INTRODUCTION TO EUROCODES

EC 0, 1, 2, 3 & 5

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Objectives

- What are Eurocodes?
- Defining limit state design concept.
- Structural design parameters
  - Design Loads or actions?
  - National annex
  - Factors of safety
  - Combination of loads.
  - Defining structural constraints (use, durability, environmental conditions, etc.)
- Structural design examples of a simple elements.
- Summary

What are EUROCODES?

- Historically British Standards, German Standards, French Standards, etc. Not ideal for free trade within the European Union, so standardisation necessary.
- Objectives
  - “The Eurocodes to establish a set of common technical rules for the design of buildings and civil engineering works which will ultimately replace the different rules in the various Member States.”
- Special agreement (BC/CEN/03/89)
  - Council Directives on Public Procurement (71/305)
  - The Construction Products Directive (CPD)

What is the CPD?

- The necessity to have commonality of structural materials irrespective of the source country or the destination country.
- Six Essential Requirements
  - Mechanical resistance & stability
  - Safety in case of fire
  - Hygiene, health and environment
  - Safety in use
  - Protection against noise
  - Energy economy and heat retention

Who are effected by CPD

- Clients
- Product manufacturers
- Consulting Civil/Structural Engineers
- Builders and Contractors
- Regulators - Building Control
- Quantity Surveyors
- Architects
- Insurers

Key elements of CPD

- Harmonised European Product Standards
  - European Standards or European technical Approvals (ETA)
- Agreed conformity for each product family
- A framework of notified bodies
  - Attestation by 3rd party OR manufacture's declaration plus supporting evidence.
- CE marking of construction products
  - Examples: structural steel not mild steel, or Grade 43 now S275
  - concrete not grade 25 now C25/30
    - It should be noted that the mode of testing concrete can be cylinder based tests or cube based testing. (25N/mm^2 and 30N/mm^2 respectfully.)

Product families

- Structural Bearings (M104)
- Structural timber products (M112)
- Wood base panels & related products (M113)
- Cement, building limes & hydraulic binders (M114)
- Reinforcing and prestressing steel for concrete (M115)
- Masonry products and ancillaries (M116)
- Structural metallic products and ancillaries (M120)
- Aggregates (M125)
- Construction adhesives (M127)
- Products related to concrete, mortar & grout (M128)

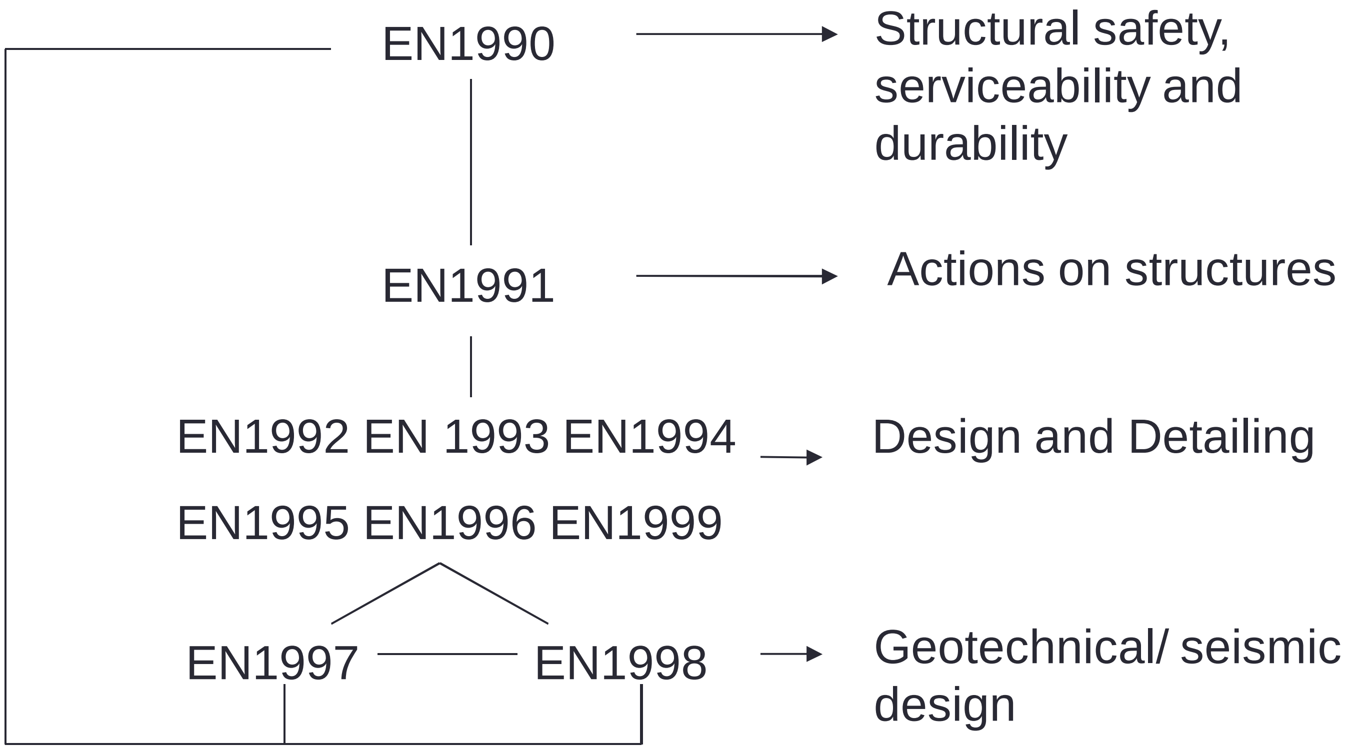
So Eurocodes!

- Intended use in structural design of buildings and civil engineering works, including:
  - Geotechnical features
  - Structural integrity and stability
  - Structural Fire design
  - Earthquakes, construction, demolition and temporary structures

What are the 58 Eurocodes?

- EN1990 Eurocode - Basis of Structural Design (1 unit with no subdivisions)
- EN1991 Eurocode 1 - Actions on structures (10 subdivisions)
- EN1992 Eurocode 2 - Design of concrete structures (4 subdivisions)
- EN1993 Eurocode 3 - Design of steel structures (20 subdivisions)
- EN1994 Eurocode 4 - Design of composite steel and concrete structures (3 subdivisions)
- EN1995 Eurocode 5 - Design of timber structures (3 subdivisions)
- EN1996 Eurocode 6 - Design of masonry structures (4 subdivisions)
- EN1997 Eurocode 7 - Geotechnical design (2 subdivisions)
- EN1998 Eurocode 8 - Design of structures for earthquake resistance (6 subdivisions)
- EN1999 Eurocode 9 - Design of aluminium structures (5 subdivisions)

Eurocode system



Basic format of Eurocodes

Part 1.1

General Rules and rules for buildings

Part 1.2

Structural Fire design

Part 2

Bridges

National Annex

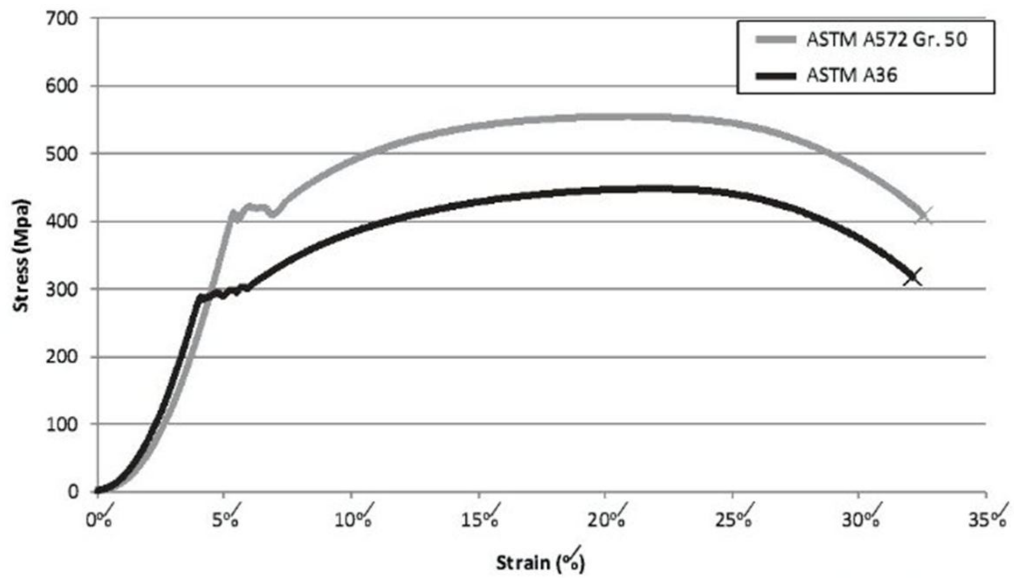
- "Eurocodes recognise the responsibility of regulatory and other relevant authorities in each Member State and have safeguarded their rights to determine values related to safety matters at National level where these continue to vary from State to State."
- In these notes where examples are used to illustrate a point the NAs of the UK will be used.
  - Definition of the applicable NA will be contract dependant but only one NA system should be used. Mixing NA from UK and Germany, for example, is not permitted.

Concept of limit state.

- Early structural design.
  - Forth Bridge 1894
  - First major steel structure.
  - Acceptable maximum stress was 6 Tons/ins^2 (93N/mm^2)
  - Design taken to be within elastic range of the materials.
- Needs result in change
  - Same material but acceptable limit increased to 10 Tons/ins^2 (153N/mm^2).
  - Design remains within the elastic range.

Elastic vs plastic design

- Elastic design
  - Factors of safety in the acceptable limit of the material.
  - Use statistical historic and experimental values for dead and live loads.
  - Originally applied to all structural materials (iron, steel, concrete, timber, etc.)
  - *But was it making the best use of the materials capacity?*



	ASTM A572 Gr. 50	ASTM A36
Yield Strength (MPa)	402 ± 12	289 ± 12
Tensile Strength (MPa)	555 ± 18	449 ± 18

Elastic vs plastic (continued)

- As defined in the Free Dictionary:
  - 1. (General Engineering) a design criterion specifying that with acceptable probabilities a structure will not reach a limit state in which it either is unfit for the use for which it was designed (unavailability limit state) or fails (ultimate limit state)
    - (http://www.thefreedictionary.com/limit-state+design December 2015)
- Reinforced concrete was the first material in Europe to consider plastic design and called it limit state design.
- Structural steel soon followed and only with the introduction of Eurocodes did timber follow.
  - Factors of safety applied to materials and loadings plus on combinations of loads.

Limit state design

- The concept of limit state design first applied in Russia in the 1930's (McKenzie) and consists of two aspects:
  - **Serviceability Limit State** in which a condition e.g. deflection, vibration or cracking, which is unacceptable to the owners of the structure.
  - **Ultimate Limit State** in which the structure, or some part of it, is unsafe for its intended purpose. Modes of failure can be compressive, tensile, shear, of flexural failure or instability lead to total or partial collapse of the structure.

