Safety Standards

of the
Nuclear Safety Standards Commission (KTA)

KTA 2201.2 (2024-12)

**Design of Nuclear Power Plants Against Seismic Events
Part 2: Subsoil**

(Auslegung von Kernkraftwerken gegen seismische Einwirkungen; Teil 2: Baugrund)

Previous versions of this Safety Standard
were issued 1982-11, 1990-06 and 2012-11

Editor:
KTA-Geschaeftsstelle c/o Bundesamt fuer die Sicherheit der nuklearen Entsorgung (BASE)
Willy-Brandt-Str. 5 • 38226 Salzgitter • Germany
Telephone +49(0)30-18333-2907

KTA SAFETY STANDARD

December 2024	**Design of Nuclear Power Plants Against Seismic Events Part 2: Subsoil**	KTA 2201.2

Previous versions of this safety standard: 1990-06 (BAnz No. 20a of January 30, 1991)
1982-11 (BAnz No. 64a of April 6, 1983)
2012-11 (BAnz AT 23.01.2013 B5 of December 27, 2012)

Contents

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 this 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 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 the AtG, the Radiation Protection Act (StrlSchG) and the Radiation Protection Ordinance (StrlSchV) as well as further detailed in the Safety Requirements for Nuclear Power Plants (SiAnf) and the Interpretations of the SiAnf.

(2) In accordance with SiAnf protective measures against seismic events are required, provided, earthquakes must be taken into consideration. Earthquakes belong to that group of external events requiring preventive plant engineering measures. The basic requirements of these preventive measures are dealt with in safety standard series KTA 2201.

(3) The present safety standard KTA 2201.2 – as part of the series KTA 2201 entitled “Design of nuclear power plants against seismic events” – deals with the determination and application of subsoil properties governing the seismic design of nuclear power plants. The series KTA 2201 is comprised of the following six parts:

Part 1: Principles,

Part 2: Subsoil (*the present safety standard*),

Part 3: Design of civil structures,

Part 4: Components,

Part 5: Seismic instrumentation,

Part 6: Post-seismic measures.

1 Scope

This safety standard applies to nuclear power plants with light water reactors to achieve the protective goals specified in safety standard KTA 2201.1.

2 Definitions

(1) Soil liquefaction

Soil liquefaction is understood to be the reduction of the shear strength of the soil on account of increasing pore water pressure. The increase of the pore water pressure in turn is caused by the tendency of soil to compact due to the repeated dynamic loading.

(2) Dynamic shear modulus

The dynamic shear modulus (G) of the soil describes the elastic deformation behavior due to the dynamic shear stresses. It is determined by laboratory tests or by in-situ tests. In soil, G decreases with increasing shear deformation; its maximum value, G_0, occurs at very small dynamic shear deformations (shear strains $\gamma \leq 10^{-5}$).

(3) Compressional wave velocity

The compressional wave velocity is the speed of propagation of the compressional waves. Compressional waves (also called primary waves or pressure waves) are elastic, longitudinally polarized spatial waves which, when passing though a medium, compresses and stretches the volume elements.

(4) Material damping

Material damping in a vibrating system or in wave propagation is the conversion of mechanical energy into thermal energy by dissipation (friction, viscosity).

(5) Shear-wave velocity

The shear-wave velocity is the speed of propagation of the shear waves. Shear waves (also called secondary waves or transverse waves) are elastic, transversely polarized spatial waves which, when passing though a medium, causes the particles to move perpendicular to the direction of wave propagation. This leads to a shear deformation of the propagation medium. A propagation of shear waves is possible in solid bodies but not in fluids or gases due to the negligible shear resistance of the latter.

3 Subsoil Investigation

(1) The evaluation of the subsoil conditions of the site shall be based, in particular, on geotechnical reports concerning geology, seismology and the subsoil.

(2) The results of the geotechnical reports of geological and seismological as well as subsoil investigations shall be documented by the geotechnical expert in characteristic soil profile cross-sections.

