EGT2
ENGINEERING TRIPOS PART IIA

Tuesday 2 May 2023 2 to 3.40

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 light, pin-jointed steel frame constructed from circular hollow sections is shown in Fig. 1. The frame is to be clad in lightweight panels. The structure will be used to provide shelter for an archeological excavation below.

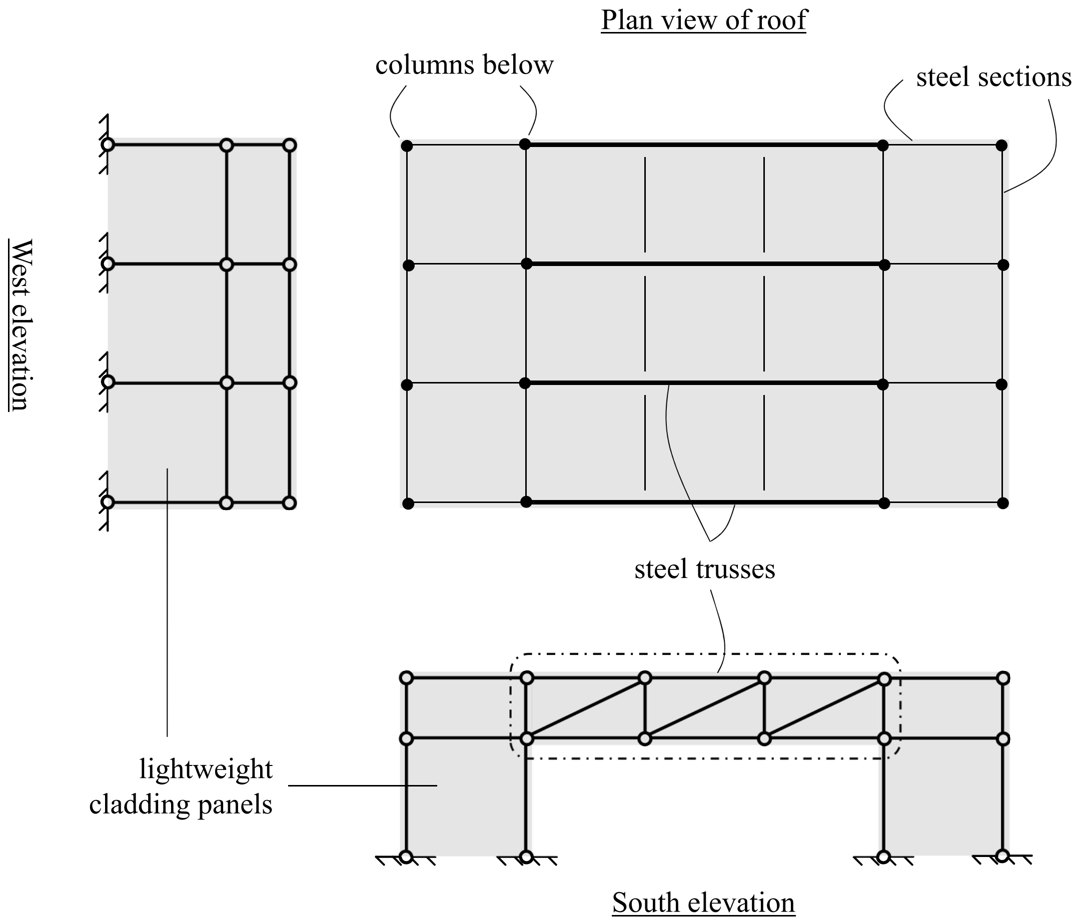


Fig. 1

(a) With the aid of sketches, devise a suitable structural scheme to provide overall stability to the structure. Summarise the principal load paths for the complete structure for both horizontal and vertical loading. Explain any assumptions made. [40%]

(b) Passing under the steel frame is a glass walkway that allows visitors to view the artefacts below. The walkway is constructed from 2-layer laminated fully toughened glass plates that span between low masonry walls as shown in Fig. 2. The glass plates can be assumed to provide effective lateral restraint to the top of the walls. A uniformly distributed characteristic transient medium-term load of 3.5 kN m^{-2} acting on the walkway is expected to govern the design. Self-weight of the glass may be assumed to be 0.5 kN m^{-2} for the purposes of initial design. The partial safety factor for transient load is 1.5 and for permanent load is 1.35.

(i) The glass has characteristic surface prestress f_{rk} = 90 MPa and characteristic annealed strength f_{gk} = 45 MPa; material partial safety factors γ_{mV} = 1.2 and γ_{mA} = 1.8; the load duration factor k_{mod} = 0.43 and the stressed area factor k_A = 1.0. The interlayer is made of polyvinyl butyral and is of negligible thickness. If both layers of the laminated glass are to be of equal thickness, determine the required total thickness of the laminated glass for adequate flexural strength. [30%]

(ii) The characteristic compressive strength of the masonry f_k = 2 MPa and the material partial safety factor for the masonry γ_m = 3. Perform suitable checks to determine whether the masonry walls are adequate to resist the loads from the walkway above. [30%]

