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Directive on the Safety of Water Retaining Facilities

Part C3: Seismic Safety

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Content

Notation

$D_{5\text{-}95}$ [s]	Significant duration, i.e.: the length of interval-time where 5% to 95 % of the seismic energy is applied
f_0 [Hz]	Fundamental frequency of a site
Ia [m/s]	Arias intensity, i.e.: the time-integral of the square of the ground acceleration
PI [-]	Plasticity index based on Atterberg limits
$PPSAx$ [g]	Peak pseudo-spectral acceleration for ground class x (i.e., *PSA* at the plateau of the target response spectrum for ground class x, *where x = R, AR, A, B, C, D or E*)
PSA [g]	Pseudo-Spectral Acceleration (ordinate value of elastic response spectrum)
R_{JB} [km]	Joyner-Boore distance, i.e.: the shortest distance from a site to the surface projection of the rupture surface
RP [year]	Return Period of the Safety Evaluation Earthquake (SEE)
S_x [-]	Amplification factor for ground class *x* with respect to ground class R. This value is the ratio between plateau of the elastic response spectrum for ground class *x* to that of ground class R.
T [s]	Period of vibration
T_B [s]	Characteristic period of elastic response spectrum
T_C [s]	Characteristic period of elastic response spectrum
T_D [s]	Characteristic period of elastic response spectrum
V_s [m/s]	Shear-wave velocity
Vs_{30} [m/s]	The time-averaged shear-wave velocity (V_S) to a depth of 30 meters, i.e.: $\bar{V}_s = \frac{\sum h_i}{\sum t_i} = \frac{\sum h_i}{\sum\left(\frac{h_i}{Vs_i}\right)}, \sum h_i = 30\ m$ where t_i is the one-way travel time in i^{th} layer, h_i is the thickness of the i^{th} layer and Vs_i is the shear-wave velocity in the i^{th} layer
γ_{normal} ; $\gamma_{extraordinary}$ [-]	Partial resistance factors for normal and extraordinary load cases as per Directive Part C1
γ_{normal}^{nb}; $\gamma_{extraordinary}^{nb}$ [-]	Partial resistance factors for normal and extraordinary load cases during the period immediately after a seismic action at level of *Seismic Evaluation Earthquake* (SEE) and prior to executing strengthening measures
$\mu_{D5\text{-}95}$ [sec]	Theoretical mean value of the significant duration
μ_{Ia} [m/s]	Theoretical mean value of the Arias intensity
ξ [-]	Viscous damping ratio
η [-]	Adjustment value of response spectrum (depending on viscous damping ratio)

1 Introduction

1.1 Legal basis

1.1.1 According to Article 5 paragraph 1 of the Federal Act on Water Retaining Facilities of 1 October 2010 (WRFA, SR 721.101) "water retaining facilities must be designed, constructed and operated in accordance with the state-of-the-art of science and technology so that their safety is guaranteed for all foreseeable operating and loading cases." On this basis, Article 5 paragraph 1 of the Water Retaining Facilities Ordinance of 23 November 2022 (WRFO, SR 721.101.1) specifies the requirements for structural safety of water retaining facilities as follows: "Any person who wishes to build, modify or operate a water retaining facility must ensure the safety of the dam, the safety-relevant auxiliary installations and the reservoir under normal, extraordinary and extreme loading cases". In the WRFO, the loading case of earthquake is considered as one of the extreme loading cases against which the safety of the water retaining facility must be guaranteed (see Article 5 paragraph 4 WRFO). The Swiss Federal Office of Energy (SFOE) is responsible for establishing directives and other technical resources for normal, exceptional and extreme loading cases (see Article 5 paragraph 5 WRFO). The Swiss Seismological Service has introduced an updated seismic hazard model, SUIhaz2015 (Wiemer et al., 2016). In order to implement the updated seismic hazard and take the "state-of-the-art" for seismic safety evaluation of water retaining facilities in Switzerland into account, the SFOE completely revised the Directive on the safety of water retaining facilities, part C3: seismic safety. In this revision, the principle of proportionality has been considered as far as possible.

1.1.2 This Directive Part concerns the verification of the seismic safety of water retaining facilities subject to the WRFA in the event of occurrence of an earthquake as an extreme loading case. In this part of the Directive, this level of seismic loading is named: Safety Evaluation Earthquake (SEE).

1.1.3 Seismic safety verification in accordance with the current directive, is required for all water retaining facilities subject to the WRFA and especially in the following cases:

- new, refurbished, modified, or heightened water retaining facilities within the framework of a planning approval procedure;
- if it is necessary to take into account changes in the inputs and/or assumptions made in a previous verification of seismic safety;
- in the event of abnormal behaviour of a dam or a sudden change in its behaviour, the supervisory authority may mandate an update of the seismic safety verification.

1.1.4 This Directive Part is to be applied in conjunction with the WRFA and WRFO, as well as the other parts of the Directive on the Safety of Water Retaining Facilities. The requirements set out in the directive are minimum safety requirements. Methods other than those mentioned in this directive may be applied, provided that an equal or higher level of safety is guaranteed.

2 Main objectives of the seismic safety and behavioural goals of the water retaining facilities

2.1 Main objectives

2.1.1 The present directive is intended to ensure that the seismic safety of dams in Switzerland, which are subject to the WRFA, is verified according to uniform criteria.

2.1.2 The main objectives of protection against earthquake are:

- Protection of human life and to ensure the safety of individuals from harm or injury.
- Protection of critical infrastructure, within the limits described in this document.
- Protection of the environment, within the limits described in this document.
- Protection of the properties, within the limits described in this document.
- Protection against major economic consequences, within the limits described in this document.

2.1.3 The purpose of this Directive Part is to implement the objectives of protection against earthquake for water retaining facilities.

2.2 General behavioural goals of the water retaining facilities

2.2.1 The objective of the seismic safety verification of a water retaining facility is to verify that during and following an earthquake a failure of the facility that could lead to an uncontrolled and potentially damaging release of water (or other retained material in case of natural hazard protection dams, e.g., debris) can be excluded. It is noted that damage, including permanent deformations that do not compromise the safety of the water retention facility, is considered acceptable.

2.2.2 After a Safety Evaluation Earthquake, the water retaining facilities shall have the necessary safety reserves so that it is possible to lower the reservoir water level and/or carry out repair works.

2.2.3 Ensuring the normal operability of the water retaining facility in the period immediately after a Safety Evaluation Earthquake is not a behavioural goal with respect to the safety of water retention facilities.

2.2.4 In the event of a Safety Evaluation Earthquake, the functionality of the monitoring system elements, which are indispensable for the detection of an imminent failure, shall be maintained or quickly restored. The identification of the indispensable monitoring system elements is part of the seismic safety verification.

2.3 Behavioural goals of safety-relevant auxiliary installations

2.3.1 The safety-relevant auxiliary installations of a water retaining facility, including spillways and outlets, shall remain free from structural damage that can induce any risk of uncontrolled release of water.

2.3.2 Immediately after an earthquake, with the aid of outlets or by other means, it shall be possible to safely lower or empty the reservoir.

2.3.3 Immediately after a Safety Evaluation Earthquake, the spillways or outlets of a water retaining facility of Category I or II shall be capable of safely conveying a flood event with a 50-year return period. Immediately after a Safety Evaluation Earthquake, the spillways or outlets of a water retaining facility of Category III shall be capable of safely conveying a flood event with a 30-year return period.

2.3.4 The behaviour of structures and installations that are not commonly considered as safety-relevant auxiliary installations shall also be included in the considerations, if their failure under seismic action could affect the safety of the water retaining facility.

2.4 Behavioural goals for the associated reservoir

2.4.1 Under seismic action, potential mass movements in the area of the reservoir as well as waves generated in the reservoir directly triggered by an earthquake shall not harm the safety of the dam, the safety-relevant auxiliary installations, humans, environment and infrastructure. Additionally, these processes shall not induce uncontrolled and damaging release of water.

2.4.2 The term "potential mass movements" (Paragraph 2.4.1) refers in particular to landslides, sliding of unstable slopes and banks, rockfalls, avalanches, icefalls from glaciers, etc.

2.4.3 For all water retaining facilities (including those intended to protect against natural hazards), the triggering of the mass movements in the catchment area due to seismic action and their potential impacts on the facility after the earthquake shall be taken into account.

2.5 Measures to ensure the seismic safety

2.5.1 If it is found that a dam, its abutments, its reservoir, the reservoir's surrounding slopes, or the safety-relevant auxiliary installations do not satisfy the above objectives and behavioural goals, adequate measures shall be proposed and implemented by the water retaining facility operator to ensure its seismic safety.

2.5.2 The knowledge gained from the seismic safety verification process shall be considered in the emergency action planning.

2.6 Serviceability

2.6.1 Ensuring serviceability of the water retaining facility following an earthquake is the responsibility of the operator.

2.6.2 The obligation to inform the supervisory authority, pursuant to Article 26 of the WRFA and Article 22 of the WRFO, remains valid. The operator shall ensure that the protection of the population and the environment is guaranteed at all times, and that the facilities to lower the water level or empty the reservoir are operational. In accordance with Appendix 1 of Directive Part D: *Commissioning and operation* after an earthquake, an inspection must be carried out by the operator. Any anomalies must be reported to the supervisory authority in accordance with Article 22 of the WRFO.

3 Categories of water retaining facilities

3.1 General categorisation of water retaining facilities

3.1.1 For the purpose of their seismic safety verification, all water retaining facilities are classified in three categories for which different requirements apply.

3.1.2 The categorisation is based on the following criteria, Table 1 and Figure 1.

- Category I: Water retaining facilities that fulfil the criteria specified in Article 19, paragraph 1a or 1b of the WRFO.
- Category II: Water retaining facilities that have a storage height of at least 5 metres and fulfil the size criteria specified in Article 3, paragraph 2 of the WRFA and are not classified in Category I.
- Category III: Water retaining facilities that do not fulfil the size criteria specified in Article 3, paragraph 2 of the WRFA or only have a storage height of less than 5 metres.

Table 1: Definition of the three water retaining facility categories (H = storage height, V= storage volume)

Category	Description	Criteria
I	Water retaining facilities that fulfil the criteria specified in Article 19, paragraph 1a or 1b of the WRFO.	$H \geq 40$ m or $H \geq 10$ m and $V \geq$ 1'000'000 m^3
II	Water retaining facilities that have a storage height of at least 5 m, fulfil the size criteria specified in Article 3, paragraph 2 of the WRFA and are not classified in Category I.	$H \geq 25$ m or $H \geq 15$ m and $V \geq$ 50'000 m^3 or $H \geq 10$m and $V \geq$ 100'000 m^3 or $H \geq 5$m and $V \geq$ 500'000 m^3
III	Water retaining facilities that do not fulfil the size criteria specified in Article 3, paragraph 2 of the WRFA or only have a storage height of less than 5 m.	25 < H < 15 m and V < 50'000 m^3 or 15< H < 10 m and V < 100'000 m^3 or 10 < H < 5 m and V < 500'000 m^3 or H < 5 m

Figure 1 shows the categories of water retaining facilities in terms of storage height and reservoir capacity according to the corresponding definitions in the Directive Part A: *Introduction*.

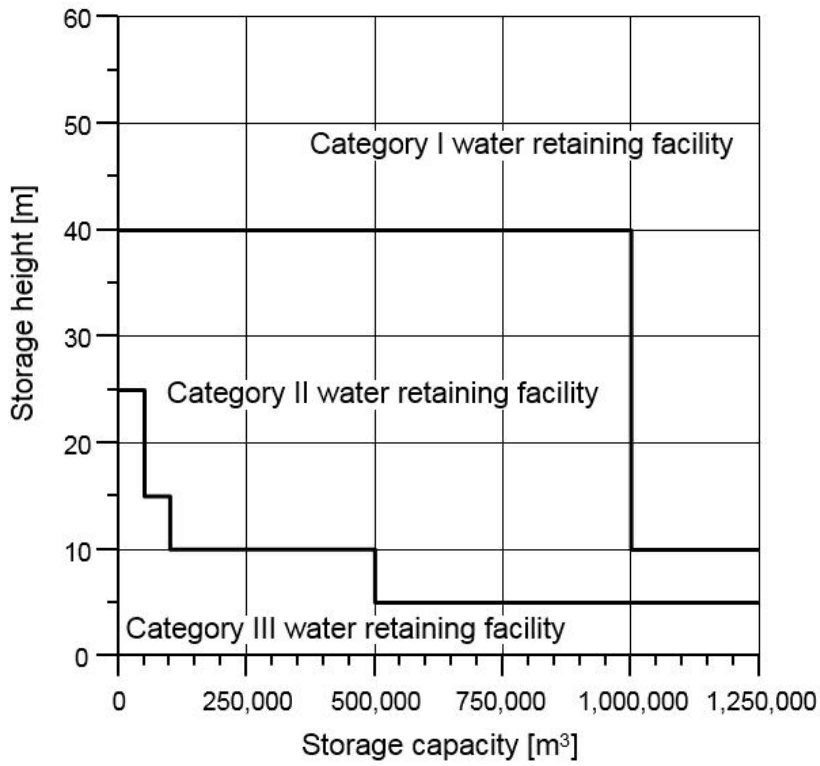


Figure 1: Definition of the three water retaining facility categories

3.1.3 In case the potential failure of a water retaining facility due to seismic loads imposes a hazard to a large number of people, critical infrastructure, or the environment (high consequences), the supervisory authority, the licensing authority or other third-party authorities may modify the classification of the facility to impose stricter requirements.

3.2 Retaining facilities for protection against natural hazards

3.2.1 Retaining facilities intended to protect against natural hazards, which only temporarily store water, mud, sediment, snow, ice, debris, etc., regardless of their storage height and reservoir capacity are classified in category III for the purpose of seismic safety evaluation.

3.3 Lateral embankment dams of run-of-river water retaining facilities

3.3.1 Lateral embankment dams of a run-of-river water retaining facilities situated beyond the vicinity of the main dam are classified in Category III, unless the supervisory authority, the licensing authority or other third-party authorities impose stricter requirements.

4 Earthquake loading case and post-earthquake conditions

4.1 Definition of load cases, initial and boundary conditions

4.1.1 Earthquake load case and initial reservoir level

4.1.1.1. The earthquake loading case is an extreme load (type 3). The earthquake effects shall be determined in accordance with Paragraph 4.3 of the Directive Part C1: *Design and construction*. Other specific effects are cited in Part C1 of the Directive.

4.1.1.2. Deformations and loads attributable to normal loading cases as defined in Part C1 of the Directive form the initial conditions of the earthquake analysis.

4.1.1.3. The partial resistance factors for the loading case of “extreme dynamic” as defined in Paragraph 4.6.5 of the Directive Part C1 may be taken as 1.0.

4.1.1.4. As a rule, the condition of the full reservoir shall be considered for the verification of the earthquake safety of the facility. Here the water level at the applicable storage height in accordance with Part A (Paragraph 6.1) of the Directive on water retaining facilities shall be assumed.

4.1.1.5. If in the event of an earthquake, a partial filling of the reservoir could lead to a more unfavourable behaviour of the retaining structure, these loading cases with partial filling of the reservoir shall also be considered in seismic safety evaluation of the water retaining facility.

4.1.1.6. For all retaining facilities intended to protect against natural hazards (e.g., flood detention dams, bedload collectors, rockfall dams, and snow avalanche dams), the seismic safety of the retaining facility shall be verified for the case of empty reservoir. Additionally, the safety of the facility shall be verified in a second seismic safety verification for the initial condition of full reservoir. For snow avalanche dams the second earthquake safety verification may be done for the reservoir level that corresponds to 10% of the volume of the reservoir. For flood detention dams, the second earthquake safety verification may be done for the reservoir level that corresponds to the average annual water level in the reservoir.

4.1.2 Post-earthquake conditions

4.1.2.1. The safety of the dam after the earthquake impact must correspond to the pre-earthquake safety in the long-term and after any necessary measures in accordance with Part C1.

4.1.2.2. In addition to the Safety Evaluation Earthquake, the safety of the water retaining facilities of Category I shall be verified for immediate aftershocks. The ordinate values of the target elastic response spectra (Paragraph 4.3.4) and the significant duration assumed for the aftershock analysis should be at least 50% of those used for the main earthquake.

4.1.2.3. Immediately after an earthquake and the corresponding aftershock (for Category I dams, according to 4.1.2.2), until relevant necessary measures have been implemented, the safety level of the water retaining facility is considered sufficient, if it is demonstrated that the water retaining facility can withstand the normal and extraordinary loading cases (see Directive Part C1, Paragraph 4.6.5) with 80% of the partial resistance factors.

$$\gamma_{normal}^{nb} = \gamma_{normal} \cdot 0.8; \tag{1}$$
$$\gamma_{extraordinary}^{nb} = \gamma_{extraordinary} \cdot 0.8; \tag{2}$$
$$\gamma^{nb} \geq 1 \tag{3}$$

where "γ^{nb}" designates the partial resistance factor for the immediate post-earthquake conditions.

For the immediate post-earthquake period, extraordinary loading cases include loading cases that result from immediate intervention measures (e.g., rapid drawdown of the water level), or that arise due to a flood occurring immediately after the earthquake. The return period of such flood is defined in Paragraph 2.3.3.

4.1.2.4. The analysis of the safety of concrete and masonry dams for the immediate post-earthquake conditions shall include a check as to whether the calculated irreversible displacements of the structure (if any) indicate damage to the watertightness and/or the drainage system of the structure. If this is the case, then the immediate post-earthquake stability of the dam shall be checked considering the corresponding modified uplift conditions.

4.1.2.5. For the analysis of embankment dams for the immediate post-earthquake phase, the permanent displacements and the generated excess pore-water pressures in the dam and its foundation shall be considered.

4.1.2.6. The operability of spillway gates, bottom outlets and other safety relevant hydromechanical equipment shall be verified for the immediate post-earthquake phase. The consequences of an eventual malfunction of these devices in the immediate post-earthquake phase shall be examined and emergency measures shall be taken to ensure the safety of the dam in the post-earthquake phase. Emergency measures may require the lowering or emptying of the reservoir and operating the spillway gates - if any -, the bottom outlets and other safety relevant hydromechanical equipment, as well as emergency power supply.

4.1.2.7. The supervisory authority defines on a case-by-case basis the duration for which reduced safety may be tolerated.

4.1.2.8. For loading cases that arise due to implementation of relevant necessary repair measures conducted in the post-earthquake period, the supervisory authority specifies the hazard and the safety requirements on a case-by-case basis.

4.1.2.9. The deformations, damages, uplift pressures, generated excess pore water pressures, seepage, and specific loads resulting from the earthquake shall serve as the initial conditions for evaluating the safety of the water retaining facility in the immediate post-earthquake period. These initial conditions must be considered, particularly in scenarios where prompt measures, such as the rapid drawdown of the reservoir water level, are planned following the earthquake.

4.2 Definition of the seismic hazard

4.2.1 The seismic hazard is based on the Swiss seismic hazard model SUIhaz2015 (Wiemer et al., 2016) developed by the Swiss Seismological Service. The seismic hazard is defined as the effect of an earthquake for a given return period at the location of the water retaining facility for a homogeneous reference rock formation defined by the seismic shear wave velocity profile of Poggi et al. (2011).

4.2.2 As an alternative the seismic hazard can be defined by a dedicated probabilistic seismic hazard analysis as specified in Appendix C of this document.

4.2.3 In accordance with the category of the water-retaining facility, the mean exceedance probability of seismic hazard or the return period of the Safety Evaluation Earthquake (SEE) is used to determine the seismic hazard. Table 2 lists the return periods to be assumed by category.

Table 2: Applicable return period according to category of water retaining facility

Category	Reference timeframe	Mean probability of exceedance in the reference time frame	Approximate return period of Safety Evaluation Earthquake
I	100 years	1%	10’000 years
II	100 years	2%	5’000 years
III	100 years	10%	1’000 years

4.3 Seismic action

4.3.1 Components of seismic action

4.3.1.1. The definition of seismic action comprises the following components:

- Elastic response spectrum
- Acceleration time-histories

4.3.1.2. The elastic response spectra are derived from the probabilistic seismic hazard and by taking into account the site effects.

