Date: 20XX -XX

Høringsudgave 01.07.2026

DS/EN 1990:2023 DK NA:202x

National Annex to: Eurocode 0 – Basis of structural and geotechnical design

National Anneks til: Eurocode 0 – Grundlag for strukturelt og geoteknisk design

Foreword

Published [måned og år].

This National Annex (NA) to DS/EN 1990:2023 forms part of the Danish national annexes to the second generation of Eurocodes.

This NA supersedes DS/EN 1990 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 1990:2023 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 Non-Contradictory Complementary Information (NCCI);
- National choice for 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 National Determined Parameter (NDP) are possible, and if a Danish value is defined, and the applicable or not applicable informative annexes. Furthermore, clauses providing complementary information (NCCI) are identified in the second list below.

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

Clause	**Subject**	**National Determined Parameter**	**Non-Contradictory Complementary Information**
3.1.2.35	Terms relating to design		Complementary information
3.1.3.31	Terms relating to actions		Complementary information
4.2(3)	Structural reliability	National choice	
4.3(1)	Consequences of failure	Unchanged	
4.4(1)	Robustness		Complementary information
4.4(2)	Robustness	National choice	
4.7(1)	Sustainability	National choice	
6.1.3.2(4)	Water actions	Unchanged	
6.1.3.2(6)	Water actions	Unchanged	
7.1.5(7)	Fire design	National choice	
8.3.1(2)	Verifications of ultimate limit states (ULS) – General Ultimate limit states caused by excessive deformations		Complementary information *awaiting information*
8.3.1(4)	Verifications of ultimate limit states (ULS) – General		Complementary information
8.3.2.1(4)	Verification of ultimate limit states (ULS) - Design values of the effects of actions	Unchanged	
8.3.3.1(5)	Permanent actions	National choice	
8.3.3.6(1)	Fatigue actions	National choice	

Clause	Subject	National Determined Parameter	Non-Contradictory Complementary Information
8.3.4.2(2)	Combination of actions for persistent and transient	National choice	
A.1.3(1)	Consequence classes	National choice	
A1.3	Consequence classes Additional paragraphs (2) – (11)		Complementary information
A.1.4(1)	Design service life	Unchanged	
A.1.6.1(1)	Ultimate limit states (ULS)	National choice	
A.1.6.1(2)	Ultimate limit states (ULS)	Unchanged	
A.1.6.2(1)	Serviceability limit states (SLS)	Unchanged	
A.1.6.3(1)	Combination factors	National choice	
A.1.6.3(2)	Combination factors	National choice see A.1.6.3 (1)	
A.1.7(1)	Partial factors for ultimate limit states (ULS)	National choice	
A.1.7(5)	Partial factors for ultimate limit states (ULS)		Complementary information
A.1.7(6)	Partial factors for ultimate limit states (ULS)		Complementary information
A.1.8.1(1)	Serviceability for buildings	National choice	
A.1.8.2.2(2)	Vertical deflections	National choice	
A.1.8.2.3(2)	Horizontal displacements	Unchanged	
A.1.8.3(1)	Vibrations	Unchanged	
A.1.8.3(3)	Vibrations	National choice	
A.1.8.3(4)	Vibrations	National choice	
A.1.8.4(2)	Limiting foundation movements (Table A.1.16)	Unchanged	
A.1.8.4(4)	Limiting foundation movements (Table A.1.13, Table A.1.14, Table A.1.15)	Unchanged	
A.1.8.4(4)	Limiting foundation movements		Complementary information

Clause	Subject	National Determined Parameter	Non-Contradictory Complementary Information
A.1.8.4(5)	Limiting foundation movements		Complementary information
A.2.3(1)	Consequence classes	See NA for bridges	
A.2.4(1)	Design service life	See NA for bridges	
A.2.7.1(1)	Ultimate limit states (ULS)	See NA for bridges	
A.2.7.3.6(1)	Actions outside the scope of Eurocodes	See NA for bridges	
A.2.7.4.1(1)	Combination rules for road bridges – General	See NA for bridges	
A.2.7.4.3(1)	Combination of traffic load on bridge deck and behind abutment	See NA for bridges	
A.2.7.4.5(1)	Combination of wind and thermal actions	See NA for bridges	
A.2.7.4.6(1)	Combinations of snow and traffic	See NA for bridges	
A.2.7.5.1(1)	Combination rules for footbridges - General	See NA for bridges	
A.2.7.5.3(1)	Combinations of wind and thermal actions	See NA for bridges	
A.2.7.5.4(1)	Combinations of snow and traffic	See NA for bridges	
A.2.7.6.1(1)	Combination rules for railway bridges – General	See NA for bridges	
A.2.7.6.4(1)	Combinations involving snow	See NA for bridges	
A.2.7.10(5)	Combination rules for integral abutment bridges	See NA for bridges	
A.2.7.10(9)	Combination rules for integral abutment bridges	See NA for bridges	
A.2.8(1)	Partial factors for ultimate limit states (ULS)	See NA for bridges	
A.2.9.1(1)	Serviceability criteria – General	See NA for bridges	
A.2.9.3.1(5)	Approach for assessment of vibrations due to pedestrian traffic	See NA for bridges	
A.2.9.3.3(1)	Critical range of natural frequency	See NA for bridges	
A.2.9.3.3(3)	Critical range of natural frequency	See NA for bridges	

Clause	Subject	National Determined Parameter	Non-Contradictory Complementary Information
A.2.9.3.3(4)	Critical range of natural frequency	See NA for bridges	
A.2.9.4.1(1)	Serviceability criteria for railway bridges – General	See NA for bridges	
A.2.9.4.2.1(3)	Vertical acceleration of the deck	See NA for bridges	
A.2.9.4.2.2(4)	Deck twist	See NA for bridges	
A.2.9.4.2.2(5)	Deck twist	See NA for bridges	
A.2.9.4.2.3(1)	Vertical deformation of deck	See NA for bridges	
A.2.9.4.2.3(2)	Vertical deformation of deck	See NA for bridges	
A.2.9.4.2.4(2)	Transverse deformation and vibration of the deck	See NA for bridges	
A.2.9.4.2.4(4)	Transverse deformation and vibration of the deck	See NA for bridges	
A.2.9.5(1)	Foundation movements	See NA for bridges	
A.2.10(1)	Fatigue	See NA for bridges	
A.2.11.1(9)	Tension components for cable bridges	See NA for bridges	
A.2.11.4.5(3)	Effect of the expansion joint on the structure	See NA for bridges	
A.2.11.4.7(1)	Expansion joint schedule	See NA for bridges	
A.3-A.6	Application for - A.3 Towers, masts and chimneys - A.4 Silos and tanks - A.5 structures supporting cranes or machines - A.6 Coastal structures	Awaiting EN 1990:2023 +A1:2026	
Annex B	Technical management measures for design and execution	Not applicable	
Annex B NA	Technical management measures for design and execution	Informative awaiting	
B.2(1)	Scope and field of application	Awaiting	
B.4(2)	Design quality	Awaiting	

Clause	**Subject**	**National Determined Parameter**	**Non-Contradictory Complementary Information**
B.5(1)	Design checking	Awaiting	
B.6(1)	Execution quality	Awaiting	
B.6(2)	Execution quality	Awaiting	
B.7(1)	Inspection during execution	Awaiting	
B.8(1)	Technical management measures	Awaiting	
Annex C	Reliability analysis and code calibration	Informative with changes in annex C NA	
Annex C NA	Reliability analysis and code calibration	Informative	
C.1(1)	Use of this annex	National choice	
C.3.1(5)	Overview of reliability verification approaches - Conditions for the use of reliability-based methods	Awaiting	
C.3.4.2(3)	Criterion for reliability-based design and assessment	National choice	
C.3.4.2(4)	Criterion for reliability-based design and assessment		Complementary information
Annex D	Design assisted by testing	Informative	
D.1(1)	Use of this annex	National choice	Complementary information
D.4.1(1)	Planning of tests	Unchanged	
Annex E	Additional guidance for enhancing the robustness of buildings and bridges	Not applicable	
Annex E1 NA	Minimum design measures to enhance structural robustness of buildings	Normative	
Annex E2 NA	Minimum design measures to enhance structural robustness of bridges	Normative	
Annex F	Rain-flow and reservoir counting methods for the determination of stress ranges due to fatigue	Informative	
F.4	Reservoir counting method	Not applicable	

Clause	**Subject**	**National Determined Parameter**	**Non-Contradictory Complementary Information**
G.2(1)	Scope and field of application	See NA for bridges	
G.3.1(6)	Basic requirements	See NA for bridges	
G.3.3.2(1)	Bearing schedule	See NA for bridges	
G.3.3.2(2)	Bearing schedule	See NA for bridges	
G.3.4(2)	Replacement of bearings	See NA for bridges	
G.3.4(3)	Replacement of bearings	See NA for bridges	
G.6(2)	Structural analysis – Effects of deformation of piers and abutments	See NA for bridges	
G.7.1.2(2)	Forces arising from the resistance of sliding elements	See NA for bridges	
G.7.1.3(2)	Forces arising from the shear resistance of elastomeric elements	See NA for bridges	
G.7.3.2(2)	Shear modulus at elastomeric bearings	See NA for bridges	
G.7.4.2(1)	Geometric uncertainty	See NA for bridges	
G.7.5.1(1)	Rigid fixed point in one abutment, other piers free-sliding bearings	See NA for bridges	
G.7.5.2(1)	Intermediate rigid fixed point, other piers free sliding bearings	See NA for bridges	
Annex H	Verifications concerning vibration of footbridges due to pedestrian traffic	See NA for bridges	
Annex FF NA	Partial factors for of resistance	Informative	Complementary information

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 (NDPs) and Non-Contradictory Complementary Information (NNCI)

3.1.2 Terms relating to design

(NCCI)

3.1.2.35 NA

Loss of static equilibrium

Loss of static equilibrium of the structure or any part of it as considered as a rigid body, where:

- minor variations of the value or the spatial distribution of actions from a single source are significant, and
- the strength of the construction materials or ground are generally not governing

3.1.3 Terms relating to actions

3.1.3.31 NA

Geotechnical action

Action transmitted to the structure by the ground, fill, surface water or groundwater.