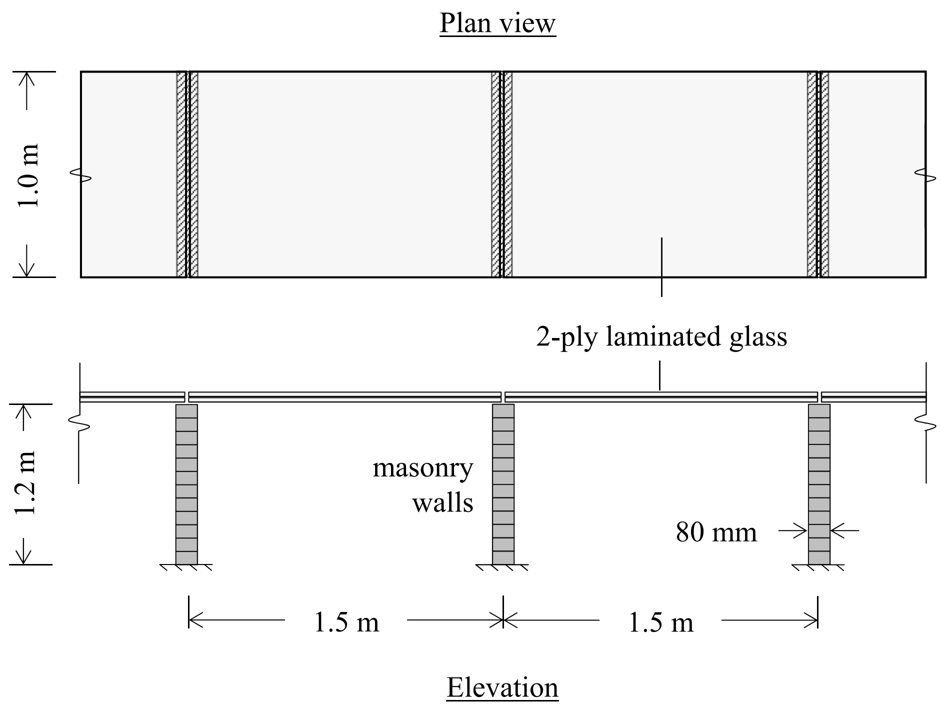


Fig. 2

2 A concrete beam of prismatic rectangular cross section spans across two simple supports as shown in Fig. 3. 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 $f_{cu} = 50$ MPa and a 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 $\gamma_c = 1.50$ 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 transient load of 20 kN m^{-1} and a uniformly distributed permanent load of 30 kN m^{-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 and breadth for the beam. [10%]

(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. [25%]

(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? Comment on the magnitude of the inadequacy. [10%]

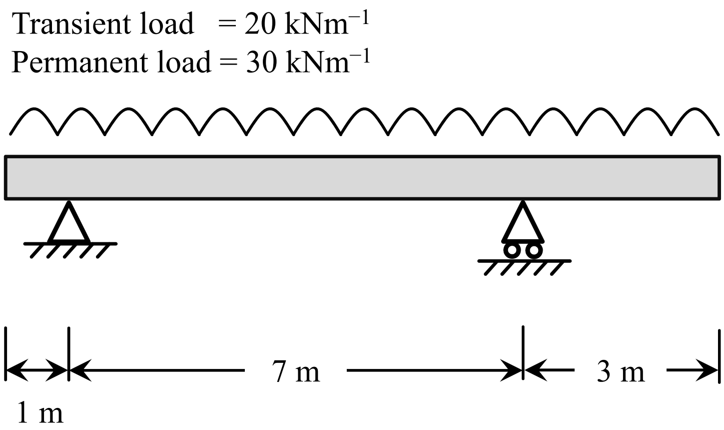


Fig. 3

(TURN OVER

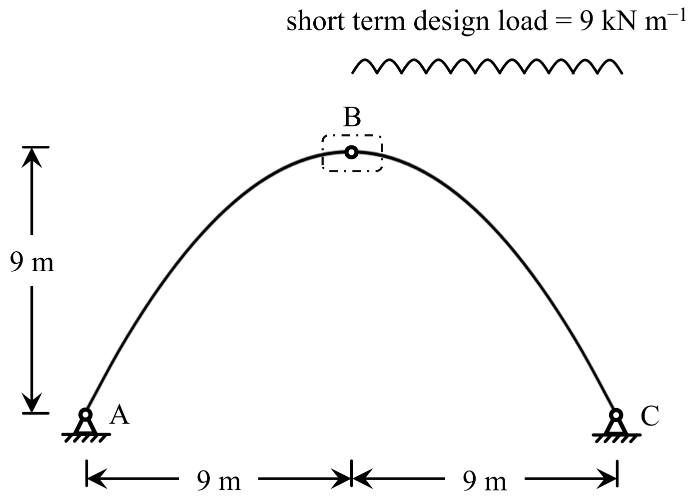
3 A series of three-pinned timber arches are to be used to form the structure of a long shed. All timber used in the design is to be C24 softwood and can be considered to be service class 1. The material partial safety factor for timber is 1.3.

(a) The structural arrangement of a single arch is idealised in Fig. 4(a). A proposed design for the pinned connection at B, at the apex of the arch, is shown in Fig. 4(b). The main compressive axial loads across the connection are transferred to the timber in end bearing by a steel end plate and this aspect of the design has already been proven by calculation. However, there is a concern that asymmetric load cases could lead to additional shear forces at the connection which must be resisted by the dowels. You decide to check the adequacy of the dowel connection for the short-term load case of a uniformly distributed vertical design load of 9 kN m^{-1} acting over only one half of the span as shown in Fig. 4(a).

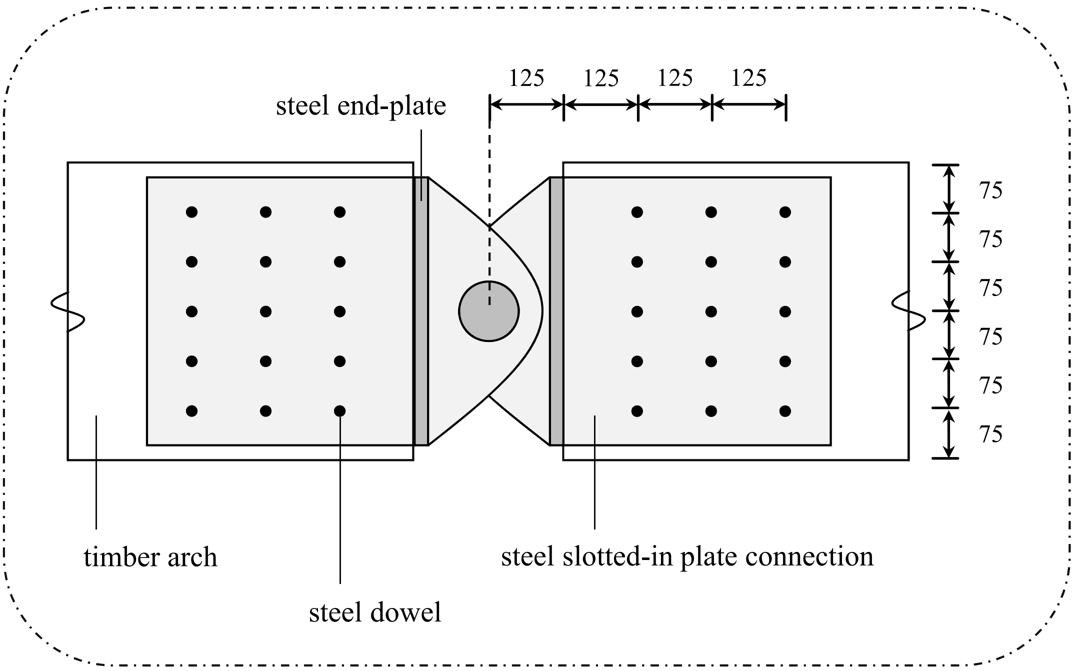
(i) Show that the design shear force at B that the connection must sustain is approximately 20 kN. [10%]

