Date: 20XX -XX

Høringsudgave 01.07.2026

DS/EN 1992-1-1:2023 DK NA:202x

National Annex to: Eurocode 2 — Design of concrete structures – Part 1-1: General rules and rules for buildings, bridges and civil engineering structures

Nationalt Anneks til: Eurocode 2 — Betonkonstruktioner – Del 1-1: Betonkonstruktioner – Generelle regler samt regler for bygninger, broer og bygningskonstruktioner

Foreword

Published [måned og år].

This National Annex (NA) to DS/EN 1992-1-1:2023 forms part of the Danish national annexes to the second generation of Eurocodes.

This NA supersedes DS/EN 1992-1-1 DK NA:2024 upon its implementation in the Building Regulations (Bygningsreglementet 2018 – BR18) as an appendix.

During the transition period stated in the Building Regulations, both the first generation and the second generation of the Eurocodes and NAs may be applied.

This NA cannot be used in conjunction with the first generation of Eurocodes.

This NA lays down the conditions for the Danish implementation of DS/EN 1992-1-1:2023 for construction works in conformity with the Danish Building Regulations; Bygningsreglementet 2018 (BR 18). Other parties may bring this NA into force by reference.

The Eurocodes establish the principle that certain parameters are left open for national determination. These parameters are referred to as *Nationally Determined Parameters* (NDP) and specify the nationally applicable values, classes or methods. Furthermore, the NA may provide *Non-Contradictory, Complementary Information* (NCCI), i.e. additional guidance and alternative methods or values that support the application of the Eurocode without altering or contradicting its provisions.

Further, this National Annex includes NOTES. Notes have the status of guidance and provide additional information intended to assist the understanding or use of text of the national annex.

In conformity with the Danish Building regulations, the verbal forms used in this Eurocode and NA are to be understood as follows:

- "shall" expresses a requirement to be followed
- "should" expresses a recommendation. Alternative approaches could be used if technically justified
- "may" expresses permission within the limits of Eurocode and NA
- "can" expresses a possibility or capability
- "is/are" expresses a statement of certainty or a fact

This NA includes:

- Foreword;
- An overview of clauses with possible Nationally Determined Parameters (NDPs) and of Non-Contradictory Complementary Information (NCCI);
- National choice to national Determined Parameters (NDP), and
  Non-Contradictory Complementary Information (NCCI)

Valid versions of the NAs as well as previous versions, addenda can be found at www.bygningsreglementet.dk.

Overview of possible Nationally Determined Parameters (NDPs) and Non-Contradictory Complementary Information (NCCI)

The list below identifies the clauses where Nationally Determined Parameters (NDPs) are possible, and whether a national choice has been made, and if an informative annex is applicable or not applicable. Furthermore, clauses providing complementary information (NCCI) are identified in the list below.

Clause	**Subject**	**Nationally Determined Parameters**	**Non-Contradictory Complementary Information**
2	Normative references		Complementary information
4.2.1.5(3)	Prestress	National choice	
4.2.1.6(1)	Effect of water or gas pressure		Complementary information
4.3.1(1)	Partial factor for shrinkage action	Unchanged	
4.3.2(1)	Partial factor for prestress action	National choice	
4.3.2(2)	Partial factor for prestress action	Unchanged	
4.3.3(1)	Partial factors for materials	National choice	
4.3.3(4)	Partial factors for materials	National choice	
5.1.3(3)	Strength	Unchanged	Complementary information
5.1.4(2)	Elastic deformation	National choice	
5.1.5(4)	Creep and shrinkage	Unchanged	
5.1.6(1)	Design assumptions	National choice	
5.1.6(2)	Design assumptions	Unchanged	
5.2.1(5)	Reinforcing steel	Unchanged	
5.2.2(1)	Properties	Unchanged	
5.3.1(3)	Prestressing steel	Unchanged	
5.3.2(1)	Properties	Unchanged	
5.4.1(1)	Prestressing systems	Unchanged	
6.3(3)	Environmental exposure conditions	National choice	
6.3(5)	Environmental exposure conditions	Unchanged	
6.4(1)	Exposure resistance classes	National choice	

Clause	Subject	Nationally Determined Parameters	Non-Contradictory Complementary Information
6.5.2.1(2)	Concrete cover – minimum cover	National choice	
6.5.2.2(1)	Minimum cover for durability	National choice	
6.5.2.2(2)	Minimum cover for durability	National choice	
6.5.2.2(3)	Minimum cover for durability	National choice	
6.5.2.2(4)	Minimum cover for durability	National choice	
6.5.2.2(5)	Minimum cover for durability	National choice	
6.5.2.2(6)	Minimum cover for durability	National choice	
6.5.2.2(9)	Minimum cover for durability	National choice	
6.5.3(1)	Allowance in design for deviation in cover	Unchanged	
7.2.1.2(6)	Representation of imperfections		Complementary information
7.3.1(6)	Linear elastic analysis		Complementary information
7.3.1(7)	Linear elastic analysis		Complementary information
7.3.2(5)	Linear elastic analysis with redistribution	Unchanged	
7.3.3.1(3)	General		Complementary information
7.3.3.1(6)	General		Complementary information
7.3.3.2(1)	Analysis for beams, frames and slabs without verification of rotation capacity		Complementary information
7.4.1(3)	Second order structural analysis of members and systems with axial force		Complementary information
7.4.3.1(1)	Methods of analysis		Complementary information
7.4.3.1(2)	Methods of analysis		Complementary information
8.2.1(2)	General verification procedure		Complementary information
8.2.1(3)	General verification procedure		Complementary information

Clause	Subject	Nationally Determined Parameters	Non-Contradictory Complementary Information
8.2.1(3)	General verification procedure	No further information	
8.2.2(5)	Detailed verification for members without shear reinforcement	Unchanged	
8.2.3(11)	Members with shear information		Complementary information
8.2.6(1)	Shear at interfaces		Complementary information
8.2.6(5)	Shear at interfaces		Complementary information
8.2.6(7)	Shear at interfaces		Complementary information
8.4.2(1)	Shear-resisting effective depth, control perimeter and shear strength	Unchanged	
8.4.4(4)	Punching shear resistance of slabs with shear reinforcement	Unchanged	
8.4.4(5)	Punching shear resistance of slabs with shear reinforcement	Unchanged	
8.4.4(6)	Punching shear resistance of slabs with shear reinforcement	Unchanged	
8.5.1(1)	Design with strut and tie models and stress fields, General		Complementary information
8.5.5(3)	Transfer of concentrated forces into a member		Complementary information
9.1(3)	Serviceability Limit States (SLS)		Complementary information
9.1(4)	Serviceability Limit States (SLS)		Complementary information
9.2.1(6)	General considerations	National choice	
9.2.2(2)	Minimum reinforcement areas to avoid yielding		Complementary information
9.2.3(2)	Refined control of cracking	Unchanged	
9.2.3(2)	Refined control of cracking		Complementary information

Clause	Subject	Nationally Determined Parameters	Non-Contradictory Complementary Information
9.2.3(3)	Refined control of cracking		Complementary information
9.2.3(4)	Refined control of cracking		Complementary information
9.3.4(4)	General method for deflection calculations		Complementary information
10	Fatigue		Complementary information Awaiting
11.4.2(2)	Anchorage of straight bars	National choice	
11.4.2(3)	Anchorage of straight bars	National choice	
11.4.7(2)	Anchorage of headed bars in tension		Complementary information
11.5.2(2)	All types of laps	National choice	
11.6.3(2)	Minimum radius of curvature and straight length of tendons adjacent to anchorages	Unchanged	
11.7(3)	Deviation forces due to curved tensile and compressive chords		Complementary information
11.7(4)	Deviation forces due to curved tensile and compressive chords		Complementary information
12.3.1(1)	Beams, General	National choice	
12.4.1(1)	Slabs, General	National choice	
12.6(1)	Columns	National choice	
12.7(2)	Walls and deep beams	National choice	
12.9.3(1)	Required resistance for ties	Awaiting	
13.4.1(1)	Structural analysis, general		Complementary information
13.7.1(15)	General		Complementary information
13.7.1(16)	General		Complementary information
14.2(1)	Concrete	National choice	

Clause	**Subject**	**Nationally Determined Parameters**	**Non-Contradictory Complementary Information**
14.4.5.2(1)	Simplified design method for walls and columns	Unchanged	
Annex A	Adjustment of partial factors for materials	Not applicable	
A.3(1)	General	Unchanged	
A.3(3)	General	Unchanged	
A.3(6)	General	Unchanged	
B.3(1)	General	Unchanged	
B.6(1)	Basic formulae for determining the shrinkage strain	Unchanged	
C.6(1)	Couplers	Unchanged	
C.7(1)	Headed bars	Unchanged	
C.8(2)	Post-installed reinforcing steel systems	Unchanged	
Annex D	Evaluation of early age and long-term cracking due to restraint	Informative	Complementary information Awaiting
Annex F	Safety formats for non-linear analysis	Informative	
F.1(1)	Use of annex F		Complementary information
F.2(1)	Scope and field of application		Complementary information
F.3(6)	General		Complementary information
F.4(1)	Partial Factor Method (PFM)	Not applicable	
F.4(2)	Partial Factor Method (PFM)		Complementary information
F.5	Global factor method (GFM)	Not applicable	
F.5.2(1)	Determination of the global resistance factor	Not applicable	
F.6	Full probabilistic method	Not applicable	
F.7	Model uncertainty	Not applicable	
F.7(2)	Model uncertainty	Not applicable	

