

EGT2
ENGINEERING TRIPOS PART IIA

Thursday 8 May 2025 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 The initial structural scheme for a small multi-storey office building is shown in Fig. 1. Ground floor loads are supported directly by a ground bearing slab. The above ground floor structure comprises an arrangement of steel beams and one-way spanning reinforced concrete slabs, as shown in Fig. 1(a). A structural core is formed by diagonal tension-only bracing at the centre of the building, as shown in Fig. 1(b).

For the purposes of preliminary design, the structure is assumed to support a uniformly distributed characteristic permanent vertical load of $g \text{ kN m}^{-2}$ and a uniformly distributed characteristic transient vertical load of $q \text{ kN m}^{-2}$ over the plan area of the building at all floor levels, including the roof. The structure is also assumed to resist a uniformly distributed characteristic transient lateral wind pressure of $p \text{ kN m}^{-2}$ on the facade. The wind pressure can be assumed to act perpendicularly to the facade and to act on only one face at a time.

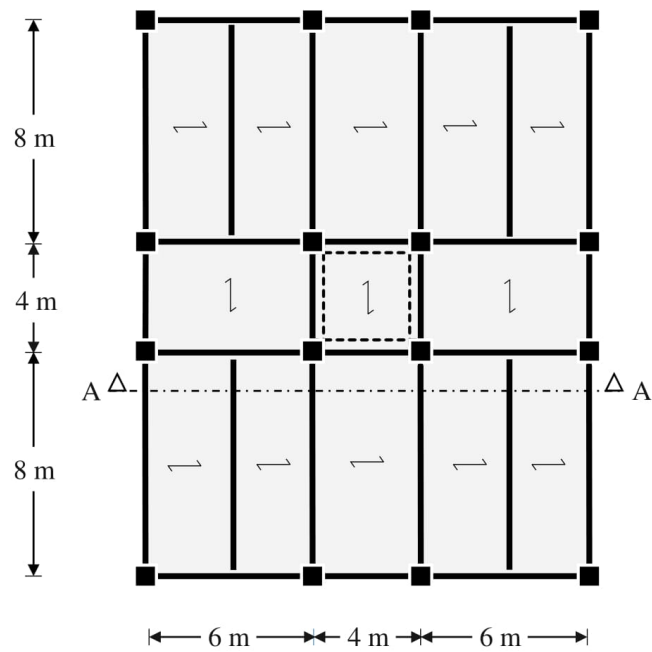
- (a) With the aid of sketches as appropriate, provide a qualitative description of the main load paths through the structure considering both vertical and lateral loads. Clearly state any assumptions made. [20%]

- (b) Initial design for the ultimate limit state is to be based on the load combination:

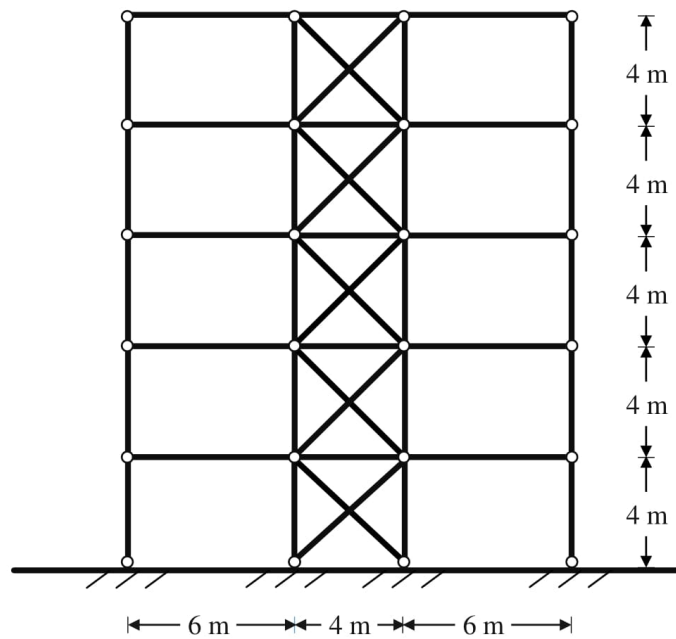
$$ULS = 1.35g + 1.5q + 0.75p$$

With the aid of sketches as appropriate and clearly stating any assumptions:

- (i) Identify the most onerously loaded perimeter columns and determine the force for which they should be designed. [20%]
- (ii) Identify the most onerously loaded bracing members and determine the force for which they should be designed. [20%]
- (iii) Identify the most onerously loaded internal columns and determine the force for which they should be designed. [20%]
- (c) You have some concerns about torsion of the structure. Without significantly increasing the amount of bracing, suggest an alteration to the structural scheme that would improve the torsional resistance of the building. [10%]
- (d) The project architect is unhappy about the aesthetic impact of the diagonal bracing members. Suggest an alternative structural approach to resisting lateral loads in a steel framed structure that does not require diagonal bracing members. [10%]



plan view – structural arrangement floors 1-5
(a)



section A-A
(b)

Fig. 1

2 A prismatic concrete beam of rectangular cross-section spans across three supports as shown in Fig. 2. The beam cross-section is designed as 300 mm wide. 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 50 mm. The concrete is to have a characteristic compressive cube strength $f_{cu} = 50$ MPa and a characteristic compressive cylinder strength $f_{ck} = 40$ MPa. Steel reinforcement is to have a characteristic yield strength $f_y = 500$ MPa. The material partial safety factor for concrete is $\gamma_c = 1.50$ and for reinforcing steel is $\gamma_s = 1.15$. 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 transient load of 8 kNm^{-1} and a uniformly distributed characteristic permanent load of 16 kNm^{-1} along the full length of the beam. The permanent load includes the self-weight of the beam.

