Reaffirmed: 6/95, 6/00

Safety Standards

of the
Nuclear Safety Standards Commission (KTA)

KTA 2201.2 (June 1990)

Design of Nuclear Power Plants Against Seismic Events
Part 2: Subsurface Materials (Soil and Rock)

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

A previous version of this Safety Standard
was issued 11/82

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KTA SAFETY STANDARD

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CONTENTS

Basic Principles 1
1 Scope 1
2 Definitions 1
3 Investigation of the Subsurface Materials 1
4 Dynamic Characteristics of the Subsurface Materials 1
5 Changes of the Subsurface Materials 1
6 Models for the Subsurface Materials 1
7 Interaction between Subsurface Materials and Structure 2
Appendix A Application Methods 3

PLEASE NOTE: Only the original German version of this safety standard represents the joint resolution of the 50-member Nuclear Safety Standards Commission (Kerntechnischer Ausschuss, KTA). The German version was made public in Bundesanzeiger No. 194a on October 14, 2000. Copies may be ordered through the Carl Heymanns Verlag KG, Luxemburger Str. 449, 50939 Koeln (Telefax +49-221-94373-603).

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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, the 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 the damage arising from the construction and operation of the facility (Sec. 7 para. 2 no. 3 Atomic Energy Act), in order to attain the protective goals specified in the Atomic Energy Act and the Radiological Protection Ordinance and further detailed in the “Safety Criteria for Nuclear Power Plants” and the “Guidelines for the Assessment of the Design of Nuclear Power Plants with Pressurized Water Reactor against Incidents pursuant to Sec. 28 para. 3 of the Radiological Protection Ordinance (Incident Guidelines)”.

(2) In order to attain these protective goals, Safety Standard KTA 2201.2 - as part of KTA 2201 entitled “Design of Nuclear Power Plants against Seismic Events” - deals with the determination and application of characteristics of the subsurface materials which have to be taken as a basis when designing a nuclear power plaint against seismic events. KTA 2201 also contains the following parts:

Part 1:	Principles
Part 3:	Design of Building Structures
Part 4:	Requirements to be Met by Methods for the Demonstration of the Aseismic Safety of Mechanical and Electrical Components
Part 5:	Seismic Instrumentation
Part 6:	Post-Seismic Measures

1 Scope

This safety standard applies to nuclear power plants.

2 Definitions

(1) Damping of Subsurface Materials

The damping of subsurface materials is equal to the energy which the soil withdraws from the oscillating system consisting of subsurface, materials and structure. It is made up of radiation damping and material damping.

(2) Soil liquefaction

Soil liquefaction is the reduction of the shear strength of a soil because of an increase in pore pressure as a result of a compression of the grain structure under dynamic loads.

3 Investigation of the Subsurface Materials

(1) The documents used to evaluate the conditions of the subsurface materials at the site shall Include in particular expert analyses concerning geology, seismology and subsurface materials.

(2) The results of the investigations of subsurface materials and the expert analyses shall be laid down in a generalized soil profile which should include not only the characteristics of the layers but also data concerning the groundwater level.

(3) The following characteristics of the respective soil layers should be quoted:

a) depth and thickness (m)
b) geological designation
c) soil classification
d) weight density of the moist soil (kN/m³)
e) coefficient of stiffness (kN/m²)
f) angle of internal friction (degree)
g) cohesion (kN/m²)

4 Dynamic Characteristics of the Subsurface Materials

(1) For the design of nuclear power plants against seismic impacts, Poisson's ratio, shear modulus and material damping shall be determined as dynamic characteristics of the subsurface materials. For this purpose, upper and lower limits shall be quoted as a function of the depth and of the stress condition of the soil when subjected to the structural load.

(2) The methods for the determination of the dynamic characteristics of the subsurface materials shall be selected as a function of the conditions of the subsurface materials.

In principle, both in-situ and laboratory tests shall be carried out. As an exception to this rule, the procedures referred to in paras. (3) and (4) may be used.

> *Note:*
> *Methods for the determination of dynamic characteristics of subsurface materials are contained as examples in Section A 1 of Appendix A.*

(3) If the subsurface materials and the geological boundary conditions of two sites are comparable, the dynamic characteristics of the subsurface materials of the one site may be assumed to apply to the other as well.

(4) The dynamic characteristics of the subsurface materials may be estimated for nuclear power plants at sites for which the maximum accelerations of the design basis earthquake were determined to be below 1.0 m/s².

(5) The shear modulus and the material damping should be determined as a function of the shear deformation and the stress condition in the soil.