(3) Type and extent of the necessary geotechnical investigations as well as the characteristics required to be determined shall be specified in accordance with DIN EN 1997-1, DIN EN 1997-1/NA and DIN 1054 in connection with DIN EN 1997-2, DIN EN 1997-2/NA und DIN 4020. In this context, the investigations shall be extended down to a depth equal to at least twice the diameter of the building or a coextensive circular foundation surrounding the same area as the building.

4 Dynamic Subsoil Properties

(1) The mechanical properties of the subsoil under dynamic loading are significantly different from those under static loading.

(2) The behavior of soil under dynamic loading is determined by a number of influencing factors. Essential factors are the shear deformation amplitude and the number of loading cycles, the mean effective confining static stress under the foundation as well as the void ratio and the degree of saturation of the soil.

(3) The design of a nuclear power plant against seismic events shall be based on geotechnical reports, which shall contain the following data on dynamic subsoil properties for the individual soil layers:

- Dynamic shear modulus, G_0, for small shear deformations,
- Poisson’s ratio, ν,
- material damping in terms of the damping ratio, D,
- density, ρ,
- shear wave velocity, v_s, and compressional wave velocity, v_p, for small shear deformations.

In this context, upper and lower limit values for G_0 shall be specified as a function of the depth and of the stress condition of the soil when subjected to the building structural load.

Note:

Examples of analytical procedures concerning the determination of dynamic subsoil properties are presented in Section A 1 of Appendix A.

(4) The shear modulus and material damping shall typically be determined as a function of the shear deformation and stress condition of the soil.

Note:

An analytical procedure concerning the determination of the shear modulus and material damping is presented in Section A 2 of Appendix A.

(5) The procedures chosen to be applied for determining the dynamic subsoil properties shall conform to the particular subsoil conditions. In-situ procedures and laboratory tests shall be applied. Alternatively, it is permissible to proceed as specified under para. 6.

(6) The dynamic subsoil properties at one site may be applied to another site, provided, the subsoil and the geological boundary conditions of these two sites are comparable.

5 Changes of the Subsoil

(1) Possible changes of the subsoil that might occur as a result of earthquakes shall be determined. These include, in particular:

a) Permanent vertical deformations as a result of compaction of the grain structure.

   Note:

   Permanent horizontal deformations, e.g., at sites with horizontal soil layers, are usually negligible.

   Examples of the basic principles concerning the evaluation of soil liquefaction are presented in Section A 3 of Appendix A.

b) Reduction of the shear strength due to soil liquefaction or to other changes in the soil.

(2) No verification with respect to soil liquefaction is required for nuclear power plant sites for which either the maximum horizontal ground acceleration is lower than 0.8 m/s^2, or where the subsoil consists of stiff geologically preloaded clays or equivalent cohesive soils.

Appendix A

Analytic Procedures

A 1 Analytic Procedures for Determining the Dynamic Subsoil Properties

Analytic Procedure	*Measurement Procedure*	*a) Measurement Parameters b) Derived Parameters*	[1]*Shear Deformation Range*
In-situ Procedures			
Borehole Procedures			
Uphole procedure	Excitation in the borehole, measurement at the surface	a) Travel times (P- and S-wave velocities) [2] b) Shear modulus, Poisson ratio	approx. 10^{-7} to 10^{-5}
Downhole procedure	Excitation at the surface, measurement in the borehole		
Crosshole procedure	Excitation in one borehole (transmitter), measurement in one or more adjacent boreholes (receiver)		
Seismic tomography	Sound transmission analysis of the area to be investigated by arranging a whole network of transmitters and receivers in boreholes		
Surface Procedures			
Swingers	Continuous excitation and measurement at ground level	a) Travel times (surface wave velocities) b) Shear modulus	approx. 10^{-7} to 10^{-5}
SASW (Spectral Analysis of Surface Waves)	Pulse excitation and measurement on the ground level		
Laboratory Procedures			
Resonant-column test	Determination of the velocities for various frequencies and various amplitudes	a) Frequency, wave length (P- and S-wave velocities) b) Shear modulus, Poisson ratio, material damping	approx. 10^{-7} to 5×10^{-4}
Ultrasonic measurement	Determination of the velocities based on the ultrasonic pulses	a) Travel times (P- and S-wave velocities) b) Shear modulus, Poisson ratio	approx. 10^{-7} to 10^{-5}
Cyclical shear test	Measurement for simple shear under uniaxial loading and impeded lateral strain	a) Deformations, stresses (characteristic of stress versus shear deformation) b) Shear modulus, Poisson ratio, material damping	approx. 5×10^{-5} to 10^{-1}
Cyclical tri-axial test	Measurement for vertical and tangential loading under various stress conditions	a) Deformations, stresses (characteristic of stress versus shear deformation) b) Shear modulus, Poisson ratio, material damping, strength properties	
Cyclical torsion test	Measurement for tangential loading and uniaxial loading		