- (a) Sketch the shear force and bending moment diagrams for this load case, clearly identifying salient values and their locations. [20%]
- (b) Determine a suitable effective depth for the beam. [5%]
- (c) Design the required longitudinal reinforcement for the beam at the most critical location for bending. [25%]
- (d) Determine whether transverse shear reinforcement is required at the most critical location for shear and design this reinforcement if required. [30%]
- (e) 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%]
- (f) Why might a design carried out using this load case alone be inadequate? For which region of the beam is this inadequacy likely to be most critical? [10%]

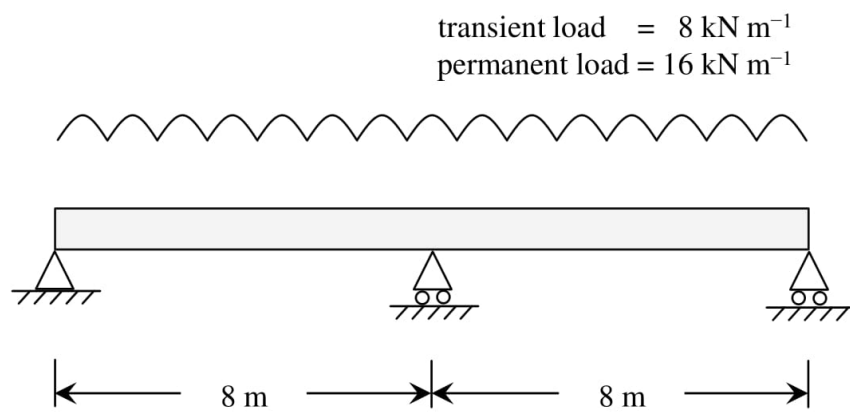


Fig. 2

3 A long series of three-pinned timber portals spaced at 4.5 m centre-to-centre are to be used to support a simple glass roof. The structural arrangement of a single portal is idealised in Fig. 3(a). The glazing that covers the portal spans simply supported between purlins spaced at 1.8 m centre-to-centre, as shown in Fig. 3(b).

All timber used in the design is to be C24 softwood and can be considered to be service class 1. The material partial safety factor for timber is $\gamma_m = 1.3$.

All glazing used in the design is 2-ply laminated fully toughened glass with characteristic surface prestress $f_{rk} = 90$ MPa and characteristic annealed strength $f_{gk} = 45$ MPa. Material partial safety factors are $\gamma_{mV} = 1.2$ and $\gamma_{mA} = 1.8$. The stressed area factor $k_A = 1.0$. The load duration factor for glass for long-term loads is $k_{mod,L} = 0.29$, for medium-term loads is $k_{mod,M} = 0.43$, and for short-term loads is $k_{mod,S} = 0.74$. The interlayer is made of polyvinyl butyral and is of negligible thickness. The interlayer provides a shear transfer coefficient of 0.2 under instantaneous loading. The interlayer provides negligible shear transfer under medium- and long-term loading.

The structural design must consider three uniformly distributed vertical loads acting on the roof: a permanent load of 0.5 kN m^{-2} assumed for the glazed roof; a medium-term snow load of 1.2 kN m^{-2} ; and an instantaneous wind load of 1.5 kN m^{-2} . The snow and wind loads may be assumed never to occur at the same time. The partial safety factor for transient load is 1.50 and for permanent load is 1.35.

- (a) If the cross-section of the horizontal beam of the portal is 200 mm wide, what is the minimum section depth required to provide adequate bending resistance at the ultimate limit state? The size effect factor is $k_h = 1$ and the load sharing factor is $k_{ls} = 1$. The beam can be assumed to be fully restrained by the purlins, such that $k_{crit} = 1$. The self-weight of the beam can be assumed to be 0.5 kN m^{-1} for the purposes of initial design. All loads on the beam resulting from the purlin reactions may be smeared into uniformly distributed loads. [50%]
- (b) The laminated glass is initially designed as 16 mm thick, comprising two 8 mm thick plies. What is the flexural utilisation ratio (i.e., the ratio of design resistance to design effect) for the glass as initially designed? [50%]