Clause	**Subject**	**Nationally Determined Parameters**	**Non-Contradictory Complementary Information**
F.7(8)	Model uncertainty	Not applicable	
Annex F NA	Safety formats for non-linear analysis	Informative	
Annex G	Design of membrane-, shells and slab elements	Normative	
G.5(1)	Refined control of cracking in membrane elements in SLS		Complementary information
Annex H	Guidance on design of concrete structures for water tightness	Informative	
H.4.2(4)	Tightness requirements	Unchanged	
Annex I	Assessment of Existing Structures	Not applicable Awaiting	
I.4.2.1(2)	Partial factors for assessment	Awaiting	
I.5.2.1(3)	General	Awaiting	
I.5.2.2(1)	Assessment assumption	Awaiting	
I.8.3.1(1)	Detailed verification of members not requiring shear reinforcement	Awaiting	
I.9.1(2)	General	Awaiting	
Annex J	Strengthening of Existing Concrete Structures with CFRP	Not applicable Awaiting	
J.4(1)	Basis of design	Not applicable	
J.5.1(2)	General	Not applicable	
Annex K	Bridges	Normative Awaiting	
K.5(2)	Materials	Awaiting	
K.6(2)	Durability and concrete cover	Awaiting	
K.6(3)	Durability and concrete cover	Awaiting	
K.6(4)	Durability and concrete cover	Awaiting	
K.7(2)	Structural analysis	Awaiting	
K.7(3)	Structural analysis	Awaiting	
K.8(2)	Ultimate limit states (ULS)	Awaiting	

Clause	**Subject**	**Nationally Determined Parameters**	**Non-Contradictory Complementary Information**
K.8(3)	Ultimate limit states (ULS)	Awaiting	
K.8(4)	Ultimate limit states (ULS)	Awaiting	
K.8(5)	Ultimate limit states (ULS)	Awaiting	
K.8(6)	Ultimate limit states (ULS)	Awaiting	
K.9(2)	Serviceability limit states (SLS)	Awaiting	
K.9(3)	Serviceability limit states (SLS)	Awaiting	
K.10.1(2)	General	Awaiting	
K.11(2)	Detailing of reinforcement and post-tensioning tendons	Awaiting	
K.11(3)	Detailing of reinforcement and post-tensioning tendons	Awaiting	
K.11(4)	Detailing of reinforcement and post-tensioning tendons	Awaiting	
K.11(5)	Detailing of reinforcement and post-tensioning tendons	Awaiting	
K.12.2(1)	Minimum reinforcement rules	Awaiting	
K.13(2)	Additional rules for precast concrete elements and structures	Awaiting	
K.13(4)	Additional rules for precast concrete elements and structures	Awaiting	
K.15(1)	Amendments to Annex G	Awaiting	
Annex L	Steel Fibre Reinforced Concrete Structures	Informative with changes in annex L NA Annex L is first applicable when the safety factors are complying with the Danish safety format.	
Annex L NA	Steel Fibre Reinforced Concrete Structures	Informative	
L.2	Scope and field application		Complementary information

Clause	**Subject**	**Nationally Determined Parameters**	**Non-Contradictory Complementary Information**
L.4(1)	Basis of design – Partial factors for materials	Awaiting sufficient experimental data to determine that the safety factors are complying with the Danish safety format.	
L.5.1(1)	Properties	Unchanged	Complementary information
L.5.5.2(1)	Stress-strain relation for structural analysis	National choice	
L.6(4)	Durability – Minimum cover	National choice	
L.7(1)	Structural analysis – plastic analysis	National choice	
L.8.2.3	Members with shear reinforcement	Not applicable	
L.8.3	Torsion – Torsional resistance of compact or closed sections	Not applicable	
L.8.4.2	Punching shear resistance of SFRC slabs with shear reinforcement	Not applicable	
L.8.5	Design with strut-and-tie models – Ties	Not applicable	
L.11.2(1)	Spacing of bars	Unchanged	Complementary information
L.12.2.2(1)	Shear and torsion reinforcement	National choice	
L.12.3.1(2)	General	National choice	
L.14	Lightly reinforced SFRC structures	Not applicable	
Annex N	Recycled aggregates concrete structures	Informative with changes	
Annex N NA	Recycled aggregates concrete structures	Informative	
Annex O	Simplified approaches for second order effects	Informative with changes	
Annex O NA	Simplified approaches for second order effects	Informative	
Annex P	Alternative cover approach for durability	Not applicable	

Clause	**Subject**	**Nationally Determined Parameters**	**Non-Contradictory Complementary Information**
Q.3(1)	General	Unchanged	
Q.4(2)	Minimum cover for durability	Unchanged	
Annex R	Embedded FRP reinforcement	Not applicable	
R.4(1)	Verification – Partial factors FRP reinforcement	Unchanged	
R.5.1(2)	General	Unchanged	
Annex S	Minimum reinforcement for crack control and simplified control of cracking	Informative with changes	
S.1(1)	Use of this Annex	National choice	
S.2(1)	Scope		Complementary information
S.3(1)	Minimum reinforcement areas for crack width control		Complementary information
S.4	Simplified control of cracking	National choice	
NOTE Unchanged: Recommendations in the Eurocode to be followed. National choice: A National Determined Parameter is chosen and defined. Complementary information: (Non-contradictory) complementary information on how to use the Eurocode. Normative: The Annex shall be used. Informative: The Annex may be used. Not applicable: The Annex cannot be used. No further information: The Eurocode allows further information. No further information is given.			

Nationally Determined Parameters (NDP) and Non-Contradictory Complementary Information (NCCI)

2 Normative references

(NCCI)

DS 2427 - EN 13670:20xx, Concrete execution – Rules for application of EN 13670 in Denmark

DS 206:20xx, Concrete – Specification, performance, production and conformity – Rules for application of EN 206 in Denmark.

DS/EN 13670 applies in conjunction with DS 2427. For prefabricated elements, DS/EN 13369 applies, provided that the elements are covered by the harmonized product standard or subject to third-party monitoring in accordance with DS/EN 13369, Annex E.

4.2.1.5(3) Prestress

(NPD)

For all three types of prestress:

r_{sup} = 1,00

r_{inf} = 1,00

4.2.1.6(1) Effect of water or gas pressure

(NCCI)

In cylindrical vessels or pipes with circular cross section and with internal fluid- or gas pressure, the effect of pressure build-up in pores and in tensile cracks may be neglected when the concrete wall thickness is less than 5% of the inner diameter of the cross section.

4.3.2(1) Partial factors for prestress action

(NDP)

$\gamma_{P,unfav}$ = 1,2

All other values are 1,00.

4.3.3(1) Partial factors for materials

(NDP)

For persistent and transient design situations the partial factors for material in Table 4.3.a NA are used. Basis for partial factors see Table 4.3.b NA.

Table 4.3.a NA — Partial factors for materials for ultimate limit states for persistent and transient design situations

Topic	Description	Symbol	Persistent design situations	In accidental or seismic situations
Structures, general and calculation				
1	Reinforced concrete [a] Compressive strength concrete Modulus of elasticity concrete	γ_C	1,31	1,00
2	Plain concrete [b] Compressive strength concrete Modulus of elasticity concrete	γ_C	1,44	1,00
3	Tensile strength concrete [c]	γ_C	1,50	1,00
4	Members without shear reinforcement	γ_V	1,38	1,00
5	Reinforcement Non-prestressed and prestressed Yield strength [e] Modulus of elasticity	γ_s	1,22	1,00
6	Reinforcement Non-prestressed and prestressed Ultimate strain limit	$\gamma_{s\varepsilon}$	1,22	1,00
Structures, testing				
7	Leading to ductile failure [d]	γ_M	1,20	1,00
8	Leading to brittle failure	γ_M	1,40	1,00

NOTE Partial factors are given with 2 decimals rounded to the nearest half decimal (0,05).

a Reinforced concrete: Concrete structure that contains minimum reinforcement for the relevant impacts in agreement with DS/EN 1991-1-1:2023 and DS/EN 1992-1-1:2023 DK NA.

b The partial factor for the compressive strength and modulus of elasticity of plain concrete, γ_c, applies to structures not provided with minimum reinforcement for the relevant impact conforming to the rules in this Eurocode. The rules for minimum reinforcement can be modified if it is documented by experiments that the type of failure will not differ from the type of failure for the structure which complies with the rules for minimum reinforcement given in the Eurocode.

c The partial factor for the tensile strength of concrete γ_c is applied in cases where the failure of the concrete is a tensile failure, and/or where the structure is not provided with minimum reinforcement, i.e. anchorage/laps in clause 11 and clause 14.

d	Members subject to transverse load are assumed to exhibit ductile failure if at least one of the following conditions is fulfilled: - Yielding of the reinforcement at failure is documented by measurement. - Prior to failure, a uniformly distributed crack pattern occurs corresponding to the load applied. - Prior to failure, deflection exceeds 3/200 of the span. Other failure modes are regarded as brittle failures. Failure of members subjected to axial forces is always assumed to be brittle failure
e	The specified value does not apply to the tensile strength of the reinforcement.

