GENERAL RULES FOR DESIGN OF REINFORCED CONCRETE STRUCTURES ACCORDING TO EUROCODE NO.2

"COMPARATIVE WITH THE ACI CODE"

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SUMMARY

Eurocode No.2 (EC2) part 1 [1] gives general rules for design and analysis of concrete structures. In this paper the main general rules for the design of reinforced concrete made of normal weight concrete according to EC2 are discussed. Material properties, design for flexure, shear, deformations and cracking control using the ultimate limit states design and the serviceability limit states are explained.

For the purpose of comparing the EC2 and the building code requirements for reinforced concrete "American Concrete Institute" (ACI) [2], a reinforced concrete rectangular cross-section with tension reinforcement is analyzed according to EC2, the results are compared with the same calculated using the ACI Code. Linear elastic analysis with limited distribution was used to determine the internal forces of the section. Design tables from [3] and [4] for rectangular cross section with tension reinforcement subject to bending with axial force using different concrete models were used to determine the area of reinforcement.

Discussion of the results and conclusion of comparing the two codes were drawn.

1. INTRODUCTION

Codes of building are made based on experimental work, experience of engineers and certain conditions and behaviour. The main purpose is to simplify the engineering work, and to establish a common unified rules for analysis and design of engineering problems.

EUROCODE No.2 (EC2) applied to the design of building and civil engineering work in plain, reinforced concrete and prestressed concrete, this concern the requirements for resistance, serviceability and durability.

Part 1 of EC2 gives general rules which are mainly applicable for the design of reinforced and prestressed concrete ordinary structures made of normal weight aggregates. The limit state method used as basis for the design to meet two requirements:

1. Ultimate limit state.
2. Serviceability limit state.

General serviceability requirements of the code, such as requirement for deflection and crack width control must be met.

2. DESIGN CONCEPT

The design of EC2 considering the Ultimate Limit States as basic requirement for design, so that the design value of an internal force or moment (S_d) smaller or equal to the corresponding design resistance with respect to the design value of material (R_d) (nominal strength). This can be expressed as follows:

$$S_d < R_d \tag{2.1}$$

To determine the design load, the service load must be multiply by a partial safety factors (γ) (see eq. 2.2). Different partial safety factors for quasi-permanent actions and for variable actions are considered. Also in the ACI Code, the different between load actions (dead, live, wind and earthquake load) are considered, an additional partial safety factor for accidental design situations was taken into consideration in the EC2. EC2 gives two different values for actions, one is a characteristical value indicated by S_k and a design action indicated as S_d, where:

$$S_d = \gamma_f \; S_k \tag{2.2}$$

For only one variable action the safety factor γ_Q is taken = 1.5. For design situation with two or more variable actions the value should be reduced to 1.35. The value of γ_G for unfavorable effect = 1.35. The value of γ_f is applicable to all accidental situations and is taken = 1.0. The expression may be written as:

$$Sd = \sum \gamma_G \cdot G + \gamma_Q \cdot Q_1 * \sum_{i>1} \gamma_Q \cdot \psi_i \cdot Q_i \tag{2.3}$$

ψ_i is another factor in EC2 taken into consideration the probability of application of one or more combination of variable action values such as live load, wind load, snow load. The factor ψ_i considers the basic combination value ψ_0 and frequent value $\psi 1$ and quasi-permanent value ψ_2. The values of ψ_i can be found in [4]. The design values d, and the characteristic values k of the variable actions have the following relation:

$$Q_d = \gamma_Q \; Q_k \quad \text{or} \quad \gamma_Q \; \psi_i \; Q_k \tag{2.4}$$

In general form this relation can be written as follows:

$$F_d = \gamma_f \; F_k \tag{2.5}$$

Also partial safety factors for material properties are given, these values assumed to take consideration of the differences between the strength of test specimen of structural material and their strength in site. Two different values for concrete and steel reinforcement or prestressing tendon for accidental and fundamental load combinations are considered. the partial safety factors for material properties are given in table (1)

Table 1: Partial safety factors for materials properties according to EC2.

Combination	Concrete γ_c	Steel reinf. or tendons γ_s
Fundamental	1.5	1.15
Accidental	1.3	1.0

3. ANALYSIS

Using the load factors to establish the distribution of internal forces and moment using linear elastic analysis. Special methods are required to use these results to obtain appropriate reinforcement areas. The common behaviour idealization used for analysis are:

-The linear elastic behaviour using the ultimate limit states
-The behaviour with limited or without redistribution
-Plastic behaviour for deep beam using the method of strut and ties modules.