(ii) The dowels have a characteristic strength perpendicular-to-grain of 12 kN per dowel and a characteristic strength parallel-to-grain of 18 kN per dowel. Making any reasonable simplifications or assumptions needed, perform suitable calculations to determine whether the proposed connection design is adequate to resist the design shear force obtained in (i). [40%]

(b) The roof build-up of the shed is supported by timber purlins that span 9 m as simply supported beams between arches. All purlins are to be 150 mm wide. Purlins can be assumed to be fully laterally restrained along their length. The load sharing factor k_{ls} and the size effect factor k_h can conservatively be taken as 1. In the permanent condition, a purlin can be considered to support a permanent design load of 3 kN m^{-1} and no transient design load. The creep factor $k_{def} = 0.6$ and the permanent deflection limit is span/300. Determine the depth of purlin required to meet both the flexural strength and stiffness requirements. [50%]



(a) Structural arrangement



(b) Pinned connection detail at B. Dimensions in mm.

Fig. 4

(TURN OVER

4 A grade S420 UB 406×140×39 steel beam is continuous over two equal spans of 6 m as shown in Fig. 5(a). The beam carries a point load P in the middle of each span. The self-weight of the beam may be neglected. The steel has a characteristic yield strength f_y = 420 MPa, a Young's modulus E = 210 GPa and a shear modulus G = 81 GPa. 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. The partial safety factor for transient load is 1.50 and for permanent load is 1.35.

(a) Assume that the beam is continuously laterally restrained along its length. Calculate the maximum design load P that the beam can sustain, based on the development of a plastic hinge mechanism. [15%]

(b) Noting that the radius of the web-to-flange transition in a UB 406×140×39 is r = 10 mm, determine which class the cross-section belongs to for local buckling. Does the result justify the calculations in (a)? [25%]

(c) If the beam were only laterally restrained at the locations of the point loads and the supports, calculate the maximum design load P which the beam can sustain. The ends of the beam are free to warp under torsion. The elastic bending moment diagram of the beam is given in Fig. 1(b) where L is the 6 m span between supports. [40%]

(d) Using the results obtained in part (c), check whether the deflections in service conditions exceed a limit value of span/300. The loads P are to be considered transient loads that are simultaneously present. [20%]

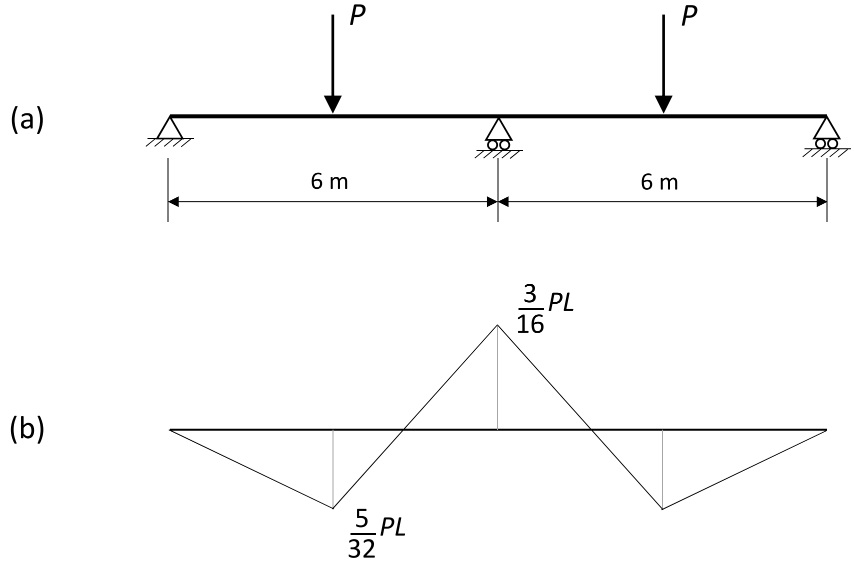


Fig. 5

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University of Cambridge
Department of Engineering

