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National Annex –
Nationally determined parameters –
Eurocode 8: Design of structures for earthquake resistance –
Part 1: General rules, Seismic actions and rules for buildings
English translation of DIN EN 1998-1/NA:2011-01

Nationaler Anhang –
National festgelegte Parameter –
Eurocode 8: Auslegung von Bauwerken gegen Erdbeben –
Teil 1: Grundlagen, Erdbebeneinwirkungen und Regeln für Hochbau
Englische Übersetzung von DIN EN 1998-1/NA:2011-01

Annexe Nationale –
Paramètres déterminés au plan national –
Eurocode 8: Calcul des structures pour leur résistance aux séismes –
Partie 1: Règles générales, actions sismiques et règles pour les bâtiments
Traduction anglaise de DIN EN 1998-1/NA:2011-01

Supersedes DIN EN 1998-1/NA:2010-08;
together with DIN EN 1998-1:2010-12 and DIN EN 1998-5:2010-12 supersedes DIN 4149:2005-04, withdrawn 2010-12

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Foreword

This document has been prepared by Working Committee NA 005-51-06 AA *Erdbeben; Sonderfragen (SpA CEN/TC 250/SC 8)* of the *Normenausschuss Bauwesen* (Building and Civil Engineering Standards Committee).

This document is the National Annex to DIN EN 1998-1:2010-12 *Eurocode 8: Design of structures for earthquake resistance — Part 1: General rules, seismic actions and rules for buildings.*

European Standard EN 1998-1 allows national safety parameters, referred to as Nationally Determined Parameters (NDPs), to be specified for a number of points. The NDPs cover alternative verification methods, the provision of individual values and the selection of classes from designated classification systems. The relevant parts of the text are identified in the European Standard by references to the possibility of national choice and are listed in Clause NA.2.1. This National Annex also includes non-contradictory complementary information (NCI) for the application of DIN EN 1998-1:2010-12.

This National Annex is an integral part of DIN EN 1998-1:2010-12.

DIN EN 1998-1:2010-12 and this National Annex, DIN EN 1998-1/NA:2011-01, together with DIN EN 1998-5:2010-12 and DIN EN 1998-5/NA supersede DIN 4149:2005-04.

Amendments

This document differs from DIN 4149:2005-04 as follows:

a) A number of provisions from DIN 4149:2005-04 have been incorporated and are to be taken into account when applying DIN EN 1998-1:2010-12 nationally.

Previous editions

DIN 4149: 1981-04; 2005-04
DIN 4149-1: 1981-04
DIN 4149-1/A1: 1992-12
DIN EN 1998-1/NA: 2010-08

NA.1 Scope

This National Annex contains national provisions relating to the design and construction of buildings and civil engineering works in seismic regions that are to be taken into consideration when applying DIN EN 1998-1:2010-12 in Germany.

This National Annex is only valid in conjunction with DIN EN 1998-1:2010-12.

NA.2 National provisions for the application of DIN EN 1998-1:2010-12

NA.2.1 General

DIN EN 1998-1:2010-12 refers to the option of choosing Nationally Determined Parameters (NDP) at the following places in the text.

Table NA.1 — National provisions — Text references in EN 1998-1

Subclause	Subject
1.1.2(7)	Informative Annexes A and B.
2.1(1)P	Reference return period T_{NCR} of seismic action for the no-collapse requirement (or, equivalently, reference probability of exceedance in 50 years P_{NCR}).
2.1(1)P	Reference return period T_{DLR} of seismic action for the damage limitation requirement (or, equivalently, reference probability of exceedance in 10 years P_{DLR}).
3.1.1(4)	Conditions under which ground investigations additional to those necessary for design for non-seismic actions may be omitted and default ground classifications may be used.
3.1.2(1)	Ground classification scheme taking account of deep geology; specification of values of parameters S, T_B, T_C and T_D defining horizontal and vertical elastic response spectra in accordance with 3.2.2.2 and 3.2.2.3.
3.2.1(1), (2),(3)	Seismic zone maps and reference ground accelerations therein.
3.2.1(4)	Governing parameter (identification and value) for threshold of low seismicity.
3.2.1(5)	Governing parameter (identification and value) for threshold of very low seismicity.
3.2.2.1(4), 3.2.2.2(1)P	Parameters S, T_B, T_C, T_D defining the shape of horizontal elastic response spectra.
3.2.2.3(1)P	Parameters a_{vg}, T_B, T_C, T_D defining the shape of vertical elastic response spectra.
3.2.2.5(4)P	Lower bound factor β on design spectral values.
4.2.3.2(8)	Reference to definitions of centre of stiffness and of torsional radius in multi-storey buildings, meeting or not meeting conditions (a) and (b) of **4.2.3.2(8)**.

Table NA.1 *(continued)*

4.2.4(2)P	Values of φ for buildings.
4.2.5(5)P	Importance factor γ_I for buildings.
4.3.3.1(4)	Decision on whether non-linear methods of analysis may be used for the design of non-base-isolated buildings. Reference to information on member deformation capacities and the associated partial factors for the Ultimate Limit State for design or evaluation on the basis of non-linear analysis methods.
4.3.3.1(8)	Threshold value of the importance factor γ_I, relating to the permitted use of analysis with two planar models.
4.4.2.5(2)	Overstrength factor γ_{Rd} for horizontal diaphragms.
4.4.3.2(2)	Reduction factor v for displacements at the damage limitation state
5.2.1(5)	Geographical limitations on the use of ductility classes for concrete buildings.
5.2.2.2(10)	q_o value for concrete buildings for which a special Quality System Plan applies.
5.2.4(1), (3)	Material partial factors for concrete buildings to be used for the seismic design situation.
5.4.3.5.2(1)	Minimum web reinforcement of large lightly reinforced concrete walls.
5.8.2(3)	Minimum cross-sectional dimensions of concrete foundation beams.
5.8.2(4)	Minimum thickness and reinforcement ratio of concrete foundation slabs.
5.8.2(5)	Minimum reinforcement ratio of concrete foundation beams.
5.11.1.3.2(3)	Ductility class of precast wall panel systems.
5.11.1.4	q-factors of precast systems.
5.11.1.5(2)	Seismic action during erection of precast structures.
5.11.3.4(7)e	Minimum longitudinal reinforcement in grouted connections of large panel walls.
6.1.2(1)	Upper limit of q for low-dissipative structural behaviour concept; limitations on structural behaviour concepts; geographical limitations on the choice of ductility classes for steel buildings.
6.1.3(1)	Material partial factors for steel buildings in the seismic design situation.
6.2(3)	Overstrength factor for capacity design of steel buildings.
6.2(7)	Information as to how EN 1993-1-10:2004 may be used in the seismic design situation.
6.5.5(7)	Reference to complementary rules on acceptable connection design.
6.7.4(2)	Residual post-buckling resistance of compression diagonals in steel frames with V-bracings.
7.1.2(1)	Upper limit of q for low-dissipative structural behaviour concept; limitations on structural behaviour concepts; geographical limitations on use of ductility classes for composite steel-concrete buildings.
7.1.3(1), (3)	Material partial factors for composite steel-concrete buildings in the seismic design situation.
7.1.3(4)	Overstrength factor for capacity design of composite steel-concrete buildings
7.7.2(4)	Stiffness reduction factor for the concrete part of a composite steel-concrete column section.
8.3(1)	Ductility classes for timber buildings.

Table NA.1 *(continued)*

9.2.1(1)	Types of masonry unit with sufficient robustness.
9.2.2(1)	Minimum strength of masonry units.
9.2.3(1)	Minimum strength of mortar in masonry buildings.
9.2.4(1)	Alternative classes for perpend joints in masonry.
9.3(2)	Conditions for use of unreinforced masonry satisfying the provisions of EN 1996 alone.
9.3(2)	Minimum effective thickness of unreinforced masonry walls satisfying the provisions of EN 1996 alone.
9.3(3)	Maximum value of ground acceleration for the use of unreinforced masonry satisfying the provisions of EN 1998-1.
9.3(4),Table 9.1	q-factor values in masonry buildings.
9.3(4), Table 9.1	q-factors for buildings with masonry systems that provide enhanced ductility.
9.5.1(5)	Geometric requirements for masonry shear walls.
9.6(3)	Material partial factors in masonry buildings in the seismic design situation.
9.7.2(1)	Maximum number of storeys and minimum area of shear walls of “simple masonry buildings”.
9.7.2(2)b	Minimum aspect ratio in plan of “simple masonry buildings”.
9.7.2(2)c	Maximum floor area of recesses in plan for “simple masonry buildings”.
9.7.2(5)	Maximum difference in mass and wall area between adjacent storeys of “simple masonry buildings”.
10.3(2)P	Magnification factor on seismic displacements for isolation devices.

