EN 1991 :EUROCODE 1
Actions on Structures

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EN 1991

EUROCODE 1 : ACTIONS ON STRUCTURES	
EN 1991-1-1	Densities, self weight and imposed loads
EN 1991-1-2	Actions on structures exposed to fire
EN 1991-1-3	Snow loads
EN 1991-1-4	Wind loads
EN 1991-1-5	Thermal loads
EN 1991-1-6	Actions during execution
EN 1991-1-7	Accidental actions
EN 1991-2	Traffic loads on bridges
EN 1991-3	Actions induced by cranes and machinery
EN 1991-4	Silos and tanks

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EN 1991-1-1

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Annex B	Vehicle barriers and parapets for car parks

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Density of construction materials

Table A.1 - Construction materials-concrete and mortar

Materials	Density γ [kN/m^3]
concrete (see EN 206)	
lightweight	
density class LC 1,0	9,0 to 10,0 [1)2)]
density class LC 1,2	10,0 to 12,0 [1)2)]
density class LC 1,4	12,0 to 14,0 [1)2)]
density class LC 1,6	14,0 to 16,0 [1)2)]
density class LC 1,8	16,0 to 18,0 [1)2)]
density class LC 2,0	18,0 to 20,0 [1)2)]
normal weight	24,0 [1)2)]
heavy weight	> [1)2)]
mortar	
cement mortar	19,0 to 23,0
gypsum mortar	12,0 to 18,0
lime-cement mortar	18,0 to 20,0
lime mortar	12,0 to 18,0

1) Increase by 1kN/m^3 for normal percentage of reinforcing and pre-stressing steel
2) Increase by 1kN/m^3 for unhardened concrete
NOTE See Section 4

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Density of construction materials

Table A.4 - Construction materials-metals

Materials	Density γ [kN/m^3]
metals	
aluminium	27,0
brass	83,0 to 85,0
bronze	83,0 to 85,0
copper	87,0 to 89,0
iron, cast	71,0 to 72,5
iron, wrought	76,0
lead	112,0 to 114,0
steel	77,0 to 78,5
zinc	71,0 to 72,0

Density of stored materials

Table A.7 - Stored materials - building and construction

Materials	Density γ [kN/m^3]	Angle of repose ϕ [°]
aggregates (see prEN 206)		
lightweight	9,0 to 20,0 [1)]	30
normal	20,0 to 30,0	30
heavyweight	> 30,0	30
gravel and sand, bulked	15,0 to 20,0	35
sand	14,0 to 19,0	30
blast furnace slag		
lumps	17,0	40
granules	12,0	30
crushed foamed	9,0	35
brick sand, crushed brick, broken bricks	15,0	35
vermiculite		
exfoliated, aggregate for concrete	1,0	-
crude	6,0 to 9,0	-

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Imposed load

Table 6.2 - Imposed loads on floors, balconies and stairs in buildings

Categories of loaded areas	q_k [kN/m^2]	Q_k [kN]
Category A		
- **Floors**	1,5 to2,0	2,0 to 3,0
- **Stairs**	2,0 to4,0	2,0 to 4,0
- **Balconies**	2,5 to 4,0	2,0 to 3,0
Category B	2,0 to 3,0	1,5 to 4,5
Category C		
- C1	2,0 to 3,0	3,0 to 4,0
- C2	3,0 to 4,0	2,5 to 7,0 (4,0)
- C3	3,0 to 5,0	4,0 to 7,0
- C4	4,5 to 5,0	3,5 to 7,0
- C5	5,0 to 7,5	3,5 to 4,5
category D		
- D1	4,0 to 5,0	3,5 to 7,0 (4,0)
- D2	4,0 to 5,0	3,5 to 7,0

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Table NA3. Imposed loads on floors, balconies and stairs in buildings

Category of loaded area		q_k (kN/m^2)	Q_k (kN)
Category A	A1	1.5	2.0
	A2	1.5	2.0
	A3	2.0	2.0
	A4	2.0	2.7
	A5	2.5	2.0
	A6	Same as the rooms to which they give access but with a minimum of 3.0	2.0 (concentrated at the outer edge)
	A7	Same as the rooms to which they give access but with a minimum of 4.0	2.0 (concentrated at the outer edge)
Category B	B1	2.5	2.7
	B2	3.0	2.7
Category C	C11	2.0	3.0
	C12	2.5	4.0
	C13	3.0	3.0
	C21	4.0	3.6
	C22	3.0	2.7

Table 1.4: Imposed load on floors in buildings

Category of loaded area	
A Area for domestic and residential activities	A1: All usages within self-contained dwelling units
	A2: Bedrooms and dormitories
	A3 : Bedrooms in hotels, hospital
B Office area	B1: General use
	B2: At or below ground floor level
C Areas where people may congregate	C11: Areas with tables – dining rooms
	C13: Areas with tables -Classrooms
	C22: Area with fixed seat – Assembly areas
D Shopping areas	D1: Areas in general retail shops

Table NA2. Categories for residential, social, commercial and administration areas including additional sub-categories for Malaysia

Category of loaded area	Specific use	Sub-category	Examples
A	Areas for domestic and residential activities	A1	All usages within self-contained dwelling units (a unit occupied by a single family or a modular student accommodation unit with a secure door and comprising not more than six single bedrooms and an internal corridor) Communal areas (including kitchens) in blocks of flats with limited use (see Note 1). For communal areas in other blocks of flats, see sub-categories A5, A6 and C3
		A2	Bedrooms and dormitories except those in self-contained single family dwelling units and in hotels and motels
		A3	Bedrooms in hotels and motels; hospital wards; toilet areas
		A4	Billiard/snooker rooms
		A5	Balconies in single family dwelling units and communal areas in blocks of flats with limited use (see Note 1)
		A6	Balconies in hostels, guest houses, residential clubs and communal areas in blocks of flats except those covered by Note 1
		A7	Balconies in hotels and motels
B	Office areas	B1	General use other than in B2
		B2	At or below ground floor level
C	Areas where people may congregate (with	C1	Areas with tables
		C11	Public, institutional and communal dining rooms and lounges, cafes and restaurants (see Note 2)
		C12	Reading rooms with no book storage

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Load values with various intensities of people loading Imposed load

1.5 kN/m² : 8 persons in a space 4m²

2.0 kN/m² : 11 persons in a space 4m²

3.0 kN/m² : 17 persons in a space 4m²

4.0 kN/m² : 22 persons in a space 4m²

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5.0 kN/m² : 28 persons in a space 4m²

EN 1992:
EUROCODE 2: Design of Concrete Structures

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Introduction

EUROCODE 2 : DESIGN OF CONCRETE STRUCTURES	
EN 1992-1-1	General rules and rules for buildings
EN 1992-1-2	General rules – Structural fire design
EN 1992-2	Concrete bridges – design and detailing rules
EN 1992-3	Liquid retaining and containment structures