Engineering Tripos Part IIA

Module 3D3
Structural Materials & Design

Datasheets

Michaelmas 2022

THE CUMULATIVE NORMAL DISTRIBUTION FUNCTION

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

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	$\cdot9^{2}0097$	$\cdot9^{2}0358$	$\cdot9^{2}0613$	$\cdot9^{2}0863$	$\cdot9^{2}1106$	$\cdot9^{2}1344$	$\cdot9^{2}1576$
2·4	$\cdot9^{2}1802$	$\cdot9^{2}2024$	$\cdot9^{2}2240$	$\cdot9^{2}2451$	$\cdot9^{2}2656$	$\cdot9^{2}2857$	$\cdot9^{2}3053$	$\cdot9^{2}3244$	$\cdot9^{2}3431$	$\cdot9^{2}3613$
2·5	$\cdot9^{2}3790$	$\cdot9^{2}3963$	$\cdot9^{2}4132$	$\cdot9^{2}4297$	$\cdot9^{2}4457$	$\cdot9^{2}4614$	$\cdot9^{2}4766$	$\cdot9^{2}4915$	$\cdot9^{2}5060$	$\cdot9^{2}5201$
2·6	$\cdot9^{2}5339$	$\cdot9^{2}5473$	$\cdot9^{2}5604$	$\cdot9^{2}5731$	$\cdot9^{2}5855$	$\cdot9^{2}5975$	$\cdot9^{2}6093$	$\cdot9^{2}6207$	$\cdot9^{2}6319$	$\cdot9^{2}6427$
2·7	$\cdot9^{2}6533$	$\cdot9^{2}6636$	$\cdot9^{2}6736$	$\cdot9^{2}6833$	$\cdot9^{2}6928$	$\cdot9^{2}7020$	$\cdot9^{2}7110$	$\cdot9^{2}7197$	$\cdot9^{2}7282$	$\cdot9^{2}7365$
2·8	$\cdot9^{2}7445$	$\cdot9^{2}7523$	$\cdot9^{2}7599$	$\cdot9^{2}7673$	$\cdot9^{2}7744$	$\cdot9^{2}7814$	$\cdot9^{2}7882$	$\cdot9^{2}7948$	$\cdot9^{2}8012$	$\cdot9^{2}8074$
2·9	$\cdot9^{2}8134$	$\cdot9^{2}8193$	$\cdot9^{2}8250$	$\cdot9^{2}8305$	$\cdot9^{2}8359$	$\cdot9^{2}8411$	$\cdot9^{2}8462$	$\cdot9^{2}8511$	$\cdot9^{2}8559$	$\cdot9^{2}8605$
3·0	$\cdot9^{2}8650$	$\cdot9^{2}8694$	$\cdot9^{2}8736$	$\cdot9^{2}8777$	$\cdot9^{2}8817$	$\cdot9^{2}8856$	$\cdot9^{2}8893$	$\cdot9^{2}8930$	$\cdot9^{2}8965$	$\cdot9^{2}8999$
3·1	$\cdot9^{3}0324$	$\cdot9^{3}0646$	$\cdot9^{3}0957$	$\cdot9^{3}1260$	$\cdot9^{3}1553$	$\cdot9^{3}1836$	$\cdot9^{3}2112$	$\cdot9^{3}2378$	$\cdot9^{3}2636$	$\cdot9^{3}2886$
3·2	$\cdot9^{3}3129$	$\cdot9^{3}3363$	$\cdot9^{3}3590$	$\cdot9^{3}3810$	$\cdot9^{3}4024$	$\cdot9^{3}4230$	$\cdot9^{3}4429$	$\cdot9^{3}4623$	$\cdot9^{3}4810$	$\cdot9^{3}4991$
3·3	$\cdot9^{3}5166$	$\cdot9^{3}5335$	$\cdot9^{3}5499$	$\cdot9^{3}5658$	$\cdot9^{3}5811$	$\cdot9^{3}5959$	$\cdot9^{3}6103$	$\cdot9^{3}6242$	$\cdot9^{3}6376$	$\cdot9^{3}6505$
3·4	$\cdot9^{3}6631$	$\cdot9^{3}6752$	$\cdot9^{3}6869$	$\cdot9^{3}6982$	$\cdot9^{3}7091$	$\cdot9^{3}7197$	$\cdot9^{3}7299$	$\cdot9^{3}7398$	$\cdot9^{3}7493$	$\cdot9^{3}7585$
3·5	$\cdot9^{3}7674$	$\cdot9^{3}7759$	$\cdot9^{3}7842$	$\cdot9^{3}7922$	$\cdot9^{3}7999$	$\cdot9^{3}8074$	$\cdot9^{3}8146$	$\cdot9^{3}8215$	$\cdot9^{3}8282$	$\cdot9^{3}8347$
3·6	$\cdot9^{3}8409$	$\cdot9^{3}8469$	$\cdot9^{3}8527$	$\cdot9^{3}8583$	$\cdot9^{3}8637$	$\cdot9^{3}8689$	$\cdot9^{3}8739$	$\cdot9^{3}8787$	$\cdot9^{3}8834$	$\cdot9^{3}8879$
3·7	$\cdot9^{3}8922$	$\cdot9^{3}8964$	$\cdot9^{4}0039$	$\cdot9^{4}0426$	$\cdot9^{4}0799$	$\cdot9^{4}1158$	$\cdot9^{4}1504$	$\cdot9^{4}1838$	$\cdot9^{4}2159$	$\cdot9^{4}2468$
3·8	$\cdot9^{4}2765$	$\cdot9^{4}3052$	$\cdot9^{4}3327$	$\cdot9^{4}3593$	$\cdot9^{4}3848$	$\cdot9^{4}4094$	$\cdot9^{4}4331$	$\cdot9^{4}4558$	$\cdot9^{4}4777$	$\cdot9^{4}4988$
3·9	$\cdot9^{4}5190$	$\cdot9^{4}5385$	$\cdot9^{4}5573$	$\cdot9^{4}5753$	$\cdot9^{4}5926$	$\cdot9^{4}6092$	$\cdot9^{4}6253$	$\cdot9^{4}6406$	$\cdot9^{4}6554$	$\cdot9^{4}6696$
4·0	$\cdot9^{4}6833$	$\cdot9^{4}6964$	$\cdot9^{4}7090$	$\cdot9^{4}7211$	$\cdot9^{4}7327$	$\cdot9^{4}7439$	$\cdot9^{4}7546$	$\cdot9^{4}7649$	$\cdot9^{4}7748$	$\cdot9^{4}7843$
4·1	$\cdot9^{4}7934$	$\cdot9^{4}8022$	$\cdot9^{4}8106$	$\cdot9^{4}8186$	$\cdot9^{4}8263$	$\cdot9^{4}8338$	$\cdot9^{4}8409$	$\cdot9^{4}8477$	$\cdot9^{4}8542$	$\cdot9^{4}8605$
4·2	$\cdot9^{4}8665$	$\cdot9^{4}8723$	$\cdot9^{4}8778$	$\cdot9^{4}8832$	$\cdot9^{4}8882$	$\cdot9^{4}8931$	$\cdot9^{4}8978$	$\cdot9^{5}0226$	$\cdot9^{5}0655$	$\cdot9^{5}1066$
4·3	$\cdot9^{5}1460$	$\cdot9^{5}1837$	$\cdot9^{5}2199$	$\cdot9^{5}2545$	$\cdot9^{5}2876$	$\cdot9^{5}3193$	$\cdot9^{5}3497$	$\cdot9^{5}3788$	$\cdot9^{5}4066$	$\cdot9^{5}4332$
4·4	$\cdot9^{5}4587$	$\cdot9^{5}4831$	$\cdot9^{5}5065$	$\cdot9^{5}5288$	$\cdot9^{5}5502$	$\cdot9^{5}5706$	$\cdot9^{5}5902$	$\cdot9^{5}6089$	$\cdot9^{5}6268$	$\cdot9^{5}6439$
4·5	$\cdot9^{5}6602$	$\cdot9^{5}6759$	$\cdot9^{5}6908$	$\cdot9^{5}7051$	$\cdot9^{5}7187$	$\cdot9^{5}7318$	$\cdot9^{5}7442$	$\cdot9^{5}7561$	$\cdot9^{5}7675$	$\cdot9^{5}7784$
4·6	$\cdot9^{5}7888$	$\cdot9^{5}7987$	$\cdot9^{5}8081$	$\cdot9^{5}8172$	$\cdot9^{5}8258$	$\cdot9^{5}8340$	$\cdot9^{5}8419$	$\cdot9^{5}8494$	$\cdot9^{5}8566$	$\cdot9^{5}8634$
4·7	$\cdot9^{5}8699$	$\cdot9^{5}8761$	$\cdot9^{5}8821$	$\cdot9^{5}8877$	$\cdot9^{5}8931$	$\cdot9^{5}8983$	$\cdot9^{6}0320$	$\cdot9^{6}0789$	$\cdot9^{6}1235$	$\cdot9^{6}1661$
4·8	$\cdot9^{6}2067$	$\cdot9^{6}2453$	$\cdot9^{6}2822$	$\cdot9^{6}3173$	$\cdot9^{6}3508$	$\cdot9^{6}3827$	$\cdot9^{6}4131$	$\cdot9^{6}4420$	$\cdot9^{6}4696$	$\cdot9^{6}4958$
4·9	$\cdot9^{6}5208$	$\cdot9^{6}5446$	$\cdot9^{6}5673$	$\cdot9^{6}5889$	$\cdot9^{6}6094$	$\cdot9^{6}6289$	$\cdot9^{6}6475$	$\cdot9^{6}6652$	$\cdot9^{6}6821$	$\cdot9^{6}6981$