Load combinations are used for analysis of continuous beams to obtain the maximum action with different variable loads locations. The load combination in both codes are different in the numerical values of the partial safety coefficients and the formulation of the equations.

4. DESIGN REQUIREMENTS

4.1 Design Requirement for Flexure

4.1.1 General

Moments calculated using linear elastic analysis is to be redistributed providing that the resulting distributed moments remain under equilibrium with the applied load. The moment redistribution is considered using the factor δ, where:

δ is the ratio between the redistributed moment $M_{red.}$ and the moment calculated using linear elastic analysis M

$$\delta = M_{red}/M \qquad (4.1)$$

The rotational capacity of critical section can be omitted if:

$\delta > 0.44 + 1.25\ x/d$ for concrete strength < C35/45

Stress strain diagram for concrete subject to uniaxial compression obtained by experiment is shown in figure 1. For calculation purpose idealized stress strain diagrams for steel and concrete are used, a parabola-rectangle idealized stress strain diagram is preferred for cross section design. in both EC2 and ACI codes any other idealized stress strain diagram can be used in the design providing effectively equivalent diagram to parabola-rectangle diagram. The design diagram is then derived from the chosen idealized diagram by means of a reduction of the stress ordinate of idealized diagram by a factor α/γ_c in which α is a coefficient taken account for sustained compression and may be assumed to be 0.85 (see Fig. 2 and 3).

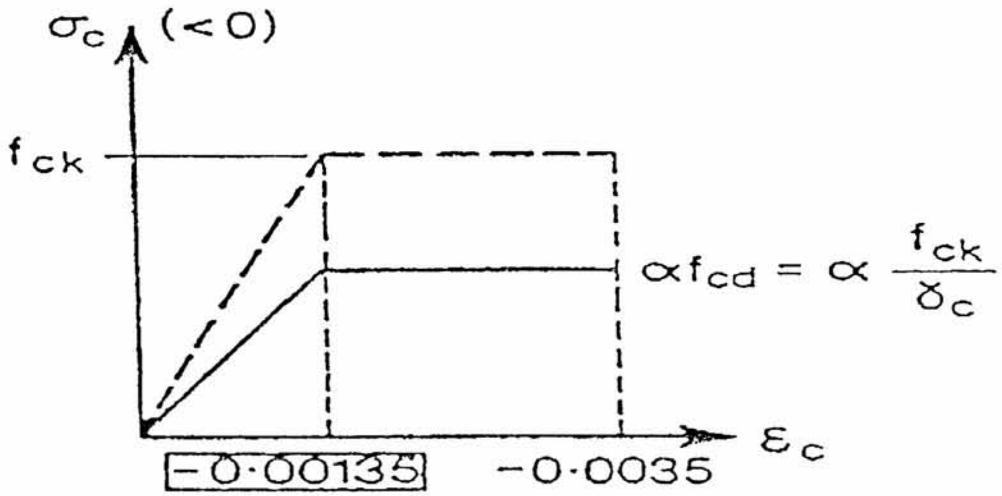


Fig. 2: Bi-linear stress-strain diagram for concrete

To simplify the determination of steel area in a cross section, tables are formed by (ref. [6]) taking into account the max. strain in concrete is equal to 0.0035 and a maximum steel strain of 0.010. Also tables were made for different concrete classes and for cross sections with or without compression steel (see also [3,5]). The parabolic-rectangular stress-strain diagrams shown in figures 2 and 3 were taken as basic for calculations.

4.2 Design Requirement for Shear

Shear design of EC2 4.3.2.1 is build in a similar way as in the ACI. A minimum amount of shear reinforcement shall be provided even when calculation shows that shear reinforcement is not necessary, this may be omitted in members as solid and ribbed slabs. The requirement for shear design of a section is that the shear design force V_{sd} should not exceed the maximum design shear resistance of the section V_{Rd}

$$V_{sd} < V_{Rd} \qquad (4.4)$$

$\delta > 0.56 + 1.25\ x/d$ for concrete strength > C35/45

$\delta > 0.7$ for high ductility steel

$\delta > 0.85$ for normal ductility steel

4.1.2 Material Classification

Strength of material in EC2 based on characteristic values, compressive cylinder strength f_{ck} for concrete and character yield strength f_{yk} for steel. Concrete classified into two values, for example: For concrete C20/25 indicates the first value the characteristic 28 day compressive strength of cylinder with 150 mm diameter and 300 mm height, the second value indicates the characteristic 150/150/150 mm cube strength. The design rules based on the cylindrical strength, the second value is mentioned only as alternative value.
The design strength values are based on strength class of material using a reduction factor γ_c for concrete and γ_s for steel. The values of the reduction values are shown above in table 1. see also fig.1. This can be written as:

$$f_{cd} = f_{ck}/\gamma_c \qquad (4.2)$$

and

$$f_{yd} = f_{yk}/\gamma_s \qquad (4.3)$$

For example: Standard steel S400 with f_{yk} = 400 MPa have a design value of f_{yd} = 400/1.15 = 348 MPa.

