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Peer Review of the Swiss Dam Safety Guideline, Part C3, version 2021 “Earthquake Safety”

Phase 1 – Chairman report

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Abbreviations

POE: Panel of Experts

C3-2021 Guideline: Swiss Dam Safety Guideline, Part C3, version 2021

GMM: Ground Motion model

ICOLD: International Commission on large Dams

P.G.A: Peak Ground Acceleration

PSHA: Probabilistic Seismic Hazard Assessment

R & D: Research & Development

RP: Return Period

SED: Swiss Seismological Service

SSHAC: Senior Seismic Hazard Advisory Committee

SFOE: Swiss Federal Office of Energy

SUIhaz2015: Swiss Seismic Hazard Model carried in 2015

1 Introduction

1.1 Background information and current situation

In 2023, the SFOE requested an external Independent Review, also referred as a “Peer Review”, of the Swiss Dam Safety Guideline, Part C3, version 2021 on “Earthquake Safety”, partial revision of 14 July 2021, referred to hereinafter as “C3-2021 Guideline”.

The “Peer Review” was carried out by an independent Panel of Experts directed by a Chairman, referred to hereinafter as “POE”.

SFOE updated C3-2021 Guideline mainly to include the results of the Swiss Seismic Hazard Model referred to hereinafter as “SUIhaz2015” carried out in 2015. Experience acquired from the first phase implementation of C3-2021 Guideline by the operators of water-retaining facilities in Switzerland, referred to hereinafter as the “Operators” motivated the need to perform a Peer Review of C3-2021 Guideline.

SFOE asked me to chair this Peer Review as Chairman and Expert in Dam Earthquake Engineering.

A common report has been delivered on October 30, 2023 and a meeting was organized to present verbally the conclusions of the panel.

Individual reports have then been submitted by the five members of the POE in November 2023.

The present report written by the chairman presents the main findings of the Peer Review.

It includes two main sections:

- a first section giving the analysis of the Swiss Seismic Hazard Model, carried out by the - seismologist experts - Ramon Secanell and Ivan Wong
- a second section providing the analysis of Swiss Dam Safety Guideline – C3-2021-, carried out the dam experts – B.Tardieu, Jonathan Hinks and Patrick Lignier.

The proposed detailed modifications of C3-2021 Guideline by the POE are presented in a separate document.

1.2 Purpose of the Peer Review

The purpose of the Peer Review is to assess the suitability, applicability, exhaustiveness of the C3-2021 Guidelines computation assumptions, evaluate whether the recommended approaches are adequate and consider if the seismic requirements allow for possible remedial measures to be executed in accordance with the concept of proportionality (balanced approach).

In a hazard evaluation, it is current practice to characterize the threatening event – hazard-, to associate it with a probability of occurrence and to give recommendations for design and construction - here the dams built or to be built in Switzerland.

The dams are supposed to resist to hazards as their collapse is judged to be unacceptable with regards to their consequences. This is clearly expressed in C3-2021 Guidelines:

“The objective of the seismic verification of a water retaining facility is to demonstrate that during and after earthquake, failure of the facility that could result in an uncontrolled discharge of water can be ruled out. Damage, such as permanent displacements if they do not interfere with the safety of the water retaining facility, is permissible.”

2 Seismic Hazard and Design Ground Motions

This Summary presents the main comments and recommendations for the SFOE suggested by the POE selected by SFOE to evaluate the bases for the increased seismic design ground motions defined following the procedures defined in the Directive Part C3. It has been prepared by Ramon Secanell and Ivan Wong.

The seismic design ground motions calculated by applying the Directive Part C3 leads to an increase compared with the seismic design levels predicted by the 2003 OFEG guidelines.

The 2003 OFEG guidelines for dams in Switzerland and the 2021 Directive Part C3 define the ground motions to be used as input for the seismic safety evaluation of dams in Switzerland. Both guidelines define the “standard” (non “site-specific”) acceleration response spectrum using:

- Swiss National Seismic Hazard Maps:
    - The 2003 Guidelines were based on intensity hazard map developed in 1977 (Figure 1-1 of OFEG, 2003).
    - The 2021 Guidelines are based on probabilistic seismic hazard maps in terms of 5%-damped pseudo-spectral acceleration developed by SED (SUIhaz2015) in 2015.
- Amplification (adjustment) factors:
    - In the 2003 Guidelines, the amplification factors were very close to 1
    - In the 2021 Guidelines, the amplification factors are significantly higher than 1.

The POE noted the significance of this increase in amplification factors in developing the standard design spectra.

2.1 Comments on the SUIHAZ2015 seismic hazard maps

The main comments identified by the POE are summarized here below:

1) The SUIhaz2015 report (Weimer et al., 2016) describes very well the methodology used to define the new seismic hazard maps for Switzerland. The methodology is consistent

with the current state-of-the-art in PSHA including some concepts mainly used, until now, in R&D but rarely used in current industrial practice (except in detailed Senior Seismic Hazard Advisory Committee [SSHAC] Level 2 and 3 studies).

2) Moreover, the SED team responsible for the development of the Swiss seismic hazard maps participated in other European projects directly related with the evolution of the EC8 (SERA project). They also participated in the past SSHAC Level 4 Pegasus Project and later Pegasus refinement. Therefore, they have experience in the development and improvement of seismic hazard maps.

3) The SED is the best qualified team to develop the national seismic hazard maps because they are the most knowledgeable with regards to the seismicity and the seismotectonic setting of Switzerland and the GMMs (ground motion models) that should be applied in Switzerland. For that, they compared the predicted ground motions in the GMMs and the recorded data in Switzerland. See following discussion.

4) The http://hazard.efehr.org/en/hazard-data-access/hazard-spectra/ website also allows one to compare the seismic hazard results of ESHM13 (SHARE Project), ESHM20 (SERA Project) and SUIhaz2015. Accelerations calculated using ESHM20 model (using Vs30=800 m/sec) and SUIhaz2015 model (using Vs30=1100 m/sec) indicate the latter is higher by about 20%. Part of this difference is due to the difference in Vs30 but this difference would be significantly higher using the proposed C_A factor (1.6, it means 60% differences) to convert spectral accelerations for Vs30=1100 m/sec into spectral accelerations for Vs30=800 m/sec. .

5) The hazard associated with the 10,000 yr-RP predicted in SUIhaz2015, may be overestimated. The slope in the hazard curves (kappa) based on the PGA ratio between the 10,000 yr-RP and 475 yr-RP seems high. Based on SUIhaz2015, kappa often exceeds 4. In southwestern Switzerland, kappa is nearly 5 in SUIhaz2015 compared to a value of about 3 estimated in site-specific studies performed for two Swiss dams in the same region. Part of the reason for the higher kappas in SUIhaz2015 may be due to the low Mmin used (see following comment). In EC8 (Chapter 2.1), kappa is 2.76 for 10000 years return period (although this value should be considered only as approximative). The table presented in Appendix 2 shows kappas observed in site-specific hazard analyses worldwide. The mean kappa for active continental regions for this small sample of sites is 4.2. We note that site-specific studies particularly those that model faults could lead to non-negligible differences compared with regional maps, where local features are not considered.

6) The minimum magnitude (Mmin) considered in the PSHA calculations for the SUIhaz15 hazard maps was **M** 4.0 (Wiemer et al., 2016). The global standard-of-practice as summarized in a paper by Bommer and Crowley (2017) generally ranges from **M** 4.5 to 5.0 although magnitudes as low as **M** 4.0 have been used. A value of **M** 4.0 may be appropriate for older vulnerable buildings in Switzerland but not for generally modern engineered dams. A Mmin of **M** 5.0 is the most commonly used value in dam analyses. The POE believes that a suite of maps appropriate for use in dam safety evaluations should be developed assuming a Mmin of **M** 5.0. The use of **M** 5.0 avoids the inclusion of non-damaging ground motions which can artificially inflate the seismic hazard, mainly at shorter return periods and in low seismicity regions (with a non-negligible contribution of the small magnitudes to the seismic hazard). The use of a lower Mmin could be required by SFOE e.g., **M** 4.5 in site-specific studies of particularly vulnerable dams which have been poorly constructed or on located on poorly compacted soils and where there may be liquefaction issues.

   If feasible, a set of SUIhaz maps should be developed using a Mmin of **M** 5.0 for use by SFOE in the next revision of Swiss seismic hazard maps (long-term recommendation).

7) In the view of the POE, the SUIhaz2015 maps can be improved through the use of more recent GMMs. Almost all of the GMMs used have been superseded by more current models as discussed in the individual reports by Wong and Secanell. The adoption of more recent GMMs is recommended for the next revision of the SUIhaz2015 (long-term recommendation).

8) The Directive Part C3 indicates that the median hazard being recommended in the SUIhaz15 should be considered instead of the standard mean hazard (McGuire et al., 2005). It is unclear why the median hazard is being used and what advantages are there to its use. The mean hazard reflects the epistemic uncertainty in the inputs into probabilistic seismic hazard analysis and given the large uncertainties in inputs for Switzerland, the mean hazard should be used. The use of mean hazard is standard practice in seismic hazard analyses worldwide.

In conclusion, the development of the SUIhaz2015 maps was performed using state-of-the-art procedures in seismic hazard analyses including the use of models typically included only in R&D projects or in very detailed “site-specific“studies (i.e., Vs-kappa adjustment, single-station sigma, etc.). The POE recognizes that significant revisions to the maps for the immediate use in Directive Part C3 is not realistic and the revisions are only recommended for the next version of the hazard model in Switzerland.

However, we believe development of hazard maps using a Mmin M 5.0 requires minimal effort and if possible, should be produced for use by SFOE.

At this time, in our judgment the SUIhaz2015 hazard maps are slightly conservative for use in dam design and safety evaluations for reasons stated above (Mmin). We understand that the SED will be updating the seismic hazard maps on a periodic basis and the next version might not be available for several years. The POE recommends that if seismic hazard maps using a Mmin of M 5.0 cannot be developed, that site-specific seismic hazard analyses be performed to address the possible over conservatism in the seismic hazard maps for evaluating dams (at least Class I). Performing site-specific seismic hazard analyses is standard practice in the world for critical and important facilities and structures and the POE believes this should be performed for all high hazard dams. This is particularly important because the SUIhaz2015 model does not address the hazard from potential active Quaternary faults and if such fault(s) exist near a dam, they need to be included in the hazard analyses.

2.2 Comments on the amplification factors used in the Directive Part C3

Two types of amplification or adjustment factors are used in the Directive Part C3:

- C_A factor:
- S factor

The S factors given in the Directive C3 are generally consistent with other amplification factors given in other seismic codes for similar site classes (i.e,. EC8 or French seismic guidelines for large dams).

Based on our review of Duverney et al. (2019), the C_A value of 1.6 was estimated based on four previous studies. The four studies yielded an average of 1.62. The POE believes this value is conservative given more current studies and guidelines (i.e., ASCE7):

- The ASCE code indicates amplification values around 1.1 to convert “hard rock (Vs30=1500 m/sec) into “standard rock” (Vs30=800 m/sec)
- An examination of the NGA-West2 GMMs indicates that the ratio of median SA values at PGA and at 0.15s for moderate ground shaking (**M** 6 and Rrup=20 km scenario) for Vs30 values of 800 and 1,100 m/sec is about 1.2.

The four studies cited in Duverney et al. (2019) have been reviewed and the POE’s evaluation is described in detail in the Appendix 1.

Based on the POE’s observations and analysis (Appendix 1), the value of C_A should be re-evaluated given the significance of the factor in developing the standard design spectrum and to avoid potential over-conservatism. Specifically analyses using global strong motion databases and GMMs (e.g., NGA-West2) indicate that an adjustment factor going from 1100 to 800 m/sec is about 1.1 to 1.2 lower than the currently recommended 1.6. Particular attention should be paid to studies that estimate C_A at ground motion levels resulting from moderate to large magnitude earthquakes. In some seismic guidelines, the amplification factors are defined for different ground motion ranges (i.e., ASCE 7, French seismic guidelines for large dams) and globally the amplification factors are smaller for high ground motions due to the nonlinear behaviour of both soils and rock. This approach should be explored in detail for future developments of seismic guidelines (long- term recommendation).

2.3 Review of the 2003 OFEG Guidelines and comparison with the 2021 directive part c3

The general procedure to define earthquake ground motions in the 2003 guidelines was defined in Section 1. Basically, the seismic design ground motions were defined using:

1) Seismic hazard maps: They were defined in terms of intensity and then, using a single model, the intensity was converted into PGA. The POE notes that the equation used was the least conservative of the equations found in the literature to convert intensity into PGA.

1) Amplification factors: The amplification factors contained in 2003 OFEG Guidelines were 1.0, 1.0, and 0.9 for soil classes A, B and C, respectively (only three soil classes are defined in 2003 Guidelines). Therefore, in general, the amplification factors were generally low compared to other guidelines (e.g., ASCE7).