4.3.1.3. Information on how to select acceleration time-histories is provided in Paragraph 4.3.5.

4.3.1.4. The seismic action is given as a free-field action at the ground surface.

4.3.2 Effects of site conditions and topography on seismic action

4.3.2.1. Site conditions (e.g., geological and/or geotechnical properties of the subsurface layers), the local topography and the bedrock geometry influence the seismic action. As far as it is of relevance, site conditions, the local topography and the geometry of relevant geological formations shall be taken into account in the seismic analysis of the water retaining facility.

4.3.2.2. The influence of the subsoil has to be taken into account either by classifying the subsoil into a ground class according to Table 3 or using a site response analysis according to Appendix C.

4.3.2.3. The relevant ground class for the seismic safety verification of the dam is determined by considering the spatial variability of the subsoil conditions. The least favourable subsoil conditions relevant for the analysis define the ground class of the dam. This concept applies similarly to the seismic safety verification of auxiliary structures and stability of the reservoir and its surrounding slopes.

4.3.2.4. For water retaining facilities of Categories I and II, the classification into a ground class shall be based on specific geophysical studies conducted on the dam site, geological survey, and if necessary, geotechnical investigations confirmed by specialists in geophysics, geology and geotechnics, respectively. The coherence of the data and the classification into a ground class shall be confirmed by experts (Table 4).

4.3.2.5. If vulnerable critical infrastructures (e.g., hospitals, buildings and facilities for disaster management or main traffic routs) are located within the potential inundation zone of water retaining facilities of Category III, the classification into a ground class shall be defined according to 4.3.2.4. Geophysical studies for all other water retaining facilities of Category III and all natural hazard protection dams are not compulsory for the classification into a ground class. However, in these cases ground classification shall be based on relevant geological and geotechnical information and shall be confirmed by experts in these fields (Table 4).

4.3.2.6. For all water retaining facilities, the classification in ground class R can only be made if the foundation has been comprehensively and quantitatively studied. A “comprehensive and quantitative study” involves a spatially comprehensive investigation by geophysical measurements on-site and a comprehensive geological survey. For a classification in ground class R, these investigations must show that in the top 30 m, the foundation consists of rock with a time-averaged shear-wave velocity (Vs_{30}) equal to or greater than 1105 m/s. In all parts of the relevant seismic profile (at least the top 35 m), the shear-wave velocities (Vs) shall be greater than 1000 m/s and shall not show significant impedance contrasts. The coherence of the data and the classification into a ground class shall be confirmed by experts.

4.3.2.7. For all water retaining facilities, the classification in ground class AR can only be made if the foundation has been comprehensively and quantitatively studied. A “comprehensive and quantitative study” involves a spatially comprehensive investigation by geophysical measurements on-site and a comprehensive geological survey. For a classification in ground class AR, these investigations must show that in the top 30 m, the foundation consists of rock with a time-averaged shear-wave velocity (Vs_{30}) equal to or greater than 800 m/s and shear-wave velocities (Vs) greater than 760 m/s. The coherence of the data and the classification into a ground class shall be confirmed by experts.

4.3.2.8. In cases where geological conditions suggest significant deviations in the shear wave velocity profile at depth from the values used for seismic hazard assessment (as per Paragraph 4.2), or where the lithostratigraphy shows significant stiffness contrasts, the shear wave velocity profile within the relevant depth shall be investigated. The implications of the above-mentioned conditions on seismic action, including adjustments to corner periods and/or location of the plateau of the elastic response spectrum, shall be considered in consultation with experts and the supervisory authority. This applies similarly to cases where the subsoil conditions are significantly better than the minimum requirement for classifications into ground class R. In these cases, a site response analysis according to Appendix C is recommended.

4.3.2.9. The classification into a ground class shall be validated by the supervisory authority.

4.3.3 **Active fault**

4.3.3.1. The relevant vicinity of the water retaining facility shall be investigated to locate faults and tectonic lineaments. The potential activity and the tectonic nature of the discontinuities shall be addressed. The search for evidence of active tectonics should not be limited to existing seismicity data but should also include geological and/or geomorphological indicators derived from e.g., geological and tectonic maps or reports, field observations, remote sensing and available geophysical surveys. A fault with evidence of Syn-Quaternary movements shall be considered potentially active. If a fault with evidence of Syn-Quaternary movements exists in the relevant vicinity of the water retaining facility, the contribution of the fault to the local seismic hazard shall be evaluated and detailed studies should be carried out relating to the specific case. The surrounding of the dam and its reservoir should also be the object of dedicated studies and relevant monitoring if based on the geological context, and spatial and temporal relationships a Reservoir-Triggered-Earthquake cannot be excluded.

4.3.3.2. If faults or tectonic lineaments are identified in the immediate vicinity of the water retaining facility and the relative displacement of the fault-bounded blocks and/or tectonic elements is relevant for the safety of the water retaining facility, the possibility for earthquake-induced relative displacements of the fault-bounded blocks and/or tectonic elements shall be addressed.

4.3.3.3. For Category I water retaining facilities, the activity or activation of faults in the vicinity of the facility must be periodically assessed. This assessment should be carried out with a periodicity of 5 years. For Category I and II water retaining facilities, the activity or activation of faults in the vicinity of the facility shall be assessed in case an earthquake of Magnitude 4 or higher with an epicentre located within a radius of 10 km from the facility is measured.

4.3.3.4. Significant oscillations of the reservoir, triggered by an earthquake, fault movement, or subsidence within the reservoir, may result in overtopping. Large waves generated by low-frequency oscillations of the reservoir, such as a seismic seiche, may also cause overtopping, potentially leading to unacceptable consequences. In cases where such potentials exist, more detailed geological and seismological investigations are required, and additional safety measures may be imposed by the supervisory authority.

4.3.4 Elastic response spectrum

4.3.4.1. The spectral acceleration of the elastic response spectrum is calculated by multiplying the peak pseudo-spectral acceleration ($PPSA_x$) for each ground class at the desired period of vibration (T) by other terms accounting for the spectrum shape and damping.

4.3.4.2. The generalised elastic response spectra of the horizontal components of the seismic action as depicted in Figure 2 are constructed according to Equations (4) to (7).

$$PSA\ (T) = PPSA_x/2.5 \cdot \left[1 + \left((2.5 \cdot \eta - 1) \cdot T\right)/T_B\right] \qquad (0 \le T \le T_B) \qquad (4)$$

$$PSA\ (T) = PPSA_x \cdot \eta \qquad (T_B \le T \le T_C) \qquad (5)$$

$$PSA\ (T) = PPSA_x \cdot \eta \cdot T_C/T \qquad (T_C \le T \le T_D) \qquad (6)$$

$$PSA\ (T) = PPSA_x \cdot \eta \cdot (T_C \cdot T_D)/T^2 \qquad (T_D \le T) \qquad (7)$$

where:

PSA (T):	Pseudo-Spectral Acceleration (ordinate value of elastic response spectrum).
$PPSA_x$:	peak pseudo-spectral acceleration for ground class *x* (i.e., *PSA* at the plateau of the target response spectrum for ground class *x*); The value of *PSA* for rigid body vibration (vibration period of 0 seconds) is equivalent to the peak ground acceleration (PGA) and can be calculated by $PPSA_x/2.5$.

T:	period of vibration.
T_B, T_C and T_D:	corner periods of the response spectrum, see Table 3.
$\eta = \sqrt{1/(0.5 + 10\xi)} \geq 0.55$:	correction factor to take into account damping, where ξ is the viscous damping ratio. Damping has to be specified depending on the specific conditions of the dam system.

4.3.4.3. The pseudo spectral acceleration at the plateau of the elastic response spectrum for ground class R ($PPSA_R$) is defined by the peak of the mean uniform hazard spectrum – *UHS* on the Swiss Reference Rock (Figure 2). The mean uniform hazard spectrum is either defined by the SUIhaz2015 model (Wiemer et al., 2016) or by site-specific probabilistic seismic hazard analysis (*PSHA*) according to Appendix C. Indicative maps for $PPSA_R$ are provided in the Appendix A of this document, the exact values of *UHS* based on the SUIhaz2015 model and the elastic response spectra shall be extracted from the Hazard Determination Tool (C3-HDT) provided on the SFOE website[1].

4.3.4.4. The site effects for each ground class are taken into account with a ground class dependent amplification factor (S_x) with respect to ground class R, see Table 3. The peak pseudo spectral horizontal acceleration (plateau of the response spectrum, $PPSA_x$) in free field are defined as:

$$PPSA_x = PPSA_R \times S_x \tag{8}$$

where x is either R, AR, A, B, C, D or E.

4.3.4.5. In order to determine the elastic response spectrum of the vertical component of the seismic action, the ordinate values are multiplied by the factor 0.7. For Category I dams for sites with R_{JB} < 10 km, V_{s30} < 500 m/s and $PPSA_R$ > 1.0 g detailed analyses for this factor (ratio of the vertical to horizontal response spectra) are required. The value of R_{JB} is defined as the Joyner-Boore distance to the seismic source for the dominant event contributing to the seismic hazard at the dam site based on the deaggreagation data of the dam location (Appendix B).

[1] https://www.bfe.admin.ch/bfe/en/home/supply/supervision-and-safety/dams/guidelines-and-tools.html

Table 3: Definition of ground classes and the parameters of the soil amplification with respect to ground class R

Ground class	Description of the stratigraphic profile	Vs_{30} [m/s]	Amplification with respect to ground class R (S_x)	T_B [s]	T_C [s]	T_D [s]
R	Massive rock without significant local impairments, significant stiffness contrasts, or loose rock inclusions; examined comprehensively and quantitatively with minimum Vs of 1000 m/s	≥ 1105	1.00	0.06	0.3	2.0
AR	Rock; examined comprehensively and quantitatively with minimum Vs of 760 m/s	>800	1.3	0.07	0.27	2.0
A	Rock or other rock like geological formation with a maximum of 5 metres of loose rock on the surface	>800	1.4 (1.5*)	0.07	0.25	2.0
B	Deposits of very dense sand, and/or gravel or very stiff clay, with a thickness of several tens of metres, characterised by a gradual increase in mechanical properties at greater depth	500... 800	1.8	0.08	0.35	2.0
C	Deposits of dense or medium dense sand, gravel or stiff clay, with a thickness of several tens up to a hundred metres	300... 500	2.2	0.10	0.4	2.0
D	Deposits of loose to medium dense sediments or soft clay	< 300	2.55	0.10	0.5	2.0
E	Surface layer of loose sediments as per C or D, with thickness between 5 and 20 metres and mean V_s-value < 500 m/s over firmer ground material with V_s > 800 m/s	-	2.55	0.09	0.25	2.0

* This value shall be used in case no geophysical studies according to Paragraphs 4.3.2.4 and 4.3.2.5 are used to determine the ground class.

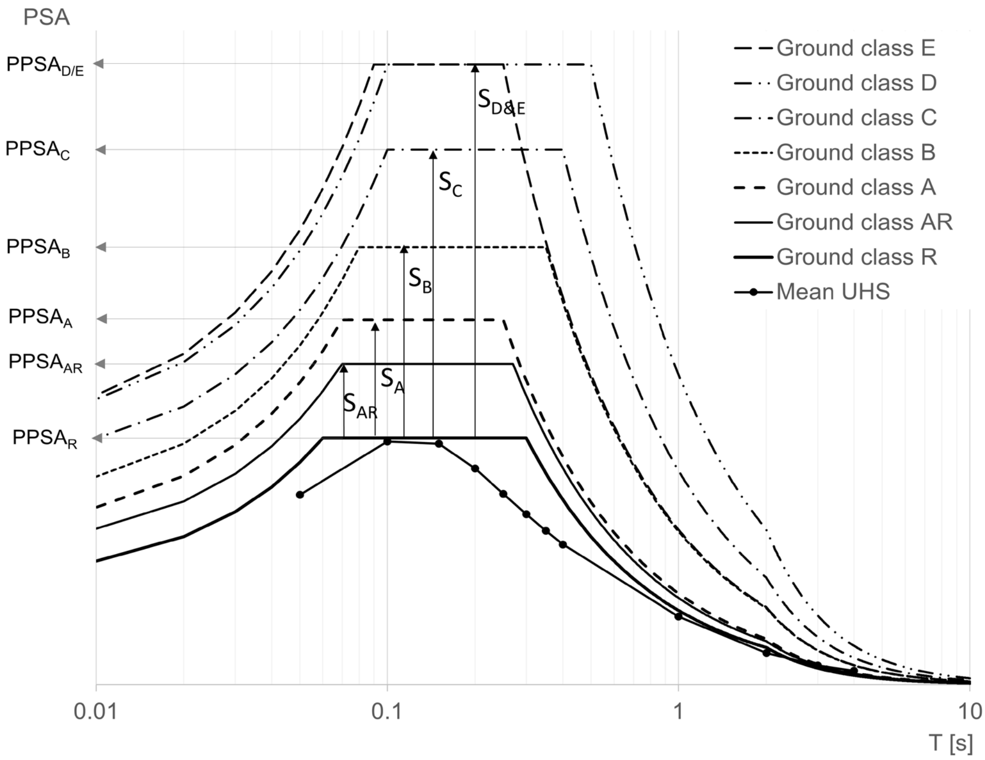


Figure 2: Elastic response spectra of the geometrical mean of the two horizontal components of the seismic action for the ground classes according to Table 3.

UHS: Uniform Hazard Spectrum defined based on SUIhaz2015 (Wiemer et al., 2016) or by site-specific probabilistic seismic hazard analysis (PSHA) according to Appendix C.

PPSA$_x$: Peak Pseudo Spectral Acceleration for ground class x (i.e., PSA at the plateau of the target response spectrum for ground class x).

S$_x$: Ground class dependent amplification factor with respect to plateau of spectral acceleration of ground class R.

PSA: Pseudo Spectral Acceleration (ordinate value of elastic response spectrum).

Table 4: Methodological requirements for site classification according to dam category and ground class.

<table>
<tr><th colspan="2">ground class / dam category</th><th>R</th><th>AR</th><th>A</th><th>B</th><th>C</th><th>D</th><th>E</th></tr>
<tr><td colspan="2">I</td><td colspan="2" rowspan="4">classification based on:
spatial comprehensive investigation by geophysical measurements (Vs profile)
+
comprehensive geological survey

the coherence of the data and the classification shall be confirmed by experts

(4.3.2.6 and 4.3.2.7)</td><td colspan="5" rowspan="3">classification based on:
specific geophysical studies
+
geological survey

the coherence of the data and the classification shall be confirmed by experts

(4.3.2.4 and 4.3.2.5)</td></tr>
<tr><td colspan="2">II</td></tr>
<tr><td rowspan="2">III</td><td>IIIa*</td></tr>
<tr><td>IIIb**</td><td colspan="5">classification based on relevant geological and geotechnical information and confirmed by experts (4.3.2.5)</td></tr>
</table>

*) IIIa: water retaining facilities in cat. III according to chapter 3 and if vulnerable critical infrastructures are located within the potential inundation zone (see 4.3.2.5)

**) IIIb: water retaining facilities in cat. III according to chapter 3 and that do not meet the subgroup IIIa criteria or all natural hazard protection dams.

4.3.5 **Acceleration time-histories**

General considerations for selection of acceleration time-histories

4.3.5.1. Suitable broadband recordings of earthquakes should be used as acceleration time-histories. If the number of suitable records is insufficient, then time-histories of physically-based simulated earthquakes may be used.

4.3.5.2. An acceleration time-history consists of three mutually orthogonal acceleration components, two of which are in the horizontal direction and one in the vertical direction.

4.3.5.3. No more than two records should be used from the same earthquake event.

4.3.5.4. When selecting seismic recordings, it is important to consider the earthquake's moment magnitude (Mw), the distance between the source and the recording site relevant to the earthquake's return period, and the regional tectonic environment, along with the local conditions of the recording site and the fitting between the spectral shape of the recording and the target response spectrum. Recordings should be taken from free-field seismic stations, excluding time-histories from stations installed in structures like buildings, dams, bridges, tunnels, or caverns.

4.3.5.5. At least one acceleration time-history with pulse characteristics of near-field recording should be included in the set of selected acceleration time-histories if a fault, or a seismicity lineament, has been identified in the near-field of the site.

If the Probabilistic Seismic Hazard Analysis (*PSHA*) of the site includes a potential near-field fault, then the selected acceleration time-histories shall include a subset with pulse characteristics of near-field recordings. The specifications and proportions of acceleration time-histories with pulse characteristics should be determined using recognised models from the literature and accounting for the relative contribution of the pulse-type motion to the seismic hazard.

Compatibility with the seismic hazard

4.3.5.6. The selected acceleration time-histories must be compatible with the deaggregation of seismic hazard for the specific seismic zone of the dam and the return period of the seismic hazard. Appendix B provides maps of Switzerland's seismic zones and their deaggregation in normalized form.

4.3.5.7. The theoretical mean ($\mu_{D5\text{-}95}$) of the significant duration ($D_{5\text{-}95}$) of the acceleration time-histories should be determined using recognized models from the literature, based on the moment magnitude (M_w) and the Joyner-Boore distance (R_{JB}) of the governing earthquake scenario. The M_w and R_{JB} for controlling earthquake event in the dam's seismic zone are identified using Appendix B, focusing on Mw and R_{JB} values with a relative significance of 1.0.

4.3.5.8. The geometric mean of the significant duration ($D_{5\text{-}95}$) of the two horizontal components of each individual acceleration time-history (without pulse characteristics) must be greater than $0.7 * \mu_{D5\text{-}95}$. The mean value of all significant durations should be higher than the mean value ($\mu_{D5\text{-}95}$).

4.3.5.9. The theoretical mean value (μ_{Ia}) of the Arias intensity (Ia) of the acceleration time-histories should be determined using recognized models from the literature, based on the moment magnitude (M_w) and the Joyner-Boore distance (R_{JB}) of the governing earthquake scenario.

The M_w and R_{JB} for controlling earthquake event in the dam's seismic zone are identified using Appendix B, focusing on Mw and R_{JB} values with a relative significance of 1.0.

4.3.5.10. The geometric mean of the Arias intensity of the two horizontal components of each individual acceleration time-history must be greater than 0.7* μ_{Ia}. The mean value of all Arias Intensities should be higher than the theoretical mean value (μ_{Ia}).

Compatibility with the target response spectrum

4.3.5.11. The geometric mean of the horizontal components, and in case of spectral matching, the vertical component, of the acceleration time-histories must be compatible with the corresponding target response spectrum in the relevant period range.

4.3.5.12. In general, the target response spectrum is the elastic response spectrum according to Paragraph 4.3.4. If a site response analysis according to Appendix C has been carried out, the corresponding elastic response spectrum (or Uniform Hazard Spectrum (*UHS*) if the site response analysis has been confirmed by means of seismic instrumentation of the site) is considered to be the target response spectrum.

Relevant period range

4.3.5.13. The relevant period range spans the periods from 0.2 T_1 to 1.5 T_1, where T_1 denotes the basic period of the structure. In addition, the relevant period range should cover at least 80% of the total modal mass. When defining the relevant period range, any shifts in the eigenperiods due to nonlinear behaviour under earthquake effects must be taken into account. At least 15 linearly spaced points should be checked in the relevant period range.

Modification of the acceleration time-histories

4.3.5.14. To ensure the compatibility of the acceleration time-histories with the target response spectrum, the acceleration amplitudes may be scaled linearly in time domain. The linear scaling factor should generally be between 0.25 - 4.

4.3.5.15. If an insufficient number of acceleration time-histories can be found using the linear scaling method (Paragraph 4.3.5.14), the spectral matching method may be used.

4.3.5.16. When using spectral matching of acceleration time-histories, existing waveform characteristics especially pulse characteristics of near-field recordings must be preserved after the matching process.

4.3.5.17. In case of linearly scaling, the scaling factors of the horizontal components may be used for the corresponding vertical component of each acceleration time-history.

4.3.5.18. In case of spectral matching, the vertical component of the acceleration time-histories should be modified separately to the vertical target response spectrum. The target response spectrum can be determined from the horizontal response spectrum in accordance with Paragraph 0.