NOTE 1 To entry: A geotechnical action is determined by the strength and deformation parameters of the ground. Examples of geotechnical actions are earth and water pressure on a retaining wall.

NOTE 2 To entry: The self-weight of ground or groundwater acting as plain self-weight ('ballast') is not considered as a geotechnical action, for instance self-weight of ground (earth), fill or water acting on a tunnel roof.

4.2(3) Structural reliability

(NDP)

The target annual reliability index is specified in C.3.4.2 NA for buildings and bridges respectively.

4.4(1) Robustness

(NCCI)

NOTE 2 is substituted by following note:

NOTE 2 For structures in consequence class CC1, design in accordance with the Eurocodes is assumed to provide an adequate level of robustness without the need for any additional design measures to enhance structural robustness.

4.4(2) Robustness

(NDP)

Design measures to enhance structural robustness for structures in consequence class CC2 and CC3 given in annex E.1 NA for buildings and annex E.2 NA for bridges shall be fulfilled.

4.7(1) Sustainability

(NDP)

Reference is made to the "building regulations" regarding buildings and "Projekteringsgrundlag for broer" regarding bridges.

7.1.5(7) Fire design

(NDP)

Reference is made to the building regulations chapter 5 regarding buildings and "Vejledning til belastnings- og beregningsgrundlag for broer" (VBB) respectively BN1-59 regarding bridges.

8.3.1(2) Verification of ultimate limit state (ULS) – General

(NCCI)

Additional text will be provided that describes possible ultimate limit states caused by excessive deformations and describes how to determine the design value of the excessive deformation in formula 8.2.

Awaiting information after the technical comment.

8.3.1(4) Verification of ultimate limit state (ULS) – General

(NCCI)

A new paragraph (4) is added.

(4) The partial factors in the national annexes should only be used together with the resistances and action effects obtained by the relevant load and resistance models specified in the appropriate Eurocode EN1991, EN1992, … EN1999 incl. Danish National Annex.

NOTE Partial factors for e.g. application of advanced numerical methods can be determined using Annex D and Annex FF NA.

8.3.3.1(5) Permanent actions

(NDP)

$$\rho = \frac{0{,}9}{1{,}2} = 0{,}75 \qquad \text{(8.a NA)}$$

8.3.3.6(1) Fatigue actions

(NDP)

Design values for fatigue loads are determined by using a partial factor γ_{Ff} equal to 1.6 for stress ranges, where the uncertainty of the individual stress ranges is described by a coefficient of variation of approximately 30 %.

For loads where the coefficient of variation is less than 10 %, a partial factor equal to 1.1 is used.

For other values of the coefficient of variation, the partial factor is determined by linear interpolation. The coefficient of variation may be stated in connection with the load models.

For bridges, reference is made to A.2.10(1).

8.3.4.2(2) Combination of actions for persistent and transient

(NDP)

Formula (8.14) is used.

(NDP)

$\xi = \frac{1,0}{1,2}$ for buildings (8.b NA)

$\xi = \frac{1,0}{1,25}$ for bridges (8.c NA)

Annex A: Application rules

(Normative)

A.1 General application and application for buildings

A.1.3(1) Consequence classes

(NDP)

(1) Buildings shall be classified into consequence classes according to the consequences of their failure with regards to expected number of lost lives.

NOTE Limits for expected number of lives lost are given in Table A.1.1 NA (left column). Examples of classified buildings in different consequence classes are given in Table A.1.1 NA (right column).

(2) Classification of a building into consequence classes should be carried out in two steps. In first step, the building as a whole should be classified in accordance with the limits for number of storeys and expected number of people given in the examples of classified buildings (Table A.1.1 NA (right column)). In second step it should be verified for CC2 and CC3 that the limits for expected number of lives lost (Table A.1.1 NA (left column)) are fulfilled for all parts / segments of the building using the design failure scenarios in E.1.5 and formula (E.1a NA) in E.1.6 for calculation of expected number of lives lost.

NOTE 1 Classification of a building into consequence classes is determining the robustness requirements. The final classification of the building as a whole and parts / segments of the building into consequence classes is directly linked to the verification of the robustness requirements, see Annex E.1. Segments and parts are explained in (6) and (11). See also table E.1.a NA and E.1.4.4 regarding segmentation.

NOTE 2 Classification of a building into consequence classes are determining the minimum level of design quality, design check and inspection during execution as well the choice of execution class, see Annex B. One minimum level can be connected to the consequence class of the building as a whole and different minimum levels can be connected to parts / segments of the building.

NOTE 3 Parts of the building structure can be classified to a different consequence class than the building if the consequences of their failure are different, see (11). For such parts a different factor k_F can be used, see Table A.1.9. See also NOTE 2 regarding different minimum levels of design quality, design checking and inspection during execution depending on the consequence class.

(3) Economic, social or environmental consequences should be included in the classification as specified by relevant authorities or where not specified as agreed for the specific project by relevant parties.

NOTE See also (7) and (8).

(4) Buildings may be classified into consequence classes directly based on the limits for expected number of lives lost provided that all relevant failure scenarios are analysed using the expected occupancy intensities given in Table A.1.a NA and fatality factor k given in E.1.6.

NOTE 1 The relevant failure scenarios cannot be limited to notional removal of elements described in E.1.5(2).

NOTE 2 Risk analysis can be used, see EN 1991-1-7 Annex B.

(5) Foundations and geotechnical structures that support a building should at least be classified with the same consequence class as the building.

NOTE Examples of geotechnical structures in different consequence classes are given in EN 1997-1.

Table A.1.1 NA - Classification of buildings in different consequence classes

Consequence class [c]	Description of consequence with associated limits for expected number of lives lost [d]		Examples of classified buildings with associated limits for number of storeys and people in the building [c]	
		Limits for expected number of lives lost	Limits for number of storeys in building	Limits for expected number of people in the building [a,b]
CC4	**Highest** Extreme risk of loss of human life – *or* economic, social, *or* environmental consequences are huge		Not intended for buildings but for special structures such as nuclear and other power plants and large-scale civil engineering structures where Eurocodes normally are only partly covering and special technical design bases and project-specific annexes need to be developed.	Not included in the classification system
CC3	**High** High risk of loss of human life - *or* economic, social *or* environmental consequences are very great	*≤ 300* *> 50*	Multi-storey buildings	7.500
			Single storey building	15.000
CC2b	**Medium high** Medium risk of loss of human life - *or* economic, social *or* environmental consequences are considerable	*≤ 50* *> 10*	Buildings up to 15 storeys above ground and a maximum 2 storeys below ground	1.500
			Single storey building	3.000
CC2a	**Medium low** Medium risk of loss of human life - *and* economic, social *or* environmental consequences are small	*≤ 10* *> 1*	Buildings up to 6 storeys above ground and a maximum of 1 storey below ground,	300
			Single storey building	600

Consequence class [c]	Description of consequence with associated limits for expected number of lives lost [d]		Examples of classified buildings with associated limits for number of storeys and people in the building [c]	
		Limits for expected number of lives lost	Limits for number of storeys in building	Limits for expected number of people in the building [a,b]
CC1	**Low** Low risk of loss of human life - *and* economic, social *or* environmental consequences are small	*≤ 1*	Single occupancy houses or terraced house up to 3 storeys above ground and a maximum 1 storey below ground,	15
			Building up to 2 storeys above ground and a maximum 1 storey below ground,	15
			Single storey agriculture or storage building	30
CC0	**Lowest** Very low risk of loss of human life - *and* economic, social *or* environmental consequences are insignificant	*0*	Non-load-bearing structures, small sheds, etc. - *and* secondary buildings where only short-term stays occur	

a The expected number of people in the building is calculated as:
∑ (utilized area) · (occupancy intensity in accordance with table A.1.a NA).
The occupancy intensity is increased by a factor of 1,5 in buildings where people are not able to escape on their own. This applies, for example, for hospitals, daycare centres, prisons and some types of nursing homes.
Rooftop terraces and unused attic floors are assumed having no occupancy.

b If the foundation of the building is significantly lower than the foundation of the neighbouring building, the people in the neighbouring building affected by a possible collapse of the foundation is included in the calculation.

c The examples of classified buildings given in right column with the associated limits for number of storeys and people in the building are assumed to fulfil the corresponding limits for the specific consequence classes with regards to expected number of human lives lost (Table A.1.1 NA (left column)) and can be used to determine the consequence class for other buildings. See also (2) including NOTE 1.

d The number of expected lives lost is connected to arbitrary failure scenarios that can be different from failure scenarios connected to notational removal of elements.