In addition, NA.2.2 includes non-contradictory complementary information for the application of DIN EN 1998-1:2010-12. This information is preceded by the letters “NCI”.

Table NA.2

Reference	**Subject**
1.2.1	Normative references
3.1.1	General
3.1.3	Geological ground*) types; for defining the ground conditions (as a combination of ground and deep geology*)) that are characteristic for the spectrum
9.4(6)	Structural analysis
9.7.2(3)	Rules for “simple masonry buildings”
Annex NA.D	Simplified design rules for simple conventional buildings

*) Translator’s note: In this National Annex, the terms “deep geology” and “geological ground” are synonymous.

NA.2.2 National provisions

In the following, the clauses are numbered as in DIN EN 1998-1:2010-12. Supplementary clauses have been added as required.

NCI re 1.2.1 General reference standards

NA DIN EN 771-1:2005-05, *Specification for masonry units — Part 1: Clay masonry units*

NA DIN EN 771-2:2005-05, *Specification for masonry units — Part 2: Calcium silicate masonry units*

NA DIN EN 771-3:2005-05, *Specification for masonry units — Part 3: Aggregate concrete masonry units (dense and light-weight aggregates)*

NA DIN EN 771-4:2005-05, *Specification for masonry units — Part 4: Autoclaved aerated concrete masonry units*

NA DIN EN 1998-2, *Eurocode 8: Design of structures for earthquake resistance — Part 2: Bridges*

NA DIN V 105-100:2005-10, *Clay masonry units — Part 100: Clay masonry units with specific properties*

NA DIN V 106:2005-10, *Calcium silicate units with specific properties*

NA DIN V 4165-100:2005-10, *Autoclaved aerated concrete masonry units — Part 100: High precision units and elements with specific properties*

NA DIN V 18151-100:2005-10, *Lightweight concrete hollow blocks — Part 100: Hollow blocks with specific properties*

NA DIN V 18152-100:2005-10, *Lightweight concrete solid bricks and blocks — Part 100: Solid bricks and blocks with specific properties*

NA DIN V 18153-100:2005-10, *Concrete masonry units (normal-weight concrete) — Part 100: Masonry units with specific properties*

NA DIN V 18580, *Special masonry mortar*

NA DIN V 20000-401, *Application of building products in structures — Part 401: Rules for the application of clay masonry units according to DIN EN 771-1:2005-05*

NA DIN V 20000-402, *Application of building products in structures — Part 402: Rules for the application of calcium silicate masonry units according to DIN EN 771-2:2005-05*

NA DIN V 20000-403, *Application of building products in structures — Part 403: Rules for the application of aggregate concrete masonry units according to DIN EN 771-3:2005-05*

NA DIN V 20000-404, *Application of building products in structures — Part 404: Rules for the application of autoclaved aerated concrete masonry units according to DIN EN 771-4:2005-05*

NA DIN V 20000-412:2004-03, *Application of building products in structures — Part 412: Rules for the application of masonry mortar according to DIN EN 998-2:2003-09*

NDP re 1.1.2(7) Scope of EN 1998-1

Annexes A and B remain informative.

NDP re 2.1(1)P Fundamental requirements

The recommended values, $T_{\mathrm{NCR}} = 475$ years and $P_{\mathrm{NCR}} = 10$ %, apply. The verification of damage limitation (T_{DLR}) may be omitted.

NDP re 3.1.1(4) General

If there is insufficient information regarding the ground, ground type C as defined in 3.1.2(1) may be assumed without carrying out a separate ground investigation unless the special case described in 3.1.2(1) applies. If it does, the influence of local ground conditions on the seismic action shall be analysed and taken into account separately.

NCI re 3.1.1 General

(NA.5) The influence of local ground conditions on the seismic action shall generally be taken into account by determining which of the three geological ground types, R, T or S (see 3.1.3), and which of the three ground types, A, B or C (see 3.1.2), exist at the respective site.

(NA.6) The following combinations of ground type and geological ground type are possible: A-R, B-R, C-R, B-T, C-T or C-S.

NDP re 3.1.2(1) Identification of ground types

(i) In Germany, a distinction is made between the following ground types:

— **Ground type A**

Unweathered solid rock of high strength.

The dominant shear wave velocities exceed 800 m/s.

— **Ground type B**

Solid rock with average weathering and solid rock with low strength

or

coarse-grained (gravels) or composite soils with high friction properties, of dense compaction or solid consistency (e.g. glacially compacted soils).

The dominant shear-wave velocities are roughly within the range of 350 m/s to 800 m/s.

— **Ground type C**

Considerably eroded or completely eroded solid rock

or

coarse-grained (gravels) or mixed-grain soils of medium compaction or a relatively solid consistency

or

fine-grained (cohesive) soils of a relatively stiff consistency.

The dominant shear wave velocities are roughly within the range of 150 m/s to 350 m/s.

(ii) If it is not possible to classify the ground according to paragraph (i), particularly if deep-reaching uncompacted loose deposits (e.g. loose sand) or soil of soft or semi-fluid consistency (e.g. lacustrine clay, silt) are present (dominant shear wave velocities are lower than 150 m/s), then the influence of the ground conditions on seismic activity shall be analysed and taken into consideration separately.

(iii) The site of a structure should normally be free from any risk of slope instability or permanent settlement due to liquefaction in the event of an earthquake.

(iv) If there is any doubt, further investigations shall be carried out to determine the type of ground at the site. Further investigations are not necessary if unfavourable ground conditions as described in paragraph (ii) above can be ruled out and the seismic action is calculated under the assumption that the site has ground type C.

NCI NA 3.1.3 Geological ground types

(i) The deep geology (classified according to the following geological ground types) shall be taken into account in addition to the types of ground referred to above:

— **Geological ground type R**

Regions with rock-like geological strata (bedrock).

— **Geological ground type T**

Transition areas between regions with geological ground type R and geological ground type S as well as regions with relatively shallow sediment basins.

— **Geological ground type S**

Regions with deep basin structures filled with very thick sediment layers.

(ii) The geological ground types found in German seismic zones are shown in Figure NA.2. Seismic zones are defined in 3.2.1 (see Figure NA.1).

NOTE Administrative districts will be classified according to the geological ground types present in each area once DIN EN 1998-1 has been approved as an acknowledged technical rule for works under the building regulations of the German *Laender*.

NDP re 3.2.1(1), (2), (3) Seismic zones

(i) The seismic zones in Germany are shown in Figure NA.1. Non-tectonic seismic events, e.g. in mining regions or areas susceptible to sinkholes, are not covered by this standard.

NOTE Administrative districts will be classified according to the seismic zone in which they are located once DIN EN 1998-1 has been approved as an acknowledged technical rule for works under the building regulations of the German *Laender*.

Figure NA.1 — Seismic zones in Germany

(ii) A reference peak ground acceleration a_{gR} is assigned to each seismic zone, as shown in Table NA.3.

Table NA.3 — Intensity ranges and reference peak ground accelerations in the various seismic zones

Seismic zone	Intensity range	Reference peak ground acceleration a_{gR} m/s^2
0	$6 \leq I < 6{,}5$	—
1	$6{,}5 \leq I < 7$	0,4
2	$7 \leq I < 7{,}5$	0,6
3	$7{,}5 \leq I$	0,8

(ii) Figure NA.2 shows the geological ground types found in German seismic zones (see Figure NA.1).