[1] The shear deformations for earthquakes in Germany lie in the range between 10^{-5} and 10^{-3}.
[2] P-wave – primary or compressional wave; S-wave – secondary or shear wave

Table A 1: Analytical procedures for determining the dynamic subsoil properties

A 2 Determining Dynamic Shear Modulus and Material Damping from In-Situ Investigations or Auxiliary Calculations

(1) The dynamic shear modulus, *G,* and material damping, *D*, may be determined following the discussion in [1] from Equations A 1 and A 2 depicted in **Figure A-1**.

$$G = \frac{1}{1+\gamma_h} G_0 \qquad \text{(A-1)}$$

$$D = \frac{\gamma_h}{1+\gamma_h} D_{\max} \qquad \text{(A-2)}$$

where

$$\gamma_h = \frac{\gamma}{\gamma_r}\left[1 + a \cdot \exp\left(-b\frac{\gamma}{\gamma_r}\right)\right] \qquad \text{(A-3)}$$

and

$$\gamma_r = \frac{\max\tau}{G_0} \qquad \text{(A-4)}$$

Nomenclature:

- γ_h : *hyperbolic shear strain*
- G_0 : *dynamic shear modulus for very small shear strains ($\gamma \le 10^{-5}$)*
- D_{max} : *damping ratio for large shear strains (material damping)*
- γ : *shear strain*
- γ_r : *reference shear strain*
- $\max\tau$: *maximum shear stress*
- *a, b* : *coefficients derived from laboratory tests*

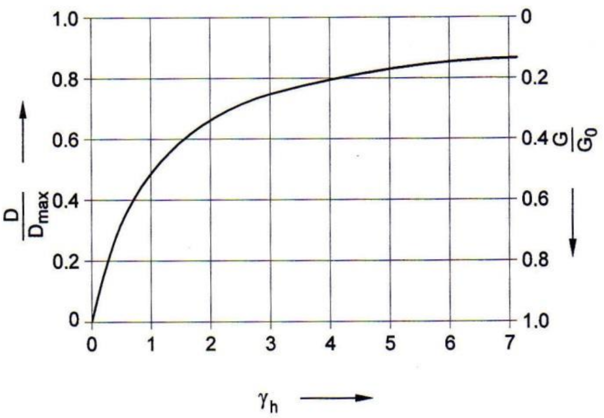

Figure A-1: Relationship between dynamic shear modulus, material damping and hyperbolic shear strain

(2) The damping ratio, D_{max}, and the coefficients, *a* and *b*, can be determined for different soil types with the aid of the equations quoted in [1]. The major influencing parameters are the number and frequency of the loading cycles as well as the stress condition prevailing in the soil. G_0 shall be determined by in-situ measurements of the shear-wave velocity, v_s.

