ANNEX 7

The Finnish National Annex to the standard SFS-EN 1992-1-1

Eurocode 2: Design of concrete structures - Part 1: General rules and rules for buildings

Foreword

This national Annex is ment to be used together with the standard EN 1992-1-1 and the standards EN 1990 and EN 1991.

In this National Annex the Nationally Determined Parameters and additional rules are presented where national choices may have to be made:

2.3.3 (3)	Deformations of concrete
2.4.2.2 (1)	Partial factors for prestress
3.1.2	Strength
3.1.6 (1)	Design compressive and tensile strengths
3.2.2 (3)	Properties
3.2.7 (2)	Design assumptions
3.3.6 (7)	Design assumptions
4.4.1.2 (5)	Minimum cover, c_{min}
4.4.1.3 (4)	Allowance in design for tolerance
5.5 (4)	Linear analysis with limited redistribution
5.8.5 (1)	Methods of analysis
5.10.1 (6)	General
5.10.2.2 (4) (5)	Limitation of concrete stress
5.10.8 (2)	Effects of prestressing at ultimate limit state
5.10.9 (1)	Effects of prestressing at serviceability limit state and limit state of fatigue
6.4.	Punching
6.8.4 (1) (5)	Verification procedure for reinforcing and prestressing steel
7.2 (5)	Stresses
7.3.1 (5)	General considerations
7.3.4 (3)	Calculation of crack widths
7.4.2 (2)	Cases where calculations may be omitted
8.3 (2)	Permissible mandrel diameters for bent bars
9.2.1.1 (3)	Minimum and maximum reinforcement areas
9.2.1.2 (1)	Other detailing arrangements
9.3.1.1(1)	General
9.4.3	Punching shear reinforcement
9.5.2 (3)	Longitudinal reinforcement
9.5.3 (3)	Transverse reinforcement
9.6.2 (1)	Vertical reinforcement
9.7 (1)	Deep beams
9.8.4 (1)	Column footing on rock
9.10.2.3 (4)	Internal ties
12.3.1 (1)	Concrete: additional design assumptions

ANNEX A Modification of partial factors for materials

The recommendations given in the standard SFS-EN 1992-1-1 apply in the following clauses:

2.4.2.1 (1)	6.2.2 (1)	9.5.2 (2)
2.4.2.2 (2) (3)	6.2.2 (6)	9.6.3 (1)
2.4.2.3 (1)	6.2.3 (2)	9.8.1 (3)
2.4.2.4 (1)	6.2.3 (3)	9.8.2.1 (1)
2.4.2.4 (2)	6.2.4 (4)	9.8.3 (1)
2.4.2.5 (2)	6.2.4 (6)	9.8.3 (2)
3.1.2 (2)	6.5.2 (2)	9.8.5 (3)
3.1.6 (2)	6.5.4 (4)	9.10.2.2 (2)
3.3.4 (5)	6.5.4 (6)	9.10.2.3 (3)
4.4.1.2(3)	6.8.6 (1)	9.10.2.4 (2)
4.4.1.2 (6)	6.8.6 (3)	11.3.5 (1)P
4.4.1.2 (7)	6.8.7 (1)	11.3.5 (2)P
4.4.1.2 (8)	7.2 (2)	11.3.7 (1)
4.4.1.2 (13)	7.2 (3)	11.6.1 (1)
4.4.1.3 (3)	7.3.2 (4)	11.6.1 (2)
4.4.1.3 (4)	8.2 (2)	11.6.2 (1)
5.1.3 (1)P	8.6 (2)	11.6.4.1 (1)
5.2 (5)	8.8 (1)	12.3.1 (1)
5.6.3 (4)	9.2.1.1 (1)	12.6.3 (2)
5.8.3.1 (1)	9.2.1.4 (1)	C.1 (1)
5.8.3.3 (1)	9.2.2 (4)	C.1 (3)
5.8.3.3 (2)	9.2.2 (5)	E.1 (2)
5.8.6 (3)	9.2.2 (6)	J.1 (2)
5.10.2.1 (1)P	9.2.2 (7)	J.2.2 (2)
5.10.2.1 (2)	9.2.2 (8)	J.3 (2)
5.10.3 (2)	9.5.2 (1)	J.3 (3)
5.10.8 (3)		

The clauses 6.4 Punching and 9.4.3 Punching shear reinforcement do not apply for the time being. For explanation please refer to these clauses in this national annex.

