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Design of nuclear power plants against seismic events —

Part 3:
Civil structures

Conception parasismique des installations nucléaires —

Partie 3: Ouvrages de génie civil

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Foreword

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A list of all parts in the ISO 4917 series can be found on the ISO website.

Any feedback or questions on this document should be directed to the user's national standards body. A complete listing of these bodies can be found at www.iso.org/members.html.

Introduction

In accordance with IAEA Safety Standards Series No. SSR-2/1 protective measures against seismic events are required, provided earthquakes is taken into consideration.

Earthquakes belong to the group of design basis events that requires taking preventive plant engineering measures against damage and which are relevant with respect to radiological effects on the environment.

Design of nuclear power plants against seismic events —

Part 3:
Civil structures

1 Scope

This document applies to civil structures of nuclear power plants with water cooled reactors in order to achieve the safety objectives given in ISO 4917-1. For other nuclear facilities the applicability of the document needs to be checked in advance, before it might be applied correspondingly.

This document specifies the requirements for civil structures for the verification of their load-bearing capacity 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).

This document will be applied under the presumption that the geology and tectonics of the plant site have been investigated with special emphasis on the existence of active geological faults and lasting geological ground displacements, and that the site has been deemed suitable for a nuclear installation.

To achieve these goals, this document deals with the requirements specific to the seismic design of civil structures above and beyond their conventional design. The basic requirements of these precautionary measures are dealt with in ISO 4917-1.

This document does not apply to cranes, to detachment devices for lifting equipment nor to the supporting and mounting constructions of components.

This document is independent of national standards. Recommendations, given in Annex A, are mainly based on the KTA Design-Philosophy and European standards. Alternatively other equivalent standards or regulations can be used in case the general requirements given in this document can be met.

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

2 Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes requirements of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

ISO 4917-1:2024, *Design of nuclear power plants against seismic events— Part 1: Principles*

ISO 4917-4:2024, *Design of nuclear power plants against seismic events — Part 4: Components*

IAEA Safety Standards Series No, SSG-67, *Seismic Design for Nuclear Installations*, INTERNATIONAL ATOMIC ENERGY AGENCY VIENNA (2021)

3 Terms and definitions

For the purposes of this document, the following terms and definitions apply.

ISO and IEC maintain terminology databases for use in standardization at the following addresses:

— ISO Online browsing platform: available at https://www.iso.org/obp

— IEC Electropedia: available at https://www.electropedia.org/

3.1
structural member damping
damping of a structural member (e.g. plate, beam) including bordering influences and effects from secondary elements (screeds, floor coverings, etc.)

Note 1 to entry: For equal load levels, structural element damping is larger than material damping.

3.2
geometric non-linearity
non-linear relationship between force and path values caused by equilibrium and kinematic considerations for the deformed system

3.3
homogenous soil
rock
soil or rock with a near constant shear-wave velocity throughout its stratum thickness and a ratio of a stratum thickness-to-foundation-radius larger than 4

Note 1 to entry: In case of squared foundations, the radius of an area equivalent circle applies.

3.4
soil-structure interaction
interaction relationship between local soil or rock conditions and the vibration behaviour of the building via its foundation

Note 1 to entry: This interaction comprises the kinematic interaction and the interaction due to the inertial forces of the structure.

3.5
impedance functions
complex frequency dependent foundation stiffness values in the subsoil; their real and imaginary parts characterize stiffness and damping

3.6
interaction
<kinematic> interaction of the foundation with the subsoil, whereby the foundation is assumed as being massless and rigid

3.7
material damping
damping caused by internal micro-plastic deformations

Note 1 to entry: The material damping is measured in the laboratory excluding bordering influences and the like; it is dependent on the load level.

3.8
material non-linearity
non-linear relationship between stresses and strains caused by a non-linear material behaviour

3.9
radiation damping
damping due to energy being radiated into an adjacent medium, e.g. from one structural member to a bordering member or from foundation to the subsoil

3.10
Rayleigh damping
damping defined by a matrix, ***C***, that is a linear combination of the mass matrix, ***M***, and the stiffness matrix, ***K***

3.11
overall building damping
damping of the total building or partial building structures made of many structural and nonstructural members, including the influence from, e.g. non-load-bearing components, interaction with equipment, friction in connections and effects from energy dissipation

Note 1 to entry: For equal load levels, overall damping is significantly higher than structural member damping.