(NCCI)

(6) A structure may be considered as statically independent if forces cannot be transferred between the adjacent independent segments, neither vertically nor horizontally. The calculation of the expected number of people in the building and the expected number of lives lost is done separately for each independent segment.

(7) Buildings for or located close to critical infrastructure may be classified into a higher consequence class as specified by relevant authorities.

NOTE Examples of buildings and structures that are often considered critical infrastructure are: Hospitals, power plants and waterworks.

(8) Buildings for activities related to potential environmental hazards (activities covered by regulation for the Control of Major-Accident Hazards involving Dangerous Substances the Executive Order on Risks – Directive Seveso III) may be classified into a higher consequence as agreed for a specific project by relevant parties.

NOTE For further information see Risikobekendtgørelsen (Bekendtgørelse No. 372) and Risikohåndbogen.

Table A.1.a NA — Expected occupancy intensities for different building applications

No.	Building application	Occupancy intensity N_{PE} [a] [number of people/m²]
1	Residential building	0,05
2	Office	0,1
3	Hospital	0,1
4	Education – Classrooms (incl. hallways) and daycare centers (excl. larger gathering areas)	0,25
5	Commercial buildings (shopping malls, libraries, museums, restaurants)	0,25
6	Assembly areas, other (auditoriums, churches, assembly halls in schools, etc.) [b]	0,8
7	Assembly areas, dense (grandstands and standing concert) [b]	2,5
8	Industry and garages [b]	0,05
9	One family residential building	0,02
10	Agriculture and storage [b, c]	0,002

a Including access routes such as hallways, stairs, etc. Stairs and access routes can be included in the calculation with the same occupancy intensity as the adjacent areas.

b Alternatively, project specific values corresponding to the maximum number of people allowed according to the fire protection strategy can be used.

c Alternatively, project specific values can be used that also accounts for the expected number of animals lost (agriculture) and economic consequences (storage).

(9) A building which occasionally is used for purposes other than its main purpose shall be classified into the highest consequence class applicable for the anticipated purposes.

NOTE Occasional uses corresponding to a one level higher consequence class can be disregarded if they occur less than 6 times a year and the number of people does not exceed 1000.

(10) The same occupancy intensity may be applied to an entire storey or building if gathering spaces such as meeting rooms, canteens, auditoriums, or similar areas are mainly used by the regular occupants of the storey or building.

(11) For structures classified into consequence class CC2b or CC3 structural members may be classified into to a consequence class one level lower provided that a comprehensive assessment of all relevant design failure scenarios for each structural member in accordance with E.1.5 NA and E.1.6 NA demonstrates that the number of lost lives resulting from structural failure does not exceed the expected number of lives lost for the lower consequence class given in Table A.1.1 NA.

NOTE See E.1.5 (6) in Annex E.1 NA.

A.1.6.1(1) Ultimate limit states (ULS)

(NDP NOTE 1)

Formula (8.14) is used.

(NDP NOTE 3)

$\xi = \frac{1{,}0}{1{,}2}$ for buildings.

(NDP NOTE 5)

For the accidental design situations ψ_2 is used

A.1.6.3(1) Combination factors

(NDP)

Value of the combination factors are given in Table A1.1.7 NA

Table A.1.7 NA — Combination factors for buildings

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings (see EN 1991-1-1):			
Category A: domestic, residential areas and similar	0,5	0,3	0,2
Category B: office areas	0,6	0,4	0,2
Category C: congregation areas	0,6	0,6	0,5
Category D: shopping areas	0,6	0,6	0,5
Category E: storage areas and industrial areas	0,8	0,8	0,7
Category F: traffic area, vehicle weight ≤ 30 kN	0,6	0,6	0,5
Category G: traffic area, 30 kN < vehicle weight ≤ 160 kN	0,6	0,4	0,2

Action	ψ_0	ψ_1	ψ_2
Category H: roofs accessible for normal maintenance and repair only	0	0	0
Construction loads (see EN 1991-1-6)	1,0	-	-
Snow loads on buildings (see EN 1991-1-3):			
Combined with imposed loads category E	0,6	0,2	0
Combined with wind loads	0	0	0
else	0,3	0,2	0
Wind actions on buildings (see EN 1991-1-4)			
Combined with imposed loads category E	0,6	0,2	0
else	0,3	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0
Water actions [a] (see 6.1.3.2)	-	-	-
a The combination value for water actions can be based on a 10 % probability that it is exceeded during a one-year reference period.			

A.1.7(1) Partial factors for ultimate limit state (ULS)

(NDP)

The values for γ_F and γ_E are given in Table A.1.8 NA.

The consequence factor k_F in Table A.1.9 NA.

Table A.1.8 NA — Partial factors on actions and effects for verification cases VC1 to VC4 for persistent and transient (fundamental) design situations

Action or effect				Partial factor γ_F and γ_E for design cases VC1 to VC4 [a, h, i, j, k, l]						
Type	Group	Symbol	Resulting effect	General		Static equilibrium	Uplift	Geotechnical structures		
Verification Case				VC1		VC2[b]		VC3		VC4
Formula				(8.14)		(8.12)		(8.14)		(8.12)
Subcase				(a)	(b)	(a)[c]	(b)[d]	(a)	(b)	
Permanent action (G_k)	Self-weight, 'Non-variable' part ($G_{k,inf}$)	γ_G	unfavourable [c, e]	1,2 k_F	1,0 k_F	1,1 k_F	1,1 k_F	1,2	1,0	1,0
		$\gamma_{G,fav}$	favourable [c]	1,0	0,9	0,9	0,9	1,0	0,9	1,0
	Self-weight, 'Variable' part ($G_{k,sup} - G_{k,inf}$)	γ_G	unfavourable [c, e]	1,3 k_F	1,0 k_F	1,1 k_F	1,1 k_F	1,3	1,0	1,0
		$\gamma_{G,fav}$	favourable [c]	0	0	0	0	0	0	0
	Self-weight, Ground and water, Geotechnical structures [f]	γ_G	unfavourable [c]	1,0	1,0	1,1 k_F	1,1 k_F	1,0	1,0	1,0
		$\gamma_{G,fav}$	favourable [c]	1,0	1,0	0,9	0,9	1,0	1,0	1,0
Prestressing action (P_k)		γ_P [g]								
Variable action (Q_k)	Leading	$\gamma_{Q,1}$	unfavourable	0	1,5 k_F	1,5 k_F	1,5 k_F	0	1,5	$\gamma_{Q,red}$ = 1,25
	Accompanying	$\gamma_{Q,i}$		0	1,5 $\psi_0 k_F$	1,5 $\psi_0 k_F$	1,5 $\psi_0 k_F$	0	1,5 ψ_0	
	All	$\gamma_{Q,fav}$	favourable	0	0	0	0	0	0	0
Effects of actions (E)		γ_E	unfavourable	γ_E not applied						1,2 k_F
		$\gamma_{E,fav}$	favourable							1,0

a VC1 applies in general for all types of structures including geotechnical structures. For some geotechnical structures VC3 and VC4 apply as well, for slopes and embankments only VC3. Geotechnical structures covered by VC3 and VC4 are structures mainly subjected to geotechnical actions. DS/EN 1997-1 DK NA, Table 4.0 NA comprises a list of Verification Cases for all geotechnical structures covered by EN 1997-3.

b VC2 applies for structures where there is a risk of loss of static equilibrium or uplift due to buoyancy or vertical tension forces.

c VC2(a) is only used for structures where the stability is sensitive to significant variations of permanent actions from a single source according to A.1.7(5) NCCI DK NA. VC2(a) can be relevant for cantilevered structures where the permanent loads are dominant especially in transient design situations. The terms 'unfavourable' and 'favourable' are substituted by 'destabilizing' and 'stabilizing' for permanent actions for this very verification, see EN 1990:2023, 8.3.3.1(5). If a tension or compression member is required, the design resistance of the member shall be greater than the force needed to obtain stability.

d VC2(b) is used to verify uplift problems. Where tension members are needed to provide stability by mobilising the ground below the structure, the loads carried by the tension members are determined assuming that the structure and ground is acting as a rigid body. The mobilised volume (and weight) of soil depends on the position of tension members, the failure type (block, pull out or a combination thereof) and the type of tension members (tension pile, bond- or compression type anchor). The tension members are also verified in VC1, VC3 and VC4. The structure is only be verified in VC1, VC3 and VC4.

e Single-source principle applies for 'non-variable' part ($G_{k,inf}$) of self-weight. Provided that the structure is not sensitive to spatial variations of self-weight, single-source principle may also apply for 'variable' part ($G_{k,sup}$ – $G_{k,inf}$) of self-weight (EN 1990:2024, 8.3.3.1(3)). Load arrangements similar to those for imposed loads according to EN 1991-1-1:2025 6.4.1(1) and 6.4.2(1) and (2) may be used for the 'variable' part of self-weight.

f Comprises the self-weight of ground and (ground) water acting on the geotechnical structure as a geotechnical action.

g γ_P depends on the type of prestress, type of structure and on the structural materials involved. See other relevant Eurocodes for the application of γ_P.