In addition instructions are given in this national annex in use of informative annexes A, B, C, D, E, F, G, H, I and J.

2.3.3 Deformations of concrete

In building structures, temperature and shrinkage effects may be omitted in global analysis provided joints are incorporated at every distance d_{joint} to accommodate resulting deformations. The joints are always designed separately especially taking into account the way of foundation.

2.4.2.2 Partial factors for prestress

(1) Prestress in most situations is intended to be favourable and for the ultimate limit state verification the value of $\gamma_{P,fav} = 0,9$ should be used. The design value of prestress may be based on the mean value of the prestressing force (see EN 1990 Section 4).

2.4.2.4 Partial factors for materials

(1) Partial factors for materials for ultimate limit states, γ_C and γ_S should be used.
The recommended values for 'persistent & transient' and 'accidental, design situations are given in Table 2.1N-FI. These are not valid for fire design for which reference should be made to EN 1992-1-2. For fatigue verification the partial factors $\gamma_{C,fat}$ and $\gamma_{S,fat}$ for persistent design situations given in Table 2.1N-FI shall be used.

Table 2.1N: Partial factors for materials for ultimate limit states

Design situations	γ_C for concrete	γ_S for reinforcing steel	γ_S for prestressing steel
Persistent & Transient	1,5	1,15	1,15
Accidential	1,2	1,0	1,0

Explanation: The partial factors are identical with the recommended values. They are presented here because they are linked with the reduction factors in annex A.

3.1.2 Strength

(4) The reduction factor k_t for the compressive strength for concrete after 28 days can be taken as 1,0.

3.1.6 Design compressive and tensile strengths

(1)P The value of the design compressive strength is defined as

$$f_{cd} = \alpha_{cc} f_{ck} / \gamma_C \quad (3.15)$$

where:

γ_C is the partial safety factor for concrete, see 2.4.2.4, and
$\alpha_{cc} = 0,85$ is the coefficient with which long term effects on the compressive strength and unfavourable effects resulting from the way the load is applied are taken into account.

3.2.2 Properties

(3)P The application rules for design and detailing in this Eurocode are valid up to a specified yield strength, f_{yk} = 400…700 MPa.

3.2.7 Design assumptions

(2) For normal design, either of the following assumptions may be made (see Figure 3.8):
  a) an inclined top branch with a strain limit of ε_{ud} and a maximum stress of kf_{yk}/γ_s at ε_{uk}, where $k = (f_t/f_y)_k$,
  b) a horizontal top branch without the need to check the strain limit.

Note 1: The value of ε_{ud} is 1 %

3.3.6 Design assumptions

(7) For cross-section design, either of the following assumptions may be made (see Figure 3.10):

- an inclined branch, with a strain limit e_{ud} = 2 %. The design may also be based on the actual stress/strain relationship, if this is known, with stress above the elastic limit reduced analogously with Figure 3.10, or

- a horizontal top branch without strain limit.

4.4.1.2 Minimum cover, c_{min}

(5) The minimum cover values for ordinary and prestressing steels in normal weight concrete taken account of the exposure classes and the structure classes is given by $c_{min,dur}$.

The values of $c_{min,dur}$ are given in Table 4.3N-FI (when the design working life is 50 years)

Table 4.3N-FI. The minimum cover values

The minimum cover values $c_{min,dur}$ according to exposure classes (mm)								
Criteria	Exposure Class according to Table 4.1							
	X0	XC1	XC2 XC3	XC4	XD1	XS1	XD2	XD3 XS2,3
Reinforcement steel	10	10	20	25	30	30	35	40
Prestressing steel	10	20	30	35	40	40	45	50
Design working life 100 years [1)]	+0	+0	+5	+5	+5	+5	+5	+5
Concrete strength ≥	C20/25 -5	C30/37 -5	C35/45 -5	C35/45 -5	C35/45 -5	C40/50 -5	C35/45 -5	C45/55 -5
Construction class 1 (RakMK B4)	-5	-5	-5	-5	-5	-5	-5	-5

[1)] If the design working life of the structure is 100 years, also the other durability requirements shall be checked according to RakMK B4 (SFS-EN 206-1 National annex).