3.12
upper limit frequency
frequency above which no significant seismic response in structures and sub-systems would occur

3.13
load bearing capacity
resistance of a structure or a structural component for forces that produce stresses or deformations

3.14
building response spectrum
<in-structure response spectrum> response spectrum at a specific point or level of the building structure (it corresponds to floor response spectrum)

4 Seismic event

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 and the strong motion duration specified in ISO 4917-1:2024, 5.5, or

b) artificial acceleration time histories compatible with the ground response spectra specified in ISO 4917-1:2024, 6.3.3, or

c) recorded acceleration time histories as specified in ISO 4917-1:2024, 5.5.

The excitation to be applied and superposed shall be as specified in ISO 4917-1:2024, 6.3.1.

5 Structure analysis

5.1 Basic requirements

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

The range of variation 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 value considerations.

5.2 Modeling

5.2.1 General

The basic requirements regarding modeling are specified in ISO 4917-1:2024, 6.3.2. Regarding modeling of civil structures, the requirements under this clause shall additionally be complied with.

Provided all decisive influences from torsion and eccentricities between the centers of gravity and the centers of stiffness are taken into account as given in ISO 4917-1:2024, 6.3.2, any torsions coming from unplanned scattering of masses and their distribution on a floor may be neglected.

NOTE In the case of decisive torsional effects coming from the excitation due to non-vertical propagating waves, incoherent waves or phase-lag between waves an approximation with an accidental torsion approach is possible (additional eccentricity between center of mass and stiffness).

Any modeling by equivalent beams subjected to axial and torsional loads shall take into account the influence from torsion and eccentricities between the centers of gravity and the centers of stiffness. Furthermore, existing deformabilities of individual structural members not accounted for in the beam model that would, however, have a relevant influence on their structural behaviour or that of connected components shall be taken into account in verifying the earthquake safety of structural members and components.

5.2.2 Geotechnical parameters, dynamic subsoil properties

In the dynamic analysis of a building the influence of the subsoil and foundation on the vibration behaviour shall be considered. The mechanical properties of the subsoil under dynamic loading shall be taken into account.

NOTE The mechanical properties of the subsoil under dynamic loading are significantly different from those under static loading. The main influencing factors are the shear strain amplitude and the number of loading cycles, the omnidirectional mean static pressure under the foundation as well as the void ratio and degree of saturation of the soil.

The design of nuclear power plants against seismic events shall be based on geotechnical assessments and investigations. The procedures chosen to be applied for determining the dynamic subsoil properties shall be in accordance with the specific subsoil conditions. In-situ procedures and laboratory tests should be applied. The whole process of assessment and determination of subsoil properties is not covered by this code. More detailed requirements may be found e.g. in IAEA NS-G 3.6.C, KTA 2201.2.

Possible changes of the subsoil that might occur as a result of earthquakes should be adressed. These are mainly the permanent vertical deformations resulting from soil compaction and reduction of the shear strength due to changes in the soil grain structure.

The following data on dynamic subsoil properties for the individual soil layers should be defined as the basis of a dynamic analysis:

— dynamic shear modulus, G_0, supplemented by appropriate upper and lower limit values;

— Poisson ratio, ν;

— material damping in terms of the damping ratio, D;

— mass density, ρ;

— shear wave velocity, v_s, and compression wave velocity, v_p, for small shear strains (if Poisson's ratio is given, v_p is not needed);

— relationships for the dynamic shear modulus reduction and the material damping to shear strains.

5.2.3 Material characteristics (parameters)

Regarding concrete, reinforcing steel, pre-stressing steel, structural steel and masonry, the material characteristics (parameters) to be used in the analysis model for static loads may be as specified in the relevant national documents.

NOTE If no national standard is available for a sufficient definition of material parameters information can be found in the European standards EN 1992-1-1, EN 1993-1-1, EN 1994-1-1 and EN 1996-1-1.

