Safety Standards

of the
Nuclear Safety Standards Commission (KTA)

KTA 2201.3 (2013-11)

Design of Nuclear Power Plants Against Seismic Events; Part 3: Civil Structures

(Auslegung von Kernkraftwerken gegen seismische Einwirkungen; Teil 3: Bauliche Anlagen)

If there is any doubt regarding the information contained in this translation, the German wording shall apply.

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KTA SAFETY STANDARD

2013-11	**Design of Nuclear Power Plants Against Seismic Events; Part 3: Civil Structures**	KTA 2201.3

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PLEASE NOTE: Only the original German version of this safety standard represents the joint resolution of the 35-member Nuclear Safety Standards Commission (Kerntechnischer Ausschuss, KTA). The German version was made public in the Federal Gazette (Bundesanzeiger) on May 06, 2015. Copies of the German versions of the KTA safety standards may be mail-ordered through the Wolters Kluwer Deutschland GmbH (info@wolterskluwer.de). Downloads of the English translations are available at the KTA website (http://www.kta-gs.de).

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Comments by the Editor:

Taking into account the meaning and usage of auxiliary verbs in the German language, in this translation the following agreements are effective:

shall	indicates a mandatory requirement,
shall basically	is used in the case of mandatory requirements to which specific exceptions (and only those!) are permitted. It is a requirement of the KTA that these exceptions - other than those in the case of **shall normally** - are specified in the text of the safety standard,
shall normally	indicates a requirement to which exceptions are allowed. However, exceptions used shall be substantiated during the licensing procedure,
should	indicates a recommendation or an example of good practice,
may	indicates an acceptable or permissible method within the scope of the present safety standard.

Basic Principles

(1) The safety standards of the Nuclear Safety Standards Commission (KTA) have the task of specifying those safety-related requirements which shall be met with regard to precautions to be taken in accordance with the state of science and technology against any damage arising from the construction and operation of the plant (Sec. 7, para. (2), subpara. (3) Atomic Energy Act - AtG) in order to attain the protective goals specified in AtG and the Radiological Protection Ordinance (StrlSchV) and further detailed in the "Safety Criteria", the “Design-Basis Accident Guidelines” and the “Safety Requirements for Nuclear Power Plants (SiAnf)”.

(2) In accordance with Criterion 2.6 of the Safety Criteria, protective measures against seismic events are required, provided, earthquakes must be taken into consideration. Table I of the Design-Basis Accident Guidelines classifies earthquakes as belonging to that group of design basis accidents that requires taking precautionary plant engineering measures against damage and which, within this group, are relevant with respect to radiological effects on the environment. The basic requirements of these precautionary measures are dealt with in the safety standard series KTA 2201.

(3) To achieve these goals, safety standard KTA 2201.3 deals with the requirements specific to the seismic design of civil structures above and beyond their conventional design.

The safety standard series KTA 2201 comprises the following six parts:

KTA 2201.1: Principles

KTA 2201.2: Subsoil (soil and rock)

KTA 2201.3: Civil structures
(*the present safety standard*)

KTA 2201.4: Components

KTA 2201.5: Seismic instrumentation

KTA 2201.6: Post-seismic measures

1 Scope

(1) This safety standard applies to civil structures of nuclear power plants with light water reactors in order to achieve the protective goals specified in safety standard KTA 2201.1. It specifies the requirements for civil structures that must be met for the verification of their load-bearing capacity (stability) in case of a seismic event. Additionally, requirements are specified pertaining to the verification of the serviceability of civil structures as far as necessary for maintaining their safety-related function in case of a seismic event (e.g., deformation and crack-width limitations).

(2) This safety standard does not apply to cranes, to detachment devices for lifting equipment nor to the supporting and mounting constructions of components.

Note:

The term civil structures as used in this safety standard shall comprise buildings and structural members made of reinforced concrete, pre-stressed concrete, steel, as well as steel composite structures and brickwork. Among others, these include the containment, crane runways, platforms, anchor constructions and canals.

2 Definitions

(1) Radiation damping

Radiation damping is the damping due to energy being radiated into an adjacent medium, e.g. the subsoil.

(2) Ground, homogenous

The ground will be considered as being homogenous within the framework of calculations if a ground stratum with a near constant shear-wave velocity throughout its thickness is shown to have a ratio of thickness-to-foundation-radius larger than 4.

(3) Ground-building interaction

Ground-building interaction is the interaction relationship between local ground conditions and the vibration behavior of the building. This interaction comprises the kinematic interaction and the interaction due to the inertial forces of the building.

(4) Damping

Damping shall be distinguished according to:

a) Material damping
This is caused by internal micro-plastic procedures; it is measured in the laboratory excluding bordering influences and the like; it is dependent on the load level.

b) Component damping
This is the damping of a component (e.g., plate, beam) including bordering influences and effects from secondary components (concrete flooring, surfacing, etc.). For equal load levels, component damping is larger than material damping.

c) Structure damping
This is the damping of the total structure comprised of many components and it includes the influence from, e.g., non-supporting components, internals, connections and effects from radiation damping. For equal load levels, structure damping is significantly higher than component damping.

(5) Dimensionless frequency, a_o, of a foundation

The dimensionless frequency, a_o, is defined as follows:

$$a_o = \frac{\omega \cdot r_o}{v_s} \qquad (2\text{-}1)$$

Nomenclature:

ω: angular frequency,

r_0: substitute radius determined from the condition of equality of areas for translatory degrees of freedom or from the moments of inertia for rotatory degrees of freedom

v_s: shear-wave velocity in the subsoil.

(6) Impedance functions

The impedance functions are the complex, frequency dependent foundation stiffness values in the subsoil; their real and imaginary parts characterize stiffness and damping.