Table 4.3.b NA — Basis for partial factors

Topic	Symbol	γ_1 Failure mode [a]	γ_2 Calculation model					γ_3 Strength parameter				
			$V_{2,\mathrm{XR}}$[b] %	$V_{2,\mathrm{XRT}}$[c] %	V_2 %	b_2[d]	γ_2	$V_{3,\mathrm{R}}$[e] %	$V_{3,\mathrm{RT}}$[f] %	V_3 %	b_3[g]	γ_3
Structures, general and calculation												
1	γ_c	WWRR	10	5	11	1,00	1,06	6,5	9	11	1,00	1,23
2	γ_c	NW	10	5	11	1,00	1,06	6,5	9	11	1,00	1,23
3	γ_c	NW	10	5	11	1,00	1,06	11	-	16	1,00	1,29
4	γ_V	NW	-	-	13	1,00	1,06	-	-	4	1,00	1,17
5	γ_s	NW	8	5	9,5	1,00	1,04	4,5	0,00	4,5	1,00	1,17
6	$\gamma_{s\varepsilon}$											1,22
Structures, testing												
7	γ_M											1,20
8	γ_M											1,40

NOTE γ_i and the sub partial factors (γ_1, γ_2 and γ_3) are based are determined following EN1990 Annex FF NA.

a Failure mode
- WWR: Warning With Residual ($\gamma_1 = 0.9$)[h]
- WWRR: Warning Without Residual Resistance ($\gamma_1 = 1.0$)
- NW: No Warning ($\gamma_1 = 1.1$)

b Coefficient of variation of uncertainty of calculation model in laboratory conditions.

c Coefficient of variation of uncertainty associated with the transfer from laboratory conditions to conditions in a real structure for calculation model.

d Bias is the conservatism included in the Eurocode calculation model.

e Coefficient of variation of strength parameter in laboratory conditions.

f Coefficient of variation of uncertainty associated with the transfer from laboratory conditions to conditions in a real structure for the strength parameter.

g Bias is the extra conservatism included in the representative (nominel) value of the strength parameter compared to the value of the characteristic 5% fractile.

h Warning with residual resistance is fulfilled when an additional resistance of the order 10% is obtained compared to the Eurocode calculation model.

For the verification of fatigue for persistent design situations, the partial factors given in Table 4.3.a NA multiplied by 1,1 are used for the values $\gamma_{c,\mathrm{fat}}$ and $\gamma_{s,\mathrm{fat}}$.

4.3.3(4) Partial factors for materials

(NDP)

The value k_{cip} = 1,0 applies in general.

5.1.3(2) Strength

(NCCI)

A f_{ck} determined after 56 days (instead of after 28 days) can be used for specific projects. This requires that documented curves for the strength development of the specified concrete and documentation that ensures the characteristic strength at 56 days are available at the start of the project. The strength class must be documented both after 28 days and 56 days.

NOTE When using the 56-day strength, it must be ensured that the strength development of the concrete has been considered in relation to the construction process - construction rate and temporary loads.

5.1.3(3) Strength

(NCCI)

Based on a coefficient of variation of 11 % for the compression strength in the structure, the ratio between the mean and the characteristic compression strength can be set to $f_{cm} = 1{,}22f_{ck}$. This relationship between f_{cm} and f_{ck} should be used instead of the one given in Table 5.1.

The formulae for f_{ctm} in Table 5.1 reflect the mean splitting tensile strength of the concrete when f_{cm} instead of f_{ck} is inserted into the formulae. As an approximation, the mean uniaxial tensile strength of the concrete may be taken as $0{,}94f_{ctm}$, where f_{ctm} corresponds to the value given by the formulae in Table 5.1. The characteristic uniaxial strength is given by the formula for f_{ctk} in Table 5.1.

5.1.4(2) Elastic deformation

(NDP)

For concrete with quartzite aggregates according to DS 206, k_E = 9000 should be assumed. In Formulae (5.1), f_{cm} is inserted in MPa and the calculated value of E_{cm} has the unit MPa.

5.1.6(1) Design assumptions

(NDP)

- k_{tc} = 1,00 for $t_{ref} \leq$ 56 days.

6.3(3) Environmental exposure conditions

(NDP)

The exposure conditions are given in Table 6.1 NA including informative examples.

For structural members, it shall be declared which exposures, stated as classes in Table 6.1 NA, to which the surfaces of the structural members are subjected. Each surface shall be associated with at least one exposure class.

NOTE 3 A structural member can be exposed to several of the exposure conditions mentioned in Table 6.1 NA, and the total exposure is described by a combination of exposure classes.

NOTE 4 The structural members can be subjected to other relevant exposures during the construction process other than those relevant for the permanent situation, e.g. frost actions and moisture exposure

NOTE 5 The exposure classes cover the typical climate and environmental impacts that normally occur in Denmark. If structural members are exposed to extra aggressive environmental actions – for instance directly on a cracked concrete surface or subject to certain industrial liquids and/or gasses – this could lead to additional requirements in terms of durability.

Table 6.1 NA – Exposure classes related to environmental conditions

Class	Description of the environment	Min. strength class [a]	Informative examples for exposure classes typical under Danish climate and environment
1 No risk of corrosion or attack			
X0	For concrete without reinforcement or other embedded metal: All exposures except where there is freeze/thaw, abrasion or chemical attack.	C12/15	Plain concrete where the structural capacity does not require reinforcement. For instance some foundations.
2 Corrosion of embedded metal induced by carbonation Where concrete containing steel reinforcement or other embedded metal is exposed to air and moisture, the exposure shall be classified as follows:			
XC1	Dry.	C12/15	Concrete surfaces inside buildings with low air humidity, where the corrosion rate will be insignificant, e.g.: - indoor structures - ground slabs on insulation

Class	Description of the environment	Min. strength class [a]	Informative examples for exposure classes typical under Danish climate and environment
XC2	Wet or permanent high humidity, rarely dry	C30/37	Concrete surfaces exposed to long-term water contact, e.g.:. - foundation piles - ground slabs and trenches - water retaining structures and waste water structures (tanks, channels, etc.) - retaining walls - buried foundations with structural reinforcement
XC3	Moderate humidity	C30/37	Concrete surfaces in moderate air humidity but sheltered from direct rain, e.g.: - some indoor structures in moderate to high humidity (for instance in unheated buildings, garages, storage, parking deck, etc.) - outdoor structures not exposed to direct rain
XC4	Cyclic wet and dry	C30/37	Concrete surfaces exposed to cyclic water contact, e.g. - external walls, facades, columns and beams - balconies - basement walls above ground - industrial structures with high moisture exposure - civil structures, bridges, columns, edge beams, etc. - parking structures
3 Corrosion of embedded metal induced by chlorides, excluding sea water Where concrete containing steel reinforcement or other embedded metal is subject to contact with water containing chlorides, including de-icing salts, from sources other than sea water, the exposure shall be classified as follows:			

Class	Description of the environment	Min. strength class [a]	Informative examples for exposure classes typical under Danish climate and environment
XD1	Moderate humidity	C35/45	Concrete surfaces exposed to airborne chlorides from de-icing salts, or limited de-icing, or surfaces in the vicinity of de-iced trafficked areas, e.g.: - balconies and stairs with limited access - retaining walls - basement walls above ground - vertical parts of parking structures - bridge structures such as columns, end abutments, retaining walls that are not part of XD3 - bridge decks and tunnels protected with membrane and/or pavement
XD2	Wet, rarely dry	C40/50	Concrete surfaces subjected to long-term chloride containing water exposure, e.g.: - swimming pool structures - concrete components exposed to industrial waters containing chlorides NOTE 1 For swimming pools refer to DS 477, 6.2.2 and 7.2. NOTE 2 If chloride content Cl^- < 1.0 g/l in water, then XD1 applies.
XD3	Cyclic wet and dry	C40/50	Concrete surfaces exposed to direct water spray containing chlorides or de-icing salt, e.g.: - balconies and stairs - parking decks - edge beams for bridge decks with de-icing - columns and end abutments for bridge sub-structure close to trafficked areas NOTE Refer to DS/EN 1992-2 DK NA for specific rules regarding bridges and tunnels.

4 Corrosion of embedded metal induced by chlorides from sea water

Where concrete containing steel reinforcement or other embedded metal is subject to contact with chlorides from sea water or air carrying salt originating from seawater, the exposure shall be classified as follows:

Class	Description of the environment	Min. strength class [a]	Informative examples for exposure classes typical under Danish climate and environment
XS1	Exposed to airborne salt but not in direct contact with sea water	C35/45	Structures near to or on the coast. NOTE If the situation is not investigated in details, normally it can be assumed that this corresponds to a distance of 1 km from sea water on SW-W-N coasts and 200 m from other coasts. On the Danish west coast, the 1 km should be increased to 2 km.
XS2	Permanently submerged	C35/45	Parts of marine structures and structures in sea water, e.g.: - marine structures in harbours, piers, exterior surfaces on submerged tunnels, etc. - buried structures in coastal areas exposed to ground water containing chlorides corresponding to $Cl^- \geq 1.0$ g/l
XS3	Tidal, splash and spray zones	C40/50	Parts of marine structures and structures temporarily or permanently directly above sea water, e.g. – harbour structures exposed to direct spray – bridge piers close above the normal water level
5 Freeze/thaw attack Where concrete is exposed to significant attack by freeze/thaw cycles whilst wet, the exposure shall be classified as follows. A XF-classification is not necessary in cases where freeze/thaw cycles are rare.			
XF1	Moderate water saturation, without de-icing agent.	C30/37	Vertical concrete surfaces exposed to rain and freeze/thaw attack, e.g.: - foundations partly above ground - retaining walls and basement walls - columns, walls and facades
XF2	Moderate water saturation, with de-icing agent.	C35/45	Vertical concrete surfaces exposed to rain and freeze/thaw attack. Same examples as for XF1 but in areas close to trafficked areas with regular de-icing and within reach of airborne chlorides, cf. XS1.