Other $c_{min,b}$ values may be used if a generally accepted working life design is made.

4.4.1.3 Allowance in design for deviation

(1)P An addition to the minimum cover shall be made in design to allow for the deviation (c_{tdev}). The required minimum cover shall be increased by the accepted negative deviation given in the standard for execution. This may depend on the type of structure.

The allowed deviation Δc_{dev} is normally 10 mm. A producer of precast elements may adopt a tolerance below 10 mm for different types of products, where it is justified according to the certified factory quality management system. However, any tolerance below 5 mm may not be adopted.

(4) For concrete cast against uneven surfaces, the minimum cover should generally be increased by allowing larger deviations in design. The increase should comply with the difference caused by the unevenness, but the cover should be at least $k_1 = c_{min} + 10$mm for concrete cast against prepared ground (including blinding) and $k_2 = c_{min} + (20 \ldots 40)$ mm for concrete cast directly against soil. The cover to the reinforcement for any surface feature, such as ribbed finishes or exposed aggregate, should also be increased to take account of the uneven surface (4.4.1.2 (11)).

5.5 Linear elastic analysis with limited redistribution

(4) In continuous beams or slabs which:
a) are predominantly subject to flexure and
b) have the ratio of the lengths of adjacent slabs in the range of 0,5 to 2, redistribution of bending moments may be carried out without explicit check on the rotation capacity, provided that:

$\delta \geq k_1 + k_2 x_u/d$ for $f_{ck} \leq 50$ MPa (5.10a)
$\delta \geq k_3 + k_4 x_u/d$ for $f_{ck} > 50$ MPa (5.10b)

$\geq k_5$ where Class B and Class C reinforcement is used (Annex C)
$\geq k_6$ where Class A reinforcement is used (Annex C)

Where:

δ is the ratio of the redistributed moment to the elastic bending moment
x_u is the depth of the neutral axis at the ultimate limit state after redistribution
d is the effective depth of the section.

$k_1 = 0{,}44$ when $f_{ck} \leq 50$ MPa
$k_2 = 1{,}10$ when $f_{ck} \leq 50$ MPa

$k_3 = 0{,}54$ when $f_{ck} > 50$ MPa
$k_4 = 1{,}25\ (0{,}6\text{-}0{,}0014/\varepsilon_{cu2})$ when $f_{ck} > 50$ MPa

$k_5 = k_6 = 1$ when $100\ \varepsilon_{uk} f_t / f_{yk} < 2{,}5$

$k_5 = k_6 = 0{,}9\text{-}3{,}21\ \varepsilon_{uk} f_t / f_{yk} \geq 0{,}67$ when $100\ \varepsilon_{uk} f_t / f_{yk} \geq 2{,}5$

$k_5 = k_6 = 1$ when $100\ \varepsilon_{uk} f_t / f_{yk} < 2{,}5$

Note: f_{yk} can also be $f_{0,2}$
$\varepsilon_{uk} f_t / f_{yk} = A_{gt}\, R_m/R_e$

For reinforcement where ($100\, A_{gt}\, R_m/R_e$) < 4 no redistribution is allowed when $x_u/d < 0{,}12$.

5.8.5 Methods of analysis

(1) The methods of analysis include a general method, based on non-linear second order analysis, see 5.8.6 and the following two simplified methods:

(a) Second order analysis based on nominal stiffness, see (2) below
(b) Method based on estimation of curvature, see (2) below

The designer chooses the case-specific method.

5.10.1 General

(6) Brittle failure should be avoided by one or more of the following methods:

Method A: Provide minimum reinforcement in accordance with 9.2.1.
Method B: Provide pretensioned bonded tendons.
Method C: Provide easy access to prestressed concrete members in order to check and control the condition of tendons by non-destructive methods or by monitoring.
Method D: Provide satisfactory evidence concerning the reliability of the tendons.
Method E: Ensure that if failure were to occur due to either an increase of load or a reduction of prestress under the frequent combination of actions, cracking would occur before the ultimate capacity would be exceeded, taking account of moment redistribution due to cracking effects.