EN1990

- No British Standard equivalent

- Provides the overall concept of the suite of structural Eurocode.
  - Ultimate limit state
    - EQU – loss of static equilibrium
    - STR – failure where material strength governs
    - GEO – failure or excessive deformation of the ground
    - FAT – fatigue failure of the structure or elements

BS EN1990

- Design values
  - $F_d = g_f F_{rep}$
    - g_f is the partial factor for action (favorable/unfavorable)
    - F_{rep} is the representative value for the action and is derived from $F_{rep} = \psi F_k$ (ψ as discussed below and F_k is the characteristic value of the action – BS EN1991)
    - ψ is 1.00 or as defined by BS EN1990 Table A1.1

Load combinations

- Characteristic
  - G is the abbreviation for permanent action (dead load)
  - Q is the abbreviation for a variable action and can be subdivided to frequent and quasi-permanent
  - P is where prestressing has been applied

$$E_d = \left\{G_{k,j}; P; Q_{k,1}; \psi_{0,1} Q_{k,1}\right\}$$

Extract of EC0

Table A1.2(A) — Design values of actions (EQU) (Set A)

Persistent and transient design situations	Permanent actions		Leading variable action (*)	Accompanying variable actions	
	Unfavourable	Favourable		Main (if any)	Others
(Eq. 6.10)	$\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_{Q,1}\ Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$

(*) Variable actions are those considered in Table A1.1

NOTE 1 The γ values may be set by the National Annex. The recommended set of values for γ are:

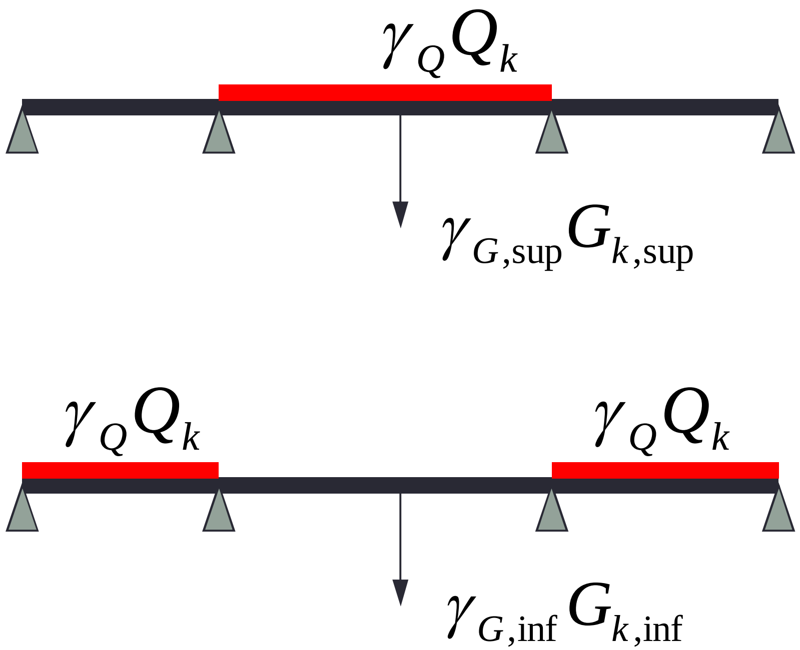
$\gamma_{Gj,sup}$ = 1,10
$\gamma_{Gj,inf}$ = 0,90
$\gamma_{Q,1}$ = 1,50 where unfavourable (0 where favourable)
$\gamma_{Q,i}$ = 1,50 where unfavourable (0 where favourable)

NOTE 2 In cases where the verification of static equilibrium also involves the resistance of structural members, as an alternative to two separate verifications based on Tables A1.2(A) and A1.2(B), a combined verification, based on Table A1.2(A), may be adopted, if allowed by the National Annex, with the following set of recommended values. The recommended values may be altered by the National Annex.

$\gamma_{Gj,sup}$ = 1,35
$\gamma_{Gj,inf}$ = 1,15
$\gamma_{Q,1}$ = 1,50 where unfavourable (0 where favourable)
$\gamma_{Q,i}$ = 1,50 where unfavourable (0 where favourable)

provided that applying $\gamma_{Gj,inf}$ = 1,00 both to the favourable part and to the unfavourable part of permanent actions does not give a more unfavourable effect.

Combination of actions



The principles of applying the various load parameters is no different from existing codes but the values have changed.

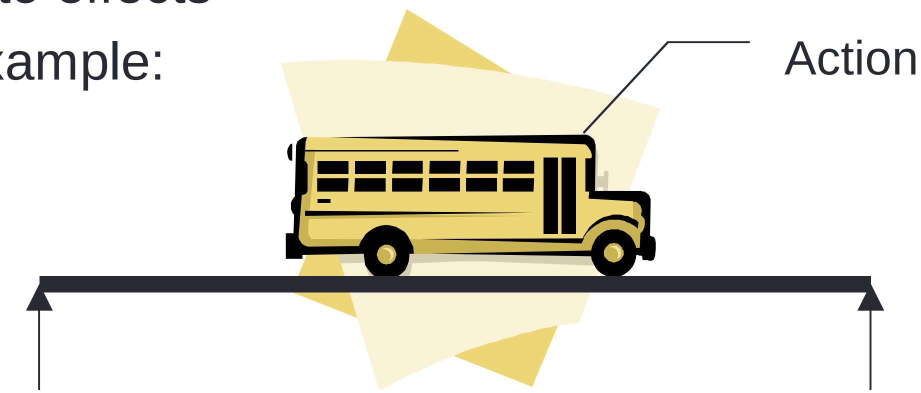
See tables A1.2(A, B and C) and BS EN1991

BS EN1991 Actions on structures

- 10 parts including General Actions (3), Wind, Snow, Thermal, Traffic, Accidental, Cranes & Machinery plus Silos & Tanks.
- Climate dependant criteria will be influenced by local conditions, hence the need for nationally determined parameters (NDPs).
  - General concepts of wind loading will apply throughout but the value of the wind pressures used will be directly related to historical metrological data and defined by the National governing body in NDPs.
  - Snow loading is also a condition that is geographical dependent and will be covered by NDP

Actions!? What happened to loads?

- Eurocode "talk"
  - Actions on a structure leads to effects
  - For example:



Effects:

Shear, bending and deformation

Eurocode Abbreviation examples

- Effect
  - Max Shear (V_{Ed})
    - V is the notation for shear force
    - E indicates it is an effect
    - d indicates it is a design consideration
  - Bending Moment (M_{Ed})
    - M is the notation for bending moment
    - E and d as above

- Resistance
  - Shear (V_{Rd})
    - Details are material dependant as with current codes
  - Bending ($M_{R,d}$)
    - Details are material dependant as with current codes

Eurocode 0

- Design Life
  - The normal working life of a structure can be defined at the design stage by the client's specification.
  - UK National Annex has define 5 categories of notional design working life.
    - Category 1: working life of 10 years such as temporary structures.
    - Category 2: working life 10 to 30 years such as replaceable elements of a structure like crane gantry girder.
    - Category 3: working life of 15 to 25 years such as agricultural buildings.
    - Category 4: working life of 50 years such as domestic and commercial buildings.
    - Category 5: working life of 120 years such bridges

BS EN1991 vs. Existing codes

- 10 Sections similar to existing codes (BS5400:2 & BS6399: 1, 2 & 3). Ranges recommended with the designer deciding.
  - Potential problem if change of use or in D&B contracts.
  - Bridge NA more complex than BS5400
  - Update of “Design Manual for Roads & Bridges” can be checked on www.highways.gov.uk/business/euro_codes

BS EN1991 (continued)

- Climate dependant actions will require the NA to provide specific data.
  - Wind Load similar to a mix of CoP 3 Ch 5 part 2 and BS6399 part 2. (NA due in August 2006)
  - Snow Loading similar in principle to BS6399 part 3 (NA due in December 2005)

BS EN1992 vs. existing codes

- Beams
  - More in line with BS5400 than BS8110.
  - Web crushing limit is shear higher so thinner webs possible in non-rectangular section .
  - More modern, e.g. non-linear analysis included
  - Material strengths based on cylinder strength not cube (C40/50 – f_{cy}/f_{cu})

- Compression members
  - Use of graphs no longer available. Compression members designed as beams with axial loading.
  - Alternative design curves for reinforcement and prestressing–
    - Idealised and design
  - No formulae given for bending resistance – application of first principles.

Fundamental Principles of EC2

- Concrete is a non-homogeneous material.
  - Cement, aggregate (course & fine) but treated as homogeneous
- Design using established theories
  - Stress and strain
  - Plastic design – ultimate and servicability
- Strong in compression weak in tension
  - Table 3.1 –see next slide.

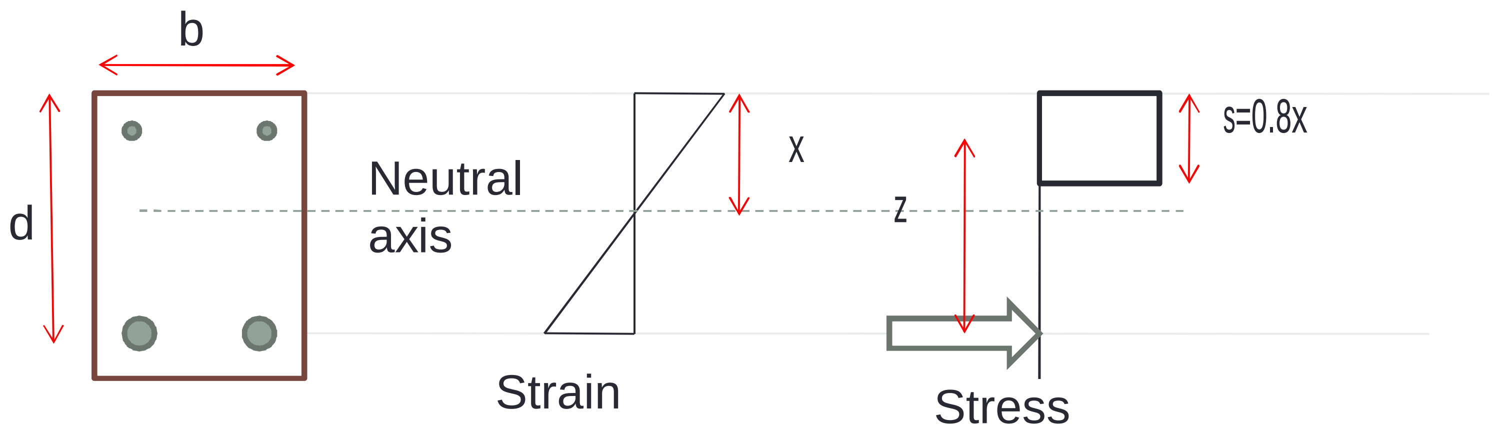
Table 3.1 — Strength and deformation characteristics for concrete

Strength classes for concrete															Analytical relation / Explanation
f_{ck} (MPa)	12	16	20	25	30	35	40	45	50	55	60	70	80	90	
$f_{ck,cube}$ (MPa)	15	20	25	30	37	45	50	55	60	67	75	85	95	105	
f_{cm} (MPa)	20	24	28	33	38	43	48	53	58	63	68	78	88	98	$f_{cm} = f_{ck} + 8$ (MPa)
f_{ctm} (MPa)	1,6	1,9	2,2	2,6	2,9	3,2	3,5	3,8	4,1	4,2	4,4	4,6	4,8	5,0	$f_{ctm} = 0{,}30 \times f_{ck}^{(2/3)} \leq$ C50/60 $f_{ctm} = 2{,}12 \cdot \ln(1 + (f_{cm} / 10)) >$ C50/60
$f_{ctk,\,0,05}$ (MPa)	1,1	1,3	1,5	1,8	2,0	2,2	2,5	2,7	2,9	3,0	3,1	3,2	3,4	3,5	$f_{ctk,\,0,05} = 0{,}7 \times f_{ctm}$ 5% fractile
$f_{clk,\,0,95}$ (MPa)	2,0	2,5	2,9	3,3	3,8	4,2	4,6	4,9	5,3	5,5	5,7	6,0	6,3	6,6	$f_{clk,\,0,95} = 1{,}3 \times f_{ctm}$ 95% fractile
E_{cm} (GPa)	27	29	30	31	33	34	35	36	37	38	39	41	42	44	$E_{cm} = 22\,[(f_{cm}) / 10]^{0,3}$ (f_{cm} in MPa)
ε_{c1} (‰)	1,8	1,9	2,0	2,1	2,2	2,25	2,3	2,4	2,45	2,5	2,6	2,7	2,8	2,8	see Figure 3.2 ε_{c1} (‰) $= 0{,}7\, f_{cm}^{0,31} < 2{,}8$
ε_{cu1} (‰)	3,5									3,2	3,0	2,8	2,8	2,8	see Figure 3.2 for $f_{ck} \geq 50$ Mpa ε_{cu1} (‰) $= 2{,}8 + 27\,[(98 - f_{cm}) / 100]^4$
ε_{c2} (‰)	2,0									2,2	2,3	2,4	2,5	2,6	see Figure 3.3 for $f_{ck} \geq 50$ Mpa ε_{c2} (‰) $= 2{,}0 + 0{,}085\,(f_{ck} - 50)^{0,53}$
ε_{cu2} (‰)	3,5									3,1	2,9	2,7	2,6	2,6	see Figure 3.3 for $f_{ck} \geq 50$ Mpa ε_{cu2} (‰) $= 2{,}6 + 35\,[(90 - f_{cm}) / 100]^4$
n	2,0									1,75	1,6	1,45	1,4	1,4	for $f_{ck} \geq 50$ Mpa $n = 1{,}4 + 23{,}4\,[(90 - f_{cm}) / 100]^4$
ε_{c3} (‰)	1,75									1,8	1,9	2,0	2,2	2,3	see Figure 3.4 for $f_{ck} \geq 50$ Mpa ε_{c3} (‰) $= 1{,}75 + 0{,}55\,[(f_{ck} - 50) / 40]$
ε_{cu3} (‰)	3,5									3,1	2,9	2,7	2,6	2,6	see Figure 3.4 for $f_{ck} \geq 50$ Mpa ε_{cu3} (‰) $= 2{,}6 + 35\,[(90 - f_{ck}) / 100]^4$

For design to EC2 use the cylinder (not cube) strength f_{ck}

Fundamental Principles

- Distribution of bending strain/stresses
  - Assume no tensile resistance from concrete
  - Assume the compressive zone is rectangular, *even though it isn't!*



$z = d - {}^{s}/_{2}$ or $s = 2(d - z)$

Depth O/A (h)
Depth to compression reinforcement (d')
Centre of compression to centre of tension (z)

Strength of materials

- Concrete
  - Characteristic strength f_{ck}
  - Design strength
    - Where

$$f_{cd} = \alpha_{cc} \frac{f_{ck}}{\gamma_c}$$

α_{cc}= coefficient to allow for the design assumptions. In the UK use 0.85 for bending and axial loading

γ_c= Partial factor for concrete. In the UK use 1.5

Strength of material (continued)

- Concrete (continued)
  - For example: It has been decided to use C25/30 concrete for the construction of a building. What is the design strength of concrete that should be used in the design?

$$\therefore f_{cd} = 0.85\frac{f_{ck}}{1.5} = 0.567 f_{ck}$$

  - Design strength in compression = 0.567x25=14.2N/mm^2

  - Note that for shear α_{cc} can be taken as 1.0 so
    - Design Shear strength = 25/1.5=16.7N/mm^2

Strength of material (continued)

- Reinforcing steel
  - ALL reinforcing steel in the UK has a strength (f_{yk}) of 500N/mm^2 – BS EN 10080
    - Only ductility differences are specified A, B or C
      - A and B are the most common with C most likely for structures that are exposed to low operation temperatures (below -20^{o}C)
    - Partial factor for reinforcement (γ_s) is 1.15
    - Applicable to cold <u>and</u> hot formed reinforcing steel (plain <u>or</u> ribbed).