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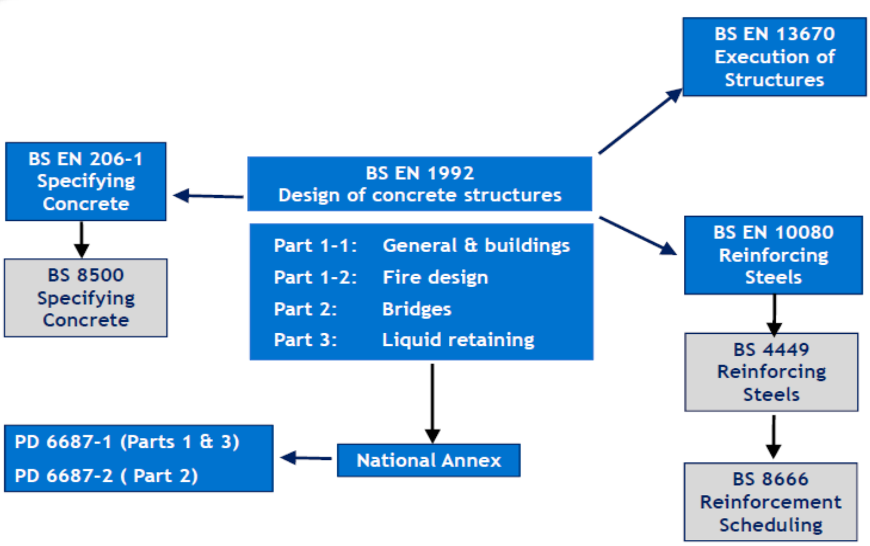

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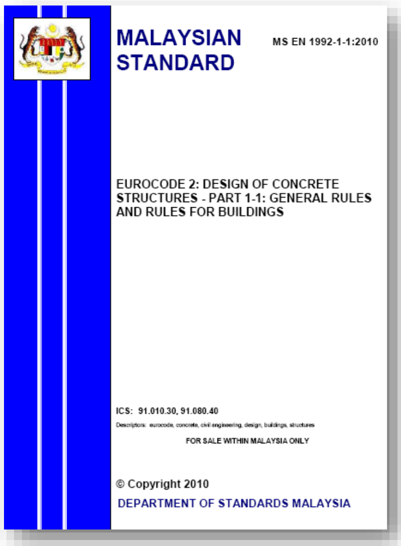
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MS EN 1992-1-1:2010

EUROCODE 2: DESIGN OF CONCRETE STRUCTURES - PART 1-1: GENERAL RULES AND RULES FOR BUILDINGS

ICS: 91.010.30, 91.080.40

Descriptors: eurocode, concrete, civil engineering, design, buildings, structures

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Contents:

- ❑ EC2 is generally laid out to give advice on the basis of phenomena (e.g. bending, shear etc) rather than by member type as in BS 8110 (e.g. beams, slabs, columns etc).
- ❑ Design is based on characteristic cylinder strength not cube strength.
- ❑ EC2 does not provide derived formulae (e.g. for bending only the details of the stress block are expressed)

Table 1.4: Concrete strength classes and modulus of elasticity

Concrete strength class	Characteristic cylinder strength f_{ck} (N/mm^2)	Characteristic cube strength $f_{ck,cube}$ (N/mm^2)	Modulus of elasticity E_{cm} (kN/mm^2)
C20/25	20	25	30
C25/30	25	30	31
C30/37	30	37	33
C35/45	35	45	34
C40/50	40	50	35
C45/55	45	55	36
C50/55	50	60	37
C55/67	55	67	38
C60/75	60	75	39

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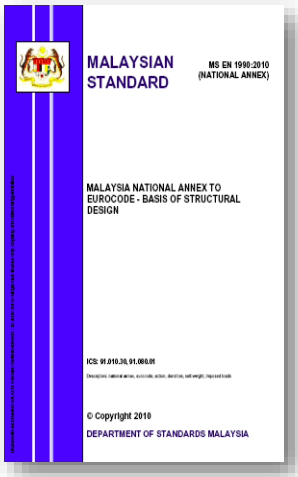
- Units for stress are mega pascals, MPa
  (1 MPa = 1 N/mm^2)
- EC2 uses comma for a decimal point.
- One thousandth is represent by ‰.
- The partial safety factor for steel reinforcement is 1.15. The characteristic yield strength is 500 Mpa.
- There is no guidance on plain steel bar or mild steel reinforcement.

MALAYSIAN NATIONAL ANNEXES



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MS EN 1990:2010 (NATIONAL ANNEX)

MALAYSIA NATIONAL ANNEX TO EUROCODE - BASIS OF STRUCTURAL DESIGN

ICS: 91.010.30, 91.080.01

Descriptors: national annex, eurocode, action, densities, self weight, imposed loads

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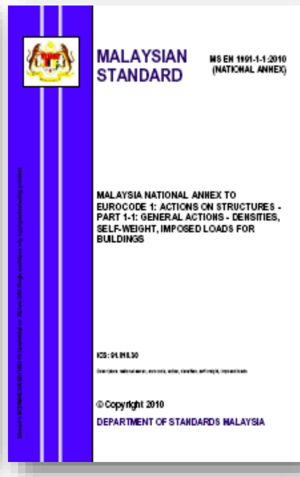

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MS EN 1991-1-1:2010 (NATIONAL ANNEX)

MALAYSIA NATIONAL ANNEX TO EUROCODE 1: ACTIONS ON STRUCTURES - PART 1-1: GENERAL ACTIONS - DENSITIES, SELF-WEIGHT, IMPOSED LOADS FOR BUILDINGS

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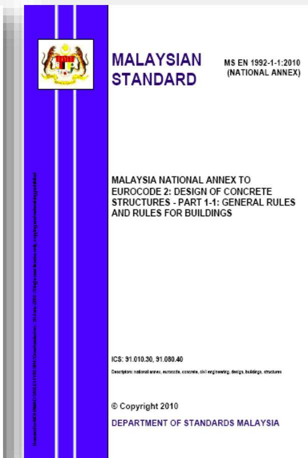

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MS EN 1992-1-1:2010 (NATIONAL ANNEX)

MALAYSIA NATIONAL ANNEX TO EUROCODE 2: DESIGN OF CONCRETE STRUCTURES - PART 1-1: GENERAL RULES AND RULES FOR BUILDINGS

ICS: 91.010.30, 91.080.40

Descriptors: national annex, eurocode, concrete, civil engineering, design, buildings, structures

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- Technical Committee (IEM-SWO) for Standards in Design of Concrete Structures, recommends the adoption of EC2 in totality with national annex & NCCI and the relevant parts of EC0, EC1 and EC2.
- Eurocodes allow for country to make their national choices where standard gives values with notes.

- The committee have prepared Malaysian National Annexes to MS EN 1990: 2009, MS 1991-1-1: 2009 and MS EN 1992-1-1: 2009.
- Hence we have these National Annexes which contain Malaysia Determined Parameters for use in design of buildings and civil engineering works.

Table A1.1: Recommended values of ψ factors for buildings

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings (see EN 1991-1-1)			
Category A: domestic, residential areas	0.7	0.5	0.3
Category B: office areas	0.7	0.5	0.3
Category C: congregation areas	0.7	0.7	0.6
Category D: shopping areas	0.7	0.7	0.6
Category E: storage areas	1.0	0.9	0.8
Category F: traffic area, vehicle weight < 30 kN	0.7	0.7	0.6
Category G: traffic area, 30 kN < vehicle weight < **200 kN**	0.7	0.5	0.3
Category H: roof (see EN 1991-1-1: Cl. 3.3.2)	0.7	0	0
Wind loads on buildings (see MS 1553: 2002)	0.5	0.7	0.7
Temperature (non-fire) in buildings (see EN 1991-1-5)	0.6	0.7	0.7
* **See also MS EN 1991-1-1: Clause 3.3.2(1)**			

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Table NA1. Indicative design working life

Design working life category	Indicative design working life (years)	Examples
1	10	Temporary structures[a]
2	10 to 30	Replaceable structural parts, e.g. gantry girders, bearings
3	15 to 25	Agricultural and similar structures
4	50	Building structures and other common structures, not listed elsewhere in this table
5	120	Monumental building structures, highway and railway bridges, and other civil engineering structures

[a] Structures or parts of structures that can be dismantled with a view of being re-used should not be considered as temporary.

