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COMPARISON OF RUSSIAN, FINNISH AND EUROPEAN NORMS FOR REINFORCED CONCRETE STRUCTURES

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ABSTRACT

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Comparison of Russian, Finnish and European norms for reinforced concrete structures

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The main purpose of the thesis was to compare the main requirements for design and construction of reinforced concrete structures given in three norms: Russian SNiP system, Eurocode 2 (EN1992) and Finnish regulations RakMK.

Nowadays the amount of cooperative projects in construction field between Russia and Finland is growing, so the knowledge of differences in requirements concerning designing and implementation of reinforced concrete structures became quite important. Also European norms will be soon applied in Finland so it is also important to reveal differences between requirements used in Finland nowadays and European norms.

The main issues regarding the design of reinforced concrete structures such as classification of concrete and steel, principles of reinforcement and tolerances for erected structures were determined, the requirements for those issues from different norms were found, studied and compared.

The study showed that there are no big differences between requirements for reinforcement detailing, but there are differences in the way of durability design and also different strength classification of concrete is applied.

Tables of comparison which include the main information are represented in Appendices.

Keywords: concrete, reinforcement, strength.

CONTENTS

1 INTRODUCTION

This thesis was done for the Finnish construction company called Bafo Ltd. Bafo was established in 1995 and is based in Helsinki, however, it operates not only in Finland. Bafo offers a wide range of services in construction - from architectural and structural design to implementation and supervision. Bafo has close relationships with Russian customers as it has made quite a large amount of projects in Russia.

The current project is construction of the living and cultural center of Russian embassy in Helsinki. The owner of the facility is Russian Ministry of Foreign Affairs, the main designer and main contractor are Russian companies.

The building has seven floors made of reinforced concrete. During construction process some problems appeared because some regulations about reinforced concrete structures in Russian norms differ from the corresponding regulations in Finnish norms.

For example concrete and steel classification in both countries is different. Requirements for quality of erected structures and some details of reinforcing are different in some cases.

It became quite useful to study these norms and find out more about differences between them.

2 INTRODUCTION TO THE RUSSIAN SYSTEM OF NORMS IN CONSTRUCTION

The Russian System of regulations in construction is based on the Town-Planning Code of the Russian Federation and on the Federal Law on Technical Regulation.

The first one regulates the whole range of issues concerning creation of the settling, town planning, development of their engineering, transport and social infrastructures, reasonable use of environment in order to provide favourable living conditions.
The Code regulates the issues concerning construction within urban territories; in particular, it determines the list of necessary town-planning documentation, permissions for construction.

The second one is a document which establishes mandatory rules and requirements to the objects of technical regulation (products, including buildings, structures and facilities, production processes, operation, storage, transportation, sale and recycling) in order to provide safety to people, property and environment.

Technical Regulation on safety of buildings and structures establishes a list of documents which include mandatory requirements in order to provide safety and good quality to buildings and structures. The main documents used in the design and implementation of buildings are:

- SNiP - Building regulations of Russian Federation
- GOST - National standards
- SP - Code of Practice in building construction

Building regulations of Russian Federation (SNiPs) establish mandatory requirement which should be fulfilled in designing and implementation of buildings and ways how to fulfill them. There is a great amount of SNiPs in

the Russian system, and they concern all types of structures and construction works. All SNiPs are listed in the Federal Law.
SNiPs used for the design and construction of concrete and reinforced concrete structures and mentioned in thesis are the following:

- SNiP 52-01-2003 “Reinforced concrete structures without prestressing”
- SNiP 2.01.07-85 “Loads and actions”
- SNiP 3.03.01-87 “Bearing and envelope structures”
- SNiP 2.03.11-85 “Protection of structures against corrosion”

National standards of Russian Federation (GOSTs) include mandatory requirements to parameters and characteristics of parts of structures, structural members and building materials. For example GOST 26633-91 ”Heavy concrete” contains requirements for parameters of concrete mixture, aggregates and binding materials that should be used and so on.

Code of Practice in building construction (SP) determines things which should be done in order to accomplish mandatory requirements given in SNiPs.
For example SNiP 52-01-2003 “Reinforced concrete structures” contains only general requirements for the design of reinforced concrete structures and it is impossible to design structures using only it. That is why there is SP 52-101-2003 which contains detailed requirements for the design and designers must use it in order to accomplish the requirements given in SNiP 52-01-2003.

SPs used for the design or reinforced concrete structures and mentioned in thesis are:

- SP 52-101-2003 “Concrete and reinforced concrete structures without prestressing”
- SP 20.13330.2011 “Loads and actions”

3 CLASSIFICATION OF LOADS

The thesis sometimes refers to different load combinations. The way loads are classified and combined is different in all studied designing norms. Thus it became necessary to explain briefly the main ideas of load classification in order not to confuse the reader by unknown terms and to give basic information about the topic.

3.2 Classification of loads according to Russian norms

For designing of loads SNiP 2.01.07-85 “ Loads and actions” and its updated version which is called SP 20.1330.2011 are used. They contain all information needed for determination and combining loads imposed on structures.

According to Russian norms loads are divided into categories:

- permanent load (self weight of structure, prestressing)
- temporary loads (all other loads- imposed loads)
- special loads (earthquake, explosion)

Temporary loads are also divided into:

- Long-term load – acting for a long period of time (storage load, people, machinery)
- Short-term – acting for short periods of time (show, wind).

Russian norms give characteristic values of distributed (residential) loads which are usually applied to floors in different types of buildings.
For assessment of a long-term effect of the load SNiP defines reduced values of residential load.
Normal value is assumed in calculations as a short-term load and a reduced value is considered as a long-term load.
Examples are given in the following table:

Table 1. Distributed (residential) loads in Russia

Area	Normal value (short-term load)	Reduced value (long-term load)
Apartments, living areas in hotels, dormitories	1.5 kN/m^2	0.5 kN/m^2
Bathrooms, shower rooms; Offices, schools	2.0 kN/m^2	0.7 kN/m^2
Eating rooms (restaurants, canteens)	3.0 kN/m^2	1.05 kN/m^2

Note: reduced values are obtained by multiplying the normal value by 0.35. /2./

The design value of a load is obtained by multiplying it by safety factor, γ_f .

The value of safety factor depends on the type of the load. Examples of safety factors for different types of loads are given in Table 2.

Table 2. Safety factor for loads

Type of load	Safety factor, γ_f
Self weight: reinforced concrete steel	 1,1 1,05
Snow load	1,4
Wind load	1,4
Distributed (residential) load: Q < 2 kN/m^2 Q ≥ 2 kN/m^2	 1,3 1,2

Load combinations (SNiP 2.01.07-85)

The basic combination of loads used for ultimate limit state design consists of permanent, long-term and short-term temporary loads:

$$C = P_d + \sum_i \psi_{Li} P_{Li} + \sum_i \psi_{Si} P_{Si} \tag{1}$$

where:

P_d - design value of permanent load

P_L - design value of long-term temporary load

P_S - design value of short-term temporary load

ψ_L - combination factor for Long-term loads

ψ_S - combination factor for Short-term loads.

Values of combination factor for long-term loads are:

$$\psi_{L1} = 1.0\,,\ \ \psi_{L2} = \psi_{L2} = ... = 0.95$$

Values of combination factor for short-term loads are:

$$\psi_{S1} = 1.0\,,\ \ \psi_{S2} = 0.9\,,\ \ \psi_{S3} = \psi_{S4} = ... = 0.7$$ /2./

For the design of concrete and reinforced concrete structures especially for serviceability limit state SNiP 52-01-2003 assumes two types of loading:

- “long acting loading”
- “not long acting loading”

The first one is a combination of permanent load and long-term temporary load, in other words it is a combination which acts for a long period of time. In that case long-term effects such as creep of concrete is taken into account.

It is also used for calculating of long-term cracking. This combination corresponds to used in Eurocode “Quasi-permanent combination”.

The second case, “not long acting loading”, is a combination of permanent load, long-term and short-term temporary loads, in other words it is a combination which acts for a short period of time, but has a bigger value than the first one. Actually it is equal to the basic combination of loads, given above (Expression 1). It corresponds to given in Eurocode Characteristic combination or Frequent combination. It is used for Ultimate limit state design and for determination of short-term cracking in SLS verification.

3.1 Classification according to Eurocode

Each design situation is characterized by the presence of several types of actions on the structure. “Action” means a load applied to the structure (direct actions), or a set of imposed deformations or accelerations caused for example, by temperature changes.
Actions are classified as:

- permanent actions (G), the duration of which is continuous and equal to the design working life of the structure, for example self-weight;
- variable actions (Q), actions with discrete and regular occurrence in time (e.g. imposed load of people on building floors, cranes, machinery and also snow and wind);
- accidental actions (A), which can not be easily predicted and have a low duration (e.g. fire, explosion).

There are characteristic values of actions: G_k and Q_k. The design values of loads are obtained by multiplying their characteristic values by the appropriate partial factor, γ.

Load combinations (Eurocode 2)

When more than one live load (variable action) is present the secondary live load may be reduced by combination factor, ψ. There can be three different combinations of loads.

The basic load combination used for ULS verification is presented below. This combination represents the biggest value of the total load that can appear:

$$\gamma_G \Sigma G_k + \gamma_Q Q_{k1} + \gamma_Q \Sigma \psi_0 Q_{ki} \tag{2}$$

Where:

- γ_G and γ_Q are the partial factors for loads, usually taken as:
  - γ_G =1.15
  - γ_Q =1.50 (values from Finnish National Annex)
- $\psi_0 Q_k$ combination value of a variable load,
- ψ_0 combination factor,
- Q_{k1} leading variable action,
- Q_{ki} accompanying action. /7./

The Leading (Q_{k1}) and Accompanying (Q_{ki}) actions should be chosen so that to obtain the most unfavourable combination.

There are three combinations of actions used for SLS: characteristic, frequent and quasi-permanent.

Characteristic combination:

$$\Sigma G_k + Q_{k1} + \Sigma \psi_{0,i} Q_{ki} \tag{3}$$

The characteristic combination is normally used for irreversible limit states. /7./

Frequent combination:

$$\Sigma G_k + \psi_1 Q_{k1} + \Sigma \psi_{2,i} Q_{ki} \qquad (4)$$

The frequent combination is normally used for reversible limit states. /7./

Quasi-permanent combination:

$$\Sigma G_k + \Sigma \psi_{2,i} Q_{ki} \qquad (4a)$$

The quasi-permanent combination is used for long-term effects and appearance of the structure, for example creep of concrete or deflection. /7./

The values of factors ψ_0, ψ_1, ψ_2 depend on the type of loading. Examples are given in table 3. /9./

Table 3. Examples of combination factors.

Action	ψ_0	ψ_1	ψ_2
Loads in residential buildings	0,7	0,5	0,3
Load in shopping areas	0,7	0,7	0,6
Wind loads	0,5	0,2	0

Examples of combinations are given in table 4.

Table 4. Combinations of loads

Combination	Expression	Example of use
Fundamental	$1.15\Sigma G_k + 1.5Q_{k1} + 1.5\Sigma \psi_0 Q_{ki}$	ULS - loss of static equilibrium or failure of structural member
Frequent	$\Sigma G_k + \psi_1 Q_{k1} + \Sigma \psi_{2,i} Q_{ki}$	SLS- calculation of cracking
Quasi-permanent	$\Sigma G_k + \Sigma \psi_{2,i} Q_{ki}$	SLS- long-term effects, calculation of deflection

4 MATERIALS FOR REINFORCED CONCRETE STRUCTURES

4.1 Concrete properties according to Russian norms

4.1.1 Concrete properties

The main properties of concrete used in designing are:

- Compression strength class B:

  (B10; B15; B20; B25; B30; B35; B40; B45; B50; B55; B60)

- Tensile strength class B_t (used seldom):

  B_t 0,8; B_t 1, 2; B_t 1,6; B_t 2,0; B_t 2,4; B_t 2,8; B_t 3, 2;

- Frost resistance class (established for structures exposed to freeze/thaw):

  (F50; F75; F100: F150; F200; F300; F400; F500)

- Water permeability class (established for structures if water impermeability is required):

  (W2; W4; W6; W8; W10; W12) /1./

4.1.2 Frost resistance class

Frost resistance class shows the maximum number of successive freeze-thaw cycles which water-saturated concrete sample can withstand without loosing more than 5% of its strength. For example a concrete with class F100 can withstand 100 successive freeze-thaw cycles.

The required frost resistance class is determined by the type of the structure and environmental conditions, nevertheless for structures subject to environment with temperature of ambient air from -5°C to -40°C frost resistance class must not be less than F75. /1./

4.1.3 Water permeability class

It shows a maximum water pressure (kg/sm^2) at which water does not penetrate through a concrete cylinder (height 150 mm). For example water permeability class W4 means that a 150 mm concrete cylinder stays water impermeable under the water pressure of 4 kg/sm^2 (0,4 MPa).

4.1.4 Compressive strength class of concrete

In the times of the USSR compressive strength class of concrete was marked by letter M, for example M150. The figure showed an average strength of concrete cubes during tests. But in fact the deviation from the theoretical value for ready concrete was often quite high, so there appeared a necessity to create a new strength grade.
The design code released in 1984 contained the new strength grade of concrete. It was principally new way of grading of concrete according to its testing strength. From that time strength class "B" is being used. The value of strength corresponding to each class is provided in 95% of tests.

The average strength of concrete sample R, MPa, corresponding to strength class is derived from the expression (only for heavyweight concrete):

$$R = \frac{B}{0,7635} \tag{5}$$

where:

B concrete strength class

R average strength of concrete sample, MPa

The following table contains the new strength classes of concrete with corresponding strength of cube and the old strength classes.

Table 5. Strength classes of concrete and corresponding strength of cube

Concrete strength class used nowadays	Average compressive strength of a cube, MPa	Old strength class (used before 1984)
B10	13,1	M150
B15	19,6	M200
B20	26,2	M250
B25	32,7	M350
B30	39,3	M400
B35	45,8	M450
B40	52,4	M550
B45	58,9	M600
B50	65,5	M600
B55	72,0	M700
B60	78,6	M800

4.1.5 Characteristic concrete strength (Russian norms)

The main concrete strength property used in design is a characteristic strength so called “normative strength”, it is obtained from the actual cubic strength by mathematical verification:

- normative compressive strength $R_{b,n}$
- normative tensile strength $R_{bt,n}$

Normative values of compressive and tensile strength according to compressive strength class are presented in table 6. /5./

Table 6. Characteristic compressive and tensile strength of concrete

Type of strength	**Characteristic (normative) strength $R_{b,n}$ and $R_{bt,n}$ (MPa) according to the compression strength class**										
	B10	**B15**	**B20**	**B25**	**B30**	**B35**	**B40**	**B45**	**B50**	**B55**	**B60**
Axial compression $R_{b,n}$	7,5	11,0	15,0	18,5	22,0	25,5	29,0	32,0	36,0	39,5	43,0
Axial tension $R_{bt,n}$	0,85	1,1	1,35	1,55	1,75	1,95	2,1	2,25	2,45	2,6	2,75

4.1.6 Design concrete strength (Russian norms)

The design compressive and tensile strengths of concrete R_b and R_{bt} are obtained by dividing the characteristic strength by the partial safety factors given in table 7 /5./:

$$R_b = \frac{R_{b,n}}{\gamma_b} \quad \text{and} \quad R_{bt} = \frac{R_{bt,n}}{\gamma_{bt}} \tag{6}$$

Table 7. Partial safety factors for concrete

Partial safety factors for concrete				
For ultimate limit state design			For serviceability limit state design	
γ_b	γ_{bt} for compression strength classes	γ_{bt} for tensile strength classes	γ_b	γ_{bt}
1,3	1,5	1,3	1,0	1,0

The design strengths of concrete R_b and R_{bt} according to compressive strength classes are presented in table 8, these values are used for ultimate limit state design (in Russia it is called 1st group of limit states) /5./.

Table 8. Design compressive and tensile strengths of concrete

Type of strength	Design values of strength R_b and R_{bt} (MPa) according to the strength class										
	B10	**B15**	**B20**	**B25**	**B30**	**B35**	**B40**	**B45**	**B50**	**B55**	**B60**
Axial compression, R_b	6	8,5	11,5	14,5	17,0	19,5	22,0	25,0	27,5	30,0	33,0
Axial tension R_{bt}	0,56	1,75	0,9	1,05	1,15	1,3	1,4	1,5	1,6	1,7	1,8

Table 9. Design values of tensile strength for tensile strength classes

Type of strength	Design value of tensile strength R_{bt} ,MPa , according to the tensile strength class						
	B_t 0,8	B_t 1,2	B_t 1,6	B_t 2,0	B_t 2,4	B_t 2,8	B_t 3,2
Axial tension, R_{bt}	0,62	0,93	1,25	1,55	1,85	2,15	2,45

The design tensile strength for tensile strength classes is determined using partial safety factor $\gamma_{bt} = 1.3$. /5./

In some cases the design strength values R_b and R_{bt} are multiplied by additional partial safety factors (Table 10), which take into account particular features (environmental impact, type of loading and etc.). /5./

Table 10. Additional safety factors for concrete strength

Partial safety factor	Description	Value
γ_{b1}	1. Depends on the static load duration	
	a) short-term loading	1,0
	b) long-term loading	0,9
γ_{b2}	2. For concrete structures without reinforcement	0,9
γ_{b3}	3. For structures concreted in vertical position	0,9
γ_{b4}	4. Environmental impact	
	a) structures exposed to freeze/thaw and extremely low temperatures	$\leq$1.0
	b) structures exposed to temperatures of t$\geq 40°C$	1,0
	c) according to special instructions	$\leq$1.0

4.1.7 Deformation properties (Russian norms)

The main deformation properties of concrete are the following:

- ultimate relative compressive and tensile strain ε_{b2} and ε_{bt2};
- modulus of elasticity E_b so called “initial modulus of elasticity”
- creep coefficient $\varphi_{b,cr}$
- Poisson’s ratio $v_{b.P}$
- linear coefficient of thermal expansion α_{bt} . /5./

According to Building code SP 52-101-2003 deformation properties such as relative strain and modulus of elasticity have different values for two different combinations of loading: Long acting loading and Not long acting loading, which are equivalent to the Eurocode Quasi-permanent and Characteristic combinations (see chapter “Classification of loads”).

The ultimate compressive and tensile strains of concrete have the following values:

- For “not long acting” loading:

  $\varepsilon_{b2} = 3,5$ ‰

  $\varepsilon_{bt2} = 0,15$ ‰

- For “long acting” loading values are given in table 11 depending on the relative humidity of ambient air. /5./

Modulus of elasticity of concrete for “not long acting” loading called “initial modulus of elasticity” is given in table 11 depending on the strength class of concrete.