The designer chooses the case-specific method A…E.

5.10.2.2 Limitation of concrete stress

(4) If prestress in an individual tendon is applied in steps, the required concrete strength may be reduced. The minimum strength $f_{cm}(t)$ at the time t should be *20* % of the required concrete strength for full prestressing given in the European Technical Approval. Between the minimum strength and the required concrete strength for full prestressing, the prestress may be interpolated between 0 and 100% of the full prestressing.

(5) For pretensioned elements the stress at the time of transfer of prestress may be increased to $0{,}65 \cdot f_{ck}(t)$, if it can be justified by tests or experience that longitudinal cracking is prevented.

5.10.8 Effects of prestressing at ultimate limit state

(2) For prestressed members with permanently unbonded tendons, it is generally necessary to take the deformation of the whole member into account when calculating the increase of the stress in the prestressing steel. If no detailed calculation is made, it may be assumed that the increase of the stress from the effective prestress to the stress in the ultimate limit state is

$\Delta\sigma_{p,ULS}$ = 50 MPa.

(3) If the stress increase is calculated using the deformation state of the whole member the mean values of the material properties should be used. The design value of the stress increase $\Delta\sigma_{pd} = \Delta\sigma_p\ \gamma_{\Delta P}$ should be determined by applying partial safety factors $\gamma_{\Delta P,sup} = \gamma_{\Delta P,inf} = 1{,}0$ in all cases.

5.10.9 Effects of prestressing at serviceability limit state and limit state of fatigue

(1)P For serviceability calculations the characteristic value of the prestressing force can be used.

6.4 Punching

The punching design is not for the time being made according to the standard EN 1991-1-2. Instead the punching design is made according to clause 2.2.2.7 in the National Building Code of Finland, part B4 Concrete Structures, guidelines.

Explanation:
In the design rules for the punching in the standard certain contradictions with the test results has been noticed, so that the capacities according to the standard are in certain cases on unsafe side.

6.8.4 Verification procedure for reinforcing and prestressing steel

(1) The damage of a single load amplitude $\Delta\sigma$ may be determined by using the corresponding S-N curves (Figure 6.30) for reinforcing and prestressing steel, the applied load should be multiplied by $\gamma_{F,fat} = 1{,}0$. The resisting stress range at N^* cycles $\Delta\sigma_{Rsk}$ obtained should be divided by the safety factor $\gamma_{S,fat} = 1{,}0$.

(5) When the rules of 6.8 are used to evaluate the remaining life of existing structures, or to assess the need for strengthening, once corrosion has started the stress range can be determined by reducing the stress exponent k_2 for straight and bent bars.

The fatigue values of $\Delta\sigma_{Rsk}$, k_1 and k_2 are taken from the steel specification.

7.2 Stresses

(5) Unacceptable cracking or deformation may be assumed to be avoided if, under the characteristic combination of loads, the tensile stress in the reinforcement does not exceed $0{,}6f_{yk}$. Where the stress is caused by imposed deformations, the tensile stress should not exceed $0{,}8f_{yk}$. The mean value of the stress in prestressing tendons should not exceed $0{,}6f_{pk}$.

7.3.1 General considerations

(5) A limiting calculated crack width, w_{max}, taking into account of the proposed function and nature of the structure and the costs of limiting cracking, should be established.

The values of w_{max} for relevant exposure classes are given in Table 7.1N-FI.

Table 7.1N-FI Values of w_{max} (mm)

Exposure Class	Reinforced members and prestressed members with unbonded tendons	Prestressed members with bonded tendons
	Quasi-permanent load combination	Frequent load combination
X0, XC1	0,4[1]	0,2
XC2, XC3, XC4 XD1, XS1	0,3	0,2[2]
XD2, XD3 XS2, XS3,	0,2	Decompression
Note 1: For X0, XC1 exposure classes, crack width has no influence on durability and this limit is set to guarantee acceptable appearance. In the absence of appearance conditions this limit may be relaxed. **Note 2:** For these exposure classes, in addition, decompression should be checked under the quasi-permanent combination of loads.		