Combining the likely low 2003 PGA values (because of the intensity to PGA conversion model used) and relatively low amplification factors, probably led to relatively “low” seismic design ground motions.

Compared to the 2003 OFEG Guidelines, the 2021 Directive Part C3 presents:

- Likely somewhat conservative seismic hazard maps.
- A rather complicated and likely conservative procedure using amplification factors (C_A) that leads to higher amplification.

In summary, the 2021 Directive Part C3 that combines spectral accelerations obtained using low Mmin (Mmin=4.0 might introduce non-damaging contributions to the seismic hazard) and large C_A amplification factor (i.e,. compared to other seismic guidelines or compared with the ratios given by direct application of NGA-2 West), has led to the perception of relatively “high” or conservative seismic design ground motions.

Also, comparing 2003 OFEG PGA values and 2021 Directive C3 PGA values, for the same soil conditions and same return period, large differences are observed (up to 100%).

2.4 Conclusions and recommendations for the seismic hazard

The main conclusions and recommendations are the following:

- **The SUIhaz2015 seismic hazard maps should be updated. They are nearly nine years old and recent scientific advances have been made in the ensuing years that indicate the maps can be improved. For example, the set of GMMs should be updated with more recent models and a Mmin of M 5.0 appropriate for dams is recommended (long-term recommendation). Similar national hazard maps such as those in the U.S. are updated every four to five years. The maps could also be developed using a Vs30 of 800 m/sec that would remove the need for defining a C_A factor. The maps should also display the mean not median hazard.**
- **The process of using and deriving C_A recommended in the 2021 Directive Part C3 should be re-evaluated and possibly revised (short-term recommendation).**
- **The combination of probably underestimated ground motions predicted from the intensity-based seismic hazard map in 2003 OFEG Guidelines and the small amplification factors (from 0.9 to 1.0), led to relatively low seismic design ground motions.**
- **The combination of spectral accelerations obtained with low Mmin from SUIhaz2015 maps used in 2021 Directive Part C3 and the C_A has likely resulted in unnecessarily conservative seismic design ground motions predicted using 2021 Directive Part C3.**
- **The combination of possible under-estimated seismic hazard in the 2003 OFEG Guidelines (Section 2.3) and possible overestimation of seismic hazard in the**

2021 Directive Part C3 (Sections 2.1 and 2.2) has led to a perceived large increase in seismic design ground motions predicted by 2021 Directive Part C3 as compared to the 2003 OFEG Guidelines.

- **To avoid possible over-conservatism and to ensure the latest information is incorporated into the seismic design ground motions for dams particularly Class 1 and possibly Class 2, we recommend that site-specific seismic hazard studies be performed. This is particularly critical where more recent information on the inputs into the hazard analysis have become available since the 2015 SUIhaz2015 maps and specifically, if Quaternary faults are recognized near the damsite.**
- **In some guidelines for buildings (ASCE7) and other seismic codes, a maximum reduction of the standard response spectrum of 20% is allowed if a site-specific seismic hazard analysis is performed. For dams in many countries (e.g., U.S.), the results of site-specific studies can be adopted as is without a minimum requirement as long as the site-specific study is peer reviewed (short-term recommendation). We recommend that decisions on the acceptability of results of a site-specific seismic hazard analysis be solely the responsibility of SFOE.**
- **The process of performing a site-specific analysis should be described in detail by SFOE and the resulting study reviewed by SFOE (internally and/or using external consultants). In the judgment of the POE, however, to impose the requirement that a SSHAC Level 2 study be performed is unnecessary and a waste of resources. A SSHAC requirement for seismic hazard assessment is not in place for dams in U.S, for example. Most site-specific seismic hazard analyses performed for dams certainly in the U.S. and likely worldwide are SSHAC Level 1, which is also a peer reviewed study. In the judgment of the POE, a SSHAC Level 2 need only be required in case of where multiple dams in the same region with similar seismotectonic characteristic are being evaluated. In such cases, the same PSHA model could be applied resulting in savings in time, effort, and costs. This is a common approach in the U.S. where the agency responsible for multiple dams in the same region has the dams evaluated together (short-term recommendation).**

For a site-specific seismic hazard analysis, we recommend that site-specific data (geological, geophysical and geotechnical) be collected through a desktop and field investigation program. In particular, this should include geophysical surveys to characterize the shear-wave velocity structure of the dam foundation and investigations into the potential for Quaternary faults occurring in the region around the site (short-term recommendation).

3 Dam Safety and Earthquake

3.1 Main comments and findings

The purpose of the Peer Review is to assess the suitability, applicability, exhaustiveness of the C3-2021 Guideline, evaluate whether the recommended approaches are adequate and consider if the seismic requirements allow for possible remedial measures to be executed in accordance with the concept of proportionality (balanced approach).”

The main comments on C3-2021 guideline by the dam experts are presented below.

1) Philosophy of the new C3-2021 guideline

The previous guidelines - 2003 OFEG guidelines - were more prescriptive with the advantage to better describe the methodology to follow to verify the safety of the dam against earthquake. On the other hand, it presented the disadvantage to be less flexible for the possible future evolution of the state of the art.

The POE believes that the philosophy followed by the new guideline is relevant.

2) Phased studies progressively more complex

The verification of the dam safety against earthquake is dependent on many parameters as the seismic hazard, the dam foundation, the size and quality of dam. It can be simple or very complex. It is therefore recommended to carry out phased studies increasing – if necessary - the complexity of the verification step by step. This approach is followed by the French Guidelines [R4] and USSD recommendations [R6]. This permits a better proportionality of the verification.

For embankment dams, the following phased studies could be followed:

1. Preliminary studies – Verification of main input data and potential liquefaction
   - Geotechnical and geological model development
   - Seismic hazard analysis
   - Verification of the potential liquefaction and cycling softening
2. Phased studies to earthquake-induced deformation analysis
   - Stage 1: Simplified verification
   - Stage 2: Sliding Block analysis
   - Stage 3: Nonlinear Deformation analysis

Depending on the Category of the dam and the PGA, it is mandatory to go up to stage 3 – for example for Category I dam. However, even if the justification needs to go to stage 3, it is recommended to start by stage 1 then stage 2.

For small dams - Category III dam-, the phased studies can be stopped as soon as the safety of the dam is demonstrated at stage 1 or 2.

3. Minimum Performance Criteria

It is implicit that damage is acceptable during and after a SEE should this damage not induce uncontrolled release of water.

It is therefore recommended to give more guidance – in the C3-2021 guideline - about the Minimum Performance Criteria of a dam during and after an earthquake. The various aspects of the consequences of the irreversible or permanent displacements should be clearly addressed.

It should be also noted that the minimum performance criteria can evolve with the state of the art.

In the C3-2021 Guideline, it is stated that “Verification of seismic safety is necessary for all water retaining facilities, and it is insisted that it is necessary to take into account changes in the state-of-the-art in science and technology and to consider these changes in the assumptions made in a previous verification of seismic safety.”

We agree with this methodology. It should be noted that the state of the art, or the good practices, are gradually changing and improving. The International ICOLD Benchmark Workshops on Numerical Analyses of Dams play an important role in this evolution. The results are published by ICOLD. However, the penetration of these good practices in the current and usual day-to-day professional production of the engineers follows a low progression. It is of the responsibility of the administrative control system to encourage this penetration.

As the normal behaviour of a dam is clearly supposed to be in the elastic reversible domain with a large margin of safety, the cracks or permanent displacements quoted in the text: “*Damage, such as permanent displacements if they do not interfere with the safety of the water retaining facility, is permissible*,” are clearly in the non-elastic domain, with a safety factor which is locally below 1. This aspect is a fundamental of the approach of the dynamic effect to avoid an excess of precaution or conservatism which could lead to reject many existing dams as unsafe.

This safety factor which is locally below 1 is acceptable because the irreversible movements or displacements occur during a short period of time and because the solicitation is not permanent. It is a vibration which stops when the earthquake and its aftershocks have stopped. In static, if $f<1$, a kinematic of movement is triggered and this movement does not stop till a change of the kinematic, including collapse and fall down. In dynamic, if $f<1$, a movement starts with the same kinematic than in static, but the solicitation stops at the end of the acceleration wave when the movement is inversed in the opposite direction. So, the displacement is stopped. At the end of the earthquake, it remains a permanent displacement which is the sum of each of the brief displacements beyond the elastic domain during the earthquake.

The definition of the acceptable or not acceptable value of the irreversible displacement should remain in the domain of the engineering judgment and of the socio-economic-political responsibility. This judgment shall be based on the analysis of the consequences of these permanent displacements on the safety of the dam and its appurtenant structures. It shall be shared and validated by the “controller” - SFOE.

The next sections analyse the state-of-the-art in modelling the dams subject to an earthquake and give some guidance about the minimum performance criteria.

4. *“Simplified method” for Category III embankment dam*

Studies and verifications should also be proportioned to the size of the dams and the ease or difficulties to demonstrate its safety.

Thus, it is possible to simplify the verification for small dams in C3-2021 Guideline as permitted by the French guidelines [R4] and stipulated by USSD [R6].

Indeed, when a small embankment is well built, the seismic load case – for moderate PGA - is not the critical load case.

The following preliminary studies analysis shall be firstly performed in all cases.

- Geotechnical and geological model development
- Seismic hazard analysis
- Verification of the potential liquefaction and cycling softening

Then, specific verification dependent on the acceleration of the earthquake could be carried out.

Acceleration below 0,1g (a_{gh})

For small dam, the verification of the safety of the dam against earthquake is not necessary when the following conditions are met:

1.Dam and Foundation materials that are not liquefiable and do not contain sensitive clay

2.Well-built, densely compacted dam with relative compaction RC > 95 % or relative density Dr > 80%

3. PGA < 0.1 g at the base of the embankment.

4. Static factor of safety before the earthquake greater than 1.5

5. At the time of the earthquake, freeboard greater than 3 percent of the dam height (including alluvial foundation) but not less than 1 m.

Acceleration above 0,1 g (a_{gh})

Should the potential liquefaction and/or cyclic softening of materials are discarded, a simplified approach could be used to demonstrate the safety of the dam. This simplified approach consists of two stages.

- Stage 1: Verification of the risk of crest settlement
- Stage 2: Verification of the risk of sliding

Correlations- empirically-based and analytically-based - shall be firstly considered.

Empirically-based correlation (e.g Sawaigsood,2014) can show if there is a potential risk of crest settlement.

Should this risk be discarded, “simplified dynamic method” can be used for verification of the stability against sliding.

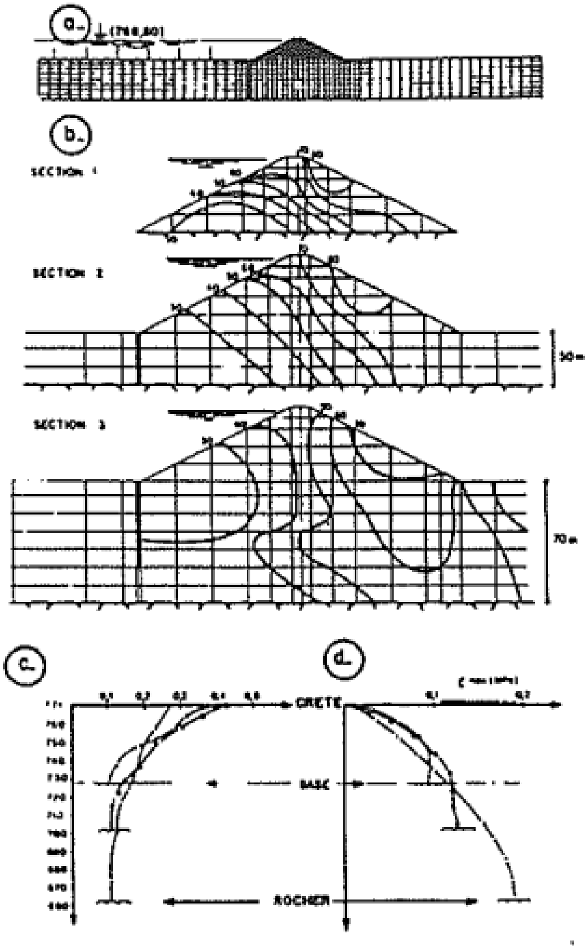
For this verification, the horizontal acceleration for sliding masses is taken equal to agh and it is not necessary to consider the vertical acceleration. If the safety factor is below 1, the other methods recommended for category II and 1 should be used.

In conclusion, the stability against sliding can be verified by a simplified approach – as pseudo-static calculation, when the following conditions are met:

- Dam and Foundation materials that are not liquefiable and do not contain sensitive clay
- There is no risk of crest settlement above the freeboard.

Should the safety of dam against sliding be not demonstrated with this simplified approach, the other methods recommended for Category II and I should be used.

It should be also noted that from the return of experience of numerical modelling, the soft foundation can act as a filter of the dynamic signal and reduce the amplification effect which is far smaller as the one calculated when the embankment dam is founded directly onto the rock. (see Figure 1 below [R12])



'ig. VIII-1.23. Barrage du Verney (France).