Class	Description of the environment	Min. strength class [a]	Informative examples for exposure classes typical under Danish climate and environment
XF3	High water saturation, without de-icing agent.	C35/45	Horisontal concrete surfaces exposed to rain and freeze/thaw attack, e.g.: - balconies and stairs with limited access - external slabs, decks and beams
XF4	High water saturation, with de-icing agent or sea water.	C40/50	Horisontal concrete surfaces exposed to rain and freeze/thaw attack, e.g.: - balconies and stairs with de-cing - parking structures - bridge structures, edge beams, columns, etc. - marine structures in splash-zone NOTE Freeze/thaw exposure on parking structures can vary significantly depending on location and design.
6 Chemical attack Where concrete is exposed to chemical attack from natural soils and ground water, the exposure shall be classified as follows, cf. Table 6.2 in DS/EN 1992-1-1 for classification limits:			
XA1	Slightly aggressive chemical environment.	C30/37	Concrete exposed to natural soil and ground water, e.g.: - parts of bottom slabs and trenches - foundation piles - tunnels - basement walls - retaining walls
XA2	Moderately aggressive chemical environment.	C35/45	Concrete exposed to natural soil and ground water. Same examples as for XA1. NOTE Concrete exposed to sea water can be classified as XA2, since natural sea water contains SO_4^{2-},
XA3	Highly aggressive chemical environment.	C40/50	Concrete exposed to natural soil and ground water. Same examples as for XA1.
7 Mechanical attack of concrete by abrasion Where concrete is exposed to mechanical abrasion, the exposure shall be classified as follows:			
XM1	Moderate abrasion.	C35/45	Members of industrial sites frequented by vehicles with pneumatic tyres.
XM2	Heavy abrasion.	C35/45	Members of industrial sites frequented by vehicles with pneumatic tyres or solid rubber tyres.

Class	**Description of the environment**	**Min. strength class** [a]	**Informative examples for exposure classes typical under Danish climate and environment**
XM3	Extreme abrasion.	C40/50	Members of industrial sites frequented by fork lifts with elastomer or solid rubber tyres.
NOTE	The examples can be deviated from if it can be demonstrated that a less aggressive exposure can be attributed.		
a	Minimum strength class for durability purpose.		

6.4(1) Exposure resistance classes

(NDP)

The exposure resistance classes, i.e. the content of clause 6.4, are substituted with relevant limiting values and requirements as described in clause 6.5.

6.5.2.1(2) General

(NDP)

The given default values for the increase in concrete cover (Δc_{min}) cast directly against soil, shall be used.

6.5.2.2(1) Minimum cover for durability

(NDP)

Exposure resistance classes designations are not used. For XC exposure classes the minimum covers in Table 6.3 NA shall be used, depending on the exposure conditions, binder combination and concrete requirements given in DS 206. For XD and XS exposure classes the minimum covers in Table 6.4 NA shall be used.

Table 6.3 NA – Minimum concrete cover $c_{min,dur}$ [mm] for carbon reinforcing steel - Carbonation

Exposure classes for carbonation, cf. Table 6.1 NA				
	XC1		XC2, XC3, XC4	
	Design service life [years]			
	50	100	50	1000
Minimum concrete cover [mm]	10	15	20	30

Table 6.4 NA – Minimum concrete cover $c_{min,dur}$ [mm] for carbon reinforcing steel - Chlorides

Exposure classes, cf. Table 6.1 NA						
	Water containing chlorides from sources other than sea water			Chlorides from sea water		
	XD1	XD2	XD3	XS1	XS2	XS3
Design service life [years]	50 *(100)*	50 *(100)*	50 *(100)*	50 *(100)*	50 *(100)*	50 *(100)*
Minimum concrete cover [mm]	30 *(45)*	40 *(60)*	40 *(60)*	30 *(45)*	30 *(45)*	40 *(60)*

6.5.2.2(1) Minimum cover for durability

(NDP)

The given default value for the decrease in concrete cover ($-\Delta c_{min,30}$ = 5 mm) shall be used.

6.5.2.2(3) Minimum cover for durability

(NDP)

The mentioned concrete cover reduction ($-\Delta c_{min,exc}$) depending on the execution shall not be used, i.e. $-\Delta c_{min,exc} = 0$.

6.5.2.2(4) Minimum cover for durability

(NDP)

The given default value for the increase in concrete cover ($\Delta c_{min,p}$ = 10 mm) shall be used.

6.5.2.2(5) Minimum cover for durability

(NDP)

The mentioned concrete cover reduction ($-\Delta c_{dur,red1}$) shall not be used unless documented by testing and supported by relevant experience.

NOTE Experience is generally provided for epoxy membranes – e.g. for parking structures.

6.5.2.2(6) Minimum cover for durability

(NDP)

The given default values for the sacrificial layer ($\Delta c_{dur,abr}$) shall be used.

6.5.2.2(9) Minimum cover for durability

(NDP)

The given default value for the decrease in cover ($\Delta c_{dur,red2} = 0$) shall be used.
If stainless steel is used the requirements in Normative Annex Q.4 shall be applied with the following additions (Table Q.3 is not applied):

– Stainless steel resistance class SSRC1 (Table Q.2) shall only be used for environmental exposure classes XC1, XC2, XC3 and XC4.

– The minimum concrete cover $c_{min,dur}$ shall be 10 mm.

– Stainless steel resistance classes SSRC2 – SSRC4 (Table Q.2) may be used for all exposure classes. The minimum concrete cover $c_{min,dur}$ shall be 10 mm.

NOTE The above-mentioned minimum concrete cover values apply to design service life of 50 years. For a design service life of 100 years they should be increased by +10 mm.

7.2.1.2(6) Representation of imperfections

(NCCI)

For buildings where the bracing system consists of floor diaphragms and shear walls or equivalent truss systems, the following simplified rules may be applied:

The overall effect of geometric imperfections is handled by designing the building for equivalent horizontal loads that act at the centre of gravity of each diaphragm. The load is determined in accordance with DS/EN 1992-1-1:2023, clause 7.2.1.2(6), formula (7.8), replacing ($N_b - N_a$) with the vertical load acting on the relevant diaphragm.

The horizontal load shall, in persistent design situations, be considered to act simultaneously with the design wind load. When assessing the stability of buildings, it may be assumed for each wind direction under consideration that the load from geometric imperfections acts in the same direction as the wind load.

For seismic design situations, the horizontal load from imperfections shall be considered to act simultaneously with the seismic load. It is assumed that the horizontal imperfection load acts in the same direction as the seismic load.

The special analysis corresponding to Figure 7.1 c1) and c2) in DS/EN 1992-1-1:2023, clause 7.2.1.2(6), may be replaced by a verification in which the vertical members are assumed to be horizontally offset between the storeys. The offset shall be at least $\Delta e = h \cdot \theta_1$, where h is the storey height and $\theta_1 = 1/200$. This corresponds to the imperfections being absorbed through moment action in the vertical members, without the appearance of separate internal forces in the overall stabilizing system.

7.3.1(6) Linear elastic analysis

(NCCI)

Formula (7.14) may be used by replacing $E_{t=0}$ with E_{le}, where E_{le} is the internal force or stress due to the total settlement or total shrinkage with no consideration for creep.

7.3.1(7) Linear elastic analysis

(NCCI)

Formula (7.15) may be used by replacing $E_{t=0}$ with E_{in}, where E_{in} is the internal force or stress in case of the initial support conditions.

7.3.3.1(3) General

(NCCI)

Adoption of the theory of plasticity presupposes that the structure has adequate deformation capacity, i.e. yielding in the reinforcement will develop to a sufficient extent before other failure modes such as instability intervene in a progressing, ductile failure. When applying the theory of plasticity, verification of sufficient deformation capacity may be omitted, if the following conditions are fulfilled:

– The distribution of internal forces and moments does not deviate significantly from that corresponding to the theory of elasticity. An accurate calculation of the distribution of internal forces and moments corresponding to the theory of elasticity is not required. It will normally be adequate to apply a qualified estimate or simple approximation methods. For lower bound solutions the following principle may be used: Where the reinforcement area associated with a plastic design at any point of the structure is denoted A_{sP} and the reinforcement area associated with the elastic solution at the same point of the structure is denoted A_{sE}, the above may be assumed to be fulfilled, if $1/3\ A_{sE} \leq A_{sP} \leq 3\ A_{sE}$ in all points of the structure. The elastic solution (assuming fully cracked structure) may be assumed to correspond to the plastic solution where the overall design reinforcement for the structure is a minimum.

– The structure is provided with normal reinforcement, i.e. requirements for minimum reinforcement are fulfilled, and the reinforcement yields at failure.

– Instability is not a pre-condition for the ultimate limit state.

Satisfactory performance of the structure in the serviceability and ultimate limit states may require an arrangement of reinforcement that takes account of the actual distribution of internal forces and moments without redistribution. Where e.g. a plastic solution is adopted disregarding torsional moments in the design, the reinforcement shall be arranged so that it allows for the actual torsional moments, e.g. by using closed links as shear reinforcement in beams and by closing free edges of slabs by U bars.

Satisfactory performance of the structure at the serviceability limit state may require that the distribution of internal forces and moments obtained does not deviate significantly from that determined by the theory of elasticity assuming cracked sections.

Where the action and thus the internal forces and moments depend on the deformation capacity of the structure, e.g. in structures subject to earth pressure, the structural deformation capacity should be assessed. Special consideration should be given to the influence of the deformation capacity on the magnitude of e.g. shear forces and reactions at bearings. For structures where the action at the serviceability limit state is greater than at the ultimate limit state, e.g. in certain structures subject to earth pressure, the serviceability limit state should always be assessed.

7.3.3.1(6) General

(NCCI)

Where a stress-strain curve is used assuming that stress increments occur after the point corresponding to the yield strength, equilibrium as well as compatibility conditions shall be fulfilled.