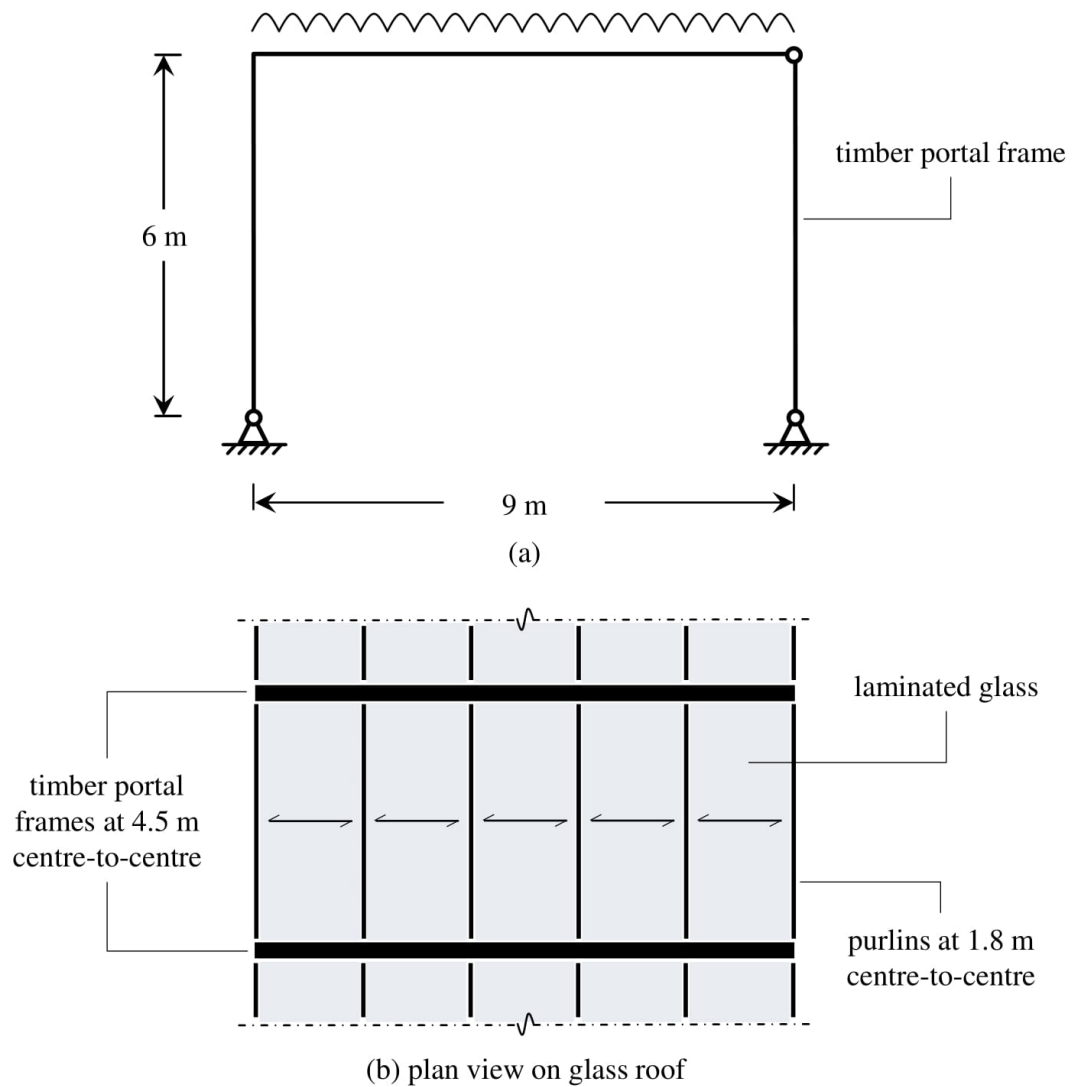


Fig. 3

4 A simply supported beam with a span length of 12 m carries two (factored) point loads of 1400 kN, as shown in Fig. 4(a). The steel grade of the beam is S355. The beam is welded together from a web plate with a depth of 1000 mm and two identical 550 mm wide flanges using 6 mm welds, as shown in Fig. 4(b).

The beam is prevented from moving in the out-of-plane direction at the supports and at the locations of the point loads. The self-weight of the beam may be neglected.

- (a) Determine the minimum thickness of the web to keep the web within Class 3. Determine the minimum thickness of the flange to keep the flange within Class 1. Round up both values to the nearest mm. [20%]
- (b) With the dimensions obtained in part (a), determine whether lateral-torsional buckling would occur under the given loading.

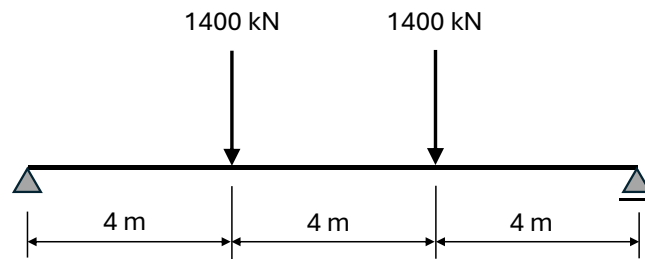
Note: The torsion constant I_T and the warping constant I_w of an I-section are given by:

$$I_T = \frac{2(t_f)^3 b + (t_w)^3 h}{3}$$

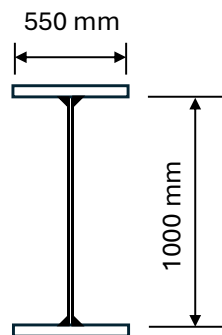
$$I_w = I_z \frac{(h + t_f)^2}{4}$$

where b is the flange width, h is the depth of the web (= 1000 mm), t_f is the flange thickness and t_w is the web thickness. I_z is the second moment of area about the minor axis. [40%]

- (c) Investigate whether shear stiffeners are needed in the beam. If so, schematically draw an elevation view of the beam showing the lay-out of the stiffeners. [40%]



(a)



(b)

Fig. 4

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University of Cambridge
Department of Engineering
Engineering Tripos Part IIA

Module 3D3
Structural Materials & Design

Datasheets
Michaelmas 2024

THE CUMULATIVE NORMAL DISTRIBUTION FUNCTION

$$\Phi(u) = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^u e^{-\frac{x^2}{2}} dx \quad \text{FOR } 0.00 \leq u \leq 4.99.$$