> *Note:*
> *A method for the determination of shear modulus and material dampings, either on the basis of in-situ investigations or by means of auxiliary calculations, is contained as an example in Section A 2 of Appendix A.*

5 Changes of the Subsurface Materials

(1) Possible changes of the subsurface materials such as they may occur as a result of earthquakes shall be determined. These include in particular:

a) permanent deformations as a result of compaction or other changes of the grain structure,
b) reduction of shear strength, either as a result of soil liquefaction or as a result of other changes of the grain structure.

> *Note:*
> *Basic principles for the evaluation of soil liquefaction are contained as examples in Section A 3 of Appendix A.*

(2) For nuclear power plants at sites for which the maximum accelerations of the reference earthquake were determined to be below 1.0 m/s², or where the subsurface materials consists of stiff and geologically preloaded clays or equivalent cohesive soils, no demonstration with respect to soil liquefaction is required.

6 Models for the Subsurface Materials

(1) Within the system consisting of structure and subsurface materials, the representation of the subsurface materials may be effected as a spring-mass, fine-element, finite-difference or boundary-element model or any combination of these models.

(2) If the generalized soil profile can be represented by a single layer (uniform subsurface material), a damped spring-mass model may be used. The parameters of this model may be determined on the basis of the theory of the elastic half-space. In a simplified approach, they may be assumed to be Independent of frequency, provided an adequate parameter variation is effected.

Note:
The representation of the subsurface materials is dealt with in further detail in KTA 2201.3 (being prepared).

7 Interaction between Subsurface Materials and Structure

(1) The influences of dynamic characteristics of subsurface materials, in particular those of shear modulus and material damping, on the oscillatory response of the structure shall be included in the calculations by the assumption of a variation range of these characteristics.

(2) The variation range of the characteristics of the oscillatory system consisting of subsurface material and structure shall be covered when calculating the plant components.

Appendix A

Application Methods

Contents:

A 1 Investigation methods for the determination of dynamic characteristics of subsurface materials 3
A 2 Determination of shear modulus and material damping on the basis of in-situ investigations or auxiliary calculation's 4
A 3 Basic principles for the evaluation of soil liquefaction 4
A 4 References 5

A 1 Investigation methods for the determination of dynamic characteristics of subsurface materials

Method	Measuring Method	a) Measured Quantity b) Derived Quantities	[1] Shear Deformation Range
In-situ methods			
Uphole method	Excitation in the borehole, measurement at the surface	a) Travel times (P and S wave velocities) b) Shear modulus, Poisson's ratio	approx. 10^{-7} to 10^{-5}
Downhole method	Excitation at the surface, measurements in the borehole		
Through transmission method	Excitation in a borehole, measurement in one or more adjacent boreholes		
Vibroseis method	Continuous excitation and measurement at the surface	a) Travel times (surface wave velocities) b) Shear modulus	approx. 10^{-7} to 10^{-5}
Laboratory methods			
Resonance test	Determination of the velocities with variable frequencies and variable amplitudes	a) Frequency, wave length (P and S wave velocities) b) Shear modulus, Poisson's ratio, material damping	approx. 10^{-6} to 10^{-4}
Triaxial dynamic test	Measurement under vertical and tangential loads with variable stress conditions	a) Movements, stresses (stress-shear deformation characteristic) b) Shear rnodulus, material damping, Poisson's ratio	approx. 10^{-4} to 5×10^{-2}
Simple dynamic shear test	Measurement with simple shear under uni-axial load and impeded lateral expansion		
Dynamic torsion test	Measurement under tangential load and uni-axial		10^{-5} to 5×10^{-2}

[1] In the case of earthquakes in the Federal Republic of experience so far shows that shear deformations are in the shear deformation range from 10^{-5} to $10^{-\S}$

A 2 Determination of shear modulus and material damping on the basis of in-situ investigations or auxiliary calculation's

The determination of shear modulus (G) and material damping (D) may be effected along the lines of [1] in accordance with equation A 1 and equation A 2 as shown in **Fig. A 1**.

