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Decision Making under Uncertainty
Developing Seismic Design Basis for Critical Infra-structures

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ABSTRACT: The planning of critical infrastructures in developed countries is a challenging task for investors accompanied by a significant amount of project risks. The development of the seismic design basis of critical infrastructures is a main contributor to these project risks and accompanied by a large amount of scientific, technical and mainly political uncertainties. A procedure is proposed how to structure the decision making process with respect to the seismic design basis from the perspective of the designer of a critical infrastructure. The procedure assures a robust and time-invariant decision-making, leading to economically acceptable design solutions and plant designs characterized by a low contribution of seismic hazard to the plant operational risk. The application of the procedure is demonstrated for the design of a new fictive nuclear power plant located at the current site of the nuclear power plant Goesgen in Switzerland. The procedure can be applied in a similar way for any other type of short-lived critical infrastructure.

1 Introduction

The planning of critical infrastructures in developed countries is a challenging task for investors accompanied by a significant amount of project risks. This is mainly caused by the low risk appetite of the society ("zero-risk" society) emerging into a large involvement of political and societal stakeholders in the decision making process. In areas exposed or presumably exposed to seismic hazards the development of the seismic design basis is a significant contributor to the uncertainty associated with the development of the design basis of critical infrastructures which bears manifold engineering and political features. Therefore the decision making on the seismic design basis of critical infrastructures has to account the "risk avoidance" behavior of modern developed societies. For this purpose a robust and an invariant with time seismic design basis has to be developed, which assures a low contribution of seismic hazard to the risk of plant operations. On the other hand the development of the seismic design basis as implemented for example for nuclear power plants (NRC RG 1.165, NRC RG 1.208) can be very time consuming increasing the political and financial risks for investors. While in the USA a part of the associated risks for nuclear power plants is covered by governmental guarantees this is not the case in most other developed countries. Therefore, an approach has to be developed which allows an effective decision making by investors meeting at the same time the risk avoidance concerns of societal stakeholders. Therefore, the procedure for the development of the seismic design basis shall be commensurate to the time schedule of the decision making process. Such a procedure for the gradual but robust development of the seismic design basis is presented in section 2 of the paper. The application of the procedure is demonstrated for the design of a new fictive nuclear power plant located at the current site of the nuclear power plant Goesgen in Switzerland including a preliminary assessment of the contribution of seismic hazard to the future operational risk of the nuclear power plant in the following sections

2 Procedure for the development of the seismic design basis for critical infrastructures

Figure 1 shows in the form of a mind map the key elements of the seismic design procedure as they are embedded into the decision making process for selecting a site and deciding on the seismic design basis for a critical infrastructure.

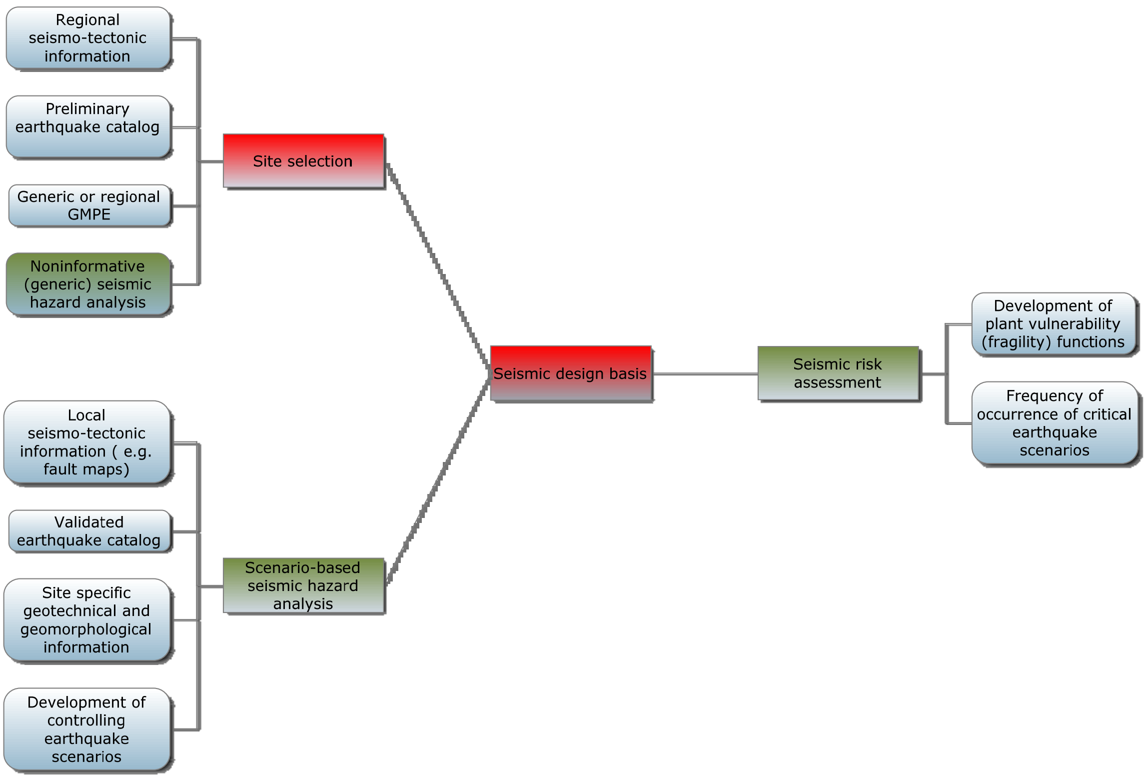

Figure 1 Mind map illustrating the key elements of the seismic design procedure

In Figure 1 steps in the management decision process are highlighted in red (e.g. site selection and the decision on the seismic design basis), while the key steps of the seismic design procedure are highlighted in green. Supporting steps (required information) are shown in light blue color.

According to the procedure the first step consists in the development of a non-informative (generic) seismic hazard for candidate sites suitable for the construction of the planned critical infrastructure. This more generic (or regional) seismic hazard analysis is based on

- a preliminary earthquake catalog,
- a regional seismo-tectonic model including f. e. fault maps on a larger regional scale
- a global geological model of the region.
- readily available or generic regional ground motion models and magnitude-fault length scaling relationships

Performing a generic seismic hazard analysis for several candidate sites includes:

- selecting the target parameter of the analysis to characterize ground motion intensity
- developing an enveloping response spectrum for the target parameter

Different seismological or engineering characteristics or combinations whereof may serve as target parameters. The selection depends on the engineering design objectives (e.g. whether deviation from linear-elastic response of a structure r component is acceptable or not). The use of combinations of parameters is preferable, because a single parameter (except for intensity which expresses the possible engineering effects statistically) can barely express the engineering effects of seismically induced ground motions. Examples of parameter combinations are:

- Elastic (pseudo) spectral acceleration response spectra and strong motion duration;
- Intensity and associated magnitude and distance dependent spectral shapes of spectral acceleration response spectra and the associated strong motion durations;
- Average spectral accelerations or maximum spectral accelerations in the frequency range of engineering interest (e.g. for structures of a nuclear power plant between 2 and 8 Hz)
- Elastic (pseudo) spectral acceleration response spectra and cumulative absolute velocity (CAV);

For a generic seismic hazard analysis it is sufficient to develop an enveloping (pseudo) spectral acceleration response spectrum and to provide an assessment of the maximum strong motion duration of the underlying controlling earthquakes (for elastic design of structures and components this is not even required). This is sufficient for robust decision making. Figure 2 shows a flow chart with the key working steps for the development of a preliminary seismic design basis by the help of a preliminary non-informative seismic hazard analysis. To derive such an enveloping spectrum three hazard input components have to be processed and evaluated:

- Historical (and if available instrumentally) recorded earthquakes (from the preliminary catalog) have to be processed into response spectra by the help of a generic or a regional ground motion prediction equation (GMPE); an envelope of all obtained response spectra has to be derived;
- The available fault maps have to be processed into fault characteristic response spectra by defining for each fault a single controlling earthquake characterized by maximum credible magnitude and the shortest distance from fault to site; an envelope of all obtained response spectra has to be derived;
- For the near-site surroundings the existence of a hidden undectable active fault has to be assumed. A controlling event for this fault has to be defined based on the resolution limits of the site investigation program and the quality of historical information available. In case of high quality long term historical information (and presuming that the site of interest is not directly located in the area of largest historical earthquake event) it is sufficient to assume a controlling event with a magnitude corresponding to the maximum magnitude observed in the same seismotectonic province reduced by the error of magnitude estimates (1.5σ= 0.5 magnitude units). A minimum value of magnitude 5.5 is suggested in case of insufficient historical information and insufficient information from site specific investigations. The distance to site has to be assumed as half of the corresponding fault length projected to the surface.
- The final step consists in the development of an envelope of all obtained response spectra and the incorporation of uncertainty. For this purpose it is suggested to perform a parametric sensitivity study on the effect of using alternate empirical ground motion prediction equations suitable for the region to define possible epistemic uncertainty. The final preliminary design basis spectrum is then defined as the envelope of the response spectra multiplied by the factor $F = \exp\left({\sigma_c}^2 \big/ 2 \right)$ where σ_c is calculated as the Gaussian error law combination of epistemic uncertainty and aleatory variability:

$$\sigma_c = \sqrt{\sigma^2{}_{epi} + \sigma^2{}_{aleatory}} \qquad \textbf{(1)}$$

The resulting factor F should be in the range of 1.3-1.4 as long as a set of empirical ground motion prediction equations are used that are suitable for the region.