5.2.4 Effective stiffness

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

A possible stiffness reduction due to cracking shall be taken into account for reinforced or pre-stressed concrete structural members, if this may have a significantly adversely effect on the vibration behaviour 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 masonry, shall be considered in the building models, provided, this has a significantly adversely effect on the vibration behaviour.

NOTE 1 In particular the torsional stiffness of reinforced concrete structural members can be reduced due to cracking and can strongly influence the design results.

NOTE 2 The contribution of non-load bearing elements can lead to an additional eccentricity between the centre of mass and stiffness. If their contribution does not vanish due to e.g. cracking it can decisively impair the overall dynamic behaviour of the building.

Stiffness values and eccentricities of connections in steel structures and their supports shall be taken into account, if they may have a significantly adversely effect on the vibration behaviour 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, for example, framework corners or bolt connections.

5.2.5 Contributing masses

The building structure analysis shall account for the seismic masses derived from the permanent masses (building structure and components), the masses of quasi-permanent live loads and the ratio of variable live loads existing during operation. Recommended ratios of variable live loads that should be considered are given in Table A.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 and the uncoupling criteria according to ISO 4917-1:2024, A.5 are satisfied.

In stick models the rotary inertias shall explicitly be considered.

NOTE In 3D models the rotary inertia is generally implicitly considered.

5.2.6 Damping

Damping may be assumed to be a viscous (velocity dependent) effect. It should be distinguished between the dynamic analyses of buildings (associated with "overall building damping") and of individual structural members (associated with "structural member damping").

NOTE 1 The overall building damping includes besides the material damping and the damping of structural members in particular the damping due to interactions with non-load-bearing components and equipment, too.

In the dynamic analysis, the different damping of the building structure and of the subsoil shall be considered.

The damping behaviour of buildings and partial building structures is in general largely determined by the overall building damping. Therefore, the analyses for verifying the ultimate limit state (ULS) and the serviceability limit state (SLS) and for determining building response spectra may be based on high damping ratios. Recommended values are given in column A of Table A.1.

For-buildings whose damping behaviour is exclusively determined by material damping and structural member damping, the verification of the serviceability limit state (SLS) and the determination of the building response spectra shall be based on reduced damping ratios without the effect of overall building-damping. Recommended values are given in column B of Table A.1.

In cases where it is proven that significant energy absorbing effects like e.g. concrete cracking and steel yielding will be expected all over the building, column A of Table A.1 may be applied for the verification of the serviceability limit state (SLS) and the generation of response spectra, too.

NOTE 1 In the verification of the ultimate limit state (ULS) it is justified to assume high energy dissipation due to e.g. cracking and steel yielding. This is because when damping is overestimated due to small utilization of the structure the real acting loads and hence the utilization automatically increases (self-regulating system). This is not the case in the verification of the serviceability limit state (SLS) and the generation of response spectra.

For the verification of the load-bearing capacity of individual structural members damping ratios as specified in column A of Table A.1 may be applied. In case of additional requirements, reduced damping ratios shall be applied. Recommended values are specified in column B of Table A.1. The use of differing values shall be well substantiated.

NOTE 2 Additional requirements can be serviceability limit state (SLS), leak tightness requirements or crackwidth limitation requirements for fastenings.

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

For oscillations of fluids, only a small damping shall be assumed. Annex A contains a recommended value of damping ratio for oscillations of fluids.

5.2.7 Hydrodynamic effects

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 structural elements shall be considered separately.

Oscillations of fluids relative to structural elements 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 structure and one mass swaying relative to the structure.

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.

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

The horizontally effective liquid masses may be considered as being constant over the entire height of the structure.

In the vertical direction, the liquid mass shall be considered as being rigidly coupled to the base of the structure.

5.3 Analysis methods

5.3.1 General requirements

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

a) response spectrum method specified in 5.3.2;

b) time history method specified in 5.3.3;

c) frequency domain method specified in 5.3.4;

d) simplified method specified in 5.3.5.

The ultimate limit state (ULS) shall basically be verified taking geometric non-linearities into account. This requirement may be waived, provided, the requirement in A.2 is met.