Design rules for retaining structures are presented in part SFS-EN 1992-3.

7.4.2 Cases where calculations may be omitted

2) Provided that reinforced concrete beams or slabs in buildings are dimensioned so that they comply with the limits of span to depth ratio given in this clause, their deflections may be considered as not exceeding the limits set out in 7.4.1 (5) and (6). The limiting span/depth ratio may be estimated using Expressions (7.16.a) and (7.16.b) and multiplying this by correction factors to allow for the type of reinforcement used and other variables. No allowance has been made for any pre-camber in the derivation of these Expressions.

$$\frac{l}{d} = K\left[11 + 1{,}5\sqrt{f_{ck}}\frac{\rho_0}{\rho} + 3{,}2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right] \quad \text{if } \rho \leq \rho_0 \qquad (7.16a)$$

(7.16.a)

$$\frac{l}{d} = K\left[11 + 1{,}5\sqrt{f_{ck}}\frac{\rho_0}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_0}}\right] \quad \text{if } \rho > \rho_0 \qquad (7.16b)$$

(7.16.b)

Values of K are given in Table 7.4N. Values obtained using Expression (7.16) for common cases (C30, σ_s = 310 MPa, different structural systems and reinforcement ratios ρ = 0,5 % and ρ = 1,5 %) are also given.

Table 7.4N-FI: Basic ratios of span/effective depth for reinforced concrete members without axial compression

Structural System	K	Concrete highly stressed ρ = 1,5%	Concrete lightly stressed ρ = 0,5%
Simply supported beam, one- or two-way spanning simply supported slab	0,8	11	16
End span of continuous beam or one-way continuous slab or two-way spanning slab continuous over one long side	1,0	15	22
Interior span of beam or one-way or two-way spanning slab	1,2	17	24
Slab supported on columns without beams (flat slab) (based on longer span)	1,0	14	20
Cantilever	0,3	4	6
Note 1: The values given have been chosen to be generally conservative and calculation may frequently show that thinner members are possible. **Note 2:** For 2-way spanning slabs, the check should be carried out on the basis of the shorter span. For flat slabs the longer span should be taken. **Note 3:** The limits given for flat slabs correspond to a less severe limitation than a mid-span deflection of span/250 relative to the columns. Experience has shown this to be satisfactory.			

The values given by Expression (7.16) and Table 7.4N have been derived from results of a parametric study made for a series of beams or slabs simply supported with rectangular cross section, using the general approach given in 7.4.3. Different values of concrete strength class and a 500 MPa characteristic yield strength were considered. For a given area of tension reinforcement the ultimate moment was calculated and the quasipermanent load was assumed as 50% of the corresponding design load. The span/depth limits obtained satisfied the limiting deflection given in 7.4.1(5).

8.2 Spacing of bars

(2) The clear distance (horizontal and vertical) between individual parallel bars or horizontal layers of parallel bars should be not less than the maximum of bar diameter, ($d_g + 3$ mm) or 20 mm where d_g is the maximum size of aggregate.

8.3 Permissible mandrel diameters for bent bars

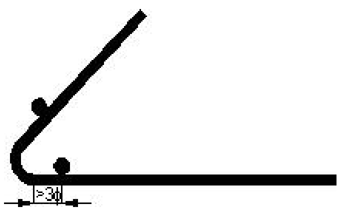
(2) In order to avoid damage to the reinforcement the diameter to which the bar is bent (Mandrel diameter) should not be less than $\phi_{m,min}$.