7.3.3.2(1) Analysis for beams, frames and slabs without verification of rotation capacity

(NCCI)

NOTE The analysis of continuous beams and slabs based on the theory of plasticity may be carried out by verifying that each span can resist the load effects corresponding to the maximum load on the entire span and the minimum load on the entire span, taking into account for both cases the total values of the chosen restraining moments. Restraining moments are chosen between the values found by the theory of elasticity and one third thereof. For continuous beams and slabs of approximately equal spans and uniformly distributed loads, verification of the position of the restraining moments in relation to the theory of elasticity may be omitted, if at restraints and intermediate supports reinforcement is applied for restraining moments, which are taken numerically as not less than 1/3 and not more than twice the design moments in adjacent spans.

7.4.1(3) Second order structural analysis of members and systems with axial force

(NCCI)

NOTE FVB can be determined according to Formula (O.1). For local seconder order effects (isolated members) simplified criteria are given in O.6.

7.4.3.1(1) Methods of analysis

(NCCI)

For stability analysis of individual structural members that are part of buildings braced by floor diaphragms and shear walls, the following applies: In addition to second-order effects, the largest of the following eccentricities must be considered:

- Geometric imperfection determined by:
    - $l_0/400$, if DS/EN 1992-1-1:2023, 7.2.1.2, formulas (7.9) and (7.10) are used.
    - $l_0/200$, if DS/EN 1992-1-1:2023, DK NA, 7.2.1.2, supplementary rules are used.
- Tolerance determined according to either DS/EN 13670 including DK NA, or DS/EN 13369 and the associated harmonized standard. If multiple tolerances occur and it can be argued that the tolerances are statistically independent, the square root method may be used. Individual values correspond to the maximum values over a segment of 1/5 l_0, symmetrically distributed around the area of greatest deflection.

- DS/EN 1992-1-1:2023, 8.1.1(5): the maximum of $h/30$ and 20 mm, where h is the cross-sectional height.

7.4.3.1(2) Methods of analysis

(NCCI)

For stability analysis of individual structural members that are part of buildings with more than one storey and which are not fully braced by floor diaphragms and shear walls, the following applies to the structural members that contribute to the building's stability: In addition to second-order effects, the sum of the eccentricities from the above-mentioned contributions (geometric imperfection and tolerance) must be considered, though at minimum the eccentricity given in DS/EN 1992-1-1:2023, 8.1.1(5) shall be applied.

8.2.1(2) General verification procedure

(NCCI)

Paragraph (2 NA) shall replace (2).

(2 NA) When, on the basis of design shear calculation, no shear reinforcement is required, minimum shear reinforcement may nevertheless be necessary for linear members, according to Clause 12.

For linear members in statically determinate structures with d > 500 mm, minimum shear reinforcement shall be provided.

8.2.1(3) General verification procedure

(NCCI)

For circular cross section, z should be determined according to 8.2.3(9).

8.2.2(3) Detailed verification for members without shear reinforcement

(NCCI)

Alternatively, as a simplified approximation to Formulae (8.30), a_{cs} may be taken as the distance from the edge of the support to the section considered. The solution requires that direct support is used and that the reinforcement is fully and sufficiently anchored at the support.

8.2.3(11) Members with shear reinforcement

(NCCI)

The second paragraph shall be replaced by:

For members subjected to high design axial compressive forces, N_{Edw} (negative value) should be chosen so that the depth of the compression chords (i.e. x) carrying $0{,}5(N_{Ed} - N_{Edw}) + 0{,}5N_{Vd}$ and $-M_{Ed}/z$ is not higher than 0,25d.

8.2.6(1) Shear at interfaces

(NCCI)

The minimum reinforcement crossing the interface is determined by:

$$\rho = \frac{0{,}02 f_{cd} - \sigma_n}{f_{yd} sin\alpha} \geq 0 \qquad (8.a\ NA)$$

(1) The minimum reinforcement may be concentrated and does not need to be distributed along the interface, provided that the structural elements on both sides of the interface have sufficient strength to distribute the effect of the concentrated minimum reinforcement as an equivalent, uniform pressure along the interface.

(2) When the interface is kept effectively together by minimum reinforcement, Clause 8.2.6(8) as well as note a) in Table 8.2 may be disregarded and the coefficients specified in Table 8.2 may be assumed to apply. Otherwise, conservative values are to be determined.

8.2.6(5) Shear at interfaces

(NCCI)

γ_c in Formula (8.76) is the partial coefficient for the tensile strength of concrete.

8.2.6(7) Shear at interfaces

(NCCI)

γ_c in Formula (8.77) is the partial coefficient for the tensile strength of concrete.

8.5.1(1) General

(NCCI)

Methods to obtain stress fields for design may e.g. include the Stringer Method, the Strut-and-tie models, and the Homogeneous Triangular Stress Fields.

Where the Stringer Method is used, the following should be fulfilled:

- The width of the stringers should not exceed 20% of the width of the adjacent shear panel that has the smallest dimension perpendicular to the stringer's longitudinal direction.

- To carry tensile forces in the stringers, the necessary reinforcement is provided. The variation in the tensile force in a stringer should not exceed what corresponds to the stringer force increasing from zero to the design yield force over a length equal to the anchorage length.

- To carry compression in the stringers, concrete as well as reinforcement may be utilized. Compressive concrete stresses in stringers should not exceed f_{cd} and the reinforcement should not carry more than a maximum of 50% of the compressive stringer force. If the reinforcement carries more than half of the design compressive force that the concrete part of the stringer can carry, lap splices must not be used.

The shear panels are designed according to Annex G and the shear reinforcement is effectively anchored at the centre of the stringers.

8.5.5(3) Transfer of concentrated forces into a member

(NCCI)

Smeared stress fields, e.g. calculated by use of the Stringer Method, may also be used to determine the transverse reinforcement.

9.1(3) General

(NCCI)

It is not possible to give precise information on when the first crack appears. Experience has shown that it occurs for concrete tensile stresses in the order of 85% of the mean uniaxial tensile strength of concrete, i.e. at an effective uniaxial tensile strength of $f_{ct,eff} = 0{,}85 \cdot 0{,}94\ f_{ctm} = 0{,}8f_{ctm}$.

[Bemærkning fra Arbejdsgruppen: The value 0,85 is under consideration and awaits experiments to be carried during the first half of 2026.]

9.1(4) General

(NCCI)

For short-term loading of concrete linear elastic behaviour may be assumed for compressive stresses up to $0{,}6f_{cm}$.

9.2.1 General considerations

(NPD)

Table 9.1 is unchanged, but for characteristic load combinations, the specified limit $0{,}8f_{yk}$ in table 9.1 may be ignored if it is demonstrated that unacceptable cracking or deformation do not occur.

NOTE Use of redistribution of forces in the ultimate limit state can lead to large stresses or yielding in the reinforcement in the serviceability limit state.

9.2.2(2) Minimum reinforcement areas to avoid yielding

(NCCI)

In formulae (9.2 – 9.4) the effective tensile strength of concrete should be taken as $f_{ct,eff} = f_{ctm}$.

If the requirements for minimum reinforcement according to Clause 12.2 are fulfilled, the specified requirements for minimum reinforcement in this paragraph will also be fulfilled.

9.2.2(4) Minimum reinforcement areas to avoid yielding

(NCCI)

In Formulae (9.7) the effective tensile strength of concrete should be taken as $f_{ct,eff} = f_{ctm}$.

9.2.3(2) Refined control of cracking

(NCCI)

Expression (9.8) applies to the fine crack system as shown a) in Figure 9.a NA, where the cracks are localized locally at the reinforcement.

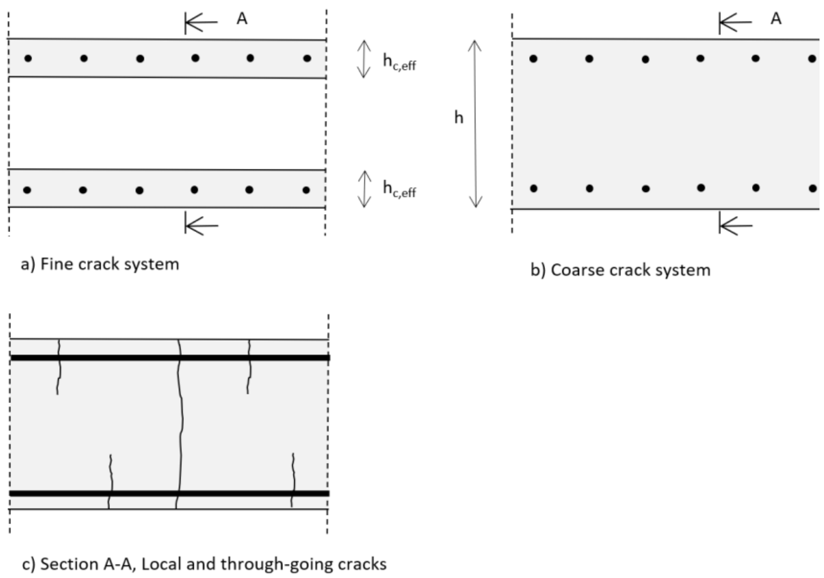
For through-going cracks in thick members (i.e. the coarse crack system), cf. c) in Figure 9.a NA formula (9.8) underestimates the crack width at the member surface.

For the through-going cracks, the crack width $w_{k,cal}$ in pure tension can approximately be calculated by multiplying the right-hand side of expression (9.8) by a factor of 0,5, and by inserting into formulas (9.11) and (9.15) following parameters:

$A_{c,eff}$ = the area of the entire cross-section

A_s = the area of all reinforcement symmetrically placed within $A_{c,eff}$, cf. b) in Figure 9.a NA.