Design Equations for Bending

- For equilibrium the compressive resistance must equal the tensile resistance.
  - $F_{cc}=F_{st}$
- Moment of resistance from compression
  - $F_{cc}\,z$
- Moment of resistance from compression
  - $F_{st}\,z$

Compressive force

- As a design assumption the compressive zone is defined by the rectangular stress block.
  - Block's dimensions are *b* and *s*
  - $\therefore F_{cc}$=stress x area = $0.567f_{ck}$ x b x s
  - $\therefore M=0.567f_{ck}bs \times z=1.134f_{ck}b(d-z)z$
  - Rearranged to be $M=Kbd^2f_{ck}$
  - and $z=d[0.5+\sqrt{(0.25-K/1.134)}]$ (max 0.95d)
  - Note if K<0.167 there is no need for compressive reinforcement

Tensile force

- The tensile force is defined by the stress on the reinforcing bars
  - Area of reinforcing bars referred to as A_s.
  - $\therefore F_{st}$=stress x area = $(f_{yk}\ /1.15)$ x A_s or $(0.87xf_{yk})$

  - $M=0.87f_{yk}A_s$ x z

  - Rearranged to be $A_s=M/(0.87f_{yk}z)=M/(435z)$

Example – simple slab

- Design assumptions
  - Design as a beam
  - Consider a 1m wide strip of slab
  - Use mesh reinforcement
  - Define if simply supported or continuous
    - At this stage we will assume simply supported
  - Consider bending and deformation criteria
  - Ignore shear

Slab details

- A span of 4,5m is required to carry a characteristic variable action of $3.0kN/m^2$ plus floor self weight, finishes and the ceiling below.
  - Strength of concrete C25/30 steel $500N/mm^2$
  - Exposure XC-1
  - Use the span - effective depth ratio graph from “The Concrete Centre Concise Eurocode 2”

Span-effective depth ratio

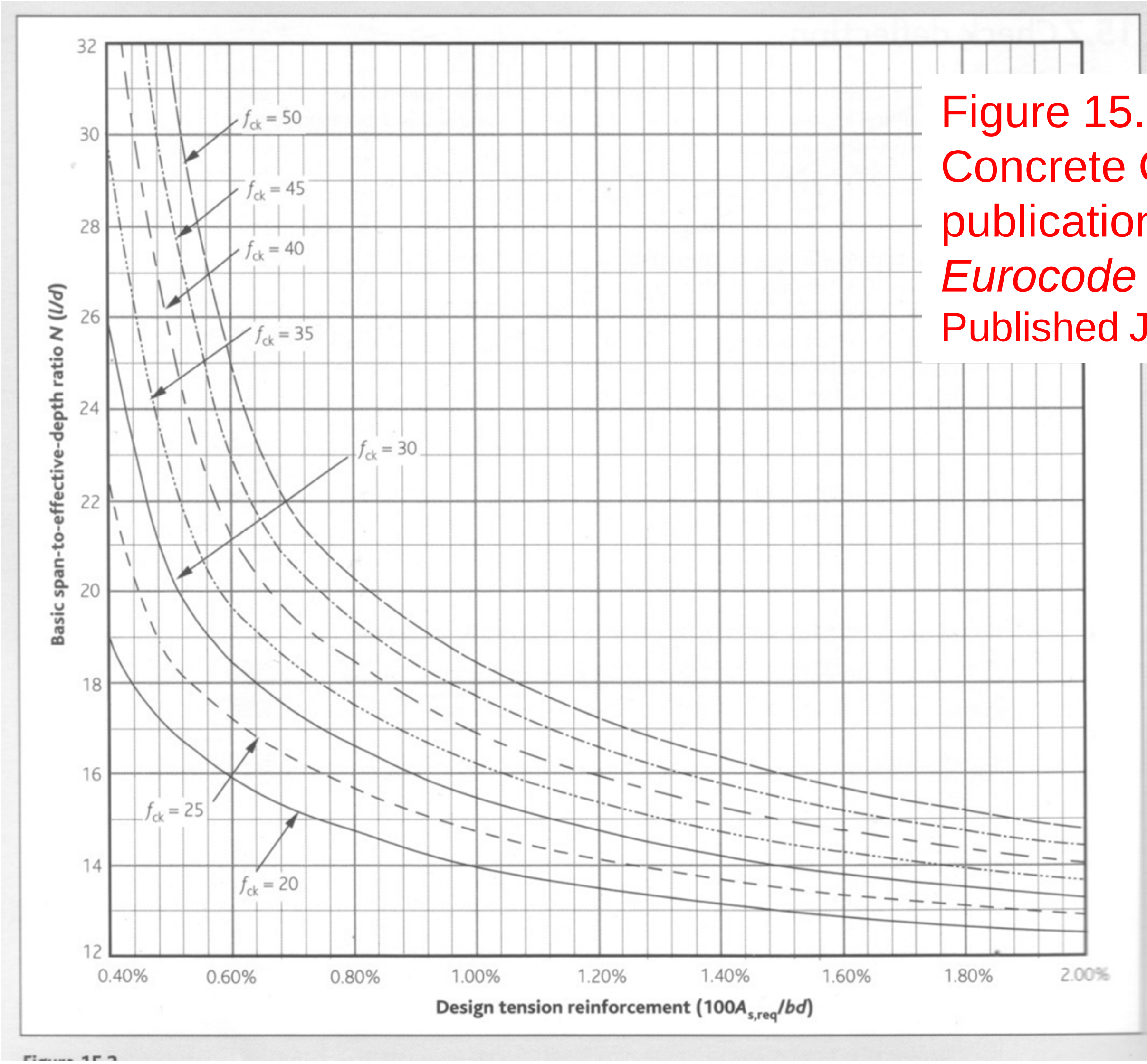
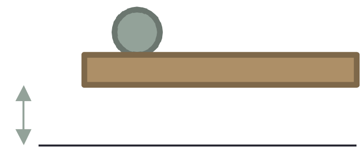


Figure 15.2 from **The** Concrete Centre's publication *Concise Eurocode 2.*
Published June 2006

Design of slab

- Estimate slab thickness
  - Assume span to effective depth 20
    - 4500/20=225mm
  - Exposure cover 15mm (which is the distance from the outermost face of rebar to exposed face of concrete
  - Assume 10mm bar AND no shear links
    - Total cover 15+(10/2)+ ΔC_{dev} =15+5+10=30mm
    - Depth of slab=225+30=255mm, say 250mm
      - Permanent action 25x0.25=6.3kN/m^2



Cover illustration

Note: ΔC_{dev} can be 10mm, 5mm or 0mm depending on conditions, e.g. normal site, precast yard, factory controlled, etc.

Design of slab (cont'd)

$$E_d = G\gamma_G + Q\gamma_Q = 7\times1.35 + 3\times1.5 = 18.7\,kN/m^2$$

$$M_{E,d} = \frac{wL^2}{8} = \frac{18.7\times4.5^2}{8} = 47kNm$$

Check for K

$$\mathrm{K} = \frac{\mathrm{M}}{\mathrm{bd}^2 f_{ck}} = \frac{47\times10^6}{1000\times225^2\times25} = 0.04 < 0167$$

$$z = d\left[0.5 + \sqrt{\left(0.25 - \frac{K}{1.134}\right)}\right] = d\left[0.5 + \sqrt{\left(0.25 - \frac{0.04}{1.134}\right)}\right] = 0.96d$$

Max z value is 0.95d $\therefore$ use 0.95d=214mm

$$\mathrm{A_s} = \frac{M}{0.87 f_{ykz}} = \frac{47\times10^6}{0.87\times500\times214} = 505mm^2$$

Design of slab (cont'd)

- The area of steel is per metre of slab
  - Adopt 10mm bar at 150mm c/c (523mm^2)
  - Span to effective depth ratio

$$\rho = \frac{100A_{s,req}}{bd} = \frac{100 \times 505}{1000 \times 225} = 0.22\%$$

From curve max is 22.5 and actual is 20 $\therefore$ satisfactory.

Example – singly reinforced beam.

- A rectangular RC beam is required to carry a design bending moment (M_{Ed}) of 185kNm.
  - The concrete is to be C25/30
  - Beam width (b) is 260mm and depth to reinforcement (d) is 440mm
- What is the minimum area of steel reinforcement?
  - (f_{yk}=500N/mm^2)

Check compressive resistance

$$K = \frac{M}{bd^2 f_{ck}} = \frac{185 \times 10^6}{260 \times 440^2 \times 25} = 0.147 < 0167$$

- As K is less than 0.167 there is no need for compression reinforcement
- Lever arm (z)

$$z = d\left[0.5 + \sqrt{\left(0.25 - \frac{K}{1.134}\right)}\right] = 440\left[0.5 + \sqrt{\left(0.25 - \frac{0.147}{1.134}\right)}\right] = 373mm$$

Tensile resistance

$$A_s = \frac{M}{0.87 f_{yk} z} = \frac{185 \times 10^6}{0.87 \times 500 \times 373} = 1141 mm^2$$

- The designer must chose the necessary reinforcement
  - But it will not be exact as was the case with peceeding codes.
  - Options
    - 3 @ 25mm dia (1470mm^2) 78% capacity
    - 2 @ 32mm dia (1610mm^2) 71%capacity
    - 6 @ 16mm dia (1210mm^2) 94% capacity

Maximum & Minimums!

- Max A_s
  - $0.04A_C$ for beams and slabs (Ac is the gross cross sectional area of the concrete member)
- Min A_s
  - $A_s = k_c k f_{ct,eff} A_{ct} / f_{yk}$
    - k_c 0.4 for beams and slabs, k 1.0 or 0.65, $f_{ct,eff}$ use f_{ctm} from Table 3.1 EC2, A_{ct} are of concrete in tension before crack initiation.

Strength	C20/25	C25/30	C30/37	C35/45	C40/50
% $A_{s,min}$	0.130	0.135	0.151	0.166	0.182

- Min bar spacing
  - Larger of bar diameter, 20mm or max. aggregate size + 5mm.