For “long acting” loading the value of the modulus of elasticity is reduced and shall be calculated from the following expression taking into account creep:

$$\mathrm{E}_{b.\tau} = \frac{\mathrm{E}_b}{1+\varphi_{b.cr}} , \tag{7}$$

where $\varphi_{b.cr}$ - creep coefficient given in the table 12 depending on the concrete strength class and relative humidity of ambient air. /5./

Table 11. Initial modulus of elasticity for concrete

The initial modulus of elasticity E_b , GPa depending on the compressive strength class										
B10	B15	B20	B25	B30	B35	B40	B45	B50	B55	B60
19,0	24,0	27,5	30,0	32,5	34,5	36,0	37,0	38,0	39,0	39,5

Table 12. Creep coefficient

Relative humidity of ambient air,%	Creep coefficient $\varphi_{b.cr}$										
	B10	B15	B20	B25	B30	B35	B40	B45	B50	B55	B60
>75	2,8	2,4	2,0	1,8	1,6	1,5	1,4	1,3	1,2	1,1	1,0
40-75	3,9	3,4	2,8	2,5	2,3	2,1	1,9	1,8	1,6	1,5	1,4
< 40	5,6	4,8	4,0	3,6	3,2	3,0	2,8	2,6	2,4	2,2	2,0

Note: Relative humidity is applied as an average relative humidity of the warmest month in the construction region (according to SNiP 23-01). /5./

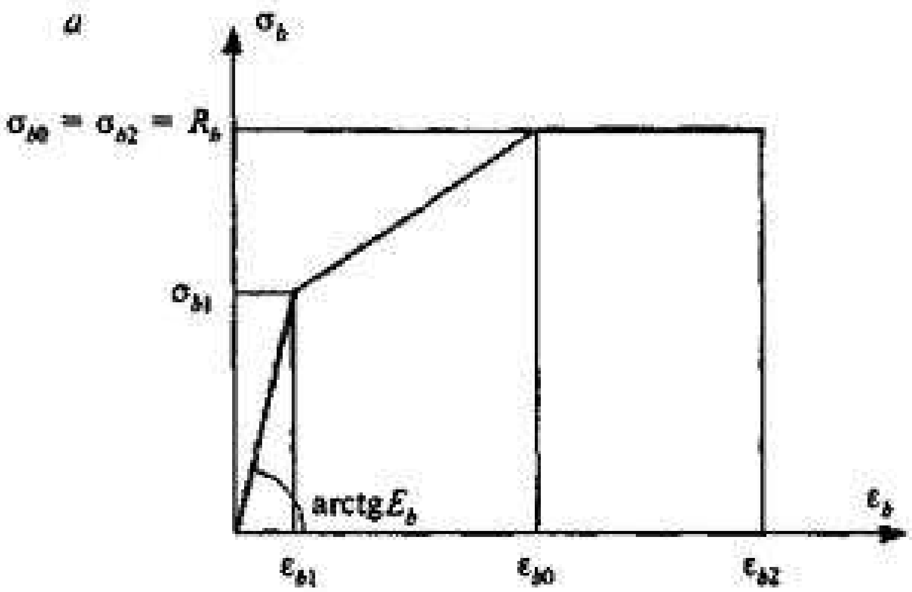
Table 13. Relative strains of concrete

Relative humidity of ambient air, %	Relative strain of concrete for “long lasting” loading, ‰					
	Compressive			Tensile		
	ε_{b0}	ε_{b2}	$\varepsilon_{b1,red}$	ε_{bt0}	ε_{bt2}	$\varepsilon_{bt1,red}$
>75	3,0	4,2	2,4	0,21	0,27	0,19
40-75	3.4	4,8	2.8	0,24	0,31	0,22
<40	4,0	5.6	3.4	0,28	0,36	0,26

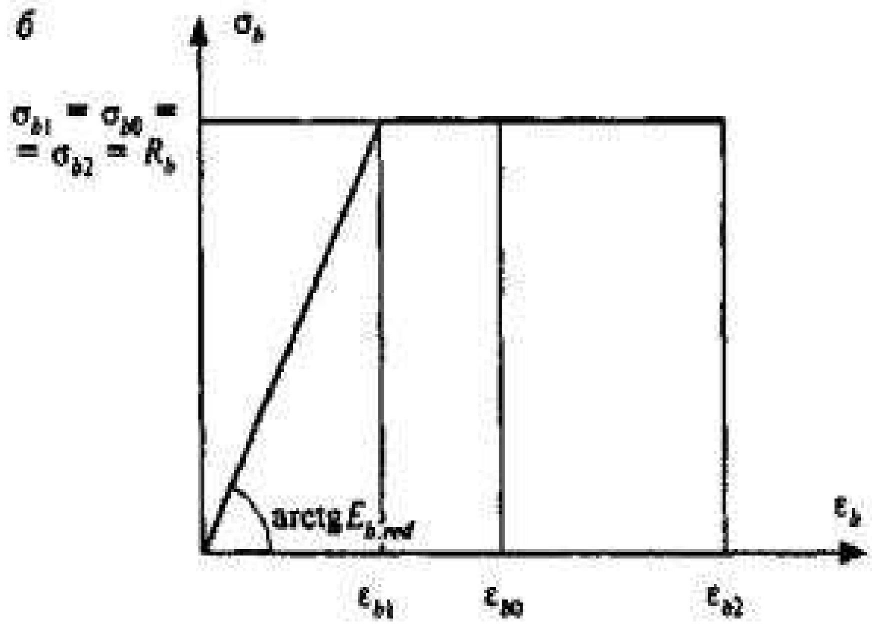
Note: Relative humidity is applied as an average relative humidity of the warmest month in the construction region (according to SNiP 23-01). /5./

4.1.8 Stress-strain diagram of the concrete (Russian norms)

There are two types of stress-strain diagrams: bilinear and trilinear (Figure 1, a, b). /5./



a. trilinear stress-strain diagram of concrete under compression



b. bilinear stress-strain diagram of concrete under compression

Figure 1. Stress-strain diagrams of concrete

The design of sections according to these diagrams is beyond the scope of this thesis, so no detailed formulas will be represented.

These stress-strain relations and diagrams are used for limit state design:

- For designing of strength of concrete elements under compression stress-strain diagrams with deformation properties for “not long lasting” loading are used; the bilinear diagram is used predominantly as the most simple.

- For calculating the appearance of cracks the trilinear diagram with properties for “not long lasting” loading is used.

- For calculating crack width bilinear diagram with properties for “long lasting” and “not long lasting” loading is used.

- For calculating deformation of concrete elements (curvature) the tri-linear diagram with properties for “long lasting” and “not long lasting” loading is used.

4.2 Concrete properties according to Eurocode 2

4.2.1 Concrete strength classes

The compressive strength of concrete is determined by concrete strength classes. According to Eurocode 2 there are strength classes from C 8/12 to C 100/115, the first figure means characteristic cylinder strength and the second means characteristic cube strength of the concrete.
Strength classes and their characteristic strength properties are presented in table 14. /8./

Table 14. Strength properties of concrete

Concrete strength class	Characteristic compression strength, MPa		Characteristic tensile strength, f_{ctk}, MPa
	Cylinder, f_{ck}	Cube, $f_{ck,cube}$	
C 8/10	8	10	0,8
C 12/15	12	15	1,1
C 16/20	16	20	1,3
C 20/25	20	25	1,5
C 25/30	25	30	1,8
C 30/37	30	37	2,0
C 35/45	35	45	2,2
C 40/50	40	50	2,5
C 45/55	45	55	2,7
C 50/60	50	60	2,9
C 55/67	55	67	3,0
C 60/75	60	75	3,1
C 70/85	70	85	3,2

The value of characteristic tensile strength is derived from the following expression:

$$f_{ctk} = 0.7 \cdot f_{ctm} \qquad (8)$$

where f_{ctm} is a mean value of axial tensile strength of concrete and it can be calculated from the following expression:

- $f_{ctm} = 0.3 \cdot f_{ck}^{2/3}$ for C $\leq$C50/60 (9)
- $f_{ctm} = 2.12 \cdot \ln(1 + \frac{f_{cm}}{10})$ for C >C50/60 (10)

In the last formula f_{cm} is a mean value of concrete cylinder strength /8./:

$$f_{cm} = f_{ck} + 8(MPa) \qquad (11)$$

4.2.2 Design compressive and tensile strength (Eurocode 2)

The value of the design compressive strength is defined as

$$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c \qquad (12)$$

where:

γ_c is a partial safety factor for concrete (see Table 15)

α_{cc} is a coefficient taking into account long term effects on the compressive strength and unfavourable effects resulting from the way the load is applied. The value $\alpha_{cc} = 0.85$ is given in the National Annex. /8;10./

The value of the design tensile strength is defined as

$$f_{ctd} = \alpha_{cc} \cdot f_{ctk} / \gamma_c \qquad (13)$$

where:

γ_c is a partial safety factor for concrete

$\alpha_{cc} = 0.85$ /8;10./

Partial factors for concrete for Ultimate Limit State design are given in table 15.
Partial factors for materials for serviceability limit state verification may be taken as $\gamma_c = 1.0$. /8./

Table 15. Partial safety factors for concrete for ULS

Design situations (type of loading)	γ_c for concrete
Persistent & Transient	1,5
Accidental	1,2

4.2.3 Deformation properties (Eurocode 2)

Stress-strain relations for the design of sections

There are 2 types of stress-strain diagrams used for static analysis: parabola-rectangle and bilinear diagrams. /8./

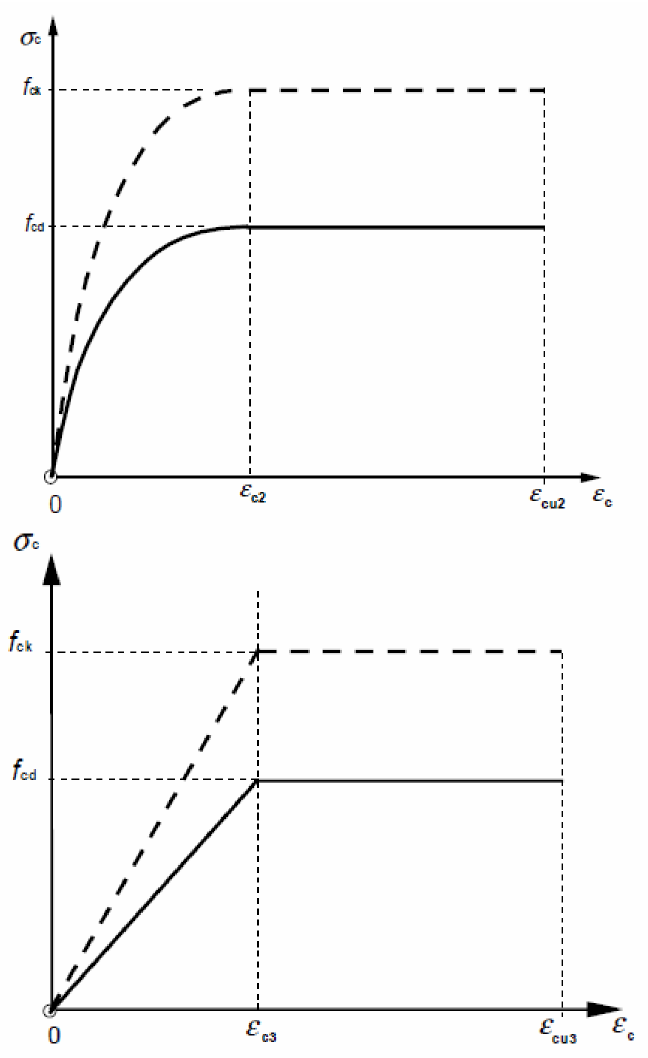


Figure 2. Parabola-rectangle and Bilinear diagrams for concrete in compression.

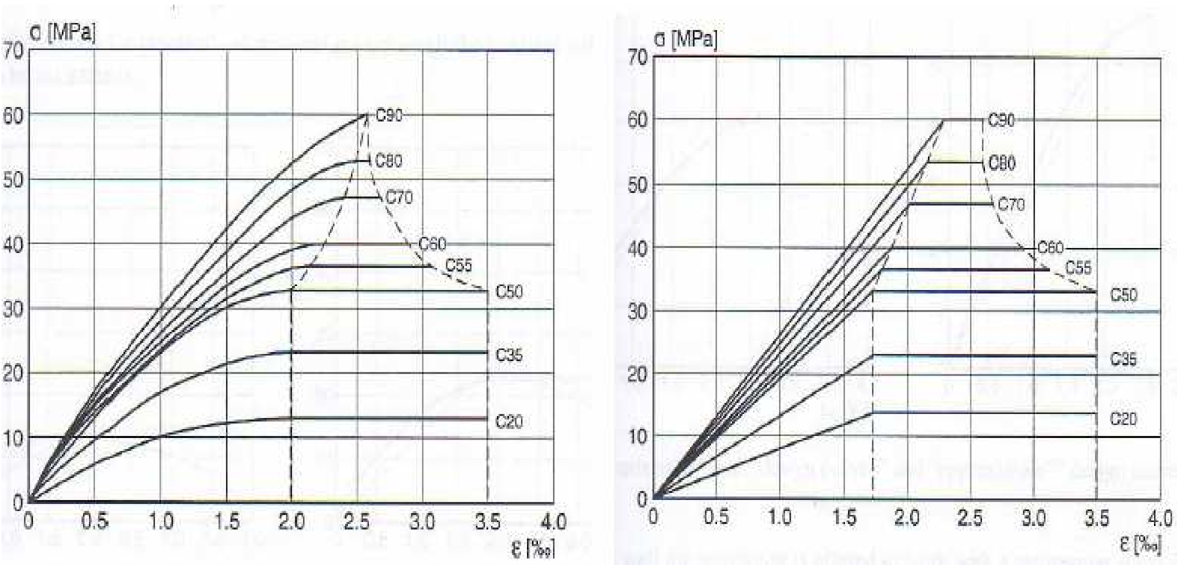
The main deformation properties used in calculations are given in table 16. Modulus of elasticity, E_{cm} , is a secant value between $\sigma_c = 0$ and $\sigma_c = 0.4 f_{cm}$.

Values ε_{c2} (ε_{cu2}) are used for design of sections using parabola-rectangle diagram and ε_{c3} (ε_{cu3})- for the design of sections using bilinear diagram. /8./

Table 16. Deformation properties of concrete

Concrete strength class	E_{cm}, GPa	ε_{c2},‰	ε_{cu2},‰	ε_{c3},‰	ε_{cu3},‰
C 12/15	27	2	3,5	1,75	3,5
C 16/20	29				
C 20/25	30				
C 25/30	31				
C 30/37	32				
C 35/45	34				
C 40/50	35				
C 45/55	36				
C 50/60	37				
C 55/67	38	2,2	3,1	1,8	3,1
C 60/75	39	2,3	2,9	1,9	2,9
C 70/85	41	2,4	2,7	2	2,7

Figure 3. Deformation properties of concrete



From the picture it is clear that the higher concrete strength shows more brittle behavior, reflected by shorter horizontal branch.

4.2.4 Creep (Eurocode 2)

Creep deformation of concrete is estimated by creep coefficient. The creep deformation $\varepsilon_{cc}(\infty;t_0)$ at time $t=\infty$ for a constant stress σ_c and a tangent modulus of elasticity E_{c0} at time t_0, is calculated from:

$$\varepsilon_{cc}(\infty;t_0)=\varphi(\infty;t_0)\cdot\frac{\sigma_c}{E_{c0}} \tag{14}$$

The value of the final creep coefficient, $\varphi(\infty;t_0)$, can be found from figure 4 for the case when the concrete is not subjected to a stress greater than $0.45 f_{ck}$. /8./

When the compressive stress of concrete exceeds the value $0.45 f_{ck}$ then creep non-linearity should be considered. Such a high stress can occur as a result of pretensioning, e.g. in precast concrete members at tendon level.

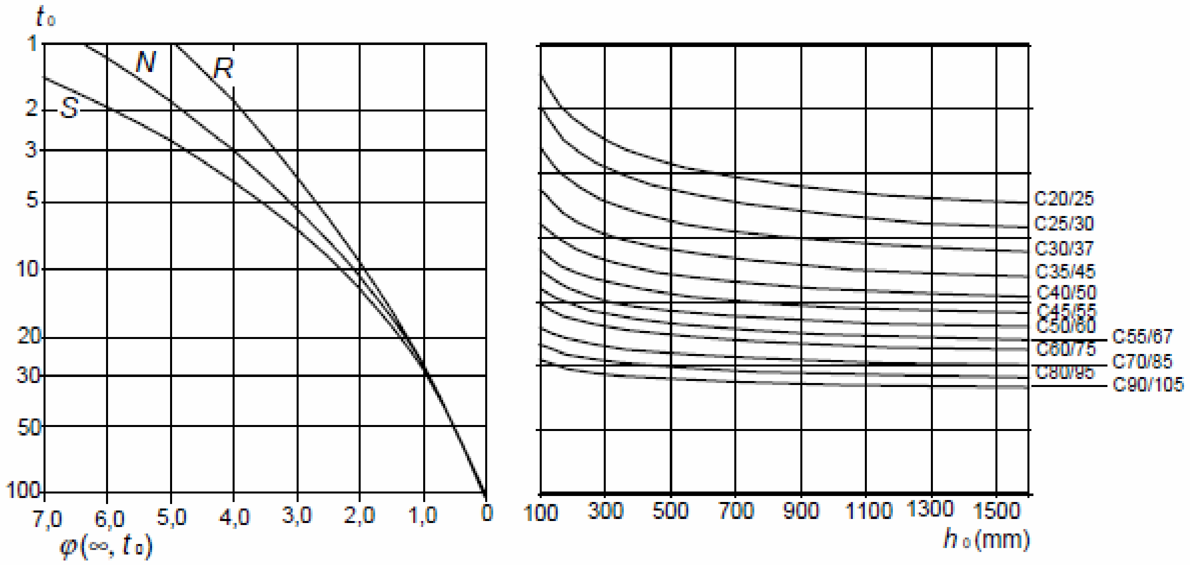
In such cases the creep coefficient should be modified as following:

$$\varphi_k(\infty;t_0) = \varphi(\infty;t_0)\cdot e^{15(k_\sigma - 0.45)} \qquad (15)$$

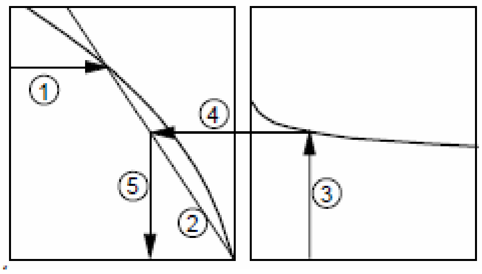
where:

$\varphi_k(\infty;t_0)$ non-linear creep coefficient, which replaces $\varphi(\infty;t_0)$

k_σ stress-strength ratio σ_c / f_{cm}, where σ_c is the compressive stress and f_{cm} is the mean concrete compressive strength at a time of loading (Expression 11). /8./



a) inside conditions - RH = 50%



Note:
- intersection point between lines 4 and 5 can also be above point 1
- for t_0 > 100 it is sufficiently accurate to assume t_0 = 100 (and use the tangent line)

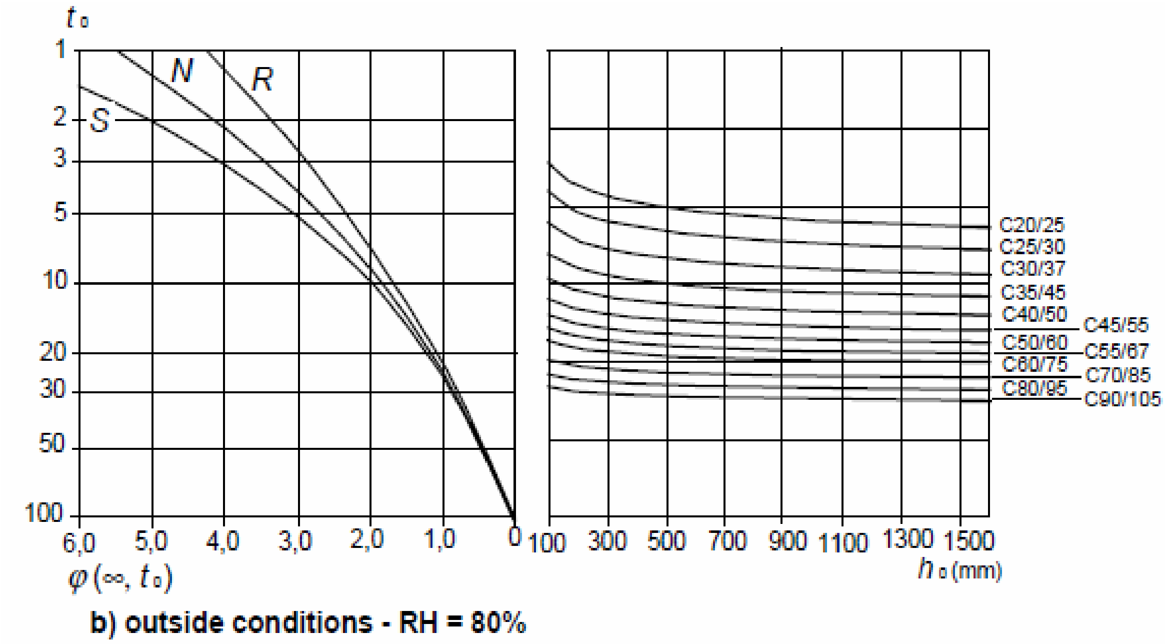


Figure 4. Final creep coefficient for concrete under normal environmental conditions

The following symbols are used in diagram:

- h_0 nominal size = $2A_c/u$, where A_c is the concrete cross sectional area and u is the perimeter of that part which is exposed to drying
- t_0 is an age of concrete in days when the load was applied
- S slowly hardening cement
- N normally hardening cement
- R rapidly hardening cement /8./

The following picture (Figure 5) is an example of determining the final creep coefficient using relations given above:

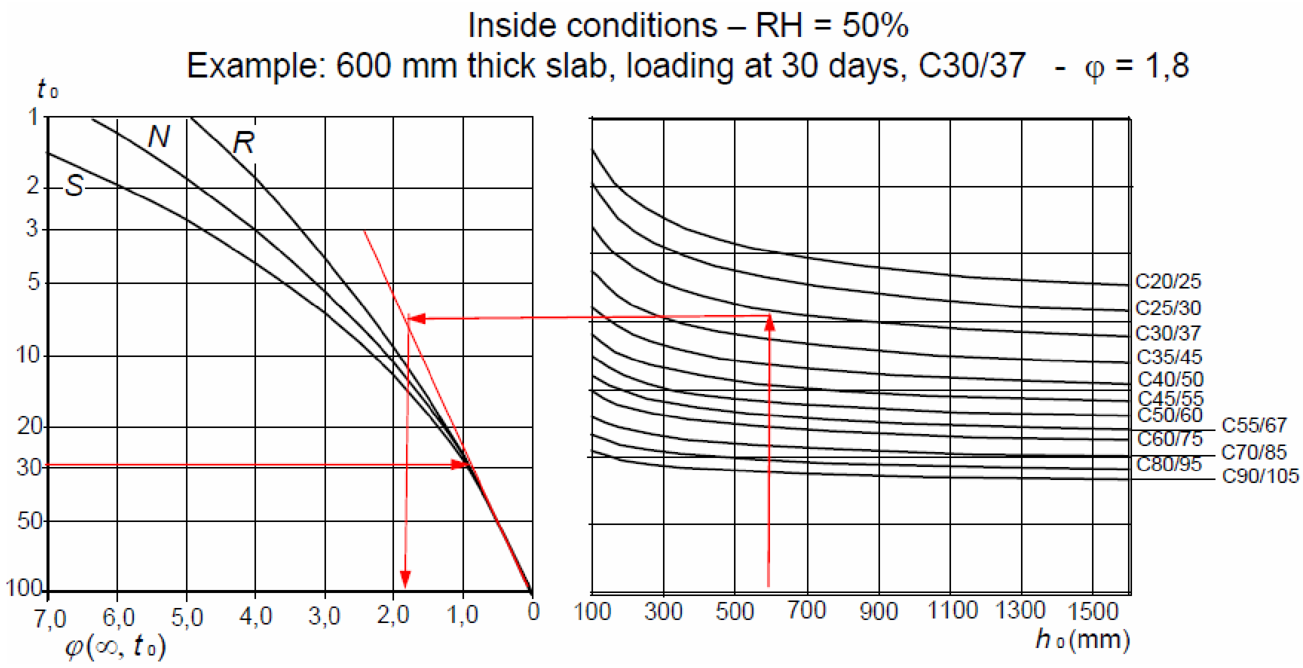


Figure 5. Determining the final creep coefficient

4.3 Concrete properties according to RakMK

4.3.1 Strength of concrete

The main strength property of concrete is compressive strength. The strength class of concrete means the characteristic compressive strength of a cube. Strength classes used in Finland and equivalent classes form Eurocode are presented in table 17. /12./

Table 17. Concrete strength classes and values of strength

Strength class	Corresponding strength class according to EN	Characteristic strength for a 150*300mm cylinder, MPa	Characteristic strength for a 150mm cube, MPa
K15	C12/15	12	15
K20	C16/20	16	20
K25	C20/25	20	25
K30	C25/30	25	30
K35	C28/35	28	35
K40	C32/40	32	40
K45	C35/45	35	45
K50	C40/50	40	50
K55	C45/55	45	55
K60	C50/60	50	60
K70	C57/70	57	70
K80	C65/80	65	80

Note: High-strength classes K90 and K100 are not represented here.

The characteristic value for effective compressive strength of concrete in a structure shall be calculated from the following expression:

$$f_{ck} = 0.7K \qquad (16)$$

The characteristic value of tensile strength of concrete shall be calculated from the following expression:

$$f_{ctk} = \alpha \cdot K^{2/3} \qquad \text{(for K≤K60)} \qquad (17)$$

where:

$\alpha = 58 \cdot \varepsilon_{cu} \leq 0.2$

ε_{cu} shall be calculated from Expression (19) /12./

The following table contains effective characteristic compressive and tensile strengths according to strength classes.

Table 18. Characteristic values of concrete compressive and tensile strength

Concrete strength class	Compressive strength of a cube, MPa	Characteristic compressive strength, MPa	Characteristic tensile strength, MPa
K15	15	10,5	1,22
K20	20	14	1,47
K25	25	17,5	1,71
K30	30	21	1,93
K35	35	24,5	2,14
K40	40	28	2,34
K45	45	31,5	2,53
K50	50	35	2,71
K55	55	38,5	2,89
K60	60	42	3,07
K70	70	49	Calculated as for high-strength concrete
K80	80	56	

The design compressive and tensile strength of concrete is obtained by dividing the characteristic value by the partial safety factor (see table 19). /12./

Table 19. Partial safety factors for concrete

Partial safety factor for concrete, γ_c	Structural class	Reinforced structure	Unreinforced structure
	1	1,4	2,0
	2	1,5	2,3
	3	1,9	2,7

4.3.2 Deformation properties of concrete (RakMK)

The value E_c for the modulus of elasticity of concrete shall be calculated from the following expression:

$$E_c = 5000 \cdot k \cdot \sqrt{K} \quad \text{(MPa)} \tag{18}$$

where:

K is the nominal strength of concrete (strength class)

k $= \dfrac{\rho_c}{2400} \leq 1.0$

ρ_c is the density of the concrete kg/m^3 /12./

The ultimate compressive strain ε_{cu} of the concrete shall be calculated from the following expression:

$$\varepsilon_{cu} = (1.1 + \frac{\rho_c}{1000}) \text{ , ‰} \qquad \geq 2‰ \tag{19}$$

≤3.5‰ /12./

4.3.3 Creep (RakMK)

When calculating deformation due to a loading which acts for a long period of time, the effect of creep of concrete must be taken into account.

The expressions presented below shall be valid if concrete stress due to long-term loads under serviceability conditions does not exceed the value $0.6 f_{ck}$. The final creep of the concrete is:

$$\varepsilon_{cc} = \varphi \cdot \varepsilon_c \tag{20}$$

where:

ε_c is the instantaneous deformation caused by long-term loading $\varepsilon_c = \sigma_c / E_c$

φ is the creep coefficient /12./

The final creep coefficient may be calculated from the following expression:

$$\varphi = k_t \cdot k_{ch} \cdot \varphi_0 \tag{21}$$

where:

φ_0 is the basic value of the creep coefficient, values for it are presented in table 20 depending on the relative humidity of ambient air;

k_{ch} coefficient depending on the thickness of structure (Table 21);

$k_t = (2.5 - 1.5 \cdot K_j / K), \geq 1.0$ where K is the nominal strength of concrete and K_j is the compressive strength of concrete at the beginning or change of loading. /12./

Table 20. The basic value of the creep coefficient

Relevant environmental conditions	Relative humidity, %	φ_0
Water	100	1
Extremely humid air	90	1,5
Outdoor air	70	2
Dry air	40	3

Table 21. Coefficient k_{ch}

h_e, mm	≤50	100	200	300	≥500
k_{ch}	1,20	1,00	0,80	0,75	0,70

So, there is not a definite expression for calculating creep of concrete in Russian norms like it is in Eurocode and RakMK. Creep is estimated by reducing the value of modulus of elasticity.

4.4 Reinforcing steel

4.4.1 Russian classification

According to Building regulations SNiP 52-01-2003 for reinforcing of concrete the following types of reinforcement should be used:

- hot-rolled smooth and ribbed reinforcement with diameter 3-80 mm;
- thermo-mechanically hardened ribbed reinforcement, diameter 6-40 mm;
- cold-worked smooth of ribbed reinforcement, diameter 3-12 mm
- steel wires, diameter 6-15 mm. /5./

The main property of reinforcement which is used in the design of structures is a tensile strength class, which is marked as:

- A - for hot-rolled and thermo-mechanically hardened reinforcement;
- B - for cold-worked reinforcement
- K - for steel wires.

Strength class of reinforcement means the assured yield strength of steel, MPa, and can take values from the following ranges:

- from A240 to A1500
- from B500 to B2000
- from K1400 to K2500

Reinforcement can also have special properties such as weldability or resistance to corrosion, for example:

- A500C, where C means weldability
- A1000K, where K means resistance to corrosion
- Aт1200, where Aт means hot-rolled thermo-mechanically hardened reinforcement

The design code SP 52-101-2003 supposes the design of structures using only classes of reinforcement represented in the following table /5./:

Table 22. Reinforcement steel grades used for design

Smooth reinforcement	Ribbed reinforcement			
A240 (A1)	A300 (A2)	A400, A400C (A3)	A500, A500C	B500, B500C (Bp1)

Note: The most frequently used grades nowadays are A400 and A500 and also B500 in the form of welded meshes.

4.4.2 Strength of reinforcement bars (Russian norms)

The main strength property of reinforcing steel is normative (characteristic) tensile strength $R_{s.n}$, which is determined according to a strength class (table 23). Normative tensile strength is used as a design value for Serviceability Limit State. /6./

Table 23. Strengths of reinforcement

Reinforcement grade	Used diameters, mm	Characteristic tensile strength $R_{s.n}$, as well as the design tensile strength for SLS $R_{s.ser}$, MPa
A240	6-40	240
A300	6-40	300
A400	6-40	400
A500	10-40	500
B500	3-12	500

The design strength of reinforcement is obtained by dividing the characteristic value by partial safety factor /5./:

$$R_s = \frac{R_{s.n}}{\gamma_s} \qquad (22)$$

Table 24. Partial safety factors for reinforcement

Reinforcement grade	A240, A300, A400	A500	B500
Partial safety factor for Ultimate Limit State	1,1	1,15	1,2

For the determination of the design value of tensile strength of transverse reinforcement (stirrups) for Ultimate Limit State an additional safety factor $\gamma_{si} = 0.8$ is used /5./:

$$R_s = 0.8 \cdot \frac{R_{s.n}}{\gamma_s}, \quad \text{but } \leq 300\text{MPa} \qquad (23)$$

Design values of tensile strength of reinforcement are presented in table 25.

Table 25. Design values of tensile and compressive strength of steel

Reinforcement class	The design value of strength, MPa		
	Tensile		Compressive, R_{sc}
	For longitudinal reinforcement, R_s	For transverse reinforcement, R_{sw}	
A240	215	170	215
A300	270	215	270
A400	355	285	355
A500	435	300	435 (400)*
B500	415	300	415 (360)

* In the brackets there are values for calculation in the case when short-term load is taken into account. /5./

The diameters of steel bars usually used in Russia are specified in table 26.

Table 26. Diameters of bars used in Russia

Diameter or bar, mm	Cross-section area, mm^2	Weight for 1 running meter, kg
3 (only B class)	7,07	0,052
4 (only B class)	12,57	0,092
5 (only B class)	19,63	0,144
6	28,3	0,22
8	50,3	0,39
10	78,5	0,62
12	113,1	0,89
14	153,9	1,21
16	201,1	1,58
18	254,5	2,00

20	314,2	2,47
22	380,1	2,98
25	490,9	3,85
28	615,8	4,83
32	804,2	6,31
36	1017,9	7,99
40	1256,6	9,86

The diameter of the bar is also specified in the marking, for example:

- 28 A500- hot-rolled steel, characteristic yield strength 500 MPa, Ø28mm;
- 3 Bp1- cold worked steel wire, characteristic yield strength 500 MPa, Ø3mm.

4.4.3 Classification of reinforcing steel according to RakMK

Nowadays in Finland hot-rolled and cold-worked reinforcement are in use. The same as in Russia the class of reinforcement shows the yield strength of steel and other special properties. The following classes of reinforcement are mostly used:

- A500HW: weldable hot-rolled ribbed steel, $f_{sk} = 500MPa$
- A700HW: weldable hot-rolled ribbed steel, $f_{sk} = 700MPa$
- B500K: cold-worked ribbed steel, $f_{sk} = 500MPa$
- B700K: cold-worked ribbed steel, $f_{sk} = 700MPa$
- B600KX: cold-worked stainless ribbed steel, $f_{sk} = 600MPa$
- S235JRG2: smooth round bar used as a lifting loops, $f_{sk} = 235MPa$
- S355J0: smooth round bar used as a lifting loops, $f_{sk} = 355MPa$ /12./

Diameters of reinforcement bars used in Finland are specified in table 27.

Table 27. Diameters of bars used in Finland

Diameter, mm	Cross-section area, mm^2	Weight for one running meter, kg
6	28,3	0,22
8	50,3	0,39
10	78,5	0,62
12	113,1	0,89
16	201,1	1,58
20	314,2	2,47
25	490,9	3,85
32	804,2	6,31

The design value of tensile strength is obtained by dividing characteristic value by the partial safety factor, γ_s , specified in table 28. Partial safety factor for reinforcing steel for serviceability limit state (SLS) is $\gamma_s = 1.0$. /12./

Table 28. Partial safety factors for steel

	Structural class	Normal steel	Prestressing steel
Partial safety factor for steel	1	1,1	1,15
	2	1,2	1,25
	3	1,35	1,35

For calculation of shear reinforcement the characteristic strength of steel may not be assumed to be more than:

- $f_{yk} = 500MPa$ for ribbed bars and meshes;
- $f_{yk} = 400MPa$ for indented bars;
- $f_{yk} = 360MPa$ for smooth bars. /12./

4.4.4 Classification of steel according to Eurocode

The design of structures according to Eurocode 2 is valid for the reinforcing steel with the yield strength up to 700 MPa. As in Finnish norms steels are divided into classes:

- A- hot-rolled,
- B- cold-worked

Partial safety factors for steel are:

- $\gamma_s = 1.15$ for Ultimate limit state design
- $\gamma_s = 1.0$ for Serviceability limit state design /8./

The design of structures according to Eurocode supposes using the same bar diameters as in Finland.

Summary

In this chapter the main characteristics of concrete and reinforcing steel used in different norms are studied. Some differences in material grading were found. In Europe and in Finland concrete is classified according to its compressive strength and only the compressive strength class is used in the design.

In Russia, besides compressive strength grading, there are frost resistance classes (e.g. F100), water permeability classes (e.g. W4) and tensile strength classes (B_t 1,6).

The last one is used in cases when the tensile strength of concrete is a main characteristic in design, for example in tensioned elements. Interesting fact is that if we choose compressive strength class as the main property of concrete, its design tensile strength is determined using partial safety factor $\gamma_{bt} = 1.5$, but when we choose tensile strength class as a main property in the design, the design value of tensile strength is determined using safety factor $\gamma_{bt} = 1.3$.

Water permeability class is used when special requirements for structures are given. Frost resistance class is determined for all outdoor structures if the temperature of ambient air can be lower than -5°C.

There is no such classification in Finnish and European norms. Water permeability can be determined when required, but concrete is not classified so.

Comparison of strength classes is presented in tables in appendix 1.

In all studied norms reinforcing steel is classified according to its tensile strength and type. Classes are almost the same. The difference is that in Europe and in Russia different diameters of reinforcing bars are used. One more thing is that partial safety factors for reinforcement are different, but not significantly. The design values of the same classes of steel are presented in appendix 4.

5 REINFORCEMENT

5.1 Regulations for reinforcement according to Building regulations of Russian Federation

5.1.1 Anchorage of reinforcement

Anchorage is implemented by one of the following methods:

- straight anchoring without bends or hooks
- bends, hooks or loops in the end of the bar
- welded transverse bar
- using other special anchoring devices

Straight anchorage without bends is allowed to use only for ribbed bars.

The basic anchorage length required to transmit a bond stress to concrete can be defined from the following expression:

$$l_{0,an} = \frac{R_s \cdot A_s}{R_{bond} \cdot u_s}, \qquad (24)$$

where A_s and u_s are the cross section and a perimeter of rebar. /5./

R_{bond} is an ultimate bond stress which is calculated from the following expression:

$$R_{bond} = \eta_1 \cdot \eta_2 \cdot R_{bt}, \qquad (25)$$

where:

R_{bt} is the design tensile stress of concrete;

η_1, η_2 coefficients taking into account the type of surface of rebar and its size, given in table 29. /5./

Table 29. Coefficients η_1, η_2

Coefficient	Type of reinforcement	Value
η_1	Bars with smooth surface	1,5
	Cold worked ribbed bars	2,0
	Hot rolled ribbed bars	2,5
η_2	Bars with diameter $d \leq 32mm$	1,0
	Bars with diameter $d > 32mm$	0,9

A required (design) anchorage length is determined by taking into account the ratio of required reinforcement to provide:

$$l_{an} = \alpha \cdot l_{0,an} \cdot \frac{A_{s,cal}}{A_{s,ef}}, \qquad (26)$$

where:

$l_{0,an}$ - basic anchorage length (24)

$A_{s,cal}$ - cross section of reinforcement required according to the design

$A_{s,ef}$ - cross section of provided reinforcement. /5./

Coefficient α for ribbed bars or for smooth bars with hooks or loops without other special anchoring devices:

- $\alpha = 1.0$ for bars under tension;
- $\alpha = 0.75$ for bars under compression. /5./

It is allowed to reduce the required (design) anchorage length up to 30% depending on the amount of transverse reinforcement in the anchoring zone and the type of anchoring devices (for example welded transverse bars or bended ends).

In any case the design anchorage length is taken not less than the following values:

- $0,3l_{0,an}$
- $15\varnothing$
- 200mm /5./

5.1.2 Longitudinal reinforcement (Russian norms)

In Russian norms the area of reinforcement in cross-section is determined by the “coefficient of reinforcing”:

$$\mu_s = \frac{A_s}{b \cdot h_0} \cdot 100\%, \qquad (27)$$

where:

A_s - area of longitudinal tension or compression (if it is necessary according to design) reinforcement;

b - width of rectangular section or width of the web for I- or T- beams

h_0 - working height of section (height of section minus concrete cover). /5./

The minimum amount of reinforcement in different types of structural members is always regulated. So the amount of reinforcement can not be less than the value required in norms.

The required amounts of main reinforcement for different elements are given in table 30. For structural members under compression (columns, wall) this value depends on the slenderness of the member. /5./

Table 30. Minimum reinforcement

Type of the structure	$\mu_{s,\min}, \%$
Tension reinforcement in bended elements or in eccentrically tensioned elements when the tension force is beyond the cross-section	0,10
Reinforcement in elements under compression with slenderness:	
a. $l_0 / i < 17$ (for rectangular sections $l_0 / h \le 5$)	0,10
b. $17 < l_0 / i \le 35$ (for rectangular sections $5 < l_0 / h \le 10$)	0,15
c. $35 < l_0 / i < 83$ (for rectangular sections $10 < l_0 / h \le 25$)	0,20
d. $l_0 / i \ge 83$ (for rectangular sections $l_0 / h > 25$)	0,25
Reinforcement in slabs (both top and bottom)	0,20
Reinforcement in centrally and eccentrically tensioned elements (all - top and bottom)	0,20

In concrete structures without working reinforcement (unreinforced concrete) helping reinforcement must be installed:

- In the places where shape or dimensions of section change
- In concrete walls above and beneath openings
- In eccentrically compressed elements in tension zone (if it appears) the minimal reinforcement is $\mu_{s,\min} = 0,025\%$ /5./

The clear distance between longitudinal bars should not exceed the following values:

In beams and slabs:

- 400 mm

In columns:

- 400 mm in the direction perpendicularly to the bending moment
- 500 mm in the direction of the plane of bending moment

In walls the distance between vertical reinforcement bars should not be more than:

- $2t$ (t - wall thickness)
- 400 mm

The distance between horizontal bars should not be more than 400 mm. /5./

If a width of cross-section of beam is more than 150 mm the number of longitudinal tension reinforcement bars should not be less than 2.
In beams 1/2 of the span reinforcement should continue up to the support and be anchored properly;
In slabs 1/3 of the span reinforcement should continue up to the support.