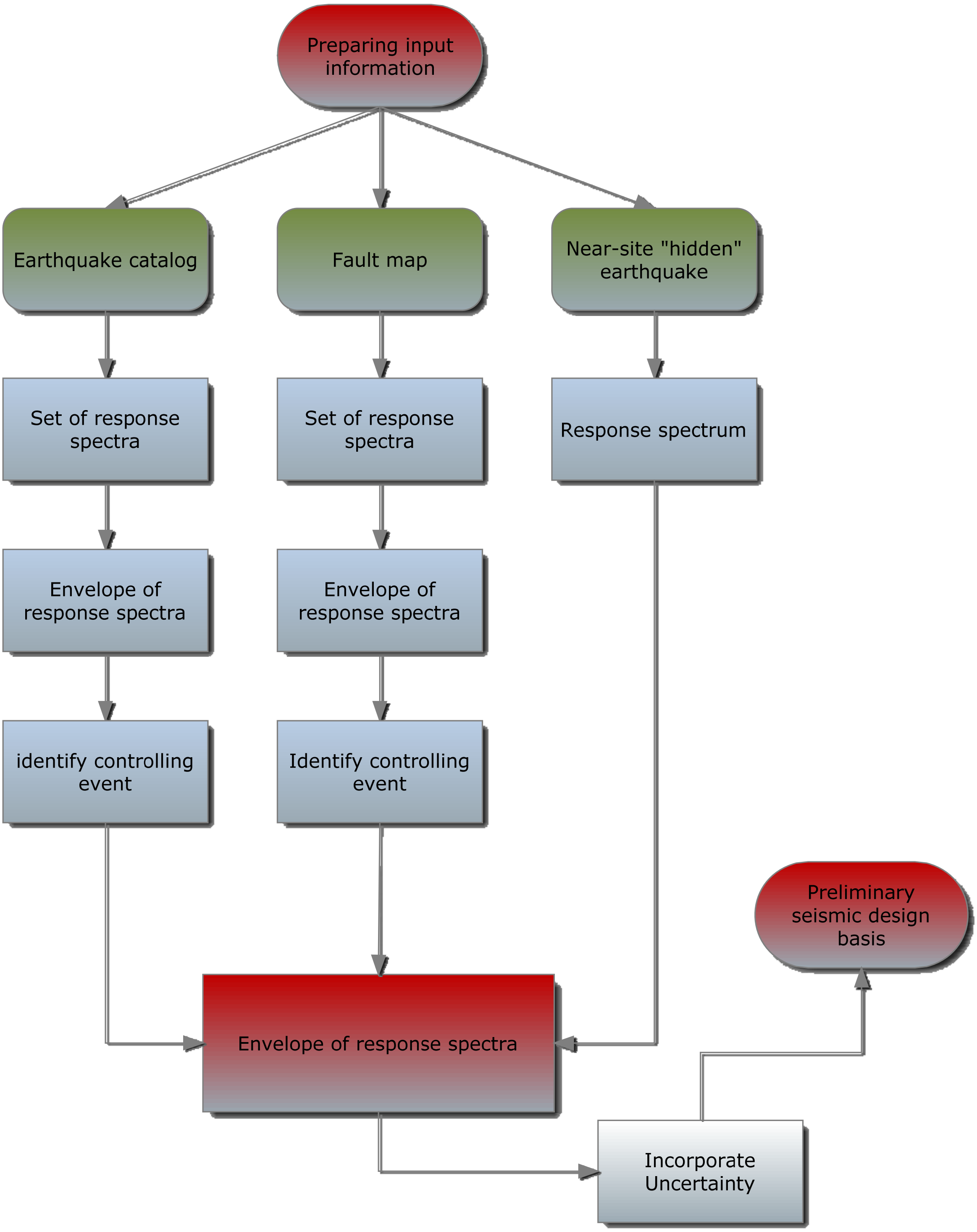

Figure 2 Flow chart illustrating the working steps of the generic seismic hazard analysis

The maximum strong motion duration has to be assessed based on the controlling events derived from each component of seismic hazard input information using the maximum strong motion duration from each of the single controlling events.

Based on the results of the preliminary seismic hazard analysis (and of course after checking the suitability of candidate sites based on other decision criteria) the site for the construction of the critical infrastructure can be selected. A preliminary decision on “suitable plant types” (e.g. for nuclear power plants the principal reactor design) can be made on this information, too. This first step of the procedure of the development of the seismic design basis is a valuable input for the technical bid specifications used by the investor in the submission process. For example it can become evident from the generic seismic hazard analysis that a base-isolated design of the infrastructure will become necessary. This might be the case for sites located in regions with significant seismic exposure (e.g. Japan, California).

The second step consists in performing a scenario-based seismic hazard analysis based on gathering the site-specific information required to perform such an analysis. This means that the (usually) available regional information has to be refined to include local characteristics. It may be necessary to refine the preliminary earthquake catalog to get a better assessment of historical seismicity. A key topic is the gathering of local information on faults in the surroundings of the site and the assessment of their seismogenic potential. The earthquake scenarios used for the refined scenario-based seismic hazard analysis are preferably developed from this gathered information (Klügel et al, 2006) using the information from the preliminary seismic hazard analysis as an input. Typically it is expected that the near-site hazard contribution can be reduced in comparison to the preliminary seismic hazard analysis by obtaining a more detailed fault mapping from geologists.

In some countries where the performance of a traditional probabilistic seismic hazard analysis (PSHA) is mandatory due to governmental regulations the “hybrid approach” may be acceptable. In the “hybrid approach” scenarios are developed from the deaggregation of PSHA results (uniform hazard spectra) into scenario-earthquakes for a predefined hazard exceedance frequency. If this approach is applied it is highly recommended to compare the deaggregation results with existing fault maps and to discuss the need of relocation of scenario-earthquakes to recognized fault systems. This is necessary because the deaggregation results of a PSHA simply represent mathematical artifacts which are very sensitive to changes in the assumptions and mathematical models used in the PSHA study.

A characteristic feature of the refined scenario-based seismic hazard analysis consists in the replacement of empirical ground motion prediction equations by waveform modeling techniques. These techniques are applied to obtain a set of source and site compatible ground motion time histories as required for nonlinear structural dynamics to support the final design of the critical infrastructure.

The third step consists in the preliminary seismic risk assessment for the plant (or for the candidate plants of different types under investigation). This step represents a joint effort of seismologists and engineers. Engineers have to provide (preliminary) vulnerability (fragility) functions for their designs. It is also possible to define requirements for the “tolerable” target vulnerability of the new infrastructure that has to be achieved by engineers to meet risk design goals. As an input for these requirements the frequency of the controlling earthquake scenarios has to be developed by seismologists. Note that traditional PSHA does not supply this information in a form suitable for risk analysis (Klügel, 2005, 2009). Hazard exceedance frequencies obtained for uniform hazard spectra cannot be equaled to frequencies of individual earthquake occurrences.

3 Application of the procedure: development of the seismic design basis for a nuclear power plant

The application of the procedure is s demonstrated for the development of the seismic design basis of a fictive new nuclear power plant located at the site of the current nuclear power plant in Gösgen in Switzerland. Due to the large amount of different types of seismic hazard studies performed in the past in Switzerland a large amount of information is available even at the beginning of the site evaluation process. The main source of input information for the first component of the generic seismic hazard analysis is the site specific earthquake catalogue of Goesgen which is based on a comparison of the Swiss ECOS 2002 catalogue (Braunmiller et al, 2005) used for the PEGASOS project (Abrahamson et al, 2004) with other published catalogues especially the Grünthal and Wahlström Catalogue (Grünthal and Wahlström, 2003). For the second component of the generic seismic hazard analysis a detailed regional geological fault map of Switzerland is available from Swisstopo (swisstopo 2005) and a detailed local fault map for the surroundings is available from Nagra (Nagra, 2008). The detailed local fault map allows making a direct estimate of the near-site seismic hazard contribution without the need of a later refinement during the detailed analysis.

The empirical ground motion prediction equations of Ambraseys et al (2005) were established as the generic empirical ground motion prediction model most suitable for Switzerland due to their broad European database. The equations for stiff soil were applied because the average shear wave velocity at the Goesgen site is between 420 and 520 m/s.

3.1 Evaluation of historical and recorded earthquake events

A detailed analysis of all earthquakes present in the Goesgen specific earthquake catalog did show that a single historical event controls the seismic hazard: the Basel earthquake event from October 18th, 1356. All other earthquakes in the catalog were found to be enveloped by the response spectrum of the Basel earthquake. This is illustrated by table 1 and figures 3 and 4.

Table 1 Earthquakes with magnitude larger 5 within 100 km distance oft he Goesgen site

YEAR	LOCATION	MW_CATALOG, GOESGEN	DISTANCE, KM
250	Kaiseraugst (Augusta Raurica)	6	25.05
1721	Aesch	5	30.01
1356	Basel	6.6	30.01
1356	Basel	5.4	34.42
1650	Basel	5.3	38.79
1777	Sarnen	5.1	57.87
1601	Unterwalden	5.9	57.89
1964	Sarnen	5.3	61.55
1774	Altdorf	5.7	78.40
1729	Frutigen	5.2	85.78

Table 1 shows all earthquakes exceeding magnitude 5 located within 100 km around the Goesgen site

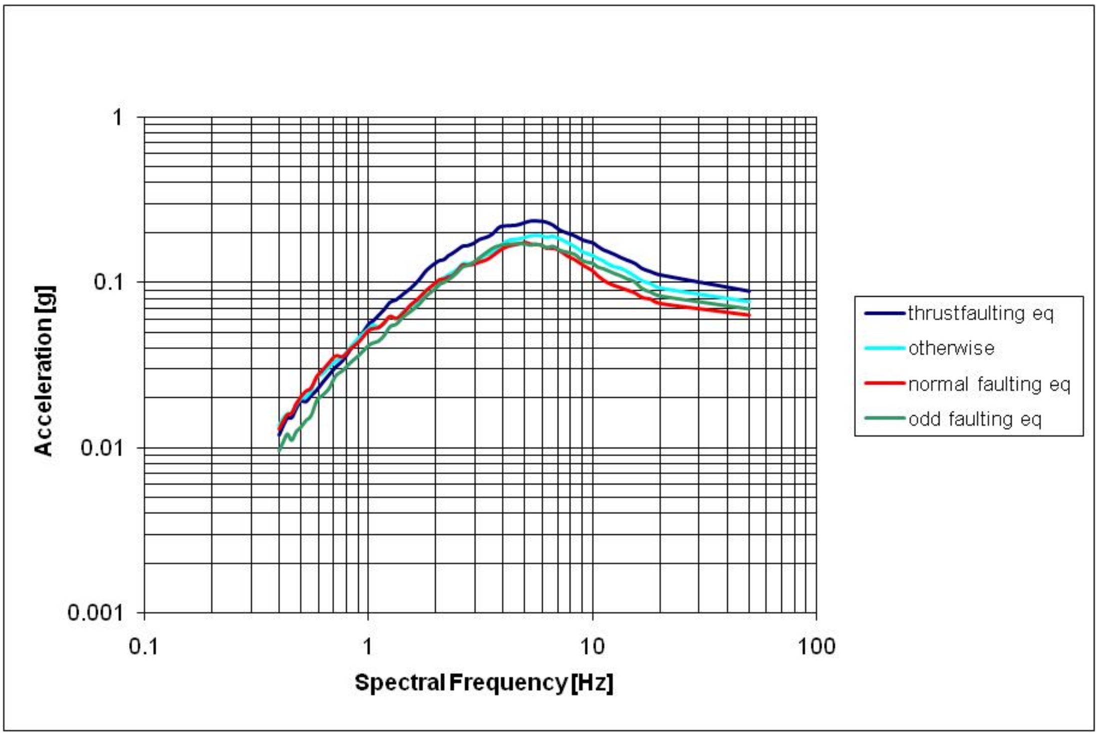

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Figure 3 Site specific response spectra of the Kaiseraugst (250) earthquake at the Goesgen site