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Table 2.1 - Indicative design working life

Design working life category	Indicative design working life (years)	Examples
1	10	Temporary structures (1)
2	10 to 25	Replaceable structural parts, e.g. gantry girders, bearings
3	15 to 30	Agricultural and similar structures
4	50	Building structures and other common structures
5	100	Monumental building structures, bridges, and other civil engineering structures

(1) Structures or parts of structures that can be dismantled with a view to being re-used should not be considered as temporary.

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Table NA1. Malaysia decisions for Nationally Determined Parameters described in MS EN 1992-1-1:2010

Subclauses	Nationally Determined Parameter	Eurocode recommendation	Malaysia decision
2.3.3 (3)	Value of d_{joint}	30 m	Use the recommended value.
2.4.2.1 (1)	Partial factor for shrinkage action γ_{SH}	1.0	Use the recommended value.
2.4.2.2 (1)	Partial factor for prestress $\gamma_{P,fav}$	1.0	0.9
2.4.2.2 (2)	Partial factor for prestress $\gamma_{P,unfav}$	1.3	1.1
2.4.2.2 (3)	Partial factor for prestress $\gamma_{P,unfav}$ for local effects	1.2	Use the recommended value.
2.4.2.3 (1)	Partial factor for fatigue loads $\gamma_{F,fat}$	1.0	Use the recommended value.
2.4.2.4 (1)	Partial factors for materials for ultimate limit states γ_C and γ_S	Table 2.1N	Use the recommended values.
2.4.2.4 (2)	Partial factors for materials for serviceability limit states γ_C and γ_S	1.0	Use the recommended value.
2.4.2.5 (2)	Value of k_f	1.1	Use the recommended value.
3.1.2 (2)P	Value of C_{max}	C90/105	Use the recommended value. However, the shear strength of concrete classes higher than C50/60 should be determined by tests, unless there is evidence of satisfactory past performance of the particular mix including the type of aggregates used. Alternatively, shear strength of concrete strength classes higher than C50/60 may be limited to that of C50/60.
3.1.2 (4)	Value of k_t	0.85	1.0
3.1.6 (1)P	Value of α_{cc}	1.0	0.85 for compression in flexure and axial loading and 1.0 for other phenomena. However, α_{cc} may be taken conservatively as 0.85 for all phenomena.

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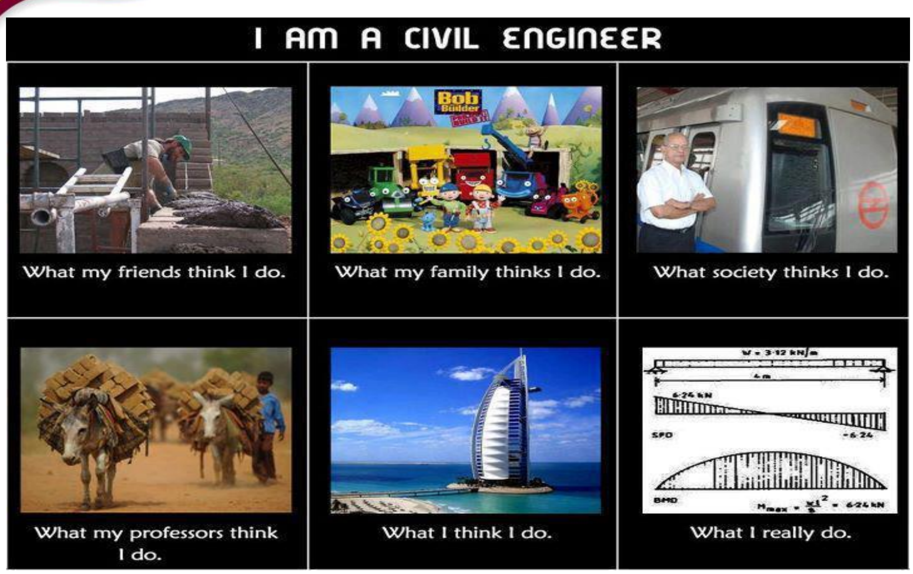

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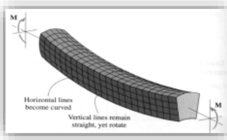
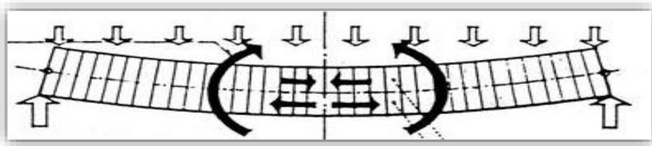
DESIGN FOR BENDING





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Introduction

Section 6.1 EC2 Part 1.1, deals with the analysis and design of section for the ultimate limit state design consideration of structural elements subjected to bending.

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SECTION 6 ULTIMATE LIMIT STATES (ULS)

6.1 Bending with or without axial force

(1)P This section applies to undisturbed regions of beams, slabs and similar types of members for which sections remain approximately plane before and after loading. The discontinuity regions of beams and other members in which plane sections do not remain plane may be designed and detailed according to 6.5.

(2)P When determining the ultimate moment resistance of reinforced or prestressed concrete cross-sections, the following assumptions are made:

- plane sections remain plane.
- the strain in bonded reinforcement or bonded prestressing tendons, whether in tension or in compression, is the same as that in the surrounding concrete.
- the tensile strength of the concrete is ignored.
- the stresses in the concrete in compression are derived from the design stress/strain relationship given in 3.1.7.
- the stresses in the reinforcing or prestressing steel are derived from the design curves in 3.2 (Figure 3.8) and 3.3 (Figure 3.10).
- the initial strain in prestressing tendons is taken into account when assessing the stresses in the tendons.

(3)P The compressive strain in the concrete shall be limited to ε_{cu2}, or ε_{cu3}, depending on the stress-strain diagram used, see 3.1.7 and Table 3.1. The strains in the reinforcing steel and the prestressing steel shall be limited to ε_{ud} (where applicable); see 3.2.7 (2) and 3.3.6 (7) respectively.

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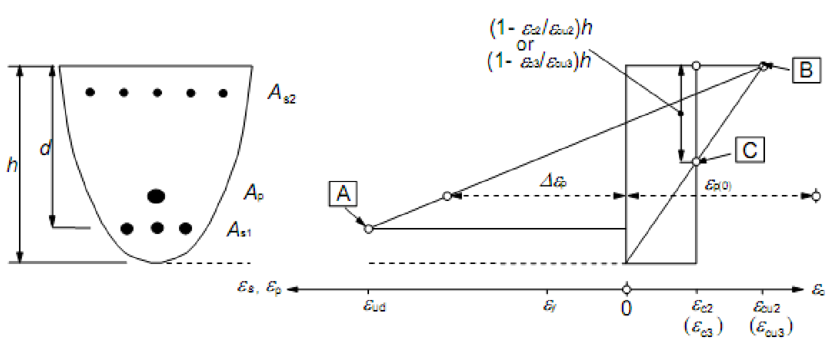


A - reinforcing steel tension strain limit

B - concrete compression strain limit

C - concrete pure compression strain limit

Figure 6.1: Possible strain distributions in the ultimate limit state

$\lambda = 0,8$ for $f_{ck} \leq 50$ MPa (3.19)
$\lambda = 0,8 - (f_{ck} - 50)/400$ for $50 < f_{ck} \leq 90$ MPa (3.20)

and

$\eta = 1,0$ for $f_{ck} \leq 50$ MPa (3.21)
$\eta = 1,0 - (f_{ck} - 50)/200$ for $50 < f_{ck} \leq 90$ MPa (3.22)

Note: If the width of the compression zone decreases in the direction of the extreme compression fibre, the value ηf_{cd} should be reduced by 10%.