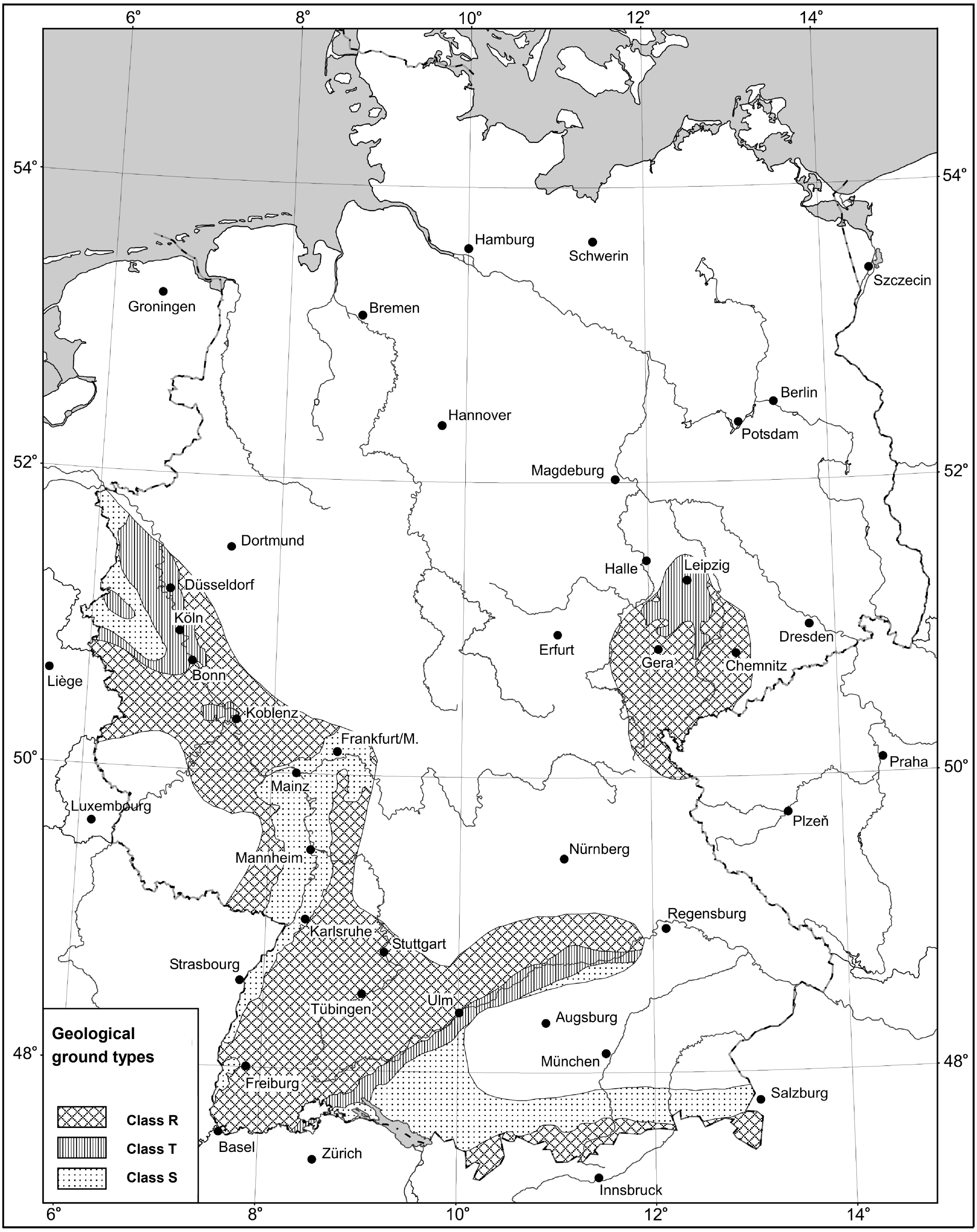

Figure NA.2 — Geological ground types (deep geology) in seismic zones in Germany

NDP re 3.2.1(4) Seismic zones

As the seismic zones 1 to 3 shown in Figure NA.1 are considered as areas of low seismicity in this standard, the simplified method described in Annex A may be used for conventional buildings*) of importance class **I**, **II** or **III** with no more than 6 storeys and with a maximum height of 20 m above the average measured ground level.

NDP re 3.2.1(5) Seismic zones

Areas not included in seismic zones 1 to 3 as shown in Figure NA.1 shall be classified as areas of very low seismicity as defined in this standard.

NDP re 3.2.2.1(4), 3.2.2.2(1)P Basic representation of seismic action

The following parameters and relationships apply to describe the horizontal elastic response spectra:

(1) The horizontal elastic response spectrum $S_e(T)$ for the reference return period (see Figure NA.3) is defined by the following expressions:

$$T_A \le T \le T_B: \quad S_e(T) = a_{gR} \cdot \gamma_I \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2{,}5 - 1)\right] \tag{NA.1}$$

$$T_B \le T \le T_C: \quad S_e(T) = a_{gR} \cdot \gamma_I \cdot S \cdot \eta \cdot 2{,}5 \tag{NA.2}$$

$$T_C \le T \le T_D: \quad S_e(T) = a_{gR} \cdot \gamma_I \cdot S \cdot \eta \cdot 2{,}5 \cdot \frac{T_C}{T} \tag{NA.3}$$

$$T_D \le T: \quad S_e(T) = a_{gR} \cdot \gamma_I \cdot S \cdot \eta \cdot 2{,}5 \cdot \frac{T_C T_D}{T^2} \tag{NA.4}$$

where

$S_e(T)$	is the ordinate of the elastic response spectrum;
T	is the vibration period of a linear single-degree-of-freedom system;
a_{gR}	is the reference peak ground acceleration according to Table NA.3;
γ_I	is the importance factor according to Table NA.6;
T_A, T_B, T_C, T_D	are the control periods of the response spectrum, with T_A = 0; in order to model the frequency range, a frequency of 25 Hz can be used instead of the period of 0 seconds, and S_e is assumed to be constant for the higher frequencies;
S	is the soil factor;
η	is the damping correction factor with a reference value of $\eta = 1$ for 5 % viscous damping, see (3) of this subclause.

*) Translator's note: In this National Annex, the term "conventional buildings" is used to describe buildings designed to sustain predominantly static, uniformly distributed imposed loads up to 5,0 kN/m^2 and, in some cases, concentrated loads up to 7,0 kN and loads from passenger cars (see definition in DIN 1045-1:2008-08).

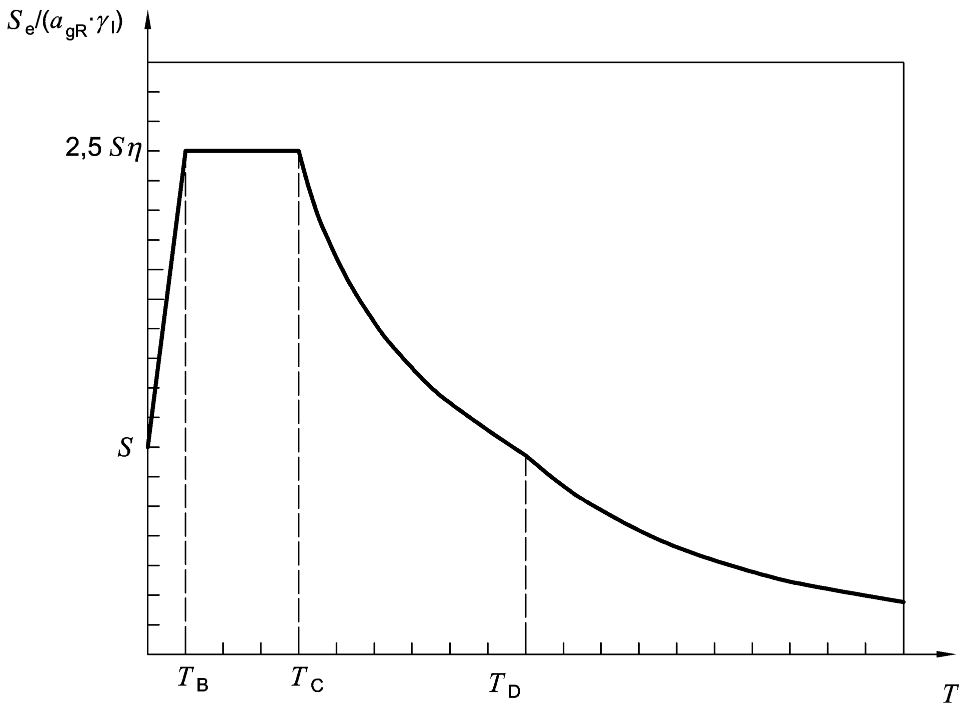

Figure NA.3 — Elastic response spectrum

(2) For horizontal ground motion, the influence of the ground conditions on the elastic response spectrum is taken into account by selecting the relevant parameters from Table NA.4.