Figure 1. Example of a long embankment dam founded on a thick alluvium deposit in the valley and on the rock on the sides.

As a consequence, for a small embankment dams founded on a thick alluvial deposit, it is acceptable to assume that the acceleration is almost uniform in the body of the dam.

5. Complex numerical modelling – State of the art

The present state-of-the-art uses numerical modelling more and more elaborated and involving many aspects of the earthquake response of dams.

The use of numerical modelling is a simplification which ignores unavoidably non-included features. The modelling represents a certain level of simplification depending on the evolution of the science, on the capacity of the computers and on the time devoted to the analysis, in particular to collect the parameters to be included in the modelling. At each period of historical time, modelling represents the state-of-the-art at this time. Regulation has to take into account this evolution.

In the practice, dynamic design of dams includes a certain level of conservatism incorporated into the safety factors and in the determination of the parameters. Modelling helps to appreciate the margin of safety included in the design currently accepted. This apparent conservatism may also cover eventual unknown negative margin namely in the seismic behaviour which is not frequently observed and measured.

It should be noted that common hypotheses are done for the pore pressure:

i)*The water is assumed to be not mobile and the water pressure is not modified during the earthquake because of a movement of water - whatever the material of dam body*. [i]

ii) In concrete dam and rock foundation, it is assumed that there is not modification of uplift during an earthquake. (Modification of uplift can occur after an earthquake)

These assumptions are done as it was never observed otherwise neither in dams nor in natural cliffs.

3.2 Gravity Dams

General comments on Minimum Performance Criteria

To guide the review of the members of the panel, several recent earthquake studies by various consulting engineers for various dams in Switzerland have been transmitted to the members. These studies show the current practice in Switzerland. These studies were developed before the publication of C3-2021 Guideline.

Two studies on concrete dam were specifically analysed.

One study concerns a high dam in a relatively high seismic area. The results of the study showed the dynamic reversible displacements during the earthquake and the extreme values of the stresses on the upstream and downstream faces of the dam.

The conclusion of the study states essentially that during the history of the stresses in the dam, the stresses do not exceed the value of the admissible compressive strength but exceed the value of the admissible tensile strength. It is said that these exceedances are “sporadic” and during a short period of time. It is concluded that these stresses do not constitute a threat for the structural safety of the dam. This statement is in agreement with the international practice and in agreement with the recommendations of USACE [R3] which introduce the Demand Capacity Ratio and the Cumulative Inelastic Duration to assess the possible failure of the structures.

As during the earthquake, the alternate movements generate almost the same stresses on both faces, the compressive stresses and the tensile stresses have the same magnitude: Compressive dynamic stresses are more or less always acceptable and dynamic tensile stresses are significantly beyond the accepted measured limit.

For an extreme earthquake, this narrative conclusion does not seem to be no more convincing. Assuming a first mode vibration of 5 Hertz, the duration of half a loop is almost one tenth of second, which is actually brief. Ten exceedances totalize 1 second. However, we are not aware of any academic research on the behaviour of massive concrete (not steel reinforced) submitted to this kind of solicitation. After the brief solicitation, the concrete is compressed again. What is its material state? Is it able to resist again to compressive stresses?

It should be recommended to go further to understand the consequences of a strong earthquake beyond the traditional and generally accepted statements.

Acceptable irreversible displacements as Minimum Performance Criteria

Coming back to the basis, the static stability of a gravity dam is demonstrated by a non-sliding proof: the dam is supposed not to slide on the rock foundation taking into account the friction angle (including the dilatancy) of the concrete rock contact and the pore pressure pattern at the contact, with an accepted safety factor. It is the main justification of the stability of a gravity dam.

During the earthquake, the effect of the acceleration at the centroid is added as well as the additional dynamic effect of the water of the reservoir. There is no permanent displacement which occurs if the safety factor remains above 1. If during peak of the acceleration, the safety factor is below 1, a permanent displacement occurs and has to be calculated.

To obtain this permanent displacement, the process is to determine the accelerogram at the centroid of the dam. Taking into account the limit acceleration of the dam, which is the permanent acceleration which provokes the sliding of the dam on its foundation. It is simple to check whether this limit acceleration is exceeded during the earthquake history.

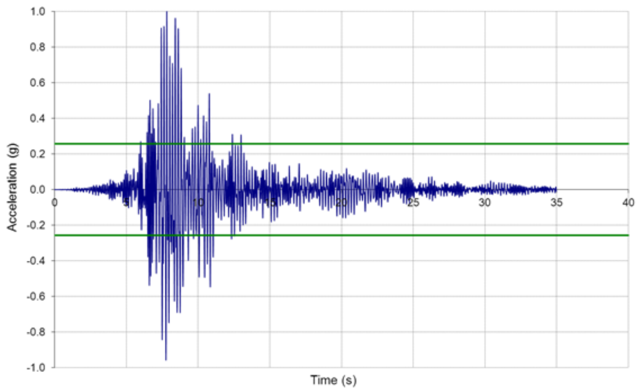


Figure2. Example of Dam acceleration response at the gravity center. In horizontal green limit gives the limit acceleration at the centroid.

In case of exceedances, the excursions beyond the limit generate a permanent displacement (calculated by double integration of the acceleration). By addition of these successive permanent displacements, the global permanent displacement of the dam is known.

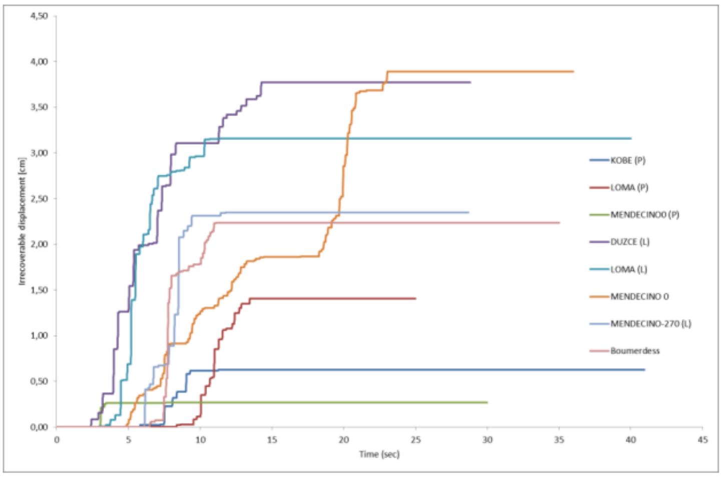


Figure 3. Example of irreversible displacement evolution with time and for various accelerogram of earth-quake.

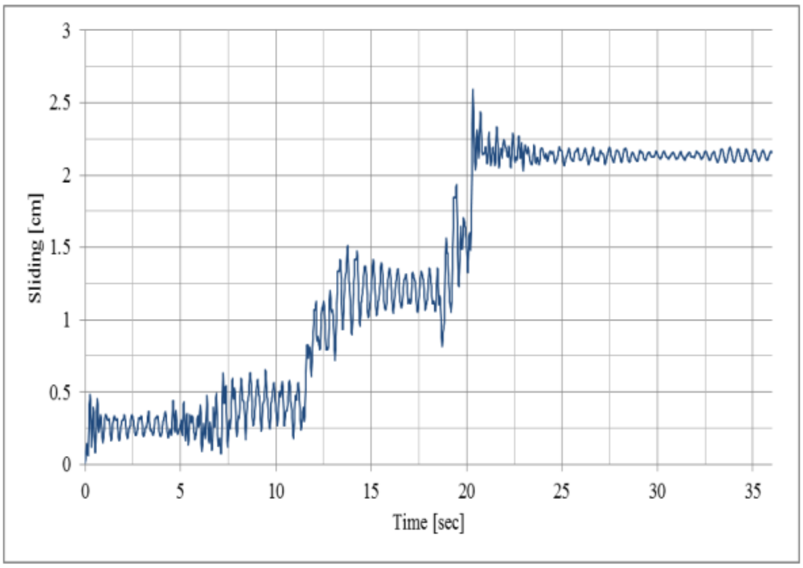


Figure 4. Example of the evolution of the irreversible sliding with time.

For any gravity dam, if the earthquake is strong, it should be recommended to calculate the acceleration at the centroid of the dam, to determine the limit acceleration of the dam and the possible irreversible displacements.

Once the irreversible displacement is calculated, it is necessary to analyze the consequences of this movement on the leakages in the dam body and in the rock foundation. The irreversible displacement is not necessarily a threat for the stability of the gravity dam, especially if the drainage system is strong and efficient enough to allow an effective control of the pore pressure.

If the irreversible displacement is high, the peak friction angle is over passed. The shear friction angle is then reduced to the value of the residual angle.

If the irreversible displacement is sufficiently small, the shear angle remains below the peak, in the dilatant phase of the joint movement.

Assuming an Unique Contact Joint (UCJ) – which is conservative -, the opening of the joint is equal to the irreversible displacement multiplied by tg (i) – i being the angle of dilatancy of the joint. A usual value of this angle (i) – to be determined in fact case by case - is 8° or 9 - giving a value of tg(i) of approximately 0,15.

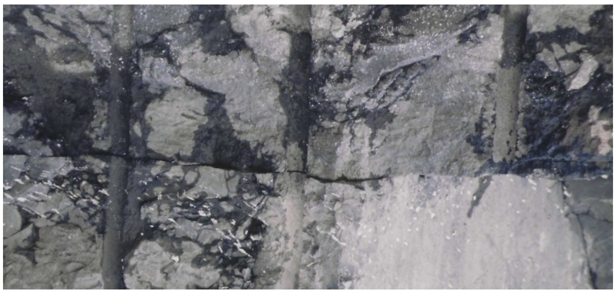


Figure 5. Example of a joint after a shear displacement and dilatancy effect

A formula gives the approximative velocity of the water (m/s) in the crack and consequently the flow of water which can reach the drainage system [R17]:

$$V=12{,}5.\ (e.Jr)^{0,5}$$

where e is the joint opening and Jr the hydraulic gradient at the drainage line.

If the flow is too large to be controlled by the drainage curtain – saturation of the drains -, the water pressure in the contact joint increases dramatically. The full uplift pressure may act as if there were no drainage at all. The stability of the gravity dam should be analysed with this new uplift water pressure. This post-seismic stability is of major importance and it should be guaranteed with a safety coefficient equivalent to the one in static conditions - as it was before the earthquake occurrence. The insurance of the control of the drainage system post-seism gives an idea of the value of the maximum acceptable irreversible displacements which is often of the order of magnitude a few centimetres.

Then the complete sequence is:

1. Evaluate the irreversible displacement at the contact between the body of the dam and the rock of the foundation,
2. Evaluate the opening of this joint taking into account its dilatancy (often between 5° and 10°)
3. Evaluate the leaks through this open joint.
4. Analyse the impact of these leaks on the pressure pattern
5. Determine whether the drainage system can accommodate these leaks without dramatic change of the uplift.

The final aim of this process is to guide the engineering judgement with clear physical phenomena. The irreversible displacement of a dam or of a part of dam – upper blocks - is immediately understandable. The increase of the leaks – often associated with the change of the colour of the water- and its impact on the pore pressure pattern at the contact concrete/ rock foundation may be interpreted by the engineer and compared to the capacity of the drainage system. Conclusion can be expressed in physical and engineering terms leading to adequate decision of reinforcement or not.

Upper blocks

It is recommended to check especially the behaviour of the upper blocks at the crest of the dam where acceleration is frequently significantly above 1 g. Numerical modelling shows that frequently, upper blocks can suffer sliding movements above the lower blocks. At the end of the earthquake, a permanent displacement toward downstream can be assessed by double integration of the excursion of the acceleration above the limit acceleration.

The upper block can also rock toward upstream or downstream. Indeed, the contact between the upper blocks and the lower part of the dam can break for a certain level of acceleration. After rupture, the upper block is independent of the dam body and its behaviour is purely ballistic like the stone of a slingshot. Either the block starts rocking and then comes back to its initial position or it continues beyond the limit of rocking and falls down. The result cannot be calculated by software using small displacements hypothesis. It depends on the velocity at the centroid of the block at the moment when the block turns out to be independent of the dam body, as a stone in a catapult. The engineer should present the consequences of this permanent displacement and decide if it may be accepted or if adequate reinforcements have to be implemented. (See references in next sections -Arch dams).

Simplified method to assess the first mode and amplification

It should be noted that a simplified method developed by Tardieu [R12] can give quickly the frequency N of the first mode:

- N=0,23 S/H for an empty reservoir
- N =0,17 S/H for a full reservoir

where S is the shear wave velocity of the dam body and H the height of the dam.

This value is an approximation useful to check whether the first mode frequency is solicited by the maximum acceleration zone of the spectrum of the earthquake. As the shear wave velocity is roughly always the same, depending on the initial formula of the concrete and its age, this very simple formula shows that the first mode frequency is inversely proportional to the height of the dam. As a consequence, a small gravity dam has a high frequency first mode, frequently beyond the zone of high amplification of the spectrum.