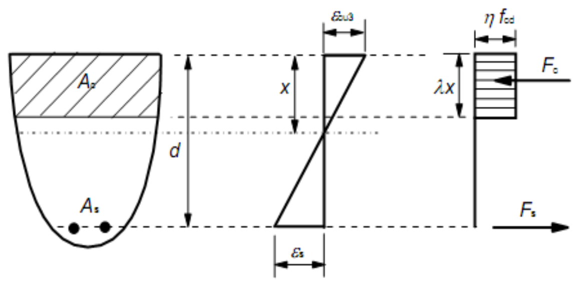
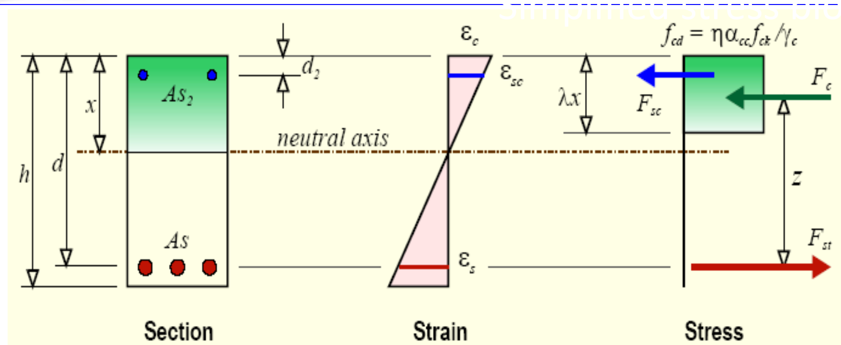


Figure 3.5: Rectangular stress distribution



f_{ck} = characteristic concrete cylinder strength (*equivalent to 80% cube strength*).

For $f_{ck} < 50$ N/mm^2: $\eta = 1$ (defining the effective strength),
$\varepsilon_c = 0.0035$, $\alpha_{cc} = 0.8$ to 1.0 (account for long term effect),
$\alpha_{cc} = 0.85$, $\lambda = 0.8$, $\gamma_c = 1.5$ so, $f_{cd} = 1.0 \times 0.85/1.5 = 0.567 f_{ck}$

Singly reinforced

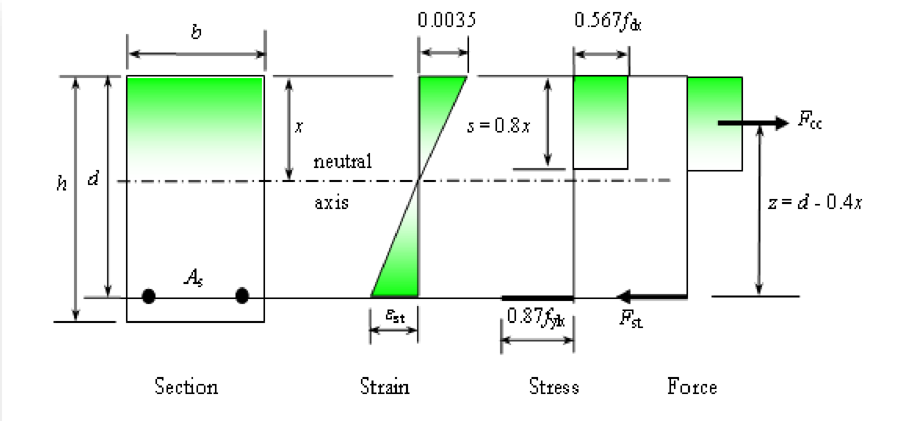


Figure 2.1 Singly reinforced section with rectangular stress block

Doubly reinforced

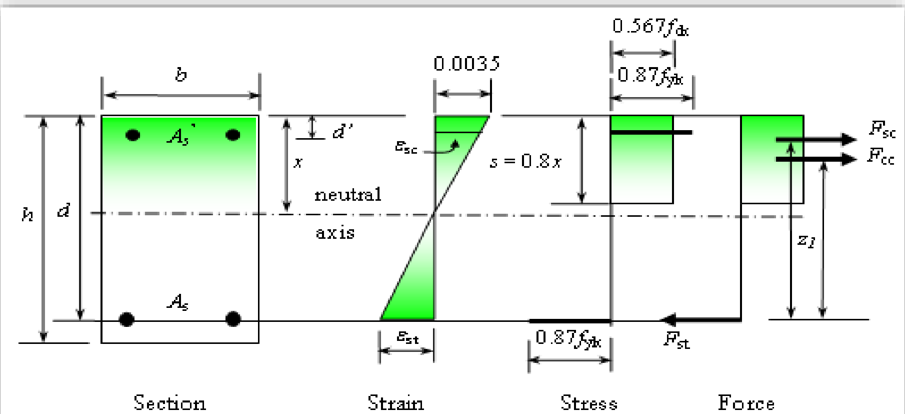


Figure 2.2 Doubly reinforced section with rectangular stress block

2.2.6 Design Procedure for Rectangular Section

Supposed the design bending moment is M, beam section is b x d, concrete strength is f_{ck} and steel strength is f_{yk}, to determine the area of reinforcement, proceed as follows.

1. Calculate $K = \dfrac{M}{bd^2 f_{ck}}$
2. Calculate $K_{bal} = 0.363(\delta - 0.44) - 0.116(\delta - 0.44)^2$

where $\delta = \dfrac{\text{momen at section after redistribution}}{\text{momen at section before redistribution}} \leq 1.0$

1. If $K \leq K_{bal}$, compression reinforcement is not required, and
   i. $z = d\left[0.5 + \sqrt{(0.25 - K/1.134)}\right]$
   ii. $A_s = \dfrac{M}{0.87 f_{yk} z}$
2. If $K > K_{bal}$, compression reinforcement is required, and
   i. $z = d\left[0.5 + \sqrt{(0.25 - K_{bal}/1.134)}\right]$
   ii. $x = (d - z)/0.4$
   iii. Check d'/x

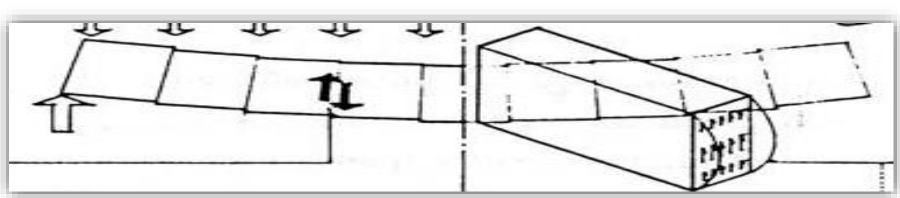
i. $A_s' = \dfrac{(K - K_{bal}) f_{ck} b d^2}{0.87 f_{yk}(d - d')}$ if $d'/x \leq 0.38$ or

$A_s' = \dfrac{(K - K_{bal}) f_{ck} b d^2}{f_{sc}(d - d')}$ if $d'/x > 0.38$

where $f_{sc} = 700(1 - d'/x)$

ii. $A_s = \dfrac{K_{bal} f_{ck} b d^2}{0.87 f_{yk} z} + A_s' \left(\dfrac{f_{sc}}{0.87 f_{yk}} \right)$

DESIGN FOR SHEAR



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Introduction

EC 2 : Section 6.2.3 introduces the strut inclination method for shear capacity checks. In this method the shear is resisted by concrete struts acting in compression and shear reinforcement acting in tension.