(3) The dynamic shear modulus shall be calculated as

$$G_0 = v_s^2 \cdot \rho \qquad \text{(A-5)}$$

where ρ is the soil density. For estimation purposes, empirical approximate equations available in literature may be used; they take on the following form:

$$G_0 = \alpha \cdot \frac{(\beta - e)^2}{1+e} \cdot \sigma'^{\,\delta}_m \cdot (OCR)^K \qquad \text{(A-6)}$$

Nomenclature:

- *e* : *void ratio of the soil*
- σ'_m : *mean effective principal stress in the soil*
- δ : *exponent, equal to approximately 0.5*
- α, β : *parameters dependent on grain shape, grain-size distribution and degree of saturation*
- *OCR* : *over-consolidation ratio*
- *K* : *exponent dependent on the plasticity index of the soil*

A 3 Basic Principles for the Evaluation of Soil Liquefaction

A 3.1 Liquefaction Potential

(1) Uniform fine-grained sands below the ground water table have a greater tendency for soil liquefaction than non-uniform coarse sands. The decisive influence is the soil layer compactness. The looser the sand layer, the greater its tendency towards liquefaction. All other conditions being the same, the tendency towards liquefaction decreases with an increasing effective stress in the soil [2].

(2) In the case of a higher groundwater table, the danger of liquefaction is greater than in the case of a deeper groundwater table. The danger of liquefaction increases with the intensity and duration of the earthquake.

(3) Furthermore in this context, the permeability of the sand and the drainage conditions shall be taken into consideration. The thinner the endangered layers and the faster they can drain into adjacent permeable zones, the shorter the sand remains in a liquefied state and the less lasting the consequences.

(4) Stiff and geologically preloaded clays or equivalent cohesive soils are insensitive to vibrations. They are not susceptible to liquefaction.

(5) Soils with a grain size in the range between medium silt and coarse sand are susceptible to liquefaction. This applies, in particular, to fine sands. In general, liquefaction of gravels is a very short-term phenomenon and, therefore, no damaging shear deformations are expected to occur. Again, the duration of liquefaction depends on the drainage conditions.

(6) In stratified soils, the liquefaction process can spread from an easily liquefiable layer to soil areas which would not be endangered under normal conditions. Therefore, the danger of liquefaction shall be evaluated by analyzing the susceptible layers.

(7) The danger of soil liquefaction shall be evaluated on the basis of the following analyses:

a) Drilling under and next to the planned structure up to a depth of 25 m below ground level, grain distribution analyses,

b) cone penetration tests (CPT). If only dynamic probing results are available, these may be used by applying appropriate conversion factors,

c) ground water measurements (maximum ground water level with an exceedance probability of 10^{-2} per annum), and

d) dynamic soil shear tests for more precise determination of the properties of individual soil layers if necessary.

(8) Soils with risk of liquefaction include sands, gravelly sands, silts and fine-grained soils with a plasticity index of less than 15. For soils with a clay content greater than 15%, the liquefaction risk assessment may be omitted.

(9) The verification is carried out by comparing the cyclic shear stress ratio (effect of action) induced by the earthquake with the shear stresses resistance ratio (resistance). The verification is carried out at all depths down to 15 m below ground level [2].

(10) The assessment of liquefaction should be carried out for the free field.

N o t e :

The influence of a structure can be both positive and negative depending on the ground conditions. In any case, it is not significant and can be neglected.

(11) To estimate resistance to liquefaction, empirical methods based on on-site exploration results should be preferred over laboratory-based methods. The latter are very complex and require a high level of qualification from the soil mechanic laboratory in order to correctly reproduce the in-situ conditions in the experiment.

(12) Methods based on CPT are considered to be reliable compared to SPT (Standard Penetration Test) or shear wave measurements, so that only the CPT-based method according to [3] and [4] is described below.

(13) The CPT measurement and evaluation must be carried out according to EN ISO 22476-1.