The amplification of the acceleration due to the shape of the dam is function of its elevation [R12]:

- P.G.A a its base
- 2/3 Spectral Acceleration (SA) at one third of its total height (H)
- Spectral Acceleration at 0.6 H
- 2,5 SA at the crest level (H)

In case of strong earthquake, the addition of the slopes of the upstream and downstream faces of a gravity dam should be significantly above 0,8 horizontal / 1 vertical to resist earthquake if the dam is supposed not to suffer any irreversible movement on the foundation. This simplified approach shows that many gravity dams will be considered as unsafe if no irreversible movement is accepted – if the safety factor shall be always above 1. This is clearly overconservative (refer to [R17]).

3.3 Arch dams

General considerations

It has been frequently said that arch dams behave well during the earthquake, namely because they are relatively light, because they are designed to resist horizontal thrust, because they are flexible. It is generally accepted. Favourable references are published.

These positive behaviours of the known arch dams do not allow to extrapolate this good behaviour to stronger earthquakes and to any type of arch dams, and do not allow to anticipate what would be the behaviour of the arch dams if the earthquakes are significantly stronger.

Numerical models

The development of the software dedicated to dam dynamic analysis have been significantly improved during the last decades. The model takes into account the reservoir and the foundation. It includes absorbing boundaries and internal material damping in the concrete of the dam. The elastic response is first analysed. Then, following the accelerogram of the earthquake, the tensile stresses in the vertical joints between blocks are released, the tensile stresses at the contact concrete rock are also released. The effect of the seismic belt (if any) is introduced to represent the behaviour of the upper part of the arch dam. Finally, a plastic Mohr behaviour in the concrete of the dam is inserted to take into account the limit of the dynamic tensile stresses.

The numerical software should also be able to release tensile stresses crossing the construction joints and to determine possible permanent movements at the contact concrete-rock. It could be also able to release the tensile stresses in the concrete mass where they are beyond the accepted dynamic tensile strength.

Opening of the construction joints during the earthquake should be analysed all the more so as there is no seismic belt. The shape and the disposition of the concrete shear boxes in the vertical construction joints between adjacent cantilever (if any) have to be analysed.

Irreversible displacement at the contact concrete-rock should be verified.

Minimum Performance Criteria

The current practice is to use the recommendations of USACE [R3] as for gravity dams involving the Demand Capacity Ratio and the Cumulative Inelastic Duration to assess the minimum performance criteria.

It is also frequently mentioned – as minimum performance criteria - that the vertical stresses in the concrete blocks do not cross the full section of the concrete, because the attention is concentrated on the behaviour of the cantilever and not on the behaviour of the arches. However, an arch dam fundamentally relies on arches.

Finally, the resistance of the dam does not rely not on the capacity of the tensile stresses but relies on the capacity of the compressive strength in the “active” vault “inscribed” inside the mass of the concrete and which transfers the thrust of the reservoir toward the abutment at any moment (see Figure 2 below). The concept of active arch has been introduced by Pigeaud and Resal in 1898 [R1] and widely used by André Coyne as presented in his teaching at Ecole des Ponts et Chaussées [R2]. (See Figure 6 below)

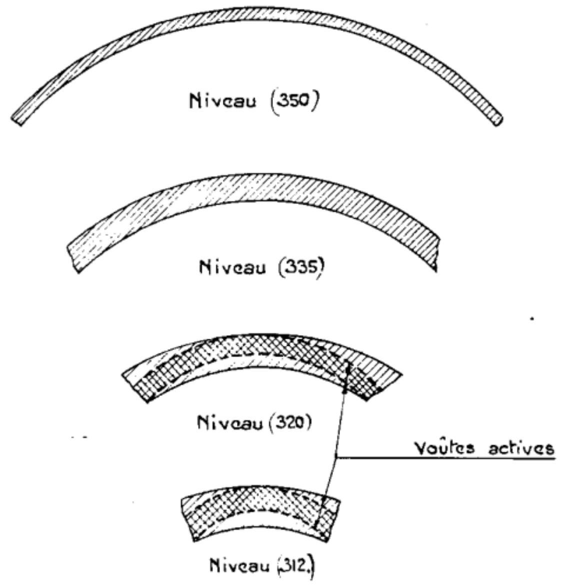


Fig. III.7 Coupes horizontales à différents niveaux.

Figure 6.: Active vault by Coyne

At the end of the day, in a non-reinforced mass of concrete, the final resistance is dictated the compressive strength within the active arches. The engineer should check that these active arches still do exist at the end of the earthquake and that they have not be weakened during the earthquake. If these active arches have been weakened without leading to a potential collapse, the post-seism safety could lead to lower the reservoir.

Upper blocks

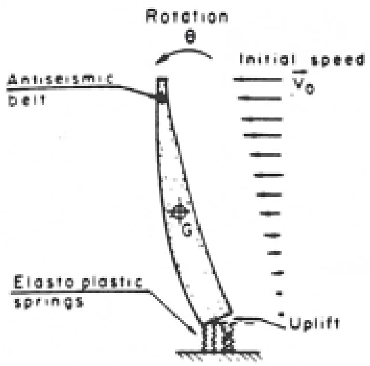
The experience of numerical modelling of strong earthquake indicates that the upper central part of an arch dam is the most sensitive to earthquake - let's say a quarter or a third of the height - and that the lower part has huge reserve of safety if the drainage system allows to control the leaks. Horizontal cracks appear in the upper part, crossing the full thickness of the arch dam and separating the upper part from the lower part for each block separated from the neighbour block by the construction joint.

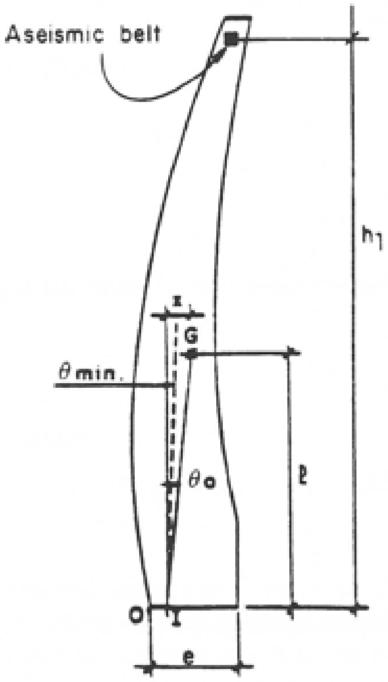
It is one of the justifications of the seismic belts to try to maintain global integrity and continuity of the upper part of the arch dam.

If we try to imagine what is the ultimate state of an arch dam submitted to an increasing earthquake, we may suggest that the lower part of the arch remain almost intact and that the upper quarter of the arch is dismantled by shearing of the concrete crossing the thickness of the arch dam. According to the results of the numerical model and without any experimental demonstration or experienced situation, the concrete in the central upper part of the dam is damaged, it may remain in place, be destroyed and fall down to downstream or for certain shapes of the upper part of the arch dam, fall down towards upstream.

The risk of overturning or rocking of the arch dam or of a part of the arch dam, namely close to the crest depends essentially on the maximum velocity at the centroid of the rocking mass [R15] [R16].

It is assumed that each cantilever resists elastically till an excess of tensile stress during a peak of acceleration. The crack crosses the cantilever whose movements become “free” and independent of the arch dam itself. Once the movement of the upper part of the cantilever is “free”, there is no longer direct transfer of acceleration and the kinematic is only guided by ballistic – i.e., the velocity of the centroid of the upper part toward upstream at the moment of its independence. Hence, if during the movement of the centroid toward upstream, the centroid passes beyond the point of rotation I, the free movement of the cantilever continues and it falls down. If not, the upper part of the cantilever comes back, more or less at its initial place. (See Figure 7 below)





$$E_k + E_g + E_s + E_p + E_f = \text{constant}$$

where

E_k = kinetic energy

$= 1/2\ (I_{yy} + Ml^2)\ \dot{\theta}^2$

E_g = potential energy

$= Mgl \cos(\theta)$

E_s = elastic deformation of the aseismic belt (if any)

E_p = work of water pressure (nil if reservoir is empty)

E_f = elastoplastic deformation of the foundation (neglected)

$\dot{\theta}$ = angular velocity

M = mass of the block

g = acceleration of gravity

I_{yy} = momentum of intertia

G = centre of gravity of the block

I = centre of block rotation

Therefore:

$$\frac{1}{2}\left(\frac{Iyy}{Mgl} + \frac{l}{g}\right)\dot{\theta}^2 + \cos(\theta) + \frac{K\ h_1^2\ lb^2}{Mgl\ 2R^2\ ls}\ (\theta_0 - \theta)^2 = cte$$

With the conditions $\theta_0, \dot{\theta}_0$, and $\dot{\theta} = 0$ when $\theta = \theta_{min}$, the equation becomes:

$$\frac{2l}{3g}\ \dot{\theta}_0^2 + \theta_0(\theta_{min} - \theta_0) - \frac{K\ h_1^2\ lb^2}{Mgl\ 2R^2\ ls}(\theta_0 - \theta_{min})^2 = 0$$

or:

$$AKX^2 - \theta_0 X - C = 0$$

Where:

$$X = \theta_{min} - \theta_0 (X \leq 0)$$

$$A = \frac{h_1^2\ lb^2}{2R^2\ ls\ Mgl}$$

$$C = \frac{2l}{3g}\ \dot{\theta}_0^2$$

Figure 7. Rotation of an independent upper block

This calculation is frequently made with the help of a numerical software call Kineblock or equivalent.

<u>Specific attention to equipment on the crest</u>

Frequently, due to amplification which could be almost 4 to 6, the horizontal acceleration at the crest may be beyond 1g. All equipment located at the crest will be displaced and may fall down. Arrangement of the crest has to be verified and eventually modified.

3.4 Embankment dams

<u>Current practice</u>

It is difficult to propose a general assessment for any kind of embankment dam on any kind of foundation. The presence of a deep alluvial deposit beneath an embankment modifies significantly the response of the dam and its foundation.

When the embankment dam is founded on a rocky foundation, the frequency of the first mode of the dam may be estimated by the following formula:

$$N = \frac{\sqrt{6}}{2\pi} \cdot \frac{S}{H}$$

Where S is the shear wave velocity and H the height of the dam.

On this basis it takes little time to have a first approach of the spectral acceleration and of the acceleration at the crest of the dam as described in [R12] 1985.

The current practice uses the method developed by Seed and Idriss (1970) and the determination of the permanent displacements by the method developed by Newmark (1965).

The curves expressing the value of G/G_{max} and the damping ratio in function of the distortion have to be determined by laboratory tests with the in-situ material collected inside the dam or the foundation.

Briefly, it is to determine the acceleration resulting of the earthquake in any location of the dam, and to select significative circular sliding lines. For each circular line, the limit acceleration is calculated. At each wave of acceleration, when the calculated acceleration exceeds the limit acceleration, the permanent displacement is computed by double integration of the acceleration and the permanent displacements are analysed. Generally, the main impact is considered as its consequences on the settlement of the crest and on the integrity of the filter, drain and the concrete facing for CFRD. Special attention should be given to the crest where acceleration is frequently high during the earthquake. The crests of the embankment dam are frequently impacted and disorganized by an earthquake.

This is the current practice, widely used which gives it a legitimation, because engineers know how to interpret the calculated permanent displacements.

Vertical settlement due to reduction of material volume

However, the observation of embankment dams suffering a strong earthquake generally shows more often a vertical settlement caused by a reduction of volume of the material of the dam body and/or of the foundation if the dam lies on an alluvial deposit.

Indeed, compaction of the granular material – and therefore reduction of volume - is linked to the alternate vibrations which displace the grains in relation to each other, reducing the void ratio.

This type of settlement may be approached by various methods. The improvement of the methods should request research, in-situ measurements and engineering consensus.

Several researches and studies have been presented in the ICOLD Proceedings “Validation of dynamic analysis of dams and their equipment” (Editors Jean Jacques Fry and Morihisa Matsumoto) CRC Press

A Balkema (2016) which is the result of several workshops organized by the Japanese committee on large dams and the French committee on dams and reservoirs [R19].

One of the most interesting studies on this matter was published in English and French in “Revue Française de genie civil 1997 Hermes on the basis of the behaviour of the dam of El Infiernillo (Mexico) under two earthquakes” [R18]. This subject was also presented during the Third Benchmark Workshop ICOLD September 1994 [R17]. Figure 5 below shows the settlement at the crest and the pore pressure in the core during 24 years including construction, impounding and two well-monitored earthquakes. In this study, the software Gefdyn - integrating the Aubry-Hujeux rheologic model with coupled analysis and self-hardening with shear strain - was used.