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6.2.3 Members requiring design shear reinforcement

(1) The design of members with shear reinforcement is based on a truss model (Figure 6.5). Limiting values for the angle θ of the inclined struts in the web are given in 6.2.3 (2).

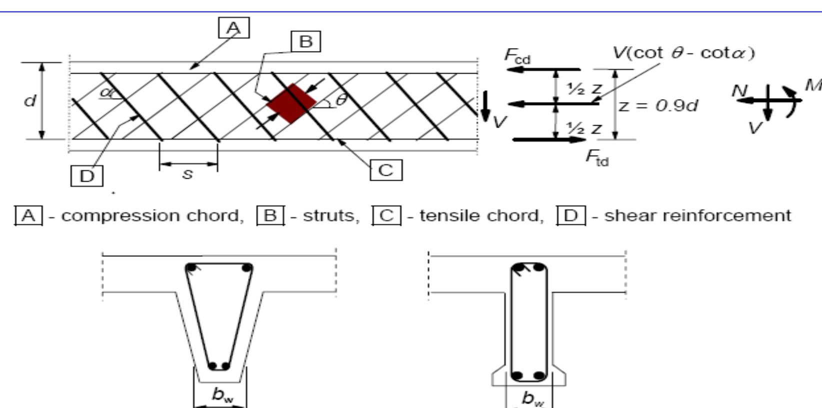
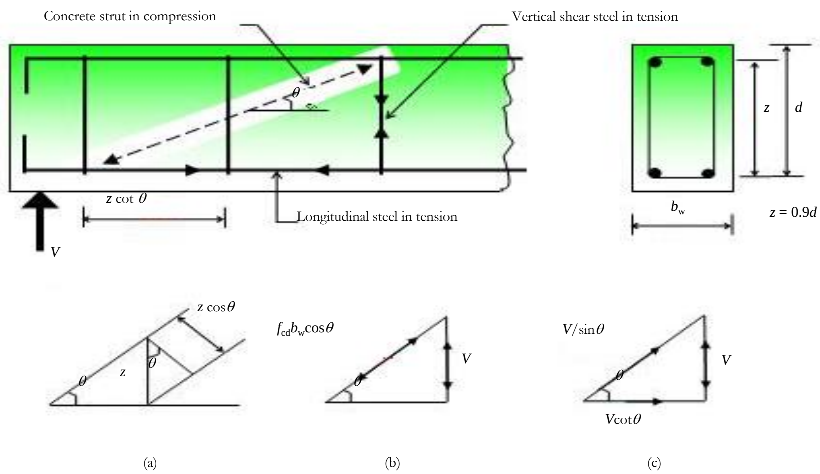


Figure 6.5: Truss model and notation for shear reinforced members

Assumed truss model for the strut inclination method



The diagonal compressive strut

$$V_{\mathrm{Rd,max}} = \frac{0.36 f_{\mathrm{ck}} b_{\mathrm{w}} d (1 - f_{\mathrm{ck}} / 250)}{(\cot\theta + \tan\theta)}$$

The shear resistance of the link is given by

$$\mathbf{V_{Rd,s} = 0.78 f_{yk} A_{sw} d (\cot\theta / s)}$$

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All shear force will be resisted by the provision of links with no direct contribution from shear capacity of concrete itself.

$$V_{Ed} = V_{Rd,s}$$
$$= 0.78 f_{yk} A_{sw} d \, (\cot\theta / s)$$

Thus rearranging

$$\frac{A_{sw}}{s} = \frac{V_{Ed}}{0.78 f_{yk} d \cot\theta}$$

EC2 (Cl. 9.2.2) specifies a minimum value for A_{sw}/s such that,

$$\frac{A_{sw}}{s} = \frac{0.08 b_w \sqrt{f_{ck}}}{f_{yk}}$$

EC2 (Cl. 9.2.2) also specifies that the maximum spacing of vertical link should not exceed 0.75*d*.

Design procedure

1. Determine design shear force V_{Ed}

2. Determine the concrete strut capacity for
   cot θ =1.0 and cot θ =2.5
   (θ = 22° and θ = 45° respectively)

$$V_{Rdmax} = \frac{0.36 b_w d f_{ck} (1 - f_{ck} / 250)}{(\cot\theta + \tan\theta)}$$

3. If $V_{Ed} > V_{Rd,max}$ cot θ = 1.0 Redesign section

4. If $V_{Ed} < V_{Rd,max}$ cot θ = 2.5, use cot θ = 2.5, and calculate the shear reinforcement as follows

$$\frac{A_{sw}}{s} = \frac{V_{Ed}}{0.78 f_{yk} d \cot\theta} \qquad (\cot\theta = 2.5)$$

$$= \frac{0.513 V_{Ed}}{f_{yk} d}$$

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5. If $V_{\mathrm{Rd,max}} \cot \theta = 2.5 < V_{\mathrm{Ed}} < V_{\mathrm{Rd,max}} \cot \theta = 1.0$

(i). Calculate $\theta = 0.5 \sin^{-1}\left[\dfrac{V_{\mathrm{Ed}}}{0.18 b_w d f_{\mathrm{ck}} (1 - f_{\mathrm{ck}} / 250)}\right]$

(ii). Calculate shear link as

$$\frac{A_{\mathrm{sw}}}{s} = \frac{V_{\mathrm{Ed}}}{0.78 f_{\mathrm{yk}} d \cot \theta}$$

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Calculate the minimum links required by EC2: Cl 9.2.2(5),

$$\frac{A_{\mathrm{sw}}}{s} = \frac{0.08 b_{\mathrm{w}} \sqrt{f_{\mathrm{ck}}}}{f_{\mathrm{yk}}}$$

Calculate the additional longitudinal tensile force caused by the shear

$$\Delta F_{\mathrm{td}} = 0.5 V_{\mathrm{Ed}} \cot \theta$$

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6.2.2 Members not requiring design shear reinforcement

(1) The design value for the shear resistance $V_{Rd,c}$ is given by:

$$V_{Rd,c} = [C_{Rd,c}k(100\,\rho_l f_{ck})^{1/3} + k_1\,\sigma_{cp}]\,b_w d \qquad (6.2.a)$$

with a minimum of

$$V_{Rd,c} = (v_{min} + k_1\sigma_{cp})\,b_w d \qquad (6.2.b)$$

$$V_{Ed} \leq V_{Rd,c}$$

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SERVICEABILITY

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INTRODUCTION

- Serviceability, implies that deformation of structures such as deflections, cracking and other distortions under load shall not be excessive

SECTION 7 SERVICEABILITY LIMIT STATES (SLS)

7.1 General

(1)P This section covers the common serviceability limit states. These are:

- stress limitation (see 7.2)
- crack control (see 7.3)
- deflection control (see 7.4)

Deflection

- Excessive deflection lead to sagging of floor, crushing of partitions, buckling of glass enclosures, ill lifting doors and windows, poor drainage, misalignment of machinery and excessive vibration.

❑ Deflection limit:

1. final deflection of a beam, slab or cantilever subjected to quasi-permanent loads should not exceed span/250

2. for the deflection which takes place after the application of finishes or fixing of partition should not exceed span/500 to avoid damage to fixtures and fittings.