h Where geotechnical structures are included only one of consequence factors k_F, k_M and k_R is used in each verification. If the factor is applied on the action side (k_F), it shall not be applied neither on the strength parameters of the soil (i.e. k_M = 1,0) nor the resistance of the geotechnical structural members (i.e. k_R = 1,0). If the factor is applied on the soil strength parameters (k_M), then it is neither applied on the action side (i.e. k_F = 1,0) nor the resistance of the geotechnical structural members (i.e. k_R = 1,0)

i For geotechnical structures the following relation applies between the verification case, VC, and the set of partial factors, M, for the strength parameters of the soil for MFA (Material Factor Approach), refer EN 1997-1 incl. DK NA and note a:

Verification case, VC	VC1	VC3	VC4
Set of partial factors, M	M2 (k_M = 1,0)	M2	M1

j In the verification of geotechnical structures by means of numerical models according to 8.2 in EN 1997-1 incl. DK NA both VC4 + M1 (Output Factoring Approach), VC3 + M2 (Input Factoring Approach) and VC1 + M2 (Input Factoring Approach) apply. This approach is particularly suitable for retaining structures supported by ground anchors and props at more levels, where the stiffness of the soil and the structural members affect the distribution of the reaction and sectional forces.

k For the application of a separate partial factor (model factor), γ_{Rd}, related to the uncertainty of the modelling of the resistance for a structural component or a structure reference is made to Annex F NA and EN 1992 – EN 1997, EN 1999 – EN 19100 incl. DK NA.

l For the application of a separate partial factor (model factor), γ_{Sd}, related to the uncertainty of the modelling of load effects reference is made to EN 1990, EN 1991, EN 1992 – EN 1997, EN 1999 – EN 19100 incl. DK NA

Table A.1.9 NA — Consequence factors for building and geotechnical structures

Consequence class	Consequence factor k_F
CC3	1,1
CC2b	1,0
CC2a	1,0
CC1	0,9
NOTE Description of consequences see table A.1.1 NA.	

A.1.7(5) Partial factors for ultimate limit states (ULS)

(NCCI)

NOTE NCCI due to error in A.1.7(5) which is expected to be corrected in a later amendment.

A.1.7(5) is changed to:

(5) When variations in the magnitude or spatial variation of permanent actions from a single source (see 6.1.1(3) and 8.3.3.1(5)) are significant, ultimate limit states that involve loss of static equilibrium and/or strength of elements contributing to the equilibrium, should be verified using partial factors for verification cases VC2(a) and VC2(b), using whichever gives the less favourable design outcome.

NOTE 1 Verification case VC2(b) may be omitted when it is obvious that verification using verification case VC2(a) governs the design outcome.

NOTE 2 Verification of loss of static equilibrium may also include cases, where the unfavourable actions are independent from the favourable actions (no correlation) and the partial factors are chosen correspondingly.

NOTE 3 Change due to error in A.1.7(5) which is expected to be corrected in a later amendment.

A.1.7(6) Partial factors for ultimate limit states (ULS)

(NCCI)

NOTE NCCI due to error in A.1.7(6) which is expected to be corrected in a later amendment.

A.1.7(6) is changed to:

(6) Verification of ultimate limit states that involve loss of static equilibrium due to uplift by water pressure or other vertical actions and/or strength of elements contributing to the equilibrium should be verified using partial factors for verification case VC2(a).

NOTE Change due to error in A.1.7(6) which is expected to be corrected in a later amendment.

A1.8.1(1) Serviceability for buildings

(NDP)

Limiting values shall be determined for the specific project taking into account the function of the adjacent elements (load-bearing and non-load-bearing) and shall be in accordance with the requirements of EN 1992 – EN 1999 incl. DK NA.

A.1.8.2.2(2) Vertical deflections

(NDP)

Limiting values should be determined for the specific project. Values in Table A.1.10 are not used.

A.1.8.3(3) Vibrations

(NDP)

Vibration behaviour of structures under normal use may be assessed using the limiting values given in Table A.1.8.3 NA as a guideline. The limiting values may be considered as requirements for user comfort and functionality in the serviceability limit state.

NOTE 1 The acceptable level of behaviour for resonant vibrations is expressed as a percentage of the gravitational acceleration (%g). The acceptable level of behaviour for transient vibrations is expressed as a velocity in millimetres per second (mm/s). The limiting values are standard deviations for accelerations and velocities, respectively.

NOTE 2 Resonant vibrations are induced by harmonic loads that can be in resonance with the structure. Transient vibrations are induced by impulsive loads. Load models are given in DS/EN 1991-1-1 DK NA.

NOTE 3 DS/EN 1992 - DS/EN 1999 incl. DK NA can provide alternative guidelines.

Table A.1.8.3 NA — Limiting values (recommended) for acceptable accelerations and velocities for given natural frequencies

Category	Load type	Resonant vibration		Transient vibration		Acceptable vibrations
		Natural frequencies	Acceptable accelerations	Natural frequencies	Acceptable velocities	Natural frequencies
		[Hz]	[%g]	[Hz]	[mm/s]	[Hz]
Floor deck for residences	Pedestrian load	≤ 8	≤ 0,1	8-15	≤ 1,6	> 15
Floor deck for offices, malls, retail stores, schools and similar	Pedestrian load	≤ 8	≤ 0,2	8-15	≤ 1,6	> 15
Balconies, roof terraces and access galleries	Pedestrian load	≤ 8	≤ 0,4	8-15	≤ 6	> 15
Monumental stairs [a]	Pedestrian load	≤ 10	≤ 2	Not relevant	Not relevant	> 10
Grandstands, fitness centres, sports halls and similar facilities	Rhythmic crowd load	≤ 10	≤ 10	Not relevant	Not relevant	> 10

NOTE 1 When calculating natural frequencies, accelerations, and velocities, the most unfavourable value between 0 % and 10 % of the static imposed load given in Table 6.1 in DS/EN 1991-1-1 is normally used for pedestrian loads.

NOTE 2 When calculating natural frequencies of structures used for rhythmic crowd loads, the natural frequency is not influenced by people.

NOTE 3 For pedestrian loads, accelerations and velocities are calculated from a single person. The vibrations for each mode shape are compared with the limit value.

NOTE 4 A structure's natural frequencies can be influenced by flexible supports. This applies, for example, to certain floor structures and stairs. In the calculation of the natural frequencies of a floor structure, orthotropic stiffness properties can influence the vibration behaviour of the structure. Orthotropic stiffness properties are, for example, relevant for ribbed slabs, hollow-core slabs, and joist systems of steel or timber.

NOTE 5 Supplementary guidance can be found in ISO 10137.

a Monumental stairs are larger staircases designed to be used by many people throughout the day.

A.1.8.3(4) Vibrations

(NDP)

See Table A.1.8.3 NA.

A.1.8.4(4) Limiting foundation movements

(NCCI)

Values of $C_{d,SLS}$ should always be assessed for the specific project with regard to the specific building's sensitivity to foundation movements.

A.1.8.4(5) Limiting foundation movements

(NCCI)

Values of $C_{d,SLS}$ should always be assessed for the specific project with regard to the specific building's sensitivity to foundation movements.

Annex B NA
Technical management measures for design and execution
(informative)

Awaiting

Annex C NA: Reliability analysis and code calibration (Informative)

C.1(1) Use of this annex

(NDP)

Annex C applies with Clause C.3.4.2 changed to C.3.4.2 NA.

C.3.1(5) Overview of reliability verification approaches

(NDP)

Awaiting

C.3.4.2 NA Criterion for reliability-based design and assessment

(NDP)

(1) The target annual reliability level is given in Table C.3 NA.

(2) When carrying out calibration of partial factors the deviation from the target annual reliability index should not be larger than 0,4.

(3) If in reliability-based design the design situation is covered by the partial safety factor design format, it should be demonstrated that the same reliability level is obtained as for either (i) a reference design or (ii) the target reliability level. This relative comparison should be made based on similar probabilistic models. The reliability level for reliability-based design should not have a deviation larger than 0,4 from the target annual reliability index and may be chosen equal to the target annual reliability index.

Table C.3 NA — Target values for reliability index β for different consequence classes (for persistent and transient (fundamental) and fatigue design situations in ULS) relevant to structures in the scope of Annex A.1

Consequence class	1-year reference period β
CC3	4,6
CC2b	4,1
CC2a	4,1
CC1	3,5

(NCCI)

(4) When referring to the 1-year reliability index β, the target should be met for every year of the required (or chosen) design service life (or remaining service life) of the structure.

Annex D: Design assisted by testing (Informative)

D.1(1) Use of this annex

(NDP)

The Annex D can be used except EN 1990 D.7.3 and EN 1990 D.8.3.

(NCCI)

(2) Annex D may be used to check characteristic values and to determine characteristic values and design values. Sections EN 1990 D.7.3 and EN 1990, D.8.3 cannot be used, as these require a reliability level corresponding to β = 3,8 and the use of the design value method in Annex C. Instead, reference is made to Annex FF NA, where the determination of material partial factors and design values is described.

Annex E.1 NA, Minimum design measures to enhancing structural robustness of buildings

(Normative)

E.1.1 Use of this annex

This normative Annex E.1 NA substitutes Annex E in EN 1990. Annex E.1 NA provides rules for enhancing the robustness of buildings.

NOTE Rules for enhancing robustness provides minimum requirements.