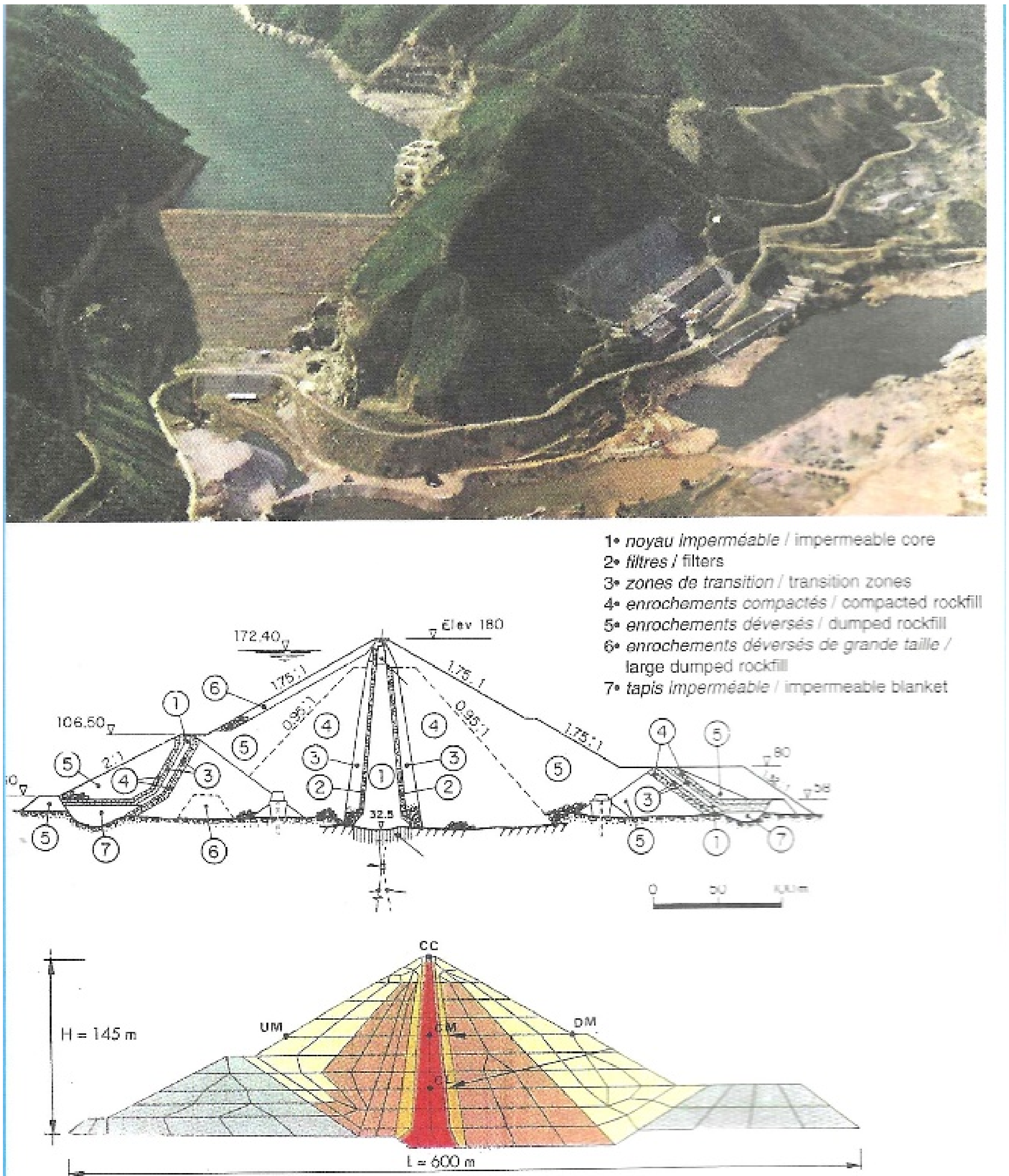


Fig. B1. ***Barrage d'El Infiernillo (Mexique, h = 145 m)***
El Infiernillo dam (Mexico, 145 m high)

Ce barrage de conception classique comporte un noyau argileux assez mince et vertical, tenu par deux recharges en enrochements. Il a subi deux séismes majeurs. Les nombreux essais de laboratoire sur l'argile et les enrochements ainsi qu'une bonne auscultation ont conduit à adopter cet ouvrage pour des ateliers internationaux de comparaison de modèles numériques.

This dam of conventional design has a quite thin vertical clay core between two rockfill shoulders. It has been subject to two major earthquakes. Numerous laboratory tests on the clay and rockfill, together with a good monitoring system, mean this dam has been adopted for international workshops on comparison of numerical models.

Figure 8. EL Infiernillo Dam – Dam materials

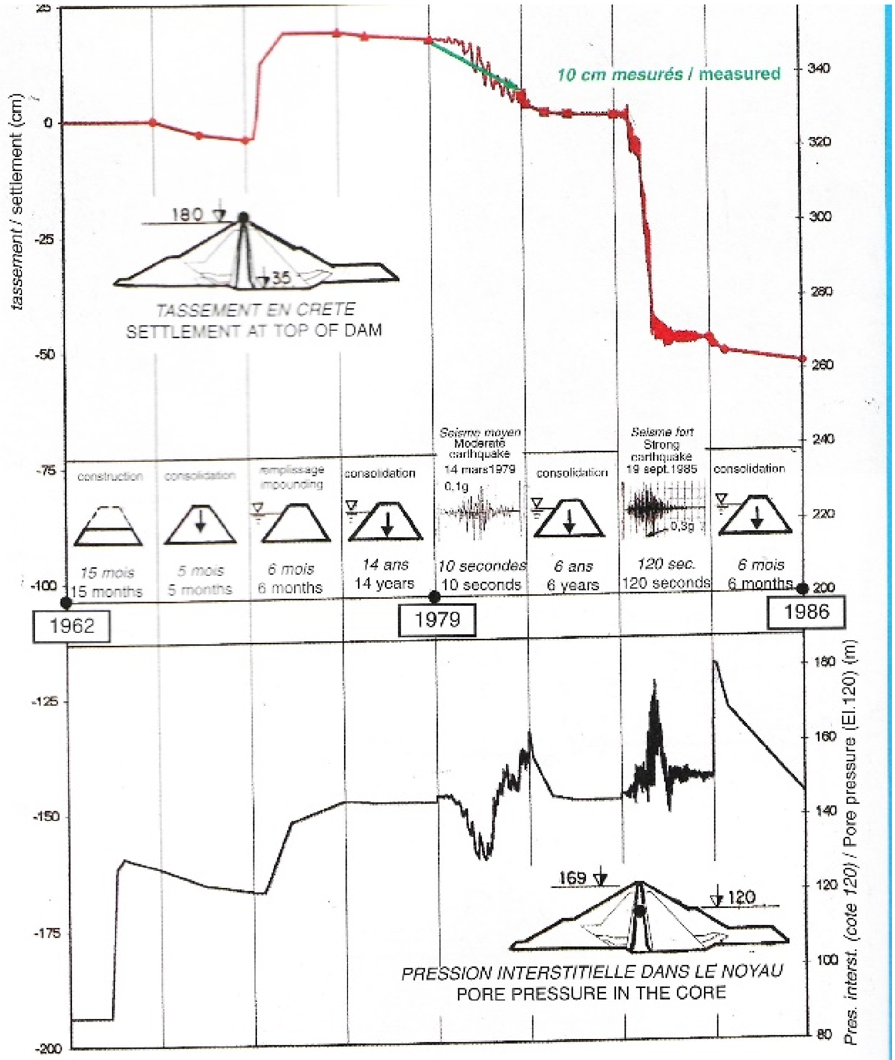


Fig. B2. ***Simulation de l'histoire complète du barrage***
Simulation of the complete dam lifetime

Les courbes de tassement en crête et de pression interstitielle au cœur du noyau caractérisent le comportement observé in situ et reproduit par le modèle numérique. Ceci n'est possible qu'en mettant en œuvre dans le modèle des mécanismes élémentaires (couplage eau/solide, effet de mémoire) qui, par leur combinaison, sont à l'origine de ce comportement.

The calculated curves for settlement of the top of the dam and the curves for pore pressure in the core characterize the behaviour observed in situ. This is only possible by implementing elementary mechanisms (water/solid coupling, memory effect) in the model. Put together, these elementary mechanisms are the cause of the dam's behaviour.

Figure 9. EL Infiernillo Dam – Evolution of the vertical displacement at the crest during the lifespan of the dam

This kind of software also allows to approach the question of liquefaction during an earthquake.

Specific attention for diaphragm wall

When the alluvial deposit is cut by a diaphragm wall, a specific dynamic analysis of the zone of the diaphragm wall is necessary, taking into account the characteristic of the foundation and of the diaphragm wall. The numerical modelling should take into consideration the progressive construction of the cut-off and of the dam.

3.5 Seismic instrumentation of dams

Dam monitoring

Whatever is the type of a dam, it is highly recommended to install adequate seismic instrumentation.

Some of the modern sensors of course, are equipped with both low level excitation and strong motion recording modes. They should be located at the crest in the central part and close to the abutments, and at the level of the centroid of the dam. They should be well protected, frequently tested with a backup of power supply system. Experience shows that many seismic systems do not work the day of the earthquake.

It is crucial to determine whether permanent displacements occurred during earthquake. These types of displacement are frequently observed in the upper part of the dam, including in the crest arrangement, and, when a bridge passes over a spillway, in the support of the bridge slab.

The evolution of the discharge and of the colour of the water collected by the drainage system - Dam foundation, abutments and banks downstream the dam - is of major importance, because it provides evidence of a movement within the rock mass.

These various aspects of instrumentation and observation are well described in the ICOLD Bulletins B027, B046, B062, B137 & B148 (refers to [R20] to [R24]). This large number of bulletins show the evolution of the practice with time.

In-situ measurements of vibrations

Various methods allow the analysis of vibration of the dams by forced vibration with an eccentric or natural low-level vibrations. These method gives the frequencies of the main vibration modes of the dam at very low damping. It is an indirect method to know the dynamic modulus of the material of the dam at low damping.

Academic research on the behaviour of the massive concrete under dynamic solicitations around 2 to 5 Hertz should be developed to determine: internal damping, state of the material after earthquakes, impact of duration and number of tensile strength exceedances.

3.6 Other considerations

In the verification of Dam safety, the following aspects posed by the earthquakes shall also be covered:

- The seiche and long oscillations of the water of the reservoir leading potentially to overtopping of the dam and the appurtenant structures, with all induced consequences. These phenomena can be triggered by fault movements in the reservoir or strong ground shaking.
- Land sliding in the banks of the reservoir creating waves, with periods shorter than the seiches, and potential overtopping on the dam and the appurtenant structures
- Modification of the hydrogeology and of the pore pressure pattern in the foundation and specifically for the arch dams in the abutments
- Progressive regressive erosion in the dykes in particular the small ones insufficiently equipped with adequate filters.
- Rockfalls on the dam and the appurtenant spillway, in particular the spillway.
- Gates disturbed or blocked by over pressure positive and negative on their plate,
- And for the Hydro Power Plant, loss of the switchyard and power transmission and excessive velocity of the turbines leading to collapse.
- Other ones learned by experience

These aspects represent a significative part of the consequences of an earthquake.

3.7 Conclusions and recommendations for the Dam Safety

The main conclusions and recommendation on C3-2021 guideline by the dam experts are summarized below.

1. Philosophy of the new C3-2021 guideline

The previous guidelines - 2003 OFEG guidelines - were more prescriptive with the advantage to better describe the methodology to follow to verify the safety of the dam against earthquake. On the other hand, it presented the disadvantage to be less flexible for the possible future evolution of the state of the art.

The POE believes that the philosophy followed by the new guideline is relevant.

The POE also notes that 2003 OFEG guidelines can still be used as an indicative reference especially for the determination of the material characteristics – in absence of in-situ or laboratory tests - and for the methodology to be used for the verification of the safety of the embankments.

2. Phased studies progressively more complex

The verification of the dam safety against earthquake is dependent on many parameters as the seismic hazard, the dam foundation, the size and quality of dam. It can be simple or very complex. It is therefore recommended to carry out phased studies increasing – if necessary - the complexity of the verification step by step. This approach is followed by the French Guidelines [R4] and USSD recommendations [R6]. This permits a better proportionality of the verification.

3. Minimum Performance Criteria

It is implicit that damage is acceptable during and after a SEE should this damage not induce uncontrolled release of water.

It is therefore recommended to give more guidance – in the C3-2021 guideline - about the Minimum Performance Criteria of a dam during and after an earthquake. The various aspects of the consequences of the irreversible or permanent displacements should be clearly addressed.

It should be also noted that the minimum performance criteria can evolve with the state of the art.

In the C3-2021 Guideline, it is stated that “Verification of seismic safety is necessary for all water retaining facilities, and it is insisted that it is necessary to take into account changes in the state-of-the-art in science and technology and to consider these changes in the assumptions made in a previous verification of seismic safety.”