Table NA.4 — Values of the parameters describing the horizontal elastic response spectrum

Ground conditions	S	T_B s	T_C s	T_D s
A-R	1,00	0,05	0,20	2,0
B-R	1,25	0,05	0,25	2,0
C-R	1,50	0,05	0,30	2,0
B-T	1,00	0,1	0,30	2,0
C-T	1,25	0,1	0,40	2,0
C-S	0,75	0,1	0,50	2,0

(3) The value of the damping correction factor η can be determined using the following expression.

$$\eta = \sqrt{\frac{10}{5 + \xi}} \geq 0{,}55 \qquad \text{(NA.5)}$$

where

ξ is the viscous damping ratio of the structure, expressed as a percentage.

If, for special investigations, viscous damping values other than 5 % are to be used, the reasons for assuming a different value shall be stated.

NDP re 3.2.2.3(1)P Vertical elastic response spectrum

The following parameters and relationships shall be used to define the vertical elastic response spectra.

The vertical elastic response spectrum $S_{ve}(T)$ for the reference return period is defined by the following expressions:

$$T_A \leq T \leq T_B: \quad S_{ve}(T) = a_{vg} \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 3{,}0 - 1)\right] \qquad \text{(NA.6)}$$

$$T_B \leq T \leq T_C: \quad S_{ve}(T) = a_{vg} \cdot \eta \cdot 3{,}0 \qquad \text{(NA.7)}$$

$$T_C \leq T \leq T_D: \quad S_{ve}(T) = a_{vg} \cdot \eta \cdot 3{,}0 \cdot \frac{T_C}{T} \qquad \text{(NA.8)}$$

$$T_D \leq T: \quad S_{ve}(T) = a_{vg} \cdot \eta \cdot 3{,}0 \cdot \frac{T_C T_D}{T^2} \qquad \text{(NA.9)}$$

The control period T_B shall be 0,05 s for all ground types.

The control periods T_C and T_D shall be 0,2 s and 2,0 s respectively.

Furthermore, $a_{vg} = 0{,}5 \cdot a_{gR} \cdot \gamma_I$ applies.

NDP re 3.2.2.5(4)P Design spectrum for elastic analysis

(NA.4.1) All buildings have a more or less pronounced ability to dissipate, by hysteretic processes, the energy imparted to them by seismic action. This, in a simplified form, allows them to be designed to withstand forces lower than they would have to withstand if only linear-elastic reactions without energy dissipation were to be assumed.

(NA.4.2) To enable favourable dissipative effects to be taken into consideration in linear calculations as well, the elastic response spectrum is reduced by introducing the behaviour factor q which is a specific characteristic of the structure and building type.

(NA.4.3) For the horizontal component of the seismic action, the design spectrum $S_d(T)$ is defined by the following expressions.

$$T_A \leq T \leq T_B: \quad S_d(T) = a_{gR} \cdot \gamma_I \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot \left(\frac{2{,}5}{q} - 1\right)\right] \qquad \text{(NA.10)}$$

$$T_B \leq T \leq T_C: \quad S_d(T) = a_{gR} \cdot \gamma_I \cdot S \cdot \frac{2{,}5}{q} \qquad \text{(NA.11)}$$

$$T_C \leq T \leq T_D: \quad S_d(T) = a_{gR} \cdot \gamma_I \cdot S \cdot \frac{2{,}5}{q} \cdot \frac{T_C}{T} \qquad \text{(NA.12)}$$

$$T_D \leq T: \quad S_d(T) = a_{gR} \cdot \gamma_I \cdot S \cdot \frac{2{,}5}{q} \cdot \frac{T_C T_D}{T^2} \qquad \text{(NA.13)}$$

where

$S_d(T)$ is ordinate of the design spectrum;

q is the behaviour factor.

NOTE The β factor according to EN 1998-1:2004 is assumed to be zero and is therefore not included in expressions NA.10 to NA.13.

(NA.4.4) The values of the parameters T_B, T_C, T_D and S are given in Table NA.4. To determine the design spectrum, T_B should be taken as 0,01 s.

(NA.4.5) The design spectrum described in paragraph (3) above is not suitable for calculating the dimensions of structures with foundation isolation or other energy dissipation systems.

NDP re 4.2.3.2(8) Criteria for regularity in plan

The specifications of DIN EN 1998-1:2010-12 apply.

NDP re 4.2.4(2)P Combination coefficients for variable actions

The values of φ for calculating ψ_{Ei} shall be taken from Table NA.5. To determine the effective mass when calculating seismic loads, snow loads shall be multiplied by a combination coefficient ψ_2 of 0,5 in expression (4.2). The reduced snow loads shall also be taken into account in stability verifications.

Note — If there is a possibility that usage of the building will result in considerable long-term asymmetrical loads (e.g. in the case of warehouses or storage buildings that are only used part of the time), then the torsional effects resulting from seismic action shall be included in the verification analyses.

Table NA.5 — Values of φ for calculating ψ_{Ei}

Type of variable action as defined in DIN EN 1991-1-1/NA	**Storey**	φ
Imposed loads of categories A – C, including imposed loads of categories T and Z	uppermost storey	**1,0**
	all other storeys	**0,7**
Imposed loads of categories D – F, including imposed loads of categories T and Z	all storeys	**1,0**

NDP re 4.2.5(5)P Importance classes and importance factors

Importance classes and importance factors shall be taken from Table NA.6.

Table NA.6 — Importance classes and importance factors

Importance class	Building	Importance factor γ_I
I	Buildings of negligible importance for public safety, with little pedestrian traffic (e.g. barns, greenhouses, etc.).	0,8
II	Buildings not belonging to the other categories (e.g. small dwellings or office buildings, workshops, etc.).	1,0
III	Buildings whose collapse as the result of an earthquake would affect a large number of people (e.g. large residential complexes, schools, assembly halls, department stores, etc.).	1,2
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection (e.g. hospitals, major facilities of disaster control authorities, fire stations, security forces, etc.).	1,4

NDP re 4.3.3.1(4) Methods of analysis

As an alternative to the “lateral force method of analysis” in **4.3.3.1(3) a)** and the “modal response spectrum analysis” in **4.3.3.1(3) b),** the non-linear static (pushover) analysis in **4.3.3.1(4) c)** or, in special cases, the non-linear time history (dynamic) analysis in **4.3.3.1(4) d)** may be used where there is sufficient experience.

NDP re 4.3.3.1(8) Methods of analysis

This clause applies to all importance classes.

NDP re 4.4.2.5(2) Resistance of horizontal diaphragms

The recommended values for the overstrength factor given in DIN EN 1998-1:2010-12 apply. They are:

γ_d = 1,3 for brittle failure modes, such as in shear in concrete diaphragms, and

γ_d = 1,1 for ductile failure modes.

NDP re 4.4.3.2(2) Limitation of interstorey drift

It shall be ensured that both the non-structural elements and their connections and means of fixation or anchorage will withstand the combination of permanent, variable and seismic actions. Particular attention shall be paid to the results of suppressed deformation (e.g. in brittle parts of façades).

NDP re 5.2.1(5) Energy dissipation capacity and ductility classes

Ductility classes DCL and DCM are recommended within the scope of this standard.

NDP re 5.2.2.2(10) Behaviour factors for horizontal seismic actions

Increasing the values of q_o to comply with a special quality system plan is not permitted.

NDP re 5.2.4(1) and (3) Safety verifications

The material partial factors for persistent and transient design situations shall be used for the seismic design situation. Values shall be taken from DIN EN 1992-1-1 and its National Annex.

NDP re 5.4.3.5.2(1) Shear resistance

The value recommended in the Note to 5.4.3.5.2(1) of DIN EN 1998-1:2010-12 applies. DIN EN 1992-1-1:2011-01, 9.6.2, applies to the transverse reinforcement of walls.