Durability and fire resistance

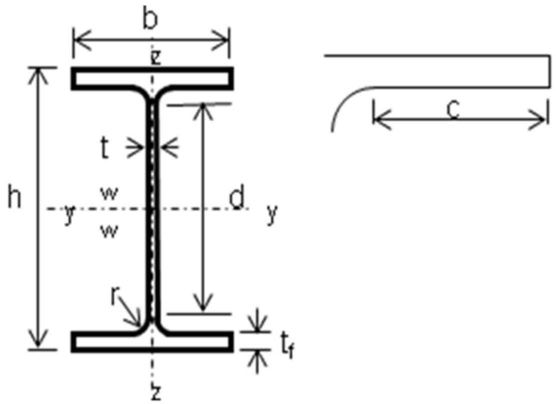
- Concrete is a durable material but it does suffer decay due to natural and "man-made" aspects – see table 4.1.
  - Protection of reinforcement from a cover of concrete.
    - Minimum cover c_{min} Section 4.1, 4.2 and 4.3
      - Cover for bond $c_{min,b}$
      - Cover for durability $c_{min,dur}$
      - Δc_{dev} to allow for deviation (10mm, 5mm or 0mm)
      - Fire protection 'a'

BS EN1993 vs. existing codes

- Beams
  - Design dependant on application of first principles for bending
  - Plastic design can be considered in Class 1, 2, 3 & 4.
  - Effect of shear lag to be considered
  - Semi-rigid connection to be considered in joint design
- Compression member
  - Same imperfections and Perry-Robertson formulae.
  - Additional check for flexural-torsional buckling
  - System slender can be considered
  - LTB depends on application of 1st principle analysis (FE)

Steel section nomenclature

Notation	Description	Units in normal use
b	breadth of section (flange width)	mm
h	Overall depth of section	mm
d	Depth of straight portion of web	mm
G	Weight per metre of section	kg/m
t_f	Thickness of flange	mm
t_w	Thickness of web	mm
W_{el}	Elastic section modulus (axis dependant)	cm3
W_{pl}	Plastic section modulus (axis dependant)	cm3
I	Second moment of area (axis dependant)	cm4
i	Radius of gyration	cm
A	Cross sectional area	cm2
A_v	Shear area	cm2
r	Root radius	mm
c/t_f	Outstand of flange divided by flange thickness	ND
d/t_w	Web width divided by web thickness	ND



Classification of section

- Check outstand of flange
  - C is the “unsupported” element of the compression zone.
  - Liable to buckle locally
    - Check outstand and internal elements
    - Check what the NA states regarding limitations for f_y as it is thickness dependant
      - (e.g. less than or equal to 40mm, and above 40mm)
    - Define using the highest classification for outstand and internal

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

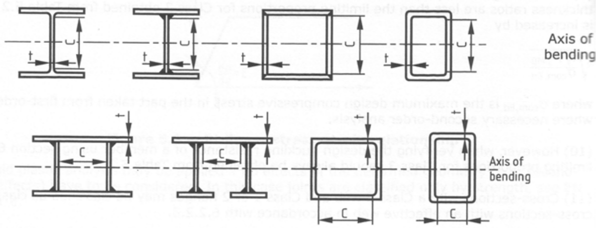
Standard and steel grade	Nominal thickness of the element t [mm]			
	$t \leq 40$ mm		40 mm $< t \leq 80$ mm	
	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]
EN 10025-2				
S 235	235	360	215	360
S 275	275	430	255	410
S 355	355	510	335	470
S 450	440	550	410	550
EN 10025-3				
S 275 N/NL	275	390	255	370
S 355 N/NL	355	490	335	470
S 420 N/NL	420	520	390	520
S 460 N/NL	460	540	430	540
EN 10025-4				
S 275 M/ML	275	370	255	360
S 355 M/ML	355	470	335	450
S 420 M/ML	420	520	390	500
S 460 M/ML	460	540	430	530
EN 10025-5				
S 235 W	235	360	215	340
S 355 W	355	510	335	490
EN 10025-6				
S 460 Q/QL/QL1	460	570	440	550

The grades of steel are those already used in UK. Note that thicker elements (>40 mm) have reduced f_y values. This is due to the manufacturing process where thinner sections have undergone more 'work hardening' by extra rolling. Using 3.2.1(1)a), i.e. taking the values directly from the standard, would also give decreasing design strength with increasing thickness but with more steps. In normal situations the flanges are both the critical and thickest element so it is reasonable that yield strength is based on them. S235 steel is a basic grade commonly available in Europe, not generally available in UK, but the situation may change in the future.

Extract of BS EN1993 for classification

Table 5.2 (sheet 1 of 3) — Maximum width-to-thickness ratios for compression parts

Internal compression parts



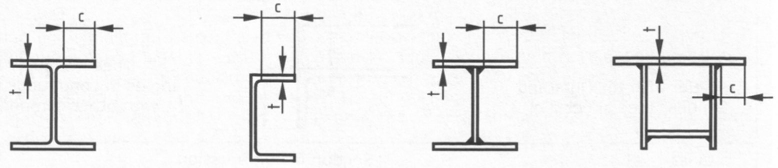
Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \le 72\varepsilon$	$c/t \le 33\varepsilon$	when $\alpha > 0,5: c/t \le \frac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0,5: c/t \le \frac{36\varepsilon}{\alpha}$
2	$c/t \le 83\varepsilon$	$c/t \le 38\varepsilon$	when $\alpha > 0,5: c/t \le \frac{456\varepsilon}{13\alpha - 1}$ when $\alpha \le 0,5: c/t \le \frac{41,5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)			
3	$c/t \le 124\varepsilon$	$c/t \le 42\varepsilon$	when $\psi > -1: c/t \le \frac{42\varepsilon}{0,67 + 0,33\psi}$ 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 $\sigma < f_y$ or the tensile strain $\varepsilon_y > f_y / E$.

Table 5.2 (sheet 2 of 3) — Maximum width-to-thickness ratios for compression parts

Outstand flanges



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	$c/t \le 9\varepsilon$	$c/t \le \frac{9\varepsilon}{\alpha}$	$c/t \le \frac{9\varepsilon}{\alpha\sqrt{\alpha}}$
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

See page 349 DoSE

Variable values of f_y

Steel Grade	EN1993-1-1			EN10025-2 (UK NA)		
	Thickness *t* (mm)	f_y (N/mm^2)	f_u (N/mm^2)	Thickness *t* (mm)	f_y (N/mm^2)	f_u (N/mm^2)
S275				$t \le 16$	275	430
	$t \le 40$	275	430	$16 < t \le 40$	265	430
				$40 < t \le 63$	255	430
	$40 < t \le 80$	255	410	$63 < t \le 80$	245	430
S355				$t \le 16$	355	510
	$t \le 40$	355	510	$16 < t \le 40$	345	510
				$40 < t \le 63$	335	510
	$40 < t \le 80$	335	470	$63 < t \le 80$	325	510

Classification Example (S275)

Section Designation Serial Size	Mass per Metre	Depth of Section	Width of Section	Thickness		Root Radius	Depth between Fillets	Ratios for Local Buckling	
				Web	Flange			Flange	Web
		h	b	t_w	t_f	r	d	c/t_f	d/t_w
	kg/m	mm	mm	mm	mm	mm	mm		
457 x 152 x 82	82.1	465.8	155.3	10.5	18.9	10.2	407.6	3.29	38.82

- Check flange thickness
  - 18.9mm ∴less than 40mm so f_y=265N/mm2
  - ε=0.92
    - Class 1 flange c/t max = 8.5>3.29
    - Class 1 web d/t max = 67.8>38.82

Simple beam design to EN1993

- To demonstrate the design process a simple example of a structural frame is considered.
  - Key features.
    - Span of beams 7m
    - Span of floor slab 3m
  - Beams to be designed
    - Edge beam (A1B1)
    - Mid floor beam (A2B2)
  - Material to be use S275

Example of simple beam
Actions on structure

- Permanent Action
  - It would be normal process to determine the permanent action (formally known as dead load).
    - In this example we will not be designing the floor structure but from previous similar contracts a precast concrete plank with voids will be used. Allowance for the weight of the steel structure can be made at this stage, as the designer decides.
      Equivalent permanent load (g_k) is 3.3kN/m^2
- Variable Action
  - Use defines class C41 so Q_k=5kN/m^2
    - Note no allowance for partitions in this case nor is there any allowance for .

Design loading

- Ultimate

$$E_d = 1.35 \times G_k + 1.5 \times Q_k$$

$$E_d = 1.35 \times 3.3 + 1.5 \times 5.0 = 12 \, {}^{kN}\!/_{m^2}$$

Using the design loading for the above it is clear the design udl for beam A1B1 is 1.5mx12kN/m^2=18kN/m^2 and A2B2 is 36kN/m^2

For beam A1A3 the loading will be the shear value from beam A2B2 (126kN) and beam B1B3 will be twice this value (252kN)

- *These values can applied throughout the structure as there is a consistency of dimensions, but this may not be the case in each structure.*

Analyse beams

- Beam A1B1

$$M_{E,d} = \frac{wL^2}{8} or \frac{WL}{8}$$

$$\therefore M_{Ed} = \frac{18 \times 7^2}{8} = 110kNm$$

$$V_{E,d} = \frac{wL}{2} or \frac{W}{2}$$

$$\therefore V_{E,d} = \frac{18 \times 7}{2} = 63kN$$

- BeamA2B2

$$M_{E,d} = \frac{wL^2}{8} or \frac{WL}{8}$$

$$\therefore M_{E,d} = \frac{36 \times 7^2}{8} = 221kNm$$

$$V_{E,d} = \frac{wL}{2} or \frac{W}{2}$$

$$\therefore V_{E,d} = \frac{36 \times 7}{2} = 126kN$$

Both beams have uniform distributed load (udl) hence the use of these standard formulae. Form of analysis not a requirement of Eurocodes.

Design beams

For all beams $M_{CRd} = \frac{W_{ply} \times f_y}{\gamma_{MO}}$ taking f_y=275N/mm^2 & γ_{MO}=1.0

- Beam A1B1
  - Min W_{ply}=400cm^3
  - Use UKB 305x102x33kg/m
  - W_{ply}=483cm^3
  - M_{CRd}=133kNm (83%)
- Beam A2B2
  - Min W_{ply}=804cm^3
  - Use UKB 406x178x54kg/m
  - W_{ply}=930cm^3
  - M_{CRd}=266kNm (83%)

For each of these beams there are other standard beams that could be used but the final decision will depend on normal engineering judgement. There is seldom a "perfect" solution and will be a compromise of many factors. For example, availability of the section, easy of detailing and construction, standardisation, etc.

Design of beams (continued)

M_{CRd} is as previously stated

- Beam A1A3
  - Min W_{ply}=687cm^3
  - Use UKB 406x140x46kg/m
  - W_{ply}=778cm^3
  - M_{CRd}=214kNm (88%)
- Beam B1B3
  - Min W_{ply}=1374cm^3
  - Use UKB 457x191x74kg/m
  - W_{ply}=1458cm^3
  - M_{CRd}=401kNm (94%)

As previously stated there are other beams that could have been chosen. Although at this stage the design is within the permissible capacities these beams are liable to suffer from Lateral Torsional Buckling (LTB) which is covered in *Design of Structures*.

Shear capacity

- A_v=greater of the two calculations.
  - Beam A1B1
    - V_{CRd}=363kN (17%)
  - Beam A2B2
    - V_{CRd}=545kN (23%)

$$A_v = A - bt_f + (t_w + 2r)t_f$$

and

$$A_v = \eta h_w t_w \text{ for simple beams } \eta = 1.0$$

$$V_{CRd} = \frac{A_v f_y}{\sqrt{3}}$$

Serviceability check

- UDL $\partial = \dfrac{5qL^4}{384EI}$
  - Beam A1B1
    - Q=1.5x5=7.5kN/m
    - $I=6501cm^4$
    - δ=17mm>35mm
  - Beam A2B2
    - W=(5x3)=15kN/m
    - $I=18720cm^3$
    - δ=12mm>35mm

In each case maximum deflection has been taken as span/200.

There are different views taken by each NA. For example,

- UK it is acceptable to consider variable action value only and deformation limited to span/360 for brittle finish to flooring and span/200 for non-brittle finish.
- Germany self weight deformation is inversely pre-set and deformation due to variable action defined by the specific needs of the flooring and Client.