Table 8.1N: Minimum mandrel diameter to avoid damage to reinforcement

a) bars and wires

Minimum mandrel diameter for bends, hooks and loops (see Figure 8.1) are at least 2 times the values of the bending test

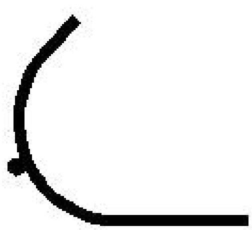
b1) for welded bent reinforcement and mesh bent after welding (outside HAZ)

	When the welding point is outside the area influenced by the welding heat (HAZ) the mandrel diameter is as in clause a). The HAZ area can be taken as 3Ø to both sides of the welding point

b2) for welded bent reinforcement and mesh bent after welding (outside HAZ) and the welding point on the inside of the bending

	$Ø_{m,min}$ = 2,0 times the value given in clause a) $Ø_{m,min}$ = 1,5 times the value given in clause a) when reinforcement SFS 1202 or CEN/TR 15481 is used

b3) for welded bent reinforcement and mesh bent after welding (inside HAZ) and the welding point on the outside of the bending

	$Ø_{m,min}$ = 5,0 times the value given in clause a) $Ø_{m,min}$ = 3,0 times the value given in clause a) when reinforcement SFS 1202 or CEN/TR 15481 is used

c) load bearing welding

Bending of reinforcement with load bearing welding requires always special safety and quality assurance arrangements

Note: Non-load bearing welding can be done in bended areas when reinforcement SFS 1202 or CEN/TR

15481 is used. The mandrel diameter shall be as in clause 8.1N-FI.

9.2.1.1 Minimum and maximum reinforcement areas

(3) The cross-sectional area of tension or compression reinforcement is not limited.

9.2.1.2 Other detailing arrangements

(1) In monolithic construction, even when simple supports have been assumed in design, the section at supports should be designed for a bending moment arising from partial fixity of at least 15% of the maximum bending moment in the span if the fixity is not analysed more accurately.

9.3.1.1 General

(3) The spacing of bars should not exceed $s_{\mathrm{max,slabs}}$.

- for the principal reinforcement, $3h \leq 400$ mm, where h is the total depth of the slab;
- for the secondary reinforcement, $4{,}0h \leq 450$ mm .

In areas with concentrated loads or areas of maximum moment those provisions become respectively:
- for the principal reinforcement, $2h \leq 250$ mm
- for the secondary reinforcement, $3h \leq 400$ mm.

9.4.3 Punching shear reinforcement

This clause of the standard does not comply for the time being, see the application rules and explanations given in clause 6.4 in this National Annex.

9.5.2 Longitudinal reinforcement

(3) The area of reinforcement should not exceed $A_{s,max} = 0{,}06\mathrm{A}_{\mathrm{c}}$. This limit should be increased to $0{,}12\ A_{\mathrm{c}}$ at laps.

9.5.3 Transverse reinforcement

(3) The spacing of the transverse reinforcement along the column should not exceed $s_{cl,tmax}$.
The value is the least of the following three distances:

- 20 times the minimum diameter of the longitudinal bars
- the lesser dimension of the column
- 400 mm

9.6.2 Vertical reinforcement

(1) The area of the vertical reinforcement should lie between $A_{s,vmin} = 0{,}002\mathrm{A}_{\mathrm{c}}$ and $A_{s,vmax} = 0{,}06\mathrm{A}_{\mathrm{c}}$

9.7 Deep beams

(1) Deep beams (for definition see 5.3.1 (2), (3)) should normally be provided with an orthogonal reinforcement mesh near each face, with a minimum of $A_{s,dbmin}$.

The minimum area is $A_{s,dbmin}$ = 0,0005 $A_{c,}$ but at least 150 mm^2/m within both sides and both directions.

9.8.4 Column footing on rock

(1) Adequate transverse reinforcement should be provided to resist the splitting forces in the footing, when the ground pressure in the ultimate states exceeds q_2 = 3 MPa. This reinforcement may be distributed uniformly in the direction of the splitting force over the height h (see Figure 9.16). A minimum bar diameter d_{min} = 8 mm should be provided.

9.10.2.3 Internal ties

(4) In floors without screeds where ties cannot be distributed across the span direction, the transverse ties may be grouped along the beam lines. In this case the minimum force on an internal beam line is:

$$F_{\mathrm{tie}} = (l_1 + l_2)/\ 2 \cdot q_3 \geq q_4 \qquad (9.16)$$

where:

$l_{1,}$ l_2 are the span lengths (in m) of the floor slabs on either side of the beam (see Figure 9.22)
q_3 = 20 kN/m and q_4 = 70 kN.