(7) Non-linearity, geometric, physical

a) Geometric non-linearity
A geometric non-linearity is the non-linear relationship between force and path values caused by equilibrium and kinematic considerations for the deformed system.

b) Physical non-linearity
A physical non-linearity is the non-linear relationship between stresses and expansions caused by a non-linear material behavior.

(8) Rayleigh damping

Rayleigh damping exists whenever the damping matrix, **C**, can be assumed to be a linear combination of the mass matrix, **M**, and the stiffness matrix, **K**, using the Rayleigh parameters α and β as follows:

$$\mathbf{C} = \alpha \cdot \mathbf{M} + \beta \cdot \mathbf{K} \qquad (2\text{-}2)$$

Equation (2-2) leads to the respective damping ratio, D, as a function of the angular frequency, ω:

$$D = \frac{\alpha}{2 \cdot \omega} + \frac{\beta \cdot \omega}{2} \qquad (2\text{-}3)$$

(9) Interaction, kinematic

The kinematic interaction describes the interaction of the foundation with the subsoil, whereby the foundation is assumed as being massless and rigid.

3 Seismic Event

(1) The seismic event to be assumed for the design basis earthquake shall comprise

a) ground response spectra both in horizontal and vertical direction together with the rigid-body accelerations specified in Sec. 3.5 of safety standard KTA 2201.1,

b) artificial acceleration time histories compatible with the ground response spectra specified in Sec. 4.3.3 of safety standard KTA 2201.1, or

c) recorded acceleration time histories as specified in Sec. 3.5 of safety standard KTA 2201.1.

(2) The excitation to be applied and superposed shall be as specified in Sec. 4.3.1 of safety standard KTA 2201.1.

4 Structure Analysis

4.1 Basic Requirements

(1) The analyses shall be performed and documented in a transparent fashion. The load-bearing behavior of the structure shall be described in the documentation.

(2) The margins of fluctuation of the analysis assumptions, in particular with regard to stiffness values, load support conditions, mass distributions and the vibration model, shall be documented and, if so required, estimated on the basis of limit state considerations.

4.2 Modeling

(1) The basic requirements regarding modeling are specified in Sec. 4.3.2 of safety standard KTA 2201.1. Regarding modeling of civil structures, the requirements under this section shall additionally be complied with.

(2) Provided all decisive influences from torsion and eccentricities between the centers of gravity and the centers of stiffness are taken into account (see Sec. 4.3.2 of safety standard KTA 2201.1), any unplanned torsions may be neglected.

4.2.1 Material characteristics

Regarding concrete, reinforcing steel, pre-stressing steel, structural steel and brickwork, the material characteristics to be used in the analysis model for static loads may be as specified in DIN EN 1992-1-1, DIN EN 1993-1-1, DIN EN 1994-1-1 and DIN EN 1996-1-1.

4.2.2 Effective stiffness

(1) The stiffness values of structural members may basically be determined under the assumption of a linear-elastic behavior of the structural material without any stiffness reduction.

(2) A possible stiffness reduction shall be taken into account for structural members from reinforced or pre-stressed concrete, provided, this has a significantly detrimental effect on the vibration behavior of the respective building structures and resulting loads of the structural members. The stiffening effect of not-load-bearing structural members, e.g., the infilling of brickwork, shall be considered in the building models, provided, this has a significantly detrimental effect on the vibration behavior.

(3) Stiffness values and eccentricities of the connecting steel components shall be taken into account, provided, they have a significantly detrimental effect on the vibration behavior of the respective building structures and on the resulting loading of structural members. The stiffness values may be varied to account for the flexibility of, e.g., framework corners or bolt connections.

4.2.3 Contributing masses

(1) The building structure analysis shall account for the resonant masses derived from the permanent masses (building structure and components), the masses of quasi-permanent live loads and variable live loads existing during operation in accordance with **Table 5-1**. In this context, Sec. 4.3.2 (6) of safety standard KTA 2201.1, shall be considered.

(2) The building analysis may be simplified by considering the components as being decoupled from the building, provided, the masses of the components are included in the mass of the building.

(3) Rotatory mass inertias shall be considered if they significantly influence the vibration behavior of the building

4.2.4 Damping

(1) Damping may be assumed to have a viscous (i.e. velocity dependent) effect. In this context, separate system analyses shall be performed for the buildings and individual structural members.

(2) The dynamic analysis shall take the different damping of the building structure and of the subsoil into consideration.

(3) The damping behaviors of buildings and partial building structures are significantly defined by structure damping. Therefore, the analyses for certifying the ultimate limit state (ULS) and the serviceability limit state (SLS) and for determining building response spectra shall be based on the damping ratios specified in Column A of **Table 4-1**. In the case of buildings whose damping behavior is exclusively defined by the damping of the materials and structural members, the determination of the building response spectra shall be based on the damping ratios specified in Column B of **Table 4-1**.

Building or Partial Building Structure built from	***A***	***B***
Reinforced concrete	7	4
Pre-stressed concrete	5	2
Steel with welded connections	4	2
Steel with bolted connections	7	4

Table 4-1: Calculation values for the damping ratio, D, in percent of critical damping; required for the analyses of buildings and partial building structures.
For an explanation of columns A and B see Section 4.2.4 para. (3).

(4) The load-bearing capacity (stability) of individual structural members shall be verified based on the damping ratios specified in Column A of **Table 4-2**. In case of additional requirements, the damping ratios specified in Column B of **Table 4-2** shall be applied. The use of differing values shall be well substantiated.

(5) Regarding the subsoil, the hysteresis related damping and the energy-radiation related damping shall be assumed in accordance with the subsoil and foundation conditions.

(6) In case of oscillations of fluids, a damping ratio of 5 % shall be assumed.

