DS/EN 1990 DK NA:2013

National Annex to
Eurocode: Basis of structural design

Foreword

This National Annex (NA) is a consolidation and revision of DS/EN 1990 DK NA 2010 and DS/EN 1990 DK NA Addendum 1:2010 and supersedes these documents as of 2013-05-15. For a transition period until 2013-09-01, this National Annex as well as the previous National Annex will be applicable. This NA does not deal with bridges and therefore EN 1990/A1:2005 and EN 1990/A1/AC:2010 have not been taken into consideration.

In addition to the consolidation and editorial changes, the contents of Table A.1.1 DK NA regarding natural actions, clause A.1.3 and Annex F (2) have been considerably modified.

Previous versions, addenda and an overview of all National Annexes can be found at www.Eurocodes.dk

This national Annex (NA) lays down the conditions for the implementation in Denmark of EN 1990 for construction works in conformity with the Danish Building Act or the building legislation. Other parties can put this NA into effect by referring thereto.

The national choices may be in the form of nationally applicable values, an option between methods given in the Eurocode, or the addition of complementary information.

This NA includes:

- an overview of possible national choices and complementary information;
- national choices;
- complementary (non-contradictory) information which may assist the user of the Eurocode.

The numbering refers to the clauses containing choices and/or complementary information. To the extent possible, the heading/subject is identical to the heading of the clause, but as references are at a more detailed level than the headings, the heading/subject has in several cases been made more explicit.

Overview of possible national choices and clauses containing complementary information

The list below identifies the clauses where national choices are possible and the applicable/not applicable informative annexes. Furthermore, clauses giving complementary information are identified. Complementary information is given at the end of this document.

Clause	Subject	Choice	Complementary information
A1.1(1)	Field of application (design working life)	Unchanged:	
A1.2.1(1)	Combinations of actions, General Modifications of combinations of actions for geographical reasons	Unchanged	
A1.2.2 / Table A1.1	Values of ψ factors	National choice	
A1.3.1(1)/ Table A1.2(A)-(C)	Design values of actions in persistent and transient design situations	National choice	
A1.3.1(5)	Design values of actions in persistent and transient design situations: Choice of approach regarding geotechnical actions	National choice	
A1.3.2 (Table A1.3)	Design values of actions in accidental and seismic design situations	National choice	
A1.4.2(2)	Serviceability criteria	National choice	
A1.4.3	Deformations and horizontal displacements		Complementary information
A1.4.4	Vibrations	National choice	
Annex B	Management of structural reliability for construction works		Complementary information
Annex C	Basis for partial factor design and reliability analysis		Complementary information
Annex D	Design assisted by testing		Complementary information
Annex E	Robustness		Complementary rules
Annex F	Partial factors for resistance		Complementary rules

NOTE Unchanged: Recommendations in the Eurocode are followed.

National choices

A1.2.2/Table A1.1 DK NA Recommended values of ψ factors for buildings

Values given in Table A.1.1 DK NA.

Table A1.1 DK NA ψ factors for buildings

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, see EN 1991-1-1			
Category A: domestic, residential areas	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	0,8	0,8	0,7
Category F: traffic area, gross vehicle weight: ≤ 30 kN	0,6	0,6	0,5
Category G: traffic area, 30 kN < gross vehicle weight ≤ 160 kN	0,6	0,4	0,2
Category H: roofs	0	0	0
Snow loads			
For combinations with leading imposed loads of category E or leading thermal actions	0,6	0,2	0
For combinations with leading wind actions	0	0	0
otherwise	0,3	0,2	0
Wind actions			
For combinations with leading imposed loads of category E	0,6	0,2	0
otherwise	0,3	0,2	0
Thermal actions	0,6	0,5	0

A1.3.1(1) / Table A1.2(A)-(C) DK NA Design values of actions in persistent and transient design situations

Combinations of actions and partial factors for EQU, UPL, STR and GEO are listed in Tables A1.2(A) DK NA and A1.2(B+C) DK NA.