Figure 9.a NA - Fine crack system



9.2.3(3) Refined control of cracking

(NCCI)

The effective tensile strength of concrete in Formula (9.11) should be taken as $f_{ct,eff} = 0,94f_{ctm}$, where f_{ctm} is defined in Table 5.1.

For the crack formation phase, see Annex S.

9.2.3(4) Refined control of cracking

(NCCI)

The effective tensile strength of concrete in Formula (9.13) should be taken as $f_{ct,eff} = 0{,}94f_{ctm}$, where f_{ctm} is defined in Table 5.1

(NCCI)

For the crack formation phase, see Annex S.

9.3.4(4) General method for deflection calculations

(NCCI)

In Equation (9.30) f_{ctm} should be replaced by $0{,}94f_{ctm}$.

(NCCI)

For the crack formation phase, see Annex S.

10 Fatigue

(NCCI)

Awaiting.

11.4.2(2) Anchorage of straight bars

(NDP)

Values in Table 11.1 should be changed as follows:

Values for l_{bd}/ϕ in Table 11.1 applies to good bond conditions and should be reduced by a factor $k_{lb}/50$. However, not less than 10ϕ.

11.4.2(3) Anchorage of straight bars

(NDP)

In Formula (11.3) k_{lb} is replaced by $25\gamma_c$, where γ_c is the partial coefficient for the tensile strength of concrete according to table 4.3 NA.

11.4.7(2) Anchorage of headed bars in tension

(NCCI)

γ_c in Formula (11.8) is the partial coefficient for the tensile strength of concrete.

11.5.2(2) All types of laps

(NDP)

k_{ls} = 1,60

11.7(3) Deviation forces due to curved tensile and compressive chords

(NCCI)

NOTE γ_c in Formula (11.24) is the partial coefficient for the tensile strength of concrete.

11.7(4) Deviation forces due to curved tensile and compressive chords

(NCCI)

NOTE γ_c in Formula (11.25) is the partial coefficient for the tensile strength of concrete

12.3.1(1) Beams, General

(NDP)

Table 12.1 NA — Detailing requirements for reinforcement in beams

	Description	Symbol	Requirement
1	Minimum longitudinal reinforcement, in those parts of the section where tension may occur	$A_{s,min}$	12.2(2), see also 12.2(3), 12.2(6). If subjected to pure tension, Formulae (12.2) is used with f_{ctm} being replaced by 0,8f_{ctm} and with f_{ctm} determined according to Table 5.1
2	Minimum shear and transverse torsional reinforcement, when required. Minimum torsion reinforcement should be provided to the full perimeter including features not counted part of the thin walled section	$\rho_{w,min}$	Formulae (12.4) for Class A reinforcement. 20% reduction is allowed for Class B and Class C reinforcement.
3	Minimum bottom reinforcement at inner supports taking account of unforeseen effects leading to positive moments at the support, e.g. unforeseen settlement, or load reversal due to explosion		0,25 $A_{s,req\ span}$

4	Minimum bottom reinforcement for end supports		$0{,}25A_{s,req\ span}$
5	Maximum longitudinal spacing of shear assemblies/stirrups[a]	$s_{l,max}$	$0{,}75d\ (1 + \cot\alpha)$
6	Maximum longitudinal spacing of bent-up bars[a]	$s_{bu,max}$	$0{,}6d\ (1 + \cot\alpha)$
7	Maximum transverse spacing of shear legs[a]	$s_{tr,max}$	$0{,}75d \leq 600$ mm
8	Minimum ratio of shear reinforcement in the form of stirrups with respect to the required reinforcement ratio (taking account of unforeseen effects e.g. compatibility torsion)	$\rho_{w,stir}$	$\geq 0{,}5\rho_{w,req}$
9	Minimum ratio of torsion reinforcement in the form of closed stirrups with respect to the required reinforcement ratio	$\rho_{w,stir}$	$\geq 0{,}2\rho_{w,req}$
10	Maximum spacing for torsion assemblies/stirrups (u defined in 8.3.2(2))	$s_{stir,max}$	$u/8 \leq \min\{b;\ h\}$
11	Minimum area and spacing of longitudinal surface reinforcement in beams with downstand ≥ 600 mm to avoid coarse cracks in SLS	$A_{s,web}$ $s_{l,surf,max}$	9.2.2(4) 300 mm
12	Minimum transverse reinforcement in flanges (those part of flanges where tension in the transverse direction may occur)	$A_{st,min}$	12.2(2) see 8.2.5, Figure 8.13. In Formulae (12.2) f_{ctm} is replaced by $0{,}8f_{ctm}$, where f_{ctm} is determined according to Table 5.1

[a] These spacings are consistent with the shear model in 8.2.3. Where alternative models are used alternative spacings may be required.

The first crack in a reinforced concrete structure subjected to pure tension is assumed to occur at $f_{ct,eff} = 0{,}8f_{ctm}$, see 9.1(3)NA.

The phase where all cracks have formed is assumed to occur at a tensile stress in the concrete that is approximately 1,4 times greater, i.e. at 1,4*0,8 = 1,12 times the uniaxial tensile strength. A controlled crack formation is assumed to occur for a reinforcement that corresponds at least to $A_{s,min} = A_c \cdot 0{,}8\ f_{ctm}/f_{yk}$, based on the direct tensile strength of the concrete being 0,94 f_{ctm}, cf. 5.1.3(3)NA and 9.1(3)NA.

12.4.1(1) Slabs, General

(NDP)

Table 12.2 NA— Detailing requirements for reinforcement in slabs

Description		Symbol	Requirement
1	Minimum longitudinal reinforcement in those parts of the cross-section where tension may occur	$A_{s,min}$	12.2(2) see also 12.2(3), (6). If subjected to pure tension, Formula (12.2) is used with f_{ctm} being replaced by $0{,}8f_{ctm}$ and with f_{ctm} determined according to Table 5.1
2	Minimum shear reinforcement, when required	$\rho_{w,min}$	12.2(4), Formulae (12.4) for Class A reinforcement. 20% reduction is allowed for Class B and Class C reinforcement. 12.4.2(3)
3	Minimum secondary reinforcement[a]		$0{,}2A_{s,req,span}$[c]
4	Minimum longitudinal bottom reinforcement at inner supports, taking account of unforeseen effects at supports		$0{,}25A_{s,req,span}$[c]
5	Minimum longitudinal bottom reinforcement at end supports		$0{,}25A_{s,req,span}$[c]
6	Minimum top reinforcement at end supports in buildings, without bearings where unintentional restraint may occur. The reinforcement should extend 0,2 length of span from the end support.		$0{,}25A_{s,req,span}$[c] (but $\geq A_{s,min}$ according to 12.2(2))
7	Maximum spacing of bars for concrete in tension	$s_{slab,max}$	$3h \leq 400$ mm
8	Maximum longitudinal spacing of shear assemblies/stirrups[b]	$s_{l,max}$	$0{,}75d \cdot (1 + \cot\alpha)$
9	Maximum longitudinal spacing of bent-up bars[b]	$s_{bu,max}$	d
10	Maximum transverse spacing of shear legs[b]	$s_{tr,max}$	$1{,}5d$
11	Along a free (unsupported) edge, a slab should normally contain longitudinal and transverse reinforcement, generally arranged as shown in Figure 12.4 in order to accommodate torsional moments near the edge.		

[a] To ensure a minimum ability to locally redistribute sectional forces transverse to the span direction, secondary reinforcement is to be placed in areas of slabs which can be considered to behave as one-way slabs.

[b] These spacing are consistent with the shear model in 8.2.3. Where alternative models are used alternative spacings may be required.

[c] $A_{s,req\ span}$ is the required reinforcement for positive bending moments at the span.

12.6(1) Columns

(NDP)

Table 12.3 NA - Detailing requirements for reinforcement in columns

	Description	Symbol	Requirement
1	Minimum amount of longitudinal reinforcement for robustness and to avoid risk of compressive yielding of reinforcement due to creep and shrinkage in SLS When all longitudinal reinforcement is prestressed the $0{,}1N_{\mathrm{Ed}}/f_{\mathrm{yd}}$ limit may be ignored.		*max* $\max\left\{0{,}1\frac{N_{\mathrm{Ed}}}{f_{\mathrm{yd}}};\ 0{,}002A_{\mathrm{c}}\right\}$
2	Minimum number of longitudinal bars[a]: — polygonal cross-section — circular cross-section		 1 at each corner with a spacing ≤ 500 mm 6 evenly distributed with a spacing ≤ 500 mm
3	Maximum longitudinal spacing of transverse reinforcement (stirrups/hoops) for columns with dimensions *h* and *b*: — intermediate region between the two end regions[b] — intermediate region between the two end regions, when longitudinal bars are not accounted for column resistance — end regions, over a length equal to the larger dimension of the column. For concrete with $f_{\mathrm{ck}} > 50$ MPa the transverse reinforcement shall provide a minimum confinement of $k \cdot f_{\mathrm{cd}}$ in accordance with 8.1.4, Formulae (8.11) to (8.14)[d] — at lap area where $\phi_{\mathrm{l}} \geq 14$ mm	 $s_{\mathrm{max,col}}$ $s_{\mathrm{max,col}}$	 $20\phi_{\mathrm{l,max}}$[c] $\leq \min\{h;\ b;\ 300\ \mathrm{mm}\}$ $\min\{h;\ b;\ 400\ \mathrm{mm}\}$ $0{,}6s_{\mathrm{max,col}}$ $0{,}6s_{\mathrm{max,col}}$
4	Minimum bar diameter for transverse reinforcement (bars in stirrups, wires in welded mesh)		$\geq 0{,}25\phi_{\mathrm{l,max}}$[a]

a) For constructability, the diameter of longitudinal bars $\phi_{\mathrm{l,max}}$ should be at least 12 mm.

b) Where all bars are prestressed a spacing of $\min\{h;\ b;\ 300\ \mathrm{mm}\}$ may be used.

c) $\phi_{\mathrm{l,max}}$ – maximum diameter of longitudinal bars.

d) This requirement is to provide a minimum level of ductility to higher strength concrete columns. $k = 0{,}02$ unless the National Annex gives a different value.

