

NATIONAL BUILDING CODE OF FINLAND

Strength and stability of structures

Concrete structures



Ympäristöministeriö
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Foreword

The Ministry of the Environment publishes the recommendations for strength and stability related to the design of concrete structures in the National Building Code of Finland. The instruction contains a compilation of all the National Annexes concerning the design of concrete structures.

The beginning of each National Annex presents those clauses in the standard where national choice is permitted, and where such a choice has been made.

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Head of the Buildings and Construction unit
Building Counsellor

Teppo Lehtinen

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1. Scope

These instructions provide additional information when applying the Ministry of Environment Decree on load-bearing structures in the design and execution of concrete structures. Where applicable, the instructions also apply to concrete structures used in composite structures, such as concrete and wood structures. A solution pursuant to these instructions is considered to meet the requirements set for load-bearing structures.

These instructions are applied when concrete structures are designed pursuant to standards SFS-EN 1992 and their National Annexes, and implemented pursuant to standards SFS-EN 13670 and SFS 5975.

Harmonised product standards concerning precast concrete products also include supplementary rules and requirements concerning the design and manufacture of concrete elements.

2. Design of structures

2.1 Execution documents for structures

Standards SFS-EN 13670 and SFS 5975 provide instructions on the preparation of the execution documents and the execution specification for concrete structures.

Usually, the execution documents include, at a minimum, the following:

- a) construction drawings
- b) requirements pursuant to standards SFS-EN 13670 and SFS 5975, such as the execution classes and tolerance classes
- c) other documents to be adhered to or references to other documents.

2.2 Contents of the structural designs

Usually, the structural designs for concrete structures present, at a minimum, the following to the extent applicable to the design task:

- a) consequence class
- b) exposure classes and the design working life of the structure
- c) the R/E/I/M fire resistance class for the structural components
- d) the adopted characteristic loads and category of use
- e) complete information on the dimensions and location of the structures
- f) execution class

- g) tolerance class
- h) concrete strength class
- i) the minimum concrete cover and its allowed deviation
- j) the upper nominal limit for the aggregate used in the concrete
- k) identification information for the reinforcing steel and the reinforcement elements
- l) the number, diameter, length, form and bending radius of the reinforcement bars and their location and splices
- m) for prestressing reinforcements, the number, diameter, length, form, bending radii, location and couplers of strands/bars, the tendon type and the location of the grouting pipes and auxiliary pipes
- n) information on the locations of the fastenings and boxouts etc.
- o) the drawings for structures in execution class 3 also present the supporting of the reinforcement and the reinforcements needed execution phase
- p) the precise location and execution method of the cold joints when the structure is in execution class 3. In other execution classes, the designer may leave the location and execution method of the cold joints which have little significance to the structure to be decided at the worksite.
- q) requirements for curing (those determined by the exposure class do not need to be presented)
- r) allowed deviations (those determined by the exposure class do not need to be presented)
- s) requirements for other materials and construction products.

The following are also presented for factory-made construction components (included in manufacturing or installation drawings):

- t) for precast concrete products, the information required for the assessment of the conformity and design
- u) the CE marking method adopted for the prefabricated elements (M1, M2, M3a or M3b)
- v) the weight and centroid location for the precast concrete element
- w) the minimum support lengths/areas
- x) lifting loops/inserts and their placement
- y) handling, support and lifting instructions if necessary.

The requirements set by the manufacturing technology should be taken into account in the structural design. If the structure or reinforcement is difficult to manufacture, the needed special measures for execution should be presented in the drawings or in the work specification. When these special measures are followed the different work phases and their supervision will meet the requirements for execution .

The load-bearing joints and joint widths should be designed in a manner that allows the joints to be filled in properly with the working method used.

2.3 Execution classes

The requirements set for the implementation of concrete structures are divided into three execution classes according to the difficulty of the structure. The execution classes are presented in standard SFS-EN 13670. The execution classes concern concrete structures that are manufactured at the worksite. As regards precast concrete products according to harmonised product standards, the execution classes only concern their erection at the worksite.

The execution class is selected on the basis of standard SFS-EN 1990 and the consequences classes (CC1, CC2 and CC3) and risk factors related to structure and its use. The execution class and its related requirements are defined as follows on the basis of the consequences class of the structure or a part thereof:

- structures in consequences class CC2 are in execution class 2, at a minimum
- structures in consequences class CC3 are in execution class 3
- structures made of high strength concrete are in execution class 3. Concrete with a strength class higher than C50/60 is considered high strength concrete
- structures and structural members whose execution is considered to be especially demanding and whose manufacturing requires special care in order to ensure their structural functionality are in execution class 3. Structural members that are critical when considering of progressive collapse and in-situ tensioned concrete structures are considered especially demanding
- if the structural design has applied tolerance class 2 and the lower partial factors enabled by it, the execution of the structure is in execution class 3.

The use of tolerance class 2 in execution class 3 is voluntary. However, partial factors may only be reduced in execution class 3 and only when tolerance class 2 is specified.

The maximum allowable concrete strength class is C20/25 when designing the load-bearing capacity of concrete structures in execution class 1.

2.4 Durability and design working life

In order to achieve the design working life, the exposure classes are defined according to the environmental conditions. The exposure class is used to determine the requirements, such as the steel grade to be used, the concrete cover and the requirements concerning the concrete and the execution of the work:

- the exposure classes are presented in standard SFS-EN 206
- the concrete cover and the design of the structure are presented in standard SFS-EN 1992-1-1 and its national annex
- instructions concerning the durability of concrete are presented in standard SFS-EN 206 and its supplementary standard SFS 7022

- instructions concerning the execution are presented in chapter 3 as well as in standard SFS-EN 13670 and its supplementary standard SFS 5975
- requirements concerning the manufacture of precast concrete elements are presented in standard SFS-EN 13369 and the harmonised product standards for prefabricated elements.

Steel parts and other metal parts whose concrete cover does not meet the requirements or which are otherwise susceptible to corrosion should be reliably protected against corrosion. In exposure classes XC3, XC4, XS2, XS3, XD2 and XD3, such parts should be manufactured from corrosion-resistant material. However, in exposure classes XC3, XC4, XS2 and XD2, the steel parts may be manufactured of corrosion-protected regular steel if their protection can be maintained. The outer chord lattice girder in sandwich elements should be made from the same material as the diagonals if the concrete cover requirements for the chord are not met.