A 3.2 Methods for the Evaluation of Soil Liquefaction

Step 1:

Calculation of Cyclic Stress Ratio (CSR):

$$CSR = 0{,}65 \ \frac{\tau_{\max}}{\sigma'_v} \tag{A-7}$$

where $\tau_{\max}$ is the maximum shear stress induced by the seismic loading and σ_v' is the vertical effective overburden stress at depth z (stress due to the buoyant weight of the soil). $\tau_{\max}$ is calculated from the following equation:

$$\tau_{\max} = \frac{a_{\max}}{g} \cdot r_d \cdot \sigma_v \tag{A-8}$$

where $a_{\max}$ is the relevant maximum horizontal ground acceleration according to KTA 2201.1, g is the gravitational acceleration, r_d is a depth-dependent stress reduction factor and σ_v is the total overburden stress at depth z (stress from the weight of the water-saturated soil)

The reduction factor r_d can be determined from the following equation (for depths $z \le 30$ m):

$$r_d = exp[\alpha(z) + M_w \beta(z)]$$

$$\alpha(z) = -1{,}01 - 1{,}13 \cdot sin\left(\frac{z}{11{,}73} + 5{,}13\right) \tag{A-9}$$

$$\beta(z) = 0{,}11 + 0{,}12 \cdot sin\left(\frac{z}{11{,}28} + 5{,}14\right)$$

The depth z is to be entered in m and the angle in radians.

The maximum shear stress $\tau_{\max}$ can also be determined from a site-specific calculation of the seismic ground response. Such a calculation is necessary for highly inhomogeneous ground conditions. This calculation can be performed using vertically propagating shear waves.

A layered profile with two consecutive layers with a stiffness ratio greater than 3 can be considered as highly inhomogeneous.

Step 2:

Calculation of Cyclic Resistance Ratio (CRR) from the measured peak pressure resistance q_c of the relevant CPT soundings at the site and application of correction factors, to take into account the moment magnitude M_w, the vertical effective overburden stress σ_v' and the fine grain fraction FC of the soil at the considered depth.

$$CRR_{M_w,\sigma'_v} = CRR_{M_w=7{,}5,\sigma'_v=p_a} \cdot MSF \cdot K_\sigma \tag{A-10}$$

MSF is a magnitude scaling factor, K_σ is a stress correction factor.

The individual components of the above equation are determined as follows.

Normalised tip resistance

$$q_{c1N} = C_N \frac{q_c}{p_a} \tag{A-11}$$

where p_a is the atmospheric pressure and

$$C_N = \left(\frac{p_a}{\sigma'_v}\right)^m \le 1{,}7 \tag{A-12}$$

$$m = 1{,}338 - 0{,}249\left(q_{c1N,cs}\right)^{0{,}264} \tag{A-13}$$

The determination of the exponent m requires an iteration. According to DIN EN ISO 22476-1, $m = 0.5$ may be used. [2].

Normalised tip resistance for clear sands $q_{c1N,cs}$

$$q_{c1N,cs} = q_{c1N} + \Delta q_{c1N} \tag{A-14}$$

$$\Delta q_{c1N} = \left(11{,}9 + \frac{q_{c1N}}{14{,}6}\right) exp\left(1{,}63 - \frac{9{,}7}{FC+2} - \left(\frac{15{,}7}{FC+2}\right)^2\right) \tag{A-15}$$

FC is determined from the grain distribution curve.

***CRR* for magnitude M_w = 7,5 and σ_v' = p_a**

$$CRR_{M_w=7{,}5,\sigma'_v=p_a} = exp\left(\frac{q_{c1N,cs}}{113} + \left(\frac{q_{c1N,cs}}{1000}\right)^2 - \left(\frac{q_{c1N,cs}}{140}\right)^3 + \left(\frac{q_{c1N,cs}}{137}\right)^4 - 2{,}80\right) \tag{A-16}$$

Magnitude-Scaling-Factor

$$MSF = 1 + (MSF_{max} - 1)\left(8{,}64 \ exp\left(-\frac{M_w}{4}\right) - 1{,}325\right) \tag{A-17}$$

$$MSF_{max} = 1{,}09 + \left(\frac{q_{c1N,cs}}{180}\right)^3 \le 2{,}2 \tag{A-18}$$

Correction factor for the effective overburden stress

$$K_\sigma = 1 - \frac{1}{37{,}3 - 8{,}27\left(q_{c1N,cs}\right)^{0{,}264}} ln\left(\frac{\sigma'_v}{p_a}\right) \le 1{,}1 \tag{A-19}$$

$K_\sigma = 1$ may be used.

