Safety Standards

of the
Nuclear Safety Standards Commission (KTA)

KTA 2201.2 (2012-11)

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 and 1990-06

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KTA SAFETY STANDARD

November 2012	**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)

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 Atomic Energy Act and the Radiological Protection Ordinance (StrlSchV) and further detailed in the “Safety Criteria” and in the "Design Basis Accident Guidelines".

(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 preventive plant engineering measures against damage and that is relevant with respect to radiological effects on the environment. 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 achieving 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 soil compaction due to the dynamic loading.

(2) Dynamic shear modulus

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

(3) Compression-wave velocity

The compression-wave velocity is the speed of propagation of the compression waves. Compression waves (also called primary waves or pressure waves) are elastic, longitudinally polarized spatial waves which, when passing though a medium, compresses and stretches the particle-filled 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 fluid or gaseous media 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 of a co-extensive 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 number of influencing factors. Essential factors are the shear deformation amplitude and the number of loading cycles of the forces, the omnidirectional mean static pressure under the foundation as well as void ratio and degree of saturation of the soil.

(3) The design of the 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 ratio, ν,
- Material damping in terms of the damping ratio, D,
- Density, ρ,
- Shear wave velocity, v_s, and compression wave velocity, v_p, for small shear deformations.

In this context, the 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 structure load.

Note:

Examples of analytic 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 normally be determined as a function of the shear deformation and stress condition of the soil.

Note:
An analytic 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 be in conformance with the particular subsoil conditions. In-situ procedures and laboratory tests shall basically be applied. As an alternative it is permissible to proceed as specified under para. 6.

(6) The dynamic subsoil properties of one site may be applied at 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:
   Lasting horizontal deformations, e.g., at sites with horizontal soil layers, are usually negligible.

   Examples of the basic principles concerning the evaluation of soil sagging are presented in Section A 3 and of soil liquefaction in Section A 4 of Appendix A.

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

(2) No verification with respect to soil liquefaction is required for nuclear power plant sites for which either the resulting maximum horizontal ground acceleration was determined as being lower than 1.0 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 uni-axial 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 uni-axial loading		

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

Table A 1: Analytic 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 deformation*
- G_0 : *dynamic shear modulus for smallest shear deformations ($\gamma \le 10^{-5}$)*
- D_{max} : *damping ratio for largest shear deformations (material damping)*
- γ : *shear deformation*
- γ_r : *reference shear deformation*
- *max* τ : *maximum shear stress*
- *a, b* : *coefficients resulting from laboratory tests*

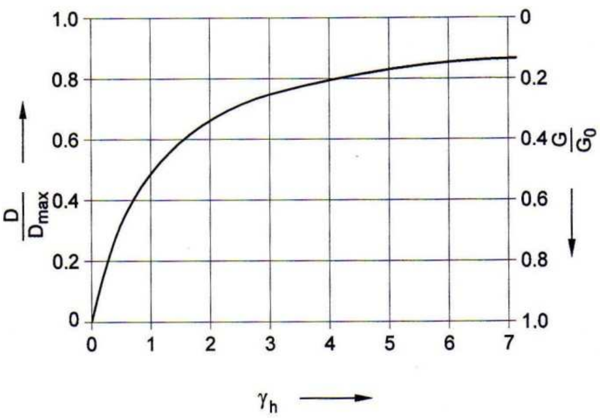

Figure A-1: Relationship between dynamic shear modulus, material damping and hyperbolic shear deformation

(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 load 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, empirically derived approximation 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* : *pore 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 Seismic Soil Compaction

A 3.1 Compaction Potential

Seismically induced shear deformations can cause a compaction of the grain structure of soils and, thereby, lead to a lasting vertical sagging of the subsoil. The tendency for seismic compaction increases with decreasing soil layer density. In the case of cohesive soils, the compaction potential is also dependent on the degree of saturation, S.

A 3.2 Methods for the Estimation of Vertical Soil Sagging (following the discussion in [2])

Step 1: Site-response analysis

A horizontally layered subsoil model shall be established for the site being analyzed with the chosen number of soil layers being large enough to sufficiently accurately represent the depth dependent shear stress distribution in the compaction sensitive layers as well as the variation of soil type and soil parameters.