In beams with the height of cross-section more than 700 mm additional longitudinal bars should be placed between the top and bottom reinforcement so that the distance between bars (in vertical direction) is not more than 400 mm. /5./

The area of additional bars should be at least 0.001 A_c^* (Figure 6), the area A_c^* is shown as a shaded zone.

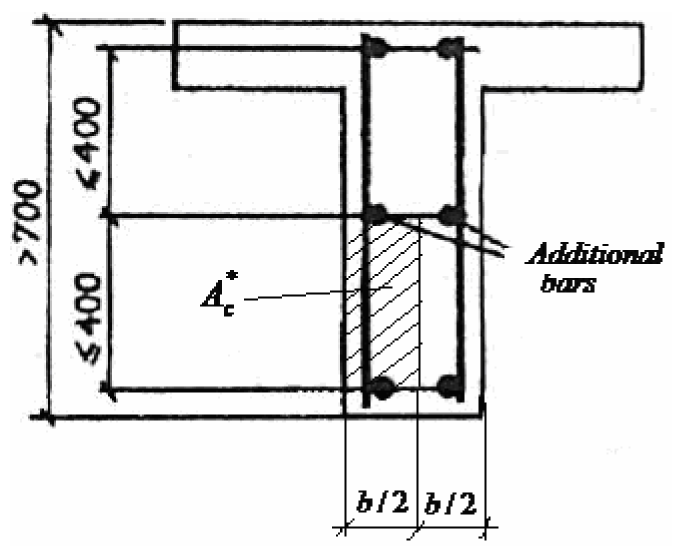


Figure 6. Additional steel if the depth of cross-section is >700 mm

5.1.3 Transverse reinforcement (SP 52-101-2003)

Transverse reinforcement (shear reinforcement) usually consists of welded or bended stirrups.

The diameter of transverse reinforcement in columns should not be less than:

- 0.25 of the biggest size of longitudinal bars
- 6 mm

In beams- not less than 6 mm. /5./

In structural members where shear force can not be taken by concrete, shear reinforcement should be installed with longitudinal spacing not more than $0.5h_0$ [1] or 300 mm.

In slabs with thickness not more than 300 mm and beams with height less than 150 mm it is not necessary to install shear reinforcement if all share force can be taken by concrete.

In slabs with thickness more than 300 mm and beams higher than 150 mm shear reinforcement must be installed with longitudinal spacing not more than $0.75h_0$ and 500 mm, even if all shear force can be taken by concrete.

In columns and beams the transverse reinforcement should be installed with spacing not more than:

- 15d, where d- diameter of longitudinal bars
- 500 mm

If the area of longitudinal reinforcement placed along one side of cross-section is more than 1.5% of the concrete area, transverse reinforcement should be placed with the spacing not more than:

- 10d
- 300 mm /5./

---

[1] h_0 - working depth of cross-section (total depth minus concrete cover)

In columns if the number of longitudinal bars along one side of cross-section is more than 4, additional stirrups should be placed. /5./

If the number of longitudinal bars along one side of cross-section is not more than 4 it is allowed to use one stirrup (Figure 7)

a) Additional stirrup b) One stirrup

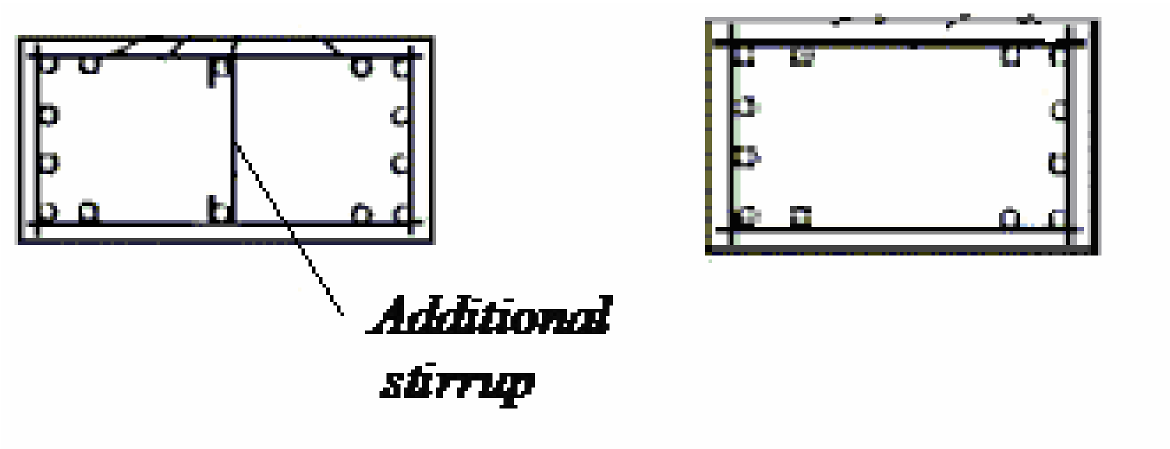


Figure 7. Additional stirrup

5.1.4 Splices of bars (SP 52-101-2003)

For connecting longitudinal steel bars and for transmitting forces from one bar to another the following types of joints are used:

- lapping of bars, with or without bends or hooks
- welding
- mechanical devices assuring load transfer from one bar to another

Lap joints

Lap joints are used for connecting steel bars with the diameter not more than 40 mm.

The design lap length is:

$$l_l = \alpha \cdot l_{0,an} \cdot \frac{A_{s,cal}}{A_{s,ef}} \tag{27}$$

The total amount of lapped bars in one section should not exceed 50% of all bars for ribbed bars and 25% for smooth bars.

The width of the section is considered to be $1.3l_l$. /5./

It is allowed to increase the percentage of laps in one section up to 100%, but in that case coefficient α in formula (27) must be taken as $\alpha = 2$.

If the total amount of lapped bars is between 50% (25% for smooth bars) and 100%, the value of α should be determined by means of interpolation.

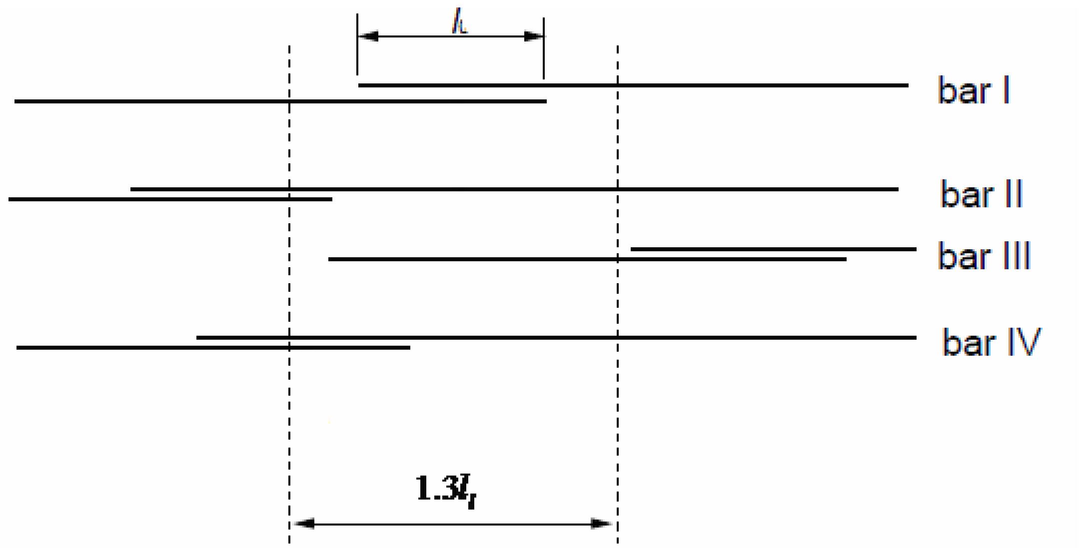


Figure 8. Section being considered in the calculation of the percentage of lapped bars

- The clear distance between two lapped bars should not be greater than 4d;

- The clear distance between two adjacent lap joints should not be less than 2d or 30 mm.

- If special anchoring devices are used, the length of lapping can be reduced, but reduction should be not more than 30%. /5./

In any case the length of lap must not be less than:

- $0.4\alpha \cdot l_{0,an}$
- $20d_s$
- 250 mm /5./

5.1.5 Spacing of bars

The spacing of bars shall be such that the concrete can be placed and compacted satisfactorily for the development of adequate bond.
The clear distance between parallel steel bars should be not less than:

- diameter of a bar
- 25 mm for bottom steel if the bars are all in one or two layers
- 30 mm for top steel (one or two layers)
- 50 mm for bottom steel if the bars are in several layer /5./

5.2 Regulations for reinforcement according to Eurocode 2

5.2.1 Anchorage

Here is the way the anchorage length is calculated according to Eurocode 2.
The design value of the ultimate bond stress for ribbed bars is:

$$f_{bd} = 2.25\eta_1 \cdot \eta_2 \cdot f_{ctd} \ , \qquad (28)$$

where:

η_1 is a coefficient related to the quality of bond conditions:

η_1 =1,0 when “good” conditions are obtained

η_1 = 0,7 for all other cases and for bars in elements built with slip-forms

η_2 is related to the bar diameter:

η_2 = 1,0 for $d \leq 32mm$

η_2 = (132- d)/100 for $d > 32mm$

The basic anchorage length for the bar of diameter d is:

$$l_b = \frac{d}{4} \cdot \frac{\sigma_{sd}}{f_{bd}}, \qquad (29)$$

where:

σ_{sd} is the design stress in the bar

f_{bd} is the ultimate bond stress. /8./

The design anchorage length l_{bd} is:

$$l_{bd} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_b \geq l_{b,\min} \qquad (30)$$

where $\alpha_1, \alpha_2, \alpha_3, \alpha_4$ and α_5 - coefficients given in table 31. /8./

Table 31. Alfa coefficients for calculation of anchorage length

Influencing factor	Type of anchorage	Reinforcement bar	
		In tension	In compression
Shape of bars	Straight	α_1 = 1,0	α_1 = 1,0
	Other than straight	α_1 = 0,7 if $c_d > 3\phi$ otherwise α_1 = 1,0	α_1 = 1,0
Concrete cover	Straight	$\alpha_2 = 1 - 0{,}15\,(c_d - \phi)/\phi$ $\geq 0{,}7$ $\leq 1{,}0$	α_2 = 1,0
	Other than straight	$\alpha_2 = 1 - 0{,}15\,(c_d - 3\phi)/\phi$ $\geq 0{,}7$ $\leq 1{,}0$	α_2 = 1,0

Influencing factor	Type of anchorage	Reinforcement bar	
		In tension	In compression
Confinement by transverse reinforcement not welded to main reinforcement	All types	$\alpha_3 = 1 - K\lambda$ $\geq 0{,}7$ $\leq 1{,}0$	$\alpha_3 = 1{,}0$
Confinement by welded transverse reinforcement*	All types	$\alpha_4 = 0{,}7$	$\alpha_4 = 0{,}7$
Confinement by transverse pressure	All types	$\alpha_5 = 1 - 0{,}04p$ $\geq 0{,}7$ $\leq 1{,}0$	-

Where:

λ $= (\Sigma A_{st} - \Sigma A_{st,min})/ A_s$

ΣA_{st} cross-sectional area of the transverse reinforcement along the design anchorage length l_{bd}

$\Sigma A_{st,min}$ cross-sectional area of the minimum transverse reinforcement $= 0{,}25\ A_s$ for beams and 0 for slabs

A_s area of a single anchored bar with maximum bar diameter

K values shown in Figure **4.1**

p transverse pressure [MPa] at ultimate limit state along l_{bd}

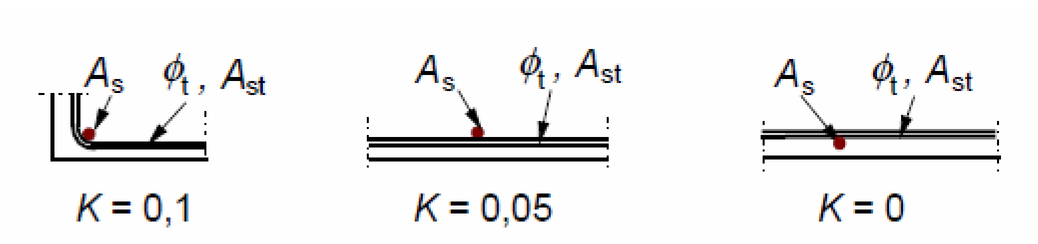


Figure 9. Values for coefficient K for beams and slabs

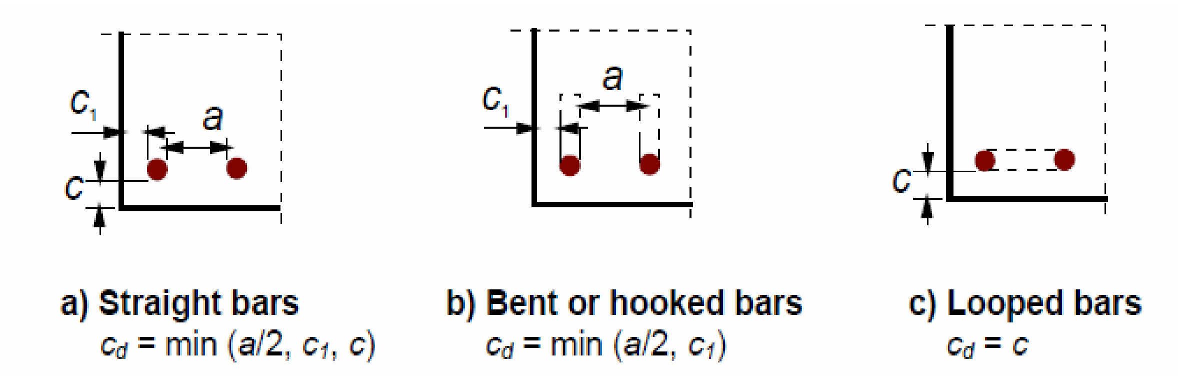


Figure 10. Values for coefficient c_d

In any case the design anchorage length should not be less than $l_{b,\min}$, which:

For anchorages in tension:

- $0.3l_b$
- 15d
- 100 mm

For anchorages in compression:

- $0.6l_b$
- 15d
- 100 mm /8./

The anchorage length is measured from the line of contact between beam and support:

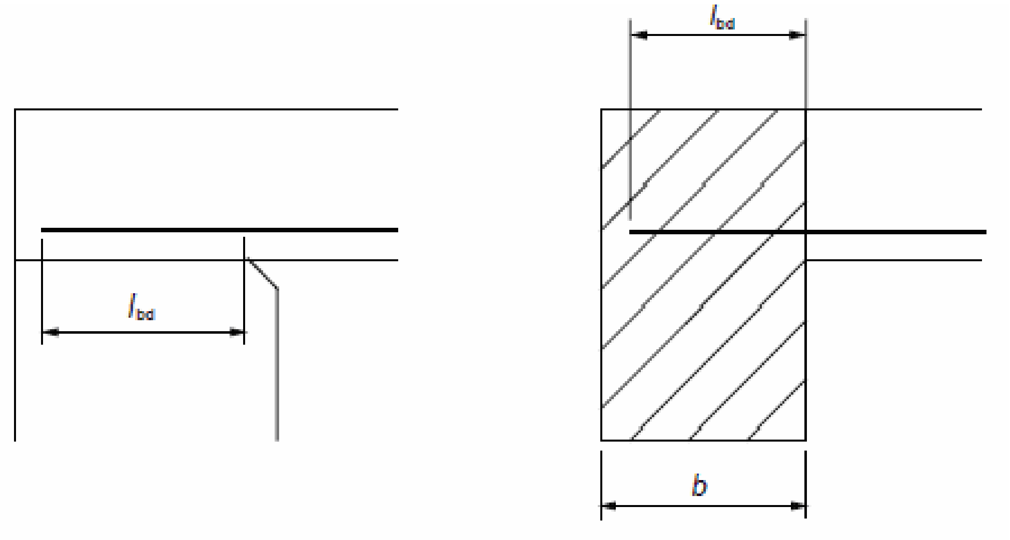


Figure 11. Anchorage of bottom reinforcement at end supports

5.2.2 Splices of bars (Eurocode 2)

The requirements for lap splicing given in Eurocode 2 are:

- the clear transverse distance between two lapped bars should not be more than $4\varnothing$ or 50 mm
- the longitudinal distance between two adjacent laps should not be less than 0.3 times the lap length, l_0
- in case of adjacent laps, the clear distance between adjacent bars should not be less than $2\varnothing$ or 20 mm. /8./

These rules are represented on the Figure 12.

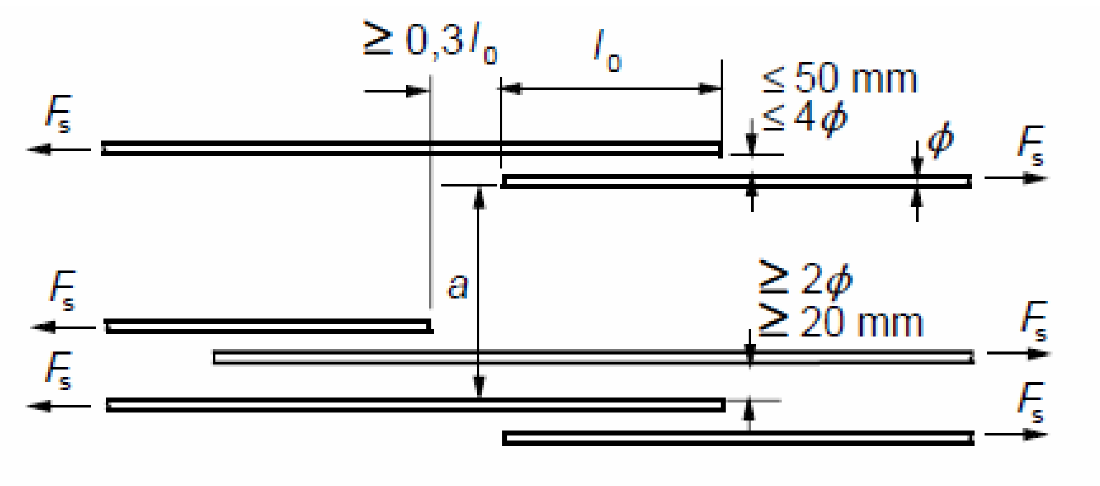


Figure 12. Regulations for laps

When provision comply with the requirements given above, the permissible percentage of lapped bars in tension may be 100% in one section where the bars are all in one layer. Where the bars are in several layers the percentage should be reduced to 50%. /8./

All bars in compression or secondary reinforcement may be lapped in one section.

The design lap length is:

$$l_0 = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_5 \cdot \alpha_6 \cdot l_b \cdot (A_{s,req} / A_{s,prov}) \geq l_{0,\min} \quad (31)$$

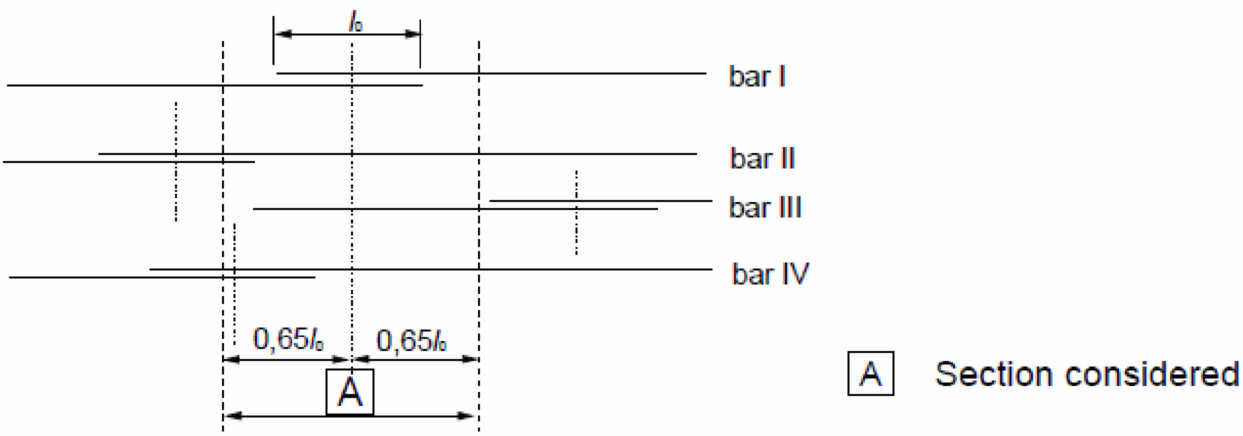
where:

- l_b is calculated from Expression 29
- values for $\alpha_1, \alpha_2, \alpha_3$ and α_5 may be taken from Table 31, however for the calculation of α_3, $\sum A_{st,\min}$ should be taken as $0.1A_s$, where A_s is the area of one lapped bar.
- α_6 is the coefficient depending on the percentage of bars lapped in one cross-section, values for it are given in table 32. /8./

Table 32. Values of the coefficient α_6

Percentage of lapped bars relative to the total cross-section area of reinforcement	<25%	33%	50%	>50%
α_6	1,0	1,2	1,4	1,5

The length of the section is considered to be $1.3l_0$:



Example: Bars 2 and 3 are outside the section being considered: 50% of all bars are lapped, α_6 =1.4.

