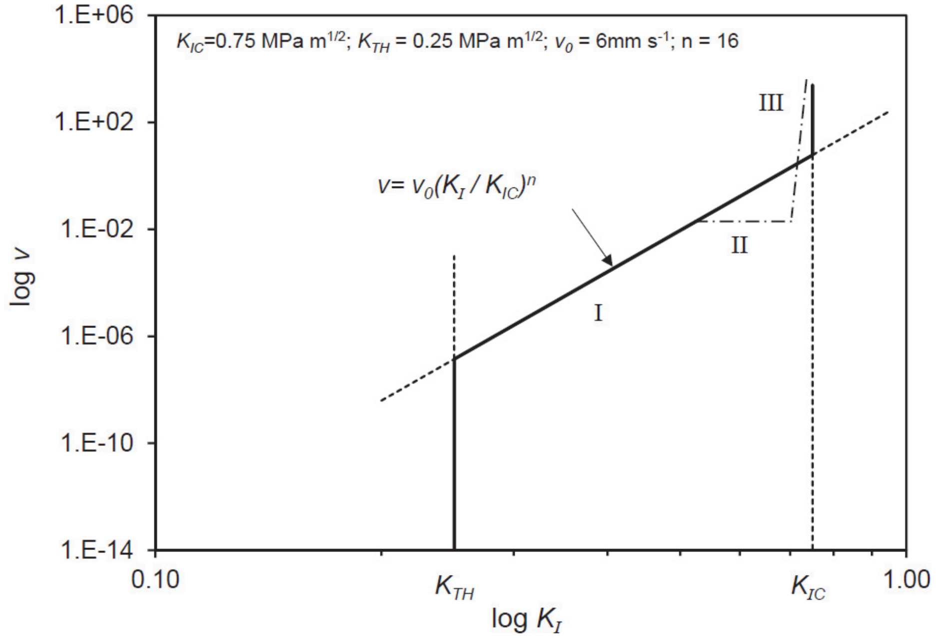
$$M = \frac{f_{kx} Z}{\gamma_m}$$

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):

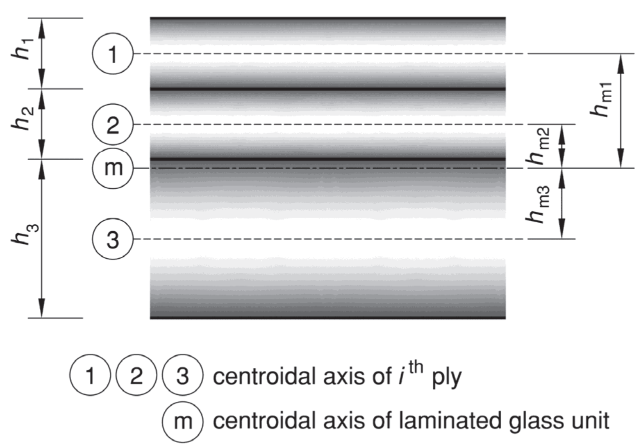
$$\frac{\sigma_f}{\sigma_{t0}} = \left(\frac{t_0}{t_f}\right)^{1/n} = k_{\mathrm{mod}}$$

Laminated glass equivalent thickness for bending deflection:

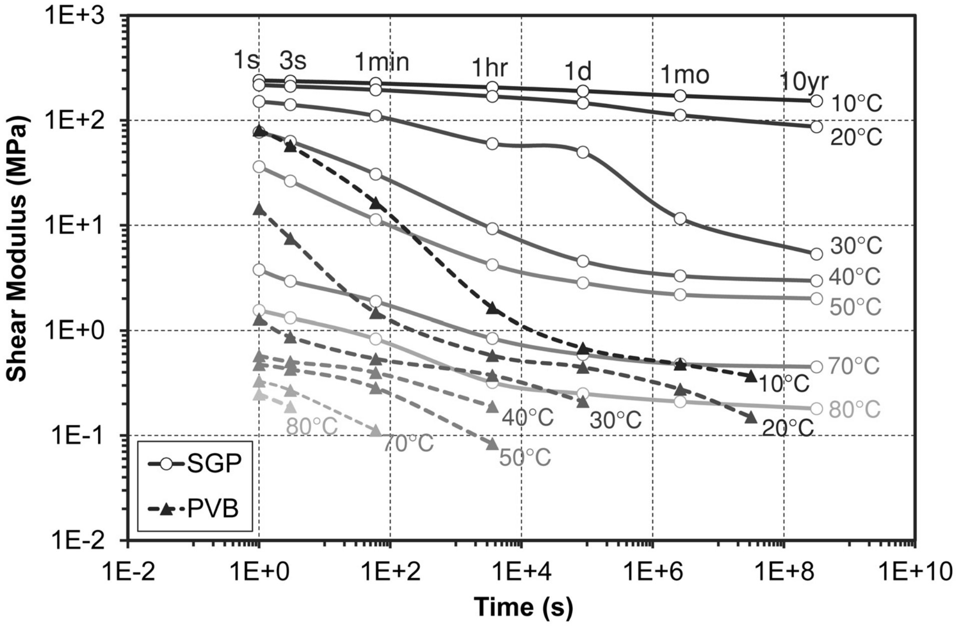
$$h_{eq,\delta} = \sqrt[3]{(1-\varpi)\sum_i h_i^{\,3} + \varpi\left(\sum_i h_i\right)^3}$$