3. Execution

3.1 Execution planning

The work plans for the execution of concrete structures are drawn up on the basis of the execution documents in adherence with standards SFS-EN 13670 and SFS 5975.

Usually, the work plans for the execution of concrete structures present, at a minimum, the following to the extent applicable to the design task:

- the required execution drawings
- the work phase plans pursuant to standard SFS-EN 13670, such as the concreting plan
- the quality documents according to standards SFS-EN 13670 and SFS 5975.

A separate concreting plan is drawn up for the execution of structures in execution classes 2 and 3.

An erection plan is drawn up for the erection of precast concrete elements.

3.2 Construction products

The characteristics of the construction and constituent materials used in concrete structures are demonstrated by means of the CE mark if they are covered by the scope of the harmonised product standard or if the manufacturer has acquired the European Technical Approval/Assessment for its product. Otherwise, they are demonstrated according to the Act on the Type Approval of Certain Construction Products (954/2012).

The characteristics of the following products are central in terms of the reliability of the concrete structures:

- concrete
- special mortars and concretes
- reinforcing steels and mesh
- prestressing tendons
- reinforcement elements
- prestressing systems
- precast concrete products
- load transferring metal components and lifting loops/inserts
- mechanical couplers for reinforcement bars
- special anchors for reinforcement bars.

Instructions regarding the specification, manufacturing and conformity of concrete are presented in standard SFS-EN 206 and its supplementary standard SFS 7022.

Special mortars and concretes cover dry ready-mix mortars and concretes that are used for load-bearing structures or for structures where weather resistance is required . Jointing and repair mortars with weather resistance requirements are also considered special mortars and concretes; the same applies to structural jointing mortar, unless it is a ready-mix concrete pursuant to standard SFS-EN 206.

The essential technical requirements for concrete reinforcing steel is laid down in the Ministry of Environment Decree 126/2016 on the essential technical requirements for weldable concrete reinforcing steel and mesh reinforcements. Instructions regarding the specification, characteristics and conformity of prestressing steel tendons are laid down in standards SFS 1265-1 and SFS 1265-3.

Standard SFS-EN 13369 presents instructions related to the manufacturing of precast concrete elements.

The effects of the special concreting methods on the development of the concrete properties should be determined in advance by means of structural tests or otherwise assessed to a necessary level of precision and taken into account in the batching of the concrete.

4. Execution supervision and the conformity of structures

4.1 Execution supervision

The inspections related to the supervision of the execution of concrete structures are carried out within the scope required by the execution documents in adherence with standards SFS-EN 13670 and SFS 5975.

During the execution of the structures, the responsible work supervisor or a separately appointed specialty site manager will supervise that the plans and instructions concerning the manufacture of concrete structures and the installation of the concrete elements are followed and that the appropriate documents are prepared for the work.

A protocol concerning the inspection of reinforcement steels in cast-in-situ structures should be drawn up within the scope defined by the execution documents and the inspection document.

If it is observed during the execution that a structure or structural detail does not meet the requirements laid down in the execution documents, the occurrence locations and causes of the deviations are analysed. This is done to determine whether the deviation can be approved without a repair. If necessary, calculations are used to demonstrate that the confidence interval required by standards SFS-EN 1992 and their national annexes is achieved. If it cannot be demonstrated that the deviation is acceptable without a repair, the repair will be carried out to the necessary extent.

If the compression strength of the concrete needs to be determined from the structure, the determination is carried out according to standards SFS-EN 13791:2007 and SFS 7022. The deviation and corrective action will be recorded in the quality control documents. In case of non-conformity, the results of the tests made in order to determine the strength, durability and other properties of the concrete are reported to the building control authority if the non-conformity affects the meeting of essential technical requirements.

The quality control documents are compiled into a single entity.

4.2 Conformity of structures

When applying these instructions, the conformity appraisal for structures is based on the concrete structures being designed appropriately pursuant to standards SFS-EN 1992 and their national annexes, and on the concrete structures being executed and inspected pursuant to the execution documents.

5. References

If the version of a reference has not been specified, the latest edition of the reference (with amendments) is applied.

| | |
|-------------------|---|
| SFS-EN 206 | Concrete. Specification, performance, production and conformity |
| SFS-EN 1990 | Eurocode. Basis of structural design |
| SFS-EN 1992-1-1 | Eurocode 2: Design of concrete structures. Part 1-1: General rules and rules for buildings |
| SFS-EN 1992-1-2 | Eurocode 2: Design of concrete structures. Part 1-2: General rules. Structural fire design |
| SFS-EN 1992-3 | Eurocode 2: Design of concrete structures. Part 3: Liquid retaining and containment structures |
| SFS-EN 12504-1 | Testing concrete in structures. Part 1: Cored specimens. Taking, examining and testing in compression |
| SFS-EN 13369 | Common rules for precast concrete products |
| SFS-EN 13670 | Execution of concrete structures |
| SFS-EN 13791:2007 | Assessment of in-situ compressive strength in structures and precast concrete components |
| SFS-EN 1265-1 | Prestressing steels. Part 1: General requirements |
| SFS-EN 1265-3 | Prestressing steels. Part 3: Strand |
| SFS 5975 | Execution of concrete structures. Use of standard SFS-EN 13670 in Finland (in Finnish) |
| SFS 7022 | Concrete. Use of standard SFS-EN 206 in Finland (in Finnish) |

6. National annexes to Eurocodes

SFS-EN 1992

National Annex to standard SFS-EN 1992-1-1 Part 1-1: General rules and rules for buildings

As regards standard SFS-EN 1992-1-1, the recommended values set forth in standard SFS-EN 1992-1-1 and all the annexes to standard SFS-EN 1992-1-1 are followed unless otherwise stated in this National Annex.

The Non-Contradictory Complementary Information (NCCI) is presented in italics.