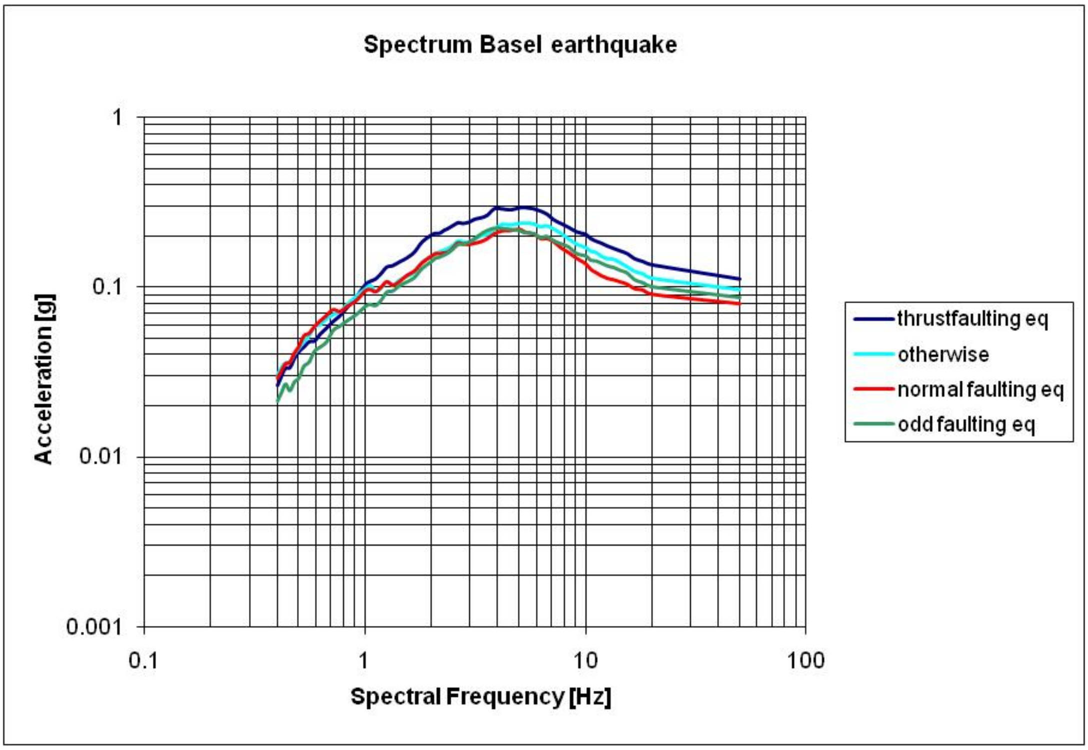

Figure 4 Site specific response spectra of the Basel (1356) earthquake at the Goesgen site

Figures 3 and 4 allow for comparing the response spectra of the largest historical events: the historical Kaiseraugst earthquake reported from Roman times and the Basel earthquake. Because the source mechanism is not quite clear all different faulting styles considered by Ambraseys et al (2005) are plotted. Not unexpectedly thrust faulting results in the highest response spectrum. Based on the analysis the Basel earthquake can be regarded as the controlling earthquake for the Goesgen site for all recorded or reported earthquakes in the catalog. At the same time the spectrum of the Basel earthquake represents the enveloping response spectrum for historical events. For a magnitude 6.6 event a strong motion duration of 14s (root mean square duration) is a reasonable estimate. The peak ground acceleration (best estimate) at the Goesgen site for the Basel event is 0.112 g. This is almost exactly the value suggested for the seismic design basis of the current Goesgen nuclear power plant in the early seventies of the past century. At that time the value was suggested for rock conditions, while the Ambraseys' et al (2005) equations were applied for stiff soil conditions. According to the suggested procedure for the incorporation of uncertainties the response spectrum in figure 4 would have been moved up by a factor of 1.3 to 1.4 to form the seismic design basis (PGA=0.15g). For the existing Goesgen nuclear power plant located at the proposed site for the new installation the design basis spectrum (safe shutdown earthquake) was originally anchored at a PGA of 0.255g on the soil surface. As a result of the PEGASOS project and the corresponding regulatory requirements the design basis was modified and the response spectrum is now anchored at a PGA value of 0.375g.

3.2 Evaluation of fault map information

In general it is difficult to identify larger *active* faults in the Swiss geological environment. Different geological hypotheses are in use in the technically informed community. In the generic analysis it is conservatively assumed that all mapped faults have the potential to reactivate during the lifetime of the future nuclear power plant. The available Swiss fault maps were analysed and maximum credible magnitudes were assigned to each of the faults using the Wells and Coppersmith equations (Wells and Coppersmith, 1994). To incorporate possible epistemic uncertainty into the analysis the maximum credible earthquakes were assigned using the median plus 1 sigma estimate from the Wells and Coppersmith equations. Figures 5 and 6 show the resulting magnitude distribution for the local near site catalogue and the resulting distance distribution. Note that the mean value of the magnitude estimates corresponds to a magnitude 4.8, and the mean "shortest distance" to site corresponds to a distance of 12.4km.

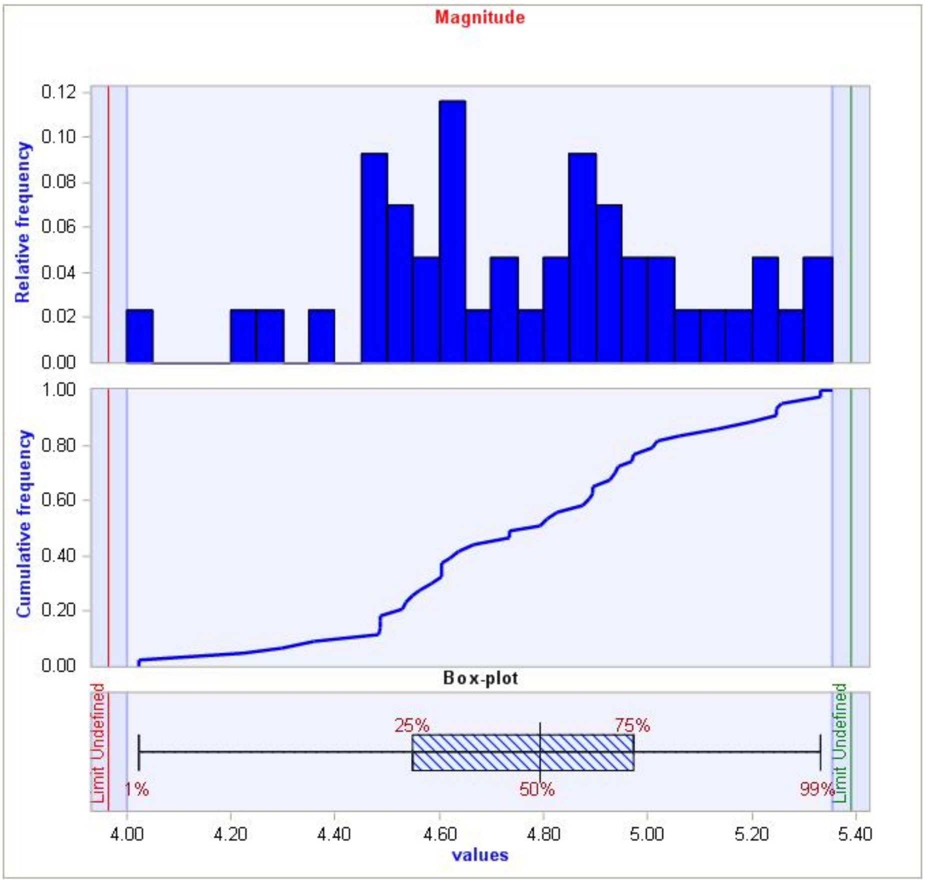

Figure 5 Estimated magnitude distribution from the local fault map (NAGRA, 2008)

Figure 6 shows the distance distribution of design relevant faults to the Goesgen site.