We agree with this methodology. It should be noted that the state of the art, or the good practices, are gradually changing and improving. The penetration of these good practices in the current and usual day-to-day professional production of the engineers follows a low progression. It is of the responsibility of the administrative control system to encourage this penetration.

The definition of the acceptable or not acceptable value of the irreversible displacements should remain in the domain of the engineering judgment and of the socio-economic-political responsibility. This judgment shall be based on the analysis of the consequences of these permanent displacements on the safety of the dam and its appurtenant structures. It shall be shared and validated by the “controller” - SFOE.

4. “Simplified method” for Category III embankment dam

Studies and verifications should also be proportioned to the size of the dams and the ease or difficulties to demonstrate its safety.

Thus, it is possible to simplify the verification for small dams in C3-2021 Guideline as permitted by the French guidelines [R4] and stipulated by USSD [R6].

Indeed, when a small embankment is well built, the seismic load case – for moderate PGA - is not the critical load case.

The following preliminary studies analysis shall be firstly performed in all cases.

- Geotechnical and geological model development
- Seismic hazard analysis
- Verification of the potential liquefaction and cycling softening

Then, specific verification dependent on the acceleration of the earthquake could be carried out.

<u>Acceleration below 0,1g (a_{gh})</u>

For small dam, the verification of the safety of the dam against earthquake is not necessary when the following conditions are met:

1.Dam and Foundation materials that are not liquefiable and do not contain sensitive clay

2.Well-built, densely compacted dam with relative compaction RC > 95 % or relative density Dr > 80%

3. PGA < 0.1 g at the base of the embankment.

4. Static factor of safety before the earthquake greater than 1.5

5. At the time of the earthquake, freeboard greater than 3 percent of the dam height (including alluvial foundation) but not less than 1 m.

Acceleration above 0,1 g (a_{gh})

Should the potential liquefaction and/or cyclic softening of materials are discarded, a simplified approach could be used to demonstrate the safety of the dam. This simplified approach consists of two stages.

- Stage 1: verification of the risk of crest settlement
- Stage 2: verification of the risk of sliding

Correlations- empirically-based and analytically-based - shall be firstly considered.

Empirically-based correlation (e.g Sawaigsood,2014) can show if there is a potential risk of crest settlement.

Should this risk be discarded, “simplified dynamic method” can be used for verification of the stability against sliding.

For this verification, the horizontal acceleration for sliding masses is taken equal to a_{gh} and it is not necessary to consider the vertical acceleration. If the safety factor is below 1, the other methods recommended for category II and 1 should be used.

In conclusion, the stability against sliding can be verified by a simplified approach – as pseudo-static calculation, when the following conditions are met:

Dam and Foundation materials that are not liquefiable and do not contain sensitive clay

There is no risk of crest settlement above the freeboard.

Should the safety of dam against sliding be not demonstrated with this simplified approach, the other methods recommended for Category II and I should be used.

5. Complex numerical modelling – State of the art

The present state-of-the-art uses numerical modelling more and more elaborated and involving many aspects of the earthquake response of dams.

The use of numerical modelling is a simplification which ignores unavoidably non-included features. The modelling represents a certain level of simplification depending on the evolution of the science, on the capacity of the computers and on the time devoted to the analysis, in particular to collect the parameters to be included in the modelling. At each period of historical time, modelling represents the state-of-the-art at this time. Regulation has to take into account this evolution.

In the practice, dynamic design of dams includes a certain level of conservatism incorporated into the safety factors and in the determination of the parameters. Modelling helps to appreciate the margin of safety included in the design currently accepted. This apparent conservatism may also cover eventual unknown negative margin namely in the seismic behaviour which is not frequently observed and measured.

Complex numerical models can be used to demonstrate the safety of the dam.

However, it should be noted that, among all these approaches, certain are not really conclusive and frequently the judgment of the engineer is sought. For frequent earthquakes, the experience, the observation of the actual behaviour of dams, in-situ measurements may also guide the engineering judgment.

When the earthquake is very strong, beyond the current experience, it should be also noted that it is not easy to find a convincing and conclusive process to determine whether or not a dam may be considered as safe. It is not easy for the engineers to determine the ultimate state of a dam just before it collapses, and how to determine what is the margin of safety before this ultimate state.

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5 APPENDIX 1: REVIEW OF C_A FACTOR

5.1 PRELIMINARY CONSIDERATIONS:

In the Directive C3, the proposed C_A factor to convert the Swiss reference rock (Vs30=1100 m/sec) into standard rock (Vs30=800 m/sec) is 1.6. This adjustment factor is to be applied to the full range of ground motions i.e., is independent of the level of ground shaking and not strain-dependent. This assumes that there is no non-linear behavior going from 1100 to 800 m/sec. Note C_A has also been called an amplification factor but in this report, adjustment factor is a more appropriate term since the factor is used to adjust from hard to standard rock. We note that a value of 1.6 is:

1) Higher than similar adjustment factors found in other guidelines.
2) The CA factor is not consistent with similar factors based on the NGA-West2 ground motion data and models.

These points are described in more detail below:

5.2 ASCE 7 Code

ASCE 7-16 *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* considers two site coefficients Fa (0.2 sec) and Fv (1.0 sec). In ASCE 7-16, the “hard rock” is identified as site class A (Vs30>1500 m/sec) and standard rock which corresponds to site class B with a Vs30 ranging from 760 to 1500 m/sec. The Fa and Fv tables are shown below:

Table 11.4-1 Short-Period Site Coefficient, F_a

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at Short Period					
	$S_S \le 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S = 1.25$	$S_S \ge 1.5$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.3	1.2	1.2	1.2	1.2
D	1.6	1.4	1.2	1.1	1.0	1.0
E	2.4	1.7	1.3	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Table 11.4-2 Long-Period Site Coefficient, F_v

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at 1-s Period					
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \ge 0.6$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4
D	2.4	2.2^a	2.0^a	1.9^a	1.8^a	1.7^a
E	4.2	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Although we recognize adjusting from site class A (1500 m/sec) to B (lower range of 760 m/sec) is not exactly the same as going from 1100 to 800 m/sec, the implied factor for Fa (0.2 sec) is significantly lower than 1.6 (0.9/0.8 = 1.125).

5.3 French seismic code for large dams

The soil amplification factors proposed by this seismic guideline are presented here below:

L’exploitant prend ensuite en compte la nature du sol sur lequel est implantée l’installation par l’intermédiaire des coefficients ci-après. Les valeurs du paramètre de sol S résultant de la classe de sol (A, B, C, D ou E au sens de la norme NF EN 1998-1) sous l’installation sont les suivantes :

Classes de sol	S (pour les zones de sismicité 1 à 3)	S (pour les zones de sismicité 4 et 5)
A	1	1
B	1,35	1,2
C	1,5	1,15
D	1,6	1,35
E	1,8	1,4

In this guideline, soil class A correspond to rock with Vs30>800 m/sec and therefore there is no hard rock site class. However, we note that:

1) The adjustment factors for soil class B (360 to 800 m/sec) and C (180 to 360 m/sec) are lower than 1.6.
2) The factors for zones 1 to 3 (low to moderate seismic hazard and lower ground motions) are higher than for zones 4 and 5 (high seismic hazard and higher ground motions).

Although these site classes have lower Vs30 values than 800 m/sec, the adjustment factors are still lower than 1.6 and it would be expected that they would be higher than going from hard to firm rock.

5.4 Amplification factors derived from NGA-2 West GMPEs

In addition to the previous observations, we note that the adjustment factors at 0.15 sec (also at PGA) going from 1100 to 800 m/sec that are implied in the five NGA-West ground motion models (GMMs) for a **M** 6.0 event at Rrup=20 km and Rrup=5km is about 1.2. This M-R combination is close to the “dominant” M-R scenarios in Switzerland, for applicable return periods (1000 y-RP to 10000 y-RP). The parameters used in the NGA-West2 calculations at 0.15 sec are listed below:

- Abrahamson et al (2014):

○ **M=6.0** ○ **Rrup=20km** ○ Focal mechanism=Reverse ○ Region=tectonically active regions ○ Vs30=800 m/sec and 1100 m/sec ○ Ztor=5 km ○ Depth Vs=1000=Negative ○ PGA(800 m/sec)=88 cm/sec2 ○ PGA(1100 m/sec)=76 cm/sec2 ○ **Adjustment factor for PGA= 1.16** ○ 0.15 sec SA (800 m/sec) = 197 ○ 0.15 sec SA (1100 m/sec) = 157 ○ **Adjustment factor T=0.15sec = 1.25**	○ **M=6.0** ○ **Rrup=5km** ○ Focal mechanism=Reverse ○ Region=tectonically active regions ○ Vs30=800 m/sec and 1100 m/sec ○ Ztor=5 km ○ Depth Vs=1000=Negative ○ PGA(800 m/sec)=300 cm/sec2 ○ PGA(1100 m/sec)=258 cm/sec2 ○ **Adjustment factor for PGA=1.16** ○ 0.15 sec SA (800 m/sec) = 692 ○ 0.15 sec SA (1100 m/se) =496 ○ **Adjustment factor T=0.15sec = 1.39**

- Campbell & Bozorgnia (2014)

<table>
<tr>
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M=6.0
Rrup=20km
Focal mechanism=Reverse
Region=tectonically active regions
Vs30=800 m/sec and 1100 m/sec
Slope fault=90 degrees
Depth =5 km
Depth Vs=2.5 km/sec=1 km
PGA(800 m/sec)=92 cm/sec2
PGA(1100 m/sec)=84 cm/sec2
Adjustment factor for PGA= 1.10
0.15 sec SA (800 m/sec) = 222
0.15 sec SA (1100 m/sec) = 198
Adjustment factor T=0.15sec = 1.12

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M=6.0
Rrup=5km
Focal mechanism=Reverse
Region=tectonically active regions
Vs30=800 m/sec and 1100 m/sec
Slope fault=90 degrees
Depth =5 km
Depth Vs=2.5 km/sec=1 km
PGA(800 m/sec)=300 cm/sec2
PGA(1100 m/sec)=276 cm/sec2
Adjustment factor for PGA= 1.08
0.15 sec SA (800 m/sec) = 670 cm/sec2
0.15 sec SA (1100 m/sec) = 606 cm/sec2
Adjustment factor T=0.15sec = 1.10

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- Chiou & Youngs (2014)

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M=6.0
Rrup=20km
Focal mechanism=Reverse
Region=tectonically active regions
Vs30=800 m/sec and 1100 m/sec
Ztor=5 km
Slope fault=90 degrees
Depth Vs=1000km/sec = 0
PGA(800 m/sec)=83 cm/sec2
PGA(1100 m/sec)=70 cm/sec2
Adjustment factor for PGA= 1.19
0.15 sec SA (800 m/sec) = 202
0.15 sec SA (1100 m/sec) = 172
Adjustment factor T=0.15sec = 1.17

</td>
<td>

M=6.0
Rrup=5km
Focal mechanism=Reverse
Region=tectonically active regions
Vs30=800 m/sec and 1100 m/sec
Ztor=5 km
Slope fault=90 degrees
Depth Vs=1000km/sec = 0
PGA(800 m/sec)=288 cm/sec2
PGA(1100 m/sec)=246 cm/sec2
Adjustment factor for PGA= 1.17
0.15 sec SA (800 m/sec) = 716
0.15 sec SA (1100 m/sec) = 616
Adjustment factor T=0.15sec = 1.16

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</table>

- Idriss (2014)

○ **M=6.0** ○ **Rrup=20km** ○ Focal mechanism=Reverse ○ Region=tectonically active regions ○ Vs30=800 m/sec and 1100 m/ecs ○ PGA(800 m/sec)=80 cm/sec2 ○ PGA(1100 m/sec)=61 cm/sec2 ○ **Adjustment factor for PGA= 1.31** ○ 0.15 sec SA (800 m/sec) = 167 ○ 0.15 sec SA (1100 m/sec) = 125 ○ **Adjustment factor T=0.15sec = 1.34**	○ **M=6.0** ○ **Rrup=5km** ○ Focal mechanism=Reverse ○ Region=tectonically active regions ○ Vs30=800 m/sec and 1100 m/sec ○ PGA(800 m/sec)=256 cm/sec2 ○ PGA(1100 m/sec)=195 cm/sec2 ○ **Adjustment factor for PGA= 1.31** ○ 0.15 sec SA (800 m/sec) = 524 ○ 0.15 sec SA (1100 m/sec) = 366 ○ **Adjustment factor T=0.15sec = 1.43**

- Boore et al. (2014)

○ **M=6.0** ○ **Rrup=20km** ○ Focal mechanism=Reverse ○ Region=tectonicallyactive regions ○ Depth Vs=1000km/ecs = 0 ○ Vs30=800 m/sec and 1100 m/sec ○ PGA(800 m/sec)=91 cm/sec2 ○ PGA(1100 m/sec)=75 cm/sec2 ○ **Adjustment factor for PGA= 1.21** ○ 0.15 sec SA (800 m/sec) = 241 ○ 0.15 sec SA (1100 m/sec) = 200 ○ **Adjustment factor T=0.15sec = 1.21**	○ **M=6.0** ○ **Rrup=5km** ○ Focal mechanism=Reverse ○ Region=tectonically active regions ○ Depth Vs=1000km/sec = 0 ○ Vs30=800 m/sec and 1100 m/sec ○ PGA(800 m/sec)=262 cm/sec2 ○ PGA(1100 m/sec)=216 cm/sec2 ○ **Adjustment factor for PGA= 1.21** ○ 0.15 sec SA (800 m/sec) = 688 cm/sec2 ○ 0.15 sec SA (1100 m/sec) = 572 cm/sec2 ○ **Adjustment factor T=0.15sec = 1.20**

The arithmetic MEAN of the 5 amplification factors obtained using the NGA- West2 GMMs is 1.19 for PGA and 1.22 for T=0.15sec, for Rrup =20 km and it is 1.18 for PGA and 1.25 for T=0.15sec, for Rrup =5 km.