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	.9014
1.3	.9032	.9049	.9065	.9082	.9098	.9114	.9130	.9146	.9162	.9177
1.4	.9192	.9207	.9222	.9236	.9250	.9264	.9278	.9292	.9305	.9318
1.5	.9331	.9344	.9357	.9369	.9382	.9394	.9406	.9417	.9429	.9440
1.6	.9452	.9463	.9473	.9484	.9495	.9505	.9515	.9525	.9535	.9544
1.7	.9554	.9563	.9572	.9581	.9590	.9599	.9608	.9616	.9624	.9632
1.8	.9640	.9648	.9656	.9663	.9671	.9678	.9685	.9692	.9699	.9706
1.9	.9712	.9719	.9725	.9730	.9738	.9744	.9750	.9755	.9761	.9767
2.0	.9772	.9778	.9783	.9788	.9793	.9798	.9803	.9807	.9812	.9816
2.1	.9821	.9825	.9830	.9834	.9838	.9842	.9846	.9850	.9853	.9857
2.2	.9861	.9864	.9867	.9871	.9874	.9877	.9880	.9884	.9887	.9890
2.3	.9892	.9895	.9898	.9901	.9904	.9907	.9910	.9913	.9916	.9919
2.4	.9921	.9924	.9927	.9929	.9932	.9934	.9937	.9939	.9941	.9943
2.5	.9945	.9947	.9949	.9951	.9953	.9955	.9957	.9959	.9961	.9963
2.6	.9965	.9967	.9969	.9971	.9973	.9975	.9977	.9979	.9981	.9983
2.7	.9985	.9987	.9989	.9991	.9993	.9995	.9997	.9999	.9999	.9999
2.8	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999
2.9	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999
3.0	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999
3.1	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999
3.2	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999
3.3	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999
3.4	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999
3.5	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999
3.6	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999
3.7	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999
3.8	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999
3.9	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999
4.0	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999
4.1	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999
4.2	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999
4.3	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999
4.4	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999
4.5	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999
4.6	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999
4.7	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999
4.8	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999
4.9	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999	.9999

Example: $\Phi(3.57) = .98215 = 0.9998215$.

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		$40 \text{ mm} < t \leq 80$ 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 :

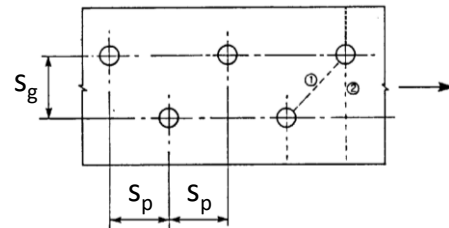
$$N_{u,Rd} = \frac{0.9 A_n f_u}{\gamma_{M2}}$$

Staggered bolt holes:

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

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:

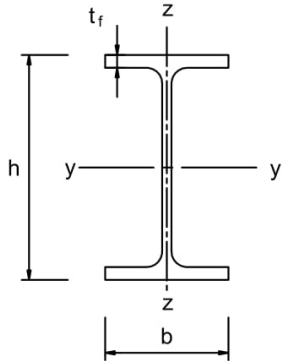
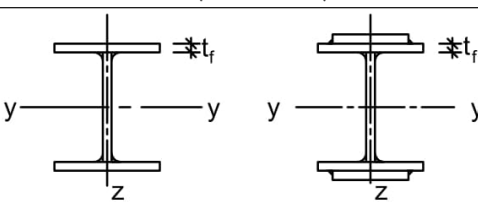
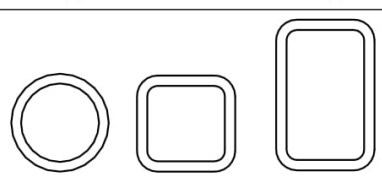
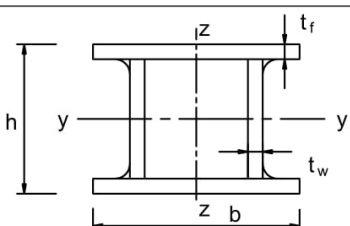
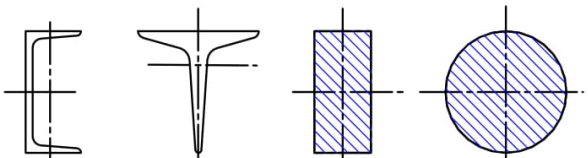
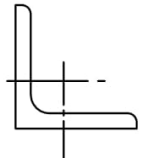
Pitch	p_1	$\leq 2,5 d_0$	$\geq 5,0 d_0$
2 bolts	β_2	0,4	0,7
3 bolts or more	β_3	0,5	0,7

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	S 460
Rolled sections		$h/b > 1,2$	$t_f \leq 40 \text{ mm}$	y-y z-z	a a ₀
			$40 \text{ mm} < t_f \leq 100$	y-y z-z	b a
		$h/b \leq 1,2$	$t_f \leq 100 \text{ mm}$	y-y z-z	b a
			$t_f > 100 \text{ mm}$	y-y z-z	d c
Welded I-sections		$t_f \leq 40 \text{ mm}$		y-y z-z	b c
		$t_f > 40 \text{ mm}$		y-y z-z	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