National choice is permitted in the following clauses of standard SFS-EN 1992-1-1:

- 2.3.3(3)
- 2.4.2.1(1)
- 2.4.2.2(1)
- 2.4.2.2(2)
- 2.4.2.2(3)
- 2.4.2.3(1)
- 2.4.2.4(1)
- 2.4.2.4(2)
- 2.4.2.5(2)
- 3.1.2(2)P
- 3.1.2(4)
- 3.1.6(1)P
- 3.1.6(2)P
- 3.2.2(3)P
- 3.2.7(2), Note 1
- 3.3.4(5)
- 3.3.6(7)
- 4.4.1.2(3)
- 4.4.1.2(5)
- 4.4.1.2(6)
- 4.4.1.2(7)
- 4.4.1.2(8)
- 4.4.1.2(13)
- 4.4.1.3(1)P
- 4.4.1.3(3)
- 4.4.1.3(4)
- 5.1.3(1)P

- 5.2(5)
- 5.5(4)
- 5.6.3(4)
- 5.8.3.1(1)
- 5.8.3.3(1)
- 5.8.3.3(2), Note 1
- 5.8.5(1), Note 1
- 5.8.6(3)
- 5.10.1(6)
- 5.10.2.1(1)P
- 5.10.2.1(2)
- 5.10.2.2(4)
- 5.10.2.2(5)
- 5.10.3(2)
- 5.10.8(2)
- 5.10.8(3)
- 5.10.9(1)P
- 6.2.2(1)
- 6.2.2(6)
- 6.2.3(2)
- 6.2.3(3), Note 1
- 6.2.4(4)
- 6.2.4(6)
- 6.4.3(6)
- 6.4.4(1)
- 6.4.5(1)
- 6.4.5(3)
- 6.4.5(4)
- 6.5.2(2)
- 6.5.4(4)
- 6.5.4(6)
- 6.8.4(1), Note 2
- 6.8.4(5)
- 6.8.6(1)
- 6.8.6(3)
- 6.8.7(1)
- 7.2(2)
- 7.2(3)
- 7.2(5)
- 7.3.1(5)
- 7.3.2(4)
- 7.3.4(3)

- 7.4.2(2)
- 8.2(2)
- 8.3(2)
- 8.6(2)
- 8.8(1)
- 9.2.1.1(1), Note 2
- 9.2.1.1(3)
- 9.2.1.2(1), Note 1
- 9.2.1.4(1)
- 9.2.2(4)
- 9.2.2(5)
- 9.2.2(6)
- 9.2.2(7)
- 9.2.2(8)
- 9.3.1.1(3)
- 9.5.2(1)
- 9.5.2(2)
- 9.5.2(3)
- 9.5.3(3)
- 9.6.2(1), Note 1
- 9.6.2(1), Note 2
- 9.6.3(1)
- 9.7(1)
- 9.8.1(3)
- 9.8.2.1(1)
- 9.8.3(1)
- 9.8.3(2)
- 9.8.4(1)
- 9.8.5(3)
- 9.10.2.2(2)
- 9.10.2.3(3)
- 9.10.2.3(4)
- 9.10.2.4(2)
- 11.3.5(1)P
- 11.3.5(2)P
- 11.3.7(1)
- 11.6.1(1)
- 11.6.2(1)
- 11.6.4.1(1)
- 12.3.1(1)
- 12.6.3(2)
- A.2.1(1)

- A.2.1(2)
- A.2.2(1)
- A.2.2(2)
- A.2.3(1)
- C.1(1)
- C.1(3), Note 1
- C.1(3), Note 2
- E.1(2)
- J.1(2)
- J.2.2(2)
- J.3(2)
- J.3(3).

A national choice has been made in the clauses marked •.

Deformations of concrete

2.3.3(3)

The distance between construction joints d_{joint} is designed for each project. The foundation type should be taken into account in the design.

Partial factors for prestress

2.4.2.2(1)

A value of 1 is adopted for the partial factor $\gamma_{p,\text{fav}}$ of the design value of prestress under persistent and transient design situations. The value may also be used for the fatigue assessments.

If different partial factors are required for favourable and unfavourable effects in the analysis of the entire structure, a value of 0.9 or 1.1 will be adopted for the partial factor of the design value of prestress.

Partial safety factors for materials

2.4.2.4(1)

The partial factors for materials γ_c and γ_s used in ultimate limit states for persistent, transient and accidental design situations are presented in Table 1. The presented values are not valid for fire design situation, in respect of which reference is made to standard SFS-EN 1992-1-2.

Table 1. Partial factors for materials for ultimate limit states

| Design situation | Partial factor for concrete, γ_c | Partial factor for reinforcing steel, γ_s | Partial factor for prestressing steel, γ_s |
|--|---|--|---|
| Persistent and transient design situation | 1.5 | 1.15 | 1.15 |
| Reduced partial factors may be adopted under persistent and transient design situations, if the following are in use: <ul style="list-style-type: none"> - execution class 3 and tolerance class 2 pursuant to SFS-EN 13670, and the quality control for concrete manufacturing is verified by third party - for precast concrete products, the reduced deviations pursuant to Table A.1 of SFS-EN 1992-1-1, and the quality control for concrete manufacturing is verified by third party | 1.35 | 1.10 | 1.10 |
| Accidental situation | 1.0 | 1.0 | 1.0 |

For fatigue analyses, the values adopted for partial factors $\gamma_{c,fat}$ and $\gamma_{s,fat}$ are the unreduced partial factors for persistent design situations pursuant to Table 1.

Strength

3.1.2(4)

A correction factor value of $k_t = 1.0$ is used when determining the strength of concrete that is over 28 days old.

Design compressive and tensile strengths

3.1.6(1)P

A value of 0.85 is adopted for the factor α_{cc} .

Reinforcing steel

3.2

The design rules in standard SFS-EN 1992-1-1 do not apply to coated reinforcing steels, stainless reinforcing steels or reinforcing steels with a yield strength higher than 600 MPa.

Coated reinforcing steels, stainless reinforcing steels or reinforcing steels with a yield strength higher than 600 MPa may be used in Eurocode design if the validity of the application rules can be demonstrated.