Figure 13. Percentage of lapped bars in one section

5.2.3 Transverse reinforcement in the lap zone (Eurocode 2)

According to Eurocode 2 transverse reinforcement is required in the lap zone.

- Where the diameter, $\varnothing$, of the lapped bars is less than 20 mm, or the percentage of lapped bars in any one section is less than 25%, then no extra transverse reinforcement is required.

- Where the diameter, $\varnothing$, of the lapped bars is $\geq$20 mm, the transverse reinforcement within the lap length should have a total area, A_{st}, of not less than the area A_s of one spliced bar. It should be placed perpendicular to the direction of the lapped reinforcement and between that and the surface of the concrete.

- If more than 50% of the reinforcement is lapped at one point and the distance, *a*, between adjacent laps at a section is $\leq 10\varnothing$ (figure 10) transverse bars should be formed by links or U bars and anchored into the body of the section. /8./

The transverse reinforcement should be positioned at the outer sections of the lap as shown in figure 12a.

Transverse reinforcement for bars permanently in compression

In addition to the rules for bars in tension one bar of the transverse reinforcement should be placed outside each end of the lap length and within $4\varnothing$ of the ends of the lap length (figure 12b). /8./

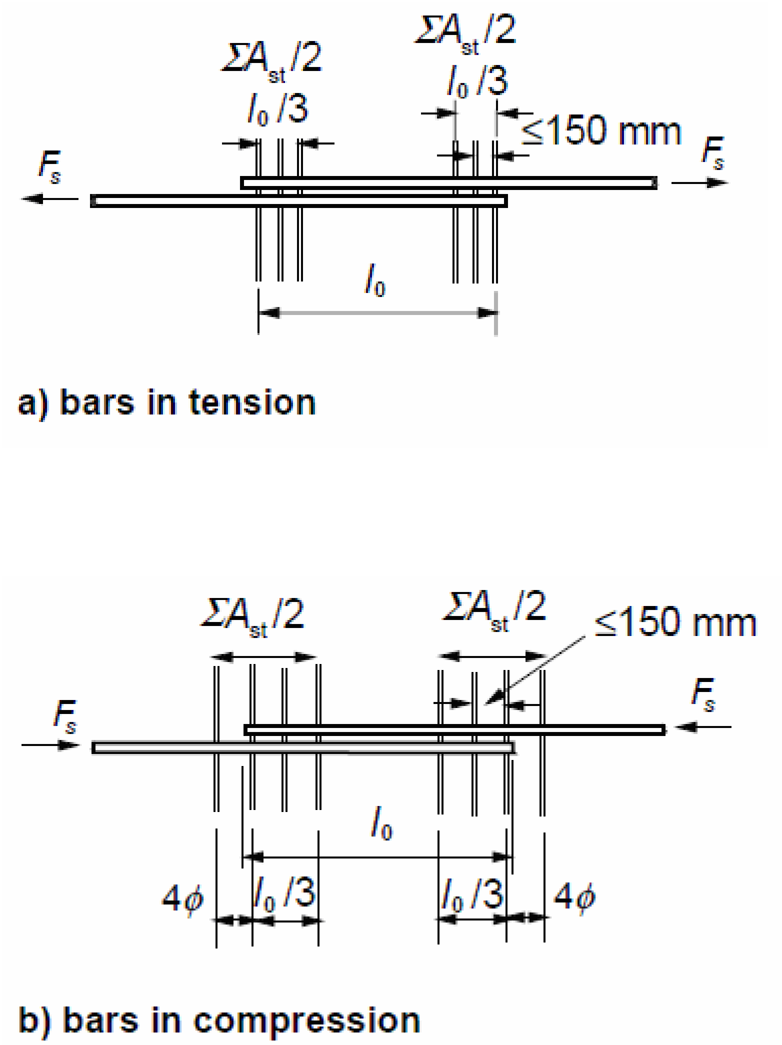


Figure 14: Transverse reinforcement for lapped splices

5.3 Detailing of reinforcement according to Eurocode 2

5.3.1 Beams

5.3.1.1 Longitudinal reinforcement

The minimum required area of longitudinal tension reinforcement is:

$$A_{s,\min} = 0.26 \frac{f_{ctm}}{f_{yk}} b_t d \quad \text{, but not less than } 0.0013 b_t d \tag{32}$$

Where:

b_t denotes the mean width of the tension zone; for a T-beam only the width of the web is taken into account

f_{ctm} is determined with respect to strength class. /8./

Alternatively, for secondary elements, where some risk of brittle failure may be accepted, $A_{s,\min}$ may be taken as 1,2 times the area required in ULS verification.

Sections containing less reinforcement than that given by Expression (32) should be considered as unreinforced.
The cross-sectional areas of the tension reinforcement or the compression reinforcement is not limited.

5.3.1.2 Anchorage of bottom reinforcement at intermediate supports

The anchorage length should not be less than 10$\varnothing$ (for straight bars) or not less than the diameter of the mandrel for bends and hooks. /8./

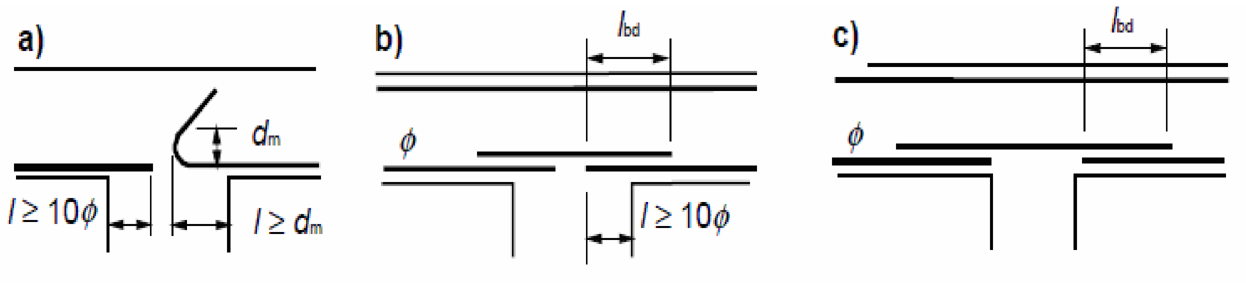


Figure 15: Anchorage at intermediate supports

5.3.1.3 Surface reinforcement (Eurocode 2)

It may be necessary to provide surface reinforcement either to control cracking or to ensure adequate resistance to spalling of the cover.

Surface reinforcement to resist spalling should be used where:

- bars with diameter greater than 32 mm or
- bundled bars with equivalent diameter greater than 32 mm. /8./

The surface reinforcement should consist of wire mesh or small diameter bars, and be placed outside the links as indicated in figure 16.

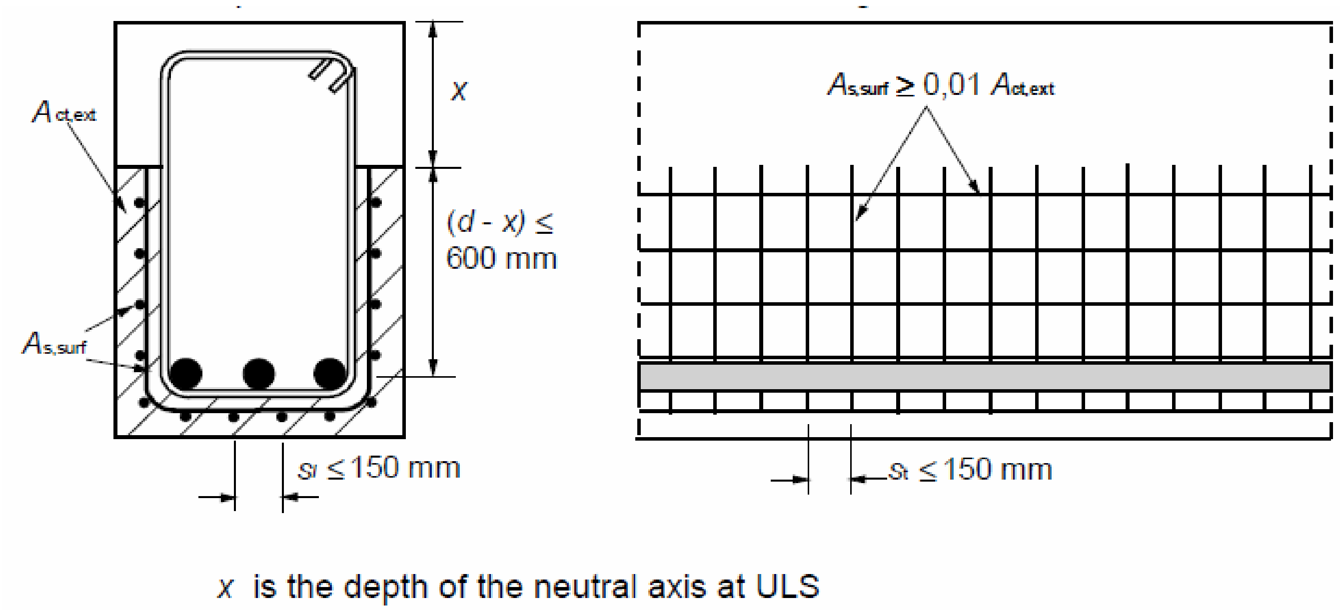


Figure 16. Example of surface reinforcement

The area of surface reinforcement $A_{s,surf}$ should be not less than $0.01A_{ct,ext}$ in the two directions parallel and orthogonal to the tension reinforcement in the beam. /.8/

$A_{ct,ext}$ is the area of the tensile concrete external to the links (see figure 16).

Where the cover to reinforcement is greater than 70 mm, for enhanced durability similar surface reinforcement should be used, with an area of $0.005A_{ct,ext}$ in each direction. /8./

5.3.1.4 Transverse (shear) reinforcement (Eurocode 2)

The shear reinforcement should form an angle of between 45° and 90° to the longitudinal axis of the structural element.
The ratio of shear reinforcement is:

$$\rho_w = \frac{A_{sw}}{s \cdot b_w \cdot \sin\alpha} \tag{33}$$

where:

A_{sw} is the area of shear reinforcement within length s

s is the spacing of the shear reinforcement measured along the longitudinal axis of the structural member

b_w is the width of cross-section or web for T-section

α is the angle between the shear reinforcement and the longitudinal axis, $\sin\alpha = 1$ for usual position of 90°. /8./

In any case ρ_w should be not less than the following value:

$$\rho_w = \frac{0.08\sqrt{f_{ck}}}{f_{yk}} \tag{34}$$

Any compression longitudinal reinforcement (diameter $\varnothing$) which is included in the resistance calculation should be held by transverse reinforcement with spacing not greater than 15$\varnothing$. /8./

5.3.2 Slabs

Regulations for the minimum and the maximum steel percentages are the same as for beams. However the minimum tensile reinforcement in slabs need not be more than 1,5 times the area required for the ultimate limit state.

The spacing of bars should not exceed $s_{\max}$:

- 400 mm or 3h for the principal reinforcement;
- 450 mm or 4h for the secondary reinforcement, where h is the depth of the slab. /8./

In areas with concentrated loads or areas of maximum moment those provisions become respectively:

- 250 mm or 2h for the principal reinforcement;
- 400 mm or 3h for the secondary reinforcement,

where h is the depth of the slab. /8./

5.3.3 Columns (Eurocode 2)

This clause deals with columns for which the larger dimension *h* is not greater than 4 times the smaller dimension *b*.

5.3.3.1 Longitudinal reinforcement

Bars should have a diameter of not less than 8 mm.

The minimum amount of total longitudinal reinforcement $A_{s,\min}$ should be derived from the following condition:

$$A_{s,\min} = 0.1\frac{N_{Ed}}{f_{yd}} \quad \text{or} \quad 0.002A_c \text{, whichever is greater} \qquad (35)$$

where:

f_{yd} is the design yield strength of the reinforcement

N_{Ed} is the design axial compression force. /8./

The area of reinforcement should not exceed $A_{s,\max} = 0.06A_c$. This limit should be increased to $0.12A_c$ at laps. /10./

5.3.3.2 Transverse reinforcement

The diameter of the transverse reinforcement (links, loops or helical spiral reinforcement) should not be less than:

- 6 mm
- one quarter of the maximum diameter of the longitudinal bars.

The diameter of the wires of welded mesh fabric for transverse reinforcement should not be less than 5 mm.

The spacing of the transverse reinforcement along the column should not exceed the lesser of the following three distances:

- 20 times the minimum diameter of the longitudinal bars
- the lesser dimension of the column
- 400 mm. /8./

The maximum spacing required above should be reduced by a factor 0.6:

- in sections within a distance equal to the larger dimension of the column cross-section above or below a beam or slab;
- near lapped joints, if the maximum diameter of the longitudinal bars is greater than 14 mm. A minimum of 3 bars evenly placed in the lap length is required. /8./

Every longitudinal bar or bundled bars placed in a corner should be held by transverse reinforcement. No bar within a compression zone should be further than 150 mm from a restrained bar. /8./

5.3.4 Walls (Eurocode 2)

This clause refers to reinforced concrete walls with a length to thickness ratio of 4 or more and in which the reinforcement is taken into account in the strength analysis.

The area of the vertical reinforcement should be between 0,002 A_c and 0,06 A_c outside lap locations. The limits may be doubled at laps.
The distance between two adjacent vertical bars shall not exceed

- 3 times the wall thickness
- 400 mm. /8./

5.3.4.1 Horizontal reinforcement

Horizontal reinforcement should be provided at each surface. It should not be less than:

- 25% of the vertical reinforcement
- $0.001A_c$

The spacing between two adjacent horizontal bars should not be greater than 400 mm. /8./

5.3.4.2 Transverse reinforcement

In any part of a wall where the total area of the vertical reinforcement in the two faces exceeds $0.02A_c$, transverse reinforcement in the form of links should be provided in accordance with the requirements for columns.

Except for welded wire mesh and bars of diameter $\varnothing \leq 16$ mm used with concrete cover larger than $2\varnothing$ transverse reinforcement should also be provided in the form of links at a number of 4 per square meter of wall area. /8./

5.3.5 Bored piles (Eurocode 2)

Bored piles with diameters not exceeding 600 mm should be provided with the minimum longitudinal reinforcement area given in table 33, which should be distributed along the periphery of the section. /8./

Table 33. Minimum reinforcement area in cast-in place bored piles

Pile cross-section, A_c	Minimum area of longitudinal reinforcement, A_s
$A_c \le 0.5m^2$	$A_s \ge 0.005 \cdot A_c$
$0.5m^2 < A_c \le 1.0m^2$	$A_s \ge 2500mm^2$
$A_c > 1.0m^2$	$A_s \ge 0.0025 \cdot A_c$

The minimum diameter for the longitudinal bars should not be less than 16mm. Piles should have at least 6 longitudinal bars. The clear distance between bars should not exceed 200 mm measured along the periphery of the pile.

5.4 Regulations for reinforcement according to RakMK

5.4.1 Anchorage of reinforcement

The anchorage capacity of a straight bar is the following:

$$F_{bu} = k_b \cdot f_{ctd} \cdot u_s \cdot l_b \tag{35}$$

And it should be sufficient for anchoring the force applied to steel:

$$F_{bu} \geq \sigma_s \cdot A_s \tag{36}$$

Therefore the design anchorage length of a straight bar should be the following:

$$l_b \geq \frac{\sigma_s A_s}{k_b \cdot f_{ctd} \cdot u_s}, \tag{37}$$

where:

u_s is the circumference (perimeter of cross section) of the bar,

k_b is the bond coefficient, the values for which are given in table 33,

σ_s is the steel stress equivalent to the design load at the ULS,

f_{ctd} is the design value of concrete tensile strength. /12./

In any case the anchorage length must be not less than 10d.

The anchorage capacity of compression bar may be increased by $3A_s f_{cd}$ if the minimum distance of the bar end from the concrete surface in the direction of the bar is 5d. /12./

The design value of anchorage length will be:

$$l_b \geq \frac{\sigma_s}{3 f_{cd} \cdot k_b \cdot f_{ctd} \cdot u_s} \quad , \tag{38}$$

where: f_{cd} is the design concrete compressive strength. /12./

Table 33. Bond coefficient k_b

Bond condition	A500HW A700HW B500K B600KX B700K	Round bar S235JRG2
1. The angle between the bar and the horizontal plane 45 or the distance of the reinforcement from the lower surface of the structure is no more than 300 mm	2,4	1,0
2. The distance of the reinforcement from the lower surface exceeds 300 mm or structure with cracking in the anchorage zone	1,7	0,7

Bond coefficients may be increased by 50% for structures with significant transverse compression in the anchorage zone. /12./

5.4.2 Splices of bars (RakMK)

According to RakMK the lap length in a straight bar in tension or compression shall be calculated from the following expression:

$$l_j = 0.25 k_j \frac{f_{yd}}{k_b \cdot f_{ctd}} \cdot \varnothing \quad , \tag{39}$$

where :

k_j is a coefficient depending on the number of bars lapped in the same cross-section from table 34,

k_b shall be selected from table 33.

The lap length for compression bars can be calculated from the following:

$$l_j = 0.25 \cdot 1 \cdot \frac{f_{yd} - 3f_{cd}}{k_b \cdot f_{ctd}} \cdot \varnothing, \tag{40}$$

In that case $k_j = 1$. /12./

Table 34. Values of the coefficient k_j

Proportion of bars to be spliced in the same cross-section relative to the total amount of reinforcement	k_j	
	a	b
≤1/5	1,0	1,2
1/3	1,2	1,6
1/2	1,3	1,8
>1/2	1,5	2,0

Values in column (a) may be used:

- If the clear distance between two adjacent laps is no less than $10\varnothing$
- If the nominal concrete cover at the point of splicing is no less than $5\varnothing$ in lateral direction, or if the splice is located at the corner of a stirrup.

Splices are considered to be in the same cross-section if their centre-to-centre spacing falls below $l_j + 20\varnothing$. /12./

5.5 Detailing of reinforcement according to RakMK

5.5.1 Slabs

The amount of reinforcement at the maximum moments in spans and at supports of cantilevers shall be no less than:

$$A_s = 0.25 \frac{f_{ctk}}{f_{yk}} \cdot A_c \tag{41}$$

With regard to reinforcement made of grade B500K or B700K steel, with bar thickness less than 10mm, the amount of steel shall be at least 1.5 times the amount derived from Expression (41). /12./

The spacing of bars at the maximum moments should be no more than:

- 3 times the depth of the slab
- 400 mm

And not less than 150 mm. /12./

The maximum spacing of bars within edge zones[2] of slabs may be 4 times the depth of the slab, but not more than 600 mm.

A minimum of 30% of the span reinforcement shall continue up to the supports. /12./

The instructions provided for beams shall apply to the positioning of shear reinforcement in shear-reinforced slabs.

[2] The edge zone of the slab is the area next to its supported edge, with a width of no more than 25% of the smaller side dimension of the slab.

5.5.2 Beams (RakMK)

The amount of main reinforcement at the maximum moments in spans or at supports of cantilevers shall be not less than:

$$A_s = 0.5 \frac{f_{ctk}}{f_{yk}} \cdot A_c \tag{42}$$

The spacing of reinforcement bars at the maximum span moments and at continuous and fixed supports shall be not more than 300 mm.
The bar diameter shall be at least 8 mm. /12./

A minimum of 30% of the span reinforcement shall continue up to the supports, however, not less than two bars if the beam width is more than 120 mm.