NDP re 5.8.2(3) Tie-beams and foundation beams

The values recommended in the Note to 5.8.2(3) of DIN EN 1998-1:2010-12 apply. These are $b_{\mathrm{w,min}} = 0{,}25$ m and $h_{\mathrm{w,min}} = 0{,}4$ m for buildings with up to three storeys or $h_{\mathrm{w,min}} = 0{,}5$ m for buildings with four or more storeys.

NDP re 5.8.2(4) Tie-beams and foundation beams

The values recommended in the Note to 5.8.2(4) of DIN EN 1998-1:2010-12 apply. These are $t_{\mathrm{min}} = 0{,}2$ m and $\rho_{\mathrm{s,min}} = 0{,}2$ %.

NDP re 5.8.2(5) Tie-beams and foundation beams

The value recommended in the Note to 5.8.2(5) of DIN EN 1998-1:2010-12 applies, i.e. $\rho_{\mathrm{b,min}} = 0{,}4$ %.

NDP re 5.11.1.3.2(3) Energy dissipation

For precast wall panel systems, the ductility class shall be DCL.

NDP re 5.11.1.4 Behaviour factors

The recommended values apply, i.e. $k = 1{,}0$ for structures with connections as described in 5.11.2.1.1 to 5.11.2.1.3 and $k = 0{,}5$ for structures with other types of connections.

NDP re 5.11.1.5(2) Analysis of transient situation

If seismic actions need to be taken into consideration during the erection of structures as described in 5.11.1.5 (1) of DIN EN 1998-1:2010-12, then the value of A_{p} shall be selected on a case-by-case basis.

NDP re 5.11.3.4(7) Precast large panel walls

The value recommended in the Note to 5.11.3.4(7) of DIN EN 1998-1:2010-12 applies, i.e. $\rho_{\mathrm{c,min}} = 1$ %.

NDP re 6.1.2(1) Design concepts

It is recommended that the maximum value of the behaviour factor q given in Table 6.2 be limited to $q \leq 4$ for seismic areas in Germany. It is also recommended that ductility class DCL or DCM be used.

NDP re 6.1.3(1) Safety verifications

For partial factor γ_{S}, $\gamma_{\mathrm{S}} = \gamma_{\mathrm{M}}$ applies. The partial factors γ_{M} shall be taken from DIN EN 1993-1-1.

NDP re 6.2(3) Materials

The recommended value for the overstrength factor applies, i.e. γ_{ov} = 1,25.

NOTE The maximum value of the yield strength used to determine the overstrength factor shall be ensured by means of compliance with the relevant product standards.

NDP re 6.2(7) Materials

In structures of ductility class DCM or DCH, exposed dissipative elements shall have a minimum toughness T_{27J} of –20 °C.

NDP re 6.5.5(7) Design rules for connections in dissipative zones

There are no complementary national provisions for the design of connections.

NOTE — In some cases, the building regulations of the German *Laender* stipulate that experimental evidence may only be used in conjunction with a general building inspectorate approval.

NDP re 6.7.4(2) Beams and columns

For the estimation of the post buckling resistance of diagonals in compression the value of 0,3 recommended for factor γ_{PB} applies.

NDP re 7.1.2(1) Design concepts

Within the scope of this standard, it is recommended that the upper limits of the behaviour factor q stated in Table 6.2 be limited to $q \leq 4$ for seismic areas in Germany.

NDP re 7.1.3(1) and (3) Safety verifications

The information relating to 5.2.4(1), 5.2.4(3) and 6.1.3(1) given in this document applies.

NDP re 7.1.3(4) Safety verifications

The information relating to 6.2(3) given in this document applies.

NDP re 7.7.2(4) Analysis

The recommended value $r = 0{,}5$ applies.

NDP re 8.3(1) Ductility classes and behaviour factors

(1) With respect to the required hysteretic energy dissipation capacity, the three ductility classes DCL (low capacity to dissipate energy), DCM (medium capacity to dissipate energy) and DCH (high capacity to dissipate energy) shall be as follows:

a) Ductility class DCL

Ductility class DCL applies to structures which are supposed to remain within the elastic limits when subjected to design seismic conditions and to which no specific ductility requirements apply. All timber buildings may be classed as belonging to this ductility class. All structures that do not have joints that will yield and in which dowel-type fasteners are used shall be assigned to this class, e.g.

— arches with three pinned joints;

— frames with three pinned joints and full dovetail joints at the frame corners;

— column-beam structures with restrained column bases.

b) Ductility class DCM

Ductility class DCM applies to structures which are not designed for ductility class DCL in the elastic state and in which the ductility requirements are limited to a few effective dissipative sections with dowel-type fasteners. These structures include:

— column-beam structures with semi-rigidly constrained (by dowelled joints) column bases;

— frames with two or three pinned joints and dowelled joints at the frame corners;

— timber panel structures with rigid (glued) diaphragms in which the individual panels are interconnected by means of mechanical fasteners.

c) Ductility class DCH

Ductility class DCH applies to structures which are not designed for ductility class DCL in the elastic state and which have a large number of dissipative sections with metallic dowel-type fasteners. These structures include:

— hyperstatic structures with metallic dowel-type fasteners where the energy is dissipated solely through the fasteners;

— wall panels with metallic dowel-type fasteners where failure due to unintentional withdrawal of the fasteners can be ruled out;

— skeleton structures and beam and column constructions in which the mechanical fasteners at the nodes are designed as horizontal bracing on the basis of plastic limit analysis;

— wall panels with metallic dowel-type fasteners where the anchorage and the connection between the panel material and timber supporting structure by means of fasteners have been shown to be particularly ductile.

The dowel-type fasteners referred to in Table NA.7 are defined in DIN EN 1995-1-1.

Table NA.7 — Design concepts, structural types and upper limit values of the behaviour factors for the three ductility classes

Ductility class and design concept	q	Examples of structures
Ductility class DCL Low capacity to dissipate energy	**1,5**	Arches with two or three pinned joints without specific dissipation mechanisms, cantilevers, trusses with rigid joints
Ductility class DCM Medium capacity to dissipate energy	**2,5**	Hyperstatic structures with flexible dowel-type fasteners, wood panels with rigid (e.g. glued) diaphragms.
Ductility class DCH High capacity to dissipate energy	**4,0**	Hyperstatic structures with metallic dowel-type fasteners where the energy is dissipated solely through the fasteners, wall panels with metallic dowel-type fasteners where failure due to unintentional withdrawal of the fasteners can be ruled out.[a]
	5,0	Wall panels with metallic dowel-type fasteners where the anchorage and the connection between the panel material and timber supporting structure by means of fasteners have been shown to be particularly ductile.

[a] For ductility class DCH, the minimum penetration depth as specified in DIN 1052 for staples and smooth shafted nails shall be increased by a factor of 1,25.

Ductility classes DCL and DCM are recommended. For combinations of structures of ductility classes DCL and DCH in seismic zones 2 and 3, the behaviour factor q shall not be higher than 1,5.

NOTE The building regulations of the German *Laender* stipulate that all experimental verifications shall be carried out in conjunction with a general building inspectorate approval.

NDP re 9.2.1(1) Types of masonry units

All masonry units defined in DIN EN 771-1, DIN EN 771-2, DIN EN 771-3 and DIN EN 771-4 may be used in seismic zones 1 to 3 as long as they are deemed suitable in accordance with the application standards DIN V 105-6, DIN V 20000-401, -402, -403 or -404 or have been granted a general building inspectorate approval or meet the following additional requirements for

— clay masonry units in accordance with DIN V 105-100;

— calcium silicate units in accordance with DIN V 106;

— concrete units in accordance with DIN V 18151-100, DIN V 18152-100 or DIN V 18153-100;

— autoclaved aerated concrete units in accordance with DIN V 4165-100.

NDP re 9.2.2(1) Minimum strength of masonry units

There are no further national requirements for the compressive strength of the masonry units permitted under 9.2.1(1) when the units are used in German seismic regions.

NDP re 9.2.3(1) Mortar

All mortars as specified in DIN EN 998-2 may be used in conjunction with DIN V 20000-412:2004-03 or DIN V 18580 in seismic zones 1 to 3.

NDP re 9.2.4(1) Masonry bond

All classes of perpend joints may be used in seismic zones 1 to 3.