The stress/strain relationship of stainless reinforcing steel differs from the stress/strain relationship for carbon steel demonstrated in section 3.2.7. Furthermore, the stress/strain relationship is not similar between different grades of stainless reinforcing steel.

In the design of reinforcing steel with a yield strength of over 600 MPa the higher strain caused by the higher tension on the steel should be taken into account.

Design assumptions

3.2.7(2), Note 1

The upper limit of strain ϵ_{ud} is 1% when inclined top branch is applied.

Design assumptions

3.3.6(7)

The upper limit of elongation ϵ_{ud} is 2% when inclined top branch is applied.

Minimum cover, c_{min}

4.4.1.2(5)

The minimum value for concrete cover $c_{min,dur}$ is given in Table 2.

Table 2. Minimum cover value requirements (design working life of 50 or 100 years)

| Requirement for value of minimum cover $c_{min,dur}$ (mm) under different environmental conditions | | | | | | | |
|--|--|-----|-----|-------------|-------------|-------------|-------------|
| Criterion | Exposure class pursuant to Table 4.1 of standard SFS-EN 1992-1-1 | | | | | | |
| | X0 | XC1 | XC2 | XC3, XC4 | XD1, XS1 | XD2, XS2 | XD3, XS3 |
| Reinforcing steel | 10 | 10 | 20 | 25 | 30 | 35 | 40 |
| Prestressing steel | 10 | 20 | 30 | 35 | 40 | 45 | 50 |
| Design working life of 100 years | +0 | +0 | +5 | +5 | +5 | +5 | +5 |

Note 1. The requirements for reinforcing steel are applied to bonded tendons with a maximum long-term stress of 400 N/mm² at the serviceability limit state.

Note 2. The minimum value of concrete cover may be reduced by 5 mm if the cylinder strength of the concrete is at least 10 MPa higher than the minimum cylinder strength required for durability.

Note 3. The concrete cover minimum value requirements also apply to prestressing steel anchors and metal parts installed in the cast concrete, unless they have been protected against corrosion in a manner commensurate with the exposure class.

Note 4. If the structure has a design working life of 100 years, the durability of the concrete must meet the design working life requirement of 100 years in other respects, as well.

When the presented minimum values for concrete cover are deviated from, or the service life goal is over 100 years, the minimum values $c_{min,dur}$ should be defined by means of a design working life calculation in which external weather exposure, the impacts of the indoor climate, exposure caused by use, the composition of the concrete, the corrosion properties of the reinforcing steel, the details and coating of the structure, curing and inspection and maintenance activities are taken into account.

Allowance in design for tolerance

4.4.1.3(1)P

The allowed deviation for concrete cover Δc_{dev} is normally 10 mm.

4.4.1.3(3)

An allowed deviation less than 10 mm may be adopted for the design of precast concrete products if this is justified according to the factory production control. However, a deviation less than $\Delta c_{dev} = 5$ mm should not be used.

4.4.1.3(4)

A minimum value of at least $k_1 = c_{min} + 10$ mm is adopted for the nominal concrete cover when concrete is cast against prepared ground (including blinding), and a value of $k_2 =$

$c_{\min} + (20...40)$ mm is adopted based on the designer's judgment for concrete that is cast directly against soil.

Linear elastic analysis with limited redistribution

5.5(4)

The redistribution of moments cannot be utilised when using steels in ductility class A; in this case, factor $k_6 = 1$. The recommended values are used for factors k_1, k_2, k_3, k_4 and k_5 .

Methods of analysis

5.8.5(1), Note 1

The designer selects the used method (a) or (b) case-specifically.

General

5.10.1(6)

In order to avoid brittle failure caused by the fracture of tendons, one of the following conditions must be met:

Method A:

Provide minimum reinforcement in accordance with 9.2.1.

Method D:

Provide satisfactory evidence concerning the reliability of the tendons. This condition is considered to be met when the cross-section bending capacity M_{Rd} is 1.5 times the ultimate limit state design moment M_{Ed} .

When using unbonded tendons, the condition is considered to be met when the structure is designed in a manner that a sufficient confidence level is achieved even if one tendon in the same cross-section is damaged. In flat slab construction, the cross-section is considered to include the tendons that are located on both sides of the tendon at a distance of $L/3$, where L is the span of the slab.

Method E:

Ensure that if failure were to occur due to either an increase of load or a reduction of prestress, cracking would occur under the characteristic combination of loads before the ultimate resistance is exceeded. The analysis should take into account of moment redistribution due to cracking.

Limitation of concrete stress

5.10.2.2(4)

A value of 20% is used for k_4 and a value of 0% is used for k_5 .

5.10.2.2(5)

A value of 0.65 is adopted for the factor k_6 .

Effects of prestressing at ultimate limit state

5.10.8(2)

A value of 50 MPa is adopted for the stress increase $\Delta\sigma_{p,ULS}$.

5.10.8(3)

A value of 1.0 is always used for the upper and lower limits of the partial factors, $\gamma_{\Delta P,sup}$ and $\gamma_{\Delta P,inf}$.

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

5.10.9(1)P

A single characteristic value may be adopted for the serviceability limit state and fatigue calculations, in which case the value for factors r_{sup} and r_{inf} is 1.

Punching shear resistance of slabs and column bases without shear reinforcement

6.4.4(1)

When defining the punching shear resistance for a structure without shear reinforcement and the upper limit for punching shear resistance, the value adopted for $C_{Rd,c}$ is:

$$C_{Rd,c} = \frac{0,3 \left(\frac{D}{d} + 1,5 \right)}{\gamma^c \left(\frac{D}{d} + 4 \right)} \quad (1.1)$$

where

D is the diameter of a round column or, for a rectangular column, $D = \sqrt{c_1 c_2}$, where c_1 and c_2 are the side lengths of the column

d is the mean effective thickness of the slab.

When defining the punching shear resistance for a structure with shear reinforcement (standard SFS-EN 1992-1-1 expression 6.52), the design value for punching shear resistance is calculated (standard SFS-EN 1992-1-1 expression 6.47) by adopting the following value of $C_{Rd,c}$:

$$C_{Rd,c} = \frac{0,3 \left(\frac{D}{d} + 1,5\right)}{4,5 \cdot \gamma_c \left(\frac{D}{d} + 4\right)} \quad (1.2)$$

A value of 0 is adopted for v_{min} and a value of 0.1 is adopted for k_1 in all cases.