The load of structural members whose vibration behaviour does not influence the overall behaviour may be determined based on building response spectra (see 5.5).

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.

5.3.2 Response spectrum method

Based on calculated eigen- (or natural) frequencies and eigenforms, the response spectrum method shall be used to determine the maximum action effects from the superposition of the contributions of the individual eigenmodes (modal analysis). If a temporal sequence of the response is required, a time history method shall be applied (see 5.3.3).

NOTE The multiple spectrum method can be used, too. This is defined in 4.3.5 b) of ASCE 4 or in ASME Boiler and Pressure Vessel Code (BVPC).

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

Basically, eigenfrequencies up to the upper limit frequency shall be considered. The eigenmodes with higher frequencies shall be applied as rigid body contribution as given in ISO 4917-4:2024, 5.4.2.

Basically, all modal response parameters shall be combined by the Complete Quadratic Combination (CQC) or Gupta or Lindley-Yow methods. Provided, all eigenforms meet Formula (1) the modal response parameters may be combined using the square root of sum of squares (SSRS) method.

$$f_i > 1{,}35 f_{i-1} \tag{1}$$

where

f_i is the eigenfrequency of the eigenform i;

i is 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 as given in ISO 4917-4:2024, 5.4.2.

The factor 1,35 in Formula (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 %.

Application of the classical 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 can be used for decoupling the equation system. This is the case when damping is distributed all over the system. Systems with local concentrated damping e.g. systems with seismic dampers (non-proportional dampened systems) require specific considerations e.g. coupling of several eigenmodes together.

5.3.3 Time history method

5.3.3.1 General requirements

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.

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 can be found in ISO 4917-1.

In well substantiated cases the modal analysis may be used as a simplified method for non-linear problems.

5.3.3.2 Modal time history method

In case of extreme damping, the solution for the undampened system may only be applied if this is well substantiated. The maximum modal damping of oscillations coming from soil-structure interaction shall be limited if not well substantiated for the individual case. Recommended values are given for the different degrees of freedom in A.3.

It is recommended to limit the modal damping of the building structures. Recommended values are specified in Table A.1 (see also 5.2.5) for the significant frequencies.

Modern non-classical modal time history methods use a complete modal damping matrix or count damping forces in a right-hand part of an equation system. These methods have no limitations in modal damping values.

In analogy to the response spectrum method (see 5.3.2), the rigid-body contribution shall be accounted for.

5.3.3.3 Direct integration

The time increment for the direct integration shall be sufficiently small 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. It is recommended to set the time step to no more than 0,1 times the reciprocal of the upper limit frequency. When a time step of more than 0,1 times the reciprocal of the upper limit frequency is used, it should be well substantiated.

NOTE 1 This value is based on the Newmark integration scheme but it is also a good reference value for other integration schemes.

NOTE 2 The time increment is sufficiently low if the results with 2 times lower time increment changes the results less than 10 %.

If the damping ratios are approximated by proportional damping (Rayleigh damping), two support points in the Rayleigh damping diagram are needed for the calculation of α and β. They shall be chosen appropriately so that the damping in the relevant frequency range is equal or less than the target damping ratio.

The Rayleigh damping of frequency-independent soil models shall be limited. Recommended values are given for the different degrees of freedom in A.3.

It is recommended to limit the Rayleigh damping of the building structures. Recommended values are specified in Table A.1 (see also 5.2.5) for the significant frequencies.

The Rayleigh damping defined by a matrix, $\boldsymbol{C}$, that is a linear combination of the mass matrix, $\boldsymbol{M}$, and the stiffness matrix, $\boldsymbol{K}$, using the Rayleigh parameters α and β according to Formula (2):

$$\boldsymbol{C} = \alpha \cdot \boldsymbol{M} + \beta \cdot \boldsymbol{K} \tag{2}$$

Formula (2) leads to the respective damping ratio, D, in Formula (3), as a function of the angular frequency, ω.

$$D = \alpha / (2 \cdot \omega) + (\beta \cdot \omega) / 2 \tag{3}$$

In the case of nonlinear analyses with hysteresis effects, the viscous damping ratios should be further limited because one portion of energy dissipation is considered in the model already. Recommended values are specified in columns B of Table A.1. However, any alternate damping values can be used with proper justification.