STEEL BEAM DESIGN WITH LATERAL TORSIONAL BUCKLING

Overview

- It is recommended the *Design of Structural Elements to Eurocodes – 2nd Edition* by William M C McKenzie would be suitable to review this complex aspect to more complex beam design. This example considers:
  - Definition of buckling
  - Lateral torsion buckling
  - Example
  - Conclusion

Buckling – examples

- Page 343
  - We are concerned with

Note that we will be considering all forms of buckling but not at this stage

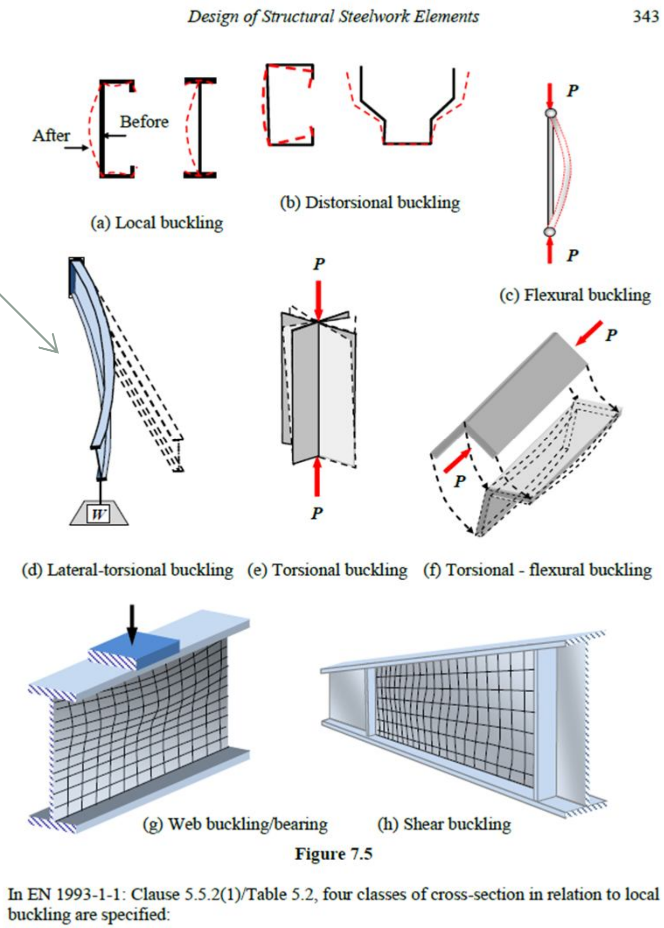

Design of Structural Steelwork Elements 343

Figure 7.5

In EN 1993-1-1: Clause 5.5.2(1)/Table 5.2, four classes of cross-section in relation to local buckling are specified:

Buckling – the theory

7.3.3 Flexural Buckling

Flexural buckling is characterised by out-of-plane movement of the cross-section at the critical load and is the predominant buckling mode in typical building structures using hot-rolled sections. In 1757 the Swiss engineer/mathematician Leonhard Euler developed a theoretical analysis of premature failure due to buckling.

The theory is based on the differential equation of the elastic bending of a pin-ended column which relates the applied bending moment to the curvature along the length of the column. The resulting equation for the fundamental critical load for a pin-ended column is known as the Euler Equation, i.e.

Euler critical buckling load $N_{cr} = \dfrac{\pi^2 EI}{L_{cr}^2}$ Equation (7.4)

where:
L_{cr} is the critical buckling length,
I is the second moment of area about the axis of buckling.
The Euler critical buckling stress and the 'slenderness' can be derived from Equation (7.3) as follows:

$$\sigma_{cr} = \frac{N_{cr}}{A} = \frac{\pi^2 EAi^2}{L_{cr}^2} \times \frac{1}{A} = \frac{\pi^2 Ei^2}{L_{cr}^2} = \frac{\pi^2 E}{L_{cr}^2/i^2} = \frac{\pi^2 E}{\lambda^2}$$ where λ is the slenderness.

This can be re-written such that the slenderness $\lambda = \pi\sqrt{\dfrac{E}{\sigma_{cr}}}$. A graph of critical stress versus slenderness, i.e. the Euler curve, is shown in Figure 7.18.

Buckling – the theory

The critical stress on the Euler curve is limited by the yield stress f_y of the material. The slenderness corresponding with this value is known as the Euler slenderness.

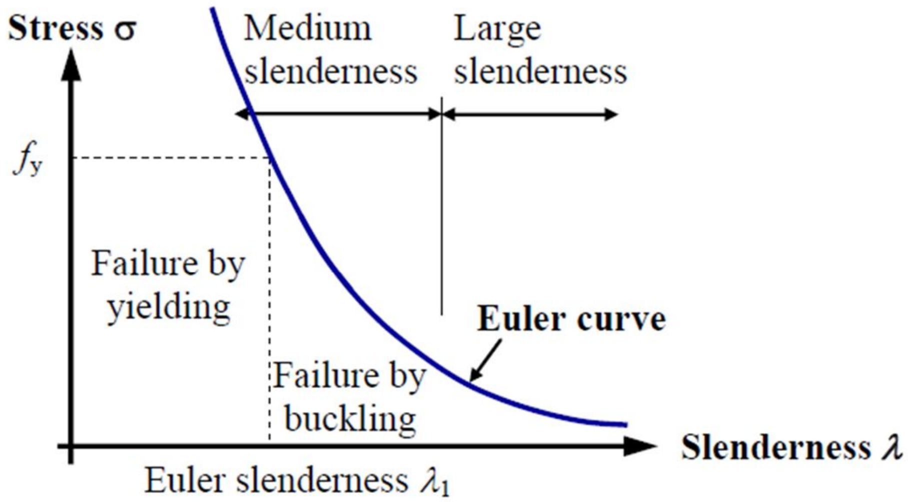


Figure 7.18

EC3 Buckling

stresses as above, e.g. for flexural buckling $\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}}$.

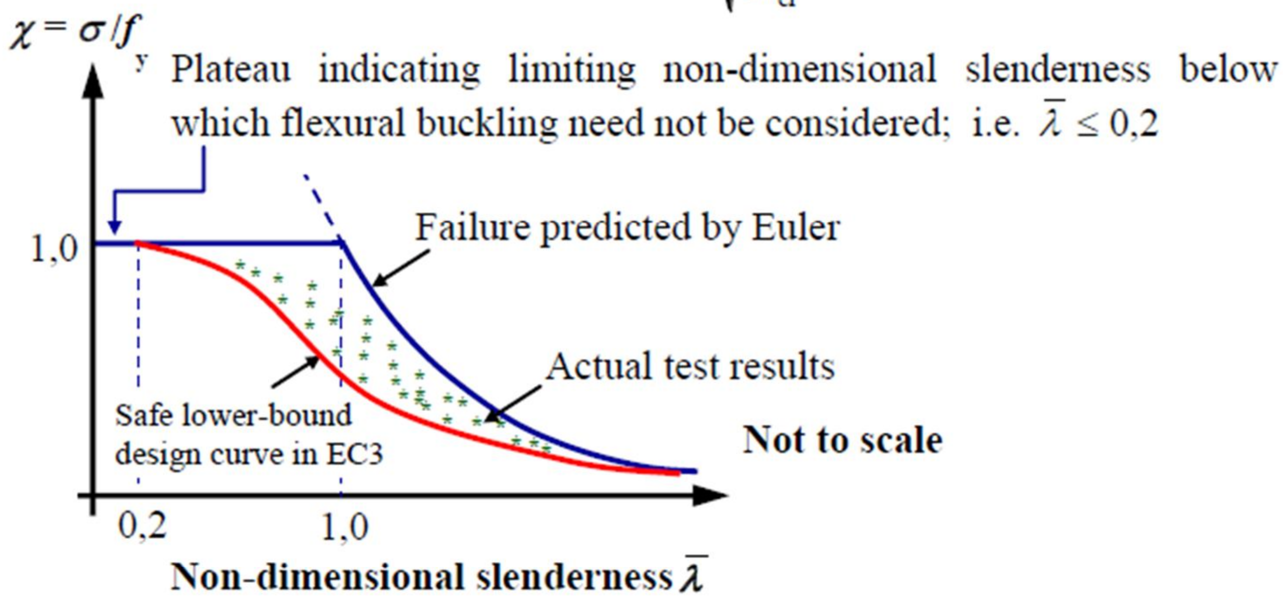


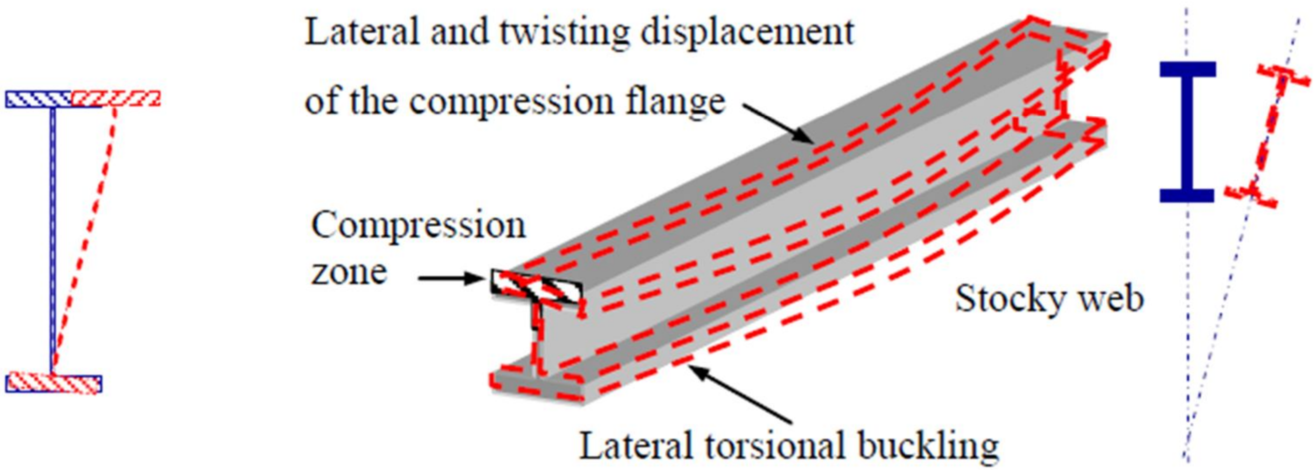
Figure 7.19

The buckling curves given in EC3 are the result of more than 1000 tests on various types of cross-sections with values of slenderness ranging from 55 to 160. The curves include the effects of imperfections such as initial out-of-straightness, residual stresses, eccentricity of applied axial load and strain-hardening.

Lateral Torsional Buckling

When the web is very slender the restraint it provides to the compression flange against buckling is negligible and lateral torsional buckling is very similar to flexural buckling of the flange by bending about the weak axis of the cross-section as shown in Figure 7.27(a).

When the web is 'stocky' it behaves in a similar manner to a rigid plate element, providing more restraint inducing lateral-torsional buckling, resulting in combined torsional and bending deformations as shown in Figure 7.27(b).



(a) very slender web (b) stocky web

Elastic critical moment (page 375)

For uniform doubly-symmetric cross-sections generally, loaded through the shear centre at the level of the centroidal axis, M_{cr} can be determined by modifying the fundamental case as follows:

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{L_{cr}^2}\left(\frac{I_w}{I_z}+\frac{L_{cr}^2 G I_T}{\pi^2 E I_z}\right)^{0,5} \quad \text{Equation (7.36)}$$

where C_1 is a factor to take account of the shape of the bending moment diagram. For the fundamental case with uniform compression along the full unrestrained length of the beam (see Figure 7.28), C_1 = 1,0, and the value of C_1 is greater than 1,0 for all other cases. (Note: k and k_w are assumed to be equal to 1,0 unless a smaller value is justified).

Due to the complexity and a lack of consensus between the contributing European nations regarding the determination of C_1 values, no information is given in EN 1993-1-1. There are numerous publications relating to this problem and acceptable approximations are given in Figure 7.32.