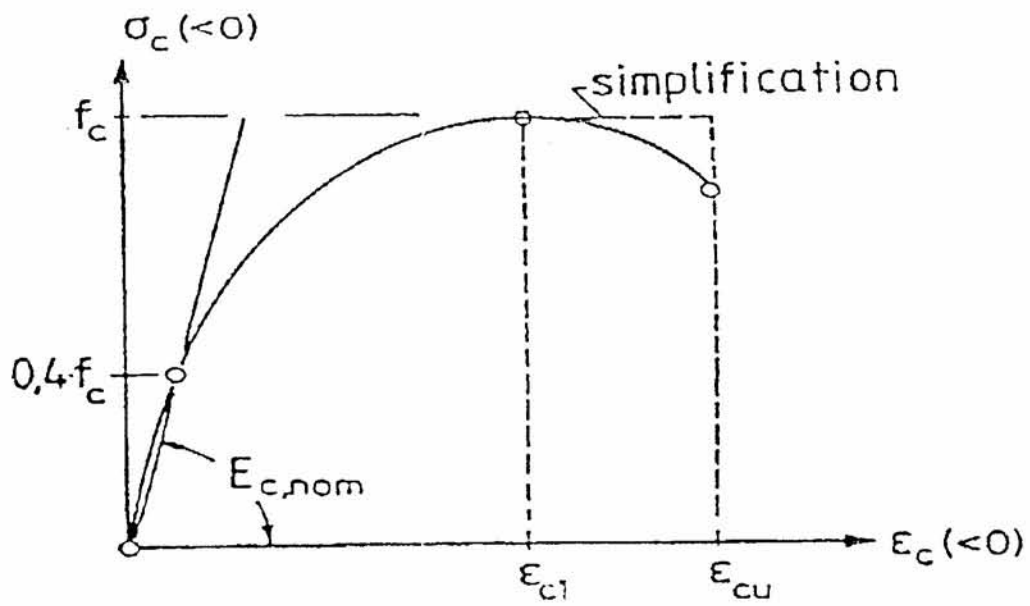


Fig.1: Schematic stress-strain diagram for structural analysis.

The design for shear uses two methods, the standard method which depends on the design value of V_{Rd}, and variable truss method in which recommended to be used for deep beams and for members subject to combined shear and torsion forces.

Two different shear values of V_{Rd} is to be checked. The design value of shear resistance force without shear reinforcement V_{Rd1}, and the maximum design force that can be carried without crushing the compressive struts of concrete V_{Rd2}.

The design shear resistance of element with shear reinforcement V_{Rd3} is given as:

$$V_{Rd3} = V_{Rd1} + V_{wd}$$

Where:
V_{wd} = The contribution of shear reinforcement.

Minimum shear reinforcement ratio are tabulated in EC2 depends on concrete class and steel, also maximum longitudinal spacing between stirrups depends on shear design values are formulated.

In general the procedure of determining the shear area is simpler formulated in EC2 than ACI Code, however the amount of shear reinforcement calculated according to ACI Code gives smaller amount than the one calculated by the Eurocode 2.