Individual Structural Member built from	***A***	***B***
Reinforced concrete	7	4
Pre-stressed concrete	5	2
Steel with welded connections	4	2
Steel with bolted connections: SL/SLP-connections *(SL - structural bolt connection with a borehole tolerance ≤ 2 mm; SLP - fitted bolt connection with a borehole tolerance ≤ 0.3 mm)*	7	4
Steel with bolted connections SLV(P)/GV(P) connections *(SLV(P) - preloaded fitted bolt connection; GV(P) - fitted friction-grip bolt connection)*	4	2
Steel composite	7	4
Brickwork	7	4

Table 4-2: Calculation values of the damping ratio, D, in percent of critical damping; required for the analyses of individual components.
For an explanation of columns A and B see Section 4.2.4 para. (4).

4.2.5 Hydrodynamic effects

(1) Fluid masses may be regarded as rigidly swinging with the building structure or may be visualized by a hydrodynamic model. The effect of the sloshing liquid on the components shall be considered separately.

(2) Oscillations of fluids relative to the components occurring in the horizontal direction may be considered using the equivalent-mass method. In this context, the liquid mass may be separated into one mass rigidly coupled to the component and one mass swaying relative to the component.

(3) In the case of open pools, the spillover may be calculated as a function of the spectral deflection of the sloshing mass under consideration of the geometry of the pool.

(4) The loading force on the pool walls shall be determined based on a hydrodynamic model.

(5) The horizontally effective liquid masses may be considered as being constant over the entire height of the component.

(6) In the vertical direction, the liquid mass shall be considered as being rigidly coupled to the base of the component.

4.3 Analysis Methods

4.3.1 General requirements

(1) The analysis may be based on one of the following analysis methods:

a) response spectrum method specified under Section 4.3.2,
b) time history method specified under Section 4.3.3,
c) frequency range method specified under Section 4.3.4, or
d) simplified method specified under Section 4.3.5.

(2) The ultimate limit state (ULS) shall basically be verified taking geometric non-linearities into account. This requirement may be waived, provided, the requirements in accordance with Sec. 4.4.2.2 of DIN EN 1998-1 are met.

(3) The load of structural members whose vibration behavior does not influence the overall behavior may be determined based on building response spectra (see Section 4.5).

(4) Non-linear analysis methods are permitted and, in individual cases, required.

Note:
The application of non-linear analysis methods requires in-depth knowledge and sufficient experience.

4.3.2 Response spectrum method

(1) Based on calculated eigen- (or natural) frequencies and eigenmodes, the response spectrum method shall be used to determine the maximum load parameters from the superposition of the contributions of the individual eigenforms (modal analysis). If a temporal sequence of the load parameters is required, a time history method shall be applied (see Section 4.3.3)

(2) The individual loads from an eigenform shall be determined based on a design response spectrum.

(3) Basically, eigenfrequencies up to at least the upper limit frequency and a resonant mass in each direction of at least 90 % of the total mass of the building shall be considered. Provided, the resonant mass of 90 % is not achieved up to the upper limit frequency, the rigid body contribution (see Sec. 4.4.2 (4) of safety standard KTA 2201.4) shall alternatively be applied. In this case, any local vibrations of relevance shall be accounted for.

(4) Basically, all unidirectional modal components of the load parameters shall be superposed by the Complete Quadratic Combination (CQC) method. Provided, all eigenforms meet equation (4-1) the modal components may be superposed using the Square Root of Sum of Squares (SSRS) method.

$$f_i > 1.35\, f_{i-1} \qquad (4\text{-}1)$$

Nomenclature:

f_i : eigenfrequency of the eigenform i

i : 1, …, n, with n equal to the number of eigenforms

The use of any other superposition method shall be well substantiated. To accurately represent the acceleration in the base regions, the rigid-body contribution shall be accounted for in the response spectrum method (see Sec. 4.4.2 of safety standard KTA 2201.4).

(5) The factor 1.35 in equation (4-1) applies to a damping ratio of 7 %. Smaller damping ratios lead to smaller factors, e.g. a factor of 1.2 applies to a damping ratio of 4 % and a factor of 1.1 to a damping ratio of 2 %.

4.3.3 Time history method

4.3.3.1 General requirements

(1) Regarding the time history method, the acceleration time history (or displacement time history) shall be applied as excitation for the entire vibrating system. The time histories of the movement and stress resultants and their maximum values shall be determined in the dynamic analysis by a modal analysis or by a direct integration of the time histories.

(2) The acceleration time history shall either be generated from the design response spectrum, or taken from the recorded time histories of the seismological expert report.

Note:

The number and generation method of the time histories as well as details regarding recorded time histories are specified in safety standard KTA 2201.1.

(3) The modal analysis may be applied as an approximation method to non-linear problems, provided, this is well substantiated for the individual case.

4.3.3.2 Modal time history method

(1) Application of the modal analysis requires differentiating between a proportionally and a non-proportionally dampened system. In the case of a proportionally dampened system, the eigenvectors of the undampened system may be used for decoupling the equation system.

(2) In case of extreme damping, the solution for the undampened system may only be applied if this is well substantiated. The maximum modal damping shall not exceed 15 % in the horizontal direction and 30 % in the vertical direction. Application of higher damping ratios shall be well substantiated for the individual case.

(3) In analogy to the response spectrum method (see Section 4.3.2 para. (3)), the rigid-body contribution shall be accounted for.

4.3.3.3 Direct integration

(1) The time increment for the direct integration shall be sufficiently small (no more than 0.1 times the reciprocal of the upper limit frequency) such that the vibration response at the maximum frequency of interest is accounted for to a sufficient accuracy and that the convergence and stability of the numeric integration is ensured.

(2) The Rayleigh damping of frequency-independent ground models shall not exceed a damping ratio of 15 % in horizontal direction and of 30 % in vertical direction for the significant frequencies. The Rayleigh damping of the building structures shall not exceed the values specified in **Tables 4-1** and **4-2** for the significant frequencies.