For structures with significant rigid body motion (for example due to sliding or isolation system), the α coefficient of the Rayleigh damping should be taken equal to 0 or procedures shall be set in place to avoid that the Rayleigh damping contribute to artificially damping these modes.

5.3.4 Frequency domain method

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.

The sensitivity of the results to different frequency discretization shall be examined basically using transfer functions analysis. The additional frequency support points required for applying the Fast-Fourier-Transformation (FFT) may be interpolated.

5.3.5 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 Simplified methods (quasi-static methods) for structures are detailed under consideration of the application limits, e.g. in ISO 4917-4:2024, 5.4.4.

5.4 Soil-structure interaction

The influence of the interaction between the building and the subsoil on the dynamic behaviour 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.

It is permissible to assume a rigid foundation for the building, when it is well provided, that the subsoil has no relevant influence on the dynamic behaviour of the building. This may be assumed if a rigid foundation assumption changes the results less than 10 % compared to a model with soil-structure-interaction.

In general, the subsoil is approximately stratified which makes specific analyses methods necessary, e.g. analysis methods in the frequency domain that consider the frequency dependent stiffness characteristics of the foundation (impedance functions). The same applies to special foundation concepts (e.g. pile foundations).

It is permissible to recalculate the ground response spectra specified as the seismic event to other subsoil horizons by deconvolution of seismic motion.

NOTE 1 It is important, that the seismological report clearly defines the soil layer or level of foundation that corresponds to the given design response spectrum.

A homogenous subsoil 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 behaviour may be accounted for by limit state considerations with adapted soil or rock characteristics (secant stiffness values). In the case of rigid circular or rectangular foundations with a dimensionless frequency, a_0, 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.

The dimensionless frequency, a_0, is defined by Formula (4) as follows:

$$a_0 = (\omega \cdot r_0) / v_s \tag{4}$$

where

ω is the angular frequency;

r_0 is the 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 is the shear-wave velocity of a half space model that represents the real soil conditions. The value shall be estimated on engineering judgement.

NOTE 2 The static rigidity corresponds to the rigidity at the frequency above zero. The damping determined from half-space approximation equations corresponds to the damping at resonance frequencies.

The possibility of a mutual subsoil related interference of neighbouring buildings (structure-soil-structure interaction) shall be evaluated.

NOTE 3 This interference is generally covered by varying the average value of the subsoil stiffness upward by multiplying it by a variation factor and downward by dividing it by a variation factor as specified in ISO 4917-1:2024, 6.3.2.

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

5.5 Building response spectra

5.5.1 General requirements

The building response spectra (secondary spectra) shall be determined based, e.g. on the acceleration time histories specified under 5.5.2 or on the substitution method specified in Annex B.

The tertiary spectra of structural elements (e.g. steel platforms) may be determined in accordance with ISO 4917-4:2024, 5.2.3.

5.5.2 Determining building response spectra based on acceleration time histories

The acceleration time histories shall be determined by applying the time history method in accordance with 5.3.3 or the frequency domain method in accordance with 5.3.4.

Response acceleration time histories shall be sampled with a time step of $1/(10 \times F_c)$ or less, where F_c is the upper limit frequency in Hertz.

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

If in-structure response spectra are calculated for the entire floor at some elevation in the building, the number of nodes where spectra are evaluated, may vary but should be enough to provide conservative results. Nodes where relevant equipment is attached to the building structure should be included to the list. The spectra from all nodes should be enveloped.

NOTE 1 Conservatism is decreased if spectra are calculated for separate rooms or exact locations in the model where equipment is fastened.

NOTE 2 The incoherency of seismic motion can generally be neglected for buildings with mat foundations. For large buildings with independent foundations national regulations can be applied.