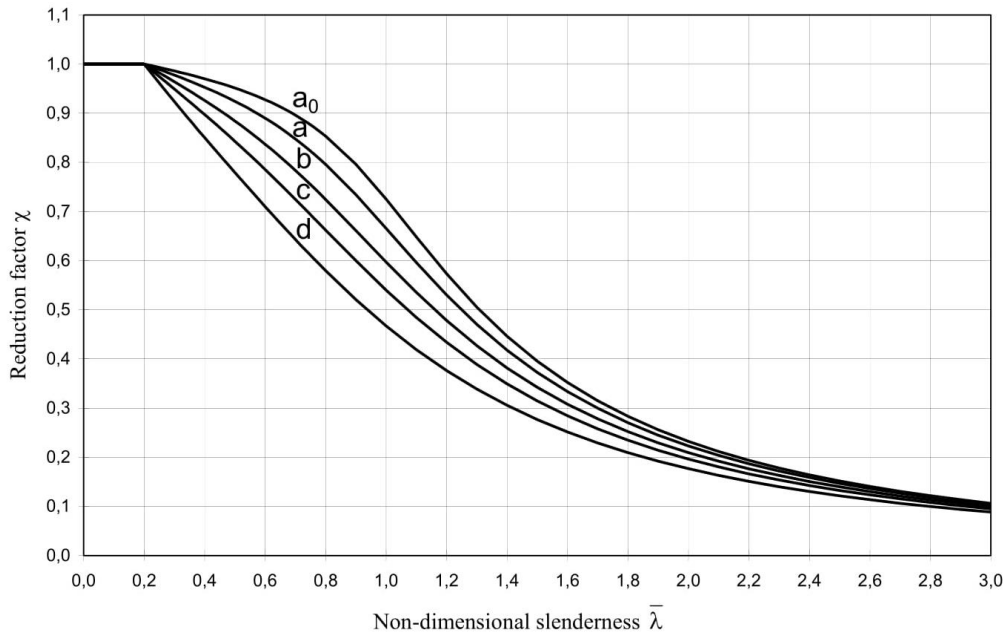


Figure 6.4: Buckling curves

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:

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

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

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

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

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

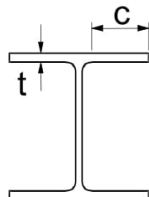
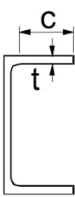
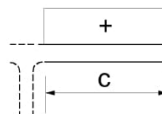
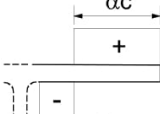
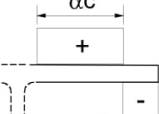
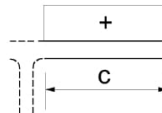
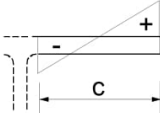
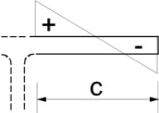
$$\sigma_{cr} = K \frac{\pi^2 E}{12(1 - \nu^2)} \left(\frac{t}{b} \right)^2$$

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		Tip in tension		
Stress distribution in parts (compression positive)						
1	$c/t \leq 9\varepsilon$	$c/t \leq \frac{9\varepsilon}{\alpha}$		$c/t \leq \frac{9\varepsilon}{\alpha\sqrt{\alpha}}$		
2	$c/t \leq 10\varepsilon$	$c/t \leq \frac{10\varepsilon}{\alpha}$		$c/t \leq \frac{10\varepsilon}{\alpha\sqrt{\alpha}}$		
Stress distribution in parts (compression positive)						
3	$c/t \leq 14\varepsilon$	$c/t \leq 21\varepsilon\sqrt{k_\sigma}$				
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 $\alpha > 0,5$: $c/t \leq \frac{396\varepsilon}{13\alpha - 1}$ when $\alpha \leq 0,5$: $c/t \leq \frac{36\varepsilon}{\alpha}$			
2	$c/t \leq 83\varepsilon$	$c/t \leq 38\varepsilon$	when $\alpha > 0,5$: $c/t \leq \frac{456\varepsilon}{13\alpha - 1}$ when $\alpha \leq 0,5$: $c/t \leq \frac{41,5\varepsilon}{\alpha}$			
Stress distribution in parts (compression positive)						
3	$c/t \leq 124\varepsilon$	$c/t \leq 42\varepsilon$	when $\psi > -1$: $c/t \leq \frac{42\varepsilon}{0,67 + 0,33\psi}$ 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:

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

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

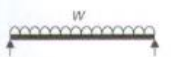
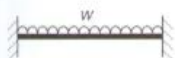
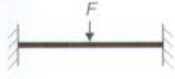
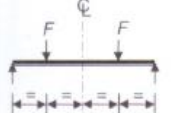
G = shear modulus

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:

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

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

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	$h/b \leq 2$	a
	$h/b > 2$	b
Welded I-sections	$h/b \leq 2$	c
	$h/b > 2$	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:

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

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

Shear

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

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

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:

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

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

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

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

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

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

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
$0,83/\eta \leq \bar{\lambda}_w < 1,08$	$0,83/\bar{\lambda}_w$	$0,83/\bar{\lambda}_w$
$\bar{\lambda}_w \geq 1,08$	$1,37/(0,7 + \bar{\lambda}_w)$	$0,83/\bar{\lambda}_w$

Web crippling:

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

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

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

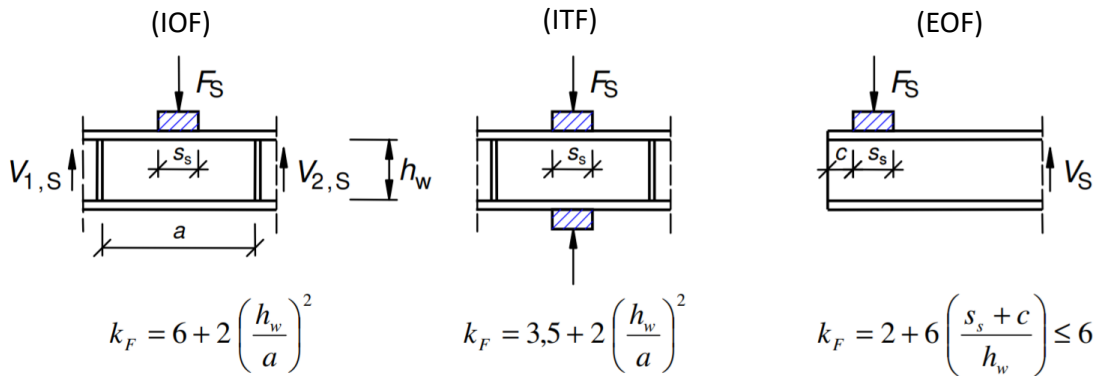
EOF: $\min \begin{cases} \ell_y = \ell_e + t_f \sqrt{\frac{m_1}{2} + \left(\frac{\ell_e}{t_f}\right)^2} + m_2 \\ \ell_y = \ell_e + t_f \sqrt{m_1 + m_2} \end{cases}$

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

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

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

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



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 ⊥	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 ⊥	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 ⊥	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

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

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)”

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

Extract from [11.2] - k_{crit}

Joints

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

$$f_{h,0,k} = 0.082(1 - 0.01d)\rho_k \text{ N/mm}^2$$

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

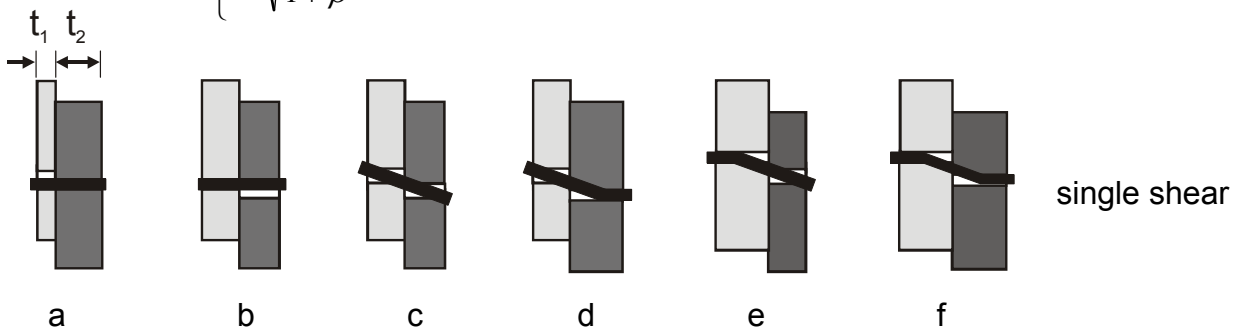
$$f_{h,\alpha,k} = \frac{f_{h,0,k}}{k_{90} \sin^2 \alpha + \cos^2 \alpha} \quad \begin{matrix} \text{for softwood} & k_{90} = 1.35 + 0.015d \\ \text{for hardwood} & k_{90} = 0.90 + 0.015d \end{matrix}$$

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

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

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



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

3D3 – Structural Materials and Design – Masonry Datasheet

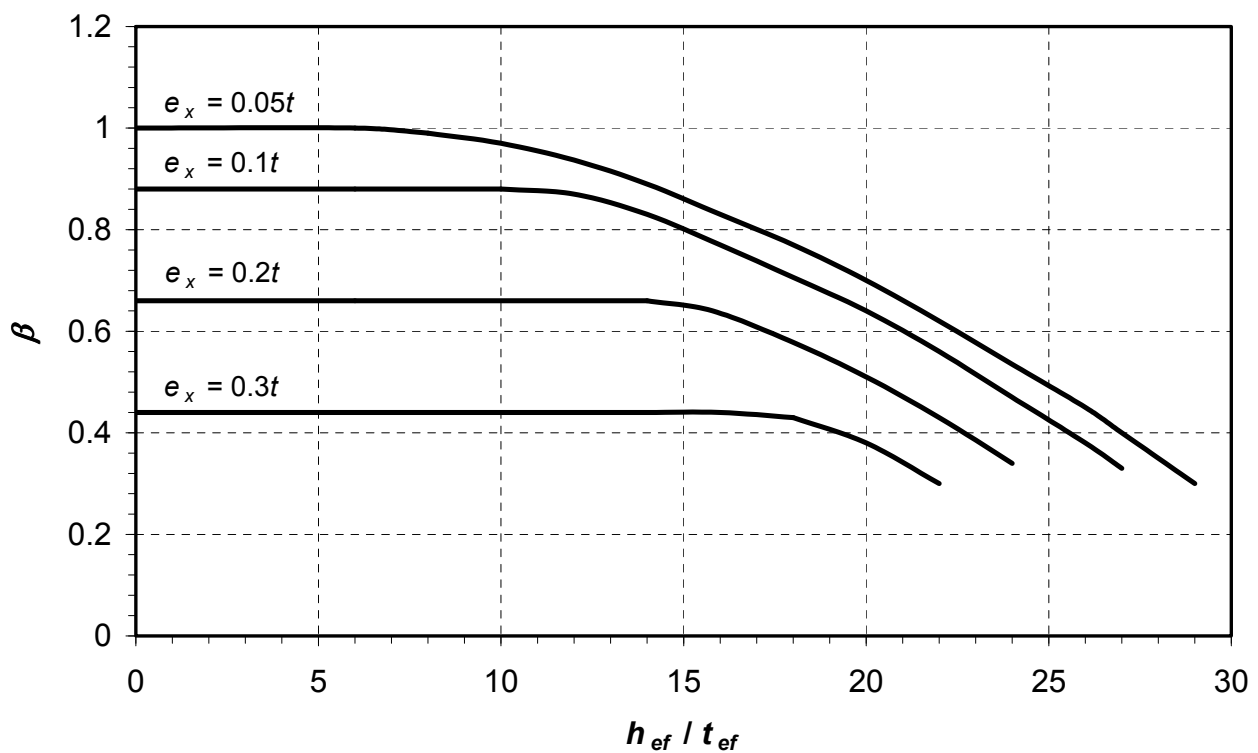
Bearing or crushing resistance per unit length