E.1.2 Scope

(1) Design according to this annex shall ensure that enhanced robustness is achieved by:

- preventing disproportionate consequences as a result of unforeseen hazardous events such as the failure of a structural member or part of a structure; or
- adding additional structural resistance to reduce the likelihood and extent of such an event.

NOTE 1 A robust structure is achieved by appropriate choice of overall static principles and structural systems, as well as by appropriate design of key members and choice of construction components and materials.

NOTE 2 Design for robustness cannot be considered as a design for gross human errors.

(2) Design for identified accidental actions shall be undertaken in accordance with EN 1991 (all parts) and other Eurocodes and not in accordance with Clause 4.4 and this annex.

NOTE 1 An identified accident action is one that can occur during the life of the structure and for which a structure is designed, either by designing it to resist the action or by making mitigation measures.

NOTE 2 The distinction between design for robustness in accordance with this annex and design for identified accidental actions in accordance with EN 1991 (all parts) is shown in Table E.1.a NA.

NOTE 3 Methods for the design for enhanced robustness are given in E.1.3 and E.1.4.

Table E.1.a NA — Design for identified accidental actions and design for enhanced robustness

Design for accidental actions (EN 1991 all parts) Explicit design of the structure (e.g. against fire, explosions or impact)	**(a) Design the structure to resist the action**
	(b) Prevent or reduce the impact e.g. protective measures, control of impacts
Design for enhanced robustness (EN 1990) Strategies based on limiting the extent of a failure	**(a) Alternative load path** either providing sufficient ductility, resistance and deformation capacity and redundancy, or applying prescriptive design rules
	(b) Key members i.e. designing selected members to resist notional action(s)
	(c) Segmentation separation of the structure into statically independent parts

E.1.3 Design strategies

(1) Design strategies for designing structures to achieve enhanced robustness may be chosen from the following:

a) Creation of alternative load paths:

a1) by providing sufficient ductility, resistance and deformation capacity for local failure scenarios and redundancy in the structure, so that the resistance may be demonstrated in an accidental design situation.

a2) in some cases by using prescriptive design rules, for example when designing tensile connections (ties) and thereby creating a coherent structure.

b) Key members:

Designing selected elements to resist notional action(s).

c) Segmentation either by:

c1) vertical segmentation of the structure into statically independent building parts, so that each part is able to collapse independently without affecting the safety in other parts.

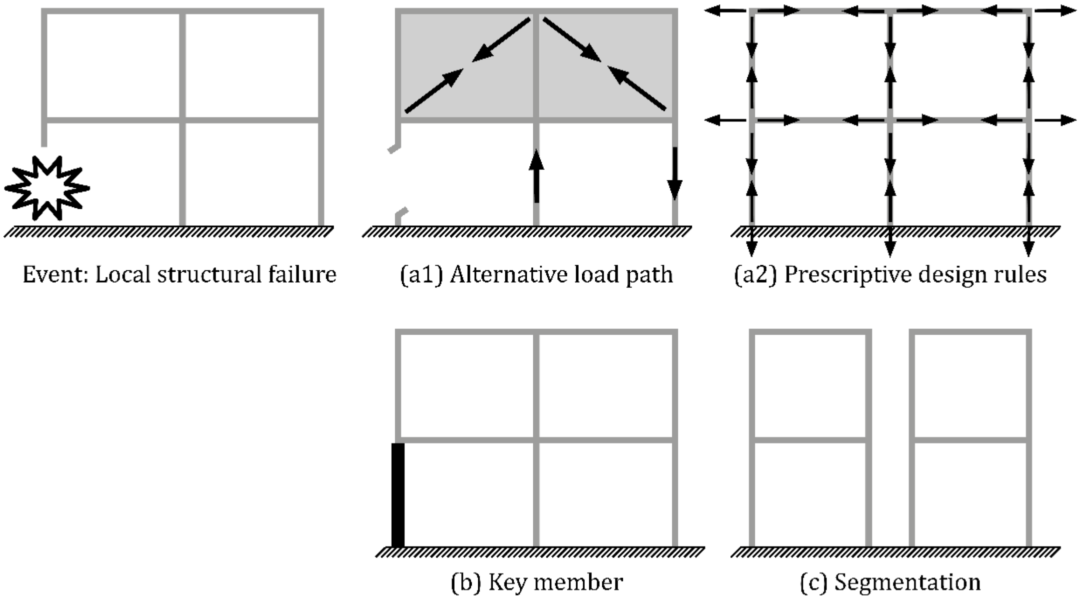
c2) horizontal segmentation in order to prevent a vertical progressive collapse.

NOTE 1 Appropriate redundancy (alternative load path) is suitable to prevent vertical progression of collapse, while vertical segmentation is suitable to prevent horizontally progressive collapse.

NOTE 2 DS/EN 1992 - DS/EN 1999 including associated national annexes can provide supplementary rules for structural systems that can be considered when documenting alternative load paths. EN 1991-1-7 provides additional rules concerning prescriptive design rules.

NOTE 3 Design strategies a)-c) for robustness are illustrated in Figure E.1.b NA.

Figure E.1.b NA — Illustration of design strategies a-c described in E.1.2(1)



E.1.4 Design methods

E.1.4.1 General

(1) Design for enhanced robustness according to the methods in this annex should be considered as accidental design situations, unless otherwise stated in the following sections regarding key members and DS/EN 1992 – DS/EN 1999 and DS/EN 19100 including associated national annexes.

(2) Methods for providing enhanced robustness shall be chosen on the basis of the building's consequence class (CC), see Table E.1.c NA.

Table E.1.c NA — Design methods for enhancing robustness in relation to consequence class

The building's consequence class	Design methods
CC3	Where the expected extent of collapse exceeds the acceptable level, see E.1.6, robustness shall be demonstrated • As for CC2b, however, prescriptive design rules d) [a] shall be supplemented by at least one of the other design methods a)-c). • Where the expected number of human lives lost exceeds 300, segmentation of the structures shall be carried out. For special complex building structures, adequate robustness should be verified by carrying out a systematic risk assessment of the structures according to (4). A systematic risk assessment of the building can be undertaken taking into account alle the normal hazards thar can reasonably be foreseen, together with any abnormal hazards.

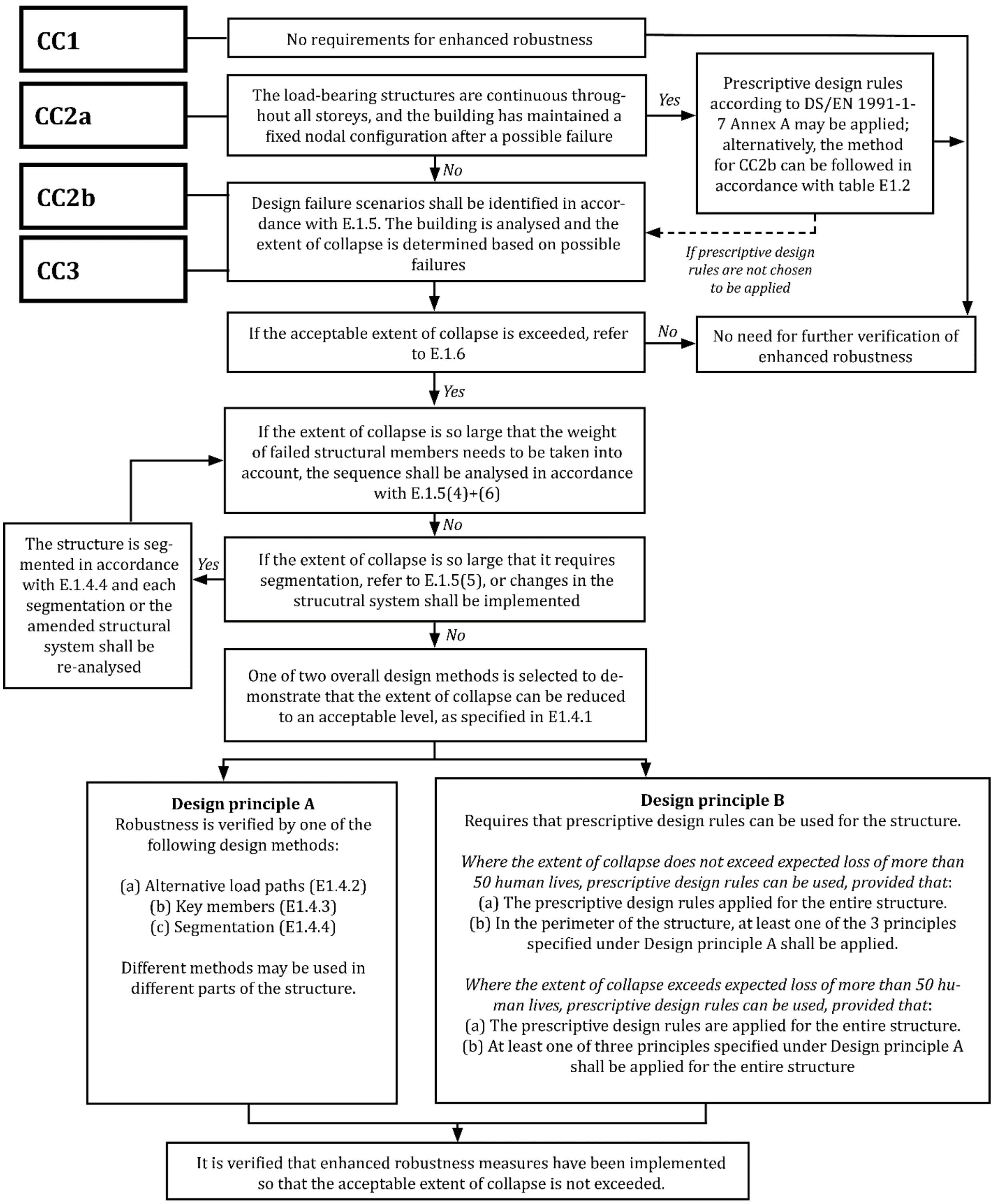
The building's consequence class	Design methods
	Critical situations for design should be selected that reflect the conditions that can reasonably be foreseen as possible during the life of the building. The structural form and concept and any protective measures should then be chosen and the detailed design of the structure and its elements undertaken in accordance with the eurocodes.
CC2b	Where the expected extent of collapse exceeds the acceptable level, see E.1.6, enhanced robustness shall be demonstrated by: • at least one of the following design methods: a) Alternative load paths a1) redundancy of the structure a2) or in some cases by use of prescriptive design rules b) Key members c) Segmentation • or by using: d) Prescriptive design rules [a] with the following additions: - The use of prescriptive design rules along the perimeter of the building [b] shall be supplemented by at least one of the other design methods mentioned above: a1), b) or c). - When the expected number of human lives lost ≥ 50 and if the prescriptive design rules do not provide a verifiable alternative load path, at least one of the design methods a1), b) or c) should supplement the design method.
CC2a	Where the expected extent of collapse exceeds the acceptable level, see E.1.6, robustness shall be demonstrated by: • at least one of the following design methods: a) Alternative load paths a1) redundancy of the structure a2) or in some cases by the use of prescriptive design rules b) Key members c) Segmentation • Or by using: d) Prescriptive design rules [a] (for the entire structure)
CC1	No requirements for enhanced robustness
a	Prescriptive design rules according to DS/EN 1991-1-7 Annex A can only be used as the sole design method if load-bearing structures are continuous through all storeys and if the main structural system of the building can remain as a fixed and stable nodal configuration after any potential failure.