Table A1.2(A) DK NA Design values of actions for persistent and transient design situations (EQU and UPL) (Set A)

Limit state				**EQU / UPL**	**UPL**
Combination of actions				**1**	**2**
Reference formula				**(6.10)**	**(6.10)**
Permanent action	Weight, general (**)	Unfavour-able	$\gamma_{Gj,sup}$	$1{,}1 \cdot K_{FI}$	$1{,}0 \cdot K_{FI}$
		Favourable	$\gamma_{Gj,inf}$	0,9	1,0
	Weight of soil and (ground) water, geotech-nical structures (***)	Unfavour-able	$\gamma_{Gj,sup}$	$1{,}1 \cdot K_{FI}$	$1{,}05 \cdot K_{FI}$
		Favourable	$\gamma_{Gj,inf}$	0,9	1,0
Variable ac-tion (*)	Leading	Unfavour-able	$\gamma_{Q,1}$	$1{,}5 \cdot K_{FI}$	$1{,}5 \cdot K_{FI}$
	Other	Unfavour-able	$\gamma_{Q,i}$	$1{,}5 \cdot \psi_0 \cdot K_{FI}$	$1{,}5 \cdot \psi_0 \cdot K_{FI}$

(*) Variable actions are those considered in Table A.1.1 DK NA.
(**) Comprises all types of permanent self weight, see clause 2.1 in EN 1991-1-1.
(***) Comprises the weight of soil and (ground) water affecting the geotechnical structure as geotechnical action, see 1.5.2.1 in EN 1997-1.

NOTE 1 Combination of actions 2 is applied only for geotechnical structures where the water pressure is maximised in the case of overflow arrangements, see DS/EN1997-1 DK NA.

NOTE 2 K_{FI} depends on the consequences class defined in Annex B, Table B3, as follows:

- consequences class CC3: $K_{FI} = 1{,}1$
- consequences class CC2: $K_{FI} = 1{,}0$
- consequences class CC1: $K_{FI} = 1{,}0$.

Consequences class CC1 is not applied for geotechnical structures.

NOTE 3 Anchors or similar devices added in order to achieve static equilibrium is to be designed to accommodate the design force that is needed to ensure static equilibrium.

Table A1.2(B+C) DK NA Design values of actions for persistent and transient design situations (STR/GEO) (sets B and C)

Limit state				**STR/GEO**				**STR**
Combination of actions				**1**	**2**	**3**	**4**	**5**
Reference formula				**(6.10a)**	**(6.10b)**	**(6.10a)**	**(6.10b)**	**(6.10a)**
Partial factors for actions								
Permanent action	Weight, general (**)	Unfavourable	$\gamma_{G;sup} \cdot K_{FI}$	$1,2 \cdot K_{FI}$	$1,0 \cdot K_{FI}$	1,2	1,0	1,0
		Favourable	$\gamma_{G;inf}$	1,0	0,9	1,0	0,9	1,0
	Weight of soil and (ground) water, geotechnical structures (***)	Unfavourable	$\gamma_{G;sup}$	1,0	1,0	1,0	1,0	1,0
		Favourable	$\gamma_{G;inf}$	1,0	1,0	1,0	1,0	1,0
Variable action (*)	Leading	Unfavourable	$\gamma_{Q,1} \cdot K_{FI}$	0	$1,5 \cdot K_{FI}$	0	1,5	0
	Other	Unfavourable	$\gamma_{Q,i} \cdot K_{FI}$	0	$1,5 \cdot \psi_0 \cdot K_{FI}$	0	$1,5 \cdot \psi_0$	0
Coefficient applied to partial factors for strength parameters and resistance								
Structural materials, cf. EN 1992 - 1996 and 1999			γ_0	1,0	1,0	K_{FI}	K_{FI}	$1,2\ K_{FI}$
Soil parameters and resistance, cf. EN 1997-1			γ_0	1,0	1,0	K_{FI}	K_{FI}	1,0 ($\gamma_M = \gamma_R = 1,0$)

(*) Variable actions are those considered in Table A.1.1 DK NA.
(**) Comprises all types of permanent self weight, see clause 2.1 in EN 1991-1-1.
(***) Comprises the weight of soil and (ground) water affecting the geotechnical structure as geotechnical action, see 1.5.2.1 in EN 1997-1.

NOTE 1 Equations 6.10a and 6.10b are applied for STR as well as GEO. Equation 6.10a relates only to permanent actions.

NOTE 2 For structures not subject to geotechnical actions, verification can be achieved solely by applying combinations of actions 1 and 2.

For structures subject to geotechnical actions, verification is to be achieved by applying combinations of actions 1 and 2.