❑ For control of deflection, two alternative methods are described in EC2 clause 7.4:

1. Limiting span to depth ratios (Clause 7.4.2)

2. Calculation of actual deflection (Clause 7.4.3)

Limiting span / depth ratio

The basic span-effective depth ratios, to control deflection to a maximum of span/250 are given in EC2 as;

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\,\frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_o}{\rho} - 1\right)^{3/2}\right] \qquad \text{if } \rho \le \rho_o$$

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\,\frac{\rho_o}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho}}\right] \qquad \text{if } \rho > \rho_o .$$

Table 4.1: Basic span/effective depth ratio (f_{yk} = 500 N/mm^2, C30/35 Concrete)

Structural System	K	Basic span-effective depth ratio	
		Concrete highly stressed, ρ = 1.5%	Concrete lightly stressed, ρ = 0.5%
1. Simply supported beam, one/two way simply supported slab	1.0	14	20
2. End span of continuous beam or one-way continuous slab or two way spanning slab continuous over one long side	1.3	18	26
3. Interior span of beam or one way or two way spanning slab	1.5	20	30
4. Slab supported on columns without beam (flat slab) based on longer span	1.2	17	24
5. Cantilever	0.4	6	8

The basic ratios are modified in particular cases as follows:

- For flange section where the ratio of the flange width to the web width exceeds 3, the values should be multiplied by 0.8.

- For beam and slabs, other than flat slab, with spans exceeding 7 m, which support partitions liable to be damaged by excessive deflection, the values should be multiplied by 7/span.

- Where more tension reinforcement is provided ($A_{s,prov}$) than that calculated ($A_{s,req}$), multiply the values by $A_{s,prov}/A_{s,req}$. (upper limit = 1.5)

Cracking

Cracks are induced in reinforced concrete elements as a result of:

- flexural tensile stress due to bending under applied loads;
- diagonal tension stress due to shear under applied load;
- volume changes due to shrinkage, creep, thermal and chemical effects; and
- splitting along reinforcement due to bond and anchorage failure.

Tension Crack

Flexural Cracking

Thermal Cracking

- **The primary objective of crack control is to limit the width of individual cracks.**
- **This is required not only for aesthetic reasons, more importantly, for durability and particularly for corrosion protection of reinforcement.**

Limiting crack width :

- In the absence of specific requirements (e.g. water tightness) the crack width may be limited to 0.3 mm in all exposure classes under quasi-permanent combination of loads.

- In the absence of requirements for appearance, this limit may be relaxed to 0.4 mm for exposure classes X0 and XC1.

For control of crack, two alternative methods are described in EC2 clause 7.3.

1. Control of cracking without direct calculation, (Clause 7.3.3)

2. Calculation of crack widths (Clause 7.3.4)

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Control of cracking without direction calculation.

- Minimum reinforcement area

$$A_{s,\,min} = k_c\, k\, f_{ct,\,eff}\, A_{ct} \,/\, f_{yk}$$

- Maximum spacing of reinforcement
- Maximum bar size

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Table 4.2: Maximum bar spacing for crack control

Steel stress (N/mm^2)	Maximum bar spacing (mm)	
	w_k = 0.4 mm	w_k = 0.3 mm
160	300	300
200	300	250
240	250	200
280	200	150
320	150	100
360	100	50

$$f_s = \frac{f_{yk}}{1.15} \times \frac{G_k + 0.3Q_k}{(1.35G_k + 1.5Q_k)} \frac{1}{\delta}$$

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Table 4.3: Maximum bar diameters for crack control

Steel stress (N/mm^2)	Maximum bar size (mm)	
	w_k = 0.4 mm	w_k = 0.3 mm
160	40	32
200	32	25
240	20	16
280	16	12
320	12	10
360	10	8
400	8	6
450	6	5

$$f_s = \frac{f_{yk}}{1.15} \text{x} \frac{G_k + 0.3Q_k}{(1.35G_k + 1.5Q_k)} \frac{1}{\delta}$$

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DETAILING & DURABILITY REQUIREMENTS

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- **Detailing and durability requirements are to ensure that a structure has satisfactory durability and serviceability performance under normal circumstances throughout its lifetime.**

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- **EC2 recommends simple rules concerning the concrete mix and cover to reinforcement, minimum member dimension, and limits to reinforcement quantities and spacing which must be taken into account at the member sizing and reinforcement detailing stage.**

SECTION 8 DETAILING OF REINFORCEMENT AND PRESTRESSING TENDONS - GENERAL

8.1 General

(1)P The rules given in this Section apply to ribbed reinforcement, mesh and prestressing tendons subjected predominantly to static loading. They are applicable for normal buildings and bridges. They may not be sufficient for:

- elements subjected to dynamic loading caused by seismic effects or machine vibration, impact loading and
- to elements incorporating specially painted, epoxy or zinc coated bars.

SECTION 9 DETAILING OF MEMBERS AND PARTICULAR RULES

9.1 General

(1)P The requirements for safety, serviceability and durability are satisfied by following the rules given in this section in addition to the general rules given elsewhere.

(2) The detailing of members should be consistent with the design models adopted.

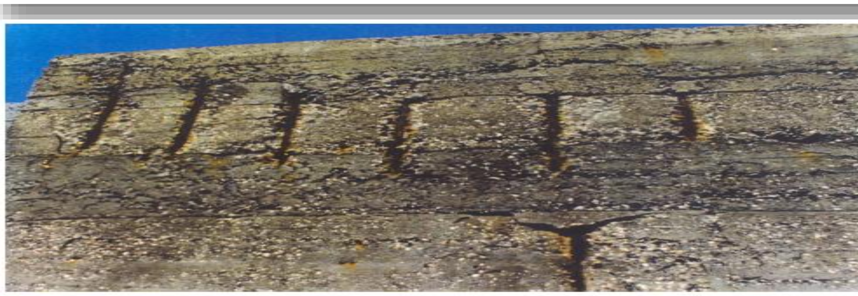
(3) Minimum areas of reinforcement are given in order to prevent a brittle failure, wide cracks and also to resist forces arising from restrained actions.

SECTION 4 DURABILITY AND COVER TO REINFORCEMENT

4.1 General

(1)P A durable structure shall meet the requirements of serviceability, strength and stability throughout its design working life, without significant loss of utility or excessive unforeseen maintenance (for general requirements see also EN 1990).

(2)P The required protection of the structure shall be established by considering its intended use, design working life (see EN 1990), maintenance programme and actions.



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Deterioration of concrete structures

Corrosion of reinforcement by chloride attack in a marine environment

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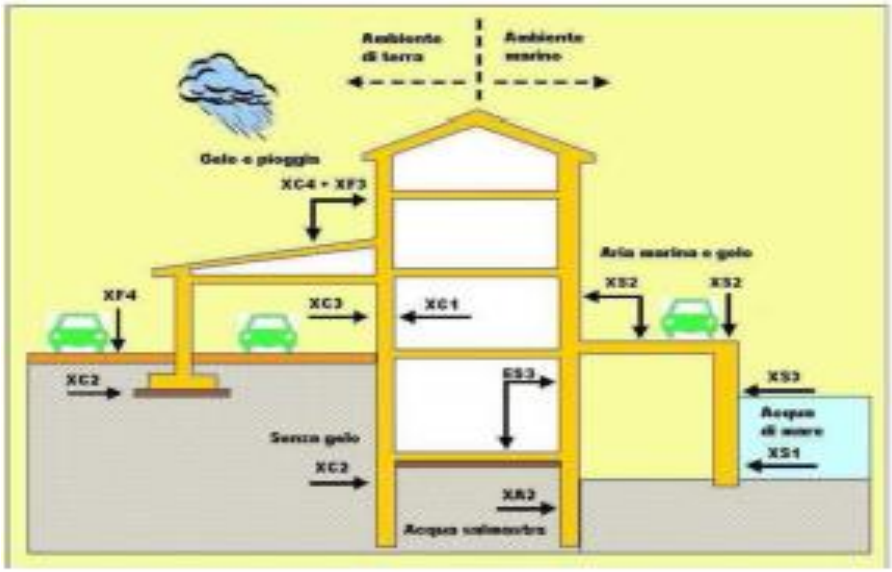
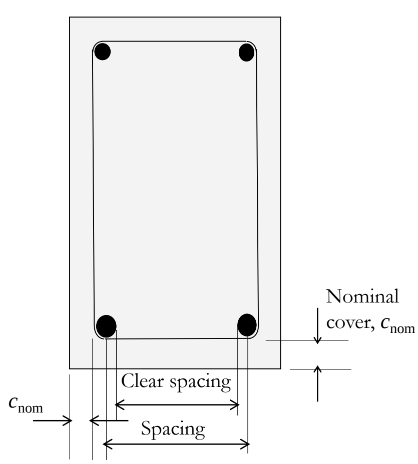
carbonation

chlorides



Concrete Cover



All reinforcement must have sufficient cover to prevent corrosion and to protect the reinforcement in the event of fire