The building's consequence class	Design methods
b	The building's perimeter is defined as the line/place where the outermost row of load-bearing structures is placed along facades, atriums and similar openings in story and roof decks and include load-bearing structures in the 3 outermost meters of the building.

(3) The procedure for verifying robustness of buildings other than those described in (4) is illustrated in Figure E.1.b NA.

(4) For special complex building structures in CC3 a systematic risk assessment should be carried out taking into account all normal hazards that can be foreseen as well as any abnormal hazards that can be identified. Critical design situations should be defined corresponding to the conditions that can be foreseen for the building structure during its lifetime. Based upon that the structural system and any protective measures should be chosen and the detailed design of the structure and its members carried out.

Figure E.1.b NA — Robustness verification procedure



E.1.4.2 Alternative load path

(1) It should be demonstrated that for the design failure scenarios in E.1.5 an alternative load path can be formed and verified in the accidental design situation.

NOTE DS/EN 1992 – DS/EN 1999 and DS/EN 19100 including DK NA can provide further guidance on which load-bearing capacities can be taken into account when verifying alternative load paths. DS/EN 1991-1-7 provides further instructions on prescriptive design rules.

E.1.4.3 Key members

(1) A Key member is a structural element whose absence will result in an extent of collapse greater than the acceptable extent of collapse ad. E.1.4.1, E.1.5 and E.1.6. Key members should be verified as described in (2) and (3).

NOTE A key member can be a structural element, e.g. a beam, a column, a deck, a connection or a bracing.

(2) Key members where the expected number of lives lost by failure is > 10 and ≤ 50 should be verified to have sufficient reliability for all following cases:

a) the design resistance R_d is reduced by 20 %
b) the load application point is moved, resulting in a 40 % reduction in the load-bearing depth in the most unfavourable direction.
c) the element is subjected to an additional load equal to 10 % of the element's normal force, with a maximum of 80 kN. The additional load is assumed to act in any direction perpendicular to the faces of the element. The additional load shall be carried by the stabilizing system.

Requirements a) and b) shall be demonstrated in cases VC1 – VC4 for persistent and transient design situations, and c) should be demonstrated for the accidental design situation.

When the load-bearing capacity of the soil determines the strength of a key member, requirements in DS/EN 1997-1 apply.

NOTE DS/EN1992-1999 + DS/EN19100 including associated national annexes can specify additional requirements on key members.

(3) Key members where:

- the expected number of human lives lost by failure is > 50 and ≤ 300 or
- the number of storeys that fail is greater than 8 should be designed as in (2) supplemented with the requirements in (4) and (5).

(4) Vertical or inclined key members including joints, the load capacity should be verified by applying the following notional actions:

- A point load of 160kN perpendicular to the element, attacking in the element's centroidal axis, at any point in any direction.
- A uniform load of 34kN/m² acting perpendicular to the surface of the key member in any direction.

Permanent load is applied when it is unfavourable.

(5) Horizontal key members including joints should be designed for for the following vertical notional actions:

- a point load of 160 kN attacking in the element's centroidal axis, any place or
- a uniform load of 15 kN/m² acting on the element.

The notional actions may have a downward or upward effect.

Permanent load is applied when it is favourable

(6) Additional safety of key members cannot be used as the only measure where the expected number of human lives lost in the event of failure is > 300. In such cases, the structural system should be changed and/or segmentation introduced, so that the expected number of human lives lost becomes ≤ 300.

E.1.4.4 Segmentation

(1) The segmentation shall be performed based on the limit of the expected number of human lives lost that can be accepted for the segment, see Table E.1.b NA. For buildings involving activities that may pose potential environmental hazards in the event of collapse or are located close to critical infrastructure, it may be relevant to apply parameters other than the expected number of human lives lost. This is decided on a project-specific basis.

NOTE Segmentation can be used to ensure enhanced robustness in all consequence classes by isolating the collapse and thereby preventing a progressive collapse and limiting the extent of collapse.

(2) Vertical segmentation should be carried out using either a total separation or by introducing one or more weak structural elements or connections (fuse-elements), such that each segment is able to collapse independently without leading to failure of other segments.

In a vertical segmentation, the load-bearing capacity of an optional fuse-element should be less than the expected load effect in the event of a failure, see formula (E.1.c NA).

$$R_{\mathrm{d}} \leq E_{\mathrm{fuse,d}} \tag{E.1.c NA}$$

where R_{d} is the design value of upper strength (load-bearing capacity) of the fuse-element constituting the segmentation,

It is determined as the 95 % percentile of the resistance multiplied by a partial factor or as a prescribed value, see DS/EN1992-1999 and DS/EN19100 incl. DK NA.

$E_{\mathrm{fuse,d}}$ is the design load effect from the forces that the fuse-element will be affected to in the event of a collapse

NOTE 1 Normally, $E_{\mathrm{fuse},d}$ depends on the self-weight ($G_{\mathrm{k,inf}}$) of the part of the structure that is assumed to collapse.

NOTE 2 Additional requirements for segmentation can be specified in DS/EN 1992-DS/EN 1999 including associated national annexes.

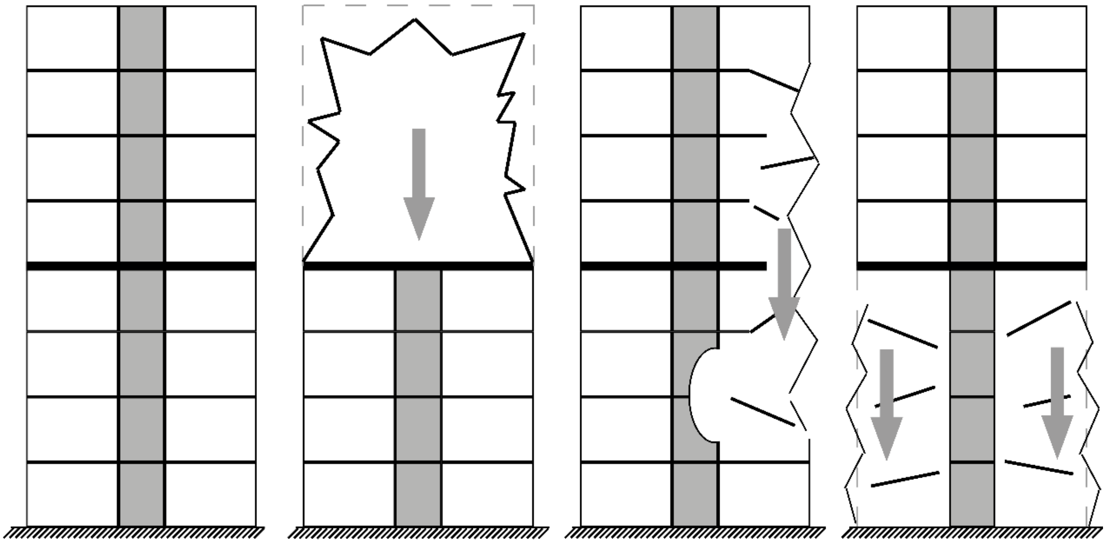
(3) Horizontal segmentation includes the horizontal separation structures that should stop the collapse, see Figure E.1.c NA.