Punching shear resistance of slabs and column bases with shear reinforcement

6.4.5(1)

A value of 1.6 is adopted for the factor k_{max} .

The expression (6.52) in standard SFS-EN 1992-1-1 is derived for radial reinforcement. In the expression the total surface area of the reinforcement within the area delimited by the circumference u_1 should be used, in which case the term $1.5 (d/s_r)A_{sw}$ is replaced by the total area of reinforcement. When calculating the area of the reinforcement, it should be considered that the reinforcement is anchored sufficiently to both sides of the punching crack. Usually, reinforcement that is at a maximum distance of $1.5 d$ from the column is considered.

Verification procedure for reinforcing and prestressing steel

6.8.4(1), Note 2

The parameter values for the S-N curves for reinforcing steel and prestressing steel presented in Tables 6.3N and 6.4N of standard SFS-EN 1992-1-1 may be adopted if the fatigue strength of the reinforcing steel has been assessed according to the Ministry of Environment Decree 126/2016 on the essential technical requirements for weldable concrete reinforcing steel and mesh reinforcements, the fatigue strength of the prestressing steel has been defined pursuant to the requirements of standards SFS 1265-1 and SFS 1265-3 and the fatigue strength class of the prestressing steel is F1 or F2.

Verification of concrete under compression or shear

6.8.7(1)

The recommended value of 10^6 is adopted for the number of stress cycles N . A value of 1.0 is adopted for the factor k_1 when the number of stress cycles is $N = 10^6$.

Stress limitation

7.2(5)

The values of factors are $k_3 = 0.6$, $k_4 = 0.8$ and $k_5 = 0.6$.

General considerations

7.3.1(5)

The limiting calculated crack width w_{\max} is given in Table 3.

Table 3. Limit values for crack width w_{\max} (mm), when the maximum design working life of the structure is 100 years.

| Exposure class | Reinforced members and prestressed members with unbonded tendons | Prestressed members with bonded tendons | |
|-------------------------|--|---|----------------------------------|
| | Quasi-permanent load combination | Frequent load combination | Quasi-permanent load combination |
| X0, XC1 | 0.40 | 0.20 | no requirements |
| XC2, XC3, XC4, XD1, XS1 | 0.30 | 0.20 | Decompression |
| XD2, XD3, XS2, XS3 | 0.20 | Decompression | no requirements |

Note 1. For exposure classes X0 and XC1 in reinforced members or members with unbonded tendons, crack width has no influence on durability and the limit is set to guarantee acceptable appearance. In the absence of appearance conditions, this limit may be relaxed.

Note 2. For prestressed members with bonded tendons, a tensile stress $f_{ctk,0.05}$ is allowed under quasi-permanent if the structure is loaded by an imposed load pursuant to standard SFS-EN 1991-1-1 with a quasi-permanent combination factor higher than 0.5.

Note 3. If the concrete cover is higher than the minimum cover required for durability $c_{\min,dur}$, the limit values for crack width may be increased by a factor of $(c_{true}-c_{dev})/c_{\min,dur} \leq 1.4$, where c_{true} is the concrete cover pursuant to the plan. If the concrete cover thickness is 50 mm, a value of 50 mm may be adopted for concrete cover c in the crack width calculation.

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

Cases where calculations may be omitted

7.4.2(2)

Values for factor K are given in Table 4. Values obtained using expression (7.16) for common cases (C30/37, $\sigma_s = 310$ MPa, with different structural systems and reinforcement ratios $\rho = 0.5\%$ and $\rho = 1.5\%$) are also given.

Table 4. Basic ratios of span/effective depth for reinforced concrete members without axial compression

| Structural system | K | Concrete highly stressed, $\rho = 1.5\%$ | Concrete lightly stressed, $\rho = 0.5\%$ |
|---|-----|---|--|
| Simply supported beam, one- or two-way 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 |
| <p>Note 1. The values given have been chosen to be generally conservative, and calculation may frequently show that thinner members are possible.</p> <p>Note 2. For two-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.</p> <p>Note 3. The limits given for flat slabs correspond to a less severe limitation than a mid-span deflection of $\text{span}/250$ relative to the columns. Experience has shown this to be satisfactory.</p> | | | |

Spacing of bars

8.2(2)

The values of factors are $k_1 = 1$ and $k_2 = 3$ mm.

Permissible mandrel diameters for bent bars

8.3(2)

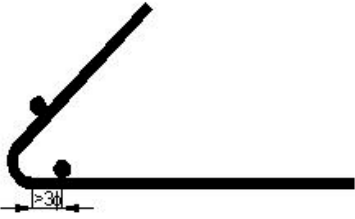
In order to avoid damage to the reinforcement, the diameter to which the bar is bent (mandrel diameter) should not be less than $\phi_{m,\min}$ in Table 5.

Table 5. Minimum mandrel diameter to avoid damage to reinforcement

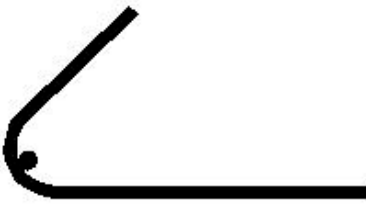
a) Bars and wires

| Bar diameter | Minimum mandrel diameter for bends, hooks and loops (see Figure 8.1) |
|--|--|
| $\phi \leq 16 \text{ mm}$ | 4.5ϕ |
| $\phi > 16 \text{ mm}$ | 9ϕ |
| Note: Minimum mandrel diameter values that are at least 2 times the values used in the bending test for the relevant steel grade may be adopted alternatively. | |