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

a) A number of particular response spectra are calculated from respective response acceleration time histories. Response spectra shall be calculated for desired frequency range and damping value. The frequency increment in each octave may normally be no larger than 1/50 of the lower limit of the octave, however, may no lower than 0,02 Hz. However, it shall be ensured that SSI frequencies are also considered while deciding the frequency steps while deriving the response spectra.

b) The average, soft and hard subsoil parameters should be defined in accordance with ISO 4917-1:2024, 6.3.2. Resulting spectra from different subsoil parameters should be enveloped.

c) Averaging the results from the different acceleration time histories in accordance with ISO 4917-1:2024, 6.3.3.

NOTE For linear analysis it is recommended to use three statistically independent acceleration time histories according to ISO 4917-1:2024, A.6. For non-linear analysis detailed information is given in ISO 4917-1:2024, 6.3.3.

d) Clipping of spectrum tips. Only spectrum tips that are not wider than 10 % of the respective center frequency should be clipped.

e) Enveloping the spectra, using the following approaches:

   1) Enveloping the subsoil with medium stiffness values by a 15 %-widening of the maxima of the spectra.

   2) Enveloping the soft, widened medium and stiff subsoil in accordance with ISO 4917-1:2024, 6.3.2.

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

f) Smoothing of the resulting response spectra by applying simplified polygon contours 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 the respective center frequency are surrounded by a plateau originating from the lower peak.

g) Calculating the design response spectra for the required damping ratios. The design response spectra should be calculated for different damping ratios, D. Recommended values are 0,5 %, 1 %, 2 %, 4 %, 7 %, 15 % and 20 % 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.

6 Seismic design verification concept

6.1 General requirements

Civil structures shall be assigned to seismic category 1, 2 or 3 and correspondingly designed in accordance with ISO 4917-1:2024, 6.1.

6.2 Combination of actions

Influential actions (effects) shall be classified as follows:

a) Permanent actions, G_k;

b) Pre-stressing actions, P_k;

c) Variable actions, $Q_{k,i}$;

d) Actions from the design-basis earthquake, A_{ed}, specified in ISO 4917-1.

The permanent actions, the pre-stressing actions and the variable actions are specified as characteristic parameters. Unusual effects from seismic events are specified as design values such that a partial safety factor of 1,0 can be implicitly presumed.

The design value of a seismic event, A_{ed}, is determined under consideration of all vertical loads from the combinations contained in Formula (5):

$$\sum_{j=1}^{m} G_{k,j} \oplus \sum_{i=1}^{n} \psi_{Ei} \cdot Q_{k,i} \tag{5}$$

where

$\oplus$ indicates "shall be combined with";

$\sum$ indicates "combined effect of";

$G_{k,j}$ is the characteristic value of the permanent actions, j;

$Q_{k,i}$ is the characteristic value of the variable actions i, causing inertial forces;

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

The design values for the seismic loading is determined from the ultimate limit states (ULS) and the serviceability limit states (SLS) of the combination of actions specified by Formula (6) taking the combination factor, Ψ_2, into account.

$$E_d = E\left\{G_K \oplus P_K \oplus A_{Ed} \oplus \sum_{i=1}^{n} \left(\psi_{2,i} \cdot Q_{k,i}\right)\right\} \tag{6}$$

Recommended combination combination factors, Ψ_E and Ψ_2, in Formula (5) and Formula (6) are specified in Table A.2.

6.3 Combinations of loads caused by directional components of a seismic event

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

Alternatively, the action effects may be determined individually for each of the three directions of the seismic event. In this case, the maximum value for each action effects of the building may be determined as specified in ISO 4917-1:2024, 6.3.1 from the individual components of the seismic event.

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

The loads resulting from the combination of actions specified in the previous three paragraphs. 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 action effects is achieved.

6.4 Ultimate limit state (ULS)

6.4.1 General requirements

It shall be verified that, at the ultimate limit state, Formula (7) is fulfilled.

$$E_d \leq R_d \tag{7}$$

where

E_d is the design value calculated from Formula (6) for the load (e.g. stress resultant) during the seismic event;

R_d is the 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.

R_d is defined in the Formula (8) as follows

$$R_d = R\{f_{k,i} / \gamma_{M,i}\} \quad (8)$$

where

f_k the characteristic strength value of the structural material;

γ_M the corresponding partial safety factor.