By applying site-specific earthquake time histories, the effective shear deformations, γ_{eff}, in the center of each soil layer shall be determined (cf. [2]). It is permissible to apply

$$\gamma_{eff} = 0{,}65 \cdot \gamma_{max} \qquad \text{(A-7)}$$

as a conservative approximation. If the effective shear deformations, γ_{eff}, are lower than the threshold value, γ_S, the seismic compaction of the corresponding layers may be disregarded. The threshold value for sand layers is in the order of γ_S = 0.01 % and for cohesive soil layers between 0.01 % and 0.04 %

Step 2: Equivalent number of similar shear deformation cycles

To enable determining the volumetric strain, ε_V, in Step 3, the transient shear deformation time history due to an earthquake shall be replaced by an equivalent number of sine waves with a constant amplitude equal to γ_{eff}. This equivalent cycle number, *N*, can be approximated (cf. [3]) by Equation A-8 as a function of the moment magnitude of the design basis earthquake, M_W, and the shortest distance, *r* (km), from its epicenter:

$$N = \frac{10^{0.5 M_w}}{70 \cdot \exp\left[\frac{1}{3}(b_1 + b_2(M_w - 5.8))\right]} + c_1 \cdot S_m + c_2 \cdot r \qquad \text{(A-8)}$$

Parameter S_m accounts for the subsoil situation at the site; S_m = 0 for rock with low sediment coverage (less than 20 m) and S_m = 1 for a sediment thickness larger than 20 m. The coefficients b_1, b_2, c_1 and c_2 were determined in [3] as follows:

b_1 = 1.53 ± 0.15,

b_2 = 1.51 ± 0.12,

c_1 = 0.75 ± 0.42,

c_2 = 0.095 ± 0.014.

A graphic representation of Equation A-8 is presented in **Figure A-2**.

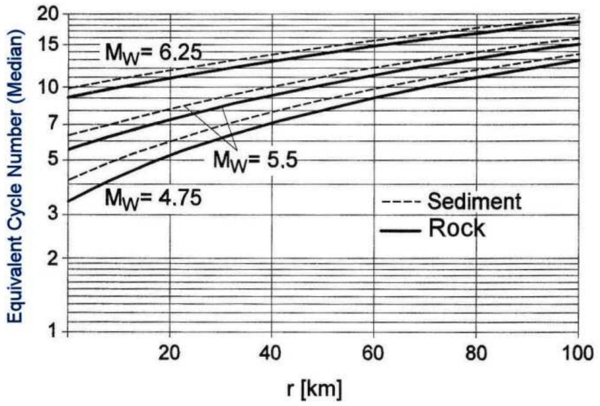

Figure A-2: Equivalent cycle number, N, as a function of the moment magnitude, M_W, and the shortest distance, r (km), from the epicenter

Step 3: Volumetric strain

An estimation of the volumetric compression of the subsoil requires performing laboratory tests (cyclic shear tests) for all compaction sensitive soil layers in order to determine their respective quantitative relationship between volumetric strain, ε_V, and the effective shear-deformation amplitude, γ_{eff}, and equivalent cycle number, N. The recommended material model (cf. [2]) is the following:

$$\begin{aligned} &\gamma_{eff} > \gamma_S: \quad \varepsilon_{V,N=15} = c \cdot (\gamma_{eff} - \gamma_S)^d \\ &\gamma_{eff} \le \gamma_S: \quad \varepsilon_{V,N=15} = 0 \end{aligned} \qquad \text{(A-9)}$$

$$\frac{\varepsilon_{V,N}}{\varepsilon_{V,N=15}} = R \cdot \ln(N/15) + 1 \qquad \text{(A-10)}$$

The term $\varepsilon_{V,N=15}$ in Equation A-9 represents the volumetric strain after 15 uniform cycles and γ_S is the threshold value specified in Step 1. The coefficients c and d shall be determined dependent on the relative compactness, I_D, and – in case of cohesive soils – on the degree of saturation, S, of the respective soil layer. Equation A-10 shall be used for the conversion of the volumetric strain after 15 cycles, $\varepsilon_{V,N=15}$, to other values of N. The coefficient R shall also be determined in laboratory tests.

In the case of sands without fine grains, the material model of **Figure A-3** together with R = 0.33 may be used as approximation.

Step 4: Determination of the overall soil sagging, s

The overall soil sagging, s, may be approximated by the sum of the individual contributions of all compression sensitive layers as follows:

$$s = 2\int_0^z \varepsilon_V \, dz \approx 2\sum_k \varepsilon_{V,k} \cdot h_k \qquad \text{(A-11)}$$

The term $\varepsilon_{V,k}$ is the volumetric strain of layer k (cf. Step 3) and h_k is the thickness of the respective layer. The factor 2 in Equation A-11 accounts for the excitation by the vertical and two horizontal earthquake components (cf. [4]).