For structures solely subject to geotechnical actions, verification may be achieved by applying combinations of actions 3 and 4 and combination of actions 5.

For $K_{FI} = 1,0$, combinations of actions 1 and 2 are identical to combinations of actions 3 and 4. For $K_{FI} \neq 1,0$, the factor K_{FI} may be applied to the load effects (internal forces and moments) instead of to the action, provided that the load effects are linearly proportional to the associated action.

Geotechnical actions are actions transmitted to the structure by the ground, fill, standing water or ground water. In addition to the weight, the action from the ground and fill is determined by the strength and deformation properties of the ground and fill, e.g. expressed as the angle of friction. Examples of geotechnical actions include earth and water pressures on a wall structure.

NOTE 3 – Coefficient γ_0 for the partial factor for strength parameters and resistances is obtained as follows.

For combinations of actions 3 and 4 used for geotechnical structures, cf. EN 1997-1, the K_{FI} factor applies to all relevant partial factors for the strength parameters and resistance of the ground and for the material strengths and resistances, respectively.

For combination of actions 5 which is used for verification of STR for structural materials forming part of geotechnical structures, the usual partial factors are applied for structural materials multiplied by 1,2 K_{FI}. For strength parameters and resistances of the ground, a partial factor of $\gamma_M = \gamma_R$ =1,0, cf. EN 1997-1, is applied.

NOTE 4 K_{FI} depends on the consequences class defined in Annex B, Table B3, as follows:

- consequences class CC3: K_{FI} = 1,1
- consequences class CC2: K_{FI} = 1,0
- consequences class CC1: K_{FI} = 0,9

Consequences class CC1 is not applied for geotechnical structures.

See also EN 1991 to EN 1999 for γ values for imposed deformations.

NOTE 5 - The characteristic values of all permanent actions from one source are multiplied by $\gamma_{Gj,sup}$ if the total resulting action effect is unfavourable and by $\gamma_{Gj,inf}$ if the total resulting action effect is favourable. As an example all actions originating from the self weight of the structure may be considered as coming from one source; this also applies if different materials are involved.

Design values for fatigue actions

(1) Design values for fatigue actions should be determined by applying a partial factor equal to 1,3 for loads where the uncertainty of the individual spans are described by a coefficient of variation of the magnitude of 30%. For loads where the coefficient of variation is less than 10%, a partial factor equal to 1,0 is applied. For other values of the coefficient of variation, the partial factor should be determined by linear interpolation. The coefficient of variation may be stated in connection with the action specification.

A1.3.1(5) Design values of actions in persistent and transient design situations - Choice of design approach for geotechnical actions

Design Approach 3 is applied, see DS/EN 1997-1 DK NA.

A1.3.2 Design values of actions in accidental and seismic design situations

Combinations of actions are listed in Table A1.3 DK NA.

Table A1.3 DK NA Design values of actions for use in accidental and seismic combinations of actions

Design situation	Permanent actions		Leading accidental or seismic action	Accompanying variable actions*	
	Unfavourable	Favourable		Main (if any)	Other
Fire (Formula 6.11*a*/*b*)	$G_{\mathrm{kj,sup}}$	$G_{\mathrm{kj,inf}}$	A_{d}	$\psi_{1,1}\,Q_{\mathrm{k,1}}$	$\psi_{2,\mathrm{i}}\,Q_{\mathrm{k,i}}$
Other accidental (Formula 6.11*a*/*b*)	$G_{\mathrm{kj,sup}}$	$G_{\mathrm{kj,inf}}$	A_{d}	$\psi_{2,1}\,Q_{\mathrm{k,1}}$	$\psi_{2,\mathrm{i}}\,Q_{\mathrm{k,i}}$
Seismic (Formula 6.12*a*/*b*)	$G_{\mathrm{kj,sup}}$	$G_{\mathrm{kj,inf}}$	A_{d}	$\psi_{2,\mathrm{i}}\,Q_{\mathrm{k,i}}$	

*) Variable actions are those considered in Table A.1.1 DK NA.

NOTE 1 – Seismic actions are used to evaluate the structure for the seismic design situation. Seismic actions do not include imperfections of the structure as imperfections are considered according to rules specified in the individual Eurocodes for materials.

Seismic actions include actions taken into account in order to safeguard the structure's strength and stability from small ground motions. Seismic actions are the smallest horizontal actions assumed to affect a structure.