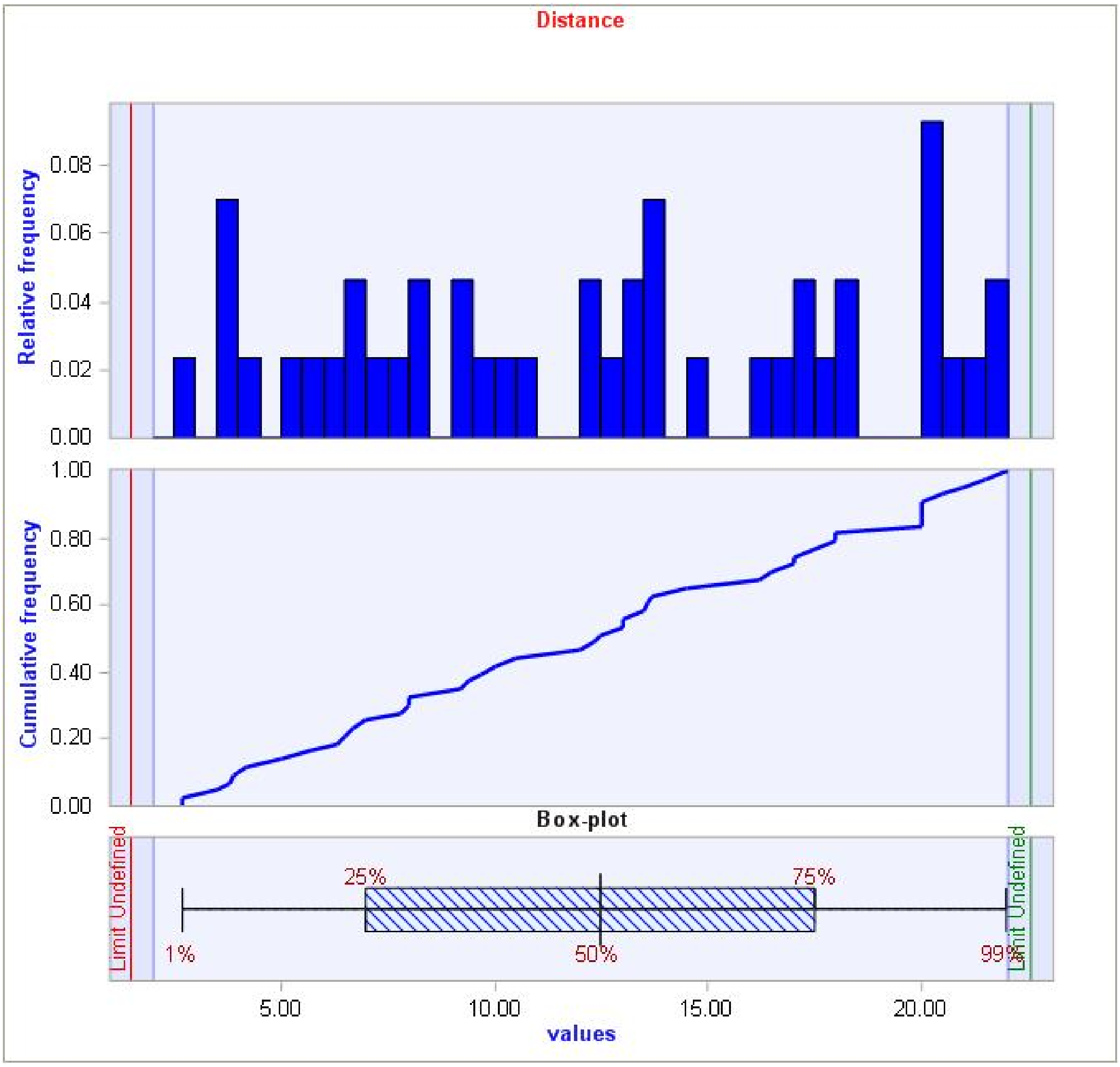

Figure 6 Estimated distance distribution from the local fault map (NAGRA, 2008)

Figure 7 shows the enveloping response spectrum derived from the evaluation of the seismic capacity of all registered faults and geological disturbances (considering them as active or assuming their reactivation during the lifetime of the infrastructure to be designed).

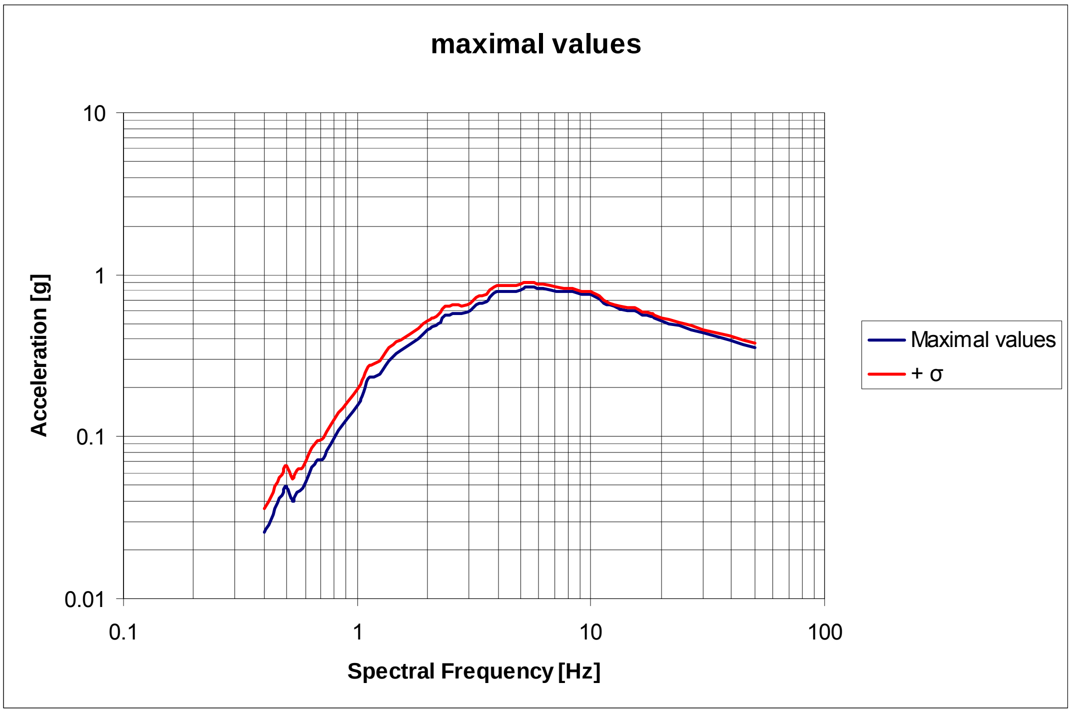

Figure 7 Enveloping response spectrum (best estimate + 1 sigma) derived from mapped geological disturbances.

The detailed analysis of the local fault map identified a possible critical near-site scenario: the “Engelberg scenario” under the assumption of a reactivitation of the corresponding fault system. The scenario is characterized by a magnitude of 5.2 (median + 1 sigma) and a shortest distance to the site of 4.2 km. The most likely fault mechanism is normal, but in the generic study a more general approach is applied. Therefore, all fault styles considered in the Ambraseys’ et al (2005) equations except for thrust (due to lack of geological evidence) faulting were included into the analysis. Figure 8 shows the response spectrum of the Engelberg scenario. The strong motion duration of the Engelberg scenario event was estimated to be around 4s.

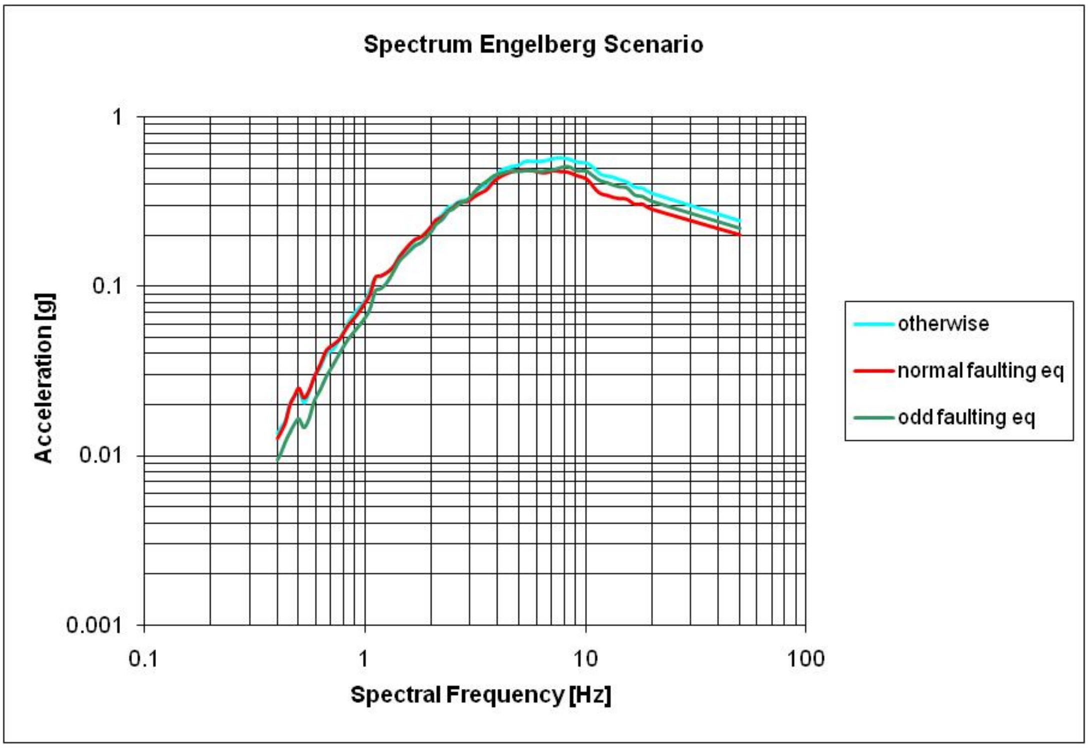

Figure 8 Site specific response spectra of the Engelberg earthquake scenario at the Goesgen site

The analysis of the regional fault map (swisstopo, 2005) did lead to an additional earthquake scenario associated with the Zeininger fault system. This fault system is currently evaluated as not active. Nevertheless in the generic seismic hazard analysis it is assumed that the fault system can be reactivated during the lifetime of the critical infrastructure. The maximum credible magnitude (median + 1 sigma) of the earthquake scenario is 5.6 at a shortest distance of 18.9 km to the site. Figure 9 shows the corresponding site specific response spectrum. Again the thrust faulting style equations were removed from the analysis because this source mechanism can be excluded from the available geological information.

Summarizing the performed generic seismic hazard analysis it can be concluded that two scenarios control the seismic hazard at the Goesgen site; the Basel earthquake scenario which leads to the largest strong motion duration and the Engelberg scenario which leads to the largest PGA value (highest spectrum) at the site.

The next step consists in the development of an enveloping response spectrum and in the incorporation of uncertainties to obtain the preliminary seismic design basis.