In a paper published by Al Atik et al. (2022), they developed hard rock scaling factors to adjust the NGA-West2 GMMs. These factors can be used to adjust ground motions for a Vs30 of 760 m/sec to any Vs30 up to 1,000 to 2,200 m/sec. Their scaling factor to go from 760 m/sec to 1,100 m/sec at 0.15 sec is 0.8606. The inverse of that would be is 1.162, significantly lower than 1.6.

A question may be raised on whether the NGA-West2 GMMs are appropriate for Switzerland. The SUIHaz2015 maps utilized four empirical GMMs for active tectonic regions: Chiou and Youngs (2008) the predecessor to Chiou and Youngs (2014), Cauzzi and Faccioli, (2008), Akkar and Bommer (2010), and Zhao et al. (2006). The NGA-West2 models are also for active tectonic regions. We understand that the GMMs used in SUIHaz2015 have been adjusted for kappa and velocity profiles but the changes are not significant enough to result in a large difference going from 1100 to 800 m/sec using the NGA-West2 models.

5.5 ANALYSIS OF DUVERNEY ET AL. (2019)

The report by Duvernay et al. (2019) provides a short summary on the justifications for the adoption of C_A=1.6. The adopted CA is justified using 4 different reports:

5.5.1 Felicetta et al. 2018:

A C_A=1.66 was estimated using the plot shown below where C_A (soil coefficient) to convert site class A (Vs30>800 m/sec) and soil class EC8-B (360 m/sec <Vs30<800 m/sec) into “reference rock”, A_{ref}, for Italy, resulting in a value of $10^{(0.22)}$. We note that the definition of “reference rock” in Felicetta et al. (2018) was defined using six proxies based on geological, topographical and geophysical data to identify reference rock sites (A_{ref}) and, therefore, it does not correspond exactly to the Swiss reference rock (based only on Vs30=1100 m/sec). The objective of Felicetta et al. (2018) study was to introduce a new site soil category and to re-calibrate the ITA10 prediction equations for this new soil class and not to determine an amplification factor between A_{ref} and soil class A or B, although it was presented (see Figure 7 here below). In Figure 7, at T=0.15 sec, the amplification factor proposed for soil class A (factor to convert reference rock into soil class rock) is higher than the amplification factor proposed for soil class B, which seems unusual.

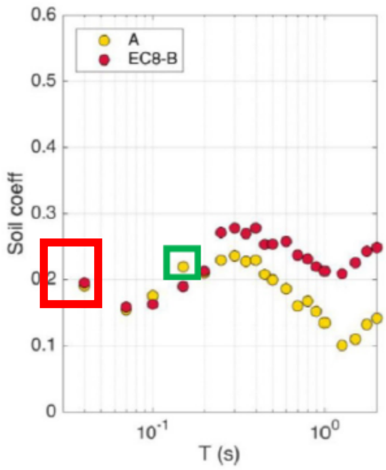

Fig 7. Soil coefficient for classes A and EC-8 in Felicetta et al. (2018).

This report justifies the adoption of C_A=1.6 using four M-R pairs and Italian data which raises the question whether such data are appropriate for Switzerland.

Additionally, we note that:

- The adjustment factor obtained for M=6.0 is lower than the value obtained using M=5.0 (see Figure 6 of Felicetta el al., 2018 here below). The amount of reduction is directly related to an amplification factor). This strain-dependent pattern is also shown in ASCE7 and the French seismic code for large dams.
- The adjustment factor for PGA (red rectangle in above figure) is significantly lower than the amplification value for T=0.15s (green rectangle).

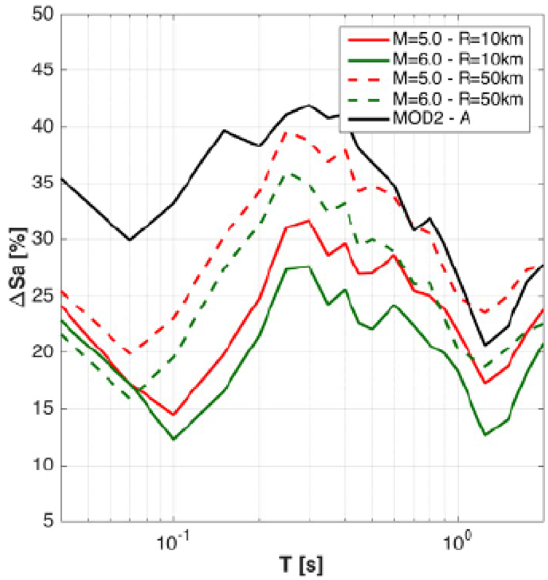

Fig. 6. Percentage of reduction between the predictions of ITA10 for EC8-A and those of MOD2-Aref for two magnitudes (M5 and M6) and distances (10 and 50 km), as a function of period. The black line represents the reduction between the prediction of MOD2 for generic rock site and reference rock site.

5.5.2 SED, 2017 report (file Vs30Kappa_SED_2017_asa.pdf)

The figure that seems to justify the CA=1.64 value is presented here below (Figure A2 in Duverney et al., 2019):

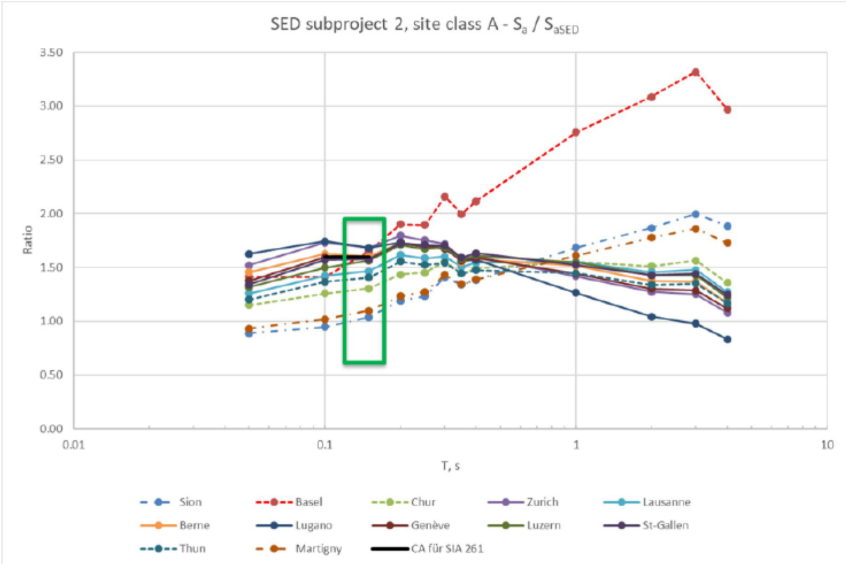

Figure A2: Relationship between response spectra for soil class A and UHS on the SED reference soil [SED 2017b].

We were unable to find Figure A2 in the report by Danciu and Fah (2017*) Adjustment of the 2015 updates of the Swiss Hazard Model to different rock conditions (Vs-kappa adjustment)*. This is a key figure which probably provides the strongest argument for a C_A of 1.6. The adjustment factors derived in Danciu and Fah (2017) are close to 1.1 or 1.2. The report's Appendix the adjustment factors for 0.15 sec, the listed C_A factor 1.1158 (red rectangle). The adjustment factor for PGA is only 1.0525 (blue rectangle). It is unclear how the stated 1.64 was derived considering the available information. Additional details about that should be requested to SED.

5 APPENDIX

Adjustment factors (ratios) with constant kappa - reference rock *Vs30=1100 m/s, kappa = 0.016.*

Periods(s)	Vs30= 500	Vs30= 600	Vs30= 700	Vs30= 800	Vs30= 900	Vs30= 1000	Vs30= 1100
0.01	1.2294	1.1454	1.0899	$\boxed{1.0525}$	1.0264	1.0097	1.0000
0.05	1.0965	1.0465	1.0191	1.0046	0.9978	0.9972	1.0000
0.10	1.2880	1.1947	1.1308	1.0844	1.0492	1.0221	1.0000
0.15	1.3690	1.2559	1.1757	$\boxed{1.1158}$	1.0691	1.0312	1.0000
0.20	1.4078	1.2838	1.1959	1.1302	1.0774	1.0351	1.0000
0.25	1.4299	1.3007	1.2084	1.1384	1.0834	1.0380	1.0000
0.30	1.4398	1.3085	1.2139	1.1423	1.0855	1.0394	1.0000
0.35	1.4437	1.3118	1.2167	1.1441	1.0867	1.0396	1.0000
0.40	1.4440	1.3121	1.2172	1.1450	1.0865	1.0401	1.0000
1.00	1.3431	1.2431	1.1704	1.1143	1.0689	1.0316	1.0000
2.00	1.1971	1.1436	1.1028	1.0704	1.0434	1.0202	1.0000
3.00	1.1358	1.1012	1.0736	1.0508	1.0317	1.0151	1.0000
4.00	1.1053	1.0789	1.0577	1.0400	1.0250	1.0117	1.0000

5.5.3 Poggi et al. (2013, 2017):

We do not have the report cited as Poggi et al. (2013a) but assume that Poggi et al. (2017) is the final version of that study. The value 1.64 proposed is the mean of the four factors indicated in green rectangles shown below. The range in values reportedly represents the epistemic uncertainty in site response analysis. The modeling was based on three modeling approaches using Japanese data. The reference rock is reportedly 800 m/sec but there is no reference to 1100 m/sec in the report. How do these factors relate to C_A?

Moreover, the ratios presented in Table 5 of Poggi et al. (2017) are not clearly explained in the article. It appears that the ratios presented could be the ratios of the amplification factors obtained using different methodologies (AUTH, SED, METU) instead of direct amplification ratios. For example, the amplification factors for class A are higher than for Vs>700 m/sec, which is unusual (see Table 5 below). Moreover, the ratios (amplification factors?) presented in Poggi 2013 and 2017 articles show decreasing ratios with Vs30, while normally we would expect increasing ratios with Vs30 (see tables here below). We understand that the site classes are adopted from AUTH but it is unclear what class A is. Is it the same as class A in Eurocode 8 where Vs30 > 800 m/sec?

	AUTH/SED - Class Selection																
Freq. (Hz)	1	1.27	1.62	2.07	2.64	3.36	4.28	5.46	6.95	8.86	11.29	14.38	18.33	23.36	29.76	48.33	100
A	3.05	2.97	2.91	2.86	2.83	2.84	2.85	2.54	2	1.71	1.66	1.75	1.82	1.94	2.09	2.01	1.95

	AUTH/SED - Velocity Selection																
Freq. (Hz)	1	1.27	1.62	2.07	2.64	3.36	4.28	5.46	6.95	8.86	11.29	14.38	18.33	23.36	29.76	48.33	100
>700m/s	2.05	2.09	2.13	2.16	2.19	2.19	2.15	2	1.73	1.57	1.63	1.88	2.15	2.21	2.16	1.9	1.76

Table 4. *AUTH versus SED average response spectral amplification factors.*

	METU/SED - Class Selection																
Freq. (Hz)	1	1.27	1.62	2.07	2.64	3.36	4.28	5.46	6.95	8.86	11.29	14.38	18.33	23.36	29.76	48.33	100
A	1.74	1.72	1.7	1.73	1.78	1.85	1.98	1.89	1.61	1.48	1.53	1.68	1.71	1.75	1.83	1.68	1.6

	METU/SED - Velocity Selection																
Freq. (Hz)	1	1.27	1.62	2.07	2.64	3.36	4.28	5.46	6.95	8.86	11.29	14.38	18.33	23.36	29.76	48.33	100
>700m/s	1.3	1.31	1.34	1.37	1.4	1.4	1.43	1.37	1.24	1.17	1.26	1.5	1.7	1.73	1.68	1.46	1.37

Table 5. *METU versus SED average response spectral amplification factors.*

Table 3
AUTH versus METU average response spectral amplification factors.

