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SEISMIC SAFETY ASSESSMENT BY AN ENHANCED MONTE CARLO METHOD

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Abstract. *A probabilistic seismic safety assessment is performed by means of Monte Carlo simulation. Emphasis is on the identification of response uncertainty as a result of the stochastic nature of ground motions in time domain. Especially, the relevance of low failure probabilities at moderate earthquakes is investigated. As application example an arch dam with nonlinear material properties and fluid-structure interaction is analysed. Exceedance probabilities for selected peak ground acceleration levels are obtained from a site-specific hazard curve. Stochastic ground motions are generated accordingly by a Kanai-Tajimi model. The fragility curve is calculated by performing Monte Carlo simulations. Its tail probabilities are estimated by a new extrapolation technique.*

1 INTRODUCTION

Since the characteristics of earthquakes are highly random, seismic safety assessments reasonably have to be based on stochastic methods somehow. In engineering practice, usually, major structures are designed to withstand the so-called maximum credible earthquake, which depends on the importance of the structure and is defined by probabilistic methods. However, the ultimate limit state is then calculated in a deterministic way, assuming that failure can only happen at or above the maximum credible earthquake.

In fact, the probability that a certain failure state happens in a given period of time depends on both, the occurrence probabilities of all the different earthquake intensities and the respective conditional failure probabilities. For n intensity levels the failure probability is calculated by

$$P_F = \sum_{k=1}^{n} P(F|I_k)Q(I_k) \tag{1}$$

in which $P(F|I_k)$ is the conditional failure probability given the earthquake intensity I_k and $Q(I_k)$ is the occurrence probability of the intensity.

In the present work, $P(F|I_k)$ is estimated by Monte Carlo simulation. For that purpose, the intensity levels are replaced by peak ground accelerations $a_{g,max}$. For many regions of the earth seismological services provide diagrams in which the seismic hazard is indicated as exceedance probability in (e.g.) 50 years as a function of the peak ground acceleration, $H(a_{g,max})$. Hence, it is straightforward to replace the sum in equation 1 by an integral. $Q(I_k)$ is then replaced by the probability density function of $a_{g,max}$. This corresponds to the negative derivative of the exceedance probabilities provided by seismological services:

$$h(a_{g,max}) = -\frac{dH(a_{g,max})}{da_{g,max}} \tag{2}$$

The failure probability is then calculated by

$$P_F = \int_0^{\infty} P(F|a_{g,max})h(a_{g,max})da_{g,max} \quad . \tag{3}$$

Obviously, it is referred to the same period of time as is the seismic hazard function $H(a_{g,max})$, typically to 50 years.

It is shown how much different earthquake intensities (corresponding to certain return periods) contribute to the overall failure probability. As application example nonlinear analysis of an arch dam is performed. Ground accelerations are created by the Kanai-Tajimi model and serve as stochastic excitation for the Monte Carlo simulations. The efficiency is increased by applying a new extrapolation technique, which is particularly suitable to estimate small failure probabilities more accurately without need for more simulation runs.

2 THE ARCH DAM

As application object for the reliability analysis a 220 m high arch dam is chosen. The earthquake safety of this (fictitious) structure has already been studied und documented in detail by Goldgruber in his PhD thesis [1]. The FE-model of the dam, see fig. 1, has kindly been made available to the authors and is used for this research. The focus of this work, however, is a Monte Carlo-based probabilistic analysis of the earthquake safety of structures in general, rather than a structural analysis of a certain dam. Hence, some changes are made on the dam described in [1]:

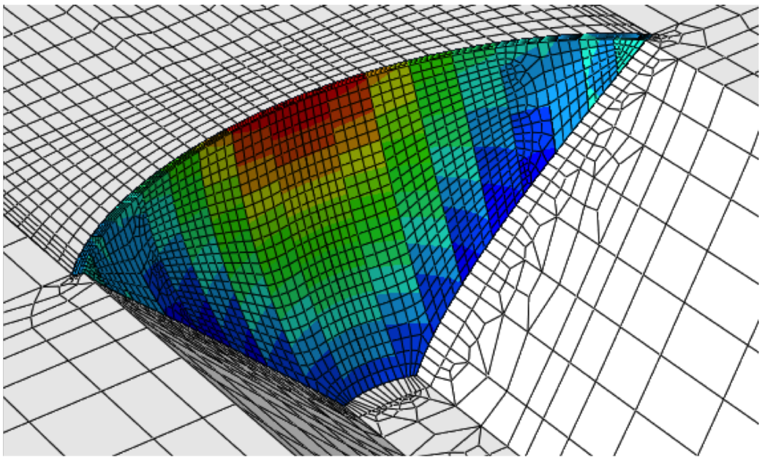


Figure 1: FE-model of the arch dam

- Crack simulation by XFEM is removed
- The block joints are removed, hence the dam behaves as a monolithic block
- Tied contact is introduced between dam and the rock foundation
- Damping of the rock mass is increased by multiplying the Rayleigh damping factors α and β by factor five.

The first three changes aim at reducing the computational effort for the Monte Carlo simulations and at guaranteeing convergence in every simulation run. The significant increase of the damping of the rock foundation is due to the necessity, in this research, to exactly excite the structure by earthquakes of certain predefined intensities. When ground motions are applied as accelerations on the model boundaries, they can be altered while travelling through the rock and possibly be intensified by reflections on the boundaries. As a result, the accelerations at the foot of the dam can significantly differ from the ones applied on the model boundaries. To avoid this, the mass density of the rock foundation is, just as in [1], chosen to be nearly zero, and, additionally in this research, a rather strong damping is implemented.

All other properties, including fluid-structure interaction with acoustic elements and nonlinear behaviour of the concrete, are equal as described in [1].

For the reliability analysis a failure criterion has to be defined. Critical zones and limiting values of quantities that lead to failure could be identified in a deterministic analysis, see [5]. Since critical values of stresses and displacements are strongly correlated with the crest displacements, the simplified assumption is made that failure (or a certain damage state) happens when the radial displacement of the crest midpoint exceeds 0.3 m. Naturally, also any other criterion, including sets of criteria, could be used.

3 SEISMIC HAZARD

It was sought to locate the dam in a region whose seismicity is well documented by a hazard curve. Hence, the city of Basle in Switzerland was chosen. Basle is the location of the largest historical earthquake on record in central Europe, with M_w 6.6. The Swiss Seismological Service recently published updated hazard curves for this city in [2]. The hazard is expressed as the probability of exceedance of $a_{g,max}$ in 50 years. In this study the hazard curve referred to as "stochastic model" in [2] is used.

The earthquake excitation is applied as acceleration boundary condition in all three directions. It is modelled as an amplitude-modulated random process

$$a(t) = e(t) \cdot b(t) \tag{4}$$

where $e(t)$ is the modulating function as given by

$$e(t) = 4 \cdot [exp(-0.25t) - exp(-0.5t)] \tag{5}$$

and $b(t)$ denotes a stationary zero-mean Gaussian random process with power spectral density according to Kanai-Tajimi:

$$S_{bb}(\omega) = S_0 \cdot \frac{4\zeta_g^2\omega_g^2\omega^2 + \omega_g^4}{(\omega_g^2 - \omega^2)^2 + 4\zeta_g^2\omega_g^2\omega^2} \quad . \tag{6}$$

The numerical values are ω_g = 17 rad/s and ζ_g = 0.3. The spectral intensity S_0 is obtained by converting the values of $a_{g,max}$ from the hazard curve in [2]. Since there is a simple relationship between these two intensity quantities, namely $a_{g,max} = const \cdot \sqrt{S_0}$, this was done by repeatedly calculating the average $a_{g,max}$ from a thousand of trials and adjusting in that way the input S_0 to the desired $a_{g,max}$. In total, twelve intensity levels are analysed. They are listed in table 1 with the corresponding probabilities of exceedance and return periods.

$a_{g,max}$ [g]	S_0 [m^2/s]	Prob. of exceedance	Return period [years]
0.105	0.00133	0.04	1225
0.124	0.00185	0.03	1642
0.155	0.00289	0.02	2475
0.185	0.00412	0.015	3308
0.230	0.00636	0.01	4975
0.260	0.00813	0.008	6225
0.280	0.00943	0.007	7118
0.300	0.01083	0.006	8308
0.365	0.01603	0.004	12475
0.500	0.03007	0.002	24975
0.670	0.05400	0.001	49975
1.000	0.12030	0.00032	156225

Table 1: Intensity levels analysed

The time interval considered is 20 s, the time step dt = 0.01 s, giving a total of 2000 time steps. The excitation in vertical direction was reduced by factor 0.7 in time domain.