Example: $\Phi(3{\cdot}57) = \cdot9^{3}8215 = 0{\cdot}9998215$.

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

<u>Tension members</u>

Yielding of the gross cross-section A_g:

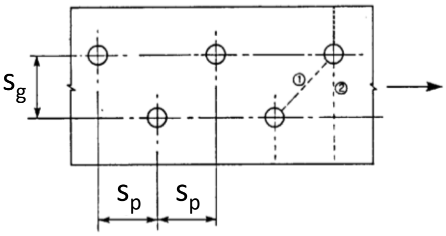
$$N_{pl,Rd} = \frac{A_g f_y}{\gamma_{M0}}$$

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

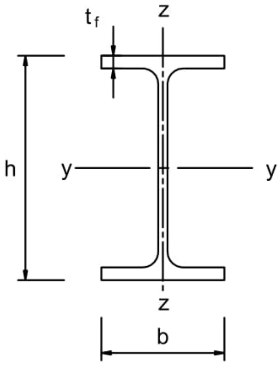
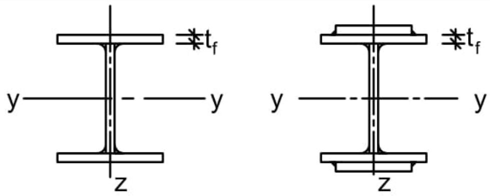
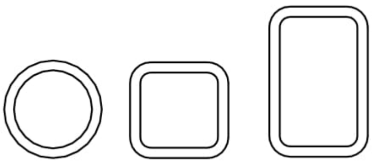
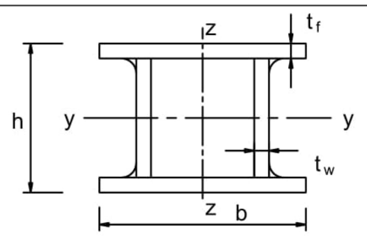
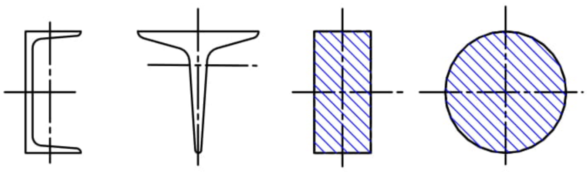
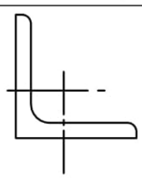
Reduction factor for shear lag in eccentrically connected angles:

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
		$h/b > 1{,}2$	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
		$h/b \le 1{,}2$	$t_f > 100$ 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

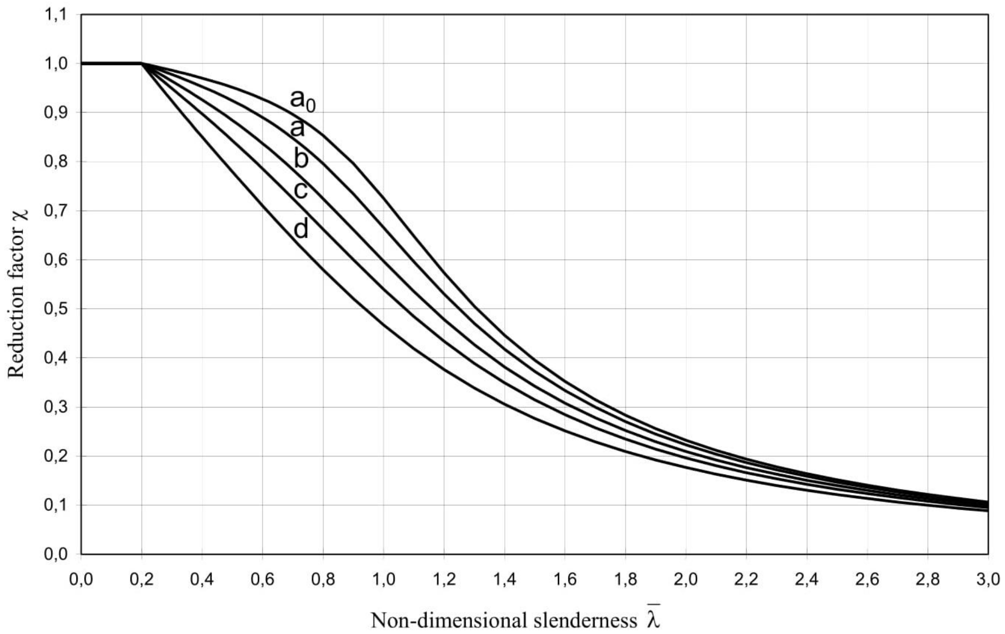


Figure 6.4: Buckling curves

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:

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

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

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

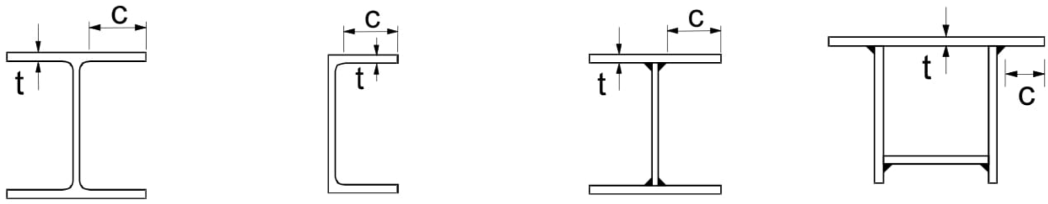
where b is the width of the plate and t is its thickness.