The partial safety factors for determining the load-bearing capacity at the ultimate limit state (ULS) are chosen as specified for the individual structural materials. Recommendations for the selection of partial safety factors are given in A.8 to A.11.

6.4.2 Ductility

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

NOTE It is possible that the serviceability is adversely affected by utilizing the ductility, see 5.3.5.

6.4.3 Equilibrium conditions

The building shall remain in a stable equilibrium even in case of seismic events. This also includes effects from overturning, sliding and uplift.

6.5 Serviceability limit state (SLS)

6.5.1 General requirements

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 Formula (6). This requires performing verifications at the serviceability limit state (SLS) with corresponding requirements (e.g. deformation and crack-width limitations).

It shall be shown that Formula (9) is met for the serviceability limit state (SLS).

$$E_d \leq C_d \quad (9)$$

where

E_d is the design value of the load (e.g. stress, deformation, crack width) during a seismic event;

C_d is the design value of the serviceability criterion (e.g. permissible stress, deformation or crack width) required for achieving the safety objectives specified in ISO 4917-1.

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

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 structural elements related requirements or crack-width limitations with respect to leak-tightness requirements or requirements regarding load-bearing capacity and deformation behaviour of fastenings. The respective requirements will be specified for the individual case.

6.5.2 Deformations

The deformations shall be determined for the combination of actions specified in 6.2.

If the analysis of the building deformations does not account for material non-linearities (e.g. by a reduction of the stiffness values) and the damping values specified in Table A.1 are used, then the building deformations determined for the seismic event shall be increased adequately. An increase of 50 % is recommended in the DBE.

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

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

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

b) When planning the size of a building joint, the stiffness and the limited compressibility of any infill material should be considered.

6.6 Beyond design considerations

In order to provide adequate seismic margins for the most important system, structures and components and to avoid cliff edge effects, the considerations for beyond design basis according to IAEA SSG-67 shall be applied. The beyond design basis earthquake shall be defined in accordance with national regulations.

6.7 Requirements on foundations

The foundations shall basically meet the requirements in accordance with the relevant national requirements with respect to load-bearing capacity and serviceability.

In the case of pile foundations, interactions with the soil and between the piles should be considered.

7 Structure-type dependent seismic verifications

7.1 Structural members of reinforced and pre-stressed concrete

7.1.1 General requirements

The requirements regarding analysis and structural design in accordance with relevant national standards for reinforced concrete shall be applied, provided, in the following no other requirements are specified.

It is not permitted to use a concrete with a nominal compressive strength less than f_c = 20 MPa.

Unless a more exact verification of the ductility is performed, the reinforcing steel should have a high ductility. Further recommendations are given in Annex A.

7.1.2 Strength parameters

The design value for the uniaxial (or unconfined) compressive strength at the ultimate limit state (ULS), f_{cd} shall take safety factors into account. A recommendation is given with Formula (10).

The design values for the strength of the reinforcing steel and pre-stressing steel shall take safety factors into account. Recommended values are specified in Table A.3.

7.1.3 Verifying the load-bearing capacity

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.

Alternatively, the biaxial bending design of support columns may be replaced by individual verifications of a 70 % flexural load bearing capacity, $M_{\mathrm{Rd},i}$, in each direction, i, as given by Formula (10).

$$M_{\mathrm{Ed},i} < 0{,}7 \cdot M_{\mathrm{Rd},i} \tag{10}$$

where

$M_{\mathrm{Ed},i}$ is the external moment;

$M_{\mathrm{Rd},i}$ is the bending load bearing capacity;

i is the each of one of the two directions.

The shear-force resistance of a reinforced or pre-stressed concrete structural element shall be verified in accordance with the relevant national standards. Recommendations are given in Annex A.

The punching shear verification (in the case of support columns on plates or foundations) shall be verified in accordance with relevant national standards. Recommendations are given in Annex A.

7.2 Steel structure parts

The relevant national requirements shall be applied to the design and construction of steel parts. Further recommendations are given in Annex A.

7.3 Masonry

7.3.1 General requirements

It is not permissible in the design for Seismic category 1 buildings to take account of masonry walls as seismic reinforcements.