The nominal cover can be assessed as follows:

$$C_{nom} = C_{min} + \Delta C_{dev}$$

Where C_{min} shall be provided in order to ensure:

- The safe transmission of bond forces
- The protection of steel against corrosion (Durability)
- An adequate fire resistance

And ΔC_{dev} is and allowance which should be made in the design for deviation from the minimum cover. It should be taken as 10 mm. It is permitted to reduce to 5 mm if the fabrication subjected to a quality assurance system.

Minimum cover for bond

EN 1992-1-1

Arrangement of bars	Minimum cover $c_{min,b}$*
Separated	Diameter of bar
Bundle	Equivalent diameter $\phi_n = \phi\sqrt{n_b} \leq 55$ mm Where n_b is the number of bars in the bundle, which is limited to $n_b \leq 4$ for vertical bars in compression $n_b \leq 3$ for all other cases
* If the nominal maximum aggregate size is > 32 mm, $c_{min,b}$ should be increased by 5 mm	

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Minimum cover for durability

EN 1992-1-1

Structural Class	Exposure Class according to Table 4.1 EC 2						
	X0	XC1	XC2/XC3	XC4	XD1/XS1	XD2/XS2	XD3/XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

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(5) The minimum cover values for reinforcement and prestressing tendons in normal weight concrete taking account of the exposure classes and the structural classes is given by $c_{min,dur}$.

Note: Structural classification and values of $c_{min,dur}$ for use in a Country may be found in its National Annex. The recommended Structural Class (design working life of 50 years) is S4 for the indicative concrete strengths given in Annex E and the recommended modifications to the structural class is given in Table 4.3N. The recommended minimum Structural Class is S1.

Table 4.3N: Recommended structural classification

Structural Class							
Criterion	Exposure Class according to Table 4.1						
	X0	XC1	XC2 / XC3	XC4	XD1	XD2 / XS1	XD3 / XS2 / XS3
Design Working Life of 100 years	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2
Strength Class [1) 2)]	≥ C30/37 reduce class by 1	≥ C30/37 reduce class by 1	≥ C35/45 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C45/55 reduce class by 1
Member with slab geometry (position of reinforcement not affected by construction process)	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1
Special Quality Control of the concrete production ensured	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1

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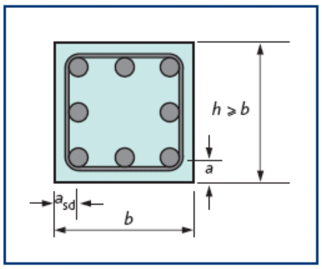
Minimum cover for fire resistance

Rather than giving a minimum cover, the tubular method based on nominal axis distance is used. This is the distance from the centre of the main reinforcement bar to the top or bottom surface of the member.

Section through structural member, showing nominal axis distances a and a_{sd}



$$a > C_{nom} + \phi_{\lambda ink} + \phi_{bar}/2$$

$$a_{sd} = a + 10 \text{ mm}$$

Minimum & Maximum Area of Reinforcement

- The minimum area of reinforcement that must be provided within tensile zone is

$$A_{s,min} = k_c \, k \, f_{ct,eff} \, A_{ct} / f_{yk}$$

- The minimum area of reinforcement for beam also specified in Section 9.2.1 as follows:

$$A_{s,min} = 0.26(f_{ctm}/f_{yk})b_t d \quad \text{but not less than } 0.0013 b_t d$$

- The limits $A_{s,max}$ specified by EC2 in Section 9.2.1 is $0.04A_c$ for tension or compression reinforcement.

Spacing of reinforcements

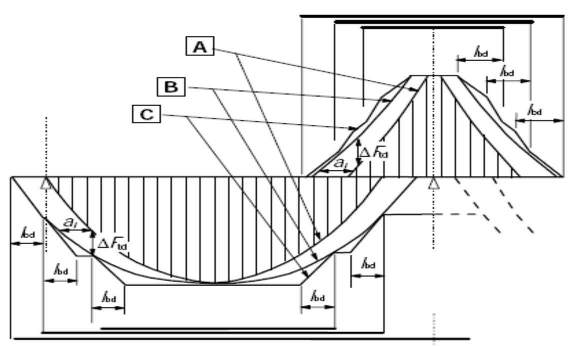
- The minimum distance between bars is to permit concrete flows around reinforcement during construction and to ensure that concrete can be compacted satisfactorily for the development of adequate bond.
- The clear distance between bars should not be less than the maximum of (i) the maximum bar size, (ii) the maximum aggregate size + 5 mm, or (iii) 20 mm. (Specified in section 8.2 EC2).

Curtailment and anchorage of reinforcements

- It is a common practice to cut off bars where they are no longer required to resist moment.
- Each curtailed bar should extend a full anchorage length beyond the point at which it is no longer needed.
- The basic required anchorage length given in section 8.4.3 EC2 is as follows;

$$l_{b,rqd} = (\phi / 4)(\sigma_{sd} / f_{bd}) = (\phi / 4)(f_{yk} / 1.15) / f_{bd}$$
$$= (f_{yk} / 4.6 f_{bd})\, \phi$$

The curtailment of the tension reinforcement is based upon the enveloped of tensile forces, F_s derive from the bending moment envelop as shown in Figure 5.2 such that at any location along the span, $F_s = M_{Ed}/z + \Delta_{Ftd}$



A - Envelope of $M_{Ed}/z + N_{Ed}$ B - acting tensile force F_s C - resisting tensile force F_{Rs}