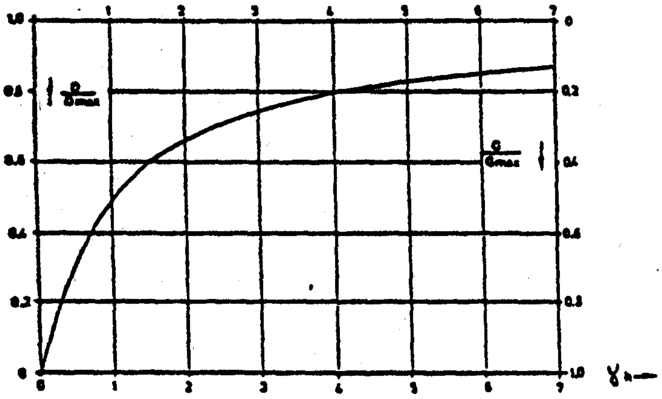

Fig. A 1: Relationship between shear modulus, material damping and hyperbolic shear deformation

$$G_{max} = \frac{1}{1+\gamma_h} \; G_{max} \qquad \text{(A 1)}$$

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

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

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

γ_h = hyperbolic shear deformation

G_{max} = shear modulus for smallest shear deformations

D_{max} = material damping for largest shear deformations

γ = shear deformation

γ_r = reference shear deformation

$\max\tau$ = maximum shear stress

The maximum material damping D_{max} and the quantities a and b can be determined for different types of soil with the aid of the reference equations quoted in [1]. The major influencing quantities include number and frequency of the stress cycles as well as the stress condition prevailing in the soil.

In general, G_{max} shall be determined by in-situ measurements. For estimation purposes, empirically derived approximate equations may be used such as they are found in literature, for example [1], and take the following form:

$$G_{max} = \alpha \cdot \frac{(\beta - e)^2}{1+e} \cdot \sigma'^{\delta}_{m} \cdot (OCR)^K \qquad \text{(A 5)}$$

e = pore ratio of the soil

σ'_m = mean effective principal stress in the soil

δ = exponent, as a rule 0.5

α,β = quantities depending on grain shape, grain size distribution and degree of saturation

OCR = degree of over-consolidation

K = exponent depending on the plasticity index of the soil

A 3 Basic principles for the evaluation of soil liquefaction

A 3.1 Liquefaction potential

Basically, uniform and fine sands exhibit a greater tendency towards soil liquefaction than non-uniform and coarse sands. The decisive influence is excercised by the compactness of the material. The tendency towards liquefaction increases with the degree of looseness. With all other conditions being identical, the tendency towards liquefaction will decrease with an increase in effective stresses in the soil.

In the case of high-lying groundwater levels, the danger of liquefaction is greater than in the case of deep-lying groundwater levels. The danger of liquefaction increases with the intensity and duration of an earthquake.

In this context, the permeability of the sand and the drainage conditions shall also be taken into consideration. The thinner the endangered layers and the faster they can drain into permeable adjacent layers, the shorter is the time during which the sand remains in the liquid state and the less persistent are the consequences.

Geologically preloaded, stiff clays and similar cohesive soils are insensitive to vibrations. They do not exhibit any tendency towards liquefaction.

Soils whose grain size ranges between middle silt and coarse sand are susceptible to liquefaction. This applies in particular to fine sands. As far as gravels are concerned, liquefaction is, in general, only a very short-term phenomenon so that no damaging shear deformations can occur. The duration of liquefaction depends on the conditions of drainage.

In stratified soils the liquefaction process, starting out from an easily liquefiable layer, may spread to soil areas which would not be endangered in normal conditions. Therefore, the danger of liquefaction shall be evaluated on the basis of the most unfavorable layer.

The following shall be effected for the evaluation of the danger of soil liquefaction:

(a) boreholes under and beside the planned structure down to a depth of 25 m below the surface of the terrain,

(b) drop penetration tests or pressure determination by ultrasonic means,

(c) determination of the highest ground water level,

(d) dynamic shear tests, if necessary.

A 3.2 Methods for the estimation of the possibility of soil liquefaction (along the lines of [2])

Step 1:

A grain size distribution curve of the soil to be investigated shall be plotted in a diagram in accordance with **Fig. A 2**.

If the grain size distribution curve is outside Zones 1 and 2, liquefaction need not be assumed.

if the principal portion of the grain size distribution curve is within Zone 1, limit line Z_1 in **Fig. A 3** is decisive for the further examination.

If the principal portion of the grain size distribution curve is within Zone 2, limit line Z_2 in **Fig. A 3** is decisive for the further examination.