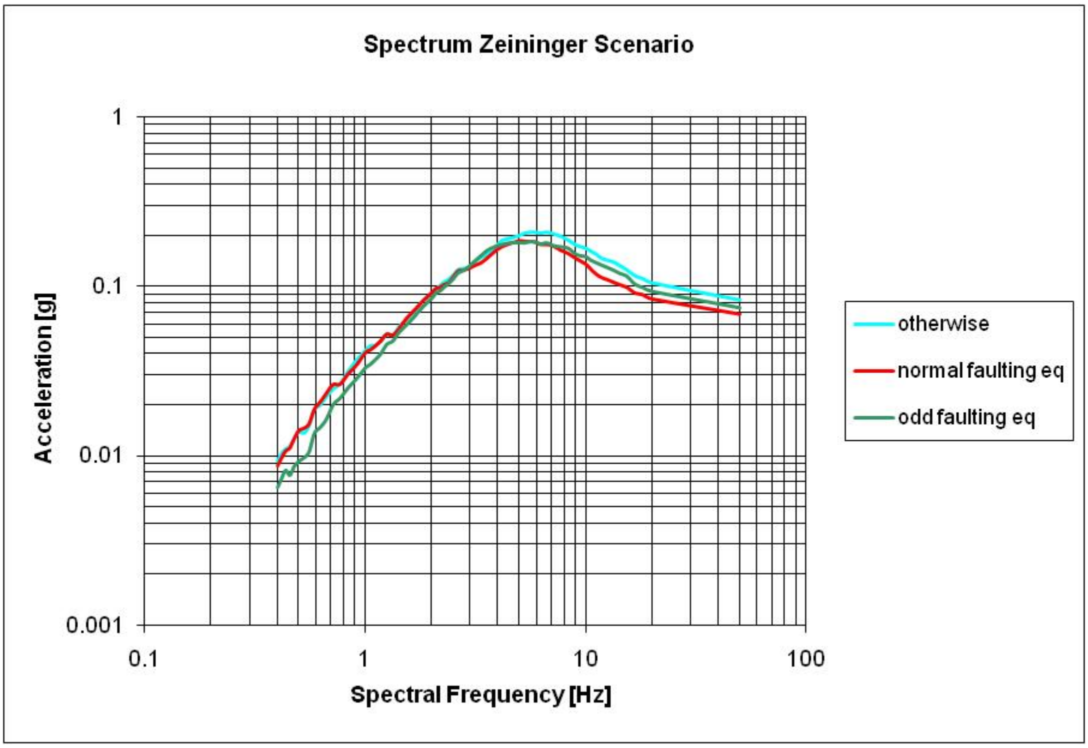

Figure 9 Site specific response spectra of the Zeininger earthquake scenario at the Goesgen site

3.3 Preliminary seismic design basis

For determining the epistemic uncertainty associated with the use of generic attenuation models a comparison between the GMPEs of Ambraseys et al (2005) with the latest version of the Akkar and Bommer (2010) equations was performed for the controlling historical event the Basel earthquake and for the controlling near site scenario – the Engelberg earthquake scenario.

Figure 10 shows the results for the Basel earthquake. A comparison with the response spectrum given in figure 4 shows that the prediction results using the Akkar and Bommer (2010) equations are enveloped by the results obtained by using the Ambraseys et al (2005) equations. The same conclusions can be made by comparing the results presented in figures 7 and 10. Therefore it can be concluded that no additional epistemic uncertainty has to be included into the analysis, because the Ambraseys et al (2005) equations represent the bounding case for European data. Therefore the combined uncertainty can be set equal to:

$$\sigma_c \approx \sigma_{aleatory} \quad \textbf{(2)}$$

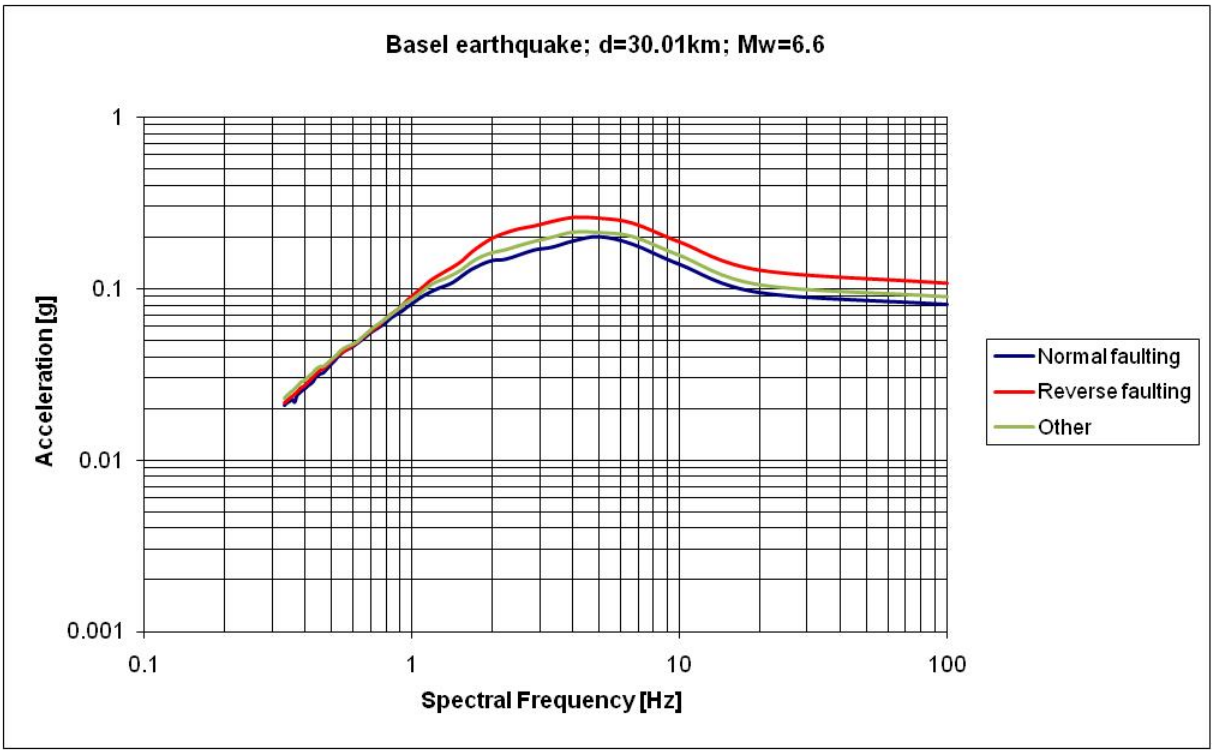

Figure 10 Site specific response spectra of the Basel earthquake (1356) computed using the Akkar and Bommer (2010) equations

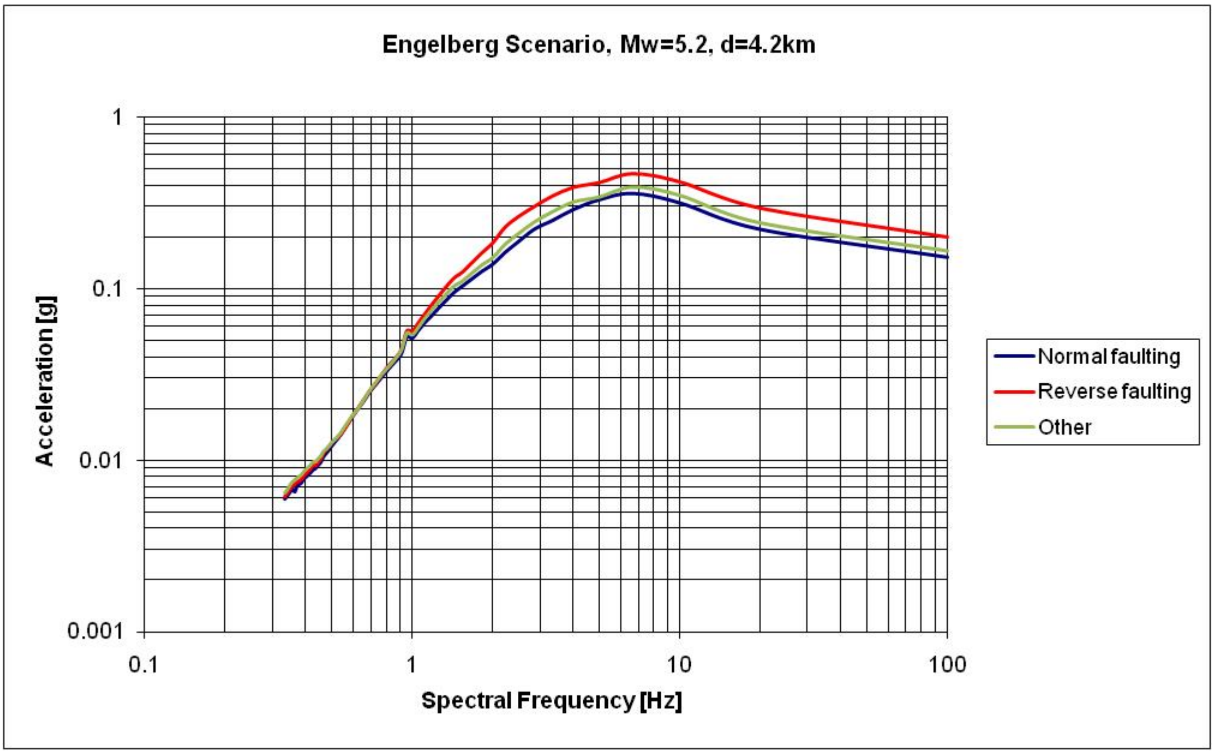

Figure 11 Site specific response spectra for the Engelberg scenario computed using the Akkar and Bommer (2010) equations

The aleatory variability for the Ambraseys equations is magnitude dependent. It is a little bit smaller than the total variability observed for the Akkar and Bommer equations (Akkar and

Bommer, 2010) in the relevant range of frequencies. In accordance to the generic, non-informative character of the preliminary seismic hazard analysis the larger value is selected.

$$\sigma_{aleatory} = 0.78$$

Therefore, the enveloping response spectrum developed from the three components of the preliminary seismic hazard analysis has to be raised by a factor of 1.36. Figure 12 shows the resulting enveloping spectra (best estimate and the mean taking into account the uncertainty).