The requirements in accordance with national standards for masonry under seismic action shall be applied. Recommendations are given in Annex A.

7.3.2 Verifying the load-bearing capacity

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.

7.3.3 Constructional design

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.

The horizontal and vertical enclosing mechanical mounts shall be interconnected and shall be anchored in the structural elements of the main structure.

The force transmission between enclosing mechanical mounts and masonry shall be ensured.

7.4 Steel composite civil structures

The requirements in accordance with national standards shall be applied to the design and construction of steel composite civil structures. Further recommendations are given in Annex A.

7.5 Fastening constructions

The fastening constructions for interconnecting structural or plant-engineering components with reinforced concrete elements 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 element, is subject to special requirements.

It shall be ensured that within the vicinity of fastening constructions using metal dowels or headed 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 of the fastening constructions can be found in national standards.

7.6 Buried pipelines and ducts

In case of buried pipelines and ducts, the soil deformations due to seismic waves transition, displacement and the additional dynamic soil pressure shall be considered.

In the case of buried pipelines and ducts assembled from multiple stiff parts, it shall be verified that in the transition regions the permissible torsions and axial displacements are not exceeded.

NOTE Specified requirements regarding buried steel pipelines can be found in national standards.

7.7 Support structures

Support structures shall be designed to resist the dynamic soil pressure. The design value for the force acting from the groundside onto the support structure may be determined in accordance with the relevant national standards.

NOTE If no national standard is available for the design of support structures to resist the dynamic soil pressure regulations can be found in EN 1998-5.

Annex A
(informative)

Recommendations with comments

A.1 Recommendation to 5.2

Table A.1 recommended values for the damping ratio, *D* for the analyses of buildings, partial buildings and individual structural elements.

Table A.1 — Recommended damping ratios, *D*, in percent of critical damping for the analyses of buildings and individual structural elements

Buildings and individual structural member built from	A[a]	B[a]
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
Masonry	7	4

[a] The application of colums A and B is explained in 5.2.6.

NOTE The degree of utilization under the seismic action combination of the considered structure influences the amount of damping. In areas of low seismicity, the seismic action combination is often not determinant for the design, so that the degree of utilization under this action combination is low. If overall building damping with the energy dissipating effect of equipment and non-load-bearing elements is not present only small damping values are justified.

In case of oscillations of fluids, a damping ratio of 0,5 % is recommended for the analyses of buildings and partial building structures.

A.2 Recommendation to 5.3

The ultimate limit state (ULS) should basically be verified taking geometric non-linearities into account. This requirement may be waived, provided, the following requirement in Formula (A.1) is met.

$$\theta = \frac{p_{tot} \cdot d_r}{V_{tot} \cdot h} \leq 0{,}10 \tag{A.1}$$

where

θ	is the interstorey drift sensitivity coefficient;
P_{tot}	is the total gravity load at and above the storey considered in the seismic design situation;
d_r	is the design interstorey drift, evaluated as the difference of the average lateral displacements d_s at the top and bottom of the storey under consideration and calculated in accordance with 6.5.2;
V_{tot}	is the total seismic storey shear;
h	is the interstorey height.

A.3 Recommendation to 5.3.3

It is recommended to limit the modal damping respectively the Rayleigh damping of frequency-independent soil or rock models to a damping ratio of 15 % in horizontal direction and of 30 % in vertical direction for the significant frequencies.

A.4 Recommendation to 6.2

Table A.2 shows recommended combination factors for actions.

Table A.2 — Combination factors for actions

Actions		Combination factor	
		ψ_2[a]	ψ_E[b]
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	[a]
	— platform surface loads	0,30	0,25
	— friction forces	0,30	[c]
	— loads during majorinspections	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 [d]	[a]
	Snow	0 [d]	0 [e]
	Climatic temperature effects	0	[a]
	Restraint force due to settlements	1,00	[a]

[a] Combination coefficient for the quasi-permanent value of a variable actions.

[b] Combination coefficient to be used when determining seismic inertia forces from masses from variable actions.

[c] Not applicable.

[d] Combination with seismic loads may be necessary according to national standards.

[e] Consideration in the seismic mass may be necessary according to national standards.