(3) In the case of nonlinear analyses with hysteresis effects, the viscous damping ratios specified in the columns B of **Tables 4-1** and **4-2** shall be applied.

4.3.4 Frequency range method

(1) The transfer function shall be calculated for a sufficient number of frequencies. In this context, it shall be ensured that they cover the entire frequency spectrum of the excitation as well as the eigenfrequencies of the analyzed structures up to the upper limit frequency.

(2) The sensitivity of the results to different support points shall be examined. The additional frequency support points required for applying the Fast-Fourier-Transformation (FFT) may be interpolated.

4.3.5 Simplified methods

In well substantiated cases simplified methods may be applied for buildings and individual structural members, provided, the horizontal and vertical excitations are accounted

for.

Note:

The simplified methods (quasi-static methods) for components are detailed under consideration of the application limits, e.g., in Sec. 4.4.4 of safety standard KTA 2201.4.

4.4 Ground-Building Interaction

(1) It is permissible to recalculate the ground response spectra specified as the seismic event to other subsoil horizons.

(2) The influence of the interaction between the building and the subsoil on the dynamic behavior of the building shall be determined. In the case of embedded buildings, the kinematic interaction shall basically be considered. The kinematic interaction may be waived for buildings that are, or are assumed to be surface founded. The assumption of a rigid restraint for the building is permissible, provided, the dominant frequency of the building assumed as being rigid is at least twice as high as the dominant frequency of the resilient building rigidly restrained in the foundation.

(3) It is permissible to assume a rigid restraint for the building, provided, the shear-wave velocity is higher than 1200 m/s.

(4) A homogenous ground may be represented by a dampened spring-mass model. The parameters for this model may be determined based on the Elastic Half-Space Theory. In this case, the nonlinear load-bearing behavior may be accounted for by limit state considerations with adapted ground characteristics (secant stiffness values). In the case of rigid circular or rectangular foundations with a dimensionless frequency, a_o, smaller than 2 for horizontal vibrations and smaller than 1 for vertical vibrations and for tilting movements, it is permissible for the sake of simplification to assume the static rigidity and damping determined from half-space equations to be frequency independent. In the case of strip foundations or non-rigid foundations, special considerations are required.

Note:

The static rigidity corresponds to the rigidity at the frequency zero. The damping determined from half-space approximation equations corresponds to the damping at resonance frequencies.

(5) In the case of stratified grounds with widely differing dynamic characteristics as well as of special foundation concepts (e.g., pile foundations), specific analyses shall be performed, e.g., calculations within the frequency-range.

(6) The possibility of a mutual subsoil related interference of neighboring buildings (building-ground-building interaction) shall be evaluated.

Note:

This interference is generally covered by varying the average value of the subsoil stiffness upward by multiplying it by 1.5 and downward by dividing it by 1.5 as specified in Sec. 4.3.2 (8) of safety standard KTA 2201.1.

(7) In the case of low frequencies (frequency is lower than the eigenfrequency of the ground stratum) and application of the modal method, energy-radiation related damping (radiation damping) may not be assumed.

4.5 Building Response Spectra

4.5.1 General requirements

(1) The building response spectra (secondary spectra) shall be determined based, e.g., on the acceleration time histories specified under Section 4.5.2 or on the substitution method specified under Section 4.5.3.

(2) The tertiary spectra of components (e.g., steel platforms) may be determined in accordance with Sec. 4.2.3 of safety standard KTA 2201.4.

4.5.2 Determining building response spectra based on acceleration time histories

(1) The acceleration time histories shall be determined by applying the time history method in accordance with Section 4.3.3 or the frequency range method in accordance with Section 4.3.4.

(2) The building response spectra calculated for the individual place or site of installation of the structural members or the components shall be converted to smoothed, broadened and enveloping design response spectra.

(3) The response spectra shall be calculated using the following procedural steps:

a) Averaging the results from the different acceleration time histories in accordance with Sec. 4.3.3 (3) of KTA 2201.1.

b) Capping spectrum tips from widths that are less than 10 % of the respective center frequency.

c) Enveloping the spectra, using the following approaches:

ca) Enveloping the low and high subsoil stiffness values in accordance with Sec. 4.3.2 (8) of KTA 2201.1.

cb) Enveloping the average subsoil stiffness values by a 15 %-widening of the maxima of the spectra.

Note:

Numerically extreme singularities caused by the modelling may be neglected in well substantiated cases.

d) Smoothing out the calculated spectra by an appropriate simplification using suitable polygon courses that ensure a robust design of the components.

Note:

Generally, this requirement is met if the spectra valleys with a base width of less than 20 % of their center frequency are closed off by a plateau extending from the lower peak.

e) Calculating the design response spectra for the required damping ratios. (The design response spectra should be calculated at least for the damping ratios, D, equal to 2, 4, 7 and 15 % of critical damping such that the curves for other damping ratios can easily be deduced by a logarithmic inter- or extrapolation. The spectra should be presented in graphic and tabular as well as digital formats.)

4.5.3 Substitution method regarding building response spectra

(1) In case of building structures with a homogenous distribution of their stiffness and mass, the following substitution method may be applied for determining building response spectra. This method leads directly to the design response spectra for given accelerations, a_G, at the place or site of installation of the structural member or the component.

(2) The spectrum shape shall be determined as shown in **Figure 4-1** and the spectrum amplification factor with respect to the building acceleration as shown in **Figure 4-2**, taking the damping ratio, D_1, for the building and the damping ratio, D_2, for the structural member or the component into account.

(3) The modal damping at the dominant eigenvibration (natural vibration) of the building may be used as its damping ratio, D_1. In this case, the damping of the building shall be applied as specified in Section 4.2.4 para. (3).

Note:

Due to the subsoil damping it is generally justified to apply a significantly higher damping ratio, D_1, than specified in **Table 4-1**.