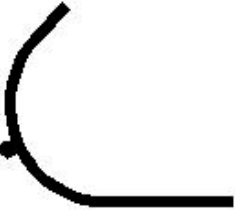
b1) for welded reinforcement and mesh bent after welding, the bending point being outside the area influenced by welding heat

| | |
|---|--|
|  | <p>When the bending point is outside the area influenced by the welding heat (HAZ), the mandrel diameter is as in a) above.</p> <p>The HAZ area can be taken as 3ϕ to both sides from the welding point.</p> |
|---|--|

b2) for welded reinforcement and mesh bent after welding, the bending point being in the HAZ area and the weld on the inside of the bending

| | |
|---|---|
|  | <p>$\phi_{m,min} = 2.0$ times the values given in a) above.</p> <p>$\phi_{m,min} = 1.5$ times the values given in a) above for reinforcement according to SFS 1202 or CEN/TR 15481.</p> |
|---|---|

b3) for welded reinforcement and mesh bent after welding, the bending point being in the HAZ area and the weld on the outside of the bending

| | |
|---|---|
|  | <p>$\phi_{m,min} = 5.0$ times the values given in a) above.</p> <p>$\phi_{m,min} = 3.0$ times the values given in a) above for reinforcement according to SFS 1202 or CEN/TR 15481.</p> |
|---|---|

c) load bearing welding

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

Note: Non-load bearing welds are allowed in bended areas for reinforcement according to SFS 1202 or CEN/TR 15481. The mandrel diameters shall be as in a) above.

Ultimate bond stress

8.4.2

Smooth bars may be used as anchorage bars for steel components or link reinforcement for piles.

The anchorage strength of smooth bar is calculated while applying clause 8.4 of standard SFS-EN 1992-1-1:

- *instead of the expression (8.2) in standard SFS-EN 1992-1-1, the value of $f_{bd} = \eta_1 \eta_2 f_{ctd}$ is adopted for the ultimate bond stress of smooth bars*
- *the product ($\alpha_2 \alpha_3 \alpha_5$) has a value of 1 (see expression (8.5) of standard SFS-EN 1992-1-1)*

This instruction may be applied for smooth reinforcing steel and structural steel with a maximum yield strength f_{yk} of 400 N/mm². Hooks should be used for anchoring smooth bar.

Lap length

8.7.3(1)

When using a value of 2.0 for the factor α_6 , it may be considered that the requirement in clause 8.7.2(3) for the longitudinal distance of the lap slice is met and the bars may be lapped at the same cross-section pursuant to clause 8.7.2(4).

When calculating the lap length of bars in compression a value of 1 may be adopted for factor α_6 .

Minimum and maximum reinforcement areas

9.2.1.1(3)

The cross-sectional area $A_{s,max}$ of tension or compression reinforcement is not limited.

Other detailing arrangements

9.2.1.2(1), Note 1

A value of 0.15 is adopted for the factor β_1 , unless the degree of restraint is analysed more accurately.

Shear reinforcement

9.2.2(4)

When the shear reinforcement is reliably anchored, the value of the minimum proportion of links, β_3 , is 0.

General

9.3.1.1(3)

The maximum value for bar spacing $s_{\max, \text{slabs}}$ is:

- for the principal reinforcement, $3h \leq 400$ mm, where h is the total depth of the slab;
- for the secondary reinforcement, $4h \leq 600$ mm.

In areas with concentrated loads or areas of maximum moment the provisions are respectively:

- for the principal reinforcement, $2h \leq 250$ mm
- for the secondary reinforcement, $3h \leq 400$ mm.

Punching shear reinforcement

9.4.3(2)

The expression (9.11) in standard SFS-EN 1992-1-1 is derived for radial reinforcement. The minimum amount of steel for the entire shear reinforcement may be alternatively calculated by replacing the term $s_r s_t$ with the area delimited by the circumference u_1 in which and the cross-section area of the column is subtracted.

Longitudinal reinforcement

9.5.2(3)

The value of the maximum area of longitudinal reinforcement at laps is $A_{s, \max} = 0.12A_c$ and outside of laps it is $A_{s, \max} = 0.06A_c$.

Transverse reinforcement

9.5.3(3)

The spacing of the transverse reinforcement along the column should not exceed $s_{cl, \text{tmax}}$.

The lowest of the following three values applies:

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

Vertical reinforcement

9.6.2(1), Note 2

The maximum value of the vertical reinforcement is $A_{s,vmax} = 0.06A_c$.

Deep beams

9.7(1)

The minimum area is $A_{s,dbmin} = 0,0005 A_c$, but at least $150 \text{ mm}^2/\text{m}$ on each face and in both directions.

Column footing on rock

9.8.4(1)

Adequate transverse reinforcement should be provided to resist the splitting forces in the footing, when the ground pressure in the ultimate limit state exceeds $q_2 = 3 \text{ MPa}$. A minimum reinforcement bar diameter of $\varphi_{min} = 8 \text{ mm}$ should be provided.

Internal ties

9.10.2.3(4)

The parameter values used are $q_3 = 20 \text{ kN/m}$ and $Q_4 = 70 \text{ kN}$.

Members not requiring design without shear reinforcement

11.6.1(1)

The recommended values are used in the shear analysis

When defining the punching resistance for a structure without shear reinforcement or the upper limit for punching resistance, the value adopted for $C_{IRd,c}$ is:

$$C_{IRd,c} = \frac{0,3 \left(\frac{D}{d} + 1,5\right)}{\gamma^c \left(\frac{D}{d} + 4\right)} \quad (1.3)$$

where

D is the diameter of a round column or, for a rectangular column, $D = \sqrt{c_1 c_2}$, where c_1 and c_2 are the side lengths of the column

d is the mean effective thickness of the slab

When defining the punching resistance for a structure with shear reinforcement (expression 11.6.52), the design value for punching resistance is calculated (expression 11.6.47) by using the following value for $C_{IRd,c}$:

$$C_{IRd,c} = \frac{0,3 \left(\frac{D}{d} + 1,5\right)}{4,5 \cdot \gamma_c \left(\frac{D}{d} + 4\right)} \quad (1.4)$$

A value of 0 is adopted for $v_{l,min}$ in all punching analyses.

Concrete: additional design assumptions

12.3.1(1)

The values $\alpha_{cc,pl} = 0.7$ and $\alpha_{ct,pl} = 0.6$ are adopted for the strength factors of plain concrete.