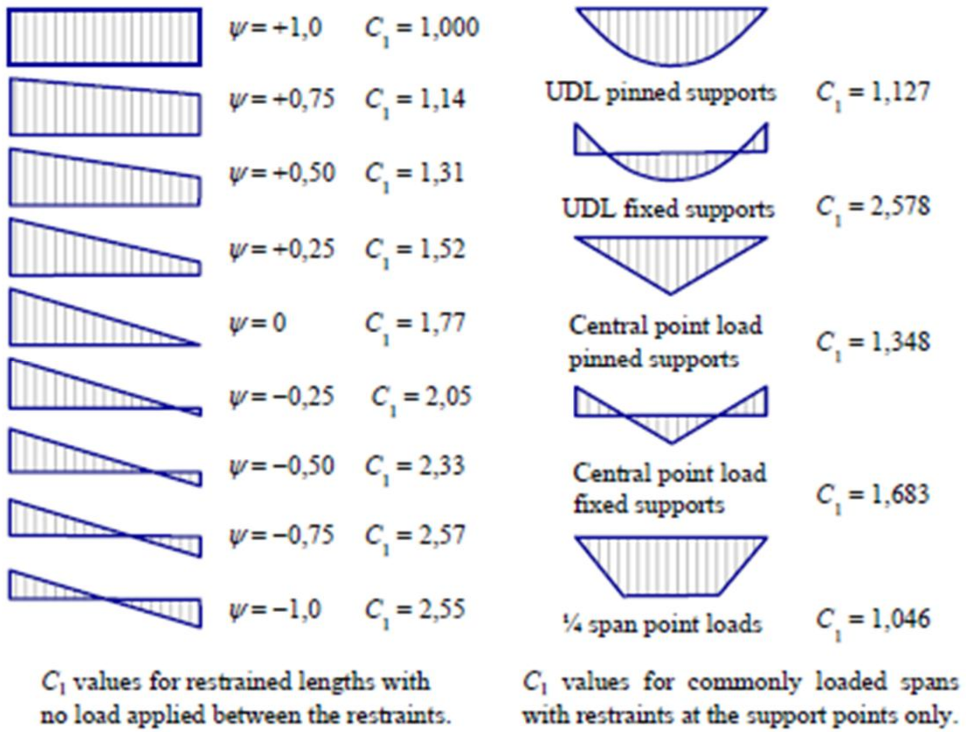


Figure 7.32

Reduction factor χ_{LT} (Chi)

$$\chi_{\mathrm{LT}} = \frac{1}{\Phi_{\mathrm{LT}} + \sqrt{\Phi_{\mathrm{LT}}^2 - \bar{\lambda}_{\mathrm{LT}}^2}} \quad \text{but} \quad \chi_{\mathrm{LT}} \leq 1{,}0$$ EN 1993-1-1: Equation (6.56)

where:

$$\Phi_{\mathrm{LT}} = 0{,}5\left[1 + \alpha_{\mathrm{LT}}\left(\bar{\lambda}_{\mathrm{LT}} - 0{,}2\right) + \bar{\lambda}_{\mathrm{LT}}^2\right] \quad \text{and} \quad \bar{\lambda}_{\mathrm{LT}} = \sqrt{\frac{W_y f_y}{M_{cr}}}$$

α_{LT} is an imperfection factor given in EN 1993-1-1: Table 6.3. The value is dependent on selection of the appropriate buckling curve from Table 6.4.

7.3.4.2 The Method for Hot-rolled Sections or Equivalent Welded Sections

No buckling curves are given in EN 1993-1-1 for this case; The value of χ_{LT} may be determined using EN 19931-1-: Equation (6.57).

$$\chi_{\mathrm{LT}} = \frac{1}{\Phi_{\mathrm{LT}} + \sqrt{\Phi_{\mathrm{LT}}^2 - \beta\bar{\lambda}_{\mathrm{LT}}^2}} \leq 1{,}0$$

$$\leq \frac{1}{\bar{\lambda}_{\mathrm{LT}}^2}$$ EN 1993-1-1: Equation (6.57)

where:

$\bar{\lambda}_{LT,0} = 0{,}4$ (in the UK National Annex $\bar{\lambda}_{LT,0} = 0{,}2$ for welded sections),

$\beta = 0{,}75$ (in the UK National Annex $\beta = 1{,}0$ for equivalent welded sections).

The imperfection factor α_{LT} is obtained from EN 1993-1-1: Table 6.3. The selection of the appropriate buckling curve is given in EN 1993-1-1: Table 6.5. In the UK National Annex EN 1993-1-1: Table 6.5 has been replaced by the Table given in Clause NA.2.17.

Moment capacity - buckling

The value of M_{cr} can also be calculated using the ***LTBeam* software** (available free of charge) which can be downloaded from http://www.cticm.com

The lateral torsional buckling resistance moment $M_{b,Rd}$ as given in EN 1993-1-1: Clause 6.3.2 is taken as the smaller of:

$$\boldsymbol{M}_{b,Rd} = \chi_{LT} W_{pl,y} \frac{f_y}{\gamma_{M1}}$$ for Class 1, Class 2 cross-sections.

$$\boldsymbol{M}_{b,Rd} = \chi_{LT} W_{el,y} \frac{f_y}{\gamma_{M1}}$$ for Class 3 cross-sections.

$$\boldsymbol{M}_{b,Rd} = \chi_{LT} W_{eff,y} \frac{f_y}{\gamma_{M1}}$$ for Class 4 cross-sections.

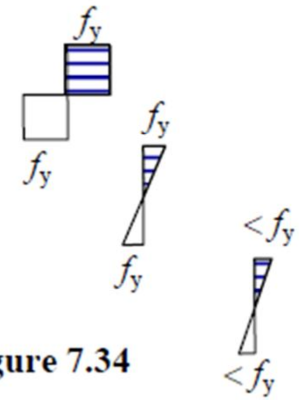


Figure 7.34

where χ_{LT} is the reduction factor for lateral torsional buckling and the W_y value is the appropriate plastic, elastic or effective section modulus. The formulation to calculate the reduction is given in two formats,

(i) a General case which may be applied to all common types of section, i.e. hot-rolled sections, plate girders, castellated beams etc., (EN 1993-1-1: Clause 6.3.2.2 and

(ii) for use with hot-rolled sections or equivalent welded sections, (i.e. with similar dimensions to rolled sections), EN 1993-1-1: Clause 6.3.2.3.

Example 7.16 (page 425)

7.3.10.8 Example 7.14: Design of a Simply Supported Beam with Mid-span and End Restraints Only

A simply supported floor beam in a braced steel industrial frame supports a column as shown in Figure 7.62. Using the design data given and EN 1990:2002: Equation (6.10), verify, or otherwise, the suitability of the proposed steel beam with respect to:

(i) shear resistance,
(ii) bending resistance,
(iii) lateral torsional bucking resistance and
(iv) deflection, (assume the beam is supporting a brittle finish).

Design Data:

Characteristic permanent load in the column (G_k) 20,0 kN
Characteristic variable load in the column (Q_k) 50,0 kN

Conditions of restraint at supports:
Compression flange laterally restrained
Beam fully restrained against torsion
Both flanges are partially restrained against rotation on plan

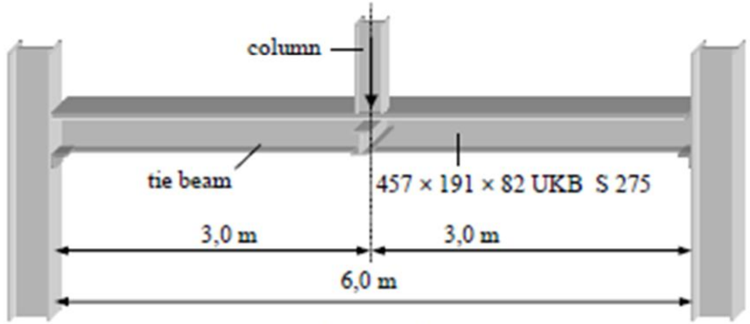


Figure 7.62

Section classification Class 1

457 × 152 × 67 UKB S 275 - Section properties:

$h = 458{,}0$ mm $d = 407{,}6$ mm $b = 153{,}8$ mm $t_w = 9{,}0$ mm $t_f = 15{,}0$ mm
$A_g = 85{,}60$ cm^2 $i_y = 18{,}4$ cm $i_z = 3{,}27$cm $r = 10{,}2$ mm
$I_y = 28900$ cm^4 $I_z = 913$ cm^4 $W_{y,pl} = 1450 \times 10^3$ mm^3 $I_T = 47{,}7$ cm^4
$I_w = 0{,}448$ dm^6 (= $0{,}448 \times 10^{12}$ mm^6)

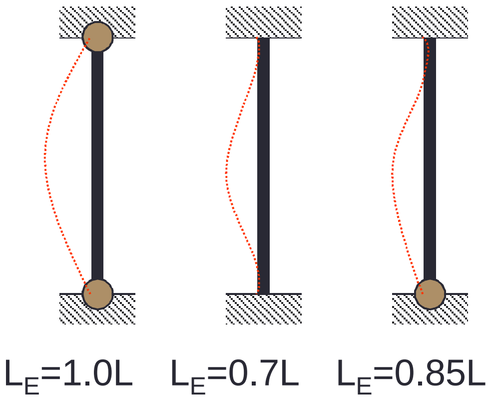
SIMPLE COLUMN DESIGN (COMPRESSION MEMBER)

Why does a column buckle?

- Buckling is when a compression member deforms laterally.
  - Causes
    - Materials inherent resistance to compression
    - Impurities in the member e.g. faults in the material, lack of straightness, minimal non-symmetry, etc.
  - Compounded by
    - Applied bending moment
    - Eccentricity of axial load

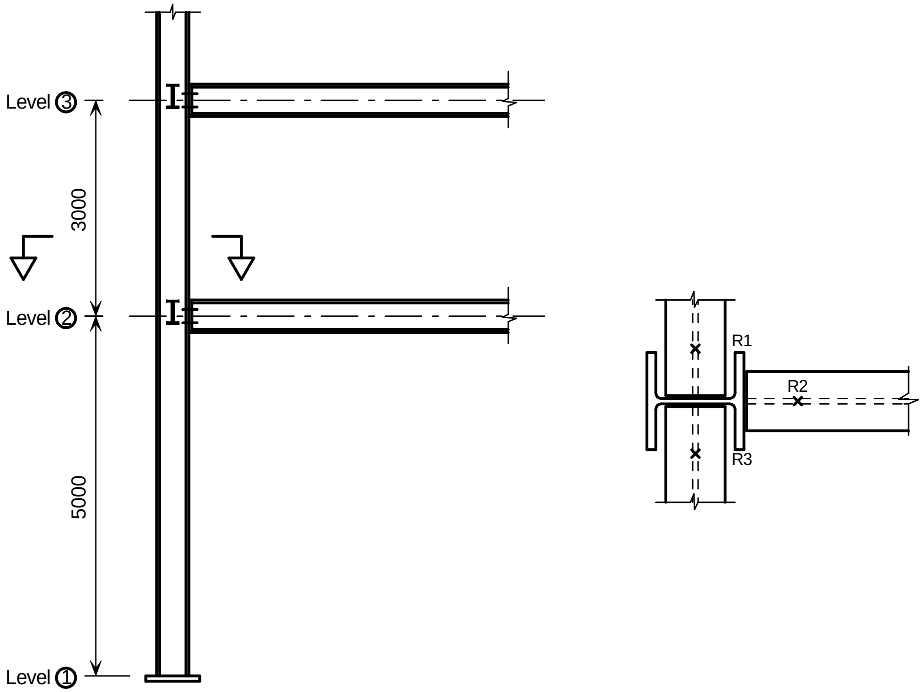
Examples of restraint

- Simple column
  - Pinned at both ends, or
  - Fixed at both ends, or
  - Fixed at one end and pinned at the other
- Continuous
  - Similar to the above
- Cantilever



Simple Column Example

- This example gives the details for the design of an edge column in simple construction.

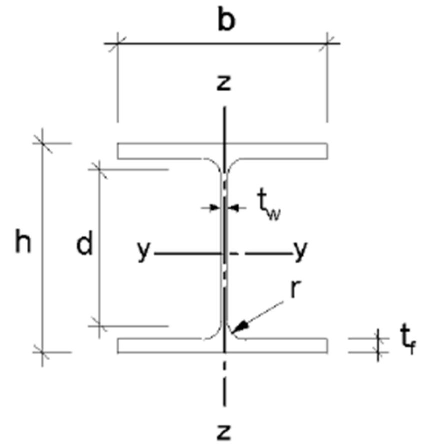


Design of lower section of column

- Axial load from upper levels
  - $N_{2,3\ Ed}$=337kN
- Reaction from beam 1
  - $R_{1,Ed}$=37kN
- Reaction from beam 2
  - $R_{2,Ed}$=147kN
- Reaction from beam 3
  - $R_{3,Ed}$=28kN
- $N_{Ed}= N_{2,3\ Ed}+R_{1,Ed}+R_{2,Ed}+ R_{3,Ed}$=589kN

Chosen Column

UKC 203x203x46$^{kg}/_{m}$



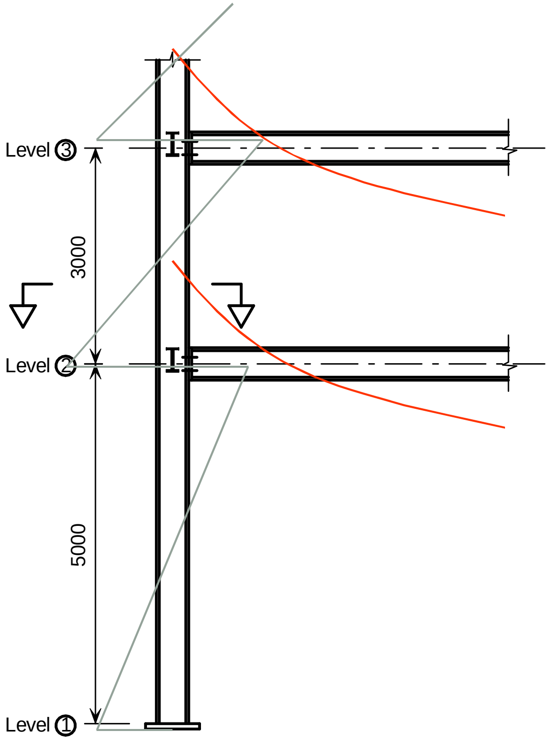
Depth	$h = 203{,}2mm$
Flange thickness	$t_f = 11.0mm$
Web thickness	$t_w = 7.2mm$
Radius of gyration	$i_z = 5.13cm$
Section area	$A = 58{,}7cm^2$
Plastic modulus, y-y axis	$W_{pl,\,y} = 497cm^3$
Plastic modulus, z-z axis	$W_{pl,\,z} = 231cm^3$
Buckling length about y-y axis	$L_{cr,y} = 5.0m$
Buckling length about z-z axis	$L_{cr,z} = 5.0m$

Note: In choosing a column there may be more advantage by using a column to suit incoming beams rather than load bearing capacity.