All vertical actions are assumed to be capable of contributing to the calculation of seismic actions. Seismic actions are taken as fixed actions. Seismic actions are assumed to occur simultaneously with the associated vertical actions only.

Seismic actions act at the centres of gravity of the associated vertical actions and are assumed to be capable of acting in any horizontal direction, but such that this direction is the same for all seismic actions occurring at the same time.

The design value of the seismic action, A_d, is determined on the basis of the vertical action as follows:

$$A_d = 1{,}5\%\left(\sum G_{k,j} \text{ "+" } \sum_{i\geq 1} \psi_{2,i} Q_{j,i}\right)$$

Structures are not to be designed for seismic and wind actions acting simultaneously.

A1.4.2(2) Serviceability criteria

Empirical values for vertical vibrations are given in clause A1.4.4 of this NA.

A1.4.3 Deformations and horizontal displacements

For serviceability limit states that relate to the functionality and appearance of the structure, reference is made to EN 1992-1999.

A1.4.4 Vibrations - Vertical

Requirements regarding natural frequencies may be based on the empirical values in Table A1.4 DK NA. If a more detailed analysis is carried out, the functioning of the structure will normally be satisfactory if the variation of the structure's accelerations originating from the stated action does not exceed the acceleration limit in the table.

The risk of unsatisfactory functioning increases with increasing span and the risk is particularly great for lightweight or poorly damped structures. For these structures, the natural frequency requirement in the table does not always result in satisfactory functioning.

Table A1.4 DK NA Empirical values for acceptable natural frequencies and acceleration limits

Structure	Action	Normally satisfactory functioning	Often unsatisfactory functioning	Acceleration limit in % of the gravity acceleration
Grandstands, fitness centres, sports halls and public premises	Rhythmic load caused by movement of people	$n_e > 10$ Hz	$n_e < 6$ Hz	10 %
Residential buildings	Load from walking	$n_e > 8$ Hz	$n_e < 5$ Hz	0,1 %
Office premises	Load from walking	$n_e > 8$ Hz	$n_e < 5$ Hz	0,2 %

NOTE - Natural frequencies and accelerations are calculated during normal use, where the fluctuating action is typically considerably less than the action corresponding to the quasi-permanent combination specified in clause 6.5.3 of EN 1990.

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Complementary (non-contradictory) information

Annex B - Management of structural reliability for construction works

Annex B may be used with the following modifications:

- Table B1 (Consequences classes)
- Table B2 (Minimum values for reliability index)
- Clause B4 (Design supervision differentiation)
- Clause B5 is not applied
- Clause B6 is not applied.

Table B1 DK NA Definition of consequences classes

Consequences class	Consequences of possible damage	Examples
CC3 High consequences class	High risk of loss of human life, *or* considerable economic, social or environmental consequences	– Buildings with several storeys where the height to the floor of the uppermost storey is more than 12 m above the ground, if they are often used for accommodating people, e.g. residential or office buildings – Buildings with large spans, if they are often used by many people, e.g. for concerts, sporting events, theatrical performances, or exhibitions – Grandstands – Large road bridges and tunnels – Large masts and towers – Large silos near a built-up area – Dams and similar structures where a failure would result in considerable damage.
CC2 Medium consequences class	Medium risk of loss of human life. Considerable economic, social or environmental consequences.	Buildings or structures not belonging to CC3 or CC1.
CC1 Low consequences class	Low risk of loss of human life, *and* small or negligible economic, social or environmental consequences	– 1 and 2 storey buildings with moderate spans, which people enter only occasionally, e.g. storage buildings, sheds and small agricultural buildings – Small masts and towers, including general street masts – Small silos – Secondary structural members, e.g. partitions, window and door lintels and cladding

(1) The consequences for adjacent structures and surroundings can be decisive when determining the consequences class.

(2) Structural members that are not part of the main structure can often be referred to a lower consequences class than the main structure.

NOTE The main structure is that part of a load-bearing structure the failure of which will have considerable consequences for the reliability and functionality of the entire structure. Examples of structural members that are often considered not to be part of the main structure include roofs, independent decks, stairways and balconies.