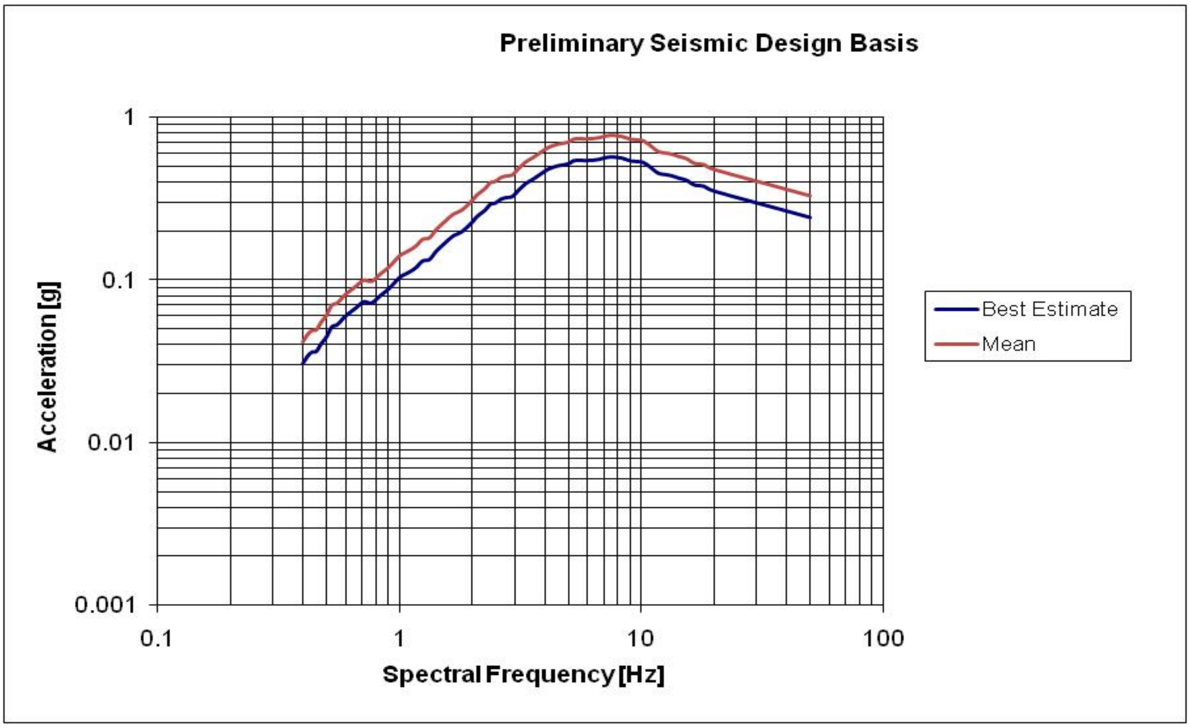

Figure 12 Preliminary seismic design basis (best estimate, mean)

The computed "mean" response spectrum is anchored at a PGA value of 0.33g, while the best estimate spectrum is anchored at 0.243g. According to the procedure the strong motion duration is set to 14s, which corresponds to the longest strong motion duration of the underlying controlling earthquake events (Basel earthquake). This preliminary seismic design basis is obviously very conservative and robust, because it considers all seismic sources of Switzerland (historical events, active and not active sources) of engineering importance. The resulting spectrum has a spectral shape which envelops the spectral shapes of the response spectra of all underlying seismic sources. The strong motion duration is set to a value which corresponds to the strongest historical event, despite the fact that this event would lead to a significantly lower response spectrum.

3.4 Refined seismic hazard analysis – final seismic design basis

The main tasks of the refined seismic hazard analysis are:

- The refinement of the input of the seismic hazard analysis by incorporating additional investigations (update of the earthquake catalogue, investigation of local site characteristics, additional geomorphological studies)
- The replacement of generic empirical ground motion prediction equations by more site and source specific models based on waveform modeling

Because in this case the preliminary seismic hazard analysis performed was already based on very detailed information including the results of local investigations the main remaining task consists in the development of a waveform model which represents the local attenuation characteristics of the region. This task was solved in Klügel et al (2009) and Klügel and Attinger (2010, accepted for publication). A stochastic source model was developed which reflects adequately the attenuation characteristics of the region as a comparison with time histories recorded at the site of Goesgen has confirmed. The model developed allows generating correlated three component (x, y, and z) time histories. The main characteristics of the model (for the larger of the two horizontal components) are summarized in Table 2.

Table 2 Parameters of the Goesgen stochastic source model (Klügel et al, 2009)

PARAMETER	**VALUE, MODEL**
Source spectrum	Brune ω-square, with equivalent circular source dimensions, source radius is magnitude dependent
Stress drop $\Delta\sigma$	Not required, explicit magnitude scaling using empirical area-magnitude scaling relations;
Geometric attenuation	Set of piecewise functions, near fault D<a, $1/(SRL+1)^2$ For D<70km, 1/D For D$\geq$70km, $1/D^{-0.71}$, near fault constraint $4/a^2$ with $a \geq 1$;
Path attenuation	$270 f^{0.5}$
Shear velocity, β [km/s]	3.5
Density, ρ [kg/m^3]	2800

PARAMETER	VALUE, MODEL
Site attenuation	$\kappa = 0.006 + 0.25\exp\left(-0.8\left(D - SRL\right)\right)$ $SRL = -3.22 + 0.69\max\left(M_w, 4.7\right)$
Site amplification	Boore et al, (1997) (equivalent linear model)

A stochastic approach was selected because this approach allows a reasonable estimate of the uncertainties of ground motion time histories avoiding the statistical mistakes resulting from pooling of empirical data as it is characteristic for many empirical ground motion equations. Table 3 gives an overview on plausibility checks performed to evaluate the validity of the model developed. A set of earthquake time histories recorded at the site was compared with the mean (= the expected value) of simulated accelerograms. Additionally, some earthquakes were included into the comparison which did not trigger the Goesgen seismic instrumentation network (0.01g set point for free field instrumentation, 0.02g for plant instruments). Considering the scatter in the simulated time histories it is explainable that these earthquakes didn't trigger a seismic registration.

Table 3 Plausibility checks for the stochastic source model – comparison with empirical data

DATE	LOCATION	DISTANCE TO GOES-GEN SITE, KM	MAGNITUDE, M_w	MEASURED, MAX X, [mg]	MEASURED, MAX Y, [mg]	MEASURED, MAX, GE-OM, [mg]	MEASURED, MAX Z, [mg]	CALCU-LATED MEAN, [mg]
22.02.2003	Rambervil-liers - St. Die	183.69	4.8	N. A	N.A.	N.A.	N.A.	11.1
23.02.2004	Besancon	187.80	4.5	N.A.	N.A.	N.A.	N.A.	7.6
12.11.2005	Mönthal (Frick)	27.93	3.6	13.51	15.76	16.85	8.5	16.7
05.12.2004	Waldkirch	80.01	4.6	11.63	15.31	17.17	4.23	14.9
21.06.2004	Liestal	31.98	3.4	7.72	9.76	11.24	5.33	10.9

Note that the site characteristics (average shear wave velocity and first natural frequency of the site) were treated as random normally distributed variates.

To reject or to approve the preliminary seismic design basis of the fictive nuclear power plant it is sufficient to simulate a set of time histories for the two driving controlling earthquake scenarios. A set of 100 time histories was generated both for the Basel earthquake scenario as well as for the Engelberg earthquake scenario. Figures 13 and 14 show the time histories corresponding to the maximal and to the minimal PGA values observed in the simulations. The mean PGA value obtained from the simulations for the Basel earthquake is approximately 0.25g. The value obtained from the simulation is significantly higher than the values obtained by the use of generic GMPEs (Ambraseys et al (2005) and Akkar and Bommer (2010)). This indicates some of the problems that arise in conjunction with the use of “imported” and “pooled” empirical ground motion prediction equations. Nevertheless, the simulation results are still well below the anchor point of the preliminary design basis spectrum.

Similar conclusions can be derived from figure 14 showing the large dispersion of possible acceleration time histories for the Engelberg scenario. The mean PGA value from the simulations is 0.297g which is lower than the preliminary seismic design basis (0.33 g) for the fictive nuclear power plant supposedly constructed at the Goesgen site.

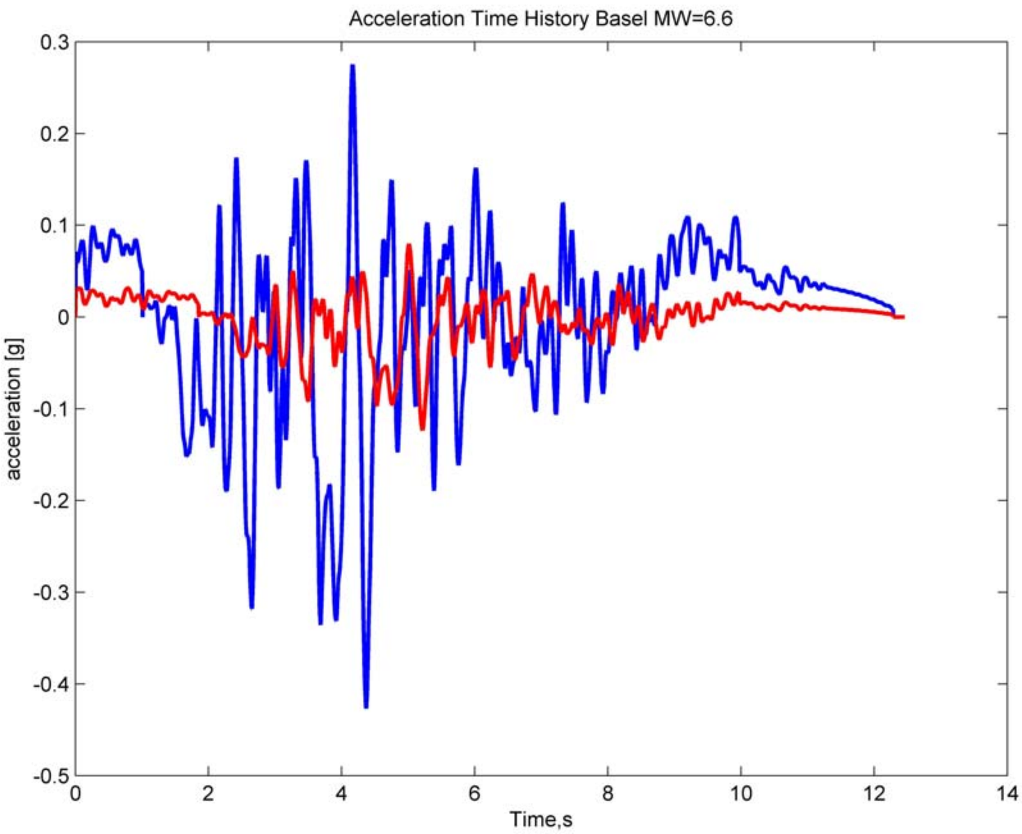

Figure 13 Simulated acceleration time histories (PGAmax (blue) and PGAmin (red)) for the Basel earthquake