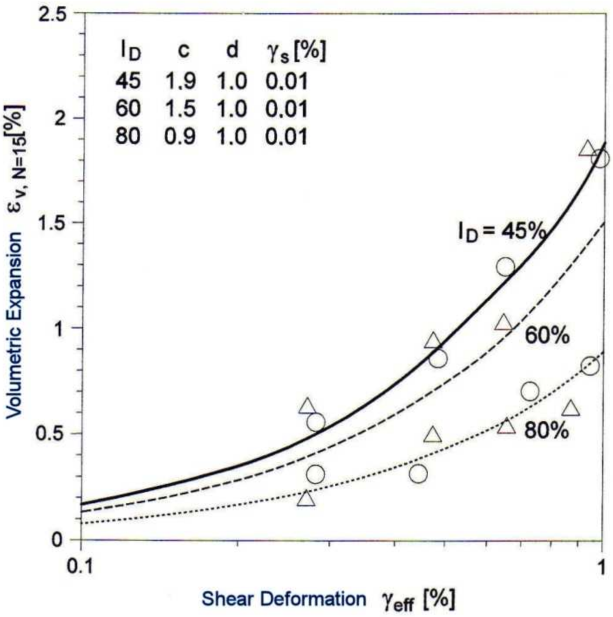

Figure A-3: Material model sand without fine grains – I_D = 45 %, 60 % and 80 % (cf. [2])

A 4 Basic Principles for the Evaluation of Soil Liquefaction

A 4.1 Liquefaction Potential

(1) Uniform and fine-grained sands below the ground water level basically have a greater tendency for soil liquefaction than non-uniform and 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) In the case of a higher groundwater level, the danger of liquefaction is greater than in the case of a deeper groundwater level. The danger of liquefaction is also increased 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 permeable adjacent 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 gravelis a very short-term phenomenon and, therefore, no damaging shear deformations can 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) Grain distribution analyses,

b) Dynamic probing or cone penetration tests,

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

d) Cyclic shear tests if the assessment according to **Figure A-5** shows that soil liquefaction cannot be ruled out.

A 4.2 Methods for Estimating the Possibility of Soil Liquefaction (following the discussion in [5])

Step 1:

A grain-size distribution curve of the soil to be investigated shall be plotted in a diagram as shown in **Figure A-4**.

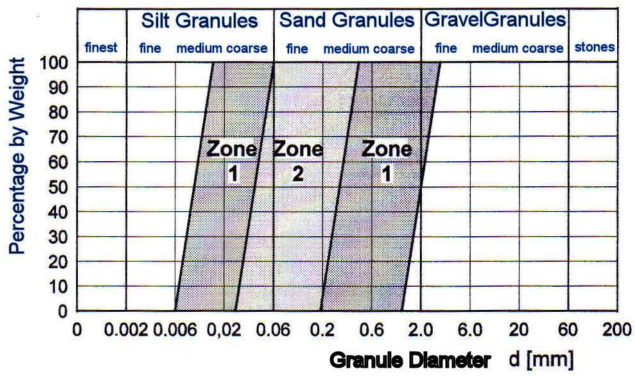

Figure A-4: Grain size distribution zones susceptible to liquefaction

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

If an essential portion of the grain-size distribution curve lies within Zone 1, then limit line Z_1 plotted in **Figure A-5** is decisive regarding further investigations.

If an essential portion of the grain-size distribution curve lies within Zone 2, then limit line Z_2 plotted in **Figure A-5** is decisive regarding further investigations.

Step 2:

The dynamic shear stress ratio, *max* τ/σ'_0, shall be determined. It may be calculated from σ'_0 and the relationship

$$\max \tau = \sigma_o \frac{\max a}{g} r_d \qquad \text{(A-12)}$$

Nomenclature:

max a : *maximum soil acceleration,*

g : *acceleration due to gravity,*

σ'_0 : *effective vertical stress in the soil at a depth, t (stress resulting from the load of the construction and weight of the soil after deduction of uplift for the maximum ground water level with an exceedance probability of* 10^{-2} *per annum)*

σ_0 : *total vertical stress in the soil at a depth, t (stress resulting from the load of the construction and weight of the water saturated soil at the maximum ground water level with an exceedance probability of* 10^{-2} *per annum)*

r_d : *reduction factor as a function of depth, t, as plotted in* ***Figure A-6***.