For deep beams ($\frac{L}{d} < 3$) in bending, the whole span reinforcement should be anchored at supports. /12./

5.5.2.1 Shear reinforcement

Should be installed when the capacity of the concrete is not enough to withstand shear forces.
The ratio of the shear reinforcement to the area of the horizontal web cross-section shall be no less than:

$$\frac{A_{sv}}{A_c} = 0.2 \frac{f_{ctk}}{f_{yk}} \tag{43}$$

where A_c is the area of the horizontal web cross-section.

Longitudinal spacing of shear reinforcement should be not more than:

- $0.7d$ (d is the depth of cross-section)
- 400 mm.

Transverse spacing should be not more than:

- d
- 600 mm. /12./

For beams with a height of more than 800 mm and the required design amount of main reinforcement is $\geq 400mm^2$, a longitudinal reinforcement should be placed in both facades of the web tension areas, with a maximum spacing of 300 mm.

The proportion of the area of this reinforcement relative to the cross-sectional area of the web in tension shall, taking the areas of reinforcement of both surfaces together, be at least:

$$\frac{A_s}{A_c} = 0.12 \frac{f_{ctk}}{f_{yk}} \quad /12./ \qquad (44)$$

5.5.3 Columns (RakMK)

The proportion of the main reinforcement area relative to the area of concrete should be not less than:

$$\frac{A_s}{A_c} = 1.5 \frac{f_{ctk}}{f_{yk}} \qquad (45)$$

The reinforcement should be evenly distributed across the cross-section. There should be a reinforcing bar at least at every corner or bend of the column. Circular columns should be provided with a minimum of 6 reinforcing bars. /12./

The spacing of the main bars should be no more than:

- twice the smallest side dimension
- 300 mm

However, in columns with a maximum side dimension up to 480 mm, bars placed in the corners will be sufficient.

The diameter of the main bars should be not less than:

- 12 mm
- 8 mm for welded reinforcing assemblies
- 10 mm for one-storey frames with the height of the column $\leq 3m$

The total area of longitudinal reinforcement should not be more than $0.06A_c$. /12./

5.5.4 Walls (RakMK)

The minimum thickness of load-bearing walls should be considered as:

- 120 mm
- 80 mm for reinforced and unreinforced walls in a building with no more than two stories. /12./

Reinforcement should be provided for both wall surfaces in both horizontal and vertical direction and its proportion of the total area of concrete cross-section should be not less than:

$$\frac{A_s}{A_c} = 0.25\frac{f_{ctk}}{f_{yk}} \tag{46}$$

The spacing of both vertical and horizontal bars should be not more than 300 mm.

The diameter of the horizontal bars shall be at least 0.5 times of vertical bars and their spacing should not more than 30 times the diameter of the vertical bars. /12./

Summary

In this chapter were studied and compared the main principles regarding designing of reinforcement of concrete structures. These main principles are the requirements of how to reinforce structural elements, which type of reinforcement to use, how to place it within structural member, how to anchor reinforcement properly. There were not significant differences found, but some of the details are different.

The first thing was mentioned is that the required amount of reinforcement in structural members is different in all norms. For example the required amount of the main reinforcement for beams and columns is approximately two times greater in Finnish RakMK than in Eurocode and SNiP, in last two this value is almost the same.

For beams the applied amount of reinforcement is usually determined by ULS calculation and is more than the minimum required value, so this difference is not so significant. But for columns reinforcement is often applied based on these requirements and the different between them becomes more significant.

Interesting fact that the required amount of reinforcement for walls given in RakMK is almost two times smaller than the value given in SNiP and Eurocode.

One more interesting thing is that Eurocode and RakMK require to reduce spacing between transverse reinforcement in columns above or below a beam or a slab. There is no such requirement in Russian norms.
The rest of the issues are without significant differences.

6 DURABILITY DESIGN

6.1 Durability design according to Eurocode

All values regarding concrete cover, limitation of cracks and other are derived from the National Annex to Eurocode 2.

In order to ensure normal working of structure throughout its intended service life the following steps of durability design should be done:

1) Definition of the design life and exposure class
2) Definition of quality parameters for the concrete composition, width of cracking, concrete cover to reinforcement and other factor that influence the service life.
3) Drawing up of other durability instructions and other instructions related to further use of the structure

One of the most important factors for provision of normal service life of reinforced concrete structure is a proper corrosion protection of steel. It depends on quality and thickness of concrete cover and control of cracking. The minimum cover layer depends on the environmental conditions to which the structure is exposed.

6.1.1 Environmental exposure classes

The designer must determine the type of stress or load to which the structure is exposed and select it from the following stress factors:

- corrosion caused by carbonation
- corrosion caused by chlorides
- corrosion caused by chlorides in sea water
- Freeze-thaw stress
- chemical load

The exposure class is selected according to the description in table 35. /8./

The structure may be simultaneously classified under several exposure classes. For instance, facades are classified under class XF1 due to freeze-thaw stress, and under classes XC3 and XC4 in terms of corrosion caused by carbonation. Figure 3.1 shows principles of exposure classification for structures.

Table 35. Environmental exposure classes

Class	Description of the environment	Informative examples where exposure class may occur
1. No risk of corrosion or chemical attack		
X0	For concrete without reinforcement or embedded metal: all exposures except where there is freeze/thaw, abrasion or chemical attack. For reinforced concrete: very dry.	Concrete inside buildings with very low air humidity. Dry, heated indoor spaces.
2. Corrosion caused by carbonation		
XC1	Dry or permanent wet	Indoor spaces with a low moisture content. Structures permanently submerged in water.
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact. Most foundations.
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity. Outdoor structures protected from direct rain. Saunas, industrial kitchens, many industrial buildings. Rain-protected bridge superstructures.

XC4	Periodical wet and dry	Concrete surface in contact with water, not within class XC2. Balcony slabs, facades exposed to rain. Bridge structures exposed to rain.
3. Corrosion caused by chlorides		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides. Noise walls on road sides.
XD2	Wet, rarely dry	Concrete exposed to industrial waters containing chlorides. Swimming pools.
XD3	Periodical wet and dry	Parts exposed to salty splashing or salting. Pavements, car park slabs, heated garages.
4. Corrosion caused by chlorides from sea water		
XS1	Exposed to airborne salt but not in direct contact with sea water	Structures near to or on the coast
XS2	Permanently under water	Sea water structures, bridge structures under sea water level
XS3	Splash and spray zones	Parts of sea water and bridge structures, for example intermediate bridge supports.
5. Freeze-thaw stress		
XF1	Moderate water saturation without de-icing agents	Vertical concrete surfaces exposed to rain and freezing. Facades, footings.
XF2	Moderate water saturation with de-icing agents	Vertical concrete surfaces of road structures exposed to freezing and airborne de-icing agents
XF3	High saturation without de-icing agents	Horizontal surfaces exposed to rain and freezing. Balconies, fresh water basins.

XF4	High saturation with de-icing agents	Horizontal concrete surfaces exposed to direct de-icing agent splashing and freezing. Road and bridge decks. Splash zones of marine structures exposed to freezing.
6. Chemical attack		
XA1	Slightly chemically aggressive environment	Natural soils and ground water. Some agricultural structures
XA2	Moderately chemically aggressive environment	Natural soils and ground water. Wood drying plants, top parts of chimneys.
XA3	Highly chemically aggressive environment	Natural soils and ground water. Agricultural structures exposed to urea or fertilizers.

Note: The level of chemical aggressiveness is determined by amount of chemical agents in ambient environment. This table is represented in Eurocode2.

6.1.2 Concrete cover (Eurocode 2)

After definition of the environmental class the required concrete cover may be determined. The value of the nominal cover layer, which is specified on the drawings is defined as a minimum cover, $c_{\min}$, plus an allowance in design for tolerance, Δc:

$$c_{nom} = c_{\min} + \Delta c \quad /8./$$

Minimum concrete cover, $c_{\min}$, is provided in order to ensure:

- safe transmission of bond forces
- protection of steel against corrosion and fire

In order to transmit bond forces safely and to ensure adequate compaction, the minimum cover should not be less than the values for c_{min} given in table 36. /8./

Table 36. Minimum cover requirements with regard to bond.

Bond Requirement	
Type of steel	Minimum cover c_{min} [1]
Ordinary	Diameter of bar
Bundled	Equivalent diameter (ϕ_n)
Post-tensioned	Circular duct for bonded tendons: diameter of the duct. Rectangular duct for bonded tendons : lesser dimension or 1/2 greater dimension but not less than 50 mm. There is no requirement for more than 80 mm for either type of duct.
Pre-tensioned	2,0 x diameter of strand or wire 3,0 x diameter of indented wire
Note 1: If the nominal maximum aggregate size is greater than 32 mm, c_{min} should be increased by 5 mm to allow for compaction.	

In order to provide an adequate protection of steel against corrosion the minimum concrete cover should be not less than the values from table 37.

Table 37. Minimum cover requirements with regard to durability

Exposure class	Minimum cover for a service life of 50 years, mm		Minimum cover for a service life of 100 years, mm		Minimum concrete strength class (equivalent K class)
	Reinforcing steel	Prestressing steel	Reinforcing steel	Prestressing steel	
X0	10	10	10	10	C20/25 (K25)
XC1	10	20	10	20	C30/37 (K35)
XC2, XC3	20	30	25	35	C35/45 (K45)
XC4	25	35	30	40	C35/45 (K45)
XD1	30	40	35	45	C35/45 (K45)
XS1	30	40	35	45	C40/50 (K45)
XD2	35	45	40	50	C35/45 (K45)
XD3, XS2, XS3	40	50	45	55	C45/55 (K55)

In the case when the concrete strength class is higher than required above the cover layer may be reduced by 5 mm.
For Structural Class 1 concrete cover may also be reduced by 5 mm. /8./

6.1.3 Allowance in design for tolerance (Eurocode 2)

The required minimum cover should be increased by the accepted negative deviation. The allowed deviation, Δc , is normally 10 mm. A producer of precast elements may adopt a tolerance below 10 mm for different types of products, however, any tolerance below 5 mm may not be adopted.

For concrete cast against uneven surfaces, the minimum cover should be increased by allowing larger deviations in design. The cover should be at least $c_{nom,1} = c_{\min} + 10mm$ for concrete cast against prepared ground and $c_{nom,2} = c_{\min} + (20...40)mm$ 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. /8./
The value c_{nom} is applied as a final value for the design cover layer and specified on drawings.

6.1.4 Cracking

The second important thing in durability design is crack control. In the case when cracks have width exceeding the maximum permissible value appears a risk of corrosion damage of reinforcing steel within structural member. Cracking is also limited in order not to make appearance of the structure unacceptable.
The limitations of crack width $w_{\max}$ for relevant exposure classes are given in table 38.

Table 38. Limitations of crack width

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

Note1: 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.

Note2: For these exposure classes, in addition, decompression should be checked under the quasi-permanent combination of loads. /8./

Note3:

- Quasi-permanent combination of actions: The combination of permanent and variable loads which is most likely to be present most of the time during the design working life of the structure.
- Frequent combination of actions: The most likely highest combination of permanent and variable loads which is likely to occur during the design working life of the structure.

6.2 Durability design according to RakMK

The main principles of durability design according to Finnish norms are based on Eurocode, however there are some differences in details. The idea about designing structure according to environmental conditions and environmental exposure classes are the same.

Table 39. Minimum strength grade according to exposure classes /12./

Exposure class	Minimum strength grade (design live 50 years)	Minimum strength grade (design live 100 years)
X0	K15	K15
XC1	K25	K25
XC2	K30	K35
XC3	K30	K40
XC4	K35	K45
XS1	K40	K40
XS2,XS3	K45	K45
XD1,XD2	K35	K35
XD3	K45	K45
XF1-XF4	-	-
XA1	K40	K40
XA2	K45	K45
XA3	K50	K50

6.2.1 Concrete cover to reinforcement (RakMK)

As in Eurocode the design concrete cover which is specified on the drawings is calculated as minimum cover plus an allowed deviation, which is usually applied as 10 mm. The values for minimum cover are specified in table 40 depending on exposure class.

Table 40. Minimum concrete cover

Exposure class	Minimum cover for a service life of 50 years, mm		Minimum cover for a service life of 100 years, mm	
	Reinforcement sensitive to corrosion	Other reinforcement	Reinforcement sensitive to corrosion	Other reinforcement
X0	10	10	10	10
XC1	20	10	20	10
XC2	30	20	35	25
XC3, XC4	35	25	40	30
XS1, XD1	40	30	45	35
XS2, XD2	45	35	50	40
XS3, XD3	50	40	55	45

Note: The reinforcement is considered to be sensitive to corrosion when the diameter is 4 mm or less or when cold-formed steels are subjected to long-term tension with a stress exceeding 400MPa. /12./

6.2.2 Crack control (RakMK)

The requirements for cracking of structure with a design service life of 50 years are represented in table 41.
For some exposure classes appearing of cracks is restricted: tensile stresses in concrete should not occur at all.

Points a) and b):

- a - requirements for long-term loads (corresponds to quasi-permanent load combination from Eurocode)
- b – requirements for short-term loads (corresponds to frequent load combination fro Eurocode)

Table 41. Limitation of crack width

Exposure class	Reinforcement sensitive to corrosion	Other reinforcement
X0, XC1	a) $w_k \leq 0.2mm$ b) $w_k \leq 0.3mm$	No regulations
XC2, XC3, XC4 XS1, XD1 XF1, XF2, XF3 XA1, XA2	a) Limit state of tensile stress b) $w_k \leq 0.1mm$	a) $w_k \leq 0.2mm$ b) $w_k \leq 0.3mm$
XS2, XS3 XD2, XD3 XF4 XA3	a) Limit state of tensile stress b) Limit state of tensile stress	a) $w_k \leq 0.1mm$ b) $w_k \leq 0.2mm$

Note: Limit state of tensile stress is a state, at which no tensile stresses occur in concrete. /12./

For different periods of working life there are different requirements for crack width. In the following table requirements for working life of 100 years, w_{k100}, are represented according to requirements for 50-years service life. /12./

Table 42. Limitations of crack width for working life of 100 years

Concrete cover c, mm	w_{k100}, where requirement for working life of 50 year life is $w_{k50} = 0.1mm$	w_{k100}, where requirement for working life of 50 year life is $w_{k50} = 0.2mm$	w_{k100}, where requirement for working life of 50 year life is $w_{k50} = 0.3mm$
50	0,07	0,14	0,21
40	0,07	0,14	0,20
35	0,07	0,14	0,20
30	0,07	0,13	0,19
25	0,07	0,12	0,17
20	0,60	0,11	0,15

If the minimum concrete cover (nominal value- permitted deviation) is more than that required for the exposure class and working life, the required crack may be multiplied by the following factor:

$$\frac{c_{act}}{c_{\min}}, \quad \text{but} \leq 1.5$$

where:

c_{act} is an actual value of cover, used in crack verifications,

$c_{\min}$ is the minimum cover required for the exposure class. /12./

6.3 Durability design according to Russian norms

The idea of durability design in Russian norms for reinforced concrete is different from mentioned above. There is not such an exact description of environmental exposure classes like in European norms.

However, there is a special Building regulations for protection of structures against corrosion under chemically aggressive environmental conditions.

It is called SNiP 2.03.11-85 “Protection of structures against corrosion” and includes detailed rules of how to protect structures and structural members from different types of corrosion under different conditions. These norms are used only when special requirements are applied.
If no special requirements are applied then simple general rules are used.

6.3.1 Concrete cover (Russian norms)

Values of concrete cover represented in Building code SP 52-101-2003 are the design value which are specified on the drawings. The allowed deviation from this value is determined by tolerance and can be found in SNiP 3.03.01-87. The following values are related to main steel.

Table 43. Minimum values for concrete cover

Environmental conditions	Minimum concrete cover, mm
Concrete inside buildings with low and moderate air humidity	20
Concrete inside buildings with high air humidity	25
Concrete outside buildings (without special protection against corrosion)	30
Concrete in ground, foundations cast against prepared ground	40
Foundations cast directly against soil	70

- If there are special protective measures undertaken, the value of cover may be decreased.
- For structures made on factory the minimum value of cover is reduced by 5 mm.
- For helping steel concrete cover may be reduced by 5 mm.

- In any case the thickness of cover should be not less than the diameter of reinforcement and not less than 10 mm. /5./

6.3.2 Limitation of cracks (Russian norms)

Calculation of crack widths is necessary when limit state verification shows that cracks will appear.
Crack widths are calculated for “long acting loading” (corresponds to the long-term loading in RakMK) and for “not long acting” loading (short-term loading from RakMK).
Building regulations SNiP 52-01-2003 /1./ contain general requirements for crack limitation for ordinary structures. But if structure is going to be used in aggressive environmental conditions the allowed cracking is determined according to SNiP 2.03.11-85 “Protection against corrosion”. /4./ There limitation of cracking depends on reinforcing steel grade and aggressiveness of the environment.

The biggest allowed crack widths are represented in the following table. Values in a) and b) refer to cracks due to quasi-permanent load combination and frequent load combination respectively. /4;5./

Table 44. Limitation of crack width

Environmental conditions	Crack width, mm
Normal environment SNiP 52-01-2003	a) $w_k \leq 0.3$ b) $w_k \leq 0.4$ If there are special requirements to impermeability of structure: a) $w_k \leq 0.2$ b) $w_k \leq 0.3$
Aggressive environment SNiP 2.03.11-85	Depending on steel grade and type of environment the biggest allowed crack width may vary: a) $w_{k,ult} = 0....0.2$ b) $w_{k,ult} = 0....0.25$

Summary

In this chapter were studied basic requirements for ensuring durability of structural members and their protection from environmental impact- concrete cover and limitations of cracking.
Method for durability design according to Eurocode and Finnish norms RakMK is the same, but values for designing parameters are different, but not very much.
One interesting thing was mentioned is that values of required concrete cover in original text of Eurocode and in Finnish National Annex differ quite a lot.

Table 45. Difference between original Eurocode and Finnish National Annex

Environmental class	Minimum value of concrete cover to reinforcing steel, $c_{\min}$, $[mm]$		Minimum value of concrete cover to prestressing steel, $c_{\min}$, $[mm]$	
	Eurocode original	Finnish National Annex	Eurocode original	Finnish National Annex
XC1	15	10	25	20
XC2, XC3	25	20	35	30
XC4	30	25	40	35
XD1, XS1	45	30	55	40
XD2	45	35	55	45
XD3, XS2, XS3	45	40	55	50

The design value of concrete cover should be increased for concrete cast against ground and this increase is also different in two norms (table 46):

Table 46. Cover to concrete cast against ground

Cover to concrete cast:	Eurocode 2 original	Finnish National Annex
against prepared ground	$c_{\min} + 40mm$	$c_{\min} + 10mm$
directly against soil	$c_{\min} + 75mm$	$c_{\min} + (20...40)mm$

The comparison of required values for concrete cover from different norms is represented in table 47. The values from Eurocode and RakMK are almost the same.

It is hard to compare values required in Europe and Russia precisely because classification of environmental conditions is different.

Requirements according to Russian norms for European exposure classes are determined approximately and are represented in the following table. For environment without chemical loading SP 52-101-2003 was used, for chemically aggressive environment (classes XD1-XD3, XS1-XS3) SNiP 2.03.11-85 was used.

Table 47. Comparison of required concrete cover

Exposure class	Minimum concrete cover for normal reinforcing steel, mm		
	EN 1992 with National Annex	RakMK	SP 52-101-2003 SNiP 2.03.11-85
XC0, XC1	10	10	20
XC2	20	20	40
XC3	20	25	25
XC4	25	25	30
XD1, XS1	30	30	35
XD2	35	35	
XD3, XS3	40	40	35
XS2	40	35	

Note: In the table are given values of concrete cover which should be provided in the structural member.

The design value which is specified on the drawings according to Eurocode and RakMK is usually 10 mm more than the minimum value from the table. So European and Finnish norms assume that a deviation for the thickness of concrete cover may be -10 mm.