Table B2 DK NA Minimum values for reliability index *ß* (ultimate limit states) for a 1 year reference period

Reliability class	Minimum values of β
RC3 corresponding to CC3	4.7
RC2 corresponding to CC2	4.3
RC1 corresponding to CC1	3.8

NOTE It is assumed for the determination of the reliability index for RC2 that permanent actions have a normal distribution and variable actions have a Gumbel distribution. All strength parameters and model uncertainties should be assumed to have a log-normal distribution. Information on the choice of coefficients of variation are given in DS/INF 172 *Background investigations in relation to the drafting of National Annexes to EN 1990 and EN 1991 - Reliability verification formats, combination of actions, partial coefficients, fatigue, snowload, windload, etc.*(Available in Danish only). The reliability index *ß* is defined in Annex C.

B4 Design supervision differentiation

(1) Design supervision includes checking of the project material relating to the load-bearing structures, viz. project basis, statistical calculations, drawings/models and execution specifications. The design brief is the specifications on which the design is based, including the static system and mode of operation, robustness, fire, material data, action data, etc.

NOTE Design supervision is to contribute to ensuring:
- that the assumptions of the design brief are correct and are used as a basis for the structural design;
- that the assumptions made in the static calculations have been correctly incorporated into any other project material;
- that drawings and execution specifications are adequate for the execution of the load-bearing structures.

(2) Design supervision, except self-checking, is to be documented in accordance with guidelines drawn up in advance. The method, scope, any points of focus and the results of the design supervision is to be stated in the documentation.

(3) For all project material, the people responsible for preparation and design supervision, respectively, are to be identified.

(4) For structures in consequences class CC3 where the consequences of failure are particularly serious, special requirements apply to design supervision.

(5) Examples of structures covered by (4) include:
- buildings with more than 15 storeys above ground level, if they are used for accommodating persons, e.g. for residential, office or educational buildings;
- hospitals with more than 5 storeys above ground level;
- industrial buildings where failure would have a particularly major impact on society;

- buildings with large spans, provided they are used by many people, e.g. for concerts, theatrical performances, exhibitions, sporting events, or entertainments;
- grandstands.

(6) The following types of supervision are applied in connection with design: self-checking, independent checking and third party checking. The types of supervision are defined in Table B4a DK NA.

Table B4a DK NA Definition of types of supervision

Type of supervision	Definition
Self-checking	Checking performed by the person who has prepared the design
Independent checking	Checking by different persons than those involved in the design of the structure
Third party checking	Checking by an organisation that is neither directly nor indirectly linked financially to the organisation(s) involved in the design of the structure

(7) The minimum requirements for the type of supervision depend on the consequences class to which the structure is assigned. The minimum requirements are specified in Table B4b DK NA.

Table B4b Minimum requirements for types of supervision for project material

Consequences class	Self-checking	Independent checking	Third party checking
CC1	X		
CC2	X	X *)	
CC3	X	X	
CC3 if covered by (4)	X	X	X

*) The requirement for independent checking in CC2 applies to the design brief only. For any other project material, checking may be carried out by persons who have not been involved in the design of the relevant section of the structure.

B6 Partial factors for resistance

Comment:

This clause is not applied. Reference is made to Annex F (7) for complementary rules concerning the determination of partial factors for resistance, according to the level of checking.

Annex C Basis for partial factor design and reliability analysis

The Annex may be used with a changed Table C2 DK NA (target reliability indices).

Table C2 - Target reliability index β for class RC2 structural members [1]

Limit state	Target reliability index	
	1 year	50 years
Ultimate	4,3	3,3
Fatigue		1,5 to 3,3 [2]
Serviceability (irreversible)	2,9	1,5

[1] See Annex B.
[2] Depends on the degree of inspectability, reparability and damage tolerance.

Annex D Design assisted by testing

The Annex may be used with the exception of D7.3 and D8.3; see comment.

Comment:
Annex D may be used to check characteristic values and to establish characteristic values and design values. Clauses D7.3 and D8.3 may not be used as they assume a reliability level corresponding to $\beta = 3,8$ and application of the design approach in Annex C. Instead reference is made to Annex F in which the determination of material partial factors and design values is described.

Annex E Robustness

Complementary rules for the verification of robustness

This Annex may be used for the examination of robustness, see 2.1.4(P) - 2.1.5(P).

(1) A structure is robust:

- when the parts of the structure that are decisive for safety are only slightly sensitive to unintended effects and defects; or
- when there is no extensive failure of the structure if a limited part of the structure fails.