NDP re 9.3(2) Types of construction and behaviour factors

The minimum thickness of unreinforced structural masonry walls complying with the provisions of DIN EN 1996 only shall be 115 mm.

NDP re 9.3(3) Types of construction and behaviour factors

Unreinforced masonry may be used in all German seismic zones.

NDP re 9.3(4) Types of construction and behaviour factors

Table NA.8 — Types of construction and upper limit of the behaviour factor

Type of construction	Behaviour factor q
Unreinforced masonry as specified in DIN EN 1996-1	1,5
Unreinforced masonry as specified in DIN EN 1998-1	Table NA.9
Confined masonry	Table NA.9
Reinforced masonry	3,0

Table NA.9 — Behaviour factor q for unreinforced and confined masonry in accordance with DIN EN 1998-1

Masonry type	Wall geometry	
	h/l[a] ≤ 1	$h/l \geq 1,6$
Unreinforced[b, c]	1,5	2,0
Confined	2,0	2,5

[a] h/l is the ratio of the clear storey height to the length of the longest wall in the building axis direction under consideration.

[b] The use of behaviour factors q greater than 1,5 is only permitted when, for the seismic design situation, the mean normal stress in the relevant walls does not exceed 15 **%** of the characteristic compressive strength of the masonry f_k as specified in DIN EN 1996-1-1.

[c] Structural modelling may be carried out in accordance with DIN EN 1996-1-1.

Intermediate values may be obtained by linear interpolation.

NCI re 9.4(6) Structural analysis

As an alternative, the base shear in individual walls as obtained by linear analysis in accordance with Section 4 may be redistributed among the walls provided that

d) the redistribution is carried out on the basis of elastic-ideal plastic load displacement curves for each wall while ensuring that the global equilibrium is satisfied. Structural modelling may be carried out in accordance with DIN EN 1996-1-1.

For all types of mortar and masonry unit commonly used for unreinforced masonry in Germany, the maximum deformation between the top and bottom of the wall may be as follows:

— flexural failure: 0,006 $\cdot H_0/L \cdot H$;

— shear failure: 0,004 $\cdot H$ if, for the seismic design situation, the mean normal stress does not exceed 15 **%** of the characteristic compressive strength of the masonry f_k as specified in DIN EN 1996-1-1 or a general building inspectorate approval. In all other cases, the deformation shall not exceed 0,003 $\cdot H$ for shear failure;

where H is the storey height, L is the wall length and H_0 is the distance between the cross-section at which flexural capacity is reached and the inflection point, all expressed in m.

For confined masonry, the deformation values given for unreinforced masonry may be increased by a factor of 2.

NDP re 9.5.1(5) Design criteria and construction rules

Table NA.10 — Minimum requirements for stiffening walls (shear walls)

Seismic zone	h_{ef}/t_{ef}	t_{ef} mm	l/h
1	as in DIN EN 1996-1-1		≥ 0,27
2	≤ 18	≥ 150[a]	≥ 0,27
3	≤ 15	≥ 175	≥ 0,27

h_{ef} Effective length as in DIN EN 1996-1-1
t_{ef} Wall thickness
l Wall length

[a] Walls of thicknesses ≥ 115 mm may be considered separately if $h_{ef}/t_{ef} \leq 15$.

Gable walls shall be sufficiently stiffened by means of cross-walls or pilasters if they are not connected to the roof structure by an interference fit.

NDP re 9.6(3) Safety verifications

Table NA.11 — Partial factors for masonry buildings for the seismic design situation

Material	Partial factor
Masonry γ_m	**1,2**
Reinforcing steel γ_s	**1,0**

NDP re 9.7.2(1) Rules for “simple masonry buildings”

Table NA.12 — Minimum requirements for the cross-sectional area of shear walls in relation to the storey floor plan area [c), d)]

Number of full storeys	$a_{gR} \cdot S \cdot \gamma_I \leq 0{,}6 \cdot k^a \cdot k_r^e$			$a_{gR} \cdot S \cdot \gamma_I \leq 0{,}9 \cdot k^a \cdot k_r^e$			$a_{gR} \cdot S \cdot \gamma_I \leq 1{,}2 \cdot k^a \cdot k_r^e$		
	Masonry unit strength class in accordance with DIN 1053-1[b]								
	4	6	≥ 12	4	6	≥ 12	4	6	≥ 12
1	0,02	0,02	0,02	0,03	0,025	0,02	0,04	0,03	0,02
2	0,035	0,03	0,02	0,055	0,045	0,03	0,08	0,05	0,04
3	0,065	0,04	0,03	0,08	0,065	0,05	Simplified verification procedure not permitted (Spnp)		
4	Spnp	0,05	0,04	Spnp					

[a] For buildings in which 70 % of the shear walls under consideration are longer than 2 m in one axis direction, the factor k is given by $k = 1 + (l_a - 2)/4 \leq 2$ where l_a is the average length, expressed in m, of the shear walls considered. For all other cases, $k = 1$. The value of γ_I is determined in accordance with Table NA.6.

[b] If units of different strength classes are used, e.g. for internal and for external walls, the required values shall be weighted in relation to the stiffness components of the respective strength classes.

[c] Intermediate values may be obtained by linear interpolation.

[d] Masonry unit strength class 2 may be used for external walls if at least 50 % of the required cross-sectional area of the wall in each axis direction of the shear walls consists of masonry of strength class 4 or higher. The total cross-sectional area of the shear walls shall then satisfy the values specified for masonry unit strength class 4 in Table NA.12.

[e] For terraced houses with the dimensions $B \leq 7$ m and $L \leq 12$ m and with at least two parallel walls in two orthogonal directions, where the length of each wall is to be at least 40 % of the length of the structure in the axis direction considered, k_r can be taken as 1,25. In all other cases, $k_r = 1{,}0$.

NDP re 9.7.2(2)b Rules for “simple masonry buildings”

The minimum aspect ratio λ_{min} shall be not less than 0,25.

NDP re 9.7.2(2)c Rules for “simple masonry buildings”

The recommended value $p_{max} = 15$ % is applicable.

NCI re 9.7.2(3) Rules for “simple masonry buildings”

Alternatively, the condition in 9.7.2 (3)d is also deemed to be satisfied when the majority of the vertical loads are supported by the shear walls in both main orthogonal directions.

NDP re 9.7.2(5) Rules for “simple masonry buildings”

The greatest permissible difference between the mass and the wall area of storeys that are directly above each other in the case of “simple masonry buildings” is as follows:

The highest permissible values are $\Delta_{m,max} = 20$ % and $\Delta_{A,max} = 30$ %.

NDP re 10.3(2)P Fundamental requirements

The magnification factor on seismic displacements for the design of isolation devices is as follows:

The recommended value $\gamma_x = 1{,}2$ is applicable.

NCI

Annex NA.D
(informative)

Simplified design rules for simple conventional buildings

NCI NA.D.1 General

(1) The following conditions shall be satisfied to enable the simplified design rules to be applied:

a) The site of the structure and the type of supporting ground shall be free from any risk of slope instability and permanent settlement due to liquefaction or compaction in the event of an earthquake.

b) The ground shall not consist of deep, uncompacted loose deposits (e.g. loose sand) or soil of soft or semi-fluid consistency (e.g. lacustrine clay, silt) (the dominant shear-wave propagation velocities being lower than 150 m/s).

(2) The simplified calculation method described in this annex may be used for conventional buildings of importance classes I to III with a maximum of six storeys and a maximum height of 20 m above the average measured ground level provided the following conditions are satisfied:

a) As regards the lateral stiffness and the distribution of masses, the building plan should be almost completely symmetrical along both main axes. If distribution is not symmetrical the structure shall be capable of resisting the resulting effects (e.g. due to torsion) described in Section NA.D.4.

b) The building plan shall be compact, i.e. not in the shape of an H, X, L, T or U, for example. Such shapes are only permitted if the various sections of the building are seismically isolated from each other by suitable joints (see NA.D.8 d). In this case, each section of the building shall be considered separately.

c) The floors shall act as quasi-rigid diaphragms and shall be capable of transmitting the horizontal forces to the stiffening elements.

d) All systems designed to absorb horizontal loads, such as cores, loadbearing walls or frames, in multi-storey buildings shall be continuous from their foundations right through to the top loadbearing floor of the building. If this is not the case, the horizontal and vertical transmission of loads shall be ensured.

e) The lateral stiffness, the actual horizontal loadbearing capacity and the mass of the individual storeys shall remain constant or be reduced only gradually from one storey to the next without abrupt changes, decreasing upwards (with the exception of the transition to basement storeys).