For plates in uniform longitudinal compression:

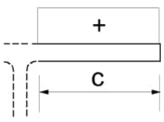
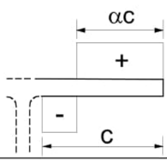
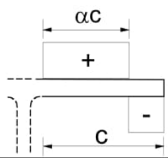
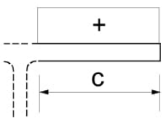
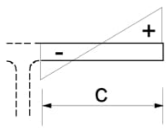
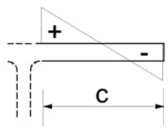
$K = 4$ for internal elements.

$K = 0.43$ for outstand elements.

Outstand flanges

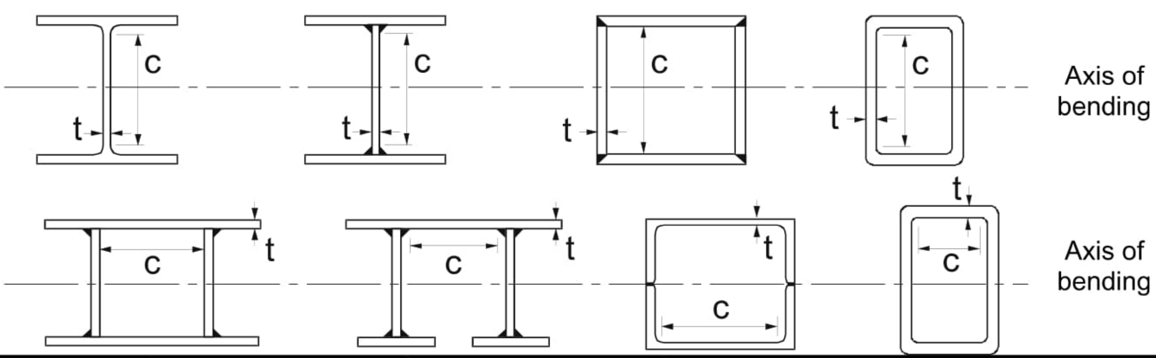


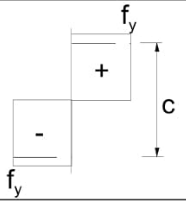
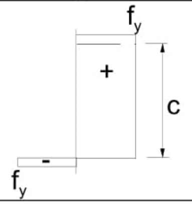
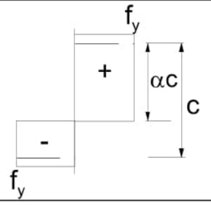
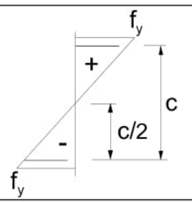
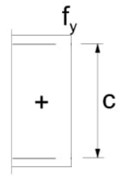
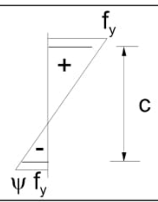
Rolled sections   Welded sections

Class	Part subject to compression	Part subject to bending and compression	
		Tip in compression	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	$c/t \le 72\varepsilon$	$c/t \le 33\varepsilon$	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:

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

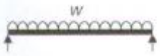
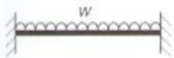
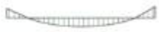
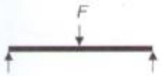
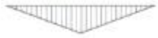
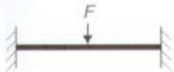
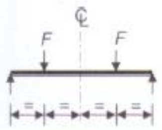
where:

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**

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

<u>Interaction between moment and shear in the cross-section</u>:

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

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

<u>Shear</u>

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

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

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:

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

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

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

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).

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

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

<u>Web crippling</u>:

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

$$= \frac{0.5}{} \le 1.0 \qquad F_{Rd} = \frac{l_y t_w f_{yw}}{\gamma_M}$$

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

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

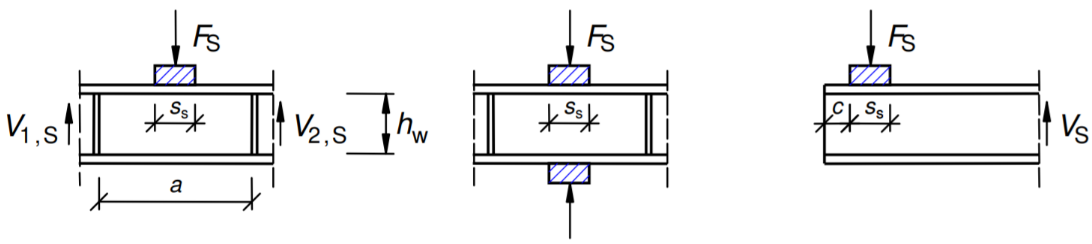
EOF: $\min \begin{cases} \ell_y = \ell_e + t_f\sqrt{\dfrac{m_1}{2}+\left(\dfrac{\ell_e}{t_f}\right)^2 + m_2} \\ \ell_y = \ell_e + t_f\sqrt{m_1+m_2} \end{cases}$

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

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

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

(IOF) (ITF) (EOF)



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

<u>Deflections:</u>

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

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			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	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

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

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)"