(4) The damping ratio of the structural member, D_2, shall be applied as specified in Section 4.2.4 para. (4). The damping ratio of the component, D_2, shall be applied as specified in accordance with KTA 2201.4.

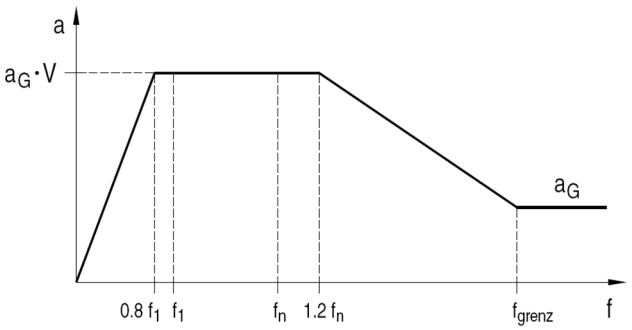

Figure 4-1: Determining the shape of the building response spectrum

Nomenclature:

f : frequency; the abscissa should be scaled logarithmically

f_1 : lowest significant eigenfrequency of the building at the lowest limit value within the range of the subsoil stiffness; it shall, however, not be higher than the right corner frequency of the highest plateau of the respective building response spectrum

f_n : highest significant eigenfrequency of the building at the upper limit value within the range of the subsoil stiffness; it shall, however, not be higher than the right corner frequency of the highest plateau of the respective building response spectrum

f_{limit} : upper limit frequency of the ground response spectrum

a : acceleration

a_G : acceleration at the place or site of installation of the structural member or component (rigid body acceleration)

V : spectrum amplification factor as specified in **Figure 4-2**

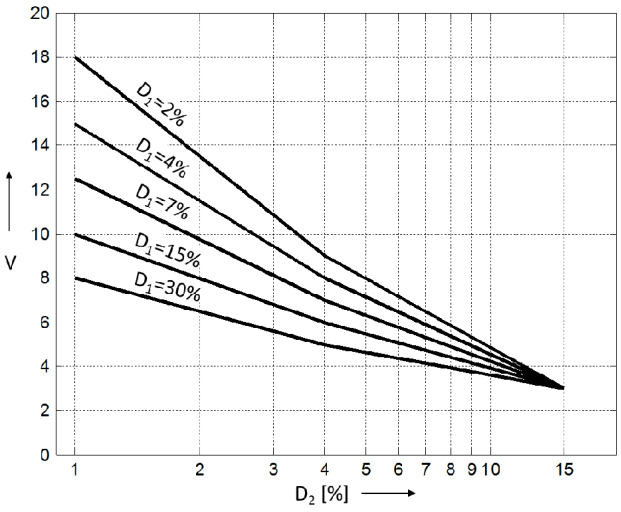

Figure 4-2: Determination of the spectrum amplification factor, V

Nomenclature:

D_1 : damping ratio of the building

D_2 : damping ratio of the structural member or the component

(5) The application of this method to building structures with an inhomogeneous distribution of stiffness and mass shall be well substantiated for the individual case.

Note:

The substitution method delivers a good approximation, provided, the response of the building is dominated by eigenfrequencies. If multiple eigenfrequencies are significant contributors the results of this method lie increasingly on the safe side.

5 Seismic Design Verification Concept

5.1 General Requirements

(1) In accordance with safety standard KTA 2201.1, civil structures shall be assigned to Class I, Class IIa or Class IIb.

(2) The load-bearing capacity (stability) and serviceability of Class I civil structures shall be designed such that they will fulfill their respective safety related function in case of a seismic event.

(3) Basically, the load-bearing capacity (stability) and, if applicable, serviceability of Class IIa civil structures shall be verified. Alternatively, it may be verified that a loss of their load-bearing capacity (stability) or an impairment of their serviceability will not hinder the Class I components and civil structures concerned from fulfilling their respective safety related function.

(4) Class IIb civil structures do not need to be designed for the design basis earthquake specified in safety standard KTA 2201.1.

5.2 Combination of Actions

(1) In accordance with DIN EN 1990, influential actions (*effects)* shall be classified as follows.

a) Permanent actions, G_k,

b) Pre-stressing actions, P_k,

c) Variable actions, Q_k, and

d) Actions from the design-basis earthquake A_{ed} specified in safety standard KTA 2201.1.

(2) The permanent actions, the pre-stressing actions and the variable actions shall be specified as characteristic parameters. Unusual effects from seismic events shall be specified as design values such that a partial safety factor of 1.0 can be implicitly presumed.

Note:

Regarding the design-basis earthquake specified in safety standard KTA 2201.1, the weighting factor, γ_1, in accordance with DIN EN 1990 and the importance factor, γ, are accounted for by the design value, A_{ed}.

(3) The design value of a seismic event, A_{ed}, shall be determined under consideration of all vertical loads from the combinations contained in equation (5-1).

$$\sum_{j=1}^{m} G_{kj} \oplus \sum_{i=1}^{n} \Psi_{Ei} \bullet Q_{ki} \qquad (5\text{-}1)$$

Nomenclature:

$\oplus$: indicates “shall be combined with”

$\sum$: indicates “combined effect of”

G_{kj} : characteristic value of the variable actions, j

Q_{ki} : characteristic value of the variable actions, i, causing inertial forces

Ψ_{Ei} : combination factor for the variable actions, i

(4) The design values for the seismic loading shall be determined from the ultimate limit states (ULS) and the serviceability limit states (SLS) of the combination of actions specified by equation (5-2) taking the combination factor, Ψ_2, into account.

$$E_d = E \{ G_k \oplus P_k \oplus A_{Ed} \oplus \sum_{i=1}^{n} (\Psi_{2,i} \bullet Q_{k,i}) \} \qquad (5\text{-}2)$$

(5) The combination factors, Ψ_E and Ψ_2, to be applied in equation (5-2) are specified in **Table 5-1**.