Criteria for compatibility

4.3.5.19. Compatibility between the target response spectrum and the response spectrum associated with the time-histories is considered to be given if all the following criteria are met:

- The arithmetic mean of the response spectra of all time-series shall not be less than 90% and not greater than 130% of the target response spectrum for all periods in the relevant period range (according to Paragraph 4.3.5.13). In case of spectral matching (4.3.5.15) the arithmetic mean of the response spectra of all time-series shall not be less than 95% and not greater than 130% of the target response spectrum for all periods in the relevant period range (according to Paragraph 4.3.5.13).

- In the relevant period range, the mean value over all periods of the ratios between the arithmetic mean of the response spectra of all time-series and the target response spectrum must not be less than 0.95.
- The response spectrum of each individual acceleration time-history shall not be less than 50% of the target response spectrum for all periods in the relevant period range.

Verification with acceleration time-histories

4.3.5.20. At least 7 different (two- or three-component) acceleration time-histories shall be used for verification.

4.3.5.21. For the verification of spatial structural models, both horizontal components and the vertical component of the same earthquake recording must be used simultaneously.

4.3.5.22. For the verification, the acceleration time-histories should be applied in the direction in which they have the greatest possible effect on the structure.

4.3.5.23. The seismic safety of the water retaining facility must be guaranteed for all acceleration time-histories used.

4.4 Notes concerning seismic actions for concrete dams (e.g., gravity dams, arch dams, buttress dams, etc.) and weirs

4.4.1 The dynamic influence of the water retained in the reservoir has to be taken into account (see Paragraphs 6.6 and 6.7).

4.4.2 The uplift acting in the contact joint between the structure and the foundation or at fracture surfaces in the foundation or in the dam body must be taken into account for the verification of stability under seismic action in the same way, i.e. with the same assumptions, as for the static loading case.

4.4.3 If seismic activity induces a change in the uplift conditions, the post-earthquake stability of the structure must be reassessed and demonstrated under the new post-earthquake uplift regime and conditions to ensure the safety and integrity of the dam.

4.4.4 In case the behaviour of a water retaining structure during an earthquake is investigated by means of the response-spectrum method as prescribed in Paragraph 6.3.4, the horizontal elastic response spectra that define the seismic action (Paragraph 4.3.4) shall be combined by taking 100% in one direction and minimum 40% in the other orthogonal direction. The structural response shall be investigated for all possible combinations of the scaled elastic response spectra.

4.4.5 The potential build-up of pore water pressure in the foundation due to the effects of an earthquake has to be taken into account.

4.5 Notes concerning seismic actions for embankment dams

4.5.1 For embankment dams without upstream lining, the hydrodynamic pressures exerted on the upstream face of the dam by the water retained in the reservoir during an earthquake may be ignored. For retaining facilities intended to protect against natural hazards, the dynamic impacts of the retained material during the earthquake need to be taken into account. The potential of liquefaction of the retained material should also be considered in this process.

4.5.2 The potential build-up of pore water pressure due to the effects of an earthquake has to be taken into account (see Paragraph 5.3).

4.5.3 Control of the potential of liquefaction must be done for all embankment dams.

5 Material parameters

5.1 Introduction

5.1.1 The determination of the values of the material parameters shall be based on the details given in the Directive Part C1, Paragraph 4.6.3. For the verification of the seismic safety, cautiously estimated values may be applied for the material parameters, taking into account the implications of the assumptions on the seismic response related to the specific verification situation. For example, if the characteristic value of the modulus of elasticity of concrete results in a lower seismic response, the mean value may need to be considered.

5.1.2 Generally, the material parameters for the dam and its foundation should be determined based on laboratory and field tests. For this purpose, tests representative of the earthquake loading case should be carried out. To account for the uncertainties in the material parameters, sensitivity analyses should be carried out.

5.2 Notes concerning material parameters for concrete and masonry dams and weirs

5.2.1 The unit weight of concrete and masonry should be determined through measurements, or a conservative value should be assumed for each verification step.

5.2.2 The dynamic strength parameters of concrete may be estimated empirically by increasing the corresponding static values by 30%. The dynamic elastic modulus of concrete may be estimated by increasing the static modulus by 25%.

5.2.3 The strength and stiffness parameters of masonry shall be defined by means of tests.

5.2.4 In the case of a structure composed of non-homogeneous materials, the different properties of the individual components must be taken into account.

5.2.5 For concrete, a viscous damping parameter of 5% of the critical damping should be used. In case higher values are used for the viscous damping parameter, they shall be justified.

5.2.6 The value assumed for the viscous damping parameter of reinforced concrete, masonry, and foundation material shall be justified.

5.2.7 For the verification of the safety against sliding at the concrete-rock interface or at the interface between two lifts or types of concrete and/or masonry in the structure, the residual shear strength should be used.

5.2.8 Reservoir silt shall be assumed to liquefy under seismic loading. Thus, the shear strength shall be assumed to be zero unless laboratory and /or site investigations rule out the potential of liquefaction.

5.3 Notes concerning material parameters for embankment dams

5.3.1 For Category I embankment dams static and dynamic soil parameters shall be determined by in-situ and /or laboratory tests. Appropriate material laws that enable describing the behaviour of the dam and foundation soils under dynamic loading shall be applied.

5.3.2 The viscous damping parameter of embankment materials is in general higher than that of concrete. The assumed level of damping for embankment dams shall be verified by the expected level of shear strain in the dam and foundation system.

5.3.3 For Category II and III embankment dams, static material parameters may be used. The details of the verification methodology specified in Paragraph 6 shall be respected.

5.3.4 For embankment dams, the potential for liquefaction in the foundation and in the dam body shall be verified according to Paragraph 5.3.8.

5.3.5 For embankment dams, a distinction shall be made between drained and undrained material behaviour. In the case of undrained condition, in particular the potential loss of shear strength as a result of excess pore water pressures shall be taken into account.

5.3.6 **Drained material behaviour**

5.3.6.1. If drained material behaviour is considered, the analysis is carried out in effective stresses.

5.3.6.2. In the case of water-saturated soils, drained material behaviour can generally not be assumed. If drained material behaviour is nonetheless assumed for saturated soils, this assumption must be validated by estimating the dissipation of the excess pore water pressures in comparison to the loading rate.

5.3.6.3. In general, residual shear strength shall be assumed unless strength softening is accounted for by the constitutive law used.

5.3.6.4. Generally, cohesion shall not be applied unless it can be substantiated by laboratory testing and is demonstrated to exist independently of external factors. Additionally, the effects of shear strain on cohesion must be duly considered.

5.3.7 **Undrained material behaviour**

5.3.7.1. If undrained (or partially drained) material behaviour is expected, the analysis can be carried out in either total or effective stresses.

5.3.7.2. When the analysis is carried out in total stresses, the undrained shear strength under cyclic loading can be assumed to be 80% of the static undrained shear strength for Category II and III embankment dams.

5.3.7.3. When the analysis is carried out in effective stresses, it shall be ensured that the influence of excess pore water pressures is correctly represented by the constitutive law. The plausibility of the mobilised shear strength has to be validated.

5.3.8 **Assessment of liquefaction potential**

5.3.8.1. The liquefaction of dam body material (for embankment dams) or the foundation is linked to a reduction in shear strength due to increased pore pressure during or immediately following the earthquake. This phenomenon poses a significant risk to the structural integrity of dams and has been identified as a potential cause of dam failures. Accordingly, it is essential to conduct a thorough and accurate assessment of liquefaction potential to evaluate the possibility of liquefaction, mitigate associated risks, and ensure the safety and stability of the water-retaining facility.

5.3.8.2. Particularly loose saturated fine sands and silts are susceptible to liquefaction. However, the potential of liquefaction due to generation of excess pore pressure is not limited to this type of soils.

5.3.8.3. To rule out the potential for liquefaction, it must be demonstrated that the soil does not have a high degree of saturation, or it is sufficiently dense, or it contains enough high-plasticity fine materials to prevent liquefaction under extreme seismic loads.

5.3.8.4. In general, materials that satisfy at least one of the following criteria are considered unlikely to have liquefaction potential:

- materials with low degree of saturation,
- materials with D_{10} > 2 mm and relative density Dr > 50%,
- materials with D_{70} < 0.063 mm and plasticity index (*PI*) > 25%,
- materials with relative density Dr > 75%.

The "D_x" values are diameters from particle size analysis, and *PI* refers to the Plasticity Index derived from Atterberg limits. However, if despite the fulfilment of the aforementioned criteria, the potential for liquefaction cannot be conclusively ruled out, detailed investigations shall be required.

5.3.8.5. Liquefaction potential is typically identified through correlations between the severity of cyclic loading during an earthquake and the results of in-situ tests that provide an index of relative density. These tests, such as the Standard Penetration Test (SPT) or Cone Penetration Test (CPT), help determine the relative density of the soil and its resistance to liquefaction. The relationship between cyclic stress from seismic events and density is crucial in assessing the potential of liquefaction in a given dam.

5.3.8.6. Alternatively, the potential of liquefaction may be quantitatively assessed using the combination of the following methods:

- Laboratory tests, for example dynamic triaxial tests.
- Finite element modelling with constitutive models that can adequately represent the phenomenon of generation of excess pore water pressure and liquefaction. The parameters of the constitutive model shall be calibrated to reflect the hydro-mechanical behaviour of the soil layers in question. The calibration of the model parameters shall be done based on laboratory and/or in-situ tests. The uncertainties in the parameters should be determined and taken into account. It is recommended that the performance of the numerical model is verified against existing physical models or relevant and well-documented cases from literature.

5.3.8.7. The assessment of safety against liquefaction does not replace the verification of the safety of the dam under seismic action under undrained conditions.

5.3.8.8. Effects of the potential liquefaction of sediments in the reservoir on the seismic behaviour of the structure shall be assessed for all types of dams (including the retaining structures intended to protect against natural hazards). If found to be present, it shall be taken into account in defining the loading.

6 Methodology for verification of the seismic safety

6.1 Introduction

Seismic safety verification consists of four main steps: development of a model, analysis, interpretation of results, and safety assessment, and refinement of the analysis.

6.2 Development of the model

6.2.1 For the purposes of the analysis, the numerical model shall take into account the foundation, the dam and the reservoir, as well as the applicable loads and actions. The load combinations to be considered shall be in accordance with Paragraph 4.3 of Directive Part C1. The model shall allow for proper simulation of the seismic actions.

6.2.2 Where changes in pore water pressure within the dam body and/or its foundation are anticipated, the pore water pressure must be appropriately modelled to reflect these conditions.

6.2.3 The geological, geotechnical and mechanical properties of the foundation, as shown by investigations or reasonable assumptions, as well as the geotechnical properties of the dam shall be represented by suitable constitutive models. Any relevant potential weak zone shall be identified and properly modelled. In case of uncertainties regarding the material properties of the dam foundation, sensitivity analyses with lower and upper bound material properties shall be considered.

6.2.4 Specific features of the water retaining structure (e.g., the sequence of injections and concreting stages during construction or special occurrences during construction or construction joints) are to be taken into account in the modelling, provided that these could have an influence on the seismic safety verification.

6.2.5 In case of an existing dam, the structural model may be calibrated based on reliable data obtained from regular and/or seismic monitoring and/or in-situ measurements. This information shall be used to verify the parameters of the material models.

6.2.6 For the seismic safety verification of dams founded on rock, the seismic input should be applied at a sufficient depth to include the potential variation of the stiffness of the foundation.

6.2.7 For the seismic safety verification of dams founded on unlithified strata, the analysis should take in consideration the soft material of the foundation down to the contact with the rock. In numerical models, the seismic input should be applied at the top of the rock foundation or at a sufficient depth within the rock formation.

6.2.8 For the purposes of Paragraphs 6.2.6 and 6.2.7, in the absence of a site response analysis (Appendix C), the input acceleration time-histories shall be determined through deconvolution of acceleration time-histories from the free field. The spectra of the output acceleration time-histories at the free field (excluding the effects of the dam) must be compatible with the target spectrum corresponding to the ground class of the dam site. The compatibility criteria are described in Paragraph 4.3.5.19. In case massless foundation is assumed, no deconvolution may be done, and the free-field signal shall be applied to the side and bottom boundaries of the massless foundation model.

6.3 Analysis

6.3.1 The analysis shall address the behaviour of the water retaining structure during the earthquake, as well as in the post-earthquake period.

6.3.2 The structural response due to seismic action, including displacements, permanent deformations and damage to the structure, shall be determined and analysed.

6.3.3 For the post-earthquake safety assessment of the structures, the permanent displacements, the damage, the excess pore water pressures, the new uplift conditions, as well as other potential effects caused by the earthquake shall be taken into account.

6.3.4 The procedure for the analysis of the behaviour during the earthquake shall be chosen according to the Category of the water retaining facility. In general, the following minimum requirements apply, but the plausibility of the results obtained can be checked using less complex methods:

6.3.4.1. **For concrete and masonry gravity dams, buttress dams and weirs:**

6.3.4.1.1. For Category III water retaining facilities: simplified response-spectrum method using only the fundamental natural mode of vibration;

6.3.4.1.2. For Category II water retaining facilities: response spectrum analysis; and

6.3.4.1.3. For Category I water retaining facilities: dynamic time-history analysis.

6.3.4.2. **For concrete arch dams:**

6.3.4.2.1. For Category III water retaining facilities: response spectrum analysis;

6.3.4.2.2. For Category II water retaining facilities: response spectrum analysis; and

6.3.4.2.3. For Category I water retaining facilities: dynamic time-history analysis.

6.3.4.3. **For embankment dams:**

6.3.4.3.1. **For Category III water retaining facilities:** the possible analysis methodologies are divided into two subcategories:

a. Sliding block analysis using applicable empirical methods or analytically based correlation models. This approach applies to:

   i. All natural hazard protection embankment dams

   ii. Dams that fulfil all of the following conditions: 1) $PPSA_R$ < 0.35 g, 2) the dam exhibits no signs of safety-relevant damage, 3) the dam satisfies the static safety requirements, i.e., normal loading case (type 1), as outlined in Directive Part C1, and 4) the flood safety verifications as set forth in Directive Part C2 are fulfilled.

b. Time-history analysis using equivalent linear method integrated with sliding block analysis. Generation of excess pore water pressure shall be considered using simplified models. This approach applies to all other Category III embankment dams that do not satisfy the conditions of Paragraph 6.3.4.3.1.a.

6.3.4.3.2. **For Category II water retaining facilities:** Time-history analysis using equivalent linear method integrated with sliding block analysis. The potential effects of generation of excess pore water pressures shall be considered.

6.3.4.3.3. **For Category I water retaining facilities:** Time-history analysis using equivalent linear method integrated with sliding block analysis. The potential effects of generation of excess pore water pressures shall be considered. If the following two conditions are not satisfied, nonlinear dynamic deformation analysis (time-history analysis) using hydro-mechanical constitutive models capable of modelling the generation and dissipation of excess pore water pressures shall also be conducted:

i. No relevant positive excess pore water pressure in the dam body and in the foundation is expected during earthquake

ii. The average cyclic shear strain due to earthquake loading remains below 0.4%.

6.3.4.3.4. In time-history analysis using equivalent linear method integrated with sliding block analysis, sensitivity analyses with upper and lower bound materials properties shall be used to account for uncertainties.

6.3.4.3.5. In sliding block analysis, the horizontal component of the acceleration acting at the centre of gravity of the sliding mass may be used; the effects of the vertical component may be neglected.

6.3.4.3.6. In case of modelling using nonlinear dynamic deformation analysis (time-history analysis) the model parameters of the integrated constitutive models shall be calibrated based on laboratory and/or in-situ tests. The uncertainties in the parameters should be determined and taken into account. It is recommended that the performance of the numerical model is verified against existing physical models or relevant and well-documented cases from literature. In addition, the results of the time-history analysis using equivalent linear method integrated with effects of generation of excess pore water pressures shall serve as a reference for comparison.

6.4 Interpretation of analysis results and assessment of water retaining facility seismic safety

6.4.1 The behaviour of the water retaining facility during the earthquake and in the post-earthquake period shall be assessed with respect to achieving the behavioural goals stated in Paragraph 2.

6.4.2 In particular, it shall be checked whether the safety of the water retaining facility is guaranteed against any local or global failure that could lead to an uncontrolled and potentially damaging release of water.

6.5 Refinement of modelling and analysis

6.5.1 If the seismic safety of Category II dams cannot be demonstrated using the analysis method corresponding to the dam class, the methods prescribed for Category I dams may be used.

6.5.2 If the seismic safety of Category III dams cannot be demonstrated using the analysis method corresponding to the dam class, the methods prescribed for Category I or II dams may be used.

6.6 Notes on verification methodology for concrete and masonry gravity dams, buttress dams and weirs

6.6.1 Modelling of concrete and masonry gravity dams, buttress dams and weirs

6.6.1.1. The hydrodynamic influence of the retained water may be modelled by means of water masses which are rigidly coupled to the dam or by representing the water with radiative boundary. In the dam-reservoir-foundation system model, the water in the reservoir can be assumed to be either compressible or incompressible. For retaining facilities intended to protect against natural hazards, the dynamic impacts of the retained material other than water during the earthquake need to be taken into account. The potential of liquefaction should be considered in this process.

6.6.1.2. For concrete and masonry gravity dams, a two-dimensional model of the relevant cross-section is usually sufficient. For masonry dams without contraction joints, a three-dimensional model may be considered. In case a spillway structure with piers is located on the top of a gravity dam, the seismic safety shall be investigated by means of a three-dimensional analysis. For a narrow valley site and/or for a site with variable foundation conditions, different cross-sections shall be considered, and a three-dimensional analysis may be necessary in order to model more realistically the behaviour of the water retaining structure. Special attention has to be paid to the abutment areas. In case a three-dimensional model of the dam-foundation-reservoir system is used, the interaction between the dam blocks at the contraction joints considered in the model shall be properly justified.

6.6.1.3. For buttress dams, a three-dimensional model of the whole dam and its foundation is required.

6.6.1.4. For weirs equipped with piers, a three-dimensional model is usually necessary, which includes at least one pier and half of each adjacent weir opening. The model boundaries should take into account the location of expansion and construction joints. If structural elements (such as bridges, engine rooms, etc.) are not taken into account in the modelling, the effect of these elements on the behaviour of the structure shall be considered by means of suitable simplifications. These have to be presented in a comprehensible way.

6.6.1.5. For concrete and masonry gravity dams, buttress dams and weirs of Category I and Category II, the model shall also include the foundation. The foundation may be modelled without or with mass. In both cases, the correct boundary conditions along the foundation borders shall be modelled.

6.6.1.6. For concrete and masonry gravity dams and weirs of Category III founded on ground classes R or AR, it is possible to take into account the effect of the foundation by specifying the response spectrum for the respective ground class directly at the bottom of the dam model, i.e., it is not necessary to include explicitly the foundation domain in the model. However, in case of presence of foundation features that may influence the dam safety, the foundation and the respective features shall be duly modelled and taken into account.

6.6.2 Analysis of concrete and masonry gravity dams, buttress dams and weirs

6.6.2.1. In two-dimensional analysis of concrete and masonry gravity dams, the horizontal component and the vertical component of the earthquake excitation shall be taken into account. In case of time-history analysis, for each acceleration-time-history, the horizontal component shall be chosen in such a manner so as to yield the maximum structural response.

6.6.2.2. For buttress dams and weirs, the three components of the earthquake excitation, i.e., the two orthogonal horizontal components and the vertical component shall be taken into account. In case of time-history analysis, for each acceleration-time-history, the combination of the horizontal components shall be chosen in such a manner so as to yield the maximum structural response.