In general, besides the seismic load also the other loads as well as the resistances of the structure are random by nature. However, in case of the examined arch dam the variability of the structural behaviour will be primarily due to the inherent randomness of strong ground motions. As for the other two relevant loads acting on the dam, the self-weight and the hydrostatic load, there is virtually no uncertainty at all. There is some randomness in the material properties of the concrete and quite much uncertainty about the properties of the rock foundation, however, these parameters are regarded as deterministic here.

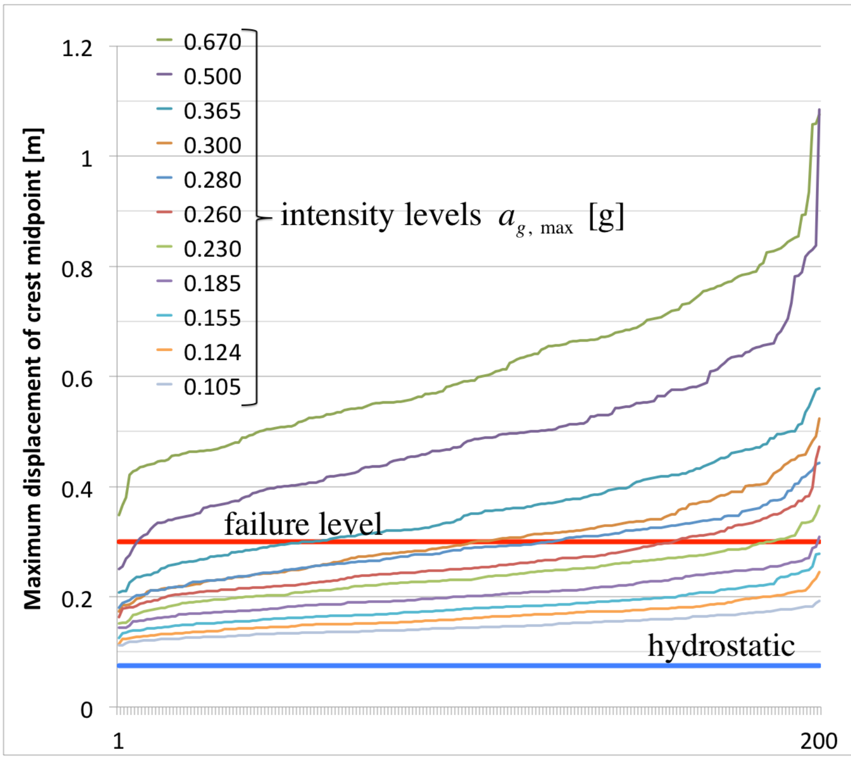


Figure 2: Maximum crest displacements

4 RESULTS

For each peak ground acceleration level listet in table 1, 200 simulation runs are performed. The maximum crest displacements during the earthquakes are shown in fig. 2 in increasing order. Besides the failure level of 0.3 m, also the displacement caused by the hydrostatic load (0.075 m) is plotted. The simulations with the three smallest earthquake intensities yield no event in which the failure level is exceeded. On the other hand, if ground motions with an average $a_{g,max}$ of 0.67 g are applied, the failure level is exceeded in each simulation run. Note that for that reason, simulations for the highest intensity level (1 g) could be avoided, since the failure level would be exceeded in each run anyway. Simulations with earthquake intensities in between lead to failure in a number of cases.

It is apparent that the variability of results based on identical ground motion intensities, but different time histories, is very high. Furthermore, it is interesting to note that the variability of the outcomes increases as the ground motion intensity increases. This is because stronger ground motions provoke more nonlinear effects.

Obviously, earthquakes with smaller $a_{g,max}$ rarely lead to failure. However, since smaller earthquakes occur more frequently, they might contribute substantially to the comprehensive failure probability according to equation 1. This means, in the present case, that 200 simulation runs might not be sufficient to estimate these small failure probabilities. Different methods exist to estimate small failure probabilities by Monte Carlo simulation at reasonable computational costs, e.g. Asymptotic Sampling [3] or Subset Simulation [4].