12.7(2) Walls and deep beams

(NDP)

Table 12.4 NA — Detailing requirements for reinforcement in walls and deep beams

	Description	Symbol	Requirement
1	Minimum amount of vertical reinforcement (each surface): – where the member carries in-plane normal and shear stresses and designed/verified by use of 8.5 or Annex G – where the member is only loaded by vertical in-plane compression and out of plane bending	$A_{s,min,v}$	$0{,}25A_c\dfrac{f_{ctm}}{f_{yk}}$ $0{,}001A_c$
2	Minimum amount of horizontal reinforcement (each face): – where the member carries in-plane normal and shear stresses and designed/verified by use of 8.5 or Annex G – where the member is only loaded by vertical in-plane compression and out of plane bending	$A_{s,min,h}$	$0{,}25A_c\dfrac{f_{ctm}}{f_{yk}}$ $0{,}25A_{s,v.}$ However, not less than $0{,}00025A_c$
3	Maximum spacing of vertical reinforcement		min{$3h^a$; 400 mm}
4	Maximum spacing of horizontal reinforcement		400 mm
5	Maximum spacing of orthogonal-to-the-surface reinforcement where $A_{s,v}$ exceeds $0{,}02A_c$ and is utilised in compression (end region is taken as $\geq 4h^a$)		see 12.7(3)
a	h – thickness of wall		

12.9.3(1) Required resistances for ties

(NDP)

Awaiting

13.4.1(1) General

(NCCI)

Detailed analysis of the influence of deformability may be omitted for a) traditional element connections, such as wall-to-wall connections, fulfilling the requirement to minimum reinforcement specified in the supplementary rules to 8.2.6(1); b) connections between hollow core slabs.

13.7.1(15) General

(NCCI)

(1) For construction joints (connections) where the reinforcement through the joint consists of overlapping U-bar loops arranged in pairs, such as joints between prefabricated concrete elements, the design strength of the interfaces may be verified according to 8.2.6(5), Formulae (8.76), provide that the strength of the U-bars is determined as the lowest of the following values:

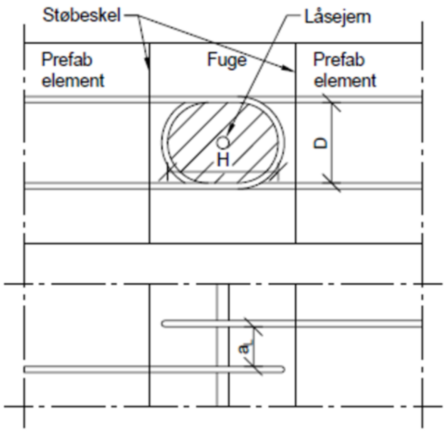
– The design yield strength of the reinforcement.

– The design lap strength of the U-bar loops embedded in the joint concrete (grout)

(2) The design lap strength may be calculated according to DS/EN 1992-1-1:2023, 11.5.4.

(3) Where the layout complies with the illustration in Figure 13.a NA, the design lap strength of the U-bar loops may also be determined by including the confining effects from the transverse pressure supplied by the lock bar (lacer bar). This pressure is considered to be distributed over the area enclosed by the U-bar loops, and the resultant of this pressure shall not exceed the design capacity of the locking bar, taking into account the actual anchorage conditions. In the calculation, the enclosed area shall be taken as $0{,}25\pi/D^2$. Verification of concrete crushing within the loop and verification of concrete shearing in a plane section between adjacent U-bar loops may be performed with utilization of the strength increase specified in DS/EN 1992-1-1:2023, 8.1.4, Formulae (8.9) & (8.10). For simplification, Formulae (8.10) may be replaced by a conservative, linear curve.

Figure 13.a NA - Layout off a construction joint with tolerance demands



$$0{,}8D \le H \le 1{,}2D$$

$$a_L \le \min\begin{cases} 3\phi \\ 30\ mm \end{cases}$$

13.7.1(16) General

(NCCI)

Use of smooth U-bars in construction joints is under preparing.

14.2(1) Concrete

(NDP)

$k_{c,pl} = 1{,}0$ and $k_{t,pl} = 1{,}0$

Annex D Evaluation of early-age and long-term cracking due to restraint (Informative)

Annex F Safety formats for non-linear analysis (Informative)

Annex F NA Safety formats for non-linear analysis (Informative)

F.1(1) Use of this annex

Annex F is informative with the supplementary requirements specified in this DK NA.

F.2(1) Scope and field of application

(NCCI)

Only the partial factor method may be used

F.3(6) General

(NCCI)

(6) The design value of the structural resistance may be determined by use of the following safety format:

- Partial factor method (PFM) as described in F.4

F.4(1) Partial factor method (PFM)

(NCCI)

Clause F.4(1) is not applicable

F.4(2) Partial factor method (PFM)

(NCCI)

(2 NA) In case the concrete tensile strength is neglected, the design value of the resistance may be calculated on the basis of values f_{cd}, f_{yd} and f_{pd} according to 5.1.6, 5.2.4 and 5.3.3(1). The partial safety factors γ_c and γ_s in Table 4.3 may be used, provided that the statistic values of the uncertaintiess of the structural model, γ_2, are similar or more favourable than the values adopted for the safety factors in Table 4.3, see NA to DS/EN 1990, Annex FF NA. Otherwise, the uncertaintiess of the structural model, γ_2, to be used shall be determined according to the method in NA to DS/EN 1990, Annex FF NA.

F.5 Global factor method (GFM)

Clause F.5 is not applicable.

F.6 Full probabilistic method

Clause F.6 is not applicable.

F.7 Model uncertainty

Clause F.7 is not applicable.

Annex G Global factor method (Normative)

G.5(1) Refined control of cracking in membrane elements in SLS

(NCCI)

NOTE The supplementary information to Clause 9.2.3 for calculation of crack widths of the fine- and the coarse crack system are also applicable to membrane elements

Annex H Guidance on design of concrete structures for water-tightness (Informative)

Annex I Assessment of existing structures (Not applicable)

Annex J Strengthening of Existing Concrete Structures with CFRP (Not applicable)

Annex K Bridges (Normative)

Awaiting.

Annex L Steel Fibre Reinforced Concrete Structures (Informative)

This informative Annex L NA provides following changes to the informative annex L.

Annex L is not compliant with the Danish safety format. Accordingly, the annex may only be used if it can be demonstrated - based on documentation or relevant testing - that there is compliance with the Danish safety format.

L.2 Scope and field of application

(1) This Informative Annex covers the design of concrete structures which comprise steel fibre reinforced concrete with reinforcing steel, pre-tensioning or post-tensioning tendons.

NOTE Steel fibre reinforced concrete structures without reinforcing steel, pre-tensioning or post-tensioning tendons may be designed using this annex provide that the structure works predominantly in compression and provide that sufficient experience exists, e.g. precast segmental tunnel lining and cast in-situ tunnel lining.

L.4(1) Basis of design – Partial Factors for materials

Awaiting

L.5.1 Properties

(1) Residual strengths………according to Table L.2.

NOTE 3 Strength values found on the basis of EN 14651 are average values. Conversion to characteristic values shall be performed in compliance to The Danish National Annex to EN 1990. See also L.15.

L.5.5.2(1) Stress-strain relation for structural analysis

(NDP, note 1)

The value of ϵ_{Ftud} is 0,02, unless a different value is determined by testing, for example assuming a larger CMOD in tests as per EN 14651 for the ULS residual strength.

L.6 (4) Durability – Minimum cover

(NDP)

$k_{dur} = 0{,}50$.

L.7(1) Structural analysis

(1) Plastic analysis of SFRC structures without any direct check of rotation capacity may be used for the ultimate limit state analysis of the following structure types, when complying with Clauses 7.3.3.1, 7.3.3.2 and/or 8.5.1(2):

- Statically determinate foundations and slabs supported directly on ground.
- For statically indeterminate foundation elements on piles (rafts and slabs) and elevated slabs with reinforcing steel and with steel fibres with ductility class of at least c according to Table 1.2, subject to the following conditions:
  - two-way systems with $L_x/L_y \leq 1{,}5$ and $A_s \geq A_{s,min}$ as defined in Clause L.12 (Lx>Ly);
  - one-way systems and two-way systems with $L_x/L_y > 1{,}5$ and $A_s \geq a_{duct} \cdot A_{s,min}$ with $A_{s,min}$ as defined in Clause L.12 (Lx>Ly).

(NDP)

The value $a_{duct} = 2{,}0$ applies.

L.8.2.2 Detailed verification for members without shear reinforcement

(2) When, on the basis of the design shear calculation, no shear reinforcement is required, minimum shear reinforcement may nevertheless be necessary for linear members, according to Clause 12. For linear members in statically determinate structures with d > 500 mm, minimum shear reinforcement shall be provided.

L.8.2.3 Members with shear reinforcement (Not applicable)

Annex L.8.2.3 shall not be a part of DK NA and shall not be used.

L.8.3 Torsion – Torsional resistance of compact or closed sections (Not applicable)

Annex L.8.3 shall not be a part of DK NA and shall not be used.

L.8.4.2 Punching shear resistance of SFRC slabs with shear reinforcement (Not applicable)

Annex L.8.4.2 shall not be a part of DK NA and shall not be used.