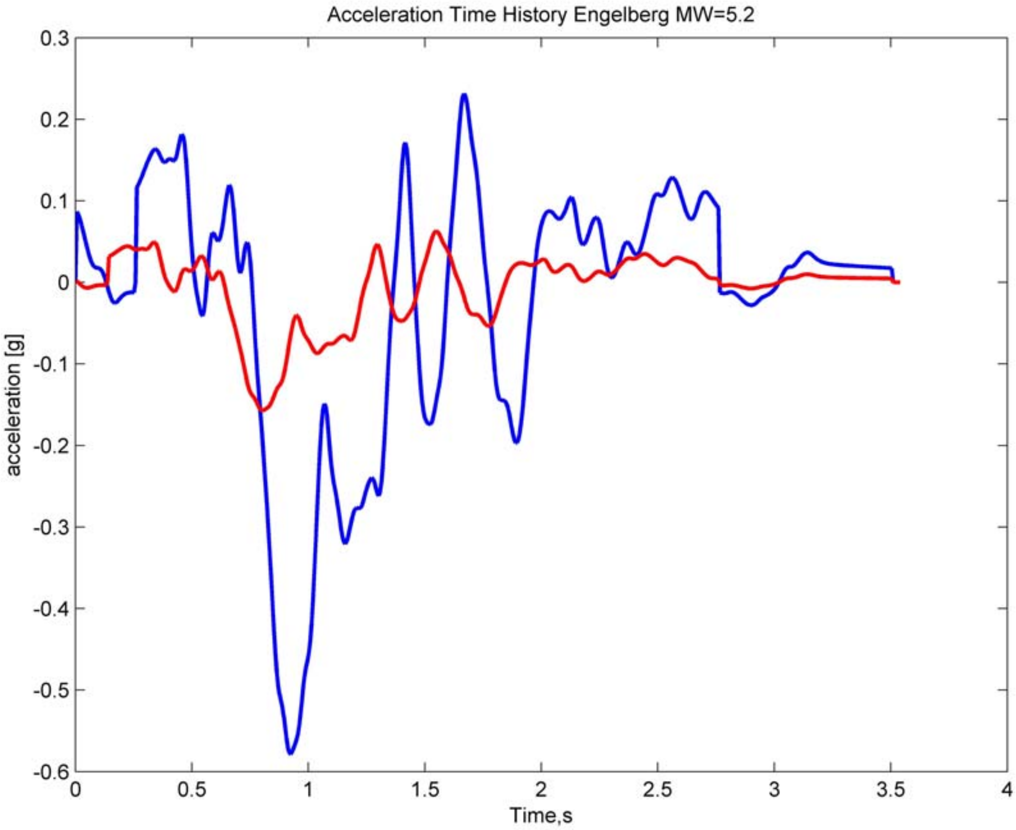

Figure 14 Simulated acceleration time histories (PGAmax (blue) and PGAmin (red)) for the Engelberg earthquake scenario

From figures 13 and 14 it can be concluded that both controlling earthquake scenarios have the potential to cause ground motion levels exceeding the enveloping response spectrum used as the preliminary seismic design basis. This is underlined by a visual inspection of the pseudospectral acceleration spectrum associated with the two extreme time histories (Figure 15). But this does not mean that the design basis would be exceeded with respect to the engineering impact. The design basis response spectrum was associated with strong motion duration of 14 s. The simulations indicate for the Engelberg scenario a strong motion duration of 3.5s (or less). Because damaging effects of earthquakes for normalized waveforms scale proportional to the square root of the strong motion duration (Klügel, 2009, Klügel et al, 2009) the preliminary seismic design basis includes an additional safety margin of a factor of two in comparison to the simulated time histories. Considering scaled response spectra the maximum response spectrum from the set of hundred time histories associated with the Engelberg scenario would still fall below the preliminary seismic design basis. A detailed analysis of the engineering consequences of possible but rare (the mean = the expected value is below the preliminary design basis) exceedances of the design basis is part of the seismic risk analysis presented in section 4.

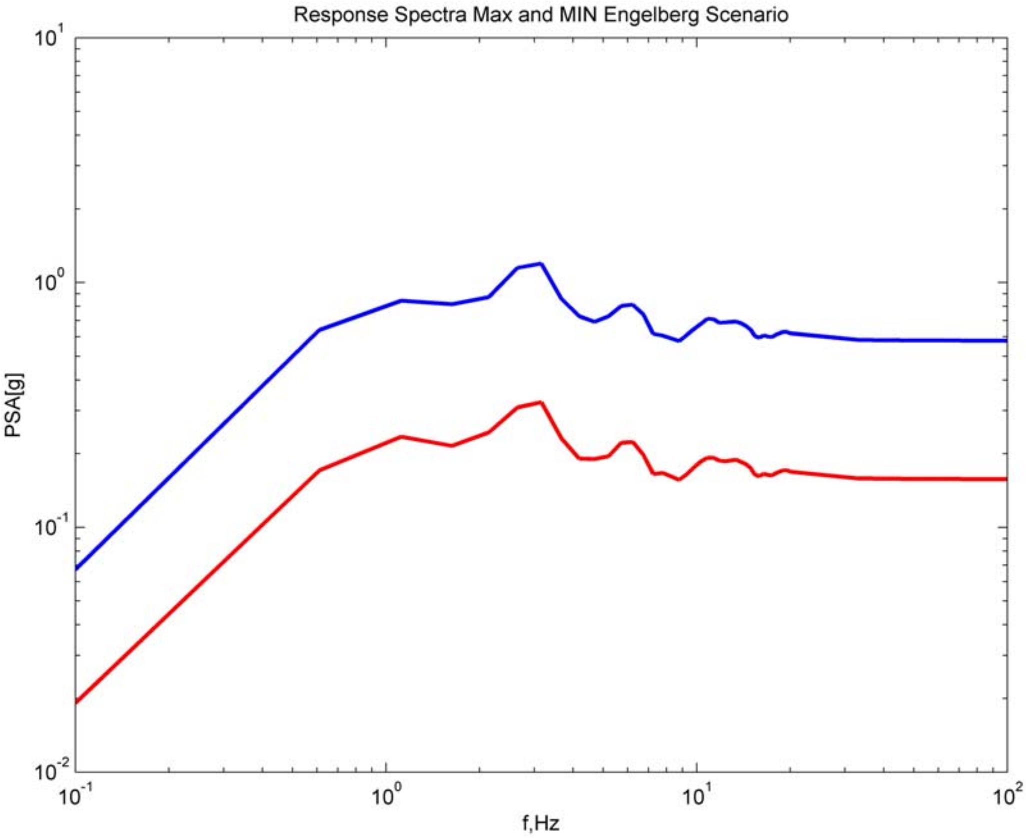

Figure 15 Pseudospectral acceleration response spectra for the two extreme time histories of the Engelberg scenario

Additionally, some statistical analysis was performed using a set of 1’000 simulated time histories for the Engelberg earthquake scenario. Using an equally weighted set of information criteria (Akaike, Schwarz and the Hannan-Quinn criteria, Vose 2008) a best parametric fit was developed comparing a set of 30 different types of distribution. The best fit appeared to be the inverse Gaussian distribution excelling the usual lognormal distribution in information performance. The fit is shown in Figure 16. Note that the assumption that site ground motions of individual earthquakes are constrained would lead to a complete rejection of the lognormal model.