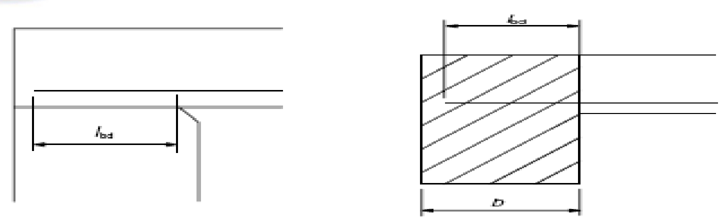


Figure 5.3: Anchorage of bottom reinforcement at end supports

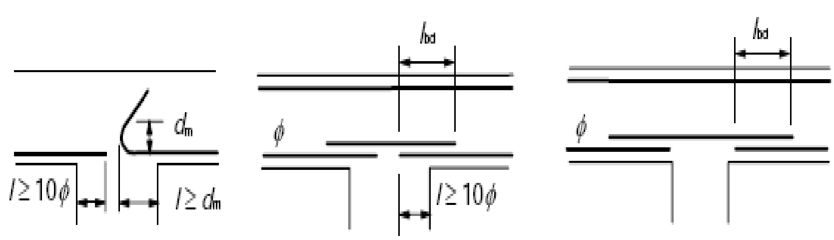
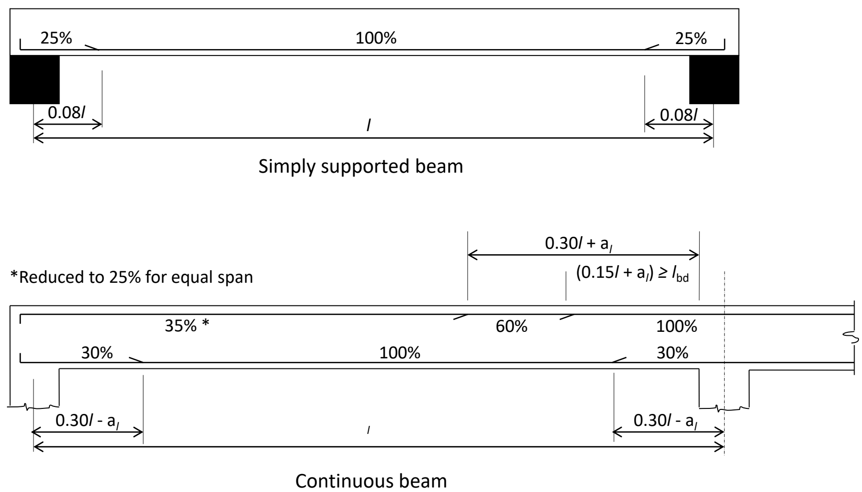


Figure 5.4: Anchorage at intermediate supports

Simplified detailing rules for beams



The design anchorage length l_{bd} mentioned above is given by,

$$l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 \, l_{bd,rqd} \geq l_{b,min}$$

where

$\alpha_1, \alpha_2, \alpha_3, \alpha_4$ and α_5 = *coefficient given in Table 5.5*

$l_{bd,rqd}$ = *equation (5.4)*

$l_{b,min}$ = *the minimum anchorage length*

for tension bars : max {$0.3\, l_{bd,rqd}$; 10ϕ ; 100 mm}

for compression bars : max {$0.6\, l_{bd,rqd}$; 10ϕ ; 100 mm}

Value of α	α allows for the effect of	Type of anchorage	Reinforcement in	
			Tension	Compression
α_1	The shape of the bars	Straight	1.0	1.0
		Other than straight	0.7 if $c_d > 3.0\phi$ or 1.0 if not	1.0
α_2	Concrete cover to reinforcement	Straight	$1.0 - 0.15(c_d - \phi)/\phi$ but ≥ 0.7 and ≤ 1.0	1.0
		Other than straight	$1.0 - 0.15(c_d - 3\phi)/\phi$ but ≥ 0.7 and ≤ 1.0	1.0
α_3	Confinement of transverse reinforcement not welded to the main reinforcement	All types of reinforcement	$1 - K\lambda$ but ≥ 0.7 and ≤ 1.0	1.0
α_4	Confinement of transverse reinforcement welded to the main reinforcement	All types, position and sizes of reinforcement	0.7	0.7
α_5	Confinement by transverse pressure	All types of reinforcement	$1 - 0.04\rho$ but ≥ 0.7 and ≤ 1.0	-

Laps in reinforcements

Laps are required when bars placed short of their required length need to be extended. Laps are also required when the bar diameter has to be changed along the length. The purpose of lapping is to transfer effectively the axial force from the terminating bar to the connecting bar with the same line of action at the junction.

Requirements for laps are discussed in Section 8.7 EC2. The code recommends that;

- Laps between bars should be staggered and should not occur in regions of high stress.
- The arrangement of lapped bars should comply with Figure 5.5 below.

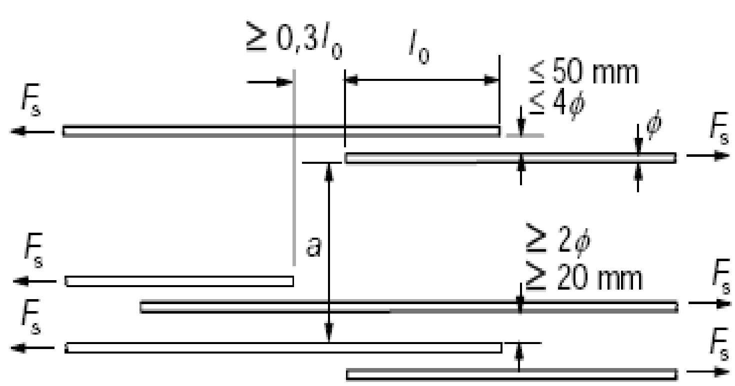


Figure 5.5: Adjacent laps

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•Transverse reinforcement must be provided around laps unless lapped bars are less than 20 mm diameter or there is less than 25 % lapped bars. In these cases minimum transverse reinforcement provided for other purposes such as shear links will be adequate. Otherwise transverse reinforcement must be provided, as shown in Figure 5.6, having a total area of not less than the area of one lapped bar.

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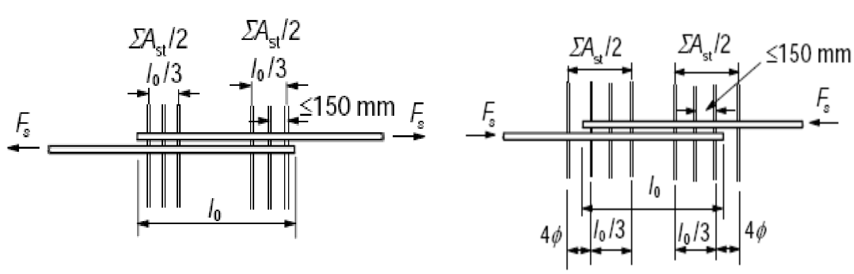

Tension lap

Compression lap

Figure 5.6: Transverse reinforcement for lapped bars

The length of laps should be based on the minimum anchorage length modified to take into account factors such as cover, etc. The design lap length required is given by,

$$l_o = \alpha_1 \alpha_2 \alpha_3 \alpha_5 \alpha_6 l_{bd,rqd} \geq l_{o,min}$$

where

α_1, α_2, α_3, α_4 *and* α_5 = *coefficient given in Table 5.5*

For the calculation of α_3, $\Sigma A_{st,min}$ *should be taken as* $1.0A_s(\sigma_{sd}/f_{yd})$, *with* A_s = *area of one lapped bar*

$\alpha_6 = (\rho_1/25)^{0.5}$ *but not exceeding 1.5 nor less than 1.0 and* ρ_1 *is the percentage of reinforcement lapped within* $0.65l_o$ *from the centre of the lap length being considered.*

$l_{bd,rqd}$ = *equation (5.3)*

$l_{o,min}$ = *the absolute minimum lap length*

= $max\{0.3\alpha_6 l_{bd,rqd}; 15\phi; 200 \text{ mm}\}$

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Table 5.8 Typical values of anchorage and lap lengths as a multiplied by bar size

Condition /Situation	Bond conditions	Concrete Strength f_{ck}/f_{cu}(N/mm^2)				
		20/25	25/30	28/35	30/37	32/40
Anchorage length (Tension or compression)	Good	47	40	37	36	34
	Poor	67	58	53	51	49
Lap length (Tension or compression)	Good	54	46	43	42	39
	Poor	77	66	61	59	56

Notes:
1. It is assumed that the bar size is not greater than 32 mm and α_1, α_2,α_3,α_4 and α_5 all equal 1.
2. It is assumed that not more than 33% of the bars are lapped at one place, α_6 = 1.15
For other situations refer to MS EN 1992-1-1 , Clause 8.4.4

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