Simplified design method for walls and columns design

12.6.5.2(1)

Creep has a substantial effect on the compression resistance of plain walls and columns. In expression (12.11), the effect of creep on the factor Φ is taken into account by means of eccentricity e_i ; however, no instruction is provided for determining it. Therefore, expression (12.11) should not be used. If the calculation is not done by general method, the factor Φ that accounts for the effects of deformation, may alternatively be defined by with expression (1.5). Expression (1.5) is based one curve fitting of the compression resistance determined with general method:

$$\Phi = \frac{1 - 2,4(e_{tot} / h_w)}{1 + 0,007(l_0 / h_w)^2(0,1 + e_{tot} / h_w)(0,8 + \varphi_{ef})(f_{ck} / 30)^{0,7}} \quad (1.5)$$

The terms e_{tot} , l_0 and h_w in expression (1.5) are defined in clause 12.6.5.2 of standard SFS-EN 1992-1-1. The effective creep ratio φ_{ef} is defined in clause 5.8.4 of standard SFS-EN 1992-1-1.

Reduction based on quality control and reduced deviations

A.2.1(1)

For cast-in-situ structures, the partial factor for reinforcement may be reduced to $\gamma_{S,red1} = 1.1$ if execution class 3 and tolerance class 2 of standard SFS-EN 13670 are used.

For reinforcement of precast concrete product, the partial factor may be reduced to $\gamma_{S,red1} = 1.1$ if the reduced deviations pursuant to Table A.1 are used.

A.2.1(2)

For cast-in-situ structures, the partial factor for concrete may be reduced to $\gamma_{C,red1} = 1.35$ if execution class 3 and tolerance class 2 of standard SFS-EN 13670 are used and the quality control of the concrete manufacturing is verified. In this case, it is considered that the variation coefficient for the mean deviation of compression strength meets the set requirement.

For precast concrete products, the partial factor for concrete may be reduced to $\gamma_{C,red1} = 1.35$ if the reduced deviations pursuant to Table A.1 are used and the quality control of the concrete manufacturing is verified by third party. In this case, it is considered that the variation coefficient for the mean deviation of compression strength meets the set requirement.

Reduction based on using reduced or measured geometrical data in design

A.2.2(1)

Partial factors may be reduced to $\gamma_{S,red2} = 1.05$ and $\gamma_{C,red2} = 1.45$.

A.2.2(2)

The partial factor for concrete may be reduced to $\gamma_{C,red3} = 1.35$.

Reduction based on assessment of concrete strength in finished structure

A.2.3(1)

The partial factor for concrete γ_C may be reduced by the conversion factor $\eta = 0.85$. If the factor η has already been taken into account when evaluating the strength in a finished structure (EN 13791: $\eta = 0.85$) the partial factor γ_C may not be reduced any further by the conversion factor η .

However, a minimum value of $\gamma_{C,red4} = 1.2$ should be used for the partial factor of the concrete.

General

C.1(1)

The fatigue strength of reinforcement bars is determined with methods described in the Ministry of Environment Decree 126/2016 on the essential technical requirements for weldable concrete reinforcing steel and mesh reinforcements. The upper limit of the stress range, and therefore also the factor β , depends on the method used. The method to be applied is selected for each project.

The minimum relative rib area should meet the values in Table C.2 N of standard SFS-EN 1992-1-1. Expressions (C.1N) and (C.2N) are not used.

C.1(3), Notes 1 and 2

Table 6. Absolute limits and parameters for test results

| Characteristic | Nominal value minus individual test result σ_2 | Lowest allowed individual result $X_{i,min}$ | Lowest allowed mean value M_{min} |
|-------------------------|---|--|-------------------------------------|
| Yield strength f_{yk} | $C_v - X_i$ | $0.97 C_v$ | $C_v + 2.27 \sigma_2$ |
| $k - 1$ ¹⁾ | $C_v - X_i$ | $0.92 C_v$ | $C_v + 1.78 \sigma_2$ |
| ϵ_{uk} | $C_v - X_i$ | $0.92 C_v$ | $C_v + 1.78 \sigma_2$ |

¹⁾ The requirement is applied to the portion of the coefficient that exceeds one.

Table 6 presents the acceptance criteria for strength and ductility of a single manufacturing batch based on three test results. X_i is an individual result that is below the nominal value and C_v is the nominal value of steel.

Annex E
Indicative strength classes for durability

Annex E is not used

Annex J
Detailing rules for particular situations

Annex J may be used with the exception of clauses J.1 and J.2.

National Annex to standard SFS-EN 1992-1-2 Part 1-2: General rules. Structural fire design

As regards standard SFS-EN 1992-1-2, the recommended values set forth in standard SFS-EN 1992-1-2 and all the annexes to standard SFS-EN 1992-1-2 are followed unless otherwise stated in this National Annex.

The Non-Contradictory Complementary Information (NCCI) is presented in italics.

National choice is permitted in the following clauses of standard SFS-EN 1992-1-2:

- 2.1.3(2)
- 2.3(2)P
- 3.2.3(5)
- 3.2.4(2)
- 3.3.3(1), Note 1
- 4.1(1)P
- 4.5.1(2)
- 5.2(3)
- 5.3.2(2), Note 1
- 5.6.1(1)
- 5.7.3(2)
- 6.1(5)
- 6.2(2)
- 6.3(1), Note 1
- 6.4.2.1(3)
- 6.4.2.2(2)

A national choice has been made in the clauses marked ●.

Parametric fire exposure

2.1.3(2)

No values are given for the average temperature rise $\Delta\theta_1$ and for the maximum temperature rise $\Delta\theta_2$ during the cooling phase of fire.

The requirement for the separation function is only based on a standard fire and on temperature limits set by it.

The fire safety requirement is also deemed to be satisfied if the building is designed and executed based on design fire scenarios which cover the situations likely to occur in the said building. The satisfaction of the requirement is attested case-by-case, taking into consideration the properties and use of the building.

Member analysis

2.4.2(3)

When using the partial factors from standard SFS-EN 1990 and the Ministry of Environment Decree concerning it, Figure 2.1 in standard SFS-EN 1992-1-2 will change as presented in Figure 1.