When designing horizontal segmentation, a vertical notional action corresponding to the failure of storeys above may be based on:

- 28kN/m² for failure of 1-2 storeys,
- 34kN/m² for failure of 3-4 storeys and
- 41kN/m² for failure of more than 4 storeys.

(4) When verifying the ability of segmentation to limit the expected number of human lives lost, it may be accepted that some design failure scenarios result in failure both above and below a segmentation, if the resulting expected number of human lives lost in no scenario exceeds 300.

Figure E.1.c NA — Examples of horizontal segmentation



E.1.5 Design failure scenarios

(1) Design failure scenarios which should be investigated as basis for calculating the extent of collapse, are identified based on the building's consequence class, see E.1.4.1.

NOTE 1 Regarding the acceptable extent of collapse and calculation of collapsed area and expected number of human lives lost, see E.1.6.

NOTE 2 Design failure scenarios are used for analysing and assessing the consequences of a local failure due to an unforeseen adverse event. The event may cause failure in one or more structural elements or parts of the structure.

(2) Design failure scenarios for buildings in consequence class CC2a, CC2b and CC3 should be investigated by the notional removal of:

– one slab structure with a width of 3 m between two structural supports, or
– one beam, or
– one column on one storey, or
– one 3 meter long wall section on a storey, or
– one connection

NOTE For buildings that do not primarily consist of walls, columns, frames and decks, the local failure that triggers a failure can be associated with any structural part.

(3) It should be demonstrated that failure does not lead to a collapse greater than the acceptable collapse extent or robustness measures should be introduced as described in Table E.1.c NA.

NOTE Sequences of failed elements are identified. A sequence is considered to be fully developed when all remaining elements posses sufficient load-bearing capacity and stability.

(4) For buildings in CC3, the weight of the collapsed structures shall be taken into account when calculating the area of collapse. In lower consequence classes, the weight of the collapsed structures may be disregard.

NOTE The weight can be considered included if notional actions are used for the collapsed structures in accordance with E.1.4.4(3). Alternatively, more detailed analyses can be carried out.

(5) For any given design failure scenario, the expected number of human lives lost shall not exceed 300.

NOTE This limit corresponds to CC3.

(6) For a part of a building structure classified into a lower consequence class according to A.1.3(11), the weight of the collapsed structures shall be taken into account as stated in (4).

E.1.6 Acceptable extent of collapse

(1) The acceptable extent of a collapse following a design failure scenario should not exceed a total area of 750 m^2 and should not result in a higher expected number of human lives lost than 10.

NOTE This limit corresponds to CC2a.

(2) Collapse extents less than the acceptable extent of collapse defined in (1) does not require measures to enhance robustness.

NOTE 1 The acceptable extent of collapse depends on:

- the use of the areas, that are expected to be affected by a collapse, and
- the expected number of people expected to be present in the areas

NOTE 2 For buildings involving activities that can pose potential environmental hazards in the event of collapse or are located close to critical infrastructure, it can be relevant to apply parameters other than the expected number of human lives lost see also A.1.3 (7) and (8).

(3) The expected number of human lives lost following a local failure shall be calculated from the formula (E.1.a NA).

$$N_F = \begin{cases} k\ A_{COL}\ N_{PE,u} & \text{for buildings with one utilized storey} \\ k\ \left(A_{COL,u}\ N_{PE,u} + \sum_i A_{COL,i} N_{PE,0,i}\right) & \text{for buildings with more than one utilized storey} \end{cases} \quad \text{(E.1.a NA)}$$

where	N_F	is expected number of human lives lost
	A_{COL}	is collapse area, calculated as described in (4)-(6)
	$A_{COL,i}$	is collapse area storey *i*, calculated as described in (4)-(6)
	$A_{COL,u}$	is collapse area of the lowest deck that collapses
	$N_{PE,o}$	is number of people per m^2 that is located above the collapsed area of the story *i*. In the case of roof structures without occupancy, the value can be set to 0. Values of N_{PE} are provided in table A.1.a NA

$N_{\mathrm{PE,u}}$	number of people per m² that is located below the collapsed area. Values of N_{PE} are provided in table A.1.a NA
k	is the fatality factor that represents the proportion of the expected number of human lives lost in relation to the people present in the building. The fatality factor depends on number of storeys: for 1-3 storey 0,15 is used, for 4 storeys or more 0,3 is used on all storeys including storey 1-3

(4) For buildings the area A_{COL} is calculated on the basis of formula (E.1.b NA).

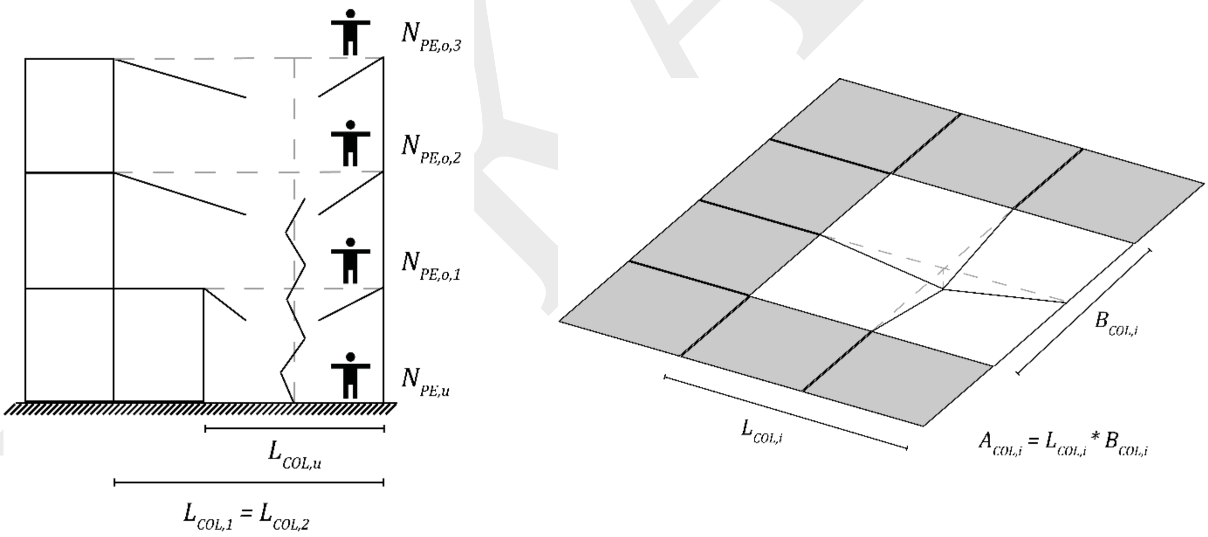
$$A_{COL} = \sum_i l_{COL,i}\, b_{COL,i} \qquad \text{(E.1.b NA)}$$

where $l_{\mathrm{COL,i}}$ is the extend of the collapse in one direction on storey i

$b_{\mathrm{COL;i}}$ is the extend of the collapse in the perpendicular direction on storey i

The area is illustrated by an example of the Figure E.1.e NA

Figure E.1.e NA — Collapse area illustrated by an example



(5) For buildings consisting of structural members other than those referred to in (4), the collapse area shall be calculated considering the specific structural system and its behaviour during a collapse.

(6) Where an alternative load path may be demonstrated after a local failure, the total collapse area in E.1.5(1) may be limited to the area that fails before the alternative load path is created.

(7) The weight from the collapsed structures shall be taken into account as stated in E.1.5.(4).

Annex E.2 NA, Minimum design measures to enhance structural robustness of bridges

(Normative)

(NDP)

This normative annex E.2 NA substitutes Annex E in EN 1990 and provides rules for enhancing the robustness of bridges.

Annex FF NA: Partial factors for of resistance (Informative)

(NCCI)

FF.1 Partial factors for resistance

(1) The design value of the resistance, R_d, should be determined either by Formula (8.19) when the determination is made on the basis of design values of strength parameters and a calculation model: the Material Factor Approach (MFA); or by Formula (8.20) when the determination is made on the basis of the Resistance Factor Approach (RFA); or by resistances determined directly by tests.

(2) The design values of resistances and the partial factors should be determined by:

a) MFA:

$$R_d = R\left\{\frac{\eta\, X_k}{\gamma_M}; a_d\, ; \sum F_{Ed}\right\} \qquad \text{(FF.1.a (8.19))}$$

where $\gamma_M = \gamma_1\, \gamma_2\, \gamma_3 \,/\, (b_2\, b_3)$

b) RFA:

$$R_d = \frac{1}{\gamma_R} R\{X_{rep}; a_d\, ; \sum F_{Ed}\} \qquad \text{(FF.1.a (8.19))}$$

or

$$R_d = \frac{1}{\gamma_R} R\{\eta\, X_k; a_d\, ; \sum F_{Ed}\} \qquad \text{(FF.1.c (8.20))}$$

where $\gamma_R = \gamma_1\, \gamma_2\, \gamma_3 \,/\, (b_2\, b_3)$

c) Resistance determined by test:

$$R_d = \frac{R_k}{\gamma_R} \qquad \text{(FF.1.d (8.21))}$$

where $\gamma_R = \gamma_1\, \gamma_3 \,/\, b_3$

(3) The sub-partial factors take into account the following:

- γ_1 failure form, see Table FF.1.a NA
- γ_2 uncertainty modelled by the coefficient of variation V_2 related to calculation model, see Table FF.1.b NA
- b_2 bias related to calculation model
- γ_3 uncertainty on strength parameter or resistance when obtained by tests modelled by the coefficient of variation V_3, see Table FF.1.c NA
- b_3 bias related to strength parameter or resistance when obtained by tests

NOTE 1 The division of the partial factors into sub-partial factors is not an expression of a probabilistic consideration only of the factors associated with the individual sub-partial factors.