Actions		Combination Factors ψ_2	Combination Factors ψ_E
Variable actions, Q_k	Live Loads / Operating Loads		
	- quasi-permanent	1.00	1.00
	- tank filling	0.80	0.80
	- temperature during operation	0.80	1)
	- platform surface loads	0.30	0.25
	- friction forces	0.30	1)
	- loads during major inspections	0.30	0.25
	- others	0.80	0.25
	Live Loads		
	- vehicle load < 30 kN	0.60	0
	- vehicle load > 30 kN	0.30	0
	Lifting device loads	0	0
	Wind	0	1)
	Snow	0	0
	Climatic temperature effects	0	1)
	Restraint force due to settlements	1.00	1)
1) not applicable			

Table 5-1: Combination factors for actions

5.3 Combinations of Loads Caused by Key Components of a Seismic Event

(1) In the analysis model, the excitation shall be considered as being simultaneously effective in all three orthogonal directions. The significant load parameters shall basically be determined by applying the combination rules for modal contributions.

(2) Alternatively, the load parameters may be determined individually for each of the three directions of the seismic event. In this case, the maximum value for each load parameter of the building may be determined as specified in Sec. 4.3.1 of safety standard KTA 2201.1 from the individual components of the seismic event.

(3) In the design of structural members, the individually determined unidirectional load parameters shall basically be considered as acting simultaneously unless more exact methods for determining the simultaneous action of the load parameters are applied.

(4) The loads resulting from the combination of actions specified in paras. (1), (2) or (3) shall be correlated to the corresponding ultimate limit state (ULS) and the serviceability limit state (SLS). The direction of each of the components in these combinations shall be chosen such that a most unfavorable value for the respective load parameter is achieved.

5.4 Ultimate Limit State (ULS)

5.4.1 General requirements

(1) It shall be verified that, at the ultimate limit state, equations (5-3) and (5-4) are fulfilled.

$$E_d \leq R_d \quad (5\text{-}3)$$

$$R_d = R\{ f_{k,i} / \gamma_{M,i} \} \quad (5\text{-}4)$$

Nomenclature:

E_d : design value calculated from equation (5-2) for the load (e.g., stress resultant) during the seismic event

R_d : design value of the load-bearing capacity as a function of the requirements specific to the structural material taking the partial safety factors into account.
The partial safety factors generally represented by the characteristic strength value of the structural material, f_k, and the corresponding partial safety factors, γ_M.

(2) The partial safety factors for determining the load-bearing capacity at the ultimate limit state (ULS) shall be chosen as specified for the individual structural materials in Sections 6.1 through 6.4.

5.4.2 Ductility

In so far as dissipative effects are utilized in verifying the load-bearing capacity (stability), a sufficient ductility shall be verified for the supporting components and total structure.

Note:

It is possible that the serviceability is detrimentally affected by utilizing the ductility, see Section 4.3.5.

5.4.3 Equilibrium conditions

The building shall remain in a stable equilibrium even in case of seismic events. This also includes effects from tipping and sliding in accordance with DIN EN 19971 and DIN EN 1998-5.

5.4.4 Load-bearing capacity (stability) of the foundations

(1) The foundations shall basically meet the requirements in accordance with DIN EN 1998-5 and DIN EN 1997-1.

(2) Provided, the simplified linear verification of eccentricity for the force-resultant at the foundation base in accordance with DIN EN 1997-1 cannot be carried out, alternative detailed verifications are permissible.

(3) In the case of pile foundations, interactions with the ground and between the piles shall be considered.

5.5 Serviceability Limit State (SLS)

5.5.1 General requirements

(1) It shall be verified that an as-specified use of the civil structure is ensured for the combination of actions of the design basis earthquake calculated by equation (5-2). This requires performing verifications at the serviceability limit state (SLS) with corresponding requirements (e.g., deformation and crack-width limitations).

(2) It shall be shown that equation (5-5) is met for the serviceability limit state (SLS).

$$E_d \leq C_d \quad (5\text{-}5)$$

Nomenclature:

E_d : design value of the load (e.g., stress, deformation, crack width) during a seismic event (see equation (5-2)

C_d : design value of the serviceability criterion (e.g., permissible stress, deformation or crack width) required for achieving the protective goals specified in safety standard KTA 2201.1.

(3) The serviceability criterion for the design value, C_d, shall be specified in accordance with plant specific requirements.

Note:

In order to sustain the safety-related function of the civil structures, additional requirements may have to be met. These are, e.g., deformation limitations from component related requirements or crack-width limitations with respect to leak-tightness requirements or requirements regarding load-bearing and deformation behavior of dowel fastenings. The respective requirements will be specified for the individual case.

5.5.2 Deformations

(1) The deformations shall be determined for the combination of actions specified in Section 5.2.

(2) If the analysis of the building deformations does not account for the nonlinear behavior of the material (e.g., by a reduction of the stiffness values) and the damping values specified in **Table 4-1** are used, then the building deformations determined for the seismic event shall be increased by 50 %.

(3) The relative deformations of independently swaying buildings may be calculated as the square root of the sum of the squared individual deformations.

(4) The following requirements shall be met regarding a seismic design of joints.

a) Buildings shall be protected against earthquake-induced collisions with neighboring buildings or structural members. This requirement is considered as being met if, at the places of a possible collision, the distance between neighboring structural members is smaller than the relative deformation calculated as specified in para. (3).

b) When designing the joint width, the stiffness and limited compressibility of a possible joint material shall be taken into account.

6 Structure-Type Dependent Seismic Verifications

6.1 Structural Members from Reinforced and Pre-Stressed Concrete

6.1.1 General requirements

(1) The requirements regarding analysis and structural design in accordance with DIN EN 1992-1-1 shall be applied, provided, in the following no other requirements are specified.