6.6.2.3. The vertical component of the seismic excitation may be neglected for Category III dams.

6.6.3 Interpretation of results and safety assessment of concrete and masonry gravity dams, buttress dams and weirs

6.6.3.1. Introduction to the steps of results interpretation

The failure of a concrete or masonry gravity dam, a buttress dam and a weir subjected to strong ground motions can be related to overstressing, that can lead to cracking and/or shearing, large displacements, increased uplift pressures, loss of stability against sliding, overturning and/or floating, and/or loss of function of spillway and outlet structures. Most vulnerable to overstressing are the zones close to the dam crest where the dynamic amplifications are the highest, as well as the zones where high stresses are already present due to the static loads, e.g. at the dam base. Cracking that crosses the whole dam section can lead to the formation of a detached block susceptible to failure due to sliding and/or overturning. Sliding can also occur on planes of low shear strength in the dam body, at the dam-foundation interface or within the foundation. In the case of a gated spillway at the top of a dam or in the case of a run-of-river gated weir, high stresses in the piers may lead to a loss of flexural stability. In addition to the above, all other possible failure modes during and after the Safety Evaluation Earthquake related to the dam under study should be investigated and analysed.

A reference case in which linear elastic behaviour is assumed for the dam-foundation system while the foundation is assumed to be massless must first be analysed before proceeding to more complex models. The results of this reference case shall serve as a basis for interpreting the outputs from further sophisticated analyses, as described in the following paragraphs.

It should be noted that the term ‘concrete gravity dams’ in the current Paragraph 6.6.3 refers to either conventional or roller-compacted concrete gravity dams, as the case may be.

6.6.3.2. Assessment of stresses and strains

6.6.3.2.1. For all dam categories, an analysis of the stresses in the dam and its foundation shall be carried out. In case of a nonlinear analysis which includes a crack model, the strains and their interpretation are also needed.

6.6.3.2.2. Linear-elastic behaviour

The safety against overstressing in the dam and its foundation can be considered guaranteed, if the maximum tensile and compressive stresses do not exceed the dynamic tensile and compressive strength, respectively, of the concrete parent material, the lift joints, the masonry mortar and the masonry blocks, the dam-foundation interface and the dam foundation, as applicable, see Checks 1 and 3 of the flowcharts in Figure 3 and Figure 4, as applicable. If this is the case, then it can be proceeded with evaluation of the safety against sliding according to Paragraph 6.6.3.3, see Checks 2 and 4 of the flowcharts in Figure 3 and Figure 4, as applicable.

6.6.3.2.3. Minor nonlinear behaviour in tension

If it is found that the tensile stresses in the dam body obtained from a linear-elastic analysis exceed the tensile strength of the concrete parent material and/or of the lift joints, the safety against overstressing may be further evaluated by considering the ratio between the maximal tensile stress and the corresponding dynamic strength, the extent of the overstressed region, and the cumulative inelastic duration substantiated by means of scientifically founded methods, e.g., the demand-capacity ratio method, see Check 5 in Figure 3. This check is optional and may be skipped. This approach shall not be used for masonry dams. In order to define cumulative inelastic duration, it is necessary to carry out a dynamic time-history analysis. If it is found that the level of nonlinear response is minor, then it can be proceeded with evaluation of the safety against sliding according to Paragraph 6.6.3.3, see Checks 4 and 6 of the flowchart in Figure 3.

6.6.3.2.4. Nonlinear behaviour

If for a given acceleration time-history the criteria of Paragraphs 6.6.3.2.2 and 6.6.3.2.3 are not met, this indicates that the corresponding earthquake can impart a high level of inelastic response to the dam, which may lead to extensive cracking of the dam such that loose/detached blocks are formed. For such sets of acceleration time-histories, nonlinear analyses with gradually increasing complexity shall be carried out. These analyses can incorporate nonlinear models for the base joint at the dam-foundation interface, the lift joints, as well as, for 3D models, the vertical contraction joints. The nonlinear analyses can also incorporate the mechanisms of crack initiation and propagation in the concrete parent material or masonry blocks. Conservative assumptions shall be made regarding the uplift at the surfaces where rupture due to overstressing and permanent displacements have been identified. It shall be verified whether permanent displacements and rotational movements of detached blocks are sufficiently small not to endanger the safety of the dam during the earthquake and the aftershocks, as well as in the post-earthquake period, so that the behavioural objectives according to Paragraph 2 are fulfilled, see Check 7 in Figure 3 and Check 6 in Figure 4, as applicable. Aftershocks shall be considered for seismic safety evaluation of Category I concrete gravity dams, masonry dams, buttress dams and weirs only if the nonlinear analysis as prescribed in the current paragraph is to be carried out.

6.6.3.2.5. The safety against overstressing in the foundation shall consider the bearing capacity of the foundation material, as well as the presence of any discontinuities of lower strength. If the safety against local failure in the foundation is not ensured, it shall be verified whether the behavioural objectives according to Paragraph 2 are fulfilled despite the resulting structural response.

6.6.3.2.6. Stresses and possible damages on elements connected to the structure (in particular spillways, outlet tunnels and other ancillary installations) shall be assessed and considered in the seismic safety evaluation process and, if necessary, in defining measures to guarantee the seismic safety.

6.6.3.2.7. The safety against flexural failure of the reinforced-concrete piers of a gated spillway or of a run-of-river gated weir shall be checked.

6.6.3.3. **Assessment of safety against sliding, overturning and uplift**

6.6.3.3.1. The safety against sliding shall be verified based on the results obtained from the stress analysis for potential sliding surfaces in the dam body, at the dam-foundation interface and in the foundation. No cohesion shall be considered for cracked zones.

6.6.3.3.2. If it is found that sliding is possible, the calculation of permanent sliding displacements along potential sliding surfaces may be carried out using a sliding block method. In this case, it shall be verified whether the behavioural objectives according to Paragraph 2 are fulfilled.

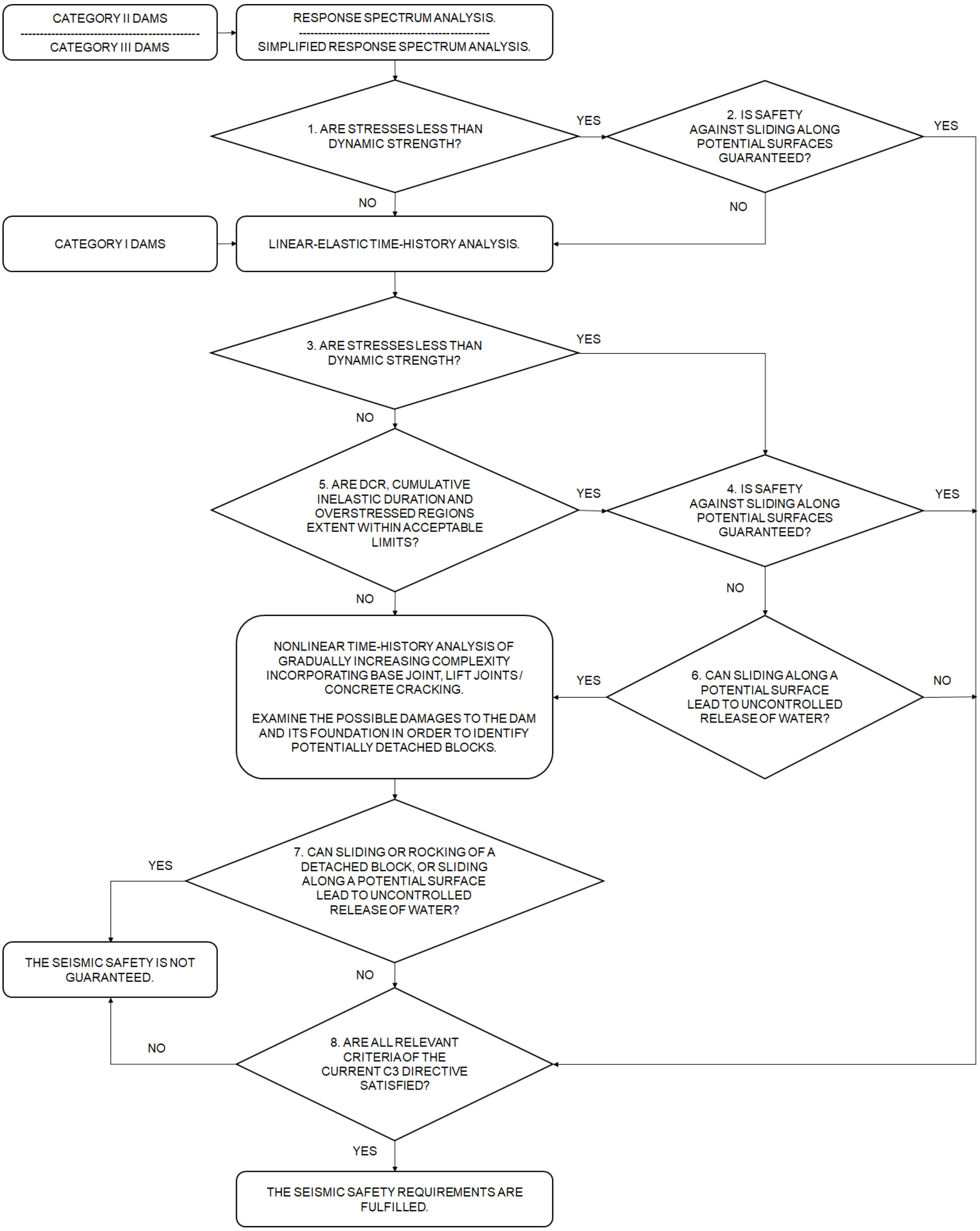
6.6.3.3.3. For the post-earthquake stability, the new uplift conditions due to damaged area, presence of cracks and lower efficiency of the drainage system after earthquake shall be considered. Based on this evaluation, the uplift forces used in the global stability analysis shall be updated.

6.6.3.3.4. The safety against overturning shall be verified at the dam - foundation contact and for blocks in the upper part of the dam where amplification is high. The safety against overturning is guaranteed if the resultant of all forces remains within the base of the cross-section at all times. If this condition is not fulfilled, it shall be verified whether the behavioural goals according to Paragraph 2 are fulfilled despite the resulting structural response.

6.6.3.3.5. The safety against uplift after the earthquake shall be verified for the updated uplift forces.

6.6.3.3.6. A flowchart to be followed for the analysis, interpretation of results and safety verification of concrete gravity dams, buttress dams and weirs is given in Figure 3.

6.6.3.3.7. A flowchart to be followed for the analysis, interpretation of results and safety verification of masonry dams is given in Figure 4.



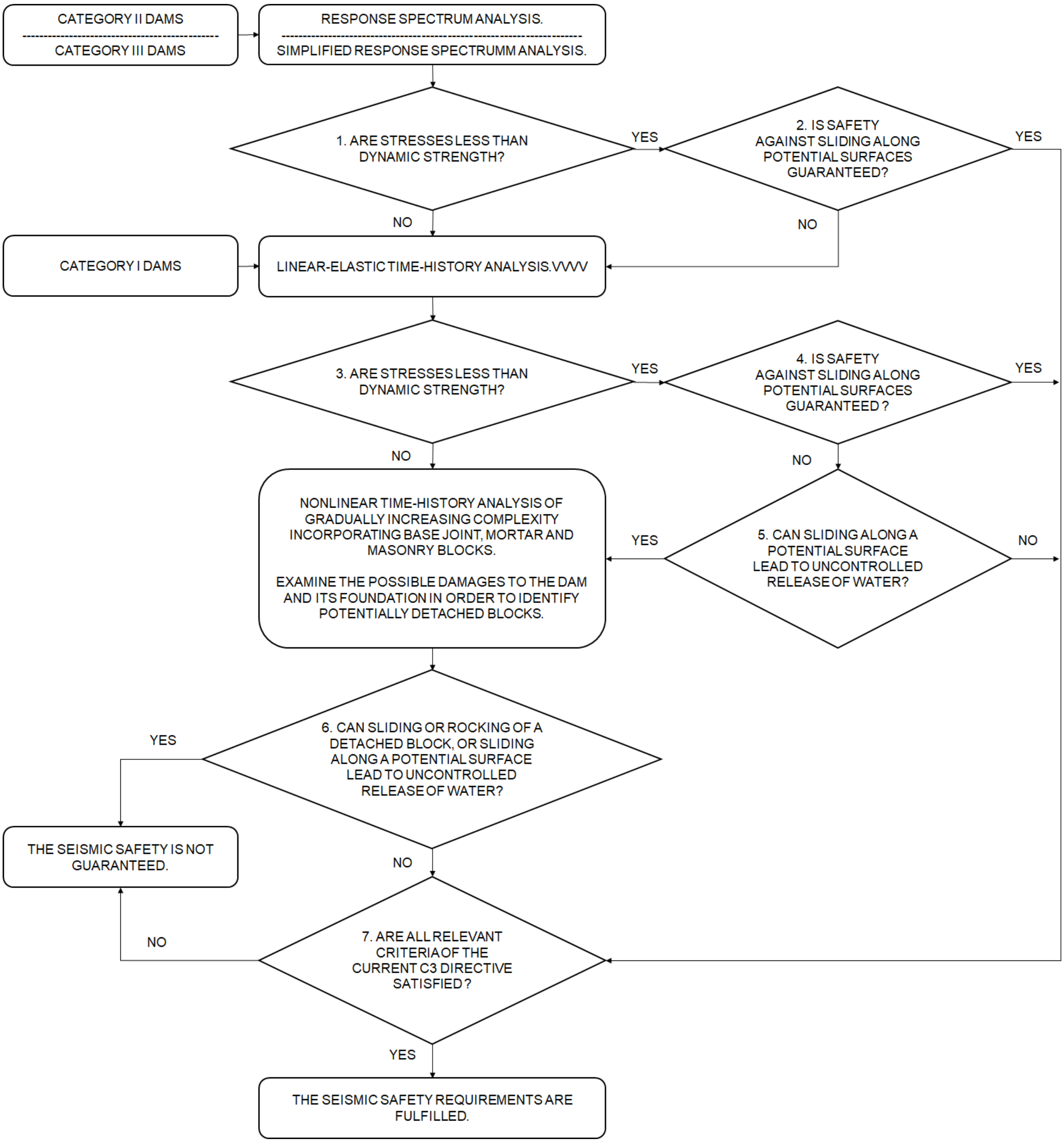
Remarks:

(1) Calibration of the material parameters should be carried out at the beginning of the seismic safety assessment.

(2) Pre-existing observed discontinuities and damages should be included in the model.

(3) In case the seismic safety assessment shows that the seismic safety is not guaranteed for the current state of the dam and mode of operation of the dam reservoir, measures shall be proposed to ensure the seismic safety.

Figure 3: Flowchart for seismic safety verification of concrete gravity dams, buttress dams and weirs (DCR: Demand Capacity Ratio)



Remarks:

(1) Calibration of the material parameters should be carried out at the beginning of the seismic safety assessment.

(2) Pre-existing observed discontinuities and damages should be included in the model.

(3) In case the seismic safety assessment shows that the seismic safety is not guaranteed for the current state of the dam and mode of operation of the dam reservoir, measures shall be proposed to ensure the seismic safety.

Figure 4: Flowchart for seismic safety verification of masonry dams

6.7 Notes regarding the verification methodology of concrete arch dams

6.7.1 Modelling of arch dams

6.7.1.1. For the analysis of arch dams, a three-dimensional model shall be used.

6.7.1.2. The hydrodynamic influence of the retained water can be modelled by means of water masses which are rigidly coupled to the dam. These masses are introduced to the model as point masses. Alternatively, more appropriate simulation of the hydrodynamic pressures during an earthquake can be achieved by explicitly modelling the dam-reservoir interaction. In this approach, the reservoir is explicitly modelled as a fluid domain.

6.7.1.3. For arch dams of all categories, i.e., Category I, Category II and Category III, the model shall also include a part of the foundation. The foundation may be modelled without or with mass. In both cases, the correct boundary conditions along the foundation borders shall be modelled.

6.7.2 Analysis of arch dams

6.7.2.1. For arch dams of all categories, the earthquake excitation shall be specified in two orthogonal horizontal directions and in the vertical direction. In case of time-history dynamic analysis, for each acceleration time-history, the combination of the horizontal components of the earthquake excitation shall be chosen so as to yield the most conservative results regarding the structural response.

6.7.3 Interpretation of results and safety assessment of arch dams

6.7.3.1. Introduction to the steps of results interpretation

The failure of a concrete arch dam subjected to strong ground motions can be related to overstressing, that can lead to cracking and/or shearing, large displacements, formation of detached blocks that may slide and/or overturn, increased uplift pressures in the abutment wedges, loss of sliding stability of abutment wedges, and/or loss of function of spillway and outlet structures. Most vulnerable to overstressing are the zones in the upper part of the arch dams where high dynamic amplifications can occur, as well as the zones where high stresses are already present due to the static loads. Large vibrations can cause excessive openings of the contraction joints which, when accompanied by significant cracking at the lift joints and/or the concrete parent material can lead to formation of detached blocks that may fail due to sliding or loss of rotational stability. In addition, sliding can occur along planes of discontinuity in the rock foundation that form kinematically capable abutment wedges. In the case of a gated spillway at the top of an arch dam, high stresses in the piers may lead to loss of flexural stability. A loss of the so-called active vault, i.e. the part of the arch dam body that transfers the thrust of the water in the dam reservoir onto the abutments, due to compression and/or shear overstressing and buckling may also lead to failure. In addition to the preceding, all other possible failure modes during and after the Safety Evaluation Earthquake related to the dam under study must be taken into account.

A reference case in which linear elastic behaviour is assumed for the arch dam-foundation system while the foundation is assumed to be massless must first be analysed before proceeding to more complex models. The results of this reference case shall serve as a basis for interpreting the outputs from further sophisticated analyses, as described in the following paragraphs.

6.7.3.2. **Assessment of stresses and strains**

6.7.3.2.1. For arch dams of all categories, an analysis of the stresses in the dam and its foundation shall be carried out. In case of a nonlinear analysis which includes a crack model, the strains and their interpretation are also needed.

6.7.3.2.2. Linear-elastic behaviour

The safety against overstressing in an arch dam and its foundation can be considered guaranteed, if the maximum tensile, compressive and shear stresses do not exceed the dynamic tensile, compressive and shear strength, respectively, of the concrete parent material, the lift joints, the contraction joints, the dam-foundation interface and the dam foundation, see Checks 1 and 3 of the flowchart in Figure 5. If this is the case, then it can be proceeded with evaluation of the safety against sliding of the abutment wedges, according to Paragraph 6.7.3.3, see Checks 2 and 4 of the flowchart in Figure 5.

6.7.3.2.3. Minor nonlinear behaviour in tension

If it is found that the tensile stresses in the dam body obtained from a linear-elastic analysis exceed the tensile strength of the concrete parent material and/or of the lift joints, and/or of the base joint, the safety against overstressing may be further evaluated by considering the ratio between the maximal tensile stress and the corresponding dynamic strength, the extent of the overstressed region, and the cumulative inelastic duration substantiated by means of scientifically founded methods, e.g., the demand-capacity ratio method, see Check 5 in Figure 5. This check is optional and may be skipped. In order to define cumulative inelastic duration, it is necessary to carry out a dynamic time-history analysis. If it is found that the level of nonlinear response is minor, then it can be proceeded with evaluation of the safety against sliding of the abutment wedges, according to Paragraph 6.7.3.3, see Checks 4 and 6 of the flowchart in Figure 5.

6.7.3.2.4. Nonlinear behaviour due to relative displacements at the contraction joints

If for a given acceleration time-history the check of the stresses in the body of an arch dam indicates high levels of inelastic response, i.e. if the criteria for minor nonlinear behaviour are not met, then the nonlinear effects due to relative displacements at the contraction joints shall first be modelled. More specifically, if high tensile stresses in the arch direction are calculated, it shall be assumed that these stresses cannot be transferred through the contraction joints and the joints can open and close. This requires carrying out a nonlinear time-history analysis. Furthermore, if tensile stresses in the direction normal to the dam-foundation interface are found to exceed the tensile strength of the base (peripheral) joint, i.e., of the concrete-rock contact, this should also be taken into account by incorporating the peripheral joint in the numerical model of the dam-foundation-reservoir system. The model of the peripheral joint shall allow for simulating at least opening and closing at the dam-foundation interface. In addition, for contraction joints with shear keys, it should be checked whether large joint openings may prevent the shear keys from closing. The safety against overstressing in the arch dam and its foundation can be considered ensured, if the shear keys close after each joint opening and if the maximum tensile, compressive and shear stresses do not exceed the dynamic tensile, compressive and shear strength, respectively, of the concrete parent material, the lift joints and the foundation, as applicable, see Check 7 in Figure 5. If this is the case, then it can be proceeded with evaluation of the safety of abutment wedges against sliding according to Paragraph 6.7.3.3, see Checks 8 and 9 of the flowchart in Figure 5.