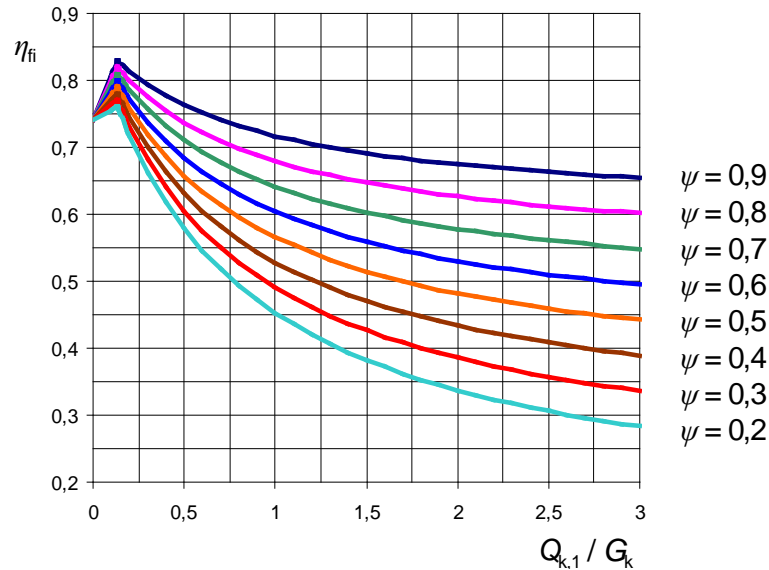


Figure 1. The variation of the reduction factor η_{fi} as a function of the load ratio of the nominal values of leading variable action and permanent action $Q_{k,1} / G_k$ according to the load combination rules presented in the Ministry of Environment Decree concerning standard SFS-EN 1990.

2.4.2(3), Note 2

Approximate values are not used.

Reinforcing steel

3.2.3(5)

Class N (Table 3.2a) may be used for all reinforcing steel conforming to the requirements in the Ministry of Environment Decree 126/2016 on the essential technical requirements for weldable concrete reinforcing steel and mesh reinforcements.

Class X (Table 3.2b) may be used if the characteristics of the steel at elevated temperatures have been demonstrated by means of testing pursuant to standard SFS 1300.

Prestressing steel

3.2.4(2)

Both classes A or B may be used.

Thermal conductivity

3.3.3(1), Note 1

Lower limit is used for thermal conductivity.

Explosive spalling

4.5.1(2)

When considering a risk of explosive spalling, the value $k = 2.5\%$ is used as a limit value.

Method A for assessing fire resistance of columns

5.3.2(2), Note 1

The limit value of $e_{\max} = 0.4h$ (and b) is used for eccentricity.

General

5.6.1(1)

Class WC is used for web thickness.

General

6.1(5)

When reducing the strength of high strength concrete at elevated temperatures, a class in accordance with Table 1 is used for all strength classes of concrete.

Table 1. Reduction of strength at elevated temperatures

| Temperature of concrete θ °C | $f_{c,\theta} / f_{ck}$ |
|--|-------------------------|
| | Class FI |
| 20 | 1.00 |
| 50 | 1.00 |
| 150 | 0.75 |
| 300 | 0.75 |
| 800 | 0.15 |
| 900 | 0.08 |
| 1000 | 0.04 |
| 1100 | 0.01 |
| 1200 | 0.00 |

Spalling

6.2(2)

Method A: Not used

Method B: May be used

Types of concrete with the following characteristics have been shown to be acceptable:

- *cement CEM I 42.5 (or 52.5) R*
- *maximum content of silica fume 10% of the weight of cement,*
- *natural aggregate and*
- *after the concrete has attained about 60% of its nominal strength, it is allowed to dry, i.e. no long-term moisture curing may be used.*

Method C: May be used

Method D: May be used

Thermal properties

6.3(1), Note 1

A lower limit value is used for thermal conductivity of high strength concrete in accordance with clause 3.3.3.

Columns and walls

6.4.2.1(3)

For Class FI, factor $k = 1.3$ is used. Class FI is defined in Table 1 above.

Beams and slabs

6.4.2.2(2)

The factors k_m in Table 6.2N are not valid for Class FI (defined in Table 1 above). More accurate methods are used instead, e.g. 400°C isotherm as for columns and walls in clause 6.4.2.1.

6.4.3 Tabulated data

6.4.3(1)

Axis distance of reinforcement may be adjusted by using more accurate methods in paragraph 5.2. When taking into account that thermal conductivity for high strength concrete according to the National Annex is the same as for normal strength concrete, the outcome of the use of more accurate methods is that there is no need to increase the axis distance by factor k .

Annex B

Simplified calculation methods

Annex may be used, but not for parametric fire.

Annex C

Buckling of columns under fire conditions

Annex is not used.

Annex D

Calculation methods concerning shear, torsion and anchorage

Annex is not used unless the results are separately verified.

National Annex to standard SFS-EN 1992-3 Part 3: Liquid retaining and containment structures

As regards standard SFS-EN 1992-3, the recommended values set forth in standard SFS-EN 1992-3 and all the annexes to standard SFS-EN 1992-3 are followed unless otherwise stated in this National Annex.

The Non-Contradictory Complementary Information (NCCI) is presented in italics.

National choice is permitted in the following clauses of standard SFS-EN 1992-3:

- 7.3.1(111)
- 7.3.1(112)
- 7.3.3 (Figures 7.103N and 7.104N)
- 8.10.1.3(103)
- 9.11.1(102)

A national choice has been made in the clauses marked ●.

General considerations

7.3.1(111)

Examples of structures in different Tightness Classes:

Tightness Class 1: small water towers, swimming pools

Tightness Class 2: water towers where aesthetically disturbing leakage is not allowed.

Tightness Class 3: big water towers, reservoirs and tanks containing detrimental substances (as at dumping places).

Minimum area of passive reinforcement and cross-sectional dimensions

9.11.1(102)

The thickness of walls forming the sides of reservoirs should be not less than $t_1 = 120$ mm in class 0 and $t_2 = 200$ mm in classes 1 and 2. Slip-formed walls should always be at least 200 mm thick regardless of class.

Annex K

Effect of temperature on the properties of concrete

Annex K may be used. Chapter K.2 may be used in design situations where the temperature of the construction is permanently between $-25 \dots -40$ °C.