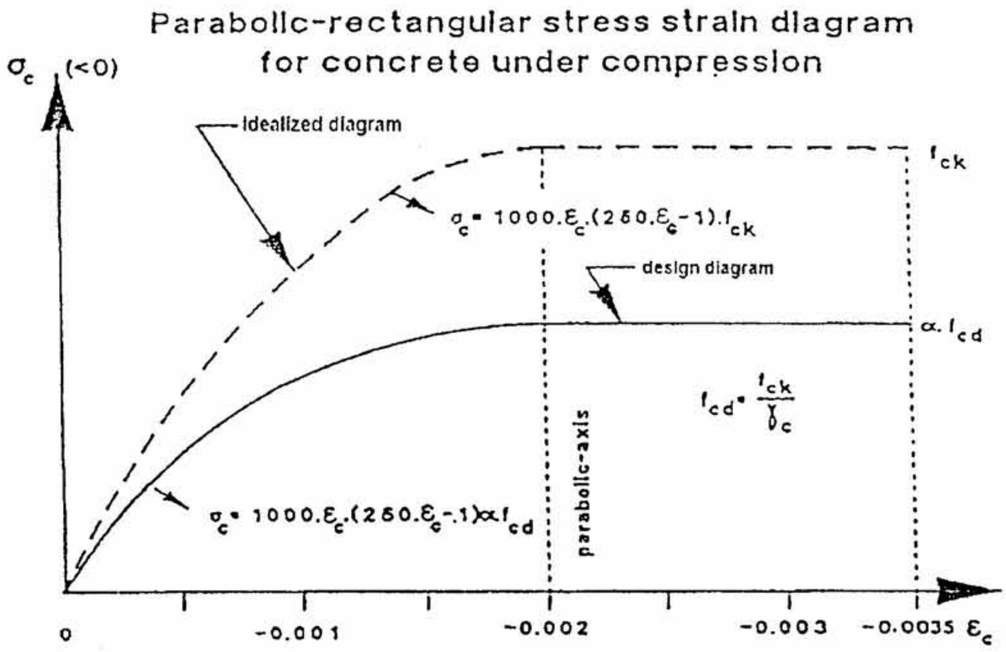


Fig. 3: Parabolic-rectangular stress-strain diagram for concrete in compression

5. LIMIT STATES OF CRACKING

The design criteria for reinforced concrete members is that the design cracked width (W_k) is smaller than the limited crack width given by the code (W_{limit}). A maximum design crack width of 0.3 mm under quasi-permanent combination of loads is satisfactory for reinforced concrete members. Calculation of the crack width can be obtain in EC2 4.4.2.4.

Control of cracking without direct calculation may be achieved by limiting bar spacings and diameters. Tables in EC2 are designed to ensure that crack widths will not generally exceed 0.3 mm for reinforced concrete and 0.2 mm for prestressed concrete.

A minimum reinforcement areas is required to ensure controlled of cracking in a member which may be subject to tensile stress due to restraint of imposed deformations. In EC2 the minimum areas of reinforcement may be calculated depends on many factors such as area of concrete within tensile zone, steel stress and coefficients considering stress distribution and size effect. In ACI the min. areas of reinforcement depends only on the steel yield stress.

6. LIMIT STATES OF DEFORMATIONS

Reinforced and prestressed concrete members subject to bending must be design to satisfy requirements for deflection. In EC2 deflection should be calculated to ensure a satisfactory behaviour in the serviceability limit states, a minimum ratio of span length to effective depth is formulated depends on structural system and concrete stress so that no calculation of deflection is necessary. In the case of computed deflection a maximum limiting deflection for long-term and for instantaneous deformations must not be exceeded.

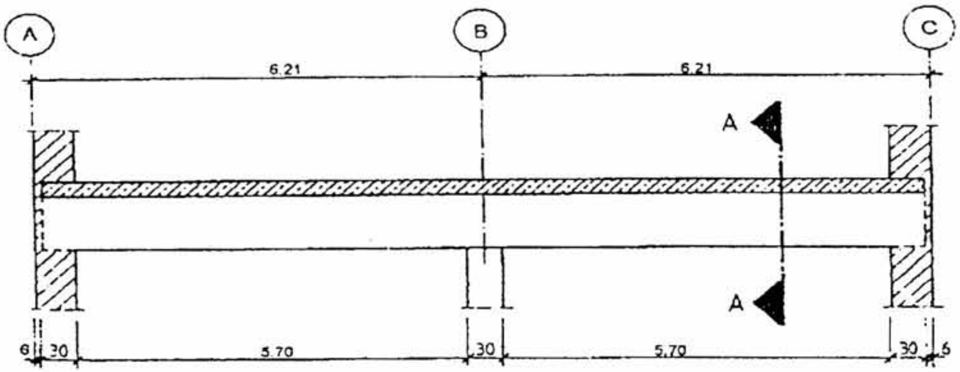
Basic principles to control deflection in EC2 and ACI are similar. The different seems obvious in the span/depth ratio and the limited calculated deflection. A higher safety factor is obviously employed in ACI Code than EC2.