6.7.3.2.5. Highly nonlinear behaviour

If for a given acceleration time-history the safety criteria related to a nonlinear model with contraction joints are not met, further nonlinear analyses with gradually increasing complexity shall be carried out. In addition to the contraction joints and the base (peripheral) joint, these analyses can incorporate nonlinear models of the lift joints and of the mechanisms of crack initiation and propagation in the concrete parent material in order to identify potentially detached blocks. Conservative assumptions shall be made regarding the uplift at the surfaces where rupture due to overstressing and permanent displacements have been identified. It shall be verified whether permanent displacements and rotational movements of potential detached blocks are sufficiently small not to endanger the safety of the dam during the earthquake and the aftershocks, as well as in the post-earthquake period, so that the behavioural objectives according to Paragraph 2 are fulfilled, see Check 10 in Figure 5. In case of overstressing due to high compressive and/or shear stresses, the behaviour of the active arches shall be verified in order to ensure that the safety against buckling is ensured in the post-earthquake period, see Check 10 in Figure 5. Aftershocks shall be considered for seismic safety evaluation of Category I concrete arch dams only if the nonlinear analysis as prescribed in the current paragraph is to be carried out.

6.7.3.2.6. The safety against overstressing in the foundation shall consider the bearing capacity of the foundation material, as well as the presence of any discontinuities of lower strength. If the safety against local failure in the foundation is not ensured, it shall be verified whether the behavioural objectives according to Paragraph 2 are fulfilled despite the resulting structural response.

6.7.3.2.7. Stresses and possible damages on elements connected to an arch dam (in particular spillways, orifices, outlets and other ancillary installations) shall be assessed and considered in the seismic safety evaluation process and, if necessary, in defining measures to guarantee the seismic safety.

6.7.3.2.8. The safety against flexural failure of the reinforced-concrete piers of a gated spillway shall be checked.

6.7.3.2.9. If a seismic belt and/or steel skin reinforcement exist in the dam, it should be included in the model and the elongation of the steel bars of the belt and/or the reinforcement shall be analysed and the possible plastification must be determined.

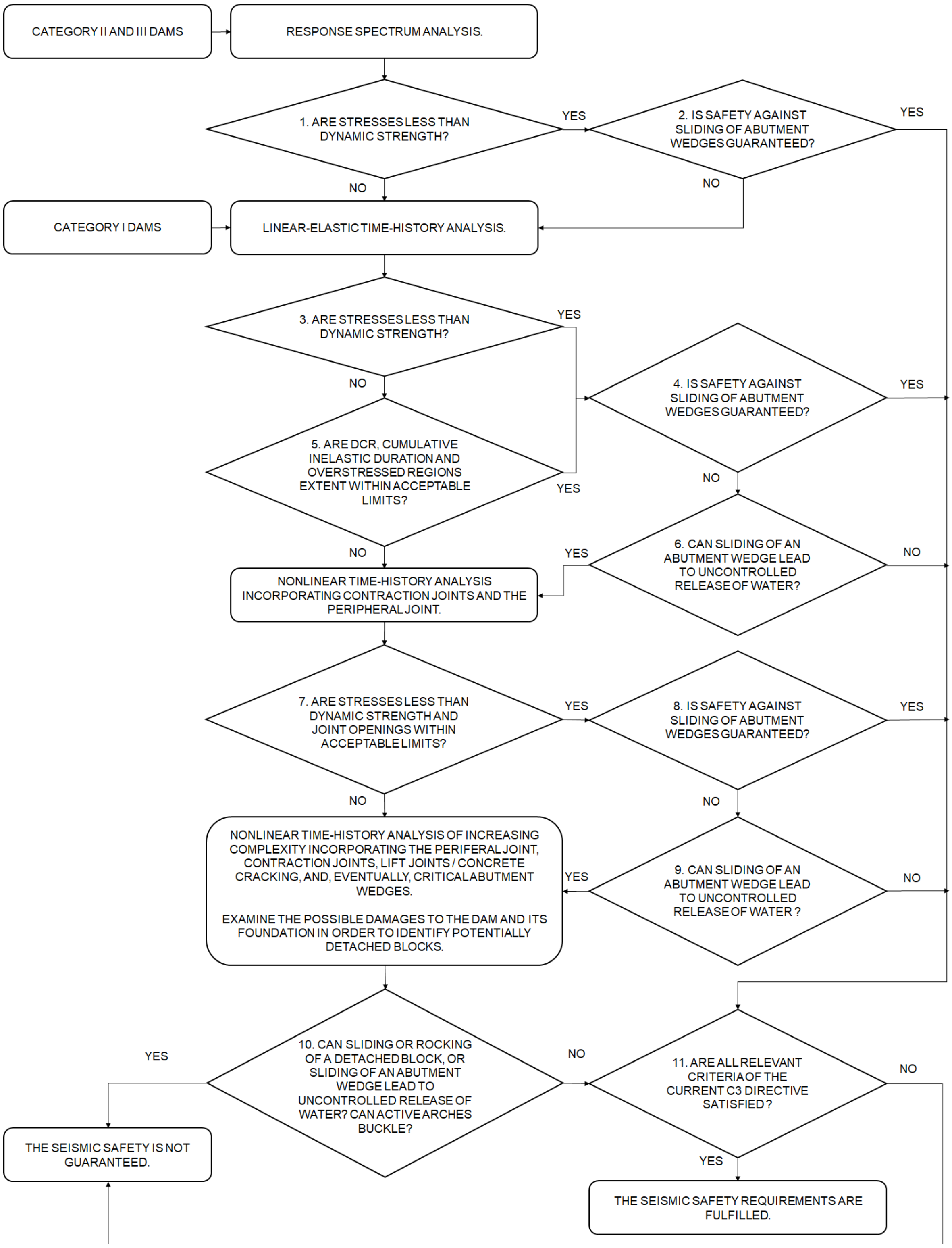
6.7.3.3. Assessment of safety against sliding of abutment wedges

6.7.3.3.1. The safety against sliding of abutment wedges shall be verified based on the results obtained from the corresponding stress analysis. If it is found that sliding is possible, the calculation of reversible and permanent sliding displacements along potential sliding surfaces shall be carried out in order to verify whether the behavioural objectives according to Paragraph 2 are fulfilled.

6.7.3.3.2. For aftershock and post-earthquake stability of the abutment wedges, any change in the uplift conditions, e.g., due to damaged area, presence of cracks and/or lower efficiency of the drainage system shall be considered. Based on this evaluation, the uplift forces used in the global stability analysis shall be updated accordingly.

6.7.3.3.3. In case of an arch dams in a wide valley, the safety against sliding of the whole dam shall also be verified.

6.7.3.3.4. A flowchart to be followed for the analysis, interpretation of results and safety verification of concrete arch dams is given in Figure 5.



Remarks:
(1) Calibration of the material parameters should be carried out at the beginning of the seismic safety assessment.
(2) Pre-existing discontinuities and damages should be included in the model.
(3) The post-seismic safety of the active arches (active vault) against buckling shall be checked.
(4) In case the seismic safety assessment shows that the seismic safety is not guaranteed for the current state of the dam and mode of operation of the dam reservoir, measures shall be proposed to ensure the seismic safety.

Figure 5: Flowchart for seismic safety verification of concrete arch dams.

6.8 Notes regarding the verification methodology for embankment dams

6.8.1 Modelling of embankment dams

6.8.1.1. The potential of liquefaction for all categories of embankment dams shall be checked and ruled out according to Paragraph 5.3.8. In case the potential of liquefaction is existing, the consequences of liquefaction on the safety of the water retaining facility shall be quantified using detailed laboratory and in-situ material characterisations combined with numerical analysis.

6.8.1.2. For embankment dams, a two-dimensional analysis of the relevant cross-section is usually sufficient. In narrow valleys or with variable subsoil conditions, different cross-sections must be considered, or a three-dimensional analysis may be necessary. Special attention has to be given to the abutment areas.

6.8.1.3. For three-dimensional modelling of embankment dams, the three components of the earthquake excitation, i.e., the two orthogonal horizontal components and the vertical component shall be taken into account. In case of time-history analysis, for each acceleration-time-history, the direction of the horizontal components shall be chosen in such a manner so as to yield the maximum structural response.

6.8.1.4. For two-dimensional modelling of embankment dams, the horizontal and the vertical components shall be taken into account. In case of time-history analysis, for each acceleration-time-history, the horizontal component shall be chosen in such a manner so as to yield the maximum structural response.

6.8.1.5. The vertical component of the seismic excitation may be neglected for Category III embankment dams that satisfy the conditions of Paragraph 6.3.4.3.1 let. a.

6.8.1.6. In the analysis of embankment dams, particular attention must be given to the location of the groundwater table, and pore water pressure distribution in the dam body and in the foundation layers.

6.8.1.7. The selected material models used in numerical analyses must accurately reflect the actual behaviour of the material under investigation to the greatest extent possible. The model parameters need to be calibrated and verified.

6.8.1.8. Potential generation of excess pore water pressures resulting from seismic loading shall be considered for all dam categories.

6.8.1.9. In the analysis of the dynamic behaviour of the embankment dams using time-history analysis, the vertical excitation must be taken into account for all embankment dam categories.

6.8.1.10. The potential of settlement due to densification or consolidation of the material of the dam body and the foundation as a result of the reduction in void ratio during or after the seismic loading shall be analysed.

6.8.1.11. For Category II dams the sliding block analyses shall be conducted based on the estimates of the accelerations developed from 2D dynamic analyses for the relevant slip surfaces. For the purpose of calculating the sliding displacements, the vertical component of the acceleration may be neglected.

6.8.1.12. For Category III dams that do not fulfil the criteria of Paragraph 6.3.4.3.1 let. a, the sliding block analyses shall be conducted based on the estimates of the accelerations developed from two-dimensional dynamic analyses for the relevant slip surfaces. For the purpose of calculating the sliding displacements, the vertical component of the acceleration may be neglected.

6.8.1.13. For Category III dams that fulfil the criteria of Paragraph 6.3.4.3.1 let. a, the acceleration in the relevant sliding body, as well as permanent deformations, may be determined by applicable semi-empirical methods.

6.8.1.14. A rapid drawdown of water level in the reservoir can cause unfavourable seepage flow within the embankment adversely affecting the stability of the dam. The potential and consequences of rapid drawdown of the reservoir after an earthquake must be evaluated.

6.8.2 Analysis, interpretation of results and safety assessment of embankment dams

6.8.2.1. To reliably assess seismic performance of an embankment dam, an appropriate evaluation of the expected seismic deformations is required. For all dam categories the calculated displacements shall be critically assessed, with particular emphasis on determining the reduction of the freeboard following the earthquake.

6.8.2.2. If permanent deformations are to be expected after the seismic action, in particular the following points shall be assessed:

- To prevent overflow of the dam due to direct or indirect seismic impacts (e.g., due to a sliding of a mass of soil or/and rock in the reservoir rims), a sufficiently large freeboard must be maintained. Earthquake-induced sliding displacements and settlements of the dam crest shall be taken into consideration in this process.
- Internal erosion must also be excluded in the deformed state.

6.8.2.3. The acceptable permanent crest settlements, due to the combined effects of densification, consolidation and formation of slip surfaces, due to seismic action, should not exceed 25% of the safety freeboard as defined in Paragraph 2.3.2. of the Directive Part C2: *Flood safety and lowering the reservoir water level*.

6.8.2.4. If seismic action could lead to the formation of cracks or other failure mechanisms that might alter the seepage regime in the dam, the potential for erosion through these anomalies in the embankment or the foundation must be investigated.

6.8.2.5. The stability of structures or elements (e.g., spillway walls, or conduits) adjacent to, within, or under the embankment which could potentially cause overtopping, erosion, or infiltration shall be verified. The safety against flexural failure of the reinforced-concrete piers of a gated spillway shall be checked.

6.8.2.6. For zoned embankments with a core, the potential for internal erosion after permanent deformation can be excluded if the following conditions are met:

i. Between the individual dam zones, the filter functionality is still satisfied.

ii. The core is still sufficiently covered and is made of material that can withstand the imposed deformations without significant change in its permeability.

iii. The residual thickness of the filter and drainage layers in the deformed state is at least half the thickness in the undeformed state.

6.8.2.7. For embankment dams with an upstream surface lining the following aspects need to be checked:

i. The integrity of the lining must be assessed.

ii. If damage to the lining may not be excluded, the resulting risks, such as internal erosion and changes in stability due to potential formation of a seepage surface in the dam body, shall be evaluated.

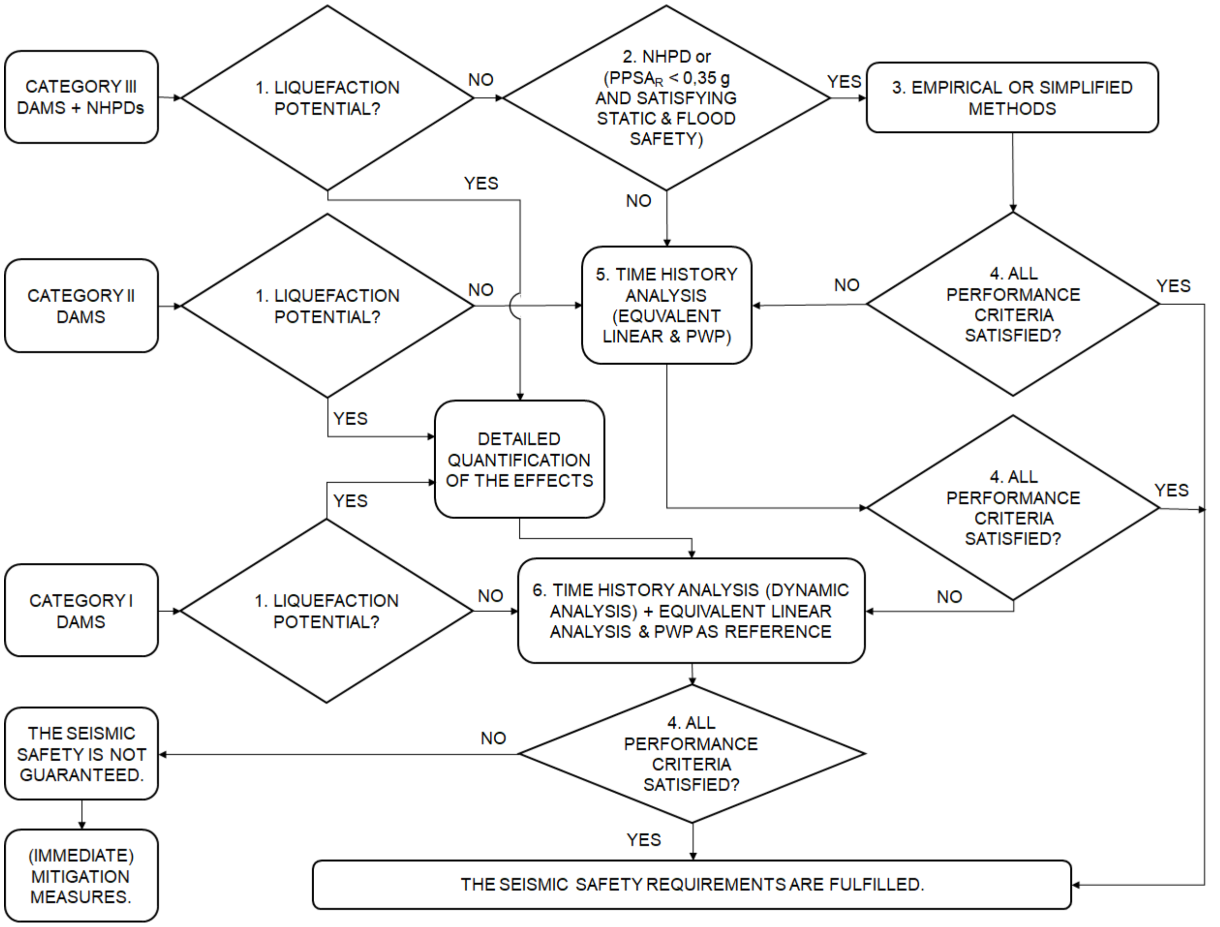
iii. The residual thickness of the filter and drainage layers beneath the lining in their deformed state shall be at least half the original thickness in the undeformed state.

iv. Between the individual dam zones, the filter functionality is still satisfied.

6.8.2.8. The stability analysis of the Category I embankment dams for aftershocks (Paragraph 4.1.2.2) shall incorporate verification methodologies listed in Paragraph 6.3.4.3.3. The effect of generated pore water pressure due to seismic action need to be considered.

6.8.2.9. The post-earthquake static stability analysis may incorporate standard slope stability methods with the partial factors mentioned in Paragraph 4.1.2.3 for all categories of water retaining facilities. The effects of generated pore water pressure due to seismic action need to be considered.

6.8.2.10. A flowchart to be followed for the analysis, interpretation of results and safety verification of embankment dams is given in Figure 6.



Remarks based on the numbers stated in the flowchart:

1. Assessment of the liquefaction potential (ref. Paragraph 5.3.8).
2. Are all conditions listed in 6.3.4.3.1 let. a satisfied? NHPD stands for Natural Hazard Protection Dams.
3. As described in 6.3.4.3.1 let. a. shear strains shall be checked, and the initial assumption of damping shall be verified accordingly.
4. Performance criteria and behavioural goals are described in Paragraphs 2 and 6.8.2.
5. As described in 6.3.4.3.1 let. b or 6.3.4.3.2. Eq. Li & PWP stands for time-history analysis using equivalent linear method integrated with sliding block analysis. The potential effects of generation of excess pore water pressures shall be considered.
6. As described in 6.3.4.3.3.

Figure 6. Flowchart for seismic safety verification of embankment dams

7 Literature

Bergamo P., L. Danciu, F. Panzera and D. Fäh (2022) Basis for the determination of waveforms for the sites of dams in Switzerland - subproject 1: disaggregation of seismic hazard for return periods of 1000, 5000, 10000 years. Technical Report SED 2021/11, Swiss Seismological Service, ETH Zurich. DOI: 10.3929/ethz-b-000517545

Poggi, Valerio & Edwards, Benjamin & Fäh, Donat. (2011). Derivation of a Reference Shear-Wave Velocity Model from Empirical Site Amplification. *Bulletin of The Seismological Society of America - BULL SEISMOL SOC AMER*. 101. 258-274. 10.1785/0120100060.

Wiemer, Stefan & Danciu, Laurentiu & Edwards, Benjamin & Marti, Michèle & Fäh, Donat & Hiemer, Stefan & Woessner, Jochen & Cauzzi, Carlo & Kästli, Philipp & Kremer, Katrina. (2016). Seismic Hazard Model 2015 for Switzerland (SUIhaz2015).