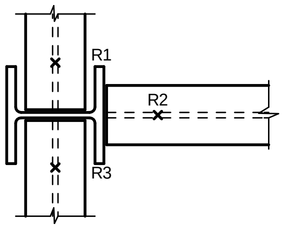
Eccentricity

- Design assumption
  - Simple construction therefore “pinned” connection between beams and column.
    - Use of rigid or semi-rigid connections would induce bending moment due to the distribution.
  - Option 1 make a decision to firmly establish the eccentricity and calculate accordingly.
  - Option 2 make a standard allowance of a 100mm eccentricity plus half the member size.

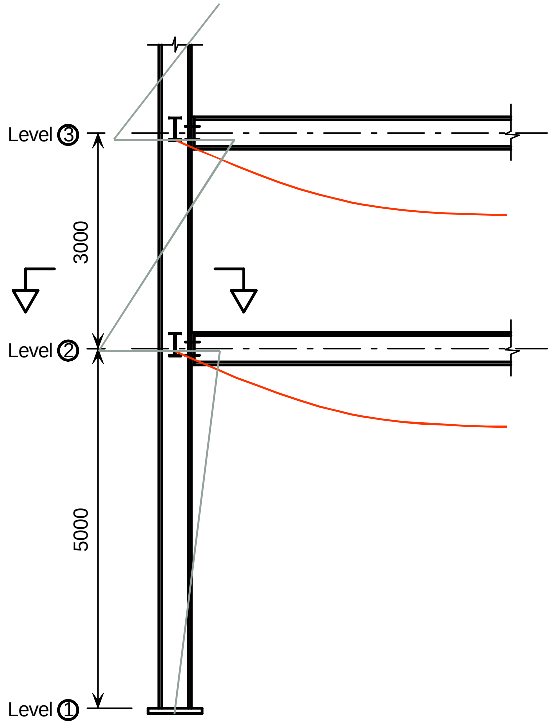
Option 1
All joints are moment resisting



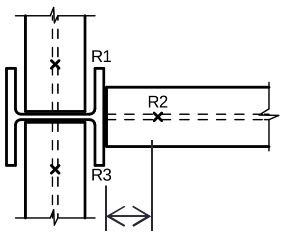
How to analyse the frame to establish the bending moment – Moment Distribution or CAD



Option 2
Simple construction with pinned joints



Analysis of beams basic beam theory. Column will require application of moment distribution theory.

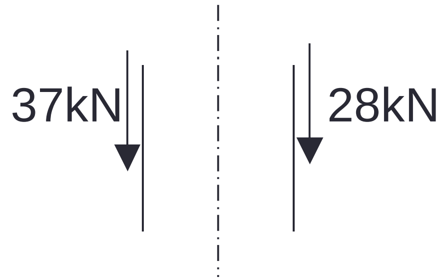


At this stage in the education process Option 2 is the preferred course of action.

Also recommended in industry as moment connections (full or partial) can be difficult to justify economically.

Design bending moments

- $M_{2.y.Ed} = ((h/2)+100)\times R_{2.Ed}$=29.6kNm
- $M_{2.z.Ed} = ((t_w/2)+100)\times(R_{1.Ed}\text{-}R_{3.Ed})$=0.9kNm



- $M_{y.Ed}=M_{2.y.Ed}\times{}^3/_8=11.1kNm$
- $M_{z.Ed}=M_{2.z.Ed}\times{}^3/_8= 0.3kNm$

Flexural Buckling Resistance

- λ_1 = 93.9×ε = 93.9×$(235/275)^{0.5}$ = 86.8
- $\lambda_{z.bar}=(L_{z.cr}/i_z)//\lambda_1=(500/5.13)/86.8=97.5/86.8$ = 1.123
- h/b<1.2 and t_f<100mm, so use buckling curve “c” for the z axis
- Find X_z from tables or graph, X_z = 0.472
- $N_{b.z.Rd} = (X_z \times A \times f_y) / \gamma_{M1} = 0.472 \times 5870 \times 275 \times 10^{-3}/1.0$ = 762kN

LTB Resistance

- Take
  - 0.9× 1.123 = 1.011

$$\bar{\lambda}_{LT} = 0.9\bar{\lambda}_z$$

- Rolled bi-symmetric I-section, h / b ≤ 2, so use buckling curve “b” for the z axis
- Find X_z from tables or graph, $X_z = 0.686$
-

$$M_{\mathrm{b,Rd}} = \chi_{\mathrm{LT}} W_{\mathrm{y}} \frac{f_{\mathrm{y}}}{\gamma_{\mathrm{M1}}} = 0.686 \times 497 \times \frac{275}{1.0} \times 10^{-3} = 93.8\,\mathrm{kNm}$$

Lateral Bending Resistance

- Due to the strength of the section in this direction there is no reduction.

$$\therefore M_{b.z.Rd} = \frac{W_{pl.z} f_y}{\gamma_{M1}} = \frac{231 \times 275 \times 10^{-3}}{1.0} = 63.5 kNm$$

Combined bending & compression buckling (simplified version)

$$\frac{N_{Ed}}{N_{b.z.Rd}} + \frac{M_{y.Ed}}{M_{b.Rd}} + 1.5\frac{M_{z.Ed}}{M_{b.z.Rd}} \leq 1.0$$

$$\frac{589}{762} + \frac{11.1}{93.8} + 1.5\frac{0.3}{63.5}$$

$$= 0.773 + 0.118 + 0.007 = 0.898 \leq 1.0$$

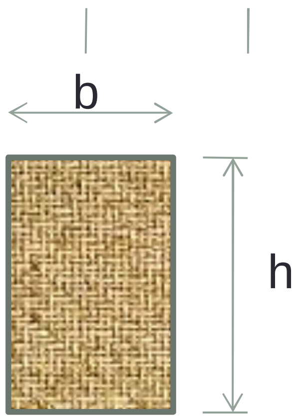
- Therefore a UKC 203 x 203 x 46 is adequate.

BS EN1995 vs. existing code

- Fundamental change from elastic design (BS5268) to the same Limit State design of all other materials.
  - Variability of strengths (parallel/perpendicular to grain, shear, etc.) recognised as a percentage of bending strength.
    - Material strengths contained with a separate code (BS EN338:2003) with no species listed only strength class.
  - Fastener type and reliability to movement to be considered at the early design stage.
  - Good TRADA guidance available and software/ freeware for design and classification
  - Named k-factors rather than numbered (e.g. k_h from height)

Timber or wood

- Wood
  - A generic term for the hard **fibrous** material from trees below the bark.
- Timber
  - Generic term for wood used in furniture, pulp (for paper) and for construction
- Nomenclature



Bending parallel to grain AND shear parallel to grain

Compression parallel to grain
Tension parallel to grain

Design principles

- Actions as define in EN1991
- Factors and principles are the same as were used for steel and concrete.
  - Plastic rather than elastic
  - Factor of safety for permanent action 1.35 and for variable action 1.5.
- Major difference: timber is anisotropic (properties are direction dependant)
  - See Table 1 of BS EN 338

Extract of BS EN338

Table 1 — Strength classes - Characteristic values

		Poplar and softwood species												Hardwood species					
		C14	C16	C18	C20	C22	C24	C27	C30	C35	C40	C45	C50	D30	D35	D40	D50	D60	D70
Strength properties (in N/mm^2)																			
Bending	$f_{m,k}$	14	16	18	20	22	24	27	30	35	40	45	50	30	35	40	50	60	70
Tension parallel	$f_{t,0,k}$	8	10	11	12	13	14	16	18	21	24	27	30	18	21	24	30	36	42
Tension perpendicular	$f_{t,90,k}$	0,4	0,5	0,5	0,5	0,5	0,5	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6
Compression parallel	$f_{c,0,k}$	16	17	18	19	20	21	22	23	25	26	27	29	23	25	26	29	32	34
Compression perpendicular	$f_{c,90,k}$	2,0	2,2	2,2	2,3	2,4	2,5	2,6	2,7	2,8	2,9	3,1	3,2	8,0	8,4	8,8	9,7	10,5	13,5
Shear	$f_{v,k}$	1,7	1,8	2,0	2,2	2,4	2,5	2,8	3,0	3,4	3,8	3,8	3,8	3,0	3,4	3,8	4,6	5,3	6,0
Stiffness properties (in kN/mm^2)																			
Mean modulus of elasticity parallel	$E_{0,mean}$	7	8	9	9,5	10	11	11,5	12	13	14	15	16	10	10	11	14	17	20
5% modulus of elasticity parallel	$E_{0,05}$	4,7	5,4	6,0	6,4	6,7	7,4	7,7	8,0	8,7	9,4	10,0	10,7	8,0	8,7	9,4	11,8	14,3	16,8
Mean modulus of elasticity perpendicular	$E_{90,mean}$	0,23	0,27	0,30	0,32	0,33	0,37	0,38	0,40	0,43	0,47	0,50	0,53	0,64	0,69	0,75	0,93	1,13	1,33
Mean shear modulus	G_{mean}	0,44	0,5	0,56	0,59	0,63	0,69	0,72	0,75	0,81	0,88	0,94	1,00	0,60	0,65	0,70	0,88	1,06	1,25
Density (in kg/m^3)																			
Density	ρ_k	290	310	320	330	340	350	370	380	400	420	440	460	530	560	590	650	700	900
Mean density	ρ_{mean}	350	370	380	390	410	420	450	460	480	500	520	550	640	670	700	780	840	1080

NOTE a Values given above for tension strength, compression strength, shear strength, 5% modulus of elasticity, mean modulus of elasticity perpendicular to grain and mean shear modulus, have been calculated using the equations given in annex A

b The tabulated properties are compatible with timber at a moisture content consistent with a temperature of 20^{0}C and a relative humidity of 65%

c Timber conforming to classes C45 and C50 may not be readily available.

Modification to Material Properties

- Structural timber must be considered for the operational conditions.
  - Consider solid timbers at this stage
  - The assumed reference timber is 150mm cross-sectional dimension.

$$k_h = \left(\frac{150}{h}\right)^{0.2} \text{ or } 1.3$$

    - Where h is the height of the solid section
- Duration of load is important
  - (See Table 3.1)
  - Timber suffers from creep when loaded over a long period.
    - Creep is when a member suffers continued deformation despite the applied actions not changing PLUS when action is removed a permanent set is a consequence.

$$k_h = \min \begin{cases} \left(\dfrac{150}{h}\right)^{0,2} \\ 1,3 \end{cases} \qquad (3.1)$$

where h is the depth for bending members, in mm.

Table 3.1 — Values of k_{mod}

Material	Standard	Service Class	Load-duration class				
			Permanent action	Long term action	Medium term action	Short term action	Instant-aneous action
Solid timber	EN 14081-1	1 2 3	0,60 0,60 0,50	0,70 0,70 0,55	0,80 0,80 0,65	0,90 0,90 0,70	1,10 1,10 0,90
Glued laminated timber	EN 14080	1 2 3	0,60 0,60 0,50	0,70 0,70 0,55	0,80 0,80 0,65	0,90 0,90 0,70	1,10 1,10 0,90
LVL	EN 14374	1 2 3	0,60 0,60 0,50	0,70 0,70 0,55	0,80 0,80 0,65	0,90 0,90 0,70	1,10 1,10 0,90
Plywood	EN 636 Part 1,Part 2,Part 3 Part 2, Part 3 Part 3	 1 2 3	 0,60 0,60 0,50	 0,70 0,70 0,55	 0,80 0,80 0,65	 0,90 0,90 0,70	 1,10 1,10 0,90

Factor for design

- Partial factor from material
  - (Table NA.3)
  - Use 1.3 for solid timber (& 1.25 for GLULAM)
- Design value of material property
  - (Eq2.9)

$$X_d = k_{\mathrm{mod}} \frac{X_k}{\gamma_M} = k_{\mathrm{mod}} \frac{X_k}{1.3}$$

Stiffness properties are NOT affected by k_{mod}.