(2) Examples of unintended effects and defects include:

- unforeseen action effects;
- unintended discrepancies between the structure's actual behaviour and the design models used;
- unintended discrepancies between the implemented project and the project material;
- unforeseen geometrical imperfections;
- unforeseen settlements;
- unforeseen degeneration.

Increased robustness may in certain cases also help to reduce the effects of any gross errors, although verification of robustness neither can nor must be regarded as designing against gross error.

(3) Robustness is discussed in more detail in DS/INF 146 *Robustness - Background and principles* (available in Danish only).

(4) The robustness of a structure should be proportional to the consequences of a failure of the structure. Documentation of robustness is only required for structures in consequences class CC3. However, for structures in consequences class CC2 an assessment of the robustness is to be made. The amount of detail of the assessment is to be increased in the case of large spans, large concentrated loads, few supports and special (rare or new) types of structures.

(5) A robust structure is achieved by an appropriate choice of materials, general static principle and construction and by appropriate design of key members. A key member is a restricted part of the structure that, in spite of its limited extent, is of central importance to the robustness of the structure such that failure of this member would result in the failure of the whole structure or significant parts of the structure.

(6) Where documentation of robustness is required, an expert engineering report is to be drawn up verifying that at least one of the robustness criteria specified in (1) is met. This is achieved

– by verifying that the essential parts of the structure, i.e. key members, have low sensitivity to unintended effects and defects, cf. (2);
– by verifying that no extensive failure of the structure occurs if a limited part of the structure fails (loss of a member), see (7)-(8);
– by verifying adequate safety of key members, such that the whole structure to which they belong attains at least the same level of system safety as an equivalent structure for which the robustness is documented by verification of adequate safety in the event of the "loss of a member".

In addition to the verification itself, the expert engineering report is to contain a critical evaluation of the construction, including identification of key members and action scenarios.

Verification that the first criterion has been fulfilled is only possible in special cases, and therefore verification is usually performed by verifying one of the two latter criteria.

(7) Where robustness is verified by "loss of a member", the acceptable extent of collapse for buildings of up to 15 storeys be taken as: collapse of no more than two floors, extending in this case to two vertically adjacent floors. At each of the two floors, the extent of collapse is not to affect more than 15% of the floor space, and no more than 240 m^2 per floor and no more than a total area of 360 m^2. Adequate resistance is verified in an accidental design situation by using the formula (6.11 a/b), see Table A1.3 DK NA.

(8) Robustness verified in the event of "loss of a member" may, for residential and grandstand structures, be regarded as met if it is verified that the damaged structure will continue to constitute a stable system even if one or more structural members are lost. It is assumed that failure may comprise the equivalent of the maximum permissible extent of collapse, cf. (7), including:
– either a floor or roof structure and an arbitrary pillar;
– or a floor or roof structure and an arbitrary piece of wall 3 m in length or width.

The ability of a structure to retain its coherence after a failure of the specified extent is primarily conditional upon the damaged structure continuing to constitute a stable system, which means that the structure or large parts of it are not transformed into a mechanism. If this condition is met, a rough calculation will be sufficient.

(9) Where robustness is verified by introducing additional reliability of key members, this can usually be achieved by applying a material partial factor γM, which has been increased by the factor 1,2 compared to the value specified in 6.3.5. With respect to modelling this is equivalent to a system with key members in series having the same system reliability as a system of parallel members.

As a general rule, every effort should be made in the design to document the robustness of a structure as far as possible without the use of increased safety factors on the key members. Where increased safety factors are applied to the key members, it should however be ensured that the resistance of the structure to unintended effects and defects is actually increased.

NOTE For example, the robustness of hinged pillars in a residential building will not generally be sufficiently secured by applying a factor of 1,2, unless at the same time a structural connection is arranged through each building floor in the form of a continuous tensile and shear connector in the pillar.

(10) The structural Eurocodes may provide guidelines for adequately ensuring robustness.

Annex F (informative) Partial factors for resistance

Complementary rules for establishing partial factors for resistance

(1) The design resistance value, R_d, should be determined either by formula (6.6a) if it is determined on the basis of design strength parameters and a calculation model, or by formula (6.6c) if it is determined on the basis of measured characteristic resistances.