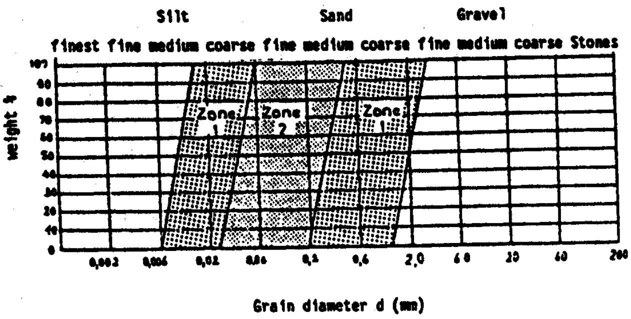

Fig. A 2: Grain size distribution areas susceptible to liquefaction

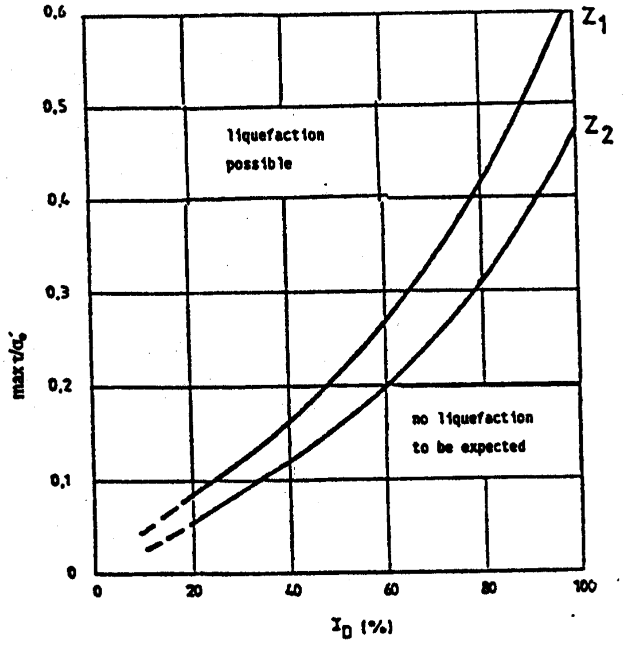

Fig. A 3: Diagram for estimating the possibility of soil liquefaction

Step 2:

The dynamic shear stress ratio τ/σ'_o shall be calculated from σ_o' and the relation

$$\max \tau = \sigma_o \frac{\max a}{g} r_d$$

where max a is the maximum acceleration, g the acceleration due to gravity and

σ'_o the effective vertical stress in the soil at depth t (stress resulting from structural load and weight of the soil after deduction of lift at the highest groundwater level)

σ_o the total vertical stress in the soil at depth t (stress resulting from structural load and weight of the water-saturated soil at the highest groundwater level)

r_d the reduction factor as a function of depth in accordance with **Fig. A 4**.

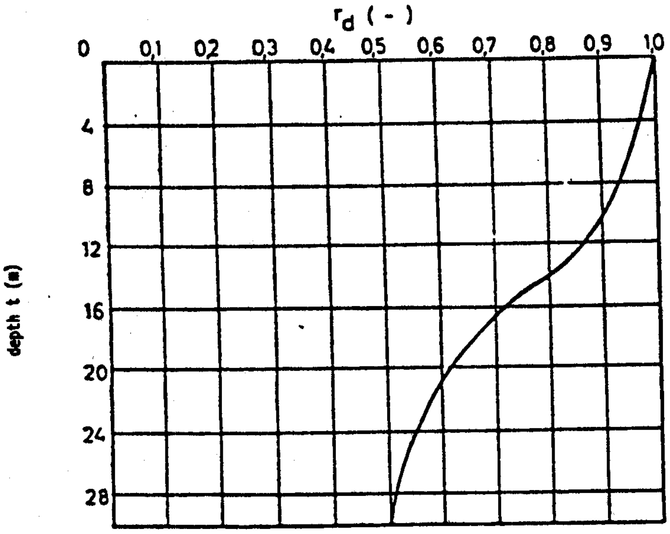

Fig. A 4: Reduction factor r_d as a function of depth t

Step 3:

If the intersection of the shear stress ratio max τ/σ_o' and the relative compactness I_D of the soil is below the relevant limit lines Z_1 and Z_2 plotted in **Fig. A 3**, there is no danger of soil liquefaction.

If the intersection is above the limit line, soil liquefaction cannot be ruled out. In such a case, more detailed investigations are needed.

A 4 References

[1] HARDIN, B.O. and DRNEVICH, V.P.: Shearmodulus and Damping in Soils: Design Equations and Curves. J. Soil Mech. and Found. Div. ASCE, 1972, Vol. 98, SM 7, p. 667-692.

[2] SEED, H.B. and IDRISS, I.M.: Simplified Procedure for Evaluating Soil Liquefaction Potential. J. Soil Mech. and Found. Div. ASCE, 1971, Vol. 97, SM 9, p. 1249-1273.