Tolerance assumed in Russia is given in Building regulations SNiP 3.03.01-87 and is not more than -5mm in any case. The required cover (given in the table) is the design value, which is specified on drawings and should be ensured with a maximum allowed tolerance of -5mm.

The values from Russian norms represented here are used for cast-in-situ structures, for prefabricated structures they may be reduced by 5 mm.

The maximum allowed crack widths are almost the same in all of three norms. For indoor structures with not high air humidity cracks are limited mostly by appearance of structure and have an ultimate width of 0,4 mm. In aggressive environment cracks are not allowed.

7 TOLERANCES

In this chapter tolerances which concern reinforced cast-in-situ concrete structures will be discussed. Tolerance is an allowable deviation from the design value of dimensions or of position of structural members. After structure is ready several taking over inspections are held. Those inspections check quality of the structure and compliance with the requirements for tolerances.

The requirements incorporate tolerances for position of main reinforcement within the cross-section, tolerances for dimensions of the cross-section, deviations from the design position of structural members and requirements for quality of cast-in-place surfaces.

7.1 Tolerances according to Russian norms

Implementation of the construction process and the following taking over expertise are held according to Building regulations SNiP 3.03.01-87 “Bearing and envelope structures”. /3./ This SNiP gives rules and requirement of how to carry out all types of works on the building site, it concerns foundations, steel structures and reinforced concrete structures both prefabricated and cast-in-situ.

7.1.1 Walls (SNiP 3.03.01-87)

Table 48. Tolerances for walls /3./

Measured parameter	Tolerance
Thickness	+6mm, -3mm
Length	±20 mm
Vertical deviation of wall (poikkeama pystysuorasta):	
for walls under cast-in-situ flooring	±15 mm
for walls under prefabricated elements	±10 mm
Horizontal deviation of the horizontal surfaces (for the whole length of wall)	±20 mm
Surface curvature or waviness (pinnan käyryys ja aaltoilu)- checking with 2m rail	5 mm
Difference between levels of two adjacent walls	3 mm
Level of the surface where prefabricated concrete elements will be based	-5 mm

The following picture represents the requirements given in the table above.

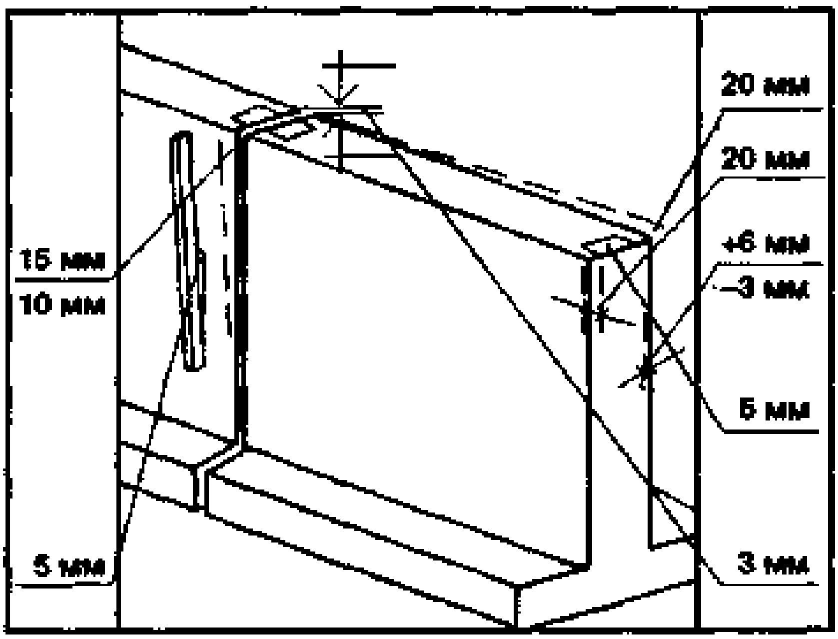


Figure 17. Tolerances for walls

7.1.2 Columns (SNiP 3.03.01-87)

Table 49. Tolerances for columns /3./

Measured parameter	Tolerance
Cross-sectional dimensions	+6 mm, -3 mm
Height	±20 mm
Vertical deviation of a column (poikkeama pystysuorasta):	
• for columns under cast-in-situ flooring	±15 mm
• for columns under prefabricated elements	±10 mm
Surface curvature or waviness (pinnan käyryys ja aaltoilu)- checking with 2m rail	5 mm
Level of the bearing surface where prefabricated elements will be based	-5 mm

Tolerances for columns are represented on Figure 18.

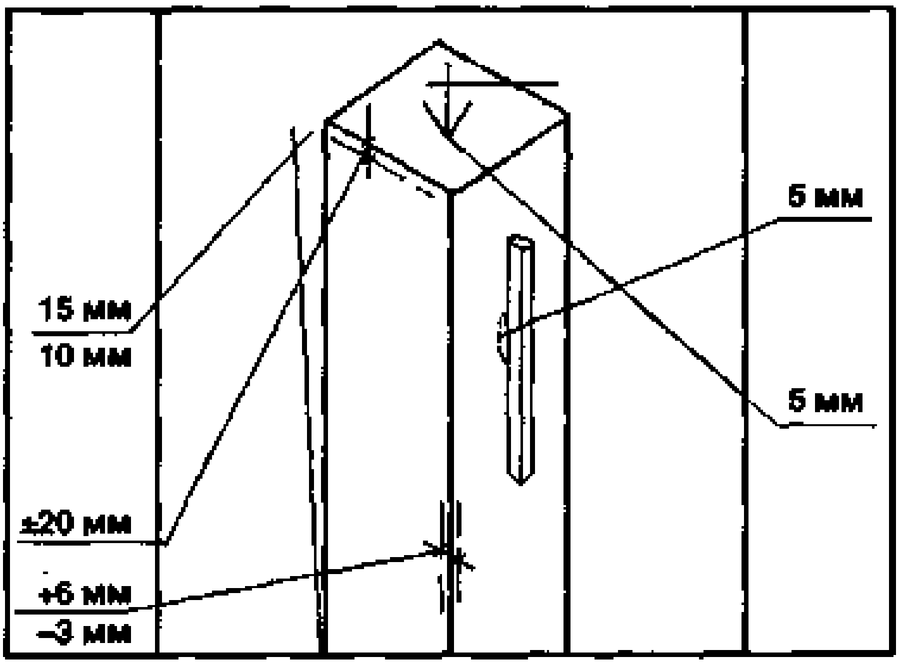


Figure 18. Tolerances for columns

7.1.3 Foundations (SNiP 3.03.01-87)

The main parameters that should be checked are the same as for other bearing structural members.

Table 50. Tolerances for foundations /3./

Measured parameter	Tolerance
Cross-sectional dimensions	+6 mm, -3 mm
Length	±20 mm
Vertical deviation (poikkeama pystysuorasta) for the whole height of foundation:	±20 mm
Difference in level of two adjacent foundations	3 mm
Maximum slope of the bearing surface, where steel column will be based	0,0007
Surface curvature or waviness (pinnan käyryys ja aaltoilu)- checking with 2m rail	5 mm
Level of the bearing surface where prefabricated elements will be based	-5 mm

For the graphical explanation see figure 19.

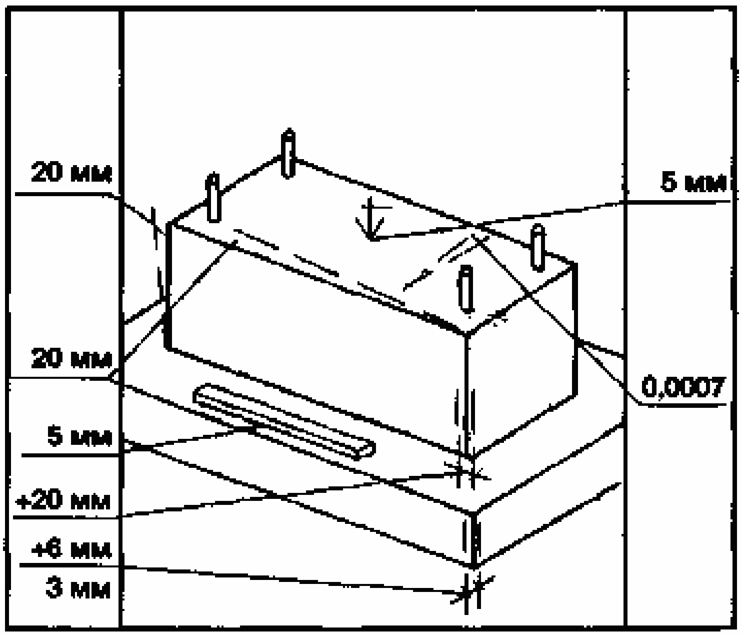


Figure 19. Tolerances for foundations

7.1.4 Reinforcement (SNiP 3.03.01-87)

The main tolerances concerning reinforce works are positioning of reinforcement within the cross-section and compliance of concrete cover. Tolerances for positioning of reinforcement include allowed deviations from the design values of distances between reinforcing bars.

Table 51. Tolerances for reinforcement /3./

Measured parameter	Tolerance
Distance between two single reinforcing bars for:	
• columns and beams	±10 mm
• foundations	±20 mm
• massive structures (e.g. dams)	±30 mm
Distance between rows of reinforcement for:	
• slabs and beams less than 1 m thick	±10 mm
• elements more than 1 m thick	±20 mm

Table is graphically represented on the following picture.

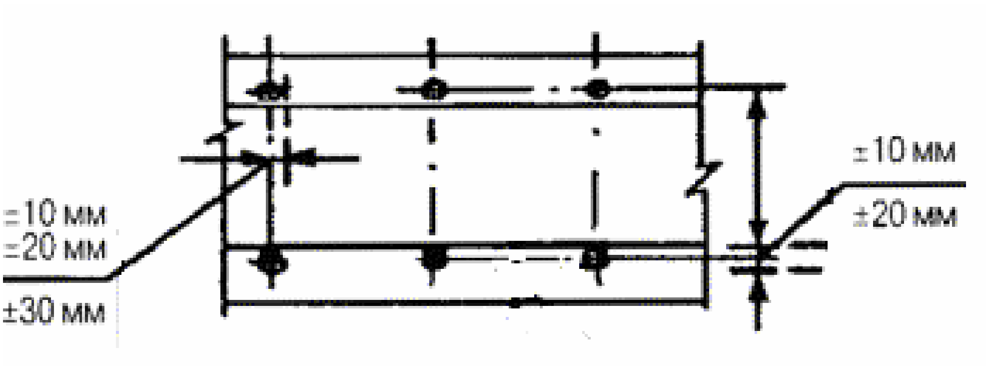


Figure 20. Tolerances for reinforcement

Tolerances for concrete cover are the allowed deviations from the design values of thickness of concrete cover.

Table 52. Tolerances for concrete cover. /3./

Deviation from the design value of thickness of concrete cover	Tolerance, mm
The design value of concrete cover is ≤15 mm and dimensions of cross-section of the structural member:	
≤100 mm	+4
101 mm….200 mm	+5
The design value of concrete cover is 16 mm…20 mm incl. and dimensions of the cross-section:	
≤100 mm	+4; -3
101 mm….200 mm	+8; -3
201 mm…300 mm	+10; -3
>300 mm	+15; -5
The design value of concrete cover is >20 mm and dimensions of the cross-section:	
≤100 mm	+4; -5
101 mm….200 mm	+8; -5
201 mm…300 mm	+10; -5
>300 mm	+15; -5

7.1.5 Quality of concrete surface

The requirements regarding the quality of concrete surface are given by National standard GOST 13015.0-83 “Prefabricated concrete and reinforced concrete constructions and products” /14./. These requirements concern only prefabricated structural members, not cast-in-situ concrete.

According to GOST prefabricated structures are divided into 7 classes regarding quality of concrete surfaces, from A1 to A7. The class should be specified in the project and information about in should be given to manufacturer.

As for cast-in-situ concrete, the only requirement concerning concrete surface is that local defects of the surface (humps or cavities) should be not more than 5mm in height or depth.

Table 53. Surface quality classes for prefabricated elements /14./

Concrete surface quality grade	The maximum linear dimension of cavity, mm	Height of local hump or depth of cavity, mm
A1	Not allowed (gloss surface)	
A2	1	1
A3	4	2
A4	10	1
A5	Not regulated	3
A6	15	5
A7	20	Not regulated

Different classes are assumed for different purposes, for example class A1 does not require finishing, because the surface is smooth, gloss and does not contain defects, class A3 can be covered with wallpapers without plastering, so humps should not be higher than 1 mm.

In order to obtain a surface with high quality it is necessary to use special devices, for example for casting elements with A1 class special plastic formwork is used.

7.2 Tolerances according to RakMK

Tolerances described here are taken from BY39 book. In some cases tolerances are divided into accuracy classes- normal and special class. Normal class is used for normal buildings for bearing and envelope structures, and special class is used when there are special demands for the quality and accuracy.

7.2.1 Foundations (BY 39)

Foundation walls are erected according to tolerances for walls.

Table 54.Tolerances for foundations /13./

Measured parameter	Tolerance, mm
Main dimensions (L, b)	±30
Level of the upper surface (K)	±20
Location of the side surface (S_n)	±30

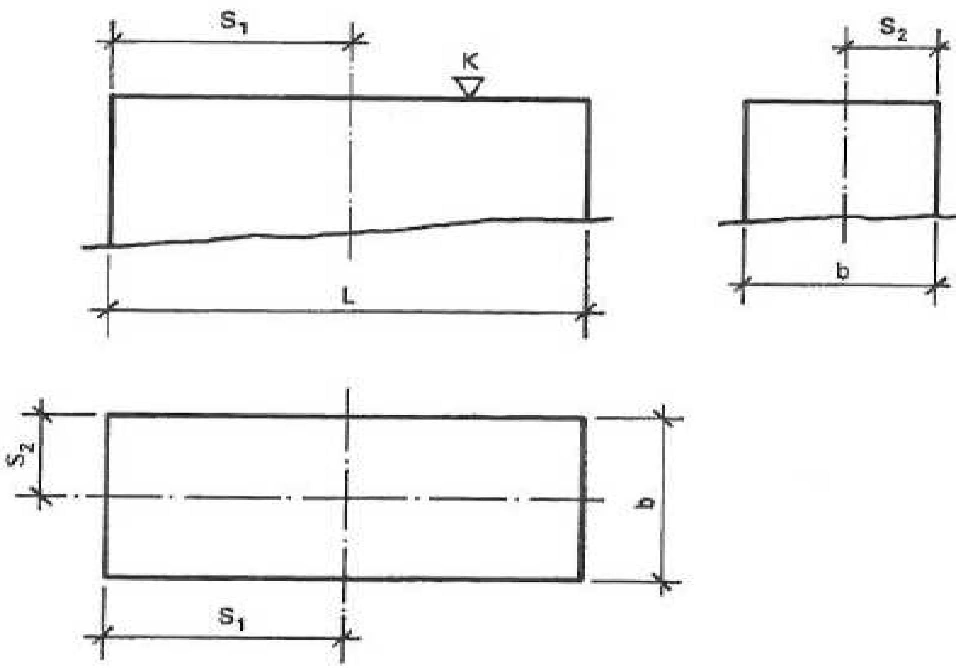


Figure 21. Measured parameters for foundations

7.2.2 Walls (BY 39)

Tolerances for walls are divided into three classes. Normal class is used for all bearing walls, for staircase walls and for partition walls in ordinary buildings. Special class is used when special demands to structures are given.

Table 55. Tolerances for walls /13./

Measured parameter	Tolerance		
	Foundation walls and slide-cast structures	Normal class	Special class
Height (H)	±15	±10	±8
Length (L)	±15 or L/350 [1)]	±10	±8 or L/500 [1)]
Thickness (b)	±10 [2)]	±8 [3)]	±5
Side curvature of wall (a)	±15	±10	±5
Curvature of openings (a1)	±8	±5	±5
Openings:			
- dimension of opening (h) and (I)	-5; +15	-5; +15	-5; +15
- dimension (e)	±20	±15	±10
- difference in position of corners │e1-e2│	15	10	10
Vertical deviation of the wall (p)	L/200	L/300	L/400
Location of the wall (S)	±20	±15	±10
Displacement of wall (s)	±15	±10	±5
Clear distance between walls (V)	±20	±15	±10
Level of the upper bearing surface of the wall (K)	±15	±10	±5

Note:

1) The biggest from two values is used.

2) For bearing wall with thickness below 200 mm tolerance is -5 , +10 $[mm]$.

3) For bearing wall with thickness below 200 mm tolerance is -5 , +8 $[mm]$.

/13./

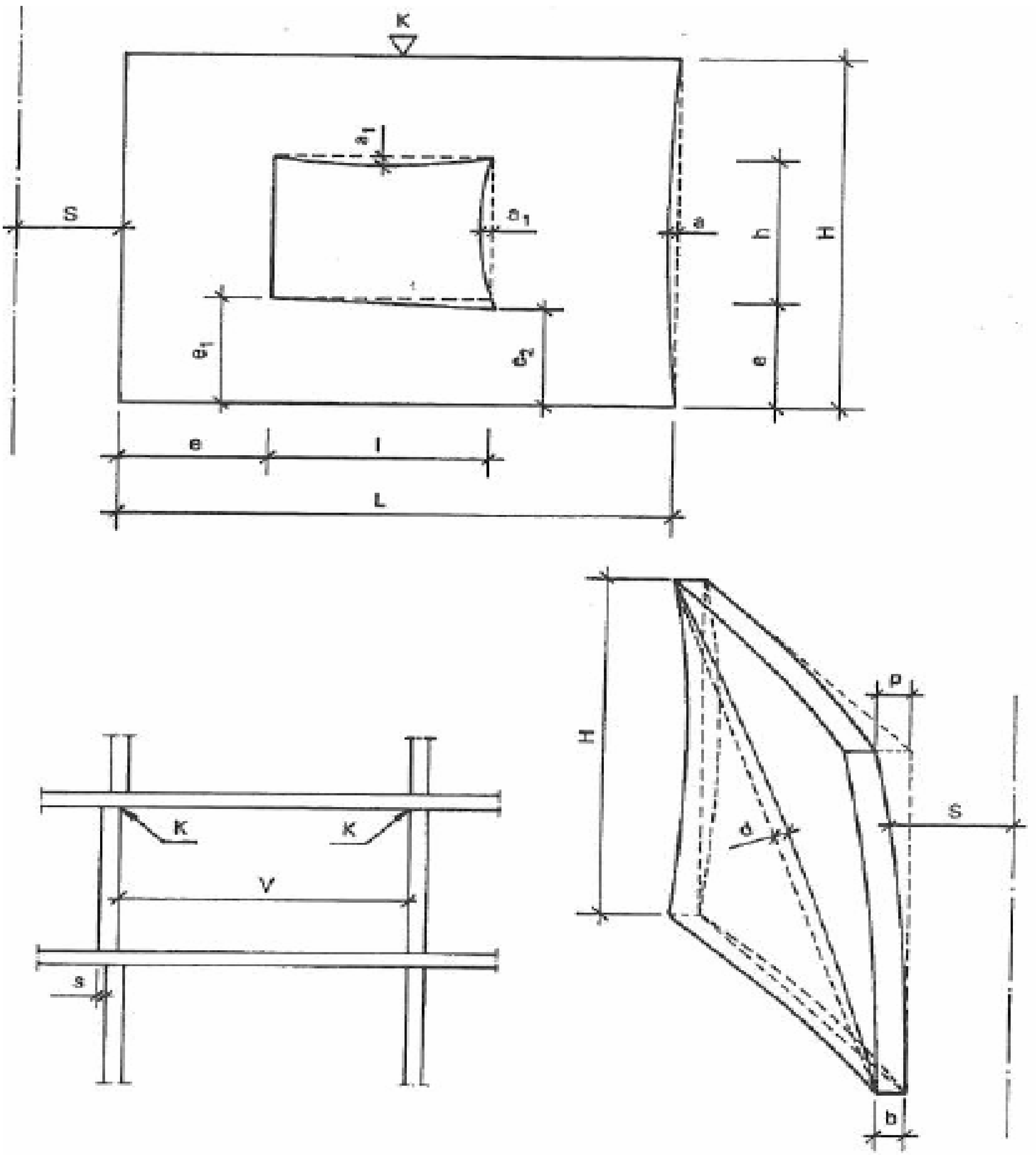


Figure 22. Measured parameters of walls

7.2.3 Columns (BY 39)

Special class is used when there are special demands to the structure and its appearance, in other cases normal class is used.