Step 3:

If the point of intersection of the shear stress ratio, *max* τ/σ'_0, and the relative soil layer compactness, I_D, lies below the decisive limit lines Z_1 and Z_2 plotted in **Figure A-5**, there is no danger of soil liquefaction.

If the intersection is above the decisive limit line, soil liquefaction cannot be ruled out. In this case, more detailed investigations are necessary.

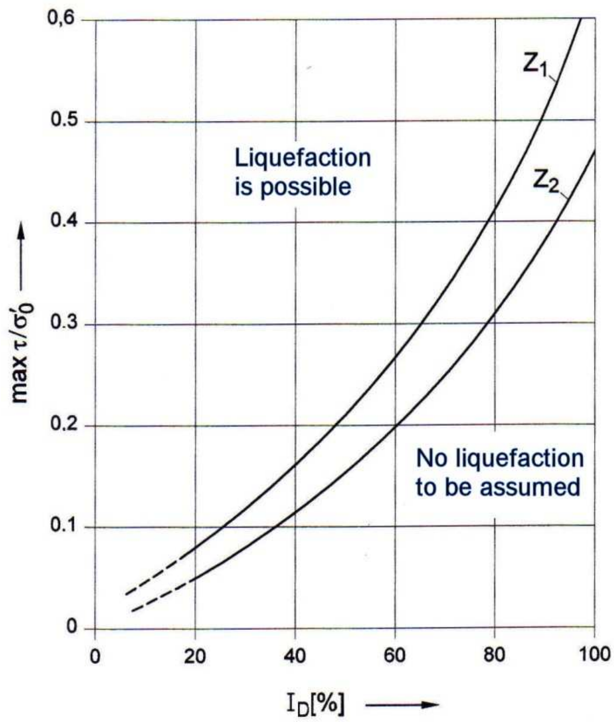

Figure A-5: Diagram for estimating the possibility of soil liquefaction

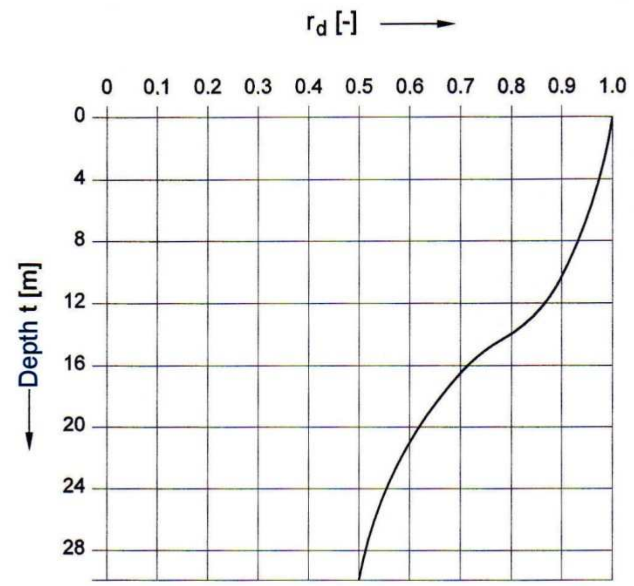

Figure A-6: Reduction factor, r_d, as a function of depth, *t*

A 5 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] STEWART, J. P., WHANG, D.H., MOYNER, M., DUKU, P.: Seismic compression of as-compacted fill soils with variable levels of fines content and fines plasticity. CUREE Publication No. EDA-05, July 2004, www.curee.org

[3] LIU, A.H., STEWART, J.P., ABRAHAMSON, N.A., MORIWAKI, Y.: Equivalent number of uniform stress cycles for soil liquefaction analysis. J. Geot. and Geoenv. Eng., ASCE, 127(12), 2001, 1017-1026

[4] PYKE, R., SEED, H.B., CHAN, C.K.: Settlement of sands under multidirectional shaking. J. Geotech. Eng., ASCE, 101(4), 1975, 379-398

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

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 – 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 February 24, 2012 (BGBl. I, p. 212)
StrlSchV		Ordinance on the protection from damage by ionizing radiation (Radiological Protection Ordinance – StrlSchV) of July 20, 2001 (BGBl. I 2001, p. 1714; 2002, p. 1459), most recently changed by Article 5 of the Ordinance of February 24, 2012 (BGBl. I, p. 212)
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
DIN 1054	(2010-12)	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	(2009-09)	Eurocode 7: Geotechnical design - Part 1: General rules; German version EN 1997-1:2004 + AC:2009
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