L.8.5 Design with strut-and-tie models – Ties (Not applicable)

Annex L.8.5 shall not be a part of DK NA and shall not be used.

L.8.6 Partial loaded areas

(1) Fibres of ductility class c, d and e may be used to replace the transverse reinforcement required to resist transverse tensile stresses arising from action effects in partially loaded areas. For the design, the idealised tensile post cracking behaviour (see L.5.5.2 and L.8.1) may be used.

L.9.3 Refined control of cracking

Note 1 and Note 2 shall not be used in Denmark.

Formulas (L.26) and (L.27) does not apply to membrane elements or beams and slabs subjected to torsional moments.

L.11.2 Spacing of bars

NOTE 3 The requirements to spacing of bars in clause 12 apply also to steel fibre reinforced concrete structures.

L.12.1 Minimum reinforcement rules

(1) In members reinforced with steel fibres with or without axial force, minimum reinforcement shall be provided so that:

$$M_{R,min}(N_{Ed}) \geq M_{cr}(N_{Ed}) \tag{L.29 NA}$$

where

$M_{R,min}$ is the bending strength of the section with $A_{s,min}$ in presence of the co-existing axial force N_{Ed}. The effects of the fibres shall not be included in the calculation of $M_{R,min.}$

(2) In members subjected to axial tension, $A_{s,min}$, shall meet the following requirement:

$$M_{R,min} \geq N_{cr} \tag{L.30 NA}$$

Similarly as above the effects of fibres shall not be included in the calculation of $N_{R,min.}$

(3) The minimum shear reinforcement ratio $\rho_{Fw,min}$ for members reinforced with steel fibres requiring shear or torsion reinforcement may be taken as:

$$\rho_{Fw,min} = \rho_{w,min} \tag{L.31 NA}$$

where

$\rho_{w,min}$ is determined according to Table 12.1 or Table 12.2.

(4) The minimum torsion reinforcement ratio $\rho_{Fw,min}$ for members reinforced with steel fibres requiring longitudinal and transverse reinforcement may be determined fromFormulae (L.31).

L.12.2.2 Shear and torsion reinforcement

(1) The shear and torsion reinforcement may not be replaced by steel fibres.

L.12.3.1 (2) General

See L.12.3.1 NA

L.12.3.1 General

(1) The longitudinal reinforcement in slabs, where the ductility class according to Table L.2 is at least c, may be partly replaced by steel fibres, subject to the provisions in L.12.1 and provided that the cross section has at least the same rotation capacity as in the case where replacement is not considered.

(2) The minimum longitudinal tensile reinforcement in slabs should not be replaced by steel fibres

(3) The minimum secondary tensile reinforcement in one-way slabs should not be replaced by steel fibres.

L.12.3.2(1) hear reinforcement

(1) The shear reinforcement in slabs, if required, shall not be replaced by steel fibres.

L.12.4(1) Wall and deep beams

Vertical and horizontal minimum reinforcement in walls and deep beams should comply with the provisions in Table 12.4.

L.14 Lightly reinforced SFRC structures (Not applicable)

Annex L.14 shall not be a part of DK NA and shall not be used.

Annex N Recycled aggregates concrete structures (Informative)

Annex N NA Recycled aggregates concrete structures (Informative)

Annex N is informative with the supplementary requirements in annex N NA

Additional choice is given in Table N.1

Table N.1, Row 2:

$f_{ck} \leq 60$ MPa

Annex O Simplified approaches for second order effects (Informative)

Annex O NA Simplified approaches for second order effects (Informative)

Annex O is informative with the supplementary requirements in annex O NA.

O.1(2) Use of this Annex

Annex O, Clauses O.7.1 – O.7.3 do not apply.

O.3(1) Critical load of building structures

EI_i is the bending stiffness of each the bracing member no. *i*

f_{ri} is the rotational restraint stiffness at the base of bracing member no. *i*

$$f_{ri} = \frac{\theta}{M} \cdot \frac{EI_i}{L} \qquad \text{(O.5 NA)}$$

O.4(1) Critical load of isolated members

(2) The following model may be used to estimate the representative effective stiffness of slender compression members with arbitrary cross section:

$$EI = K_c E_{cd} I_c + K_s E_s I_s \qquad \text{(O.a NA)}$$

where

E_{cd}	is the design value of the modulus of elasticity of concrete, see DS/EN 1991-1-1:2023, 7.4.3.3.
I_c	is the moment of inertia of concrete cross section
E_s	is the design value of the modulus of elasticity of reinforcement, see DS/EN 1991-1-1:2023, 7.4.3.3.
I_s	is the second moment of area of reinforcement, about the centre of area of the concrete
K_c	is a factor for effects of cracking, creep etc.
K_s	is a factor for contribution of reinforcement

(3) The following factors may be used for determining the representative effective stiffness, provided r ≥ 0,002:

$$K_s = 1 \qquad \text{(O.b NA)}$$

$$K_c = \frac{k_1 k_2}{(1 + \varphi_{eff,b})} \qquad \text{(O.c NA)}$$

where

ρ	is the geometric reinforcement ratio, A_s/A_c.
A_s	is the total area of reinforcement
A_c	is the area of concrete section
$\varphi_{eff,b}$	is the effective creep coefficient, see DS/EN 1991-1-1:2023, 7.4.2(2), Formula (7.27)

$$K_1 = \frac{f_{ck}}{20}\,(f_{ck}\text{ in MPa}) \quad \text{(O.d NA)}$$

$$K_2 = n \cdot \frac{\lambda}{170} \leq 0{,}20 \quad \text{(O.e NA)}$$

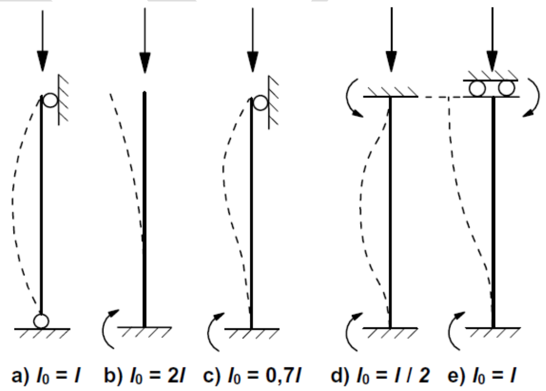
where

n	is the relative axial force, N_{Ed} / $(A_c f_{cd})$
λ	is the slenderness ratio, see DS/EN 1991-1-1:2023, O.5.

O.4(1) Critical load of isolated members

(3) Examples of effective length for isolated members with constant cross section are given in Figure O.a NA.

Figure O.a NA — Examples of different buckling modes and corresponding effective lengths for isolated members



O.7.4 Simplified analysis of isolated members based on nominal curvature

For simply supported or clamped members with cracked cross sections, the second order deflection, e_2, may conservatively be determined as follow:

$$e_2 = \frac{1}{10}\frac{\epsilon_{cu1} + \epsilon_{yd}}{d} l_0^2 \quad \text{(O.f NA)}$$

where

ϵ_{cu1}	is determined according to DS/EN1992-1-1:2023, Formula (5.10)
ϵ_{yd}	is f_{yd}/E_s, see DS/EN 1992-1-1:2023, Fig. 5.2
l_0	is the effective length, see Figure O.a NA a) and b)

Annex P Alternative cover approach for durability (Not applicable)

Annex R Embedded FRP reinforcement (Not applicable)

Annex S - Minimum reinforcement for crack control and simplified control of cracking (Informative)

Annex S NA - Minimum reinforcement for crack control and simplified control of cracking (Informative)

S.1(1) Use of this Annex

Annex S applies with the supplementary requirements specified in this annex S NA.

S.2(1) Scope

This formulation can be used to determine the base reinforcement in structural members or parts of structural members subjected to tension.

S.3(1) Minimum reinforcement areas for crack width control

(1) Regardless of analysis, fulfilment of a specific crack width may require a minimum amount of reinforcement that exceeds the minimum reinforcement determined according to Clauses 9.2.2 and 12.2. This amount of reinforcement is denoted minimum reinforcement for crack width control. The minimum reinforcement according to Clauses 9.2.2 and 12.2 only ensures a controlled crack formation.

As a simplification, the crack width can be considered constant for the phase where cracks are formed. For the phase where all cracks have formed, i.e. the stabilized phase, the crack width is determined by the formulas in Clause 9.2.3.

(2) Minimum reinforcement to ensure that the crack width requirement $w_{\mathrm{lim,cal}}$ is met is for a tension zone determined by:

$$\rho_{min,Wlim} = \sqrt{\frac{\phi f_{ct,eff}}{4E_{sk}k_{fc}w_{\mathrm{lim,cal}}}} \qquad \text{(S.a NA)}$$

where

$f_{ct,eff}$ formally is taken as $0{,}5\sqrt{0{,}1f_{ck}}$. For the fine crack system k_{fc} = 1 is assumed and for the coarse crack system, k_{cc} = 2 is assumed, cf. Figure 9.a NA.

(3) For the fine crack system, the effective concrete area $A_{c,eff}$ is the sum of the largest concrete area with a centre of gravity that coincides with the centroid of the reinforcement, however, with a thickness not greater than ½*h*.

NOTE It should be noted that if the specified minimum reinforcement is not provided and a crack forms, the crack width will be larger than the prescribed requirement $w_{lim,cal}$.

The reinforcement given by (S.a NA) may be applied in cases where a structure or parts thereof to some extent is restrained against deformations due to shrinkage and/or temperature variations, and where joints are not provided to prevent cracking, or where any subsequent repair of single cracks of considerable widths is unacceptable.

S.4 Simplified control of cracking

Annex S, S.4(1) and S.4(2) do not apply.