Appendix A: Maps of $PPSA_R$ for 3 different earthquake return periods

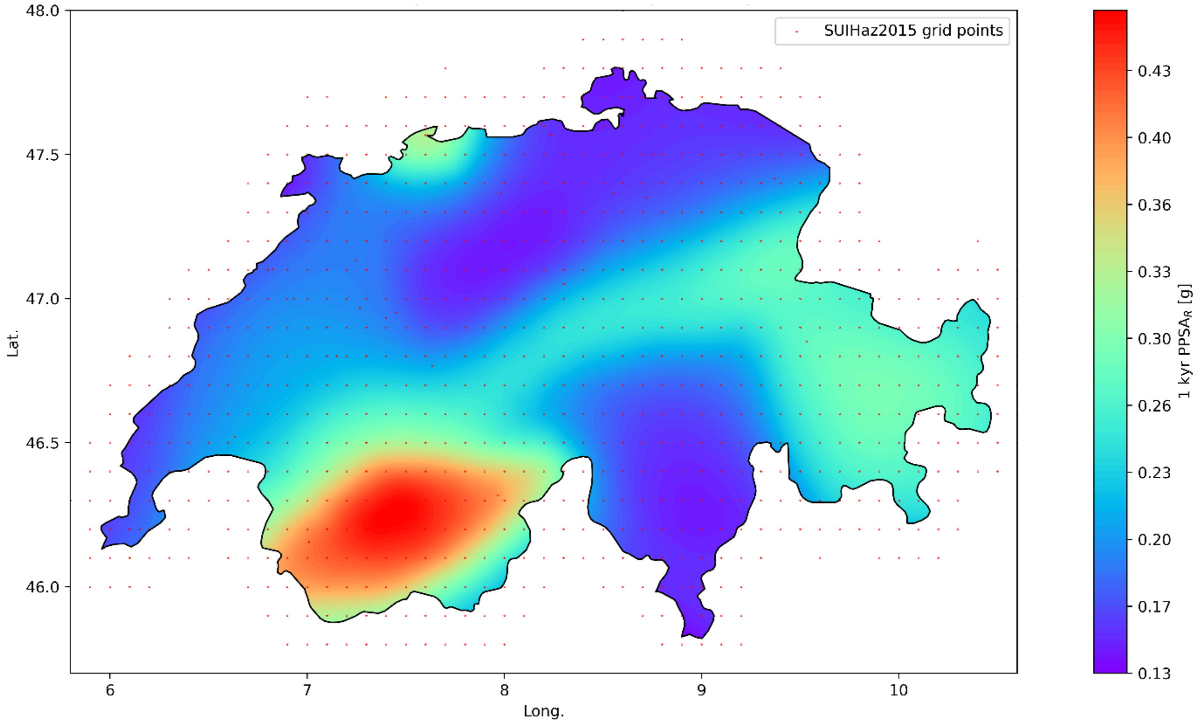


Figure A1: The map of $PPSA_R$ for Safety Evaluation Earthquake with return period of 1’000 years

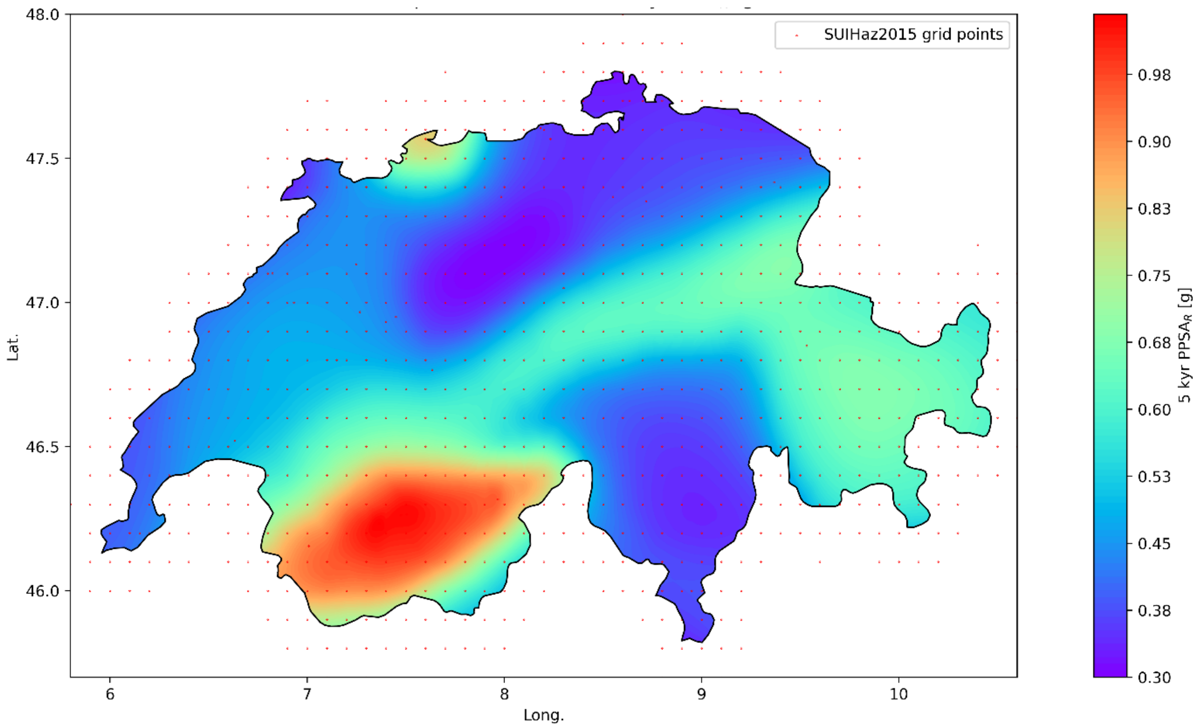


Figure A2: The map of PPSAR for Safety Evaluation Earthquake with return period of 5’000 years

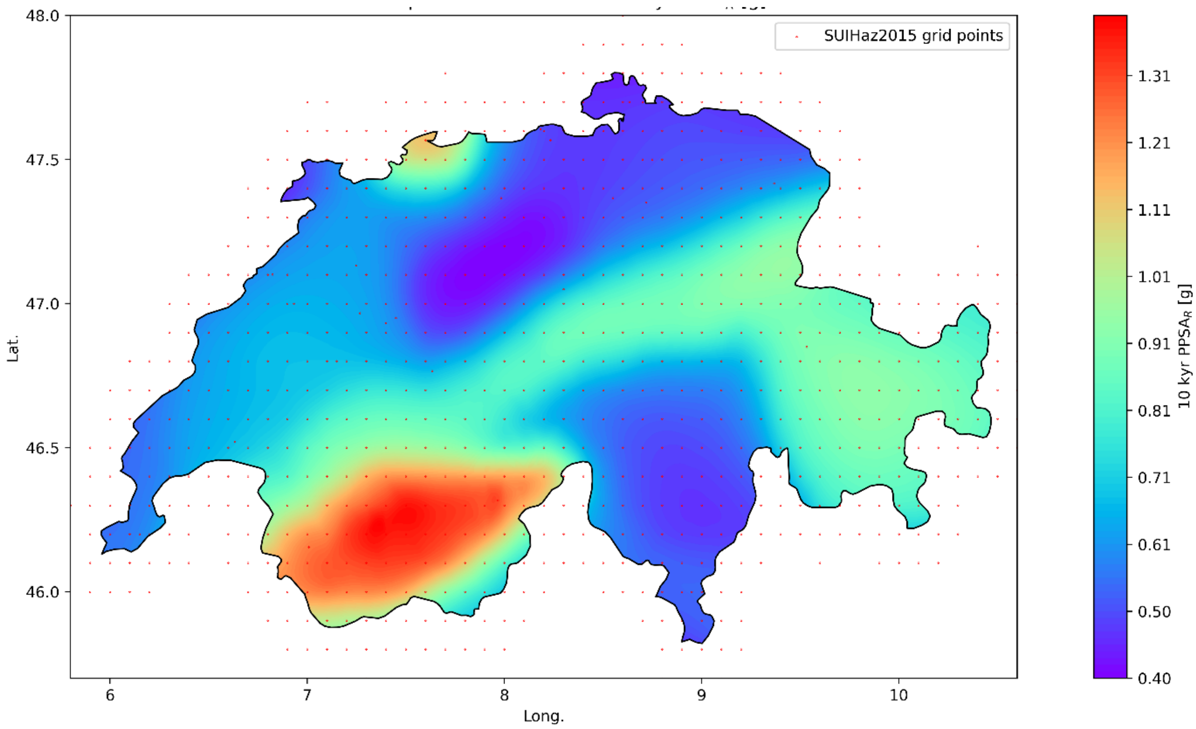


Figure A3: The map of PPSAR for Safety Evaluation Earthquake with return period of 10’000 years

Appendix B: Seismic zones and deaggregation of the hazard

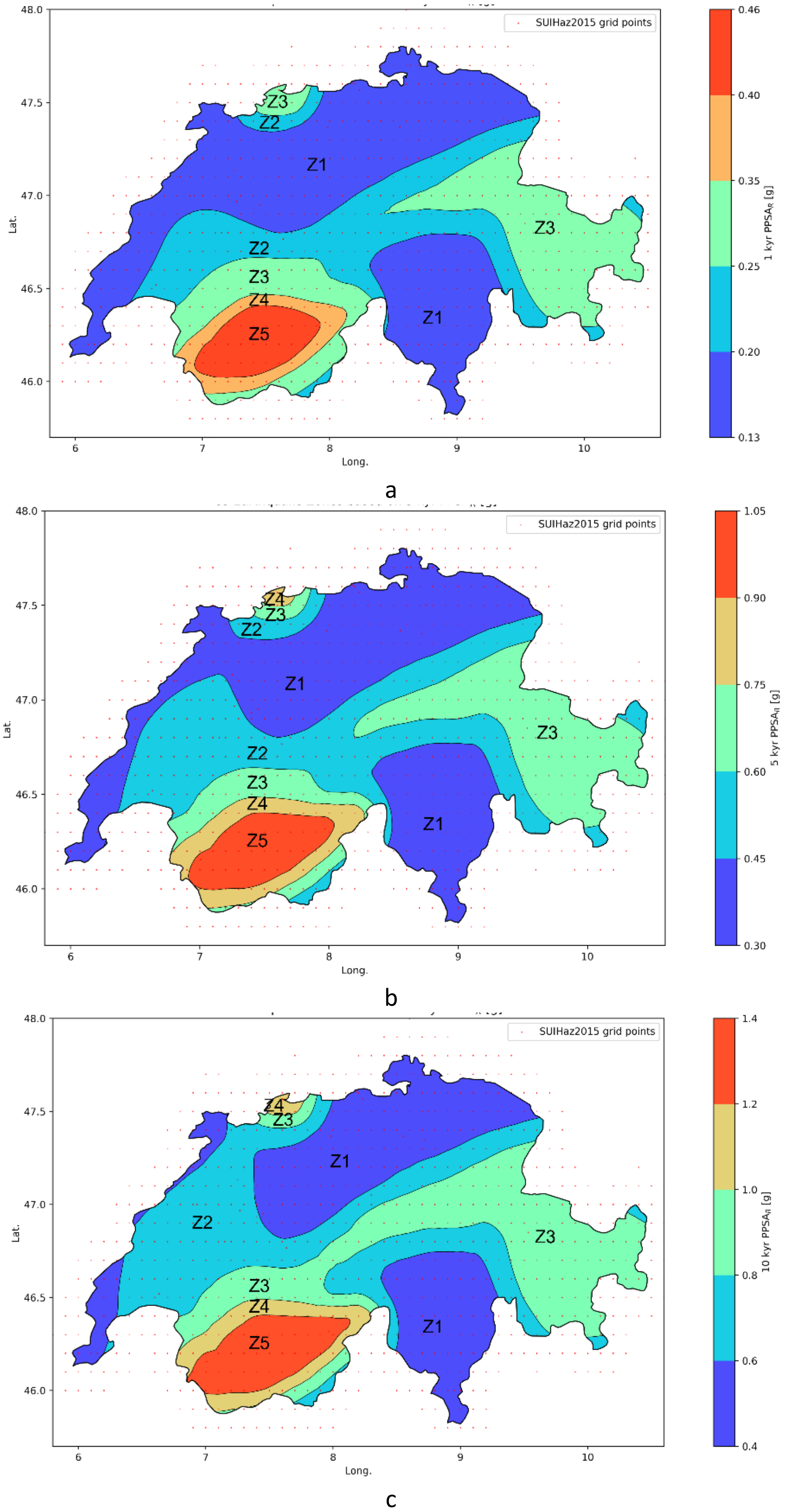


Figure B1: Seismic zones for water retaining facilities in Switzerland. Top: for return period of 1'000 years. Middle: for return period of 5'000 years. Bottom: for return period of 10'000 years.

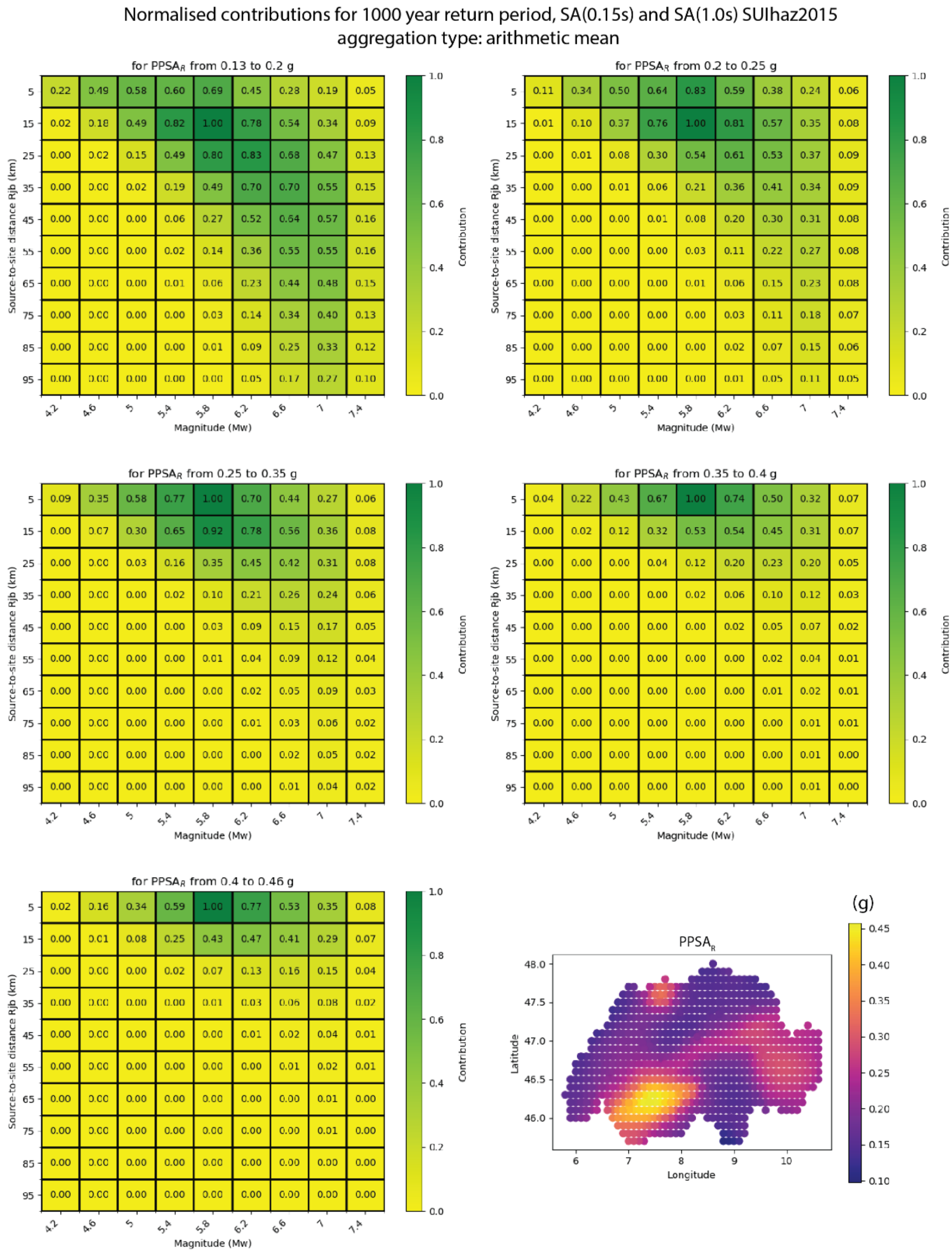


Figure B2: Normalised heatmaps of the sum of contributions to the exceedance of two intensity measures (IMT) (spectral acceleration at periods of 0.15 s and 1.0 s), derived from the average values over all SUIhaz2015 nodes belonging to a given seismic zone defined in Figure B1a (RP = 1'000 years). The colour and the number in each cell indicate the relative significance of each magnitude-distance combination. The relative significance is equal to 1 for the magnitude-distance combination that reaches the highest mean value of contribution to the exceedance of intensity measures (IMT). The relative significance is equal to 0 for the magnitude-distance combination with no contribution to the exceedance of any intensity measure at any node. For information on the data and methodology please refer to Bergamo et al. (2022). Bottom-right: distribution of the PPSA$_R$ (g) for the individual nodes and for a return period of 1'000 years.

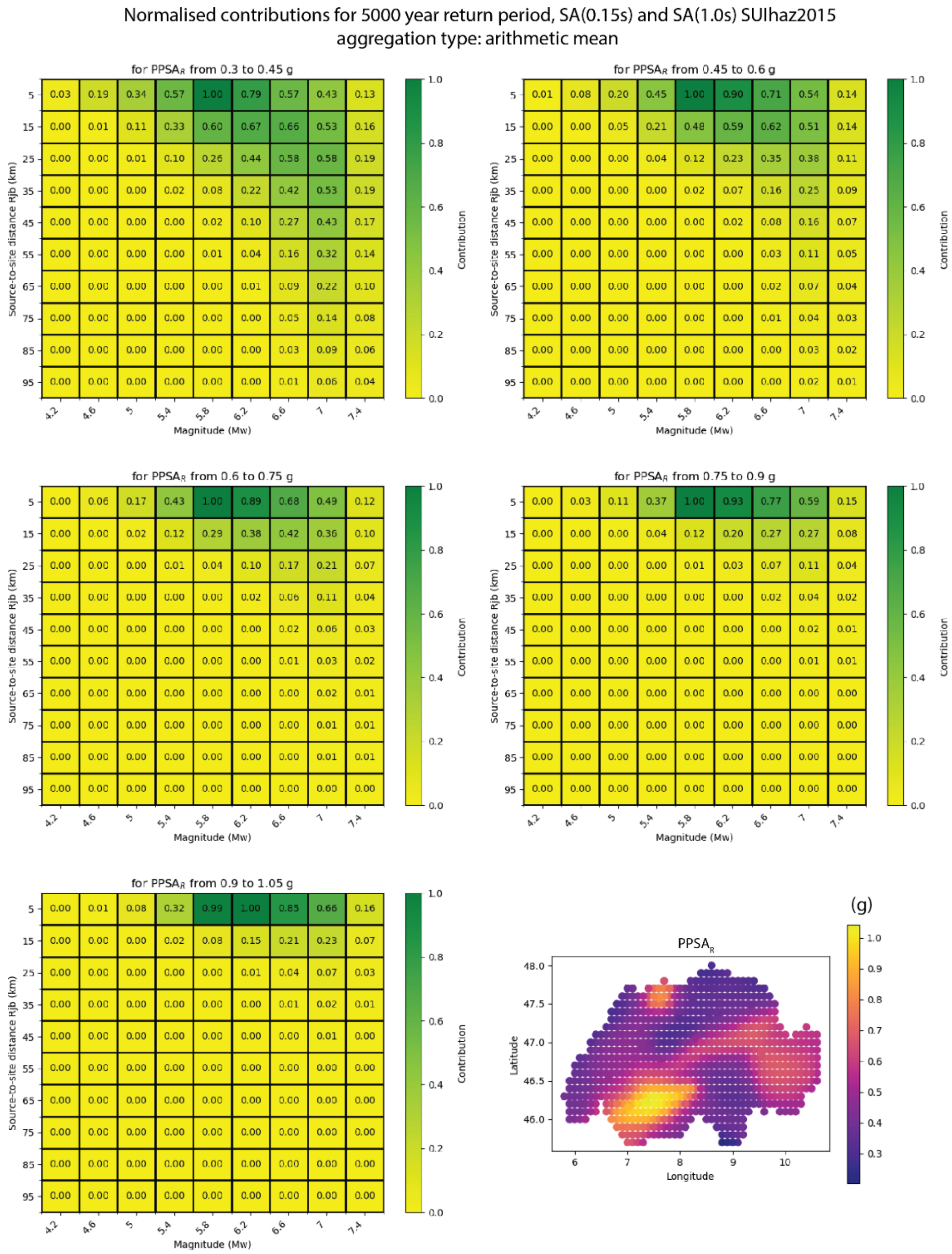


Figure B3: Normalised heatmaps of the sum of contributions to the exceedance of two intensity measures (IMT) (spectral acceleration at periods of 0.15 s and 1.0 s), derived from the average values over all SUIhaz2015 nodes belonging to a given seismic zone defined in Figure B1b (RP = 5'000 years). The colour and the number in each cell indicate the relative significance of each magnitude-distance combination. The relative significance is equal to 1 for the magnitude-distance combination that reaches the highest mean value of contribution to the exceedance of intensity measures (IMT). The relative significance is equal to 0 for the magnitude-distance combination with no contribution to the exceedance of any intensity measure at any node. For information on the data and methodology please refer to Bergamo et al. (2022). Bottom-right: distribution of the PPSAR (g) for the individual nodes and for a return period of 5'000 years.