However, for this research, another, very simple approach is used. It is applied to the results of the crude Monte Carlo simulations for the four smallest intensities, i.e. for the simulations which didn't yield any failure event, resp. just one (i.e. for $a_{g,max}$ = 0.185 g). For that purpose, the safety margin of each simulation run is calculated, which is simply the difference between the failure level and the maximum crest displacements. This is plotted versus its cumulative distribution function (CDF) expressed as the reliability index β, see fig. 3. The reliability index β is related to the failure probability by the standardized Gaussian distribution function: $P_F = \phi(\beta)$. Apparently, the failure probabilities cannot be determined from the diagrams in fig. 3 directly, since no simulation run yielded a negative safety margin, i.e. resulted in failure (except one in diagram (d), which, thus, cannot be regarded as very representative). However, a new regression technique is applied, which approximates the CDF by a logarithmic function:

$$\beta = A \cdot ln(B \cdot \text{safety margin} + C) + D \tag{7}$$

The logarithmic approach guarantees monotonicity. The four coefficients A, B, C and D are obtained by an optimization algorithm. It is evident that the regression curve retraces the Monte Carlo results very well. The failure probabilities are then obtained by extrapolation to the value of β for which the safety margin is zero.

It can be assumed that the failure probabilities can be estimated relatively accurately with this simple technique. Considerably inaccurate estimates could potentially result from a sudden change of the structural behaviour, such as change of boundary conditions, at large displacements which did not appear in the Monte Carlo simulation. However, there is no reason to assume such a behaviour for the analysed dam in case of excitation by moderate earthquakes.

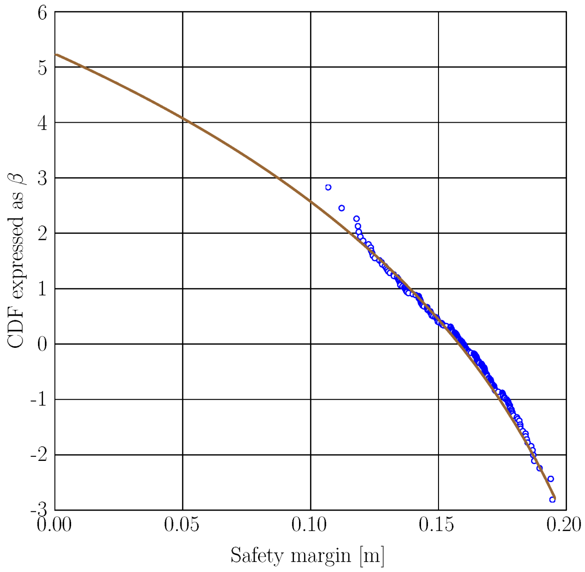
The conditional failure probabilities $P(F|a_{g,max})$ obtained by extrapolation as well as the ones obtained from crude Monte Carlo simulation are shown in fig. 4. The dots mark the evaluation points. This object-related function is called fragility curve. The diagram also shows the location-related exceedance probability in 50 years, $H(a_{g,max})$, as given in [2], and its negative derivative, the probability density function, $h(a_{g,max})$. It can be seen that ground motions with $a_{g,max}$ = 0.2 g almost never lead to failure, while the failure limit is exceeded nearly every time if ground accelerations reach up to 0.5 g. Obviously, for the peak ground acceleration $a_{g,max}$, the probabilities are distributed in the opposite way.

The comprehensive failure probability P_F is calculated by the convolution integral given in equation 3. It is of high interest to which extend different sections of $a_{g,max}$ contribute to P_F. Hence, the product $P(F|a_{g,max}) \cdot h(a_{g,max})$ is shown in fig. 5. Apparently, the peak is at $a_{g,max}$ = 0.3 g. This means that the ground motions associated with a return period of 8303 years contribute for the largest part of the long term failure probability. It is also interesting to note that the probability distribution in fig. 5 is very widebanded. This means that a Monte Carlo-based reliability analysis cannot reasonably be done for any single ground motion intensity, albeit, obviously, this probability distribution depends on the particular structure and location. Another outcome is that the contribution of the four smaller ground motion intensities, for which the extrapolation technique has been applied, is very small. However, this might be different in other applications.