(See Eq2.10)

- Design resistance
  - (See Eq2.12)

$$R_d = k_{\mathrm{mod}} \frac{R_k}{\gamma_M} = k_{\mathrm{mod}} \frac{R_k}{1.3}$$

Table NA.3 — Partial factors γ_M for material properties and resistances

Fundamental combinations	
Solid timber, untreated	1,3
Solid timber, preservative-treated	1,3
Glued laminated timber	1,25
LVL, plywood, OSB	1,2
Particleboard	1,3
Fibreboards, hard	1,3
Fibreboards, medium	1,3
Fibreboards, MDF	1,3
Fibreboards, soft	1,3
Connections (except for punched metal plate fasteners)	1,3
Punched metal plate fasteners, anchorage strength	1,3
Punched metal plate fasteners, plate (steel) strength	1,15
Accidental combinations	1,0

Load Duration & Service Class

- Load duration

	>10 years	Self weight
Long term	6 months – 10 years	Storage loading (including lofts) water tank
Medium term	1 week to 6 months	Imposed floor loading
Short term	< 1 week	Snow, maintenance or man loading on roofs, etc
Instantaneous		Wind, impact loading, explosions

- Service Class
  - Clause 2.3.1.3
    - **Class 1**: 20% moisture content or less and surrounding air only exceeding 65% for a few week per year.
    - **Class2**: 20% moisture content or less and surrounding air only exceeding 85% for a few week per year.
    - **Class 3**: conditions leading to moisture conditions greater than Class 2.
      - See table NA.2

Table NA.2 — Service classes

Type of construction	Service class
Cold roofs	2
Warm roofs	1
Intermediate floors	1
Ground floors	2
Timber-frame walls, internal and party walls	1
Timber-frame walls, external walls	2
External uses where member is protected from direct wetting	2
External uses, fully exposed	3

2.4 Verification by the partial factor method

Section 3, to which this contains a forward reference, is based on the safety format of EN 1990. The partial factor γ_m for material properties and resistances have a similar basis and format to γ_m values in Eurocodes for other materials. However, materials used in timber structures also require special adjustments. First introduced here is an important modification factor, k_{mod} – the modification taking into account the effect of the duration of load and moisture content. The modification factor k_{mod} is further discussed in the commentary on section 3.

The partial factor for material properties and resistances, γ_M, is given in EN 1995-1-1, Table 2.3 for fundamental combinations (see EN 1990, Section 6) and for accidental

Examples

- A C24 timber is in a Service Class 2 with a Medium term action.
  $$\therefore k_{\text{mod}} = 0.8$$
  - Compressive strength
    $$f_{c,od} = \frac{k_{\text{mod}} f_{c,ok}}{\gamma_M} = \frac{0.8 \times 21}{1.3} = 12.9 \, {}^{N}\!/\!_{mm^2}$$
  - Bending strength
    $$f_{m,y,d} = \frac{k_{\text{mod}} f_{m.y.k}}{\gamma_M} = \frac{0.8 \times 24}{1.3} = 14.8 \, {}^{N}\!/\!_{mm^2}$$
- For compression members there is a check necessary for buckling.
  - Relative slenderness as quoted in Clause 6.3.2
    - Note for a rectangular section

$$\lambda_y = \frac{L_{Ey}}{i_y} \text{ where } i_y = \sqrt{\frac{I_y}{A}} = \sqrt{\left(\frac{bh^3}{12}\right) \times \left(\frac{1}{bh}\right)} = \frac{h}{\sqrt{12}}$$

$$\therefore \lambda_y = \frac{L_{Ey} \times \sqrt{12}}{h}$$

Example of a beam

- A timber beam is required to carry a design bending moment (M_{Ed}) of 6.7kNm and a shear (V_{Ed}) of 12.8kN. Using C16 timber what is the best size to provide adequate resistance to:
  - Bending
  - Shear
    - Do not consider deformation OR vibration at this stage.

Example of a beam (cont'd)

- Although the design is considered "plastic" the elastic modulus of the section is used.
  - $M_{cR}=W_{el}f_{m,d}$
  - For C16 solid rectangular timber.

$$f_{m.d} = \frac{k_{\text{mod}} f_{m,d}}{\gamma_M} = \frac{0.8 \times 16}{1.3} = 9.8 \, {}^{N}\!/\!{}_{mm^2}$$

$$f_{v,d} = \frac{k_{\text{mod}} f_{v,d}}{\gamma_M} = \frac{0.8 \times 1.8}{1.3} = 1.1 \, {}^{N}\!/\!{}_{mm^2}$$

  - To prevent lateral torsional buckling of the beam keep the breadth to height ratio to no more than 1 to 5.

Example of beam (cont'd.)

$$W_{el} = \frac{bh^2}{6}$$

$$\text{If h} = \text{5b}$$

$$W_{el} = \frac{b(5b)^2}{6} = \frac{25b^3}{6}$$

$$W_{el} = \frac{M_{Ed}}{f_{m,d}} = \frac{25b^3}{6} \text{ or } b = \sqrt[3]{\frac{6M_{Ed}}{25\,f_{m,d}}} = \sqrt[3]{\frac{6\times 6.7\times 10^6}{25\times 9.8}} = 54.7mm$$

From this information 60 wide by 300 high would provide resistance $M_{R,d}$ of 8.82kNm (76%)

Check for shear.

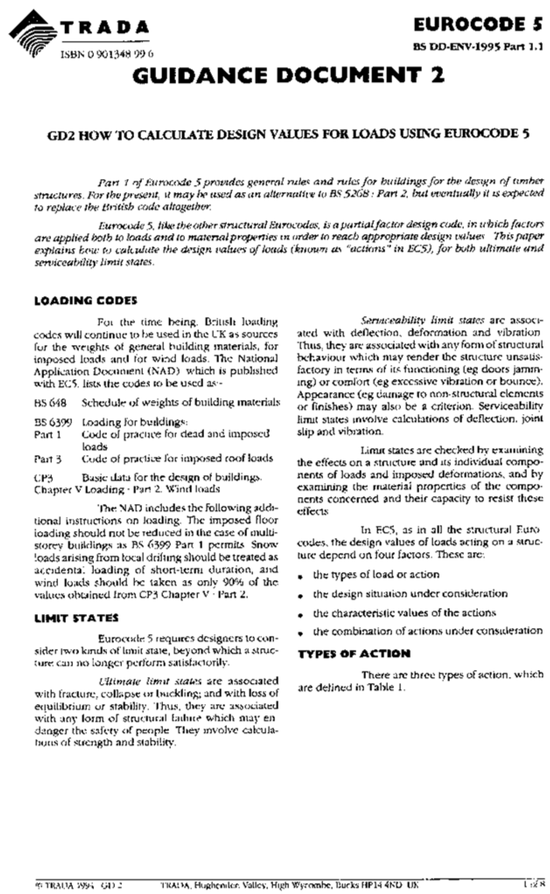
Maximum shear stress for a rectangular section is 1.5 times the average shear stress.

$$\sigma_{\mathrm{v,d}} = \frac{1.5 \times V_{E,d}}{XSA} = \frac{1.5 \times 1.8 \times 10^3}{60 \times 300} = 1.07\, N/mm^2 \langle 1.1\, N/mm^2$$

The section is satisfactory, but only just!
The designer should decide to accept or change the section size – so what do you think?

Load share and LT buckling

- Correction factors for load sharing and for sections where buckling is not critical.
  - Load share
    - k_{ls}=1.1 for spans not greater than 6m and the attached flooring is continuous over at least 2 spans and joints are staggered.
  - Lateral Torsional Buckling
    - k_{crit} is a reduction factor to allow for buckling so 1.0



TRADA

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EUROCODE 5

BS DD-ENV-1995 Part 1.1

GUIDANCE DOCUMENT 2

GD2 HOW TO CALCULATE DESIGN VALUES FOR LOADS USING EUROCODE 5

Part 1 of Eurocode 5 provides general rules and rules for buildings for the design of timber structures. For the present, it may be used as an alternative to BS 5268 : Part 2, but eventually it is expected to replace the British code altogether.

Eurocode 5, like the other structural Eurocodes, is a partial factor design code, in which factors are applied both to loads and to material properties in order to reach appropriate design values. This paper explains how to calculate the design values of loads (known as "actions" in EC5), for both ultimate and serviceability limit states.

LOADING CODES

For the time being, British loading codes will continue to be used in the UK as sources for the weights of general building materials, for imposed loads and for wind loads. The National Application Document (NAD) which is published with EC5, lists the codes to be used as:-

BS 648 Schedule of weights of building materials

BS 6399 Loading for buildings:
Part 1 Code of practice for dead and imposed loads
Part 3 Code of practice for imposed roof loads

CP3 Basic data for the design of buildings. Chapter V Loading - Part 2. Wind loads

The NAD includes the following additional instructions on loading. The imposed floor loading should not be reduced in the case of multi-storey buildings as BS 6399 Part 1 permits. Snow loads arising from local drifting should be treated as accidental loading of short-term duration, and wind loads should be taken as only 90% of the values obtained from CP3 Chapter V - Part 2.

LIMIT STATES

Eurocode 5 requires designers to consider two kinds of limit state, beyond which a structure can no longer perform satisfactorily.

Ultimate limit states are associated with fracture, collapse or buckling; and with loss of equilibrium or stability. Thus, they are associated with any form of structural failure which may endanger the safety of people. They involve calculations of strength and stability.

Serviceability limit states are associated with deflection, deformation and vibration. Thus, they are associated with any form of structural behaviour which may render the structure unsatisfactory in terms of its functioning (eg doors jamming) or comfort (eg excessive vibration or bounce). Appearance (eg damage to non-structural elements or finishes) may also be a criterion. Serviceability limit states involve calculations of deflection, joint slip and vibration.

Limit states are checked by examining the effects on a structure and its individual components of loads and imposed deformations, and by examining the material properties of the components concerned and their capacity to resist these effects.

In EC5, as in all the structural Eurocodes, the design values of loads acting on a structure depend on four factors. These are:

- the types of load or action
- the design situation under consideration
- the characteristic values of the actions
- the combination of actions under consideration

TYPES OF ACTION

There are three types of action, which are defined in Table 1.

Shear

There are factors associated with notched ends.

For notch on opposite side of support

k_v=1.0

For notch on same side!

$$f_{v,d} = \frac{k_{\mathrm{mod}} \times k_{ls} \times f_{v,k}}{\gamma_m}$$

Modification Factor for Shear of timber beam with notched end

Height of main beam (h)	196	mm	kn	5	
height of notched end (heff)	80	mm	alpa	0.408	
Length of notch	70	mm	x	90	mm
h-heff	116	mm	Breadth	50	
i	0.603		kv	0.6715	
Design shear stress	2.5	N/mm2			
Shear Capacity	6.7	kN			

– For beams notched on the same side as the support (see Figure 6.11a)

$$k_v = \min\left\{\begin{array}{l} 1 \\ \dfrac{k_n\left(1+\dfrac{1,1\, i^{1,5}}{\sqrt{h}}\right)}{\sqrt{h}\left(\sqrt{\alpha(1-\alpha)}+0,8\dfrac{x}{h}\sqrt{\dfrac{1}{\alpha}-\alpha^2}\right)} \end{array}\right. \quad (6.6$$

where:

i is the notch inclination (see Figure 6.11a);
h is the beam depth in mm;
x is the distance from line of action of the support reaction to the corner of the notch;

$$\alpha = \frac{h_{ef}}{h}$$

$$k_n = \begin{cases} 4,5 & \text{for LVL} \\ 5 & \text{for solid timber} \\ 6,5 & \text{for glued laminated timber} \end{cases} \quad (6.6$$

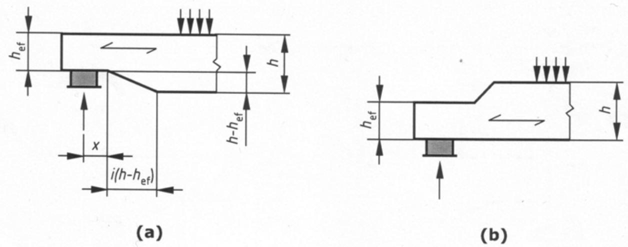

Figure 6.11 — End-notched beams