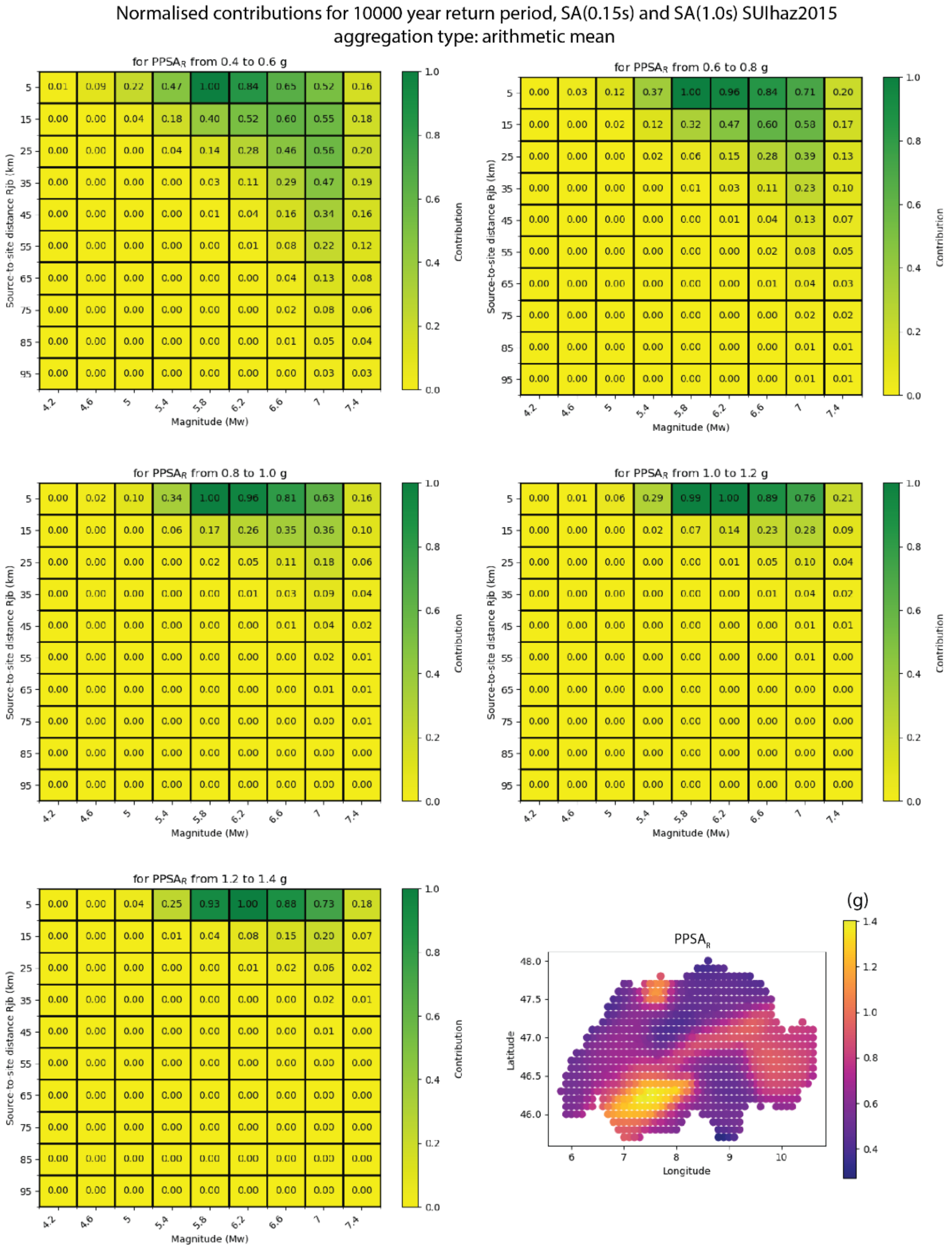


Figure B4: Normalised heatmaps of the sum of contributions to the exceedance of two intensity measures (IMT) (spectral acceleration at periods of 0.15 s and 1.0 s), derived from the average values over all SUIhaz2015 nodes belonging to a given seismic zone defined in Figure B1c (RP = 10'000 years). The colour and the number in each cell indicate the relative significance of each magnitude-distance combination. The relative significance is equal to 1 for the magnitude-distance combination that reaches the highest mean value of contribution to the exceedance of intensity measures (IMT). The relative significance is equal to 0 for the magnitude-distance combination with no contribution to the exceedance of any intensity measure at any node. For information on the data and methodology please refer to Bergamo et al. (2022). Bottom-right: distribution of the PPSA$_R$ (g) for the individual nodes and for a return period of 10'000 years.

Appendix C: Requirements on site-specific determination of seismic action for water retaining facilities in Switzerland

C1 Introduction

This Appendix outlines minimum requirements of the Swiss Federal Office of Energy (SFOE) for site response analysis (*SRA*) and probabilistic seismic hazard analysis (*PSHA*) for a specific water retaining facility. In general, three options are foreseen for determination of seismic action to be used as the input for seismic safety verification of a water retaining facility in Switzerland and schematically shown in Figure C1. The specifications of each of these three options are detailed in the following paragraphs.

In Directive Part C3, the seismic action at any location in Switzerland is determined by combining two elements:

- seismic hazard on the reference rock[2] (Paragraph 4.2)
- site effects that, relative to the reference rock, influence the ground motion at a specific location (Paragraph 4.3.2).

For the determination of the seismic hazard on the reference rock, two options are foreseen:

1) use of the SUIhaz2015 model (Wiemer et al., 2016) (Path 1 in Figure C1),
2) conducting a Probabilistic Seismic Hazard Analysis (*PSHA*) (Path 2 in Figure C1).

For the determination of the site effects, also two options are foreseen:

a) use of the amplification factors as summarised in Table 3 (Paragraph 4.3.4),
b) performing a Site Response Analysis (*SRA*), i.e., evaluation of the seismic response of the site, using numerical ground-motion simulation techniques preferably accompanied by in-situ measurements.

It shall be noted that the completion of Paths 1a and 1b constitutes a prerequisite to determination of seismic action based on Path 2.

C1.1 Path 1a

According to Paragraph 4.2, the seismic hazard on the reference rock may be taken in the form of the mean Uniform Hazard Spectrum (*UHS*) on the “Swiss Reference Rock” (Figure C2). The “Swiss Reference Rock” is defined by Poggi et al. (2011) and is used in the SUIhaz2015 model (Wiemer et al., 2016). According to Paragraph 4.3.2, site conditions may be accounted for by classifying the foundation into different ground classes. An amplification factor, S_x, is defined for each ground class. The amplification factor S_x is used to scale the peak spectral acceleration on the Swiss Reference Rock ($PPSA_R$), to the peak spectral acceleration expected on site ($PPSA_x$). $PPSA_x$ is used to construct the elastic response spectrum according to the equations given in Paragraph 4.3.4.2. It should be noted that in the process of defining the amplification factors S_x various sources of uncertainties, including the uncertainties of determination of the ground class, have been taken into account. Therefore, a certain level of conservatism is inherent in this approach. Determination of the seismic action following Path 1a is a prerequisite for assessing the seismic safety of all water retaining facilities in Switzerland, it will serve as a reference in cases where a different path is to be followed.

[2] In Directive Part C3, the shear-wave velocity profile of the reference rock has a minimum shear-wave velocity of $V_s \geq 1000$ m/s, a minimum time-averaged shear-wave velocity $V_{s30} \geq 1105$ m/s and should follow the form defined by Equation C1. In seismic hazard analysis, the shear-wave velocity profile of the reference rock is used as the target velocity profile in the host-to-target adjustment.

C1.2 Path 1b

The Directive Part C3 also allows for determining the seismic action based on local conditions where comprehensive detailed characteristics of the site (including shear wave velocity profile and other characteristics relevant for the seismic action) are available. The site effects can be determined with a site response analysis (*SRA*), instead of applying the amplification factor S_x based on an assigned ground class, as described in Path 1a. After having completed Path 1a and having obtained a first estimate of the seismic action at the location of the dam, the seismic hazard on the reference rock defined by the SUIhaz2015 model (Wiemer et al., 2016) may be combined with a site response analysis. This process delivers a more realistic view of the site effects present at the location of the water retaining facility and can substitute the results of Path 1a. The minimum requirements of the SFOE for site response analysis are detailed in paragraph C2.

C1.3 Path 2

In both paths 1a and 1b, the seismic hazard is defined based on the SUIhaz2015 model (Wiemer et al., 2016) for the Swiss Reference Rock. After having completed Path 1b, for justified cases it is possible, or necessary, to determine the local seismic hazard based on a site-specific probabilistic seismic hazard analysis (*PSHA*). For this purpose, the hazard shall be determined on the reference rock, and it is possible to use a reference rock other than the Swiss Reference Rock. The results of the probabilistic seismic hazard analysis (*PSHA*) are then integrated with Site Response Analysis (*SRA*). The possibility for a site-specific seismic hazard analysis, is described in path 2 of Figure C1.

If Path 2 is followed, it is expected that the seismic hazard is appropriately and robustly investigated, that the evaluation is objective, not influenced by cognitive bias, that the overall process is fully documented and that the results are reproducible. Therefore, it is required that the seismic hazard assessment be accompanied by a participatory peer review process and for this purpose it is advisable that the probabilistic seismic hazard analysis (*PSHA*) be carried out in accordance with the SSHAC[3] updated guidelines, which provide a multi-level assessment process.

The decision on the reference procedural SSHAC level and deviations from SSHAC guidelines are the responsibility of the operator and shall be made in consultation with experts and the supervisory authority. The minimum requirements of the SFOE for probabilistic seismic hazard analysis (*PSHA*) are detailed in Paragraph C3.

[3] The information on SSHAC (Senior Seismic Hazard Analysis Committee) updated guidelines can be found in the document "Updated Implementation Guidelines for SSHAC Hazard Studies" (NUREG-2213) at the following link:
https://www.nrc.gov/reading-rm/doc-collections/nuregs/staff/sr2213/index.html.

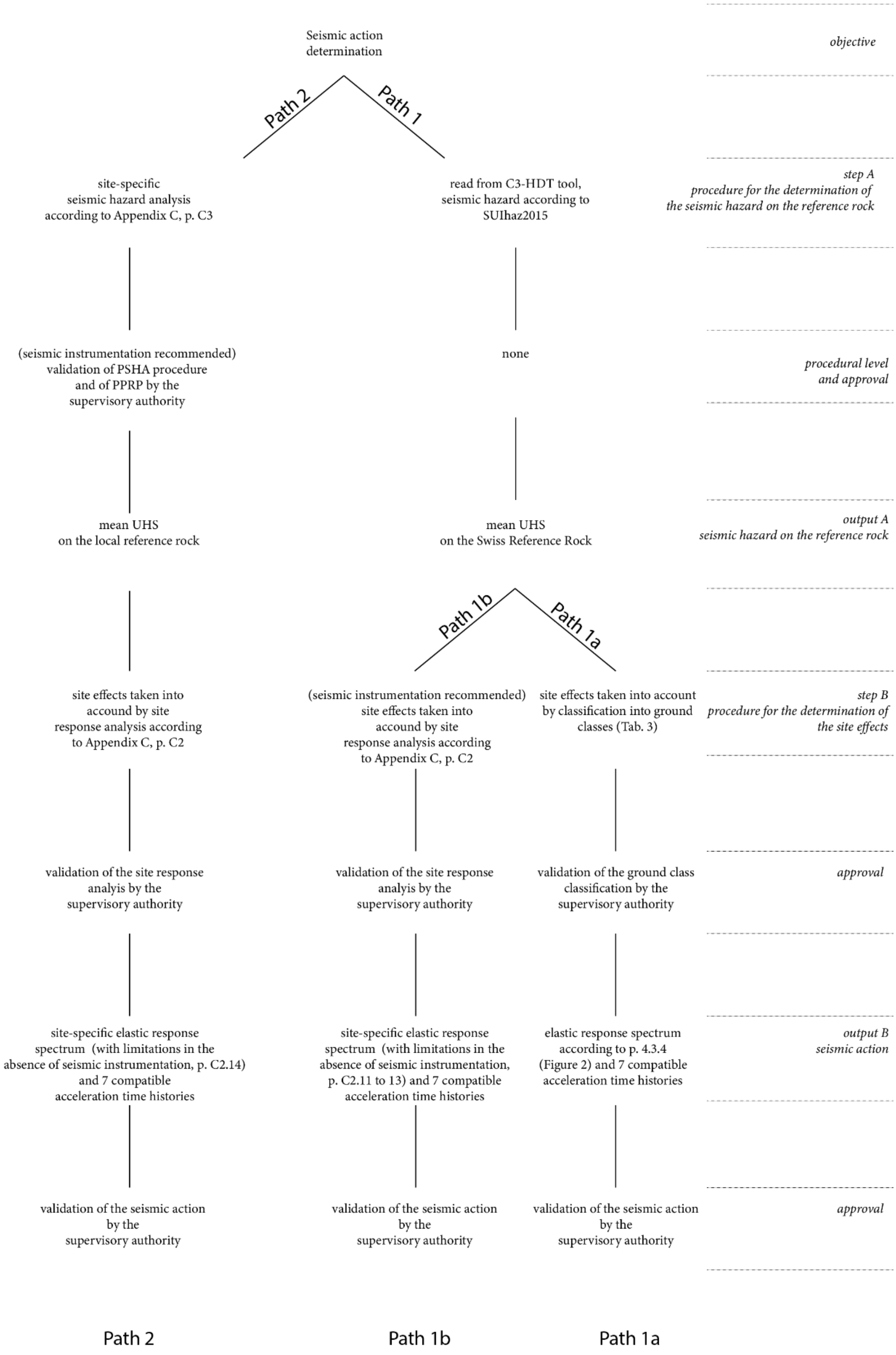


Figure C1: Procedures for the determination of the seismic action

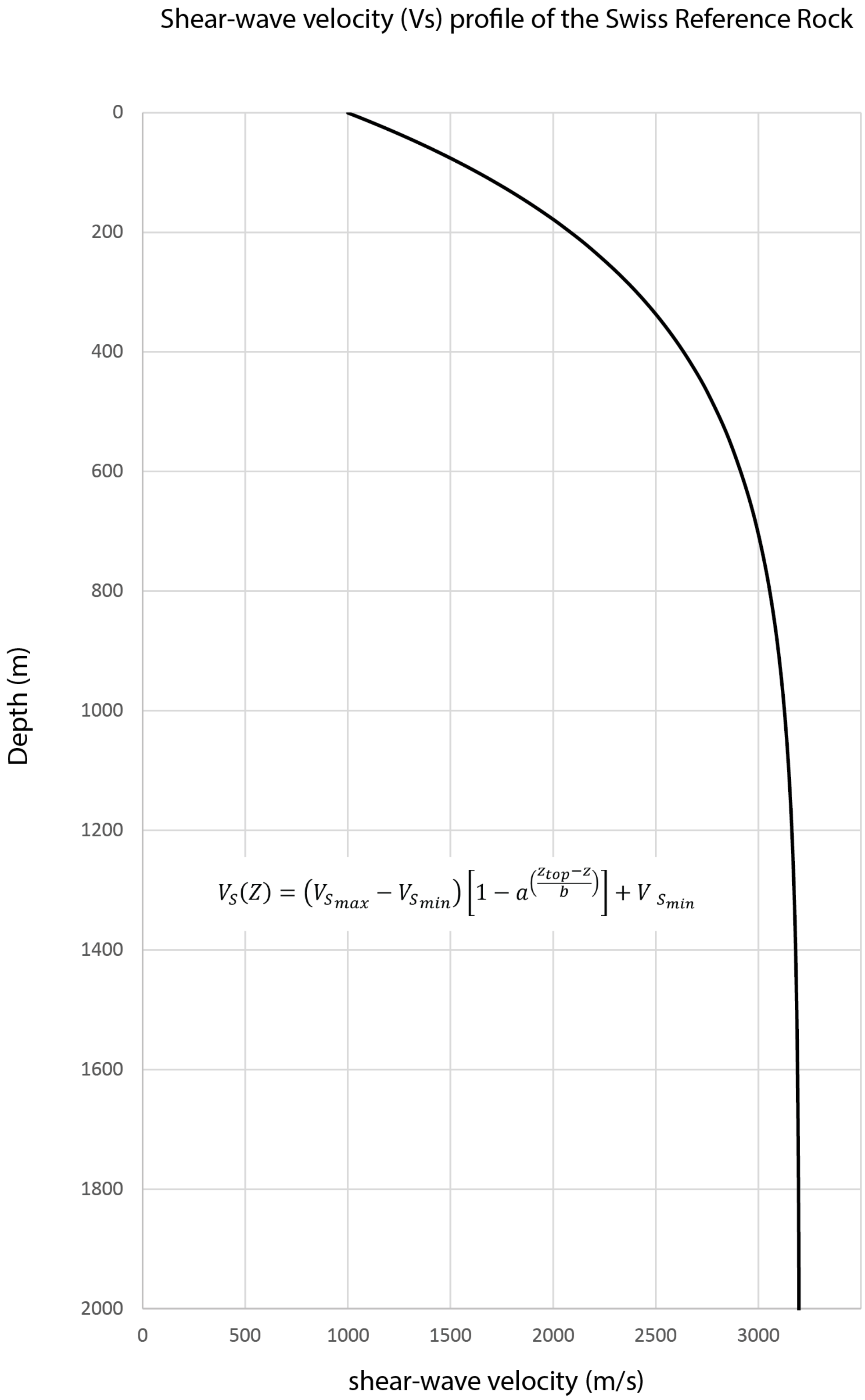


Figure C2: Shear-wave velocity profile of the Swiss Reference Rock from Poggi et al. (2011). Vs(Z): shear-wave velocity at depth Z; V_{Smax} = 3200 m/s; V_{Smin} = 1000 m/s, a = 1.3046; b = 78.1674 and Z_{top} = 0.5 m (starting depth of the shear-wave velocity profile), for further details please refer to Poggi et al. (2011).

C2 Requirements for site response analyses (*SRA*)[4]

C2.1 This section specifies the requirements for the site response analysis and the development of the corresponding elastic response spectrum.

C2.2 The site-specific elastic response spectrum developed from the site response analysis can replace the elastic response spectrum from Path 1a as described in Paragraph C1.1.

C2.3 The reference hazard, i.e., the seismic hazard on the reference rock, is defined as the mean Uniform Hazard Spectrum (*UHS*) obtained either from the SUIhaz2015 model (Wiemer et al., 2016) (Path 1, in Figure C1) or from a dedicated probabilistic seismic hazard analysis (Path 2, in Figure C1 and as described in Paragraph C2).

C2.4 Site response analysis (*SRA*) must be documented in a comprehensible manner with regard to the principles, methodology, interim results and decisions. Recording seismic events by means of permanent or temporary seismic instrumentation is recommended. It allows a verification of the site response analysis and a reduction of epistemic uncertainties.