(2) The partial factors for strength parameters and resistance should be determined using the following expressions:

$$R_d = R\left\{\eta_i \frac{X_{k,i}}{\gamma_{M,i}\gamma_0}\right\}; a_d, \tag{6.6a}$$

where

$$\gamma_M = \gamma_m \gamma_R$$

$$\gamma_m = \gamma_4$$

$$\gamma_R = \gamma_1 \gamma_2 \gamma_3$$

$$R_d = \frac{R_k}{\gamma_M \gamma_0} \tag{6.6c}$$

$$R_d = \frac{1}{\gamma_{,1}\gamma_0} R\left\{\eta_1 X_{k,1}; \eta_i X_{k,i(i>1)} \frac{\gamma_{m,1}}{\gamma_{m,i}}; a_d\right\}$$

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The sub-partial factors take account of the following:

- γ_1 failure mode, see Table F.2
- γ_2 uncertainty related to the calculation model, see Table F.3
- γ_3 scope of checking, see Table F.4
- γ_4 uncertainty of measured strength parameter or resistance, see Table F.1.

The factor γ_0 is applied to the partial factor γ_M for strength parameters and resistances (and γ_R for resistance according to EN 1997-1), depending on the combination of actions, see Table A1.2(B+C) DK NA.

(3) Division of the partial factors into sub-partial factors does not imply a probability theoretical consideration of the conditions associated with the individual sub-partial factor only.

(4) The sub-partial factor γ_4 depends on the coefficient of variation for the measured strength parameter or resistance. The coefficient of variation is to include the uncertainty associated with the transfer from laboratory conditions to conditions in an actual structure. γ_4 is given in Table F1 DK NA.

Table F.1 DK NA Sub-partial factor γ_4 for measured strength parameter or resistance

Coefficient of variation for measured strength parameter or resistance	**≤ 5 %**	**10 %**	**15 %**	**20 %**	**25 %**	**30 %**
γ_4	1,15	1,20	1,25	1,30	1,35	1,40

(5) The sub-partial factor, γ_1, depends on the type of failure of the structure. γ_1 is given in Table F2 DK NA.

No warning of failure refers to failure that occurs without prior warning (e.g. in the form of increased crack formation or deformation) and significant reduction of resistance immediately after a failure (e.g. in the event of stability failure or brittle fracture).

Warning of failure without residual resistance refers to failure where a warning is given of exhausted resistance (e.g. in the form of increased crack formation or deformation) and the resistance is retained for some time after the warning.

Warning of failure with residual resistance refers to failure where the resistance increases (e.g. as a result of strain hardening) after a formal failure has occurred (e.g. in the event of the permissible strain being exceeded). If the residual resistance is utilised in the calculation models, the failure type is to be taken as "Warning of failure without residual resistance".

Table F2 DK NA Sub-partial factor γ_1 depending on type of failure

Type of failure	Warning of failure with residual re-sistance	Warning of failure without residual re-sistance	No warning
γ_1	0,90	1,00	1,10

(6) The sub-partial factor γ_2 depends on the coefficient of variation for the calculation model. The coefficient of variation is 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. γ_2 is given in Table F3 DK NA.

Table F.3 DK NA Sub-partial factor γ_2 for uncertainty of the calculation model

Coefficient of variation of the calculation model	$\leq$5 %	10 %	15 %	20 %	25 %
γ_2	1,05	1,10	1,15	1,20	1,25

(7) Sub-partial factor γ_3 depends on the level of checking in connection with the production of components and execution at the construction site. Requirements for levels of checking may be given in EN 1992 to EN 1999 and in the Danish national annexes thereto. γ_3 is given in Table F4 DK NA. The extended level of checking is used on the condition that third party checking is conducted.

Table F4 DK NA Sub-partial factor γ_3 dependent on the scope of checking in connection with the production of components and execution at the construction site.

Level of checking	Extended	Normal	Reduced
γ_3	0,95	1,00	1,10

(8) In (2), γ_4 covers the variation of the strength parameter. By checking the strength parameter it will be possible to evaluate both the characteristic value and the coefficient of variation, which may differ from what was assumed when the partial factor was set, see EN 1992 to EN 1998.

(9) When examining accidental design situations or seismic design situations, the partial factor γ_M = 1,0 is used unless otherwise stated in EN 1992, EN 1993, EN 1994, EN 1995, EN 1996, EN 1997, EN 1998 or EN 1999.