(2) It is not permitted to use a Concrete Strength Class smaller than C 20/25.

(3) Unless a more exact verification of the ductility is performed, the reinforcing steel should have a high Type B ductility with a minimum value $k = (f_t/f_y)_k$ greater than 1.08 and a characteristic percentage of elongation at maximum load, ε_{uk}, greater than 5 %.

6.1.2 Strength parameters

(1) In accordance with DIN EN 1992-1-1, the design value for the uniaxial (or unconfined) compressive strength at the ultimate limit state (ULS), f_{cd}, shall be based on equation (6-1).

$$f_{cd} = \alpha \cdot f_{ck} / \gamma_c \qquad (6\text{-}1)$$

Nomenclature:

f_{ck} : characteristic value for the uniaxial (or unconfined) compressive strength in accordance with DIN EN 1992-1-1

α : reduction factor during a seismic event. In the case of standard concrete; a value of $\alpha = 0.85$ shall be assumed.

γ_c : partial safety factor; its values shall be as specified in **Table 6-1**

Structural Material	***Partial Safety Factor***
Concrete	$\gamma_c = 1.0$
Reinforcing and pre-stressing steel	$\gamma_s = 1.0$
Nonlinear Analysis Methods	
Load-bearing capacity [1] in accordance with DIN EN 1992−1−1/NA	$\gamma_R = 1.0$
Design value for the compressive strength [2] of concrete, f_{cR}	$1.0 \cdot \alpha \cdot f_{ck}$
Design value for the yield strength of reinforcing steel, f_{yR}	$1.0 \cdot f_{yk}$
Design value for the 1%-proof stress of pre-stressing steel	$1.0 \cdot f_{pk}$
[1] Called system resistance in Table 2 of DIN EN 1992-1-1/NA	
[2] Reduction factor, α, according to DIN EN 1992−1−1/NA	

Table 6-1: Partial safety factors for determining the ultimate limit state (ULS) for the load-bearing capacity (stability) during a seismic event

(2) Deviant design values for the characteristics for the structural material concrete other than those in accordance with DIN EN 1992-1-1 may be applied if they are verified.

(3) The design values for the strength of the reinforcing steel and pre-stressing steel shall take the partial safety factors specified in **Table 6-1** into account.

6.1.3 Verifying the load-bearing capacity (stability)

(1) The design of biaxially loaded structural members (e.g., support structures) shall take into account the effect from a simultaneous action in the two horizontal directions of the seismic event.

(2) Alternatively, the biaxial bending design of support columns may be replaced by individual verifications of a 70 % flexural load bearing capacity, M_{Rdi}, in each direction, i, as given by equation (6-2).

$$M_{Edi} \leq 0.7\, M_{Rdi} \qquad (6\text{-}2)$$

Nomenclature:

M_{Edi} : external moment

M_{Rdi} : bending load bearing capacity

i : each of one of the two directions

(3) The shear-force resistance of a reinforced or pre-stressed concrete component shall be verified in accordance with DIN EN 1992-1-1. The partial safety factor, γ_c, specified in **Table 6-1** shall be applied as design value for the shear-force resistance, $V_{Rd,ct}$.

(4) The punching shear verification (in the case of support columns on plates or foundations) shall be verified in accordance with DIN EN 1992-1-1. The partial safety factor, γ_c, specified in **Table 6-1** shall be applied as design value for the shear-force resistance, $V_{Rd,ct}$.

6.2 Steel Components

(1) The requirements in accordance with DIN EN 1993-1-1 shall be applied to the design and construction of steel components, provided, in the following no other requirements are specified.

(2) The partial safety factor, γ_M, of the load-bearing capacity shall be differentiated according to the various verification methods:

γ_{M0} : stress analysis of the cross-section,

γ_{M1} : stability analysis of structural members regarding flexural buckling and flexural-torsional buckling,

γ_{M0} : tensile strength analysis for the net section,

(3) The partial safety factors γ_{M0} and γ_{M1} shall be assumed as being equal to 1.0. In the case of stress loads that are dependent on tensile strength, the value for γ_{M2} shall be assumed as being equal to 1.15.

6.3 Brickwork

6.3.1 General requirements

(1) It is not permissible in the design for Class I buildings to take account of brickwork walls as seismic reinforcements.

(2) The requirements in accordance with DIN EN 1998-1 shall be applied to the design and construction of brickwork, provided, in the following no other requirements are specified.

(3) Only bricks and brickwork mortar may be used that meet the minimum requirements in accordance with DIN EN 1998-1.

6.3.2 Verifying the load-bearing capacity (stability)

(1) Provided, no significant loads are to be expected from induced deformations of the main structure that act parallel to the plane of the wall, verification of the distribution of seismic loads perpendicular to the plane of the wall may take arch effects between horizontal and perpendicular stiffening elements (e.g., steel or reinforced concrete beams) into account. In this case, it shall be verified that the arch effects accounted for are not negated by the flexibility (resilience) of the stiffening elements.

(2) The value of the partial safety factor, γ_M, to be applied to brickwork in the verifications of the ultimate limit state (ULS) shall be assumed as being equal to 1.2.

6.3.3 Constructional design

(1) In the case that a horizontal stiffening is accounted for by arch effects, it shall be ensured that the butt joints are completely mortared over the entire width of the wall. Furthermore, a weakening of bricks due to holes shall be considered.

(2) The horizontal and vertical enclosing components shall be interconnected and shall be anchored in the components of the main structure.

(3) The force transmission between enclosing components and brickwork shall be ensured.

6.4 Steel Composite Civil Structures

(1) The requirements in accordance with DIN EN 1994-1-1 shall be applied to the design and construction of steel composite civil structures, provided, in the following no other requirements are specified.