7. EXAMPLE: Design of Rectangular Beam with Tension Reinforcement Only

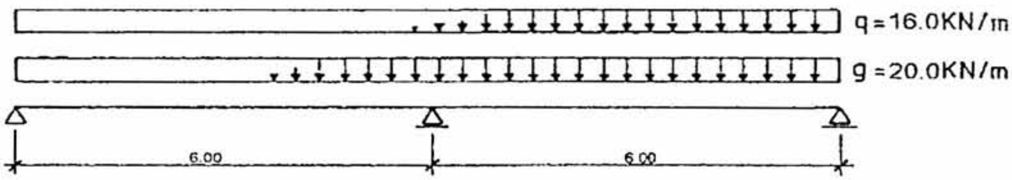
Design a reinforced concrete continuous beam over two 6.0 m equal spans according to EC2. Assume beam width is 35 cm at 4.0 m centers. The results are to be compared with the ACI Code. The structural system and cross-section are shown in the figures below.

Design criteria:
Superimposed permanent action = 1.94 KN/m
Variable action = 4 KN/m^2

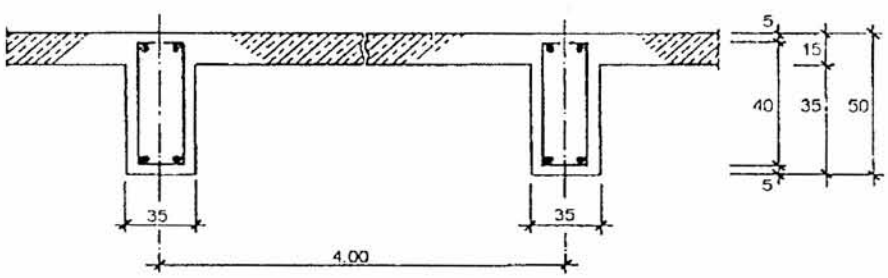
Material properties:
Concrete C 20/25
Steel S400



Statical system



SECTION A - A

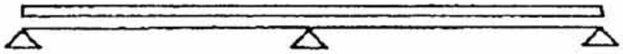


7.1 Loading:

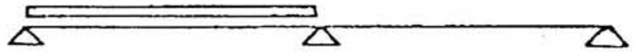
7.1.1 service load

Quasi-permanent action	$g_1 = 0.15 x 25 x 4.0$	= 15 KN/m
Web action	$g_2 = 0.35 x 0.35 x 25$	= 3 KN/m
Superimposed action		= 2 KN/m
Total permanent load		= 20 KN/m
Variable action	= 4 x 4.0	= 16 KN/m

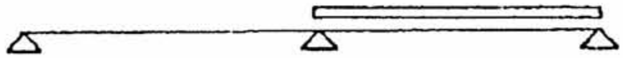
Load Pattern I



Load Pattern II



Load Pattern III



7.1.2 Ultimate limit states of loading
Determine the ultimate factored loads:

Code reference

$S_d = \gamma_g(N_g) + \gamma_Q(N_{1Q} + \psi_0 N_{2Q})$

$= 1.35 \cdot 20 + 1.5 \cdot 16 = 51$ KN/m

EC2
2.3.2.2(1)

Tab. 2.2

7.2 Load Pattern

Determine the maximum negative bending moment at support B

From load pattern I

$M_{sd,B} = -51 \cdot 6^2/8 = -230$ KN.m

Redistribution of the max. negative moment

Assume $\delta = 0.85$

EC2
2.5.3.4.2(3)

$M_{sd'},B = -0.85 \cdot 230 = -195.5$ KN.m

$U_{sd} = M_{sd}/bd^2 . f_{cd} = 195.5 \ 10^3/35 \cdot 45^2 \cdot 13.33$
$= 0.206$

$x/d = 0.423$

$\delta = 0.44 + 1.25\ x/d = 0.87$

$= 0.85$ for normal ductile steel

Tab.7.1
REF. [4]
EC2
2.5.3.4.2
Eq. 2.17

Adjusted $M_{sd'},B = 0.87 \cdot 230 = 200$ KN.m

Determine the shear force

$V_A = V_C = 51 \cdot 6/2 - 200/6 = 120$ KN

$V_{BL} = V_{BR} = 51 \cdot 6/2 + 200/6 = 186$ KN

Positive moment:
$M_{sd'},2 = 120^2/(2x51) = 141$ KN.m
Location of max. moment $X' = 120/51 = 2.35$ m

Code reference

Determine the max. positive bending moment:
Load pattern II and III

Moment at support B:

$M_{sd},B = -27x6^2/8 + 0.063x24x6^2 = -176$ KN.m

Shear forces:

$V_C = 51x6/2 - 176/6 = 124$ KN
$V_A = 27x6/2 - 176/6 = 52$ KN
$V_{BR} = 51x6/2 + 176/6 = 182$ KN
$V_{BL} = 27x6/2 + 176/6 = 110$ KN