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

Extract from [11.2] - k_{crit}

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/mm^2

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

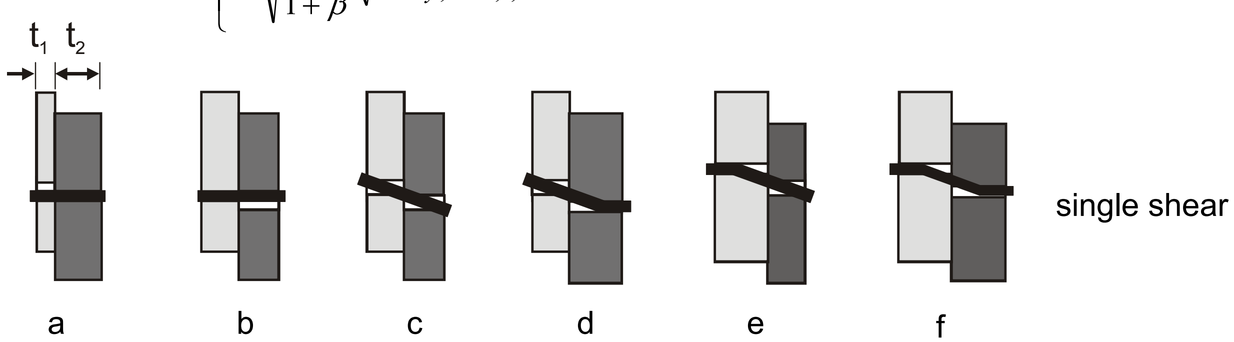
$$f_{h,\alpha,k} = \frac{f_{h,0,k}}{k_{90}\sin^2\alpha + \cos^2\alpha} \qquad \begin{array}{ll} \text{for softwood} & k_{90} = 1.35 + 0.015d \\ \text{for hardwood} & k_{90} = 0.90 + 0.015d \end{array}$$

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

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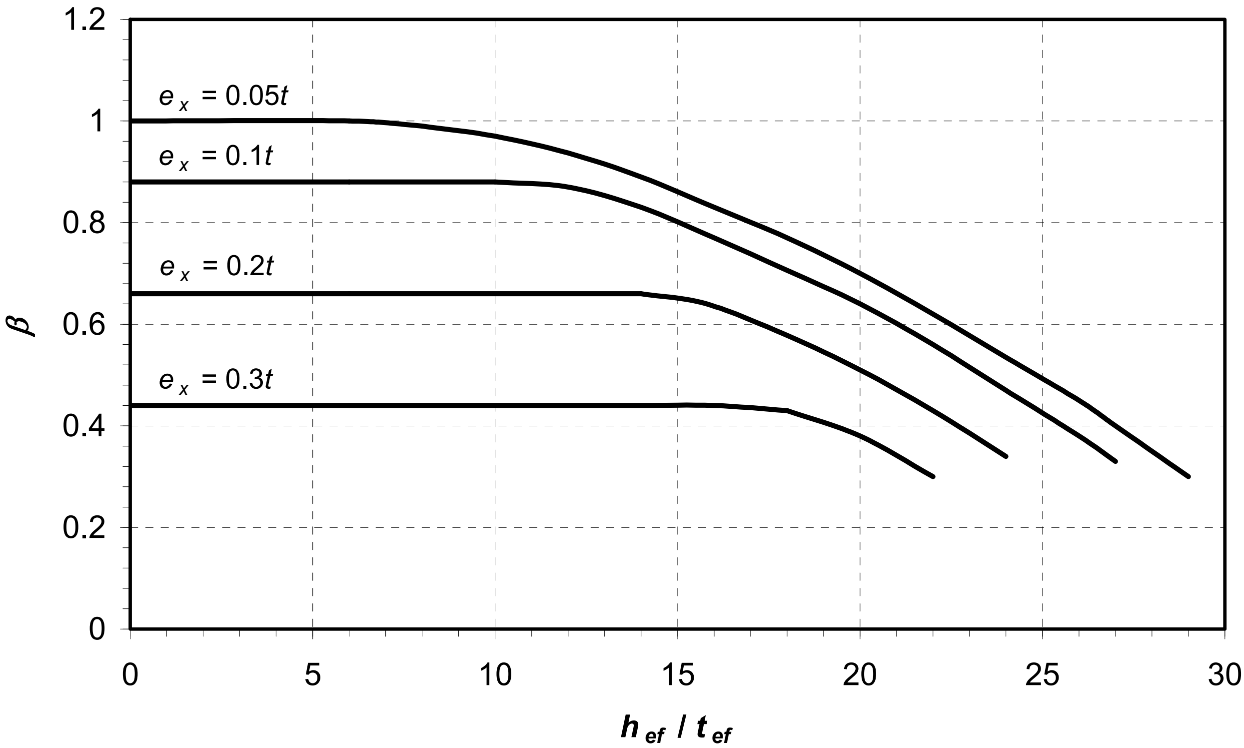
Bearing or crushing resistance per unit length

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

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

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

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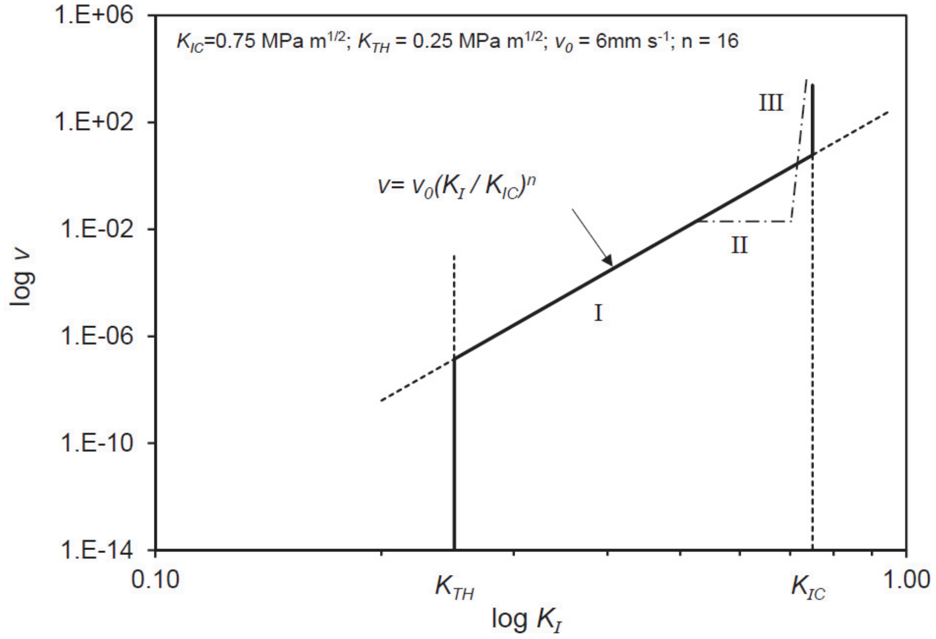
Explicit relationship between the flaw opening stress history and the initial flaw size:

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

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):