Step 3:

The proof is fulfilled if:

$$\frac{CRR_{M_w,\sigma'_v}}{CSR} \ge 1{,}25 \tag{A-20}$$

A 4 Literature

[1] IAEA Safety Standards: Geotechnical Aspects of Site Evaluation and Foundations for Nuclear Power Plants. Safety Guide No. NS-G-3.6, 2004, S. 22-26

[2] Vrettos, C. (2018). Geotechnisches Erdbebeningenieurwesen. Grundbau-Taschenbuch, Teil 2: Geotechnische Verfahren, 8. Auflage, Ernst & Sohn, Berlin, 863-914

[3] Boulanger, R.W., Idriss, I.M. (2016). CPT-based liquefaction triggering procedure. Journal of Geotechnical and Geoenviron-mental Engineering 142(2): 04015065

[4] DIN EN 1998-5: 2022-10 - Draft Eurocode 8 - Design of structures for earthquake resistance - Part 5: Geotechnical aspects, foundations, retaining and underground structures; German and English version prEN 1998-5:2022

Appendix B

Regulations Referred to in this Safety Standard

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

AtG		Act on the Peaceful Utilization of Atomic Energy and the Protection against its Hazards (Atomic Energy Act) Atomic Energy Act in the version promulgated on July 15, 1985 (BGBl. I, p. 1565), most recently changed by article 1 of the act dated December 4, 2022 (BGBl. I, p. 2153)
StrlSchG		Act on the Protection against the Harmful Effect of Ionising Radiation (Radiation Protection Act - StrlSchG) Radiation Protection Act of June 27, 2017 (BGBl. I, p. 1966), most recently changed by the promulgation of January 3, 2022 (BGBl. I, p. 15)
StrlSchV		Ordinance on the Protection against the Harmful Effects of Ionising Radiation (Radiation Protection Ordinance - StrlSchV) Radiation Protection Ordinance of November 29, 2018 (BGBl. I, p. 2034, 2036), most recently changed by article 1 of the ordinance dated October, 2021 (BGBl. I p. 4645)
SiAnf	(2022-02)	Safety Requirements for Nuclear Power Plants (SiAnf) of November 22, 2012, amended version of March 3, 2015 (BAnz AT 30.03.2015 B2), most recently changed as promulgated by BMUV on February 25, 2022 (BAnz AT 15.03.2022 B3)
Intepretationen	(2015-03)	Interpretations of the safety requirements for nuclear power plants of November 22, 2012, of November 29, 2013 (BAnz AT 10.12.2013 B4), changed on March 3, 2015 (Banz AT of March 30, 2015 B3)
KTA 2201.1	(2011-11)	Design of Nuclear Power Plants against Seismic Events; Part 1: Principles
DIN 1054	(2021-04)	Subsoil - Verification of the safety of earthworks and foundations - Supplementary rules to DIN EN 1997-1
DIN 4020	(2010-12)	Geotechnical investigations for civil engineering purposes - Supplementary rules to DIN EN 1997-2
DIN EN 1997-1	(2014-03)	Eurocode 7: Geotechnical design - Part 1: General rules; German version EN 1997-1:2004 + AC:2009 + A1:2013
DIN EN 1997-1/NA	(2010-12)	National Annex - Nationally determined parameters - Eurocode 7: Geotechnical design - Part 1: General rules
DIN EN 1997-2	(2010-10)	Eurocode 7: Geotechnical design - Part 2: Ground investigation and testing; German version EN 1997-2:2007 + AC:2010
DIN EN 1997-2/NA	(2010-12)	National Annex - Nationally determined parameters - Eurocode 7: Geotechnical design - Part 2: Ground investigation and testing
DIN EN ISO 22476-1	(2023-04)	Geotechnical investigation and testing - Field testing - Part 1: Electrical cone and piezocone penetration test (ISO 22476-1:2022); German version EN ISO 22476-1:2023