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:

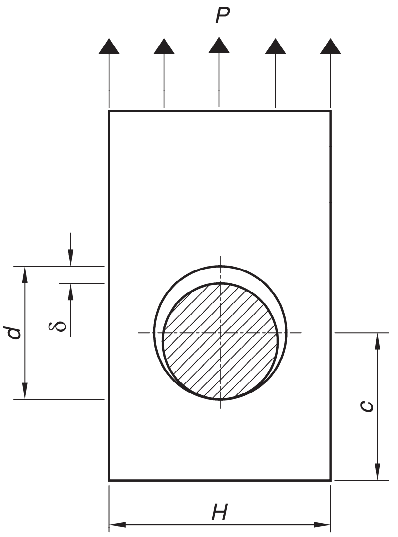
$$\frac{\sigma_{1,S}}{f_{gd,S}} + \frac{\sigma_{1,M}}{f_{gd,M}} + \frac{\sigma_{1,L}}{f_{gd,L}} \le 1$$

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

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



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]

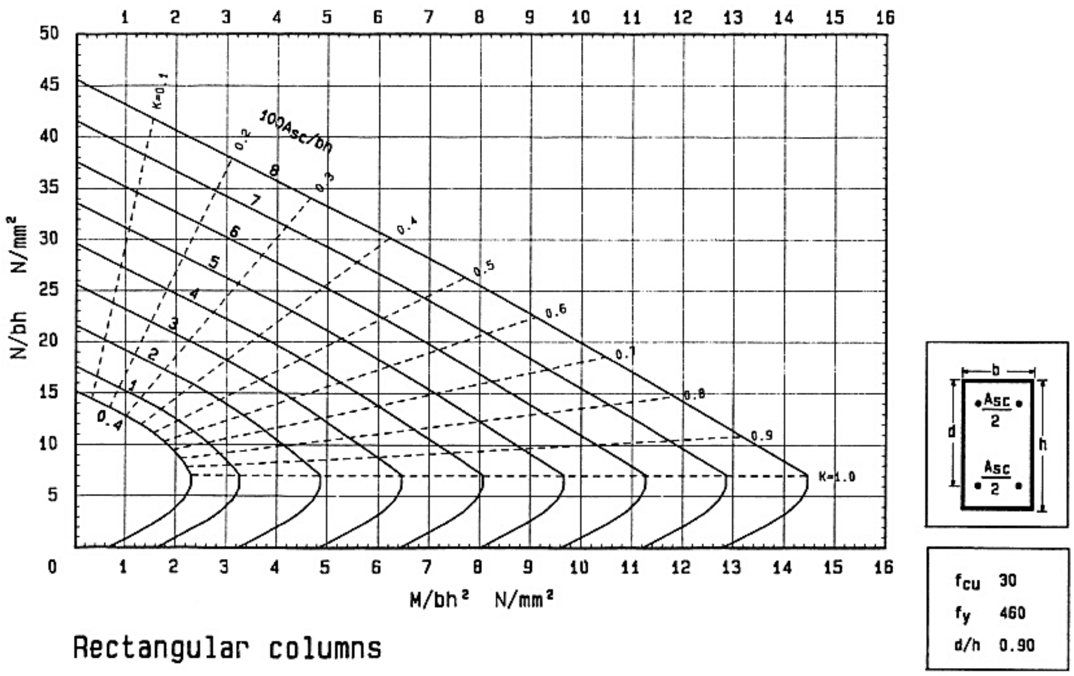


Fig 1.1 Interaction diagram from [1.3]

[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

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

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

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