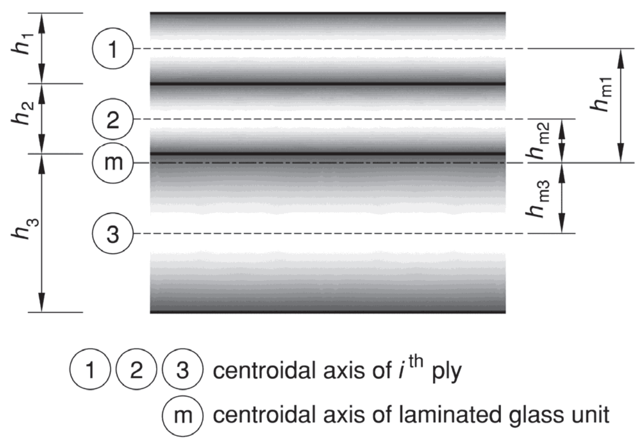
$$\frac{\sigma_f}{\sigma_{t0}} = \left(\frac{t_0}{t_f}\right)^{1/n} = k_{\mathrm{mod}}$$

Laminated glass equivalent thickness for bending deflection:

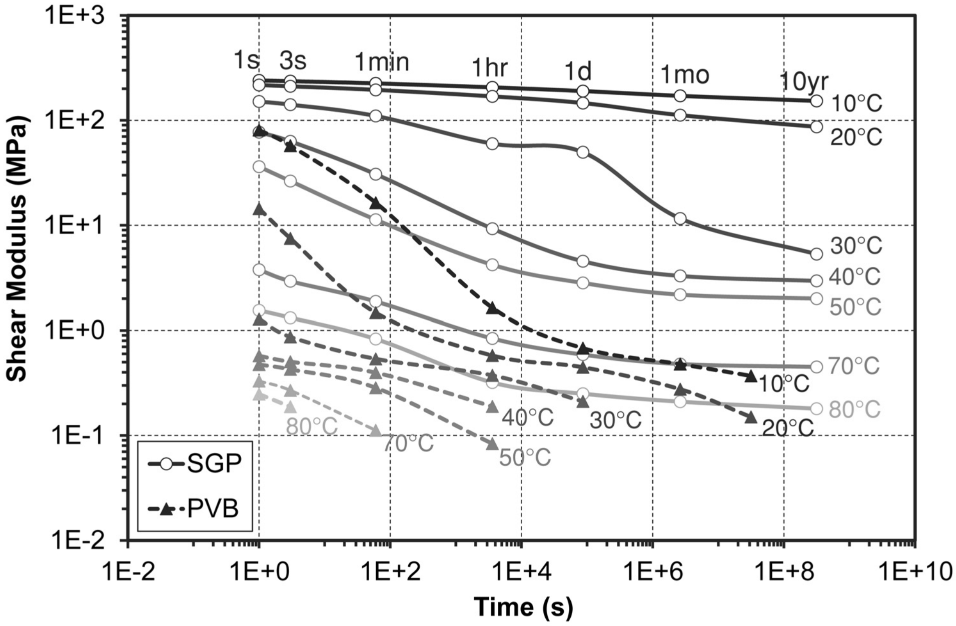
$$h_{eq,\delta} = \sqrt[3]{(1-\varpi)\sum_i h_i^{\,3} + \varpi\left(\sum_i h_i\right)^3}$$

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:

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

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

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

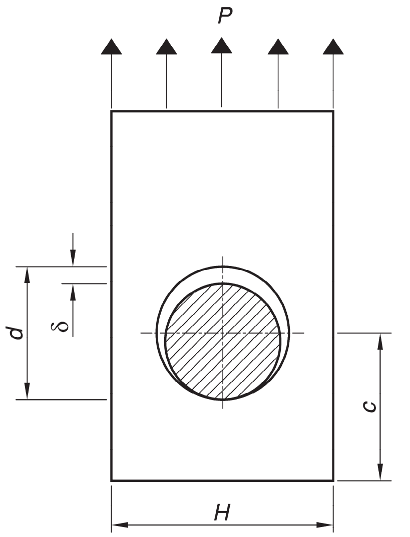


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Table 1.1 Span versus depth ratio

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]

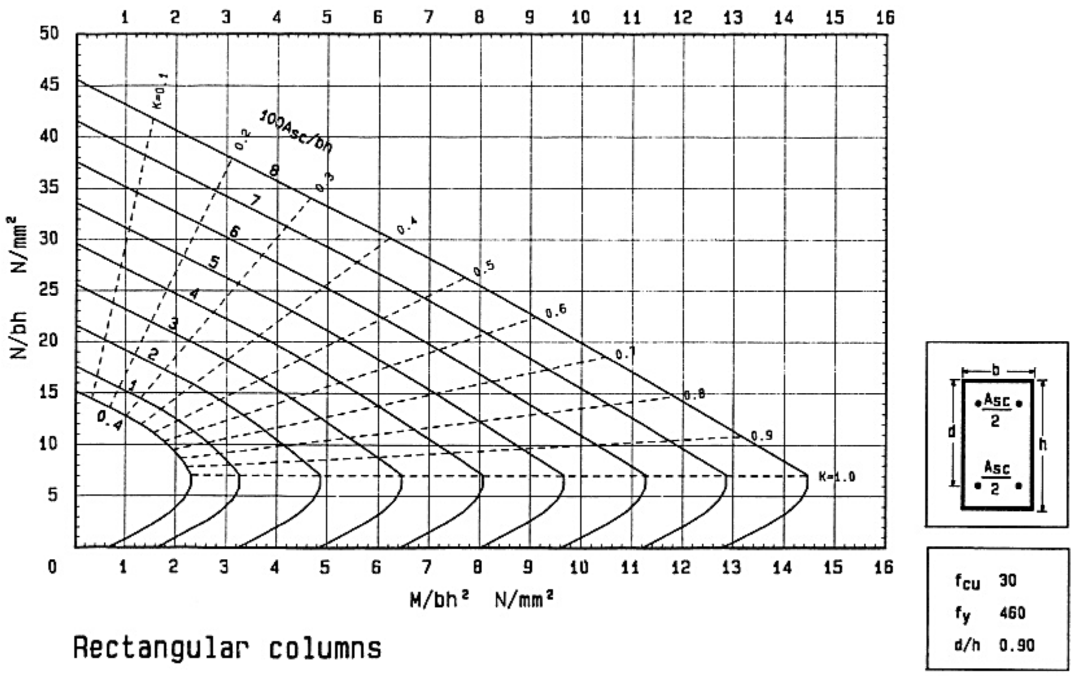

Fig 1.1 Interaction diagram from [1.3]

[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