C2.5 In particular the following points must be appropriately addressed and documented:

- geological and geotechnical conditions;
- aleatory and epistemic uncertainties;
- sensitivity analysis considering the uncertainties;
- geophysical description of the soil layers and the underlaying rock profile, including:
  - H/V measurements and their interpretation;
  - S-wave measurements and their interpretation:
    - The desired exploration depth should be established in a preliminary step based on local geology and H/V measurements (f_0). The site should be investigated to a sufficient depth to locate and characterize the deepest impedance contrast relevant for the analysis.
    - A combination of active and passive seismic methods should be favoured. For S-wave profiles derived by the inversion of surface waves generated by active sources (e.g., Multichannel Analysis of Surface Waves (MASW)), the analysis of both Rayleigh and Love waves should be combined, if possible. An effort should be done to reduce as much as possible the non-univocal character of the inversion process.
    - The compatibility of the velocity profiles with the local geology shall be sufficiently assessed.
  - The P and S-wave velocities of the relevant outcropping rocks.
- in the case of loose rock sites or unlithified layers, for Category I and II dams, the nonlinear material behaviour may need to be taken into account. In this case the process of parameter determination and verification shall be documented;
- depth of the groundwater table; and
- choice of 1-D, 2-D or 3-D site response analysis and justifications of the choice.

[4] Some of these requirements are in line with those of Chapter 11 of SIA 261/1 (2020)

C2.6 The uncertainties of knowledge (epistemic uncertainty) must not be averaged out, are to be set sufficiently large and shall be taken into account at least for the following aspects:

- shear-wave velocity profile;
- depth and geometry of the main impedance contrast;
- material properties.

C2.7 For cases where site response analysis has not been verified by means of recording local earthquakes with seismic instrumentation, the form of the elastic response spectra shall generally be specified by analogy with Paragraph 4.3.4.

C2.8 For the determination of the site-specific elastic response spectrum or *UHS*, from the site response analysis for the horizontal acceleration, the mean values of the reference rock *UHS*, multiplied by the envelope of the amplification functions from various ground modelling assumptions, are to be used. For the construction of the envelope of the amplification functions, individual isolated peaks in the plateau region may be undercut by a maximum of 20%.

C2.9 An amplification function may be derived through the convolution of deconvoluted acceleration time-histories (Figure C3). In this case, the deconvolution should be applied to free-field acceleration time-histories recorded on rock and compatible with the seismic hazard at the dam location[5]. Moreover, the following shall be considered:

- The convolution process shall start from a point which is sufficiently below the bedrock/soil transition. Convolution shall be applied at a starting depth where the shear wave velocity in the local rock profile is approximately the same as the shear wave velocity in the reference profile. For this purpose, shear-wave velocities below the exploration potential for geophysical methods, can be estimated based on local geology and reliable correlations.
- The amplification function is calculated by dividing the values of the elastic response spectrum derived from the acceleration time-history convoluted along the site-specific profile by the corresponding values of the elastic response spectrum derived from the same acceleration time-history but convoluted along the reference rock (see Figure C3).
- For a single model of a site, the geometric mean of the amplification functions shall be used, calculated with at least 11 acceleration time-histories that are representative of the local earthquake hazard on the reference rock.
- In order to take the epistemic uncertainty associated with the site into account, the envelop of the amplification functions of all models shall be used.

[5] The deconvolution process should be verified by the procedure outlined in Figure C3. On a case-by-case basis, a linear downscaling of the free-field acceleration time-history may be used instead of the deconvolution process.

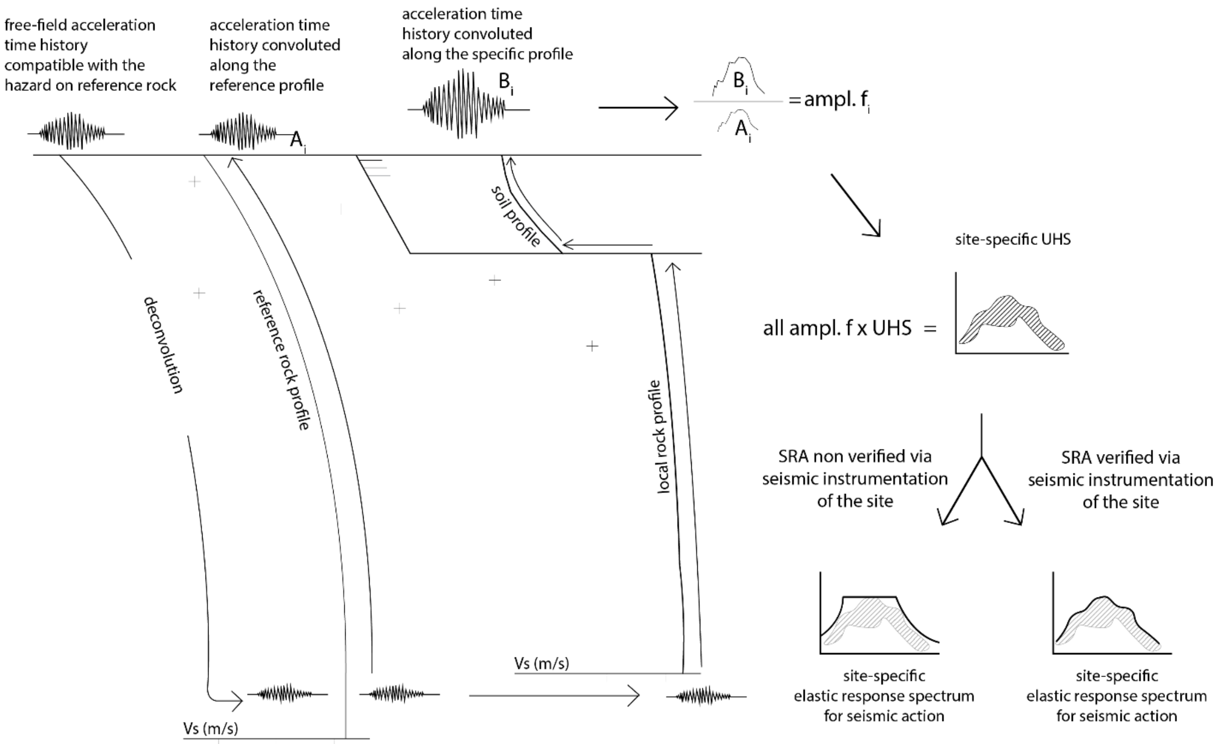


Figure C3: simplified representation of the site response analysis procedure by amplification functions

C2.10 The compatibility criteria listed in Paragraph 4.3.5 for the selection of acceleration time-histories shall be applied to site response analysis, with the following exceptions:

(i) The seismic hazard on the reference rock in the form of the mean *UHS* can be used as target spectrum for site response analysis.

(ii) For site response analysis, the number of acceleration time-histories should not be less than 11.

(iii) For site response analysis, the arithmetic mean of the response spectra of all time-series shall not be less than 75% and not greater than 130% of the target response spectrum for all periods in the relevant period range.

C2.11 For site response analysis with the seismic hazard on the reference rock defined by the SUIhaz2015 model (i.e., Path 1b) with the site response analysis that has not been verified by means of seismic instrumentation, the derived elastic response spectrum, can replace the elastic response spectrum according to Paragraph 4.3.4 (Path 1a). In this case, for sites with Vs_{30} < 800 m/s, the ordinate values of the elastic response spectrum from the site response analysis (i.e., output B in Path 1b) shall not be lower than the ordinate values of the elastic response spectrum determined for the same location according to Paragraph 4.3.4 for a hypothetical ground class A (with S_A = 1.4).

C2.12 In Path 1b with the site response analysis that has not been verified via seismic instrumentation, for sites with $Vs_{30} \geq 800$ m/s but < 1105 m/s, the ordinate values of the elastic response spectrum from the site response analysis (i.e., output B in Path 1b) shall not be lower than 75% of the ordinate values of the corresponding elastic response spectrum determined according to Paragraph 4.3.4 for an equivalent ground class (i.e., output B in Path 1a) (Figure C4).

C2.13 In Path 1b with the site response analysis that has not been verified via seismic instrumentation, for sites with $Vs_{30} \geq 1105$ m/s, the ordinate values of the elastic response spectrum, from the site response analysis (i.e., output B in Path 1b) shall not be lower than 75% of the ordinate values of the elastic response spectrum determined for the same location according to Paragraph 4.3.4 for a hypothetical ground class R.

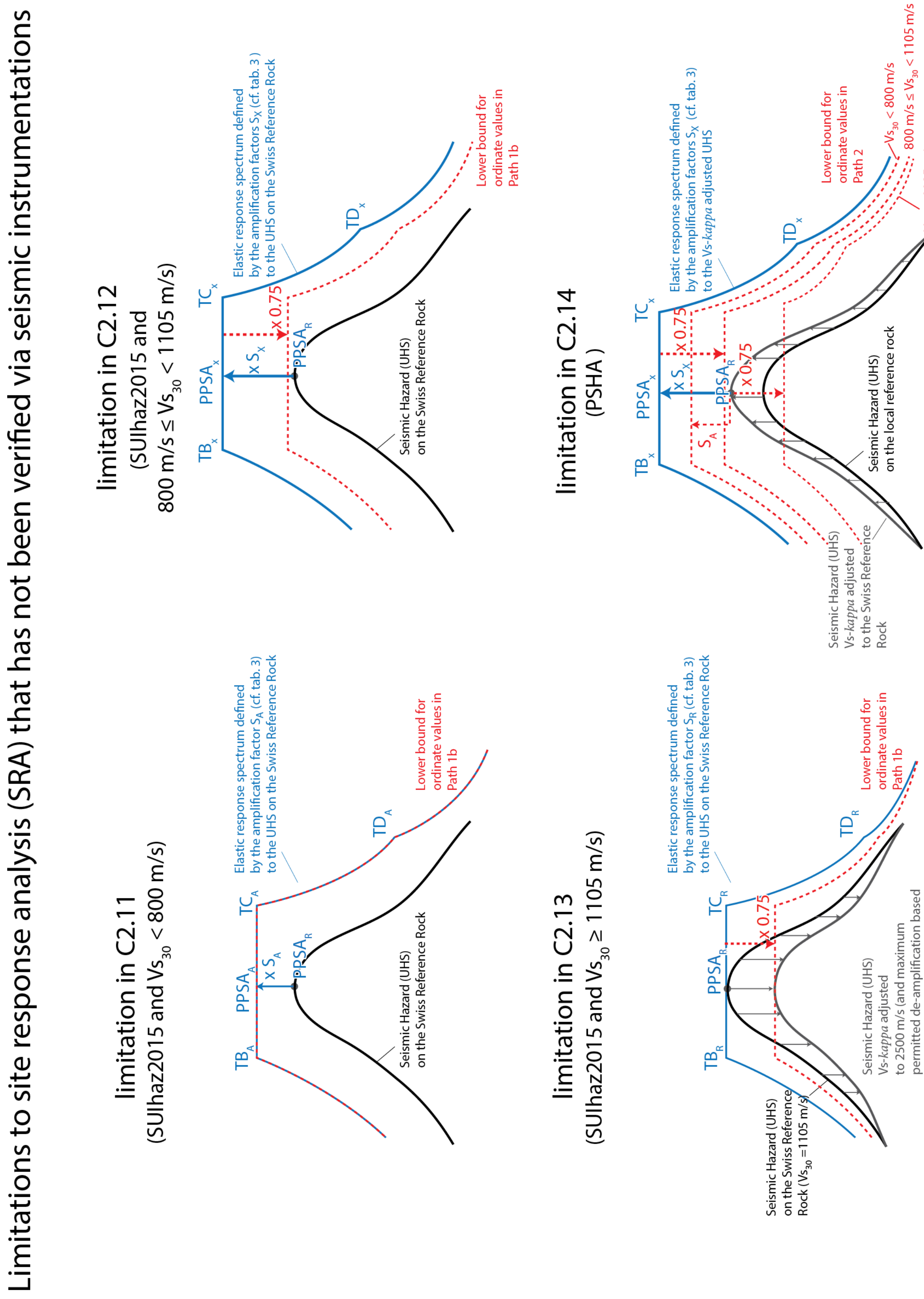


Figure C4: Schematic illustration of the conditions imposed to site response analysis in case the seismic hazard on the reference rock has been determined by the SUIhaz2015 model (Path 1b) (two Figures on the top and bottom left Figure) or according to path 2 by a site-

specific seismic hazard analysis (bottom right), and the site response analysis has not been verified with the seismic instrumentation at free field.

C2.14 For site response analysis with the seismic hazard on the reference rock determined according to a site-specific probabilistic seismic hazard analysis (Path 2 in Figure C1), with the site response analysis that has not been verified via seismic instrumentation, limitations by analogy with C2.11, C2.12 and C2.13 apply. In this case the theoretical $PPSA_R$ to be considered to set the lowest admissible ordinate values of the elastic response spectrum is defined by the peak of the mean *UHS* derived from the seismic hazard analysis and adjusted to the Swiss reference rock by a Vs-*Kappa* adjustment (e.g., Danciu and Fäh, 2017).

C2.15 For site response analysis that has been verified via seismic instrumentation, the mean values of hazard on reference rock (*UHS*), multiplied by the envelope of the amplification functions derived from the various modelling assumptions can replace an elastic response spectrum from Directive Part C3 (2025).

C2.16 The supervisory authority shall validate the outcome of the site-response analysis.

C3 SFOE requirements for the probabilistic seismic hazard analysis (*PSHA*)

C3.1 For seismic hazard analysis, probabilistic seismic hazard analysis (*PSHA*) may be carried out using seismic source models and ground motion models. The characterisation of the seismic source and the ground motion shall account for both aleatory variability and epistemic uncertainty. To conduct *PSHA* it is required to incorporate the necessary regional geological and seismological information. Moreover, it is required to utilise the most updated and relevant procedures and models for estimating the seismic hazard.

C3.2 A seismic hazard analysis may be performed in the form of a regional and/or site-specific study. Multiple sites may be combined into a single study, but a single site-specific output dataset shall be calculated for each site.

C3.3 Seismic instrumentation of the site is generally recommended. It allows to verify the site response analysis and to reduce the epistemic uncertainties. The verification of the site response analysis via seismic instrumentation is required for the direct use of the site-specific *UHS* as determined in C2.15 as target spectrum for the selection of compatible acceleration time-histories for the verification of the seismic safety of a water retaining facilities.

C3.4 It is recommended that probabilistic seismic hazard analysis for water retaining facilities in Switzerland be carried out according to SSHAC guidelines. It is possible to consider deviations from SSHAC guidelines and carry out a non-SSHAC procedure. The SSHAC procedural level and deviations shall be defined at the beginning of the procedure on a case-by-case basis in consultation with experts and the supervisory authority. For *PSHA* performed according to SSHAC Level 1 or according to a non-SSHAC procedure the minimum requirements expressed in paragraphs C3.5 to C3.10 shall be applied. For SSHAC Level 2 and higher, the SSHAC updated recommendations for the respective level set the minimum requirements to conduct a *PSHA*.

C3.5 A Participatory Peer-Review Panel (*PPRP*) shall be included in the process.

C3.6 For water retaining facilities in Switzerland the members of the *PPRP* shall be validated by the supervisory authority before the process of seismic hazard analysis begins. The minimum number of members of the *PPRP* is 2. The members of the *PPRP* must be independent of the experts, or corporate entities carrying out the *PSHA* study, the operator and the owner of the installation. The *PPRP* shall include at least two members with demonstrated expertise in two complementary fields relevant to seismic hazard analysis. At least one member shall possess demonstrated extensive knowledge of local seismology and/or seismotectonics and/or tectonics. It is recommended that at least one member should possess demonstrated experience in SSHAC procedures.

C3.7 A seismic hazard analysis carried out for a water retaining facility shall be composed at least by the following phases:

A) **Study plan**, outlining, but not limited to, the study strategy, the procedure to be followed and the list of experts involved in the study.

B) **Evaluation phase**, consisting in the consideration of the set of data, models, and methods potentially relevant to the seismic hazard analysis. The choice of data, models and methods shall be subjected to sufficient technical challenge and defence.

C) **Integration phase**, consisting in the incorporation of data, models and methods of phase B for hazard calculation. This may include, if necessary, the development of new methods and the construction of new models. Justification for the choices adopted and for exclusions shall be properly documented.

D) **Documentation**, including a detailed discussion of phases A, B and C. The provided information shall be transparent and traceable. The documentation shall be sufficiently detailed to allow the hazard analyses to be reproduced by an external reviewer.

The study plan (phase A) shall be validated by the supervisory authority.

The outreach to additional experts in phase B is not a minimum requirement but should be evaluated on a case-by-case basis and the *PPRP* recommendation. Sensitivity analysis to identify significant issues and the relative importance of input parameters is required in the evaluation process and one or more feedback and review cycles in Phase C may be conducted on a case-by-case basis and upon recommendation of the *PPRP*. Phase D shall involve at least the preparation of one draft report, the final report shall be finalised only after at least a review cycle with the *PPRP*.

C3.8 The *PPRP* conducts a technical and procedural review and shall be involved in the study starting from the development of the study plan (i.e., phase A in C3.7).

For phase B in C3.7 the *PPRP* should confirm that in the evaluation process all relevant aspects have been considered and that the evaluation process was conducted objectively and without cognitive bias. The *PPRP* evaluates the necessity of considering outreach to experts not yet involved in the process. The *PPRP* evaluates the justification for the weighting in the logic tree, the use of sensitivity analysis to address significant issues and/or the consequences of the modelling choices.

In phase C in C3.7 the *PPRP* evaluates the necessity for one or more feedback and review cycles. It verifies the calculations and estimates the extent to which the models captured the range of the technically defensible interpretations.

For phase D in C3.7 the *PPRP* evaluates the completeness of the documentation and writes a final closure report describing the evaluations reached for phases A to D in C3.7 and at which extent their evaluations, comments and/or propositions have been integrated in the study. The procedural review shall address in particular the appropriateness of the study procedure to the specific case, and the overall adherence or deviations from the study plan validated by the supervisory authority.

C3.9 **Benchmarking:** in cases where the *PSHA* results display relevant deviations from the SUIhaz2015 model, an additional document shall be provided. The aim of this additional document is to illustrate explicitly the differences, to identify in the *PSHA* the reasons for the deviations and to provide justifications. The preparation and the finalisation of the benchmarking document shall involve the *PPRP* as well. This document serves as a basis for an objective evaluation and acceptance of the proposed deviation from the SUIhaz2015 model. In this case the supervisory authority may accept the seismic hazard results with reservation and/or exceptions and set limitations to the ordinate values of the resulting *UHS* or limit the use of the new seismic hazard.

C3.10 The input for the *PSHA* model including geological, tectonic and seismotectonic frameworks as well as the historic earthquake records and the logic tree adopted should be validated by the supervisory authority or their mandated external experts before the final derivation of the local Uniform Hazard Spectrum (*UHS*).

C3.11 For water retaining facilities in Switzerland, the local seismic hazard is determined at a free-field surface on the reference rock. The reference rock shall have $Z_{top} \leq 0.5$ m (starting depth of the shear-wave velocity profile), $V_{Smin} \geq 1000$ m/s, $V_{S30} \geq 1105$ m/s and should follow the form defined by Equation C1 (for the derivation and optimisation of parameters *a* and *b* please refer to Poggi et al., 2011). The choice of the velocity profile should be justified and supported by data.

$$V_S(Z) = \left(V_{S_{max}} - V_{S_{min}}\right)\left[1 - a^{\left(\frac{z_{top}-z}{b}\right)}\right] + V_{S_{min}} \qquad \text{Equation C1}$$

C3.12 For water retaining facilities in Switzerland, the mean value of the seismic hazard is to be used.

C3.13 The use of local ground parameters, which do not correspond to those of the reference rock as defined in section C3.11, directly in the Ground Motion Models (GMMs) is not permitted, i.e., layers with Vs < 1000 m/s shall only be considered in site response analysis (*SRA*).

C3.14 A *PSHA* study may include site response analysis in addition to determination of the hazard on the reference rock. In this case, if it is decided to treat both the seismic hazard on the reference rock and the seismic hazard including the site response probabilistically, this shall be done in two stages. The first stage shall determine the seismic hazard on the reference rock and the second stage shall determine the seismic hazard including the site response. In this case, the procedure shall be performed at SSHAC level 2 or higher.

C3.15 The final results of the site-specific seismic hazard analysis, including the *UHS*, the deaggregation data for the relevant frequencies, and response spectra shall be validated by the supervisory authority. The provided documentation shall be sufficiently detailed to allow the hazard analysis to be reproduced by the supervisory authority or its mandated experts.

C4 Literature

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