A greater value than F_{tie} = 150 kN is not to be used unless resulting from the loading .

12.3.1 Concrete: additional design assumptions

(1) Due to the less ductile properties of plain concrete the following values $\alpha_{cc,pl} = 0{,}8\alpha_{cc}$ and $\alpha_{ct,pl} = 0{,}6\alpha_{ct}$ are used.

$\alpha_{ct,pl} = 0{,}6\alpha_{ct}$ is chosen smaller than because the internal stresses decrease the tension strength more than the compression strength.

Annex A (informative)

Modification of partial factors for materials

This annex A can be used as informative. The rules in clause A.2 are also to be used concerning the prefabricated constructions in A.3.

Note: Construction class 2 in RakMK B4 can be taken as the execution class 1 inspected as in class 2 in the standard ENV 13670-1. Also the partial coefficients given in table 2.4.2.4 correspond with the Construction class 2 in RakMK B4.

A.2.1 Reduction based on quality control and reduced deviations

(1) If execution is subjected to a quality control system, which ensures that unfavourable deviations of cross-section dimensions are within the reduced deviations given in Table A.1, the partial safety factor for reinforcement may be reduced to $\gamma_{s,red1.}$

The stipulations are considered to be fulfilled in Construction class 1 according to RakMK B4 with the tolerances given there, or with CE-marked prefabricated constructions, which have a certified quality assurance and reduced deviations according to table A.1.

The partial safety factor can be reduced to $\gamma_{s,red1} = 1, 1$.

(2) Under the condition given in A.2.1 (1), and if the coefficient of variation of the concrete strength is shown not to exceed 10 %, the partial safety factor for concrete may be reduced to $\gamma_{C,red1.}$

The stipulations are considered to be fulfilled in Construction class 1 according to RakMK B4 with the tolerances given there, or with CE-marked prefabricated constructions, which have a certified quality assurance and reduced deviations according to table A.1.

The partial safety factor can be reduced to $\gamma_{c,red1} = 1, 35$.

A.2.2 Reduction based on using reduced or measured geometrical parameters in design

(1) If the calculation of design resistance is based on critical dimensions, including effective depth (see Figure A.1), which are either:

- reduced by deviations, or
- measured in the finished structure,

the partial safety factors may be reduced to $\gamma_{s,red2}$ = 1,05 *and* $\gamma_{c,red2}$ = 1,45.

(2) Under the conditions given in A.2.2 (1) and provided that the coefficient of variation of the concrete strength is shown not to exceed 10%, the partial factor for concrete may be reduced to $\gamma_{c,red3}$ = 1,35.

A.2.3 Reduction based on assessment of concrete strength in finished structure

(1) For concrete strength values based on testing in a finished structure or element, see EN 13791, EN 206-1 and relevant product standards, γ_c may be reduced by the conversion factor
η = 0,85.

The value of γ_c to which this reduction is applied may already be reduced according to A.2.1 or A.2.2. However, the resulting value of the partial factor should not be taken less than
$\gamma_{c,red4}$ = 1,2.

If the factor η has already been taken into account when estimating the strength in a finished structure (EN 1379 1: η = 0,85 or RakMK B4 6.3.3.4 η = 0,85 Construction Class 1 and η = 0,80 Construction Class 2), the safety factor for concrete γ_c may not be reduced.

ANNEX B (Informative)
Creep and shrinkage strain

ANNEX D (Informative)
Detailed calculation method for prestressing steel relaxation losses

Annex E (Informative)
Indicative strength classes for durability

The table E.1N doesn't apply in Finland. The table F.1-FI in the National Annex of standard EN 206-1 (RakMK B4 annex 3).

Annex F (Informative)
Tension reinforcement expressions for in-plane stress conditions

Annex G (Informative)
Soil structure interaction

Annex H (Informative)
Global second order effects in structures

Annex I (Informative)
Analysis of flat slabs and shear walls

LIITE J Examples of areas where operations model of the structure changes.

J.1 Surface reinforcement

This annex doesn't apply.

J.2 Frame corners

This annex doesn't apply.

J.3 Corbels

This annex can be used as informative.