(3) For buildings satisfying the criteria referred to in paragraphs (1) and (2) calculations may be performed using two planar models, one for each main horizontal axis. The stability of the building shall be verified for each direction, taking NA.D.5 and the possible torsional effects dealt with in NA.D.4 into account.

NCI NA.D.2 Base shear force

(1) The base shear force F_b for each main direction is determined as follows:

$$F_b = S_d\,(T_1) \cdot M \cdot \lambda \qquad \text{(NA.D.1)}$$

where

$S_d(T_1)$ is the ordinate of the design spectrum (see NA.3.2.2.5) with the fundamental vibration period T_1;

T_1 is the fundamental vibration period of the building for translation displacement in the direction being analysed;

M is the total mass of the building, calculated as described in (2);

λ is a correction factor with a value of $\lambda = 0{,}85$ for $T_1 \leq 2 \cdot T_C$ for buildings with more than 2 storeys and with a value of $\lambda = 1{,}0$ in all other cases.

(2) The total mass M of the building is determined taking account of all permanent actions and 30 % of the imposed loads (80 % for warehouses, libraries, department stores, multi-storey car parks, workshops and factories). The calculations shall include 50 % of the snow loads.

(3) The value of T_1 [s] may be determined approximately by means of the following expression:

$$T_1 = 2 \cdot \sqrt{u} \qquad \text{(NA.D.2)}$$

where u [m] is the notional degree of sway at the top of the building under the assumed permanent and quasi-permanent horizontal loads determined from the masses calculated according to paragraph (2).

(4) The behaviour factor q assumed for all types of structure shall not exceed 1,5.

(5) As an alternative to paragraph (1), a conservative value of the base shear force F_b for each main direction can be determined as follows, using a behaviour factor $q = 1{,}5$:

$$F_b = S_{dmax} \cdot M \qquad \text{(NA.D.3)}$$

(6) Anchorages of timber panel elements shall be designed to withstand forces due to the full S_{dmax}-values shown in Table NA.D.1.

Table NA.D.1 — Maximum spectral acceleration S_{dmax} [m/s^2]

	SA I	SA II	SA III
Seismic zone 1	0,8	1	1,2
Seismic zone 2	1,2	1,5	1,8
Seismic zone 3	1,6	2	2,4

NCI NA.D.3 Distribution of the horizontal seismic forces

(1) The action-effects due to seismic action shall be determined by applying horizontal forces F_i acting on the masses m_i of all storeys in the two planar models.

(2) The forces F_i can be distributed linearly over the height of the structure (Figure NA.D.1):

$$F_i = F_b \cdot \frac{z_i \cdot m_i}{\sum_{j=1}^{n} z_j \cdot m_j} \qquad \text{(NA.D.4)}$$

where

m_i, m_j are the storey masses, calculated in accordance with NA.D.2 (2).

z_i, z_j is the height of the masses m_i, m_j above the level of application of the seismic action (foundation or top of a rigid basement);

F_i is the horizontal force acting on storey *i*;

F_b is the base shear force calculated by means of expression (NA.D.1) or expression (NA.D.3).

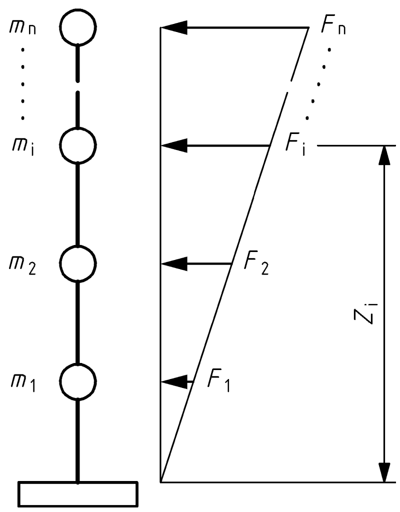

Figure NA.D.1 — Distribution of seismic forces in proportion to the height of a building

(3) The horizontal forces F_i determined in accordance with this subclause shall be distributed over the structural system that absorbs the horizontal loads, assuming that the floors are rigid.

NA.D.4 Torsional effects

(1) If the lateral stiffness and mass are distributed almost completely symmetrically in both main directions of the plan of the building, the effect of torsion will be covered by increasing the seismic action effects by 15 %.

Note — If there is a possibility that usage of the building will result in considerable long-term asymmetrical loads (e.g. in the case of warehouses or storage buildings that are only used part of the time), then the torsional effects resulting from seismic action shall be included in the verification analyses.

(2) To enable the torsional effect to be taken into account, the eccentricities of the resultant equivalent forces of the upper storeys in the main directions shall be taken as follows (Figure NA.D.2):

$$e_{\text{max,i}} = e_{0,\text{i}} + e_{1,\text{i}} + e_{2,\text{i}}$$
$$e_{\text{min,i}} = 0{,}5 \cdot e_{0,\text{i}} - e_{1,\text{i}} \qquad \text{(NA.D.5)}$$

where

i are the main directions i = x, y;

$e_{0,\text{i}}$ is the actual eccentricity;

$e_{1,\text{i}}$ is the accidental eccentricity, $e_{1\text{i}} = 0{,}05 \cdot L_\text{i}$, L_i: dimensions of the structure perpendicular to the direction of the seismic action;

$e_{2,\text{i}}$ is the additional eccentricity.

The additional eccentricity $e_{2,\text{i}}$ is calculated as follows:

$$e_{2,\text{i}} = 0{,}1 \cdot \left(L_\text{x} + L_\text{y}\right) \cdot \sqrt{\frac{10 \cdot e_{0,\text{i}}}{L_\text{i}}} \leq 0{,}1 \cdot \left(L_\text{x} + L_\text{y}\right) \qquad \text{(NA.D.6)}$$

(3) If adequate stiffening is provided against torsional effects, the additional eccentricity $e_{2,\text{i}}$ can also be determined using expression (NA.D.7):

$$e_{2,\text{i}} = \frac{1}{2e_{0,\text{i}}} \left[l_\text{s}^2 - e_{0,\text{i}}^2 - r_\text{i}^2 + \sqrt{\left(l_\text{s}^2 + e_{0,\text{i}}^2 - r_\text{i}^2\right)^2 + 4 \cdot e_{0,\text{i}}^2 \cdot r_\text{i}^2} \right] \qquad \text{(NA.D.7)}$$

The torsional radius is calculated with the moments of inertia of the stiffening elements, I_J and I_k, in the main direction i under consideration and at right angles to the main direction respectively as well as with their respective distances from the centres of stiffness, r_j and r_k, as follows:

$$r_\text{i} = \sqrt{\frac{\left(\sum_j I_\text{j} r_\text{j}^2 + \sum_k I_\text{k} r_\text{k}^2\right)}{\sum_j I_\text{j}}}$$

The radius of gyration of a rectangular area (Figure NA.D.2) can be determined as follows:

$$l_\text{s}^2 = \frac{L_\text{x}^2 + L_\text{y}^2}{12}$$

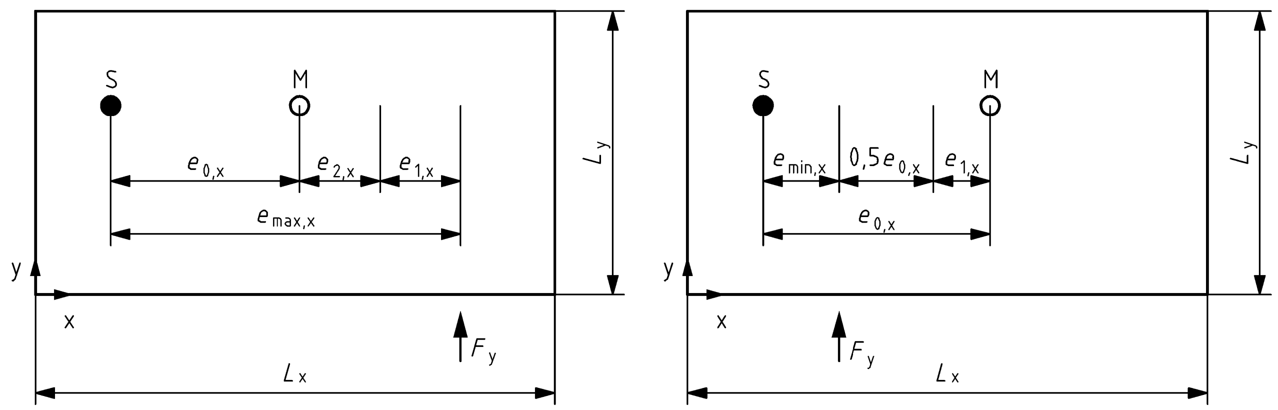

Figure NA.D.2 — Eccentricities to be assumed for a seismic force acting in the y-direction