Freq. (Hz)	1	1.27	1.62	2.07	2.64	3.36	4.28	5.46	6.95	8.86	11.29	14.38	18.33	23.36	29.76	48.33	100
AUTH/METU - Class Selection																	
A	1.75	1.73	1.71	1.65	1.59	1.53	1.44	1.35	1.24	1.16	1.09	1.04	1.07	1.11	1.14	1.2	1.22
B1 + B2	1.25	1.25	1.26	1.26	1.21	1.22	1.23	1.24	1.25	1.25	1.25	1.24	1.23	1.22	1.21	1.19	1.17
C1 + C2 + C3	1.37	1.38	1.41	1.18	1.03	1.08	1.15	1.22	1.3	1.36	1.38	1.4	1.38	1.34	1.31	1.27	1.26
E	1.03	1.04	1.06	1.09	1.18	1.31	1.36	1.4	1.46	1.5	1.51	1.51	1.48	1.44	1.41	1.36	1.33
AUTH/METU - Velocity Selection																	
>700 m/s	1.64	1.64	1.62	1.59	1.58	1.58	1.53	1.48	1.41	1.36	1.31	1.27	1.27	1.29	1.3	1.32	1.31
600–700 m/s	1.31	1.3	1.3	1.3	1.29	1.28	1.27	1.26	1.25	1.23	1.22	1.2	1.19	1.19	1.18	1.17	1.15
500–600 m/s	1.38	1.37	1.36	1.3	1.26	1.31	1.32	1.34	1.36	1.38	1.38	1.38	1.37	1.35	1.33	1.31	1.29
400–500 m/s	1.27	1.27	1.29	1.18	1.09	1.14	1.18	1.22	1.27	1.31	1.34	1.35	1.33	1.31	1.28	1.25	1.23
300–400 m/s	1.13	1.14	1.18	1.09	1.02	1.09	1.17	1.24	1.32	1.39	1.42	1.45	1.42	1.37	1.33	1.28	1.26
200–300 m/s	1.03	1.03	1.07	0.97	0.91	1	1.11	1.2	1.32	1.4	1.4	1.41	1.39	1.33	1.28	1.22	1.21

Table 4
AUTH versus SED average response spectral amplification factors.

Freq. (Hz)	1	1.27	1.62	2.07	2.64	3.36	4.28	5.46	6.95	8.86	11.29	14.38	18.33	23.36	29.76	48.33	100
AUTH/SED - Class Selection																	
A	3.05	2.97	2.91	2.86	2.83	2.84	2.85	2.54	2	1.71	1.66	1.75	1.82	1.94	2.09	2.01	1.95
B1 + B2	1.55	1.54	1.53	1.52	1.43	1.37	1.32	1.28	1.24	1.21	1.31	1.54	1.79	1.79	1.67	1.42	1.31
C1 + C2 + C3	1.41	1.29	1.2	0.93	0.74	0.72	0.71	0.73	0.82	0.9	1.02	1.18	1.36	1.3	1.16	0.95	0.88
E	1.36	1.29	1.23	1.2	1.17	1.06	0.94	0.91	0.97	1.03	1.16	1.38	1.62	1.57	1.4	1.15	1.05
AUTH/SED - Velocity Selection																	
>700 m/s	2.05	2.09	2.13	2.16	2.19	2.19	2.15	2	1.73	1.57	1.63	1.88	2.15	2.21	2.16	1.9	1.76
600–700 m/s	1.76	1.77	1.78	1.79	1.76	1.66	1.59	1.5	1.38	1.3	1.38	1.64	1.92	1.94	1.84	1.57	1.44
500–600 m/s	1.88	1.86	1.83	1.72	1.59	1.44	1.33	1.27	1.25	1.23	1.35	1.62	1.92	1.91	1.76	1.48	1.36
400–500 m/s	1.69	1.61	1.56	1.36	1.15	1.02	0.92	0.91	0.96	1.02	1.14	1.36	1.61	1.57	1.42	1.18	1.09
300–400 m/s	1.19	1.1	1.04	0.88	0.74	0.69	0.66	0.68	0.77	0.87	1	1.16	1.33	1.26	1.11	0.91	0.83
200–300 m/s	0.91	0.75	0.64	0.49	0.42	0.45	0.49	0.54	0.63	0.74	0.84	0.95	1.06	0.98	0.85	0.69	0.63

Table 5
METU versus SED average response spectral amplification factors.

Freq. (Hz)	1	1.27	1.62	2.07	2.64	3.36	4.28	5.46	6.95	8.86	11.29	14.38	18.33	23.36	29.76	48.33	100
METU/SED - Class Selection																	
A	1.74	1.72	1.7	1.73	1.78	1.85	1.98	1.89	1.61	1.48	1.53	1.68	1.71	1.75	1.83	1.68	1.6
B1 + B2	1.24	1.22	1.2	1.19	1.16	1.1	1.05	1.02	0.98	0.96	1.04	1.23	1.44	1.45	1.37	1.18	1.1
C1 + C2 + C3	1.01	0.92	0.84	0.78	0.72	0.66	0.62	0.61	0.63	0.66	0.74	0.85	0.99	0.97	0.88	0.75	0.7
E	1.29	1.21	1.14	1.08	0.97	0.79	0.68	0.63	0.65	0.68	0.76	0.9	1.08	1.07	0.98	0.83	0.77
METU/SED - Velocity Selection																	
>700 m/s	1.3	1.31	1.34	1.37	1.4	1.4	1.43	1.37	1.24	1.17	1.26	1.5	1.7	1.73	1.68	1.46	1.37
600–700 m/s	1.39	1.39	1.4	1.41	1.39	1.3	1.25	1.18	1.1	1.04	1.12	1.36	1.61	1.63	1.55	1.33	1.24
500–600 m/s	1.34	1.33	1.32	1.31	1.25	1.1	1	0.95	0.92	0.9	0.98	1.17	1.41	1.42	1.32	1.13	1.05
400–500 m/s	1.33	1.26	1.2	1.15	1.05	0.89	0.78	0.74	0.76	0.78	0.86	1.01	1.21	1.21	1.11	0.95	0.89
300–400 m/s	1.04	0.95	0.87	0.8	0.73	0.63	0.56	0.55	0.58	0.63	0.7	0.8	0.94	0.92	0.84	0.71	0.66
200–300 m/s	0.85	0.71	0.59	0.51	0.46	0.46	0.44	0.45	0.49	0.53	0.61	0.68	0.77	0.74	0.67	0.57	0.53

5.5.4 Personal communication from Laurentiu Danciu, SED 2018.

In Duverney et al. (2019), it is said that “*Hazard data for spectral accelerations at T = 0.15 s and return periods of 475 and 2475 years were used. From these analyses, the C_A factor ranges from 1.0 to 2.64, with an average value of 1.7*”.

Again, this average value seems high based on our arguments above. The global NGA-West2 database suggests an adjustment factor of 1.1 to 1.2. Is the difference between the values cited in Duverney et al. (2019) and the global database due to regional differences? The global database contains a much more robust range of magnitudes and distances and hence ground motion amplitudes. Even though the adjustment factor is to go from 1100 to 800 m/sec, the C_A of 1.6 seems to reflect levels of ground motions in the small to moderate magnitude range rather than at high ground motions that are of engineering relevance.

<u>Summary of comments:</u>

1) Existing seismic codes present systematically smaller adjustment factors to convert “standard rock” into “hard rock” and they are globally around 1.2. We note that the definition of “standard” rock and “hard” rock could differ depending on the seismic code.
2) Existing seismic codes propose systematically smaller adjustments as the ground motions increase. i.e., strain-dependent.
3) The direct application of five NGAWest-2 GMMs (taking into account an hypothetical scenario, M=6.0, Rrup=20 km) for Vs30 values of 800 and 1,100 m/sec gives a mean adjustment factor of 1.22 at 0.15 sec. At PGA, the adjustment factors are in general agreement with the values proposed in some seismic codes. The study by Al Atik et al. (2022) gives an adjustment factor of 1.16.

4) Felicitta et al. (2018) seems to confirm that adjustment factors decrease as ground motions increase (adjustment factors using **M** 6.0 are lower than using **M** 5.0).
5) The C_A factor given in Felicitta et al. (2018) is developed using Italian data (not Swiss data) and for application to seismic ground motions of the Eurocode 8, essentially associated with a short 475 yr return period.
6) The SED 2017 report appears to “predict” adjustment factors ranging from 1.1 to 1.2. However, Figure A2 in Duverney et al. (2019) shows higher values. Maybe other considerations were included (“median” to mean” conversion, for example?). This needs more clarification. This is a key issue. The Vs-Kappa adjustments performed by SED seems to lead to CA factors ranging from 1.1 to 1.2.
7) Independently of the CA factor, we would like to note that the final objective of the Directive C3 procedure is to define response spectra at PGA for standard rock (called a_{gd}) and a soil factor S. Using them, the response spectra can be defined (using also other parameters indicated in the Directive C3, as T_B, T_C and T_D).

The procedure to define a_{gd} is:

- Take the MEDIAN spectral acceleration from SUIhaz2015 ($S_{a0.15SED}$)
- Multiply $S_{a0.15SED}$ by C_A factor to convert the Swiss reference rock (Vs30=1100 m/sec) into “standard rock” (Vs30=800 m/sec).
- Divide the previous value by 2.5 to get, in fact, a PGA on “standard rock” (2.5 is the ratio between the *plateau*, where 0.15 sec is situated and the PGA)

$$a_{gd,act} = S_{a0,15SED} \cdot c_A$$

$$a_{gd} = \frac{a_{gd,act}}{2.5}$$

The C_A factor is defined using 0.15sec. However, we note again the final objective is to define the PGA on rock (a_{gd}). It is unclear why an adjustment factor based on PGA is not used in place of using 0.15 sec and then dividing by 2.5. This introduces additional uncertainty since there is uncertainty in the value of 2.5 (ratio of the spectral peak to PGA).

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6 APPENDIX 2: RATIOS BETWEEN PGA FOR 10000 y-RP and 475 y-RP

κ Hazard Curve Slopes

Site	475-yr	10,000-yr	κ	
Thermalito Forebay, CA	0.133	0.446	3.4	ACR
Avista Long Lake, WA	0.041	0.26	6.3	ACR
Avista Post Falls, WA	0.032	0.17	5.3	ACR
Sites Dam, CA	0.21	0.75	3.6	ACR
Golden Gate Dam, CA	0.2	0.71	3.6	ACR
Saddle Dam 3, CA	0.19	0.69	3.6	ACR
Morena Dam, CA	0.17	0.47	2.8	ACR
Panama 4th Bridge, Panama	0.557	1.6	2.9	ACR
Lucky Friday, Idaho	0.061	0.28	4.6	ACR
Fatima Sur, Mexico	0.048	0.236	4.9	ACR
Berkeley, CA	0.63	1.4	2.2	ACR
Sierrita, AZ	0.03	0.21	7.0	ACR
Soto Norte Mine, Colombia	0.276	0.86	3.1	ACR
Soda Springs, Oregon	0.06	0.3	5	ACR
Clear Branch, Oregon	0.11	0.61	5.5	ACR
Graphite Creek, Alaska	0.17	0.7	4.1	ACR
Smith Dam, Oregon	0.21	0.9	4.3	ACR
Trail Bridge Dam, Oregon	0.23	0.9	3.9	ACR
Carmen Diversion Dam, Oregon	0.28	1	3.6	ACR
Hawaii	0.15	0.65	4.3	ACR
Mean K	-	-	4.2	ACR

Site	475-yr	10,000-yr	κ	
Matrix NC	0.02	0.105	5.3	SCR

Elba, Georgia	0.04	0.45	11.3	SCR
La Porte, Texas	0.0055	0.045	8.2	SCR
Amandelbult, South Africa	0.047	0.295	6.3	SCR
Mogalakwena, South Africa	0.05	0.309	6.2	SCR
Motolo, South Africa	0.047	0.253	5.4	SCR
Unki, South Africa	0.026	0.148	5.7	SCR
Venetia, South Africa	0.059	0.352	6.0	SCR
Mean K	-	-	6.8	SCR

Site	475-yr	10,000-yr	κ	
Mirador, Ecuador	0.586	1.57	2.7	Subduction zone
Josemaria, Argentina	0.58	1.7	2.9	Subduction zone
Curipamba, Ecuador	0.559	1.459	2.6	Subduction zone
Anchorage, Alaska	0.58	2	3.4	Subduction zone
EWEB Leaburg Forebay, Oregon	0.15	0.59	3.9	Subduction zone
EWEB Leaburg Canal, Oregon	0.145	0.53	3.7	Subduction zone
EWEB Walterville Forebay, Oregon	0.16	0.62	3.9	Subduction zone
EWEB Walterville Canal, Oregon	0.155	0.6	3.9	Subduction zone
Mean k	-	-	3.4	Subduction zone

$\kappa = \frac{10{,}000-Yr\ Return\ Period\ PGA}{475-Yr\ Return\ Period\ PGA}$

ACR Active Continental Region

SCR Stable Continental Region