Max. positive bending moment at span A-B pattern II and span B-C Pattern III

$M_{sd}',1,2 = 124^2/(2x51) = 150$ KN.m

Location of max. moment X = X' = 124/51 = 2.43 m

Table 2: Summary of bending moment and shear forces:

L.pat.	M_{sd},B	$M_{sd},1$	$M_{sd},2$	V_A	V_C	V_{BL}	V_{BR}
I	200	141	141	120	120	186	186
II	176	150	50	52	124	110	182
III	176	50	150	124	52	182	110

7.3 Design for Flexure: — EC2 3.1.2.4

7.3.1 Characteristic design value for concrete — Tab. 3.1
Concrete class C 20/25 — EC2 3.1.2.5.2
$f_{ck} = 20$ N/mm² — Tab. 3.2
$E_{cm} = 29$ N/mm²

$f_{cd} = f_{ck}/\gamma c \qquad \gamma_c = 1.5$ — EC2 4.2.1.3.3, Tab. 2.3

$f_{cd} = 13.33$ N/mm²

7.3.2 Characteristic design value for steel — EC2 3.2.1(2)
Steel S400
$f_{yk} = 400$ N/mm² — EC2 2.3.3.2.1

$f_{yd} = f_{yk}/\gamma s \qquad \gamma_s = 1.15$
$f_{cd} = 400/1.15 = 348$ N/mm²

7.3.3 Concrete cover

A minimum concrete cover depends on relevant — EC2 Tab. 4.1

exposure class related to environmental condition should be provided, this can be estimated as follows:

Code reference

Reinforcement diameter $\phi_1 < 25$ mm

$\min C_l = \min C_w + \phi_w$

ϕw = stirrups diameter

$\min C_w = 15$

EC2 Tab. 4.1

The allowance (Δh) = 10 mm for insitu rein forced concrete

EC2 4.1.3.3.(8)

$\min C_l = 15 + 12 = 27$ mm

$\text{nom } C_l = \min C_l + \Delta h = 37 \text{ mm} > \phi l = 25$

EC2 4.1.3.3(5)

$d_1 = \text{nom } C_l + \phi_1/2 = 37 + 25/2 = 50$ mm

Effective depth $d = h - d_l = 45$ cm

Web width $b_w = 35$ cm

7.3.4 Design for negative moment at support B:

The critical design moment at the support may be taken at the face of support

EC2 2.5.3.3(5)

$M'_{sd},B = 200 - V(0.15)$

$= 200 - 186 \cdot 0.15 = 172.1$ KN.m

check min. moment for design

$0.65 \cdot 51 \cdot 6^2/12 = 99.6 \text{ KN.m} < 172.1$ KN.m

EC2 2.5.3.4.2(7)

For design purpose the idealized stress-strain diagram of concrete with parapolic-rectangle diagram is used. A max. ultimate strain of concrete = .0035 and a max. strain of steel .010 are used.

EC2 4.2.1.3.3

$u_{ad} = M'_{sd}/bd^2.f_{cd} = 172 \times 10^{-3}/0.35(0.45)^2\ 13.33$

$= 0.182$

$x/d = 0.303$

$x = 13.6$ cm

$K_z = z/d = 0.876$

$z = 39.4$ cm

$w = 0.209$

Ref.[4] Tab.7.1

Required reinforcement

$As_{req.} = w\ b\ d\ f_{cd}/f_{yd}$

$= 0.209 \cdot 35 \cdot 45 \cdot 13.33/348 = 12.6\ cm^2$

As provided $7\phi16 = 14.1\ cm^2$

7.3.5 Design for positive moment at span 1 and 2

Max. $M_{sd,1} = M_{sd,2} = 150$ KN.m

Effective width $b_{eff.} = b_w' + L_0/5$

EC2 2.5.2.2.1

$L_0 = 0.85L$ for exterior span of continuous beam

$b_{eff} = 0.35 + 0.85 \cdot 6.0/5 = 1.37 \text{ m} < 4.0$ m

Code reference

$u_{ad} = M_{sd}/bd^2 . f_{cd}$

$= 150 . 10^{-3}/1.37 (0.45)^2 . 13.33 = 0.048$

$x/d = 0.105$

x = 0.105 . 45 = 4.72 cm neutral axis is in the flange

$w = 0.0425$ — Ref.[4] Tab.7.1

Required reinforcement

$As_{req.} = w\ b\ d\ f_{cd}/f_{yd}$

$= 0.0425 . 137 . 45 . 13.33/348 = 10\ cm^2$

As provided 6ϕ16 = 12.6 cm^2

7.4. DESIGN FOR SHEAR — EC2 4.3.2.0

7.4.1 Determine the design criteria for shear:
Determine the shear resistance of a section without shear reinforcement force V_{Rd1}