Table 56. Tolerances for columns /13./

Measured parameter	Tolerance, mm	
	Normal class	Special class
Length (L)	±15	±10
Cross-sectional dimensions (b, h, d)	±10 [1)]	±5
Curvature (a)	±10 or L/750 [2)]	±5 or L/1000 [2)]
Displacement of corners on the cross-section (p)	±5	
Displacement of the corner on a head of a column (r) [3)]	±5	±3
Vertical deviation of side surface (S)	±15	
Level of the upper bearing surface (K)	±15	
Clear distance between two columns (V)	±15	
Vertical deviation (poikkeama pystysuorasta) (P)	±15 or L/750 [2)]	±10 or L/1000 [2)]

Note:

[1)] If cross-sectional dimensions of the column are less than 200 mm, the tolerance will be -5, +10 $[mm]$.

[2)] From two values the biggest is used.

[3)] This concerns also the upper surface of a cantilever. /13./

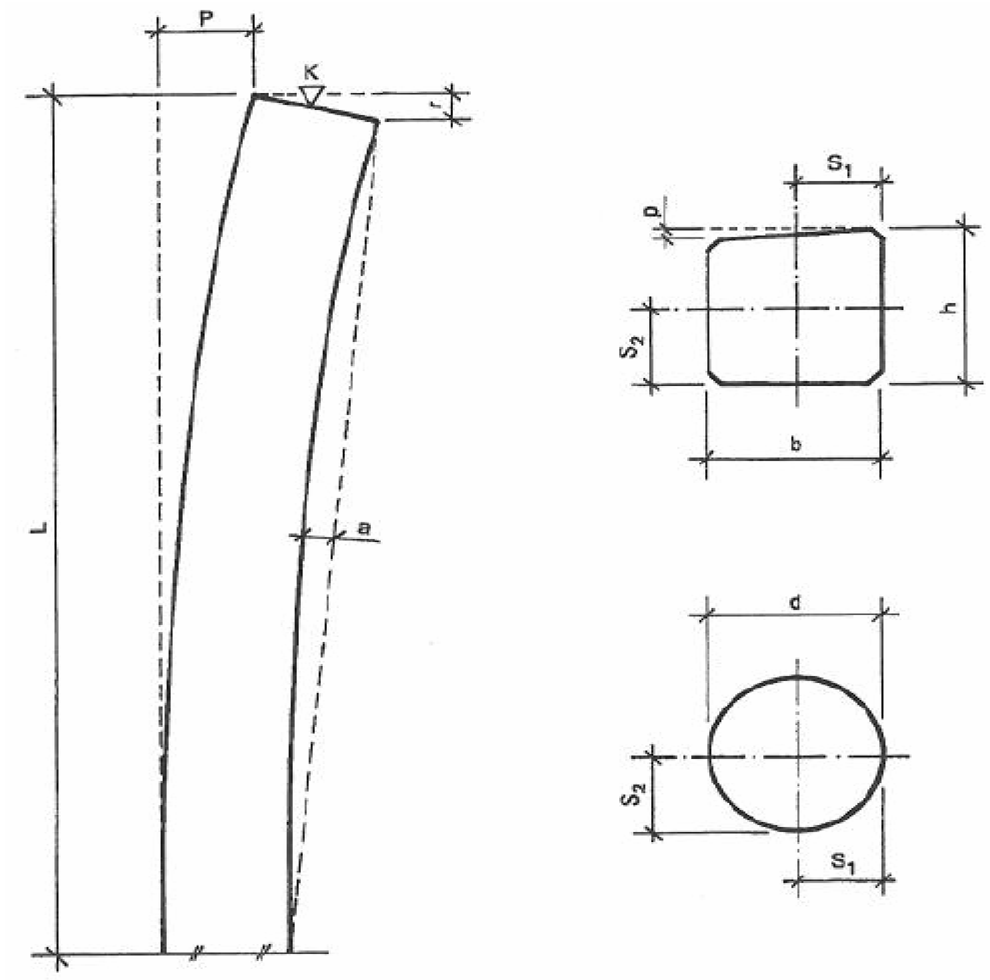


Figure 23. Measured parameters of columns

7.2.4 Reinforcement (BY 39)

Reinforcement should be designed so that to provide a concrete cover not less than 5mm an all directions. It can also be provided if the design length of a reinforcing bar is smaller than the length of a structural member- that does not let a bar too stick out of a cross-section. /12./

Tolerances for reinforcement describe allowed deviations both for length of reinforcing bars and for distance between them in cross-section and depend on the nominal length of a bar.

Table 57.Tolerances for reinforcement /13./

Measured parameters of reinforcement	Tolerance, mm	
	Normal class	Special class
Length of reinforcing bars:		
L < 500 mm	±10	±5
L = 500…1000 mm	±15	±10
L = 1000…2000 mm	±20	±15
L > 2000 mm	±30	±20
Anchoring, splice and starter length:		
Ø ≤ 16 mm	-20	
Ø > 16 mm	-40	

SUMMARY

The main conclusion appeared after studying this chapter is that most of the requirements for tolerances in Russian norms are more strict than in Finnish norms. RakMK allows to build structural elements with bigger tolerances for dimensions and position than Building regulations SNiP allow. So the structure erected in order to comply the Finnish requirements can be not approved by Russian authorities if they will hold inspection according to Russian norms.

Nevertheless, being quite big, RakMK tolerances are still very small to affect a proper work of a structure, so the structure with deviations not more than allowed will keep its bearing capacity and durability.

Comparison of tolerances for the same measured parameters of structures is represented in appendix 3.

8 CONCLUSION

The main goal of the study was to compare general requirements and details in design and construction of reinforced concrete structures given by Finnish, Russian and European norms and to discover significant differences between them.

The scope of the study did not include methods of calculation of strength of reinforced concrete structural elements and static analysis. This topic is considered to be less significant because the way structure behaves and collapses under applied load does not depend on the country where calculation is carried out and its norms.

More important issues here are the requirements for details of reinforcement of concrete structures, classification of materials used for reinforced concrete structures and information for quality control. Studying of these issues helps to implement international construction projects between Finland and Russia.

The study showed that there is not significant differences between requirements given by different norms. This conclusion is quite predictable because the theory of strength of structures is based on the same principles al over the world and the way strength is provided should be similar.

Nevertheless there are different requirements for minimal reinforcement, concrete cover and tolerances. More detailed results of comparisons are given in the end of each chapter.

One more thing is that classification of concrete used in all studied norms is different, so it is necessary to know how concrete is classified in different norms, what strength classes are used and how concrete classes used in one norm correspond to classes used in another norm.

TABLES

FIGURES

REFERENCES

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2. SNiP 2.01.07-85 “Loads and actions”, 1985

3. SNiP 3.03.01-87 “Bearing and envelope structures”, 1987

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6. GOST 5781-82 “Hot-rolled reinforcing steel”, 1982

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9. Finnish National Annex to EN 1990

10. Finnish National Annex to EN 1992

11. EN 206-1 “Concrete”, December 2000

12. Concrete code by50, Concrete Association of Finland, 2004

13. Concrete structures tolerances by39, Concrete Association of Finland, 1995

14. GOST 13015.0-83 “Prefabricated concrete and reinforced concrete constructions and products”

Appendix 1. Concrete strength classes

Table1. Strength classes from SP 52-101-2003 and their main properties.

Concrete strength class	Design value of compressive strength for ULS, f_{cd}, MPa	Characteristic value of compressive strength, f_{ck}, MPa	Design value of tensile strength for ULS, f_{ctd}, MPa	Characteristic value of tensile strength, f_{ctk}, MPa	Modulus of elasticity, E, GPa
B10	6,0	7,5	0,56	0,85	19,0
B15	8,5	11,0	1,75	1,10	24,0
B20	11,5	15,0	0,90	1,35	27,5
B25	14,5	18,5	1,05	1,55	30,0
B30	17,0	22,0	1,15	1,75	32,5
B35	19,5	25,5	1,30	1,95	34,5
B40	22,0	29,0	1,40	2,10	36,0
B45	25,0	32,0	1,50	2,25	37,0
B50	27,5	36,0	1,60	2,45	38,0
B55	30,0	39,5	1,70	2,60	39,0
B60	33,0	43,0	1,80	2,75	39,5

Table2. Strength classes from RakMK and their main properties.

Concrete strength class	Design value of compressive strength for ULS, f_{cd}, MPa	Characteristic value of compressive strength, f_{ck}, MPa	Design value of tensile strength for ULS, f_{ctd}, MPa	Characteristic value of tensile strength, f_{ctk}, MPa	Modulus of elasticity, E, GPa
K15	7,0	10,5	0,81	1,22	19,4
K20	9,3	14	0,98	1,47	22,4
K25	11,7	17,5	1,14	1,71	25,0
K30	14,0	21	1,29	1,93	27,4
K35	16,3	24,5	1,43	2,14	29,6
K40	18,7	28	1,56	2,34	31,6
K45	21,0	31,5	1,69	2,53	33,5
K50	23,3	35	1,81	2,71	35,4
K55	25,7	38,5	1,93	2,89	37,1
K60	28,0	42	2,04	3,07	38,7
K70	32,7	49	2,44	3,30	41,8

Table3. Strength classes from EN 1992 and their main properties.

Concrete strength class	Design value of compressive strength for ULS, f_{cd}, MPa	Characteristic value of compressive strength, f_{ck}, MPa	Design value of tensile strength for ULS, f_{ctd}, MPa	Characteristic value of tensile strength, f_{ctk}, MPa	Modulus of elasticity, E, GPa
C12/15	6,8	10,2	0,73	1,10	27
C16/20	9,1	13,6	0,89	1,33	29
C20/25	11,3	17	1,03	1,55	30
C25/30	14,2	21,25	1,20	1,80	31
C30/37	17,0	25,5	1,35	2,03	32
C35/45	19,8	29,75	1,50	2,25	34
C40/50	22,7	34	1,64	2,46	35
C45/55	25,5	38,25	1,77	2,66	36
C50/60	28,3	42,5	1,90	2,85	37
C55/67	31,2	46,75	1,97	2,95	38
C60/75	34,0	51	2,03	3,05	39

Table 4. Comparison between strength classes.

Strength class SP 52-101-2003	Design compressive strength, MPa	Strength class, EN 1992	Design compressive strength, MPa	Strength class, RakMK	Design compressive strength, MPa
B10	6,0	C12/15	6,8	K15	7,0
B15	8,5	C16/20	9,1	K20	9,3
B20	11,5	C20/25	11,3	K25	11,7
B25	14,5	C25/30	14,2	K30	14,0
B30	17,0	C30/37	17,0	K35	16,3
B35	19,5	C35/45	19,8	K45	18,7
B40	22,0	C40/50	22,7	K45	21,0
B45	25,0	C45/55	25,5	K55	25,7
B50	27,5	C50/60	28,3	K60	28,0
B55	30,0	C55/67	31,2	K70	32,7
B60	33,0	C60/75	34,0	K70	32,7

Appendix 2. Reinforcement

Table 1. Comparison of reinforcing

Parameter	SP 52-101-2003	EN 1992	RakMK
1. Anchorage			
Minimum values of the anchorage length	• $0,3l_{0,an}$, where $l_{0,an}$ is the basic anchorage length • $15\varnothing$ • 200mm	• $0.3l_b$ tension bars • $0.6l_b$ compression bars • $15\varnothing$ • 100mm For bars at intermediate supports: • 10Ø l_b is the basic anchorage length	The design anchorage length is determined by an expression for both tension and compression bars But not less than: • $10\varnothing$
2. Lap joints			
2.1 Length of lap	Not less than: • $0.4\alpha \cdot l_{0,an}$ • $20d_s$ • $250mm$	Only formula for calculation is represented, no minimum values are given	Only formula for calculation is represented, no minimum values are given
2.2 Other regulations	The clear distance between two lapping bars must not be more than $4\varnothing$	In some cases extra transverse reinforcement in the lap zone is required. The clear distance between two lapping bars should be not more than $4\varnothing$ or 50mm whichever is smaller.	The clear distance between two lapping bars should be not more than $50mm$

3. Reinforcement details

3.1 Beams

Minimum and maximum reinforcement areas:	Minimum area of tension reinforcement: $A_s = 0.001A_c$ The maximum area of the working reinforcement is not limited.	Minimum area of tension reinforcement: • $A_{s,\min} = 0.26\dfrac{f_{ctm}}{f_{yk}}A_c$ • $0.0013A_c$ which is greater. The maximum area of the working reinforcement is not limited.	Minimum area of tension reinf.: $A_s = 0.5\dfrac{f_{ctk}}{f_{yk}} \cdot A_c$
Rules for longitudinal reinforcement	The clear distance between reinf. bars should not be more than: • 400mm	If the diameter of the main bars is more than 32mm, surface reinforcement need to be placed outside stirrups.	The bar diameter is at least 8mm. The spacing is not more than 300mm. For beams with a height of >800mm and the required area of main reinf. is $\geq 400mm^2$, a longitudinal reinf. should be placed in both facades of the tension areas, with a maximum spacing of 300mm. A ratio for this reinf. to the area of concrete in tension: $\dfrac{A_s}{A_c} = 0.12\dfrac{f_{ctk}}{f_{yk}}$

Rules for transverse reinforcement	Diameter of transverse reinforcement should not be less than 6mm. Spacing not more than: • $15\varnothing$, $\varnothing$ - longitudinal bars • $500mm$	The ratio of shear reinforcement: $\rho_w = \frac{A_{sw}}{s \cdot b_w \cdot \sin\alpha} \geq \frac{0.08\sqrt{f_{ck}}}{f_{yk}}$ Where a is an angle between shear reinf. and longitudinal axe of the member	The ratio of shear reinf. to the area of cross-section: $\frac{A_{sv}}{A_c} \geq 0.2\frac{f_{ctk}}{f_{yk}}$ Longitudinal spacing of shear reinf. not more than: • 0.7d (d is a depth of cross-section) • 400mm Transverse spacing not more than: • d • 600mm
3.2 Columns			
Minimum and maximum reinforcement	Minimal reinf. depends on l_0 / h ratio: $As = 0.001Ac$ for $l_0 / h \leq 5$ $As = 0.0015Ac$ for $5 < l_0 / h \leq 10$ $As = 0.002Ac$ for $10 < l_0 / h \leq 25$ $As = 0.0025Ac$ for $l_0 / h > 25$	The minimum area of reinf.: • $As, \min = 0.1\frac{N_{Ed}}{f_{yd}}$ or • 0.002Ac which is greater The maximum area of reinf.: $As, \max = 0.06Ac$	Minimum value: $A_s \geq 1.5\frac{f_{ctk}}{f_{yk}} \cdot A_c$ The total area of longitudinal reinf. should not be more than 0.06Ac

Rules for longitudinal reinforcement	The distance between bars should be not more than: • 400mm perpendicular to bending moment • 500mm parallel to bending moment	The diameter of bars should not be less than 8mm;	The spacing of the main bars should be no more than: • twice the smallest side dimension • 300mm The diameter of the main bars should be not less than: • 12mm • 8mm for welded reinforcing assemblies • 10mm for one-storey frames with the height of the column $\leq 3m$
Rules for transverse reinforcement	The diameter should be not less than: • 6mm • ¼ of the max. size of main bars The spacing is not more than: • 15$\varnothing$ ($\varnothing$ is the minimum diameter of main bars) • 500mm In prefabricated columns several secondary steel meshes are needed in the top of the column.	The diameter should be not less than; • 6mm • ¼ of the max. size of main bars The spacing of transverse reinf. should not be more than: • 20$\varnothing$ ($\varnothing$ is the minimum diameter of main bars) • the lesser dimension of the column • 400 mm The spacing should be reduced by 0.6 above or`1 below a beam or slab	The minimum diameter: • 0.25d The maximum spacing: • 15d where d is the main bars' diameter

3.3 Walls

Minimum and maximum reinforcement	As for columns, the minimum reinf. depends on l_0 / h ratio: $As = 0.001Ac$ for $l_0 / h \le 5$ $As = 0.0015Ac$ for $5 < l_0 / h \le 10$ $As = 0.002Ac$ for $10 < l_0 / h \le 25$ $As = 0.0025Ac$ for $l_0 / h > 25$	Vertical reinforcement: 0.002Ac≤As≤0.06Ac Horizontal reinforcement area, As, should be not less than: • 25% of the vertical reinforcement • 0.001Ac	Minimum value: $A_s \ge 0.25 \frac{f_{ctk}}{f_{yk}} \cdot A_c$
Rules for longitudinal reinforcement	The distance between vertical bars should not be more than: • 2t (t - wall thickness) • 400mm The distance between horizontal bars should not be more than 400mm.	The distance between two adjacent vertical bars shall not exceed • 3t (t- wall thickness) • 400 mm The distance between horizontal bars should not be more than 400mm.	The diameter of horizontal reinf. is at least half the diameter of vertical bars. The spacing of both vertical and horizontal bars should be not more than 300mm.
Rules for transverse reinforcement	No regulations	If the total area of vertical reinf. in the two faces exceeds $0.02A_c$, transverse reinf. should be provided at a number of 4 per m^2 of wall area.	No regulations

3.4 Slabs

Minimum and maximum reinforcement	Minimum reinforcement (all- top and bottom): As=0.002Ac	Minimum area of tension reinforcement: • As,min= 0.26 $\cdot \dfrac{f_{ctm}}{f_{yk}} A_c$ • 0.0013Ac, which is greater. The maximum reinf. should be not more than 1,5 times the area required for the ULS.	Minimum area of reinforcement: $A_s = 0.25 \dfrac{f_{ctk}}{f_{yk}} \cdot A_c$
Rules for longitudinal reinforcement	The clear distance between bars should not be more than: • 400mm	The clear distance between bars should not be more than: • 400mm for the principal reinf. • 450mm for the secondary reinf. For areas of maximum moments: • 250mm for the principal reinf. 400mm for the secondary reinf.	The spacing of bars at the maximum moments should be no more than: • 3 times the depth of the slab • 400 mm • but ≥150 mm. The maximum spacing of bars within edge zones of slabs: • 4 times the depth of the slab • 600mm
Rules for transverse reinforcement	For slabs with thickness more than 300mm shear reinf. should be installed with the span not more than: • 0.75h • 500mm, which is smaller	The rules for slabs are the same as for beams	The rules for slabs are the same as for beams

Appendix 3. Tolerances

Table 1. Comparison of tolerances

Measured parameter	Tolerance, mm	
	RakMK (Normal class)	SNiP 3.03.01-87
Dimensions of cross-section: - thickness of wall - column cross-sectional dimensions	 ±8 ±10	 +6, -3 +6, -3
Vertical deviation (inclination): - wall - column	 L/300 (10mm for 3m wall) ±15	 ±15 (cast-in-situ) ±10 (prefabricated)
Length of structural element: - wall - column - foundation	 ±10 ±15 ±30	 ±20 ±20 ±20
Level of the upper bearing surface: - wall - column	 ±10 ±15	 -5 -5
Reinforcement: distance between bars	Depends on the length of the bar: ±15….±30	±10
The minimum value of concrete cover that should be ensured in any cases	5	13

Appendix 4. Reinforcing steel

Table 1. Strength classes used in Russia, GOST 5781-82

Strength classes used in Russia		
Old marking	New marking, corresponding to European	Characteristic value of tensile strength, MPa
A-I	A240	240
A-II	A300	300
A-III	A400	400
A-IV	A600	600
A-V	A800	800
A-VI	A1000	1000
Bp-I	B500	500

Table 2. Strength classes commonly used in Russia

Smooth reinforcement	Ribbed reinforcement			
A240 (A1)	A300 (A2)	A400, A400C* (A3)	A500, A500C	B500, B500C* (Bp1)

*****C means weldable

Table 3. Design values of tensile strength of steel

Steel class, the most commonly used	Design value of tensile strength, MPa		
	Eurocode 2	RakMK	SNiP
A500, hot rolled	435	415	435
B500 (Bp1 in Russia), cold worked	435	415	415