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

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

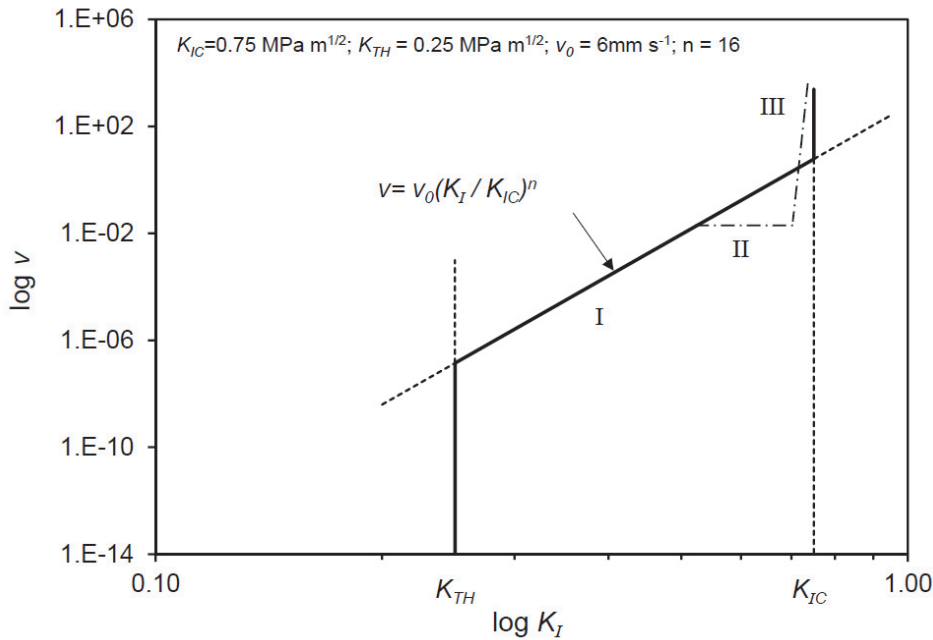
$$M = \frac{f_{kx} Z}{\gamma_m}$$

3D3 – Structural Materials and Design – Glass Datasheet

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

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

Idealised v–K relationship:



2-parameter Weibull distribution:

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

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):

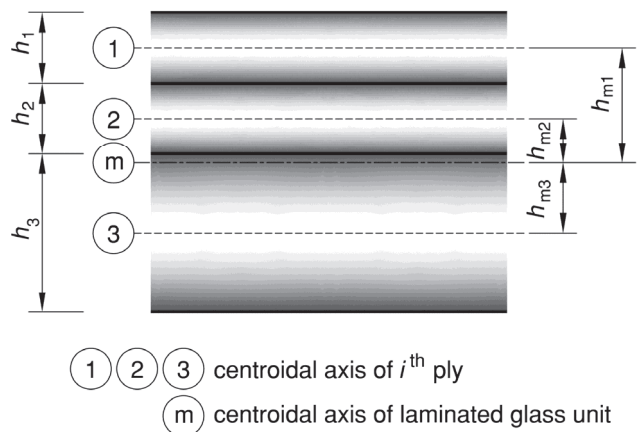
$$\frac{\sigma_f}{\sigma_{t0}} = \left(\frac{t_0}{t_f} \right)^{1/n} = k_{\text{mod}}$$

Laminated glass equivalent thickness for bending deflection:

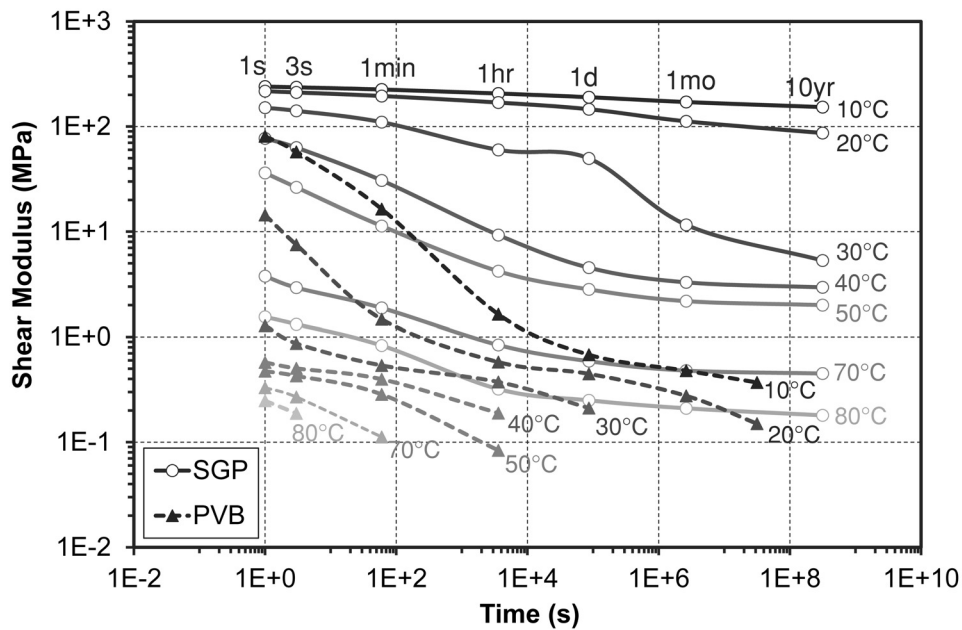
$$h_{eq, \delta} = \sqrt[3]{(1-\varpi) \sum_i h_i^3 + \varpi \left(\sum_i h_i \right)^3}$$

Laminated glass equivalent thickness for bending stress:

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



$G(t)$ 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:

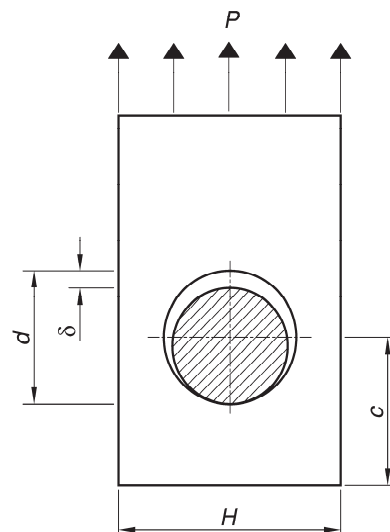
$$\frac{\sigma_{1,S}}{f_{gd,S}} + \frac{\sigma_{1,M}}{f_{gd,M}} + \frac{\sigma_{1,L}}{f_{gd,L}} \leq 1$$

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

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



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	240	200	200
	cover	70	50	45
Slabs with plain soffit	thickness	170	125	100
	cover	45	35	35

Extracts from Table 4.1 [1.1]

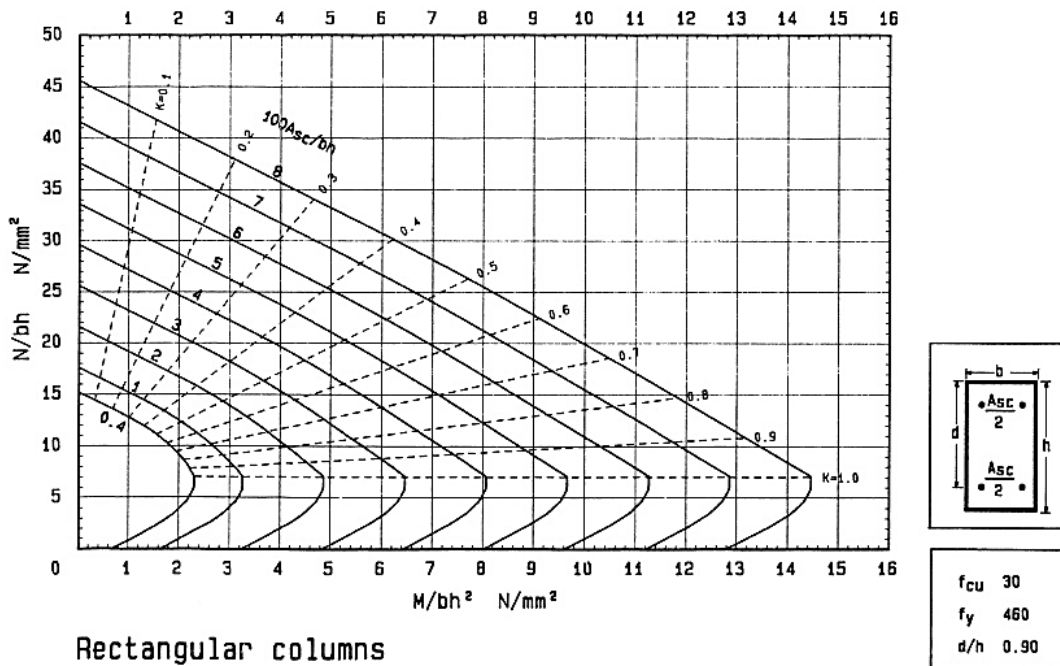


Fig 1.1 Interaction diagram from [1.3]

[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

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

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

if $x/d = 0.5$

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

Balanced section

$$\rho_b = \frac{A_s}{b d} = \frac{\gamma_s 0.6 f_{cu}}{\gamma_c f_y} \cdot \frac{\epsilon_{cu}}{\epsilon_y + \epsilon_{cu}}$$

Shear

Without internal stirrups

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

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

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

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

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