(2) The partial safety factors γ_c for concrete and γ_s for reinforcing steel shall be assumed as specified in Section 6.1.2.

(3) The partial safety factors γ_M for the structural steel, the profiled metal sheets and the connecting elements shall be assumed as specified in Section 6.2.

(4) The partial safety factor γ_V for connecting compounds shall be assumed as being equal to 1.25.

6.5 Anchor Constructions

(1) The anchor constructions for interconnecting structural or plant-engineering components with the reinforced concrete components shall be designed to safely transmit the loads occurring during a seismic event. In this context, the anchor base, i.e. the corresponding reinforced concrete components, is subject to special requirements.

(2) It shall be ensured that within the range of anchor constructions using metal dowels or head studs no plastic deformations and only permissible crack widths will occur in the case of a seismic event.

Note:

Crack-width limitations and additional requirements are specified in the Building Inspectorate Suitability Certifications of the anchor constructions.

6.6 Subsoil-Embedded Pipelines and Canals

(1) In the case of subsoil-embedded pipelines and canals, the subsoil deformations due to seismic waves and the additional dynamic soil pressure shall be taken into account.

(2) In the case of subsoil-embedded pipelines and canals assembled from multiple stiff parts, it shall be verified that in the transition regions the permissible torsions and axial displacements are not exceeded.

Note:

Safety standard KTA 32011.2 specifies requirements regarding embedded steel pipelines.

6.7 Support Structures

Support structures shall be designed to resist the dynamic soil pressure. The design value for the force acting from the ground-side onto the support structure may be determined in accordance with DIN EN 1998-5.

Appendix A
Regulations Referred to in the Present Safety Standard

(Regulations referred to in the present safety standard are valid only in the versions cited below. Regulations which are referred to within these regulations are valid only in the version that was valid when the latter regulations were established or issued.)

AtG		Act on the peaceful utilization of atomic energy and the protection against its hazards (Atomic Energy Act – AtG) of December 23, 1959, revised version of July 15, 1985 (BGBl. I, p. 1565), most recently changed by Article 5 of the Act of August 28, 2013 (BGBl. I 2013, No. 52, p. 3313)
StrlSchV		Ordinance on the protection from damage by ionizing radiation (Radiological Protection Ordinance – StrlSchV) of July 20, 2001 (BGBl. I, p. 1714; 2002 I, p. 1459), most recently changed by Article 5, Sec. 7 of the Act of February 24, 2012 (BGBl. I 2012, No. 41, p. 212)
SiAnf	(2012-11)	Safety requirements for nuclear power plants of November 22, 2012 (BAnz AT of January 24, 2013)
Safety Criteria	(1977-10)	Safety criteria for nuclear power plants of October 21, 1977 (BAnz No. 206 of November 3, 1977)
Design-Basis Accident Guidelines	(1983-10)	Guidelines for the assessment of the design of nuclear power plants with pressurized water reactors against design-basis accidents as defined in Sec. 28, para. (3) StrlSchV (Design-basis accident Guidelines) of October 18, 1983 (Addendum to BAnz No. 245 of December 31, 1983)
KTA 2201.1	(2011-11)	Design of nuclear power plants against seismic events; Part 1: Principles
KTA 2201.4	(2012-11)	Design of nuclear power plants against seismic events; Part 4: Components
DIN EN 1990	(2010-12)	Eurocode: Basis of structural design; German version EN 1990:2002 + A1:2005 + A1:2005/AC:2010 together with
DIN EN 1990/NA	(2010-12)	National Addendum: Nationally specific parameters for DIN EN 1990
DIN EN 1992-1-1	(2011-01)	Eurocode 2: Design of concrete structures Part 1-1: General rules and rules for buildings; German version EN 1992-1-1:2004/A1:2014 together with
DIN EN 1992-1-1/NA	(2011-01)	National Addendum: Nationally specific parameters for DIN EN 1992-1-1
DIN EN 1993-1-1	(2010-12)	Eurocode 3: Design of steel structures Part 1-1: General rules and rules for buildings; German version EN 1993-1-1:2005 + AC:2009 together with
DIN EN 1993-1-1/NA	(2010-12)	National Addendum: Nationally specific parameters for DIN EN 1993-1-1
DIN EN 1994-1-1	(2010-12)	Eurocode 4: Design of composite steel and concrete structures Part 1-1: General rules and rules for buildings ; German version EN 1994-1-1:2004 + AC:2009 together with
DIN EN 1994-1-1/NA	(2010-12)	National Addendum: Nationally specific parameters for DIN EN 1994-1-1
DIN EN 1996-1-1	(2010-12)	Eurocode 6: Design of masonry structures Part 1-1: General rules for reinforced and unreinforced masonry structures; German version EN 1996-1-1:2005 + AC:2009 together with
DIN EN 1996-1-1/NA	(2011-04)	National Addendum: Nationally specific parameters for DIN EN 1996-1-1
DIN EN 1997-1	(2009-09)	Eurocode 7: Geotechnical design - Part 1: General rules; German version EN 1997-1:2004 + AC:2009 together with
DIN EN 1997-1/NA	(2010-12)	National Addendum: Nationally specific parameters for DIN EN 1997-1
DIN EN 1998-1	(2010-12)	Eurocode 8: Design of structures for earthquake resistance Part 1: General rules, seismic actions and rules for buildings; German version EN 1998-1:2004 + AC:2009 together with
DIN EN 1998-1/NA	(2011-12)	National Addendum: Nationally specific parameters for DIN EN 1998-1
DIN EN 1998-5	(2010-12)	Eurocode 8: Design of structures for earthquake resistance Part 5: Foundations, retaining structures and geotechnical aspects; German version EN 1998-5:2004 together with
DIN EN 1998-5/NA	(2011-07)	National Addendum: Nationally specific parameters for DIN EN 1998-5