$V_{Rd1} = [\tau_{Rd} \cdot k \cdot (1.2 + 40 \cdot \rho_l) + 0.15 \cdot \sigma_{cp}] \cdot b_w \cdot d$ — EC2 4.3.2.3, EC2 Eq. 4.18

$\tau_{Rd} = 0.26$ — EC2 Tab.4.8

k = 1 assume more than 50% of the bottom reinforcement is curtailed.

$\rho_l = As_l/b_w\ d = .0038 < 0.02$

$\sigma_{cp} = N_{sd}/A_c = 0$

$V_{Rd1} = 55.4$ KN

Determine the max. design shear force V_{Rd2}

$V_{Rd2} = 1/2\ v\ f_{cd}\ b_w\ 0.9\ d$ — EC2 Eq. 4.19

$v = 0.7 - f_{ck}/200 = 0.6 > 0.5$ — EC2 Eq. 4.20

$V_{Rd2} = 1/2 x 0.6 x 13.33/10 x 35 x 0.9 x 45 = 567$ KN

7.4.2 Design for shear at support B

from table 4.1 Max. $V_B = 186$ KN

Determine the design shear force at a distance d from the face of a direct support V_{sd} — EC2 4.3.2.2(10)

$V_{sd,B} = 186 - 51(0.15 + 0.5) = 153$ KN

$V_{sd} > V_{Rd1}$ Design for shear is required

Determine the design shear resistance of a section V_{Rd3} — EC2 4.3.2.4.3(1)

$V_{Rd3} = V_{cd} + V_{wd}$

V_{cd} = Shear capacity of the concrete compression zone $= V_{Rd1}$ — Eq. 4.22

The contribution of shear reinforcement V_{wd} can be written as:	Code reference
$V_{wd} = V_{Rd3} - V_{cd} = 97$ KN	
$V_{wd} = A_{sw}/s\ 0.9\ \ d\ f_{ywd}$	EC2 4.3.2.4.3(2) Eq. 4.23
A_{sw} = the cross- sectional area of the shear reinforcement. s = the spacing of the stirrups. f_{ywd} = the design yield strength of the shear reinforcement.	
Equation 4.23 can be written as:	
$A_{sw}/s = V_{wd}/\ (0.9\ d\ f_{ywd})$ $= 97(10)/(0.9\ .\ 45\ .\ 348) = 6.88\ cm^2/m$	
with stirrups dia = 8 mm < 12 mm $s = 1.006\ \ 10^2/6.88 = 14.6$ cm	
A_{sw} provided Ø8/14 cm = 7.18 cm^2/m	
7.4.3 Design for shear at support A and C	
Determine the shear at a distance d from the face of support	EC2 4.3.2.2(10)
$V_{sd} = 124 - 51(0.15 + 0.5) = 91$ KN	
$V_{sd} > V_{Rd1}$ Design for shear is required	
$V_{wd} = V_{Rd3} - V_{cd} = 35$ KN	
$V_{wd} = Asw/s\ 0.9\ \ d\ f_{ywd}$	EC2 4.3.2.4.3(2) Eq. 4.23
$A_{sw}/s = 2.48\ cm^2/m$	
7.4.4 Check Minimum Shear Reinforcement Ratio	
Minimum value for ρ_w for C 20/25 and S400 is:	
Min $\rho_w = 0.0009$	EC2 5.4.2.2(5) Tab. 5.5
The shear ratio for vertical stirrups is given by equation 5.16	
$\rho_w = A_{sw}/s.b_w.\ \sin\alpha$	EC 2 5.4.2.2(5) Eq. 5.16
where: α = the angle between the shear reinforcement and the longitudnal reinforcement	
$\rho_w = 2.48\ \ 10^{-2}/35\ \ = 0.00071 <$ min $\rho_w = 0.0009$	
Req. $\rho_w = 0.0009$	
Req. $A_{sw}/s = 0.0009\ .\ 10^2\ .\ 35 = 3.15\ cm^2/m$	

A_{sw} provided $\phi 8/30$ cm = 3.35 cm^2/m
The max longitudinal spacing Smax.
for $V_{sd} < 1/5\ V_{Rd2}$:

$S_{max.}$ = 0.8 d = 0.8 45 = 36 cm > 30 cm

Check minimum shear reinforcement at support B

$\rho_w = A_{sw}/s.b_w = 0.0021 > \rho_{w\,min.}$

The max longitudinal spacing Smax. at support B
where $1/5\ V_{Rd2} < V_{sd} < 2/3\ V_{Rd2}$ is:

$S_{max.}$ = 0.6 d = 0.6 45 = 27 cm > 14 cm

provided stirrups $\phi 8/14$ cm satisfy the minimum shear reinf. and max. spacing requirements

Code reference

EC2 5.4.2.2(7) Eq. 5.17

EC2 5.4.2.2(7) Eq. 5.18

7.5 Control of Deformation and Cracking

7.5.1 Control of cracking: — EC2 4.4.2.3

Control of cracking can be generally achieved by limiting bar spacing and bar and/or bar diameter. — EC2 4.4.2.3(3)

Control of cracking for max. moment at support B

Determine the moment under quasi-permanent actions

$M_{sd,B}$ = 27 . $6^2/8$ = 121.5 KN.m
$As_{prov.}$ = 14.1 cm^2
$As_{req.}$ = 12.6 cm^2

The steel stress can be calculated as follows:

$$\sigma_{s,B} = M_{sd}/(As_{prov.} \cdot 0.85d)$$

$$= 121.5/(14.1 \cdot 0.85 \cdot 0.45)) = 225 \text{ MPa}$$

The max. bar diameter to control the cracking without direct calculation — EC2 Tab. 4.11

ϕ^*_s = 20 mm
ϕ^*_s can be modified as follows:
$\phi_s = \phi^*_s\ h/10(h-d) > \phi^*s$
$= 20$ mm $> \phi_{s\ prov.} = 16$ mm

The max. bar spacing = 220mm > $S_{prov.}$ = 83 mm — EC2 Tab. 4.14

7.5.2 Control of Deformation — EC2 4.4.3

The deformation can be controled without direct calculations as follows:
Detemine the steel ratio — EC2 4.4.3.2(5)

$\rho = As/bd = 14.1/(35x45) = 0.9\%$
Determine L_{eff}/d ratio

for ρ = 0.5% L_{eff}/d = 32 for = 1.5% L_{eff}/d = 23 for = 0.9% L_{eff}/d = 28.4 linear interpoliation	Code reference
Check steel stress under design service load	EC2 4.4.3.2(4)
$250/\sigma_s = 400/(f_{yk}.As_{req.}/As_{prov.})$ σ_s = 234 MPa The values of table 4.14 should be multiplied by $250/\sigma_s = 1.07$	
Required effective depth $d_{req.} = 600/(28.4x1.07) = 19.1$ cm > $d_{prov.} = 45$ cm	
7.5.3 Check the min. reinforcement	
The required min. reinforcement ($As_{min.}$) may be calculated from the given relation as: $As_{min.} = K_c\ K\ f_{ct.ef}\ A_{ct}/\sigma_s$	
K_c = 0.14 for sections subject to bending without normal compessive forcer K = 0.5 for rectangular sections with h>80 cm = 0.8 for rectangular sections with h<30 cm = 0.68 for h = 50 cm linear interpoliation	
$f_{ct.ef}$ = 2.2	EC2 Tab. 3.1
A_{ct} = 0.5 bh = 0.5x35x50 = 875 cm^2	
σ_s = 240 for Ø = 16 mm	EC2 Tab. 4.11
As_{min} = 0.4x0.68x2.2x875/240 = 2.2 cm^2 > $As_{prov.}$	

8. CONCLUSION AND COMPARISON

The amount of tension reinforcement for the maximum negative and positive bending moment, and the amount of shear reinforcement calculated according to EC2 and ACI Code are tabulated:

		EC2	ACI Code
M_{sd}, B	As(cm^2)	12.6	17.6
M_{sd}, 1,2	As(cm^2)	10.0	10.3
V_B	Asw/s(cm^2/m)	6.88	4.8
V_A, V_C	Asw/s(cm^2/m)	3.15	2.5

The amount of tension reinforcement calculated according to EC2 gives smaller amount at support B than the one calculated according to the ACI Code. This can be explained in the following:

-Moment calculated by elastic analysis is redistributed using EC2.
-The partial safety factors used by EC2 are smaller for permanent and variable action than the one used by the ACI Code.
-The EC2 uses also partial safety factor for material properties.

The amount of shear reinforcement calculated by the ACI Code is smaller than the one calculated by the EC2.

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