NCI NA.D.5 Combination of the effects of the components of the seismic action

(1) The seismic action may be assumed to act separately along the two main orthogonal horizontal axes of the building. If torsional effects have to be taken into account as specified in NA.D.4 (2), 30 % of the action effects due to the seismic action (including those due to torsional effects) acting in one direction shall also be taken into account in the other direction.

(2) The vertical component of the seismic action may generally be neglected except in the case of beams and floors supporting columns or stiffening or loadbearing walls.

NCI NA.D.6 Calculation of deformations

(1) Deformations due to seismic action can be determined, by approximation, on the basis of the elastic deformation of the structural system by applying the following expression:

$$d_{\mathrm{s}} = q \cdot d_{\mathrm{e}} \tag{NA.D.8}$$

where

d_{s} is the deformation of the structural system due to seismic action;

q is the behaviour factor;

d_{e} is the deformation of the same point of the structural system, determined by elastic analysis on the basis of the design spectrum.

NCI NA.D.7 Non-structural elements

(1) It shall be verified that secondary, non-structural elements of buildings (e.g. glazing constructions, brittle wall claddings, parapets, gables, aerials, mechanical appendages and equipment, non-loadbearing external walls, non-loadbearing internal partitions with heights greater than 3,50 m, railings, chimneys) that might, in case of failure, constitute a risk to persons or affect the loadbearing structure of the building, are able, together with their supports, to withstand the design seismic action.

(2) The ability to withstand seismic load situations need not be verified for non-structural internal partitions with heights of less than 3,50 m or the non-loadbearing outer leaves of double-leaf masonry.

(3) It shall be ensured that both the non-structural elements and their connections and means of fixation or anchorage will withstand the combination of persistent, variable and seismic action. Special attention shall be paid to the results of suppressed deformation (e.g. in brittle parts of façades).

(4) The action-effects due to seismic action may be approximated by assuming that a horizontal force F_a as calculated by the following expression acts on the non-structural elements in the most unfavourable direction:

$$F_a = 4 \cdot S \cdot a_{gR} \cdot m_a \cdot \gamma_a \tag{NA.D.9}$$

where

F_a is the horizontal seismic force acting in the most unfavourable direction in the centre of mass of the non-structural element;

m_a is the mass of the element;

S is the soil factor according to Table NA.4;

a_{gR} is the reference peak ground acceleration according to Table NA.3;

γ_a is the importance factor of the non-structural element

γ_a = 1,5 for anchorage elements of machinery and equipment required for life-saving systems. In all other cases, γ_a may be assumed to be 1,0.

NCI NA.D.8 Stability analysis

(1) In order to ensure that the building will not collapse in a seismic design situation (i.e. exceed the ultimate limit state), the following conditions with respect to structural strength, equilibrium, foundation stability and seismic joints shall be met. No allowance need be made for 2nd order effects (P-Δ-effects) when determining the seismic action.

a) Structural load resistance requirement: The following relationship shall be satisfied for all structural elements — including connections — as well as by all significant non-structural elements (NA.D.7).

$$E_d \leq R_d \tag{NA.D.10}$$

where

$$E_d = E\left\{\sum G_{k,j} \oplus P_k \oplus \gamma_1 \cdot A_{Ed} \oplus \sum \psi_{2,i} \cdot Q_{k,i}\right\} \tag{NA.D.11}$$

is the design value of the respective action-effect in the seismic design situation (DIN EN 1990) and

$$R_d = R\left\{\frac{f_k}{\gamma_M}\right\} \tag{NA.D.12}$$

is the design load resistance of the element, determined according to material-related requirements (characteristic value of the property f_k and partial safety factor γ_M). Furthermore, the weighting factor for seismic action γ_1 defined in DIN EN 1990 shall be given the value 1,0.

b) Equilibrium requirement: The structure shall remain in a state of stable equilibrium, even when exposed to seismic action. This includes effects such as tilting and sliding, taking DIN EN 1997-1 into consideration.

c) Load resistance of foundations: Foundations shall fulfil the conditions specified in NA.D.9.

d) Seismic joint requirements: Buildings shall be protected against the possibility of impacting adjacent structures or elements as a result of seismic action. This requirement is deemed to be satisfied if the distance to adjacent elements located at the point of potential impact is not less than the square root of the sum of the squares of the respective maximum expected horizontal displacements calculated using expression (NA.D.8). When planning the dimensions of joints, the limited compressibility of any joint filler (e.g. fibre matting) shall be taken into account. If no detailed verification calculations are provided in this context, the joint should be 1,5 times the sum defined above. This requirement is also deemed to be satisfied if the floors of adjacent buildings with the same number of storeys are at the same heights or, in the case of buildings with not more than three storeys, the distance between the double-leaf party walls is at least 40 mm.

(2) It is not necessary to provide calculated proof of the ultimate limit state of residential and other similar buildings (e.g. office buildings) as well as simple industrial buildings and hall-type buildings as described in paragraph (1) if the following conditions are met:

a) The number of full storeys above foundation level does not exceed the values given in Table NA.D.2. The top storey of a building is not classed as a full storey if the mass (due to permanent actions and imposed loads as described in NA.D.2, (2)) of the top storey or roof structure does not exceed 50% of the full storey immediately below it. For basements see paragraph (4).

b) The base shear force described in NA.D.2 (1) in each direction is less than 1,5 times the characteristic resultant wind force in the relevant direction.

(3) It is not necessary to perform stability analysis calculations for masonry structures if the conditions stated in NA.D.10 are satisfied.

(4) If the basement or storey located directly above foundation level is designed as a rigid box construction and is founded on one level only, then it does not need to be counted when the number of storeys is being determined. If it is not possible to assess the design beyond doubt in this context, then this condition may be assumed to be met if the total stiffness of this storey in each direction (i.e. the bending and shear stiffness of all structural elements that are primarily involved in transmitting the horizontal seismic loads) is at least five times greater than that of the storey immediately above it.

Table NA.D.2 — Importance class and permissible number of full storeys in buildings requiring a stability analysis by comparison with wind

Seismic zone	Importance class	Maximum number of full storeys
1	I to III	4
2	I and II	3
3	I and II	2

NCI NA.D.9 Foundations

(1) The integrity of the structure and/or the individual dynamically independent parts of a structure at its foundations shall be ensured by means of the structural design.

(2) In the case of separate foundation elements it shall be ensured (e.g. by the appropriate design of the elements) that no unpermitted displacements or action-effects in the structure occur as a result of possible displacements of the elements of the foundation relative to one another. A special design is not required in the following cases:

— for ground type A in all seismic zones;

— for ground type B and seismic zone 1.

(3) The simplified rules do not cover the foundations of buildings or of dynamically independent sections of buildings

a) at different depths, where this has a substantial effect on the vibration behaviour;

b) on different types of foundation element with considerable differences in deformation behaviour;

c) on different types of ground with considerable differences in deformation and settlement behaviour;

d) on pile foundations if these have a considerable influence on the dynamic behaviour.

(4) These simplified rules do not require the stability of foundations to be verified for the seismic load situation by calculation.

NCI NA.D.10 Specific rules for masonry buildings

Verification of the stability of masonry buildings by calculation is not required if the rules stated in DIN EN 1998-1:2010-12, 9.7, are complied with, taking account of the National Annex.