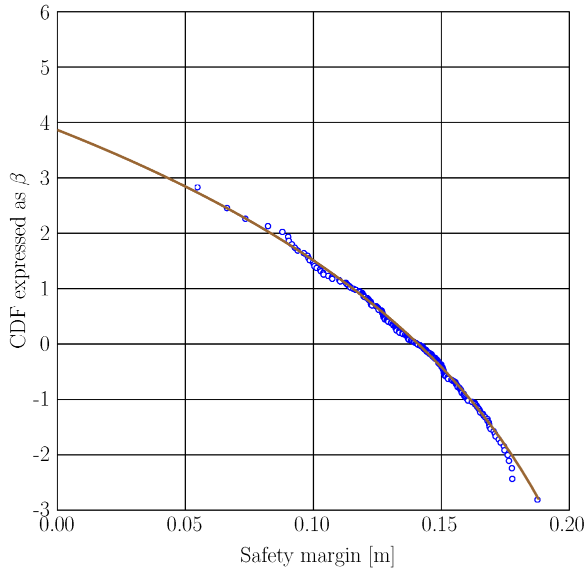
For the evaluation of the integral, only peak ground accelerations up to 1 g are considered; higher values are assumed to be geophysically impossible. The calculated failure (resp. damage) probability for a period of 50 years is 0.00609, which is equivalent to one failure in 8186 years.

5 CONCLUSION

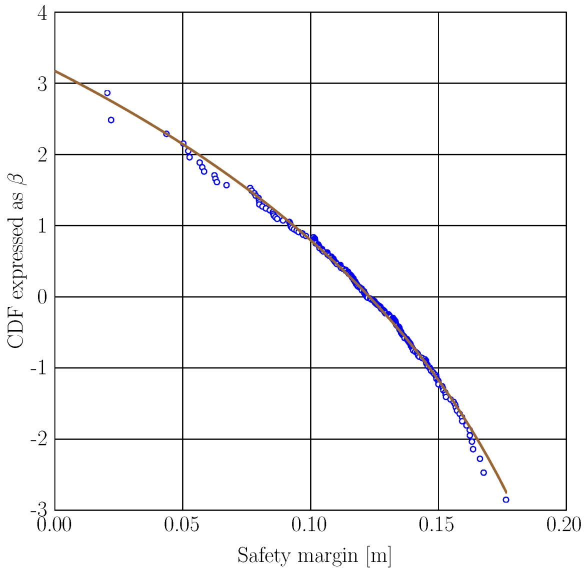
The fragility curve of an arch dam was determined by Monte Carlo simulation. It was shown that earthquakes with identical intensities but different time histories cause substantially differ-