NOTE 2 Bias can alternatively be included in the calculation model and in determining the characteristic value of the resistance.

(4) The sub-partial factor γ_1 depends on the type of failure of the structure. γ_1 is given in Table FF.1.a NA.

The type of failure is divided into 3 groups:

- Failure with no warning signs
- Failure with warning signs but no residual resistance
- Failure with warning signs and residual resistance

NOTE 1 Failure with no warning refers to failure that occurs without prior warning (no warning sign appears e.g. increased cracking or deformation) and with significant reduction of the resistance immediately after a failure (e.g. in the event of stability failure or brittle fracture).

NOTE 2 Failure with warning but no residual resistance refers to failure where a warning sign appears and can be clearly seen when the max resistance is reached (e.g. increased cracking or deformation) and the resistance is retained for some time after the warning appears.

NOTE 3 Failure with warning and residual resistance refers to failure where the resistance increases (e.g. due to strain hardening) after a formal failure resistance has been reached corresponding to a defined yield strength used to obtain the resistance.

(5) At least one of following conditions should be met if "Warning of failure" is assumed::

- It is verified that the reinforcement is yielding at failure. Strain at failure should be at least 5%.
- Before failure, a pattern of evenly distributed cracks corresponding to the applied load is clearly seen.
- Before failure, the deflection exceeds 3/200 of the span length.

(6) If the residual resistance is utilised directly in the calculation models, then γ_1 = 1,00 applies corresponding to "Warning without residual resistance" (e.g. in case where a stress-strain diagram with an inclined post-elastic branch is used for the reinforcement). If the ultimate resistance is at least 10% larger than the resistance corresponding to yielding strength used in the calculations, then γ_1 = 0,9 applies. If the ultimate resistance is at least 5 % larger than the resistance corresponding to yielding strength used in the calculations, then γ_1 = 0,95 applies.

Table FF.1.a NA — Sub-partial factor γ_1 depending on type of failure

Type of failure	Warning of failure with residual resistance	Warning of failure without residual resistance	No warning of failure
γ_1	0,90 / 0,95	1,00	1,10

(7) The sub-partial factor γ_2 depends on the coefficient of variation for the calculation model. The coefficient of variation should be established by comparing resistances determined by testing the structural members and by applying the calculation model, with the use of measured/given strength parameters and geometric dimensions. As an exception, the coefficient of variation may be determined as an estimate. The coefficient of variation should include the uncertainty associated with the transfer from laboratory conditions to conditions in a real structure. γ_2 is given in Table FF.1.b NA.

NOTE 1 Uncertainty of the calculation model is assumed to have a lognormal distribution with bias b and coefficient of variation V_2 which is equal to V_δ if it is determined by experiments, see D8.3.

NOTE 2 the coefficient of variation V_2 can be obtained from

$$V_2 = \sqrt{V_{2,XR}^2 + V_{2,XRT}^2}$$

where $V_{2,XR}$ is the coefficient of variation of uncertainty of calculation model in laboratory conditions

$V_{2,XRT}$ is the coefficient of variation of uncertainty associated with the transfer from laboratory conditions to conditions in a real structure for calculation model.

Table FF.1.b NA — Sub-partial factor γ_2 for calculation model

Coefficient of variation of the calculation model V_2	**≤5 %**	**10 %**	**15 %**	**20 %**	**25 %**
γ_2	1,00	1,05	1,10	1,15	1,20

(8) The sub-partial factor γ_3 depends on the coefficient of variation for the strength parameter or the resistance when determined based on tests. The coefficient of variation should include the uncertainty associated with the transfer from laboratory conditions to conditions in a real structure. γ_3 is given in Table FF.1.c NA.

NOTE 1 Strength parameter and resistance are assumed to have a lognormal distribution with coefficient of variation V_3 which is equal to V_{rt} if it is determined by experiments, see D8.3.

NOTE 2 The coefficient of variation V_3 can be obtained from

$$V_3 = \sqrt{V_{3,R}^2 + V_{3,RT}^2}$$

where $V_{3,R}$ is the coefficient of variation of strength parameter in laboratory conditions

$V_{3,RT}$ is the coefficient of variation of uncertainty associated with the transfer from laboratory conditions to conditions in a real structure of strength parameter

NOTE 3 If one strength parameter is dominant in a calculation model for the resistance, V_3 can be assumed to be equal to the coefficient of variation for the dominant strength parameter.

Table FF.1.c NA — Sub-partial factor γ_3 for strength parameter or resistance

Coefficient of variation for strength parameter or resistance V_3	**≤5 %**	**10 %**	**15 %**	**20 %**	**25 %**	**30 %**
γ_3	1,175	1,225	1,275	1,325	1,40	1,475

(9) When examining accidental design situations or seismic design situations, the partial factor γ_M = 1,0 should be used unless otherwise stated in EN 1992 to EN 1999 incl. DK NA.

(10) If design is assisted by a limited number tests the material partial factor should be increased by a factor γ_{mod} that depends on the number of tests n, the coefficient of variation of the material or resistance and if the coefficient of variation is known or unknown.

NOTE 1 In most applications with tests, the coefficient of variation of the population is unknown.

NOTE 2 EN1990 Annex D.7.1 (3) NOTE 3 states 'In practice, it is often preferable to use the case "V_X known" together with a conservative upper estimate of V_X, rather than to apply the rules given for the case "V_X unknown". Moreover V_X, when unknown, is assumed to be not smaller than 0,10.' This application is used as basis for the modification factors γ_{mod} in Table FF.1.d NA. and Table FF.1.e NA with known coefficient of variation. In this case the characteristic value is also determined with the conservative known upper estimate of V_X.

(11) Table FF.1.d NA shows the modification factor γ_{mod} to the material partial factor γ_M or γ_R when V_2 and V_3 are of the same magnitude, and the coefficient of variation is unknown and the number of tests is n.

Table FF.1.d NA – Modification factor γ_{mod} to the material partial factor γ_M or γ_R when V_2 and V_3 are of the same magnitude, the coefficient of variation is unknown and the number of tests is *n*.

n **V_2 and V_3**	**3**	**4**	**5**	**6**	**8**	**10**	**15**	**20**
0,05	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
0,10	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
0,15	1,42	1,18	1,11	1,08	1,05	1,03	1,02	1,01
0,20	2,27	1,44	1,25	1,17	1,11	1,08	1,04	1,03
0,25	5,26	2,00	1,52	1,34	1,20	1,14	1,08	1,06

(12) Table FF.1.e NA shows the modification factor γ_{mod} to the material partial factor γ_M or γ_R when V_2 or V_3 is dominant, and the coefficient of variation is unknown and the number of tests is *n*.

Table FF.1.e NA – Modification factor γ_{mod} to the material partial factor γ_M or γ_R when V_2 or V_3 is dominant, the coefficient of variation is unknown and the number of tests is *n*.

n **V_2 or V_3**	**3**	**4**	**5**	**6**	**8**	**10**	**15**	**20**
0,07	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
0,14	1,89	1,32	1,19	1,13	1,08	1,06	1,03	1,02
0,21	-	3,71	2,10	1,65	1,35	1,24	1,13	1,09
0,28	-	-	4,92	2,85	1,83	1,52	1,26	1,18

(13) Table FF.1.f NA shows the modification factor γ_{mod} to the material partial factor γ_M or γ_R when V_2 and V_3 are of the same magnitude, and the coefficient of variation is known and the number of tests is *n*.

Table FF.1.f NA – Modification factor γ_{mod} to the material partial factor γ_M or γ_R when V_2 and V_3 are of the same magnitude, the coefficient of variation is known and the number of tests is *n*.

n **V_2 and V_3**	**3**	**4**	**5**	**6**	**8**	**10**	**15**	**20**
0,05	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
0,10	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
0,15	1,01	1,01	1,01	1,01	1,00	1,00	1,00	1,00
0,20	1,02	1,02	1,01	1,01	1,01	1,01	1,00	1,00
0,25	1,04	1,03	1,02	1,02	1,01	1,01	1,01	1,01

(14) Table FF.1.g NA shows the modification factor γ_{mod} to the material partial factor γ_M or γ_R when V_2 or V_3 is dominant, and the coefficient of variation is known and the number of tests is *n*.

Table FF.1.g NA – Modification factor γ_{mod} to the material partial factor γ_M or γ_R when V_2 or V_3 is dominant, the coefficient of variation is known and the number of tests is *n*.

n **V_2 or V_3**	**3**	**4**	**5**	**6**	**8**	**10**	**15**	**20**
0,07	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
0,14	1,02	1,01	1,01	1,01	1,01	1,01	1,00	1,00
0,21	1,04	1,03	1,03	1,02	1,02	1,01	1,01	1,01
0,28	1,08	1,06	1,05	1,04	1,03	1,02	1,02	1,01