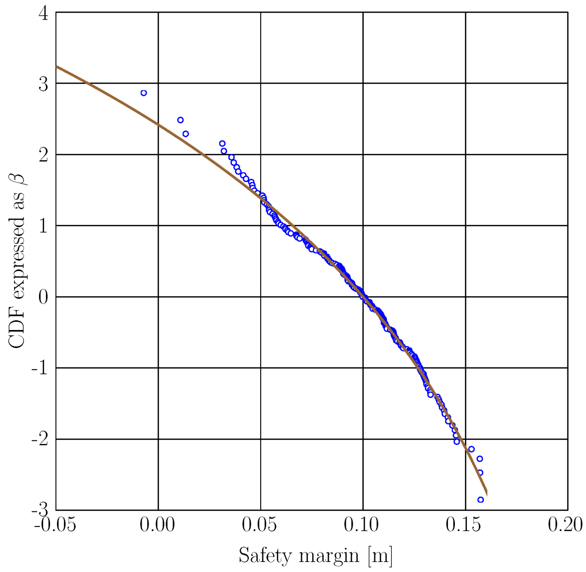
(a) $a_{g,max}$ = 0.105 g



(b) $a_{g,max}$ = 0.124 g



(c) $a_{g,max}$ = 0.155 g



(d) $a_{g,max}$ = 0.185 g

Figure 3: CDFs of safety margins with regression functions

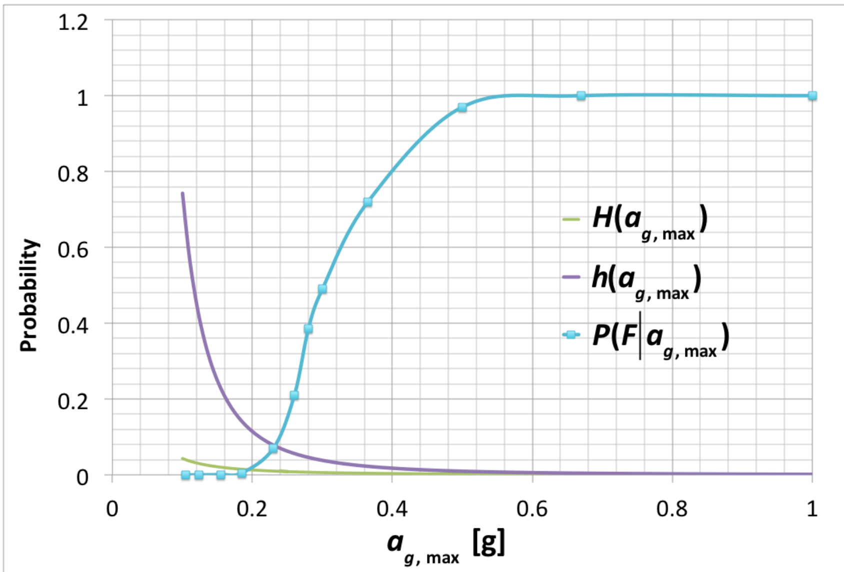


Figure 4: Fragility curve together with probability distribution and density of peak ground acceleration

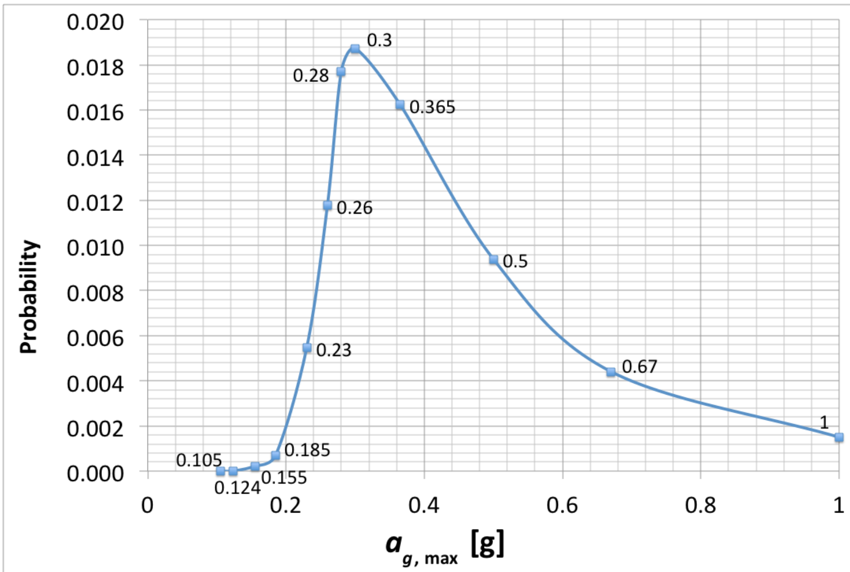


Figure 5: convolution of fragility curve and probability density of peak ground acceleration

ent structural responses. This applies especially for strong motions which lead to strong nonlinear behaviour. Hence, it can be concluded that earthquakes are the major source of uncertainty for the performance of structures in seismically active regions.

It has been shown that the range of intensities which contribute significantly to the overall seismic endangerment is wide. This means that a probabilistic seismic safety assessment has to include the whole range of intensities, i.e. return periods. The conventional method to assess the seismic safety as a result of one single acceleration time history is a completely different approach. Obviously, it is not able to reflect the large variability of the response mentioned before.

A new extrapolation technique was introduced to efficiently estimate the small failure probabilities associated with earthquakes of moderate intensities. For the examined structure it has become clear that these small failure probabilities contribute little to the overall failure probability. However, it may be assumed that this is not the case for other structures and other hazard curves. Moreover, the extrapolation technique could be effectively used to estimate failure probabilities when the Monte Carlo simulations are based on smaller sample sizes.

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