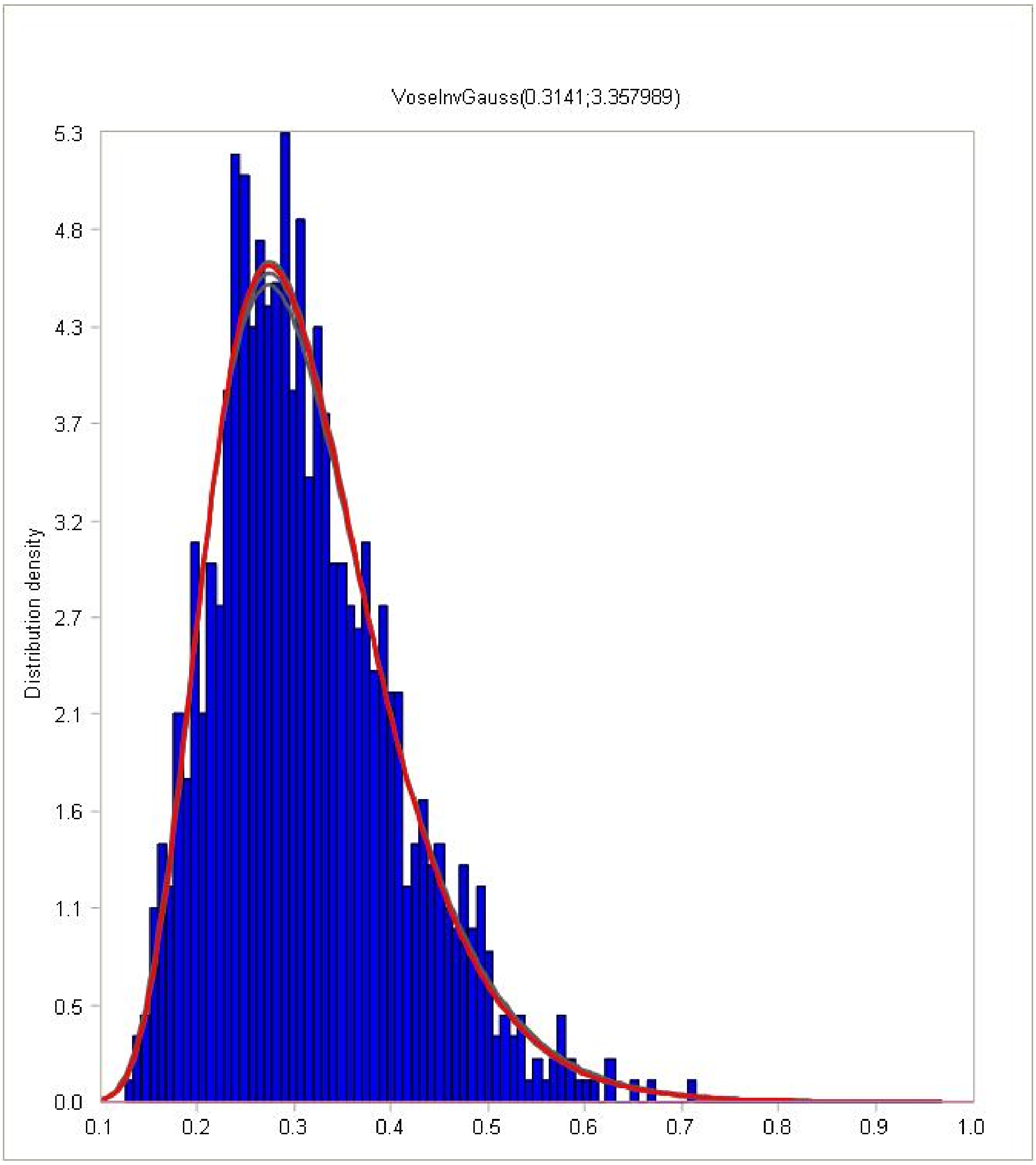

Figure 16 Statistical analysis of 1000 simulated PGA values (in g) for the Engelberg scenario

The Inverse Gaussian distribution has the following probability density function:

$$f(x;\mu,\lambda) = \sqrt{\frac{1}{2\pi x^3}} \exp\left(\frac{-\lambda (x-\mu)^2}{2\mu^2 x} \right) \quad \textbf{(3)}$$

The parameters of the best fit are μ=0.3141 and λ=3.357989.

A similar analysis (on 5000 simulations) was performed for the Basel earthquake scenario, again demonstrating that the lognormal distribution is not the best parametric fit for the simulated PGA values:

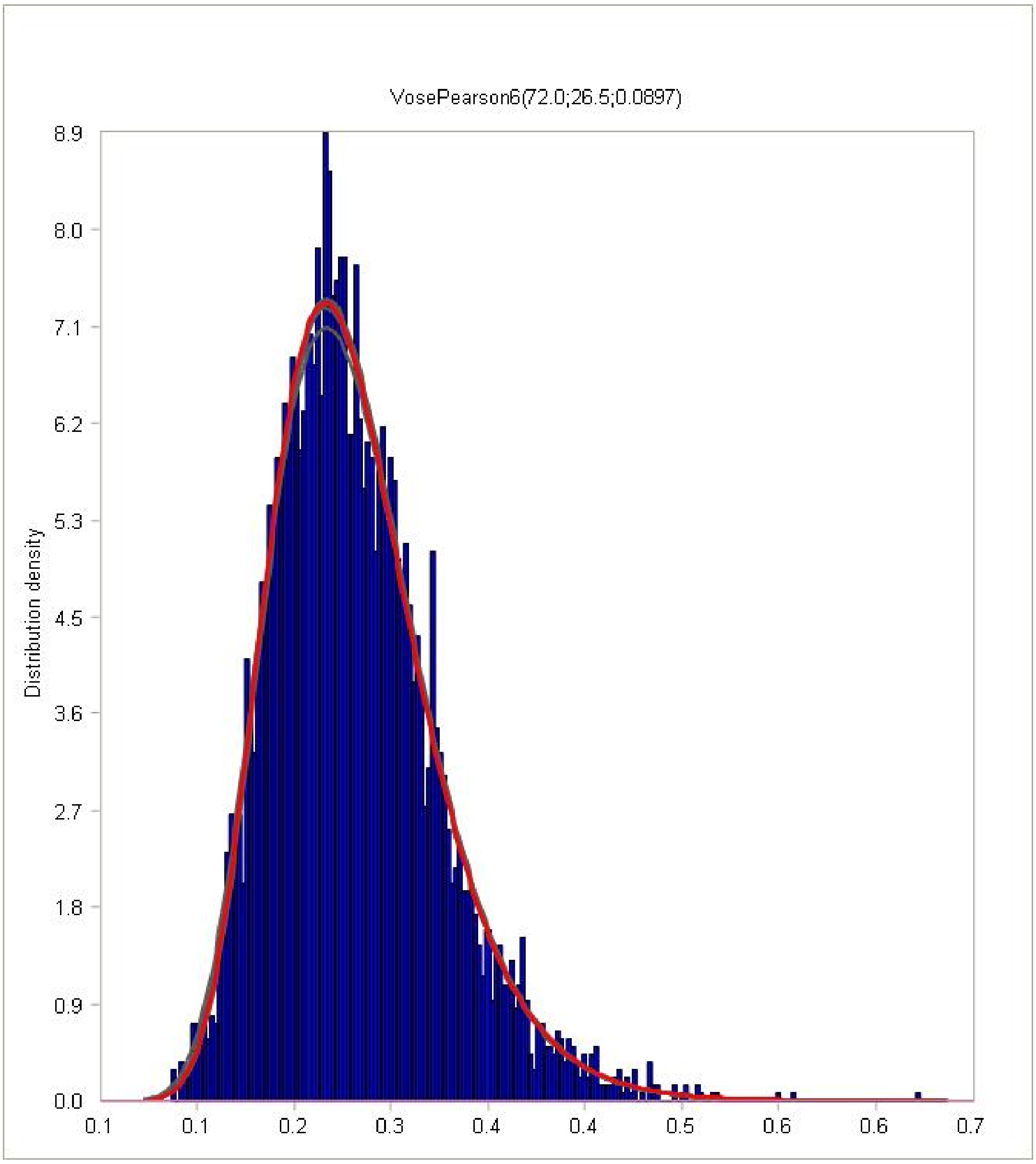

Figure 17 Statistical analysis of 5000 simulated PGA values (in g) for the Basel earthquake scenario

In this case the Pearson 6 distribution was established to be the best performing fit. This distribution belongs to the Pearson system of distributions. The Pearson system of probability distributions is defined by the requirement that for every member the probability density function f(x) satisfies the following differential equation:

$$\frac{1}{p}\frac{P}{dx} = -\frac{a+x}{c_0 + c1x + cx^2} \qquad \textbf{(4)}$$

The type 6 distribution corresponds to the case that the roots of the quadratic equation in the denominator of equation (4) are real. The probability density function of the Pearson

Type 6 distribution is defined via the Beta function with the positive distribution parameters α_1, α_2, and β.

$$f\left(x;\alpha_1,\alpha_2,\beta\right)=\frac{1}{\beta B\left(\alpha_1,\alpha_2\right)}\frac{\left(x/\beta\right)^{\alpha_1-1}}{\left(1+x/\beta\right)^{\alpha_1+\alpha_2}} \quad \textbf{(5)}$$

Generally it can be concluded, that the preliminary seismic design basis is sufficiently robust. The design basis envelopes the mean response spectra derived from simulated time histories for the controlling earthquake scenarios. Additionally, it shall not be forgotten that the faults mapped in the surroundings of the Goesgen site are evaluated by geologists as presently not active. Ignoring this assessment leads to significant conservatism of the design basis. A final answer how robust the design basis is can be given by a subsequent seismic risk analysis.

4 Seismic risk analysis

The best way of performing a seismic risk analysis consists in a direct scenario-based approach extending traditional deterministic scenario-based seismic hazard analysis methods to risk analysis (Klügel et al, 2006). Advanced risk analysis methods include the time-dependency of seismic activity removing simplifications associated with traditional seismic risk assessment methods as suggested by Cornell (1968). Unfortunately such advanced methods are not (yet) accepted by regulators. This is partially explainable due to the large sensitivity of the results of time-dependent models to changes in the modeling assumptions, although this sensitivity has a physical basis: the seismic activity in a region is not invariant with time. For the named reasons a more traditional approach to risk analysis was used (despite the manifold problems associated with this approach, Klügel (2008), Klügel 2009). The seismic risk analysis is performed based on a traditional PSHA approach. The results of the PSHA are used as an input for the probabilistic risk assessment of a generic nuclear power plant which is designed in accordance with the developed seismic design basis. To allow a comparison between different PSHA approaches two methods have been used:

(1) A non-informative PSHA approach which corresponds to current PSHA-methodology using non-informative probabilistic models for the spatial distribution of seismicity, the truncated extrapolation of the Gutenberg-Richter relationship for the magnitude-frequency relationship and a generic ground motion prediction

equation. For the later the Akkar and Bommer (2010) equations are used. Additionally the standard (although known as inadequate) model of lognormal distribution of ground motions is used to extrapolate the magnitude of ground motions to low values of hazard exceedance frequencies. The hazard was truncated at the 3 sigma level. The seismic zonation model developed by expert group EG1a during the original PEGASOS project (Abrahamson et al, 2004) was used as an input without modifications. In the analysis only the aleatory inter-event variability was considered to reduce the effects of the ergodic assumption (Klügel, 2008).

(2) An informative PSHA approach which attempts to incorporate the available geological and statistical information (distribution of the ground motion distribution tails for individual earthquakes observed at a specific site) as long as this is compatible with the traditional PSHA method. Additionally, it is considered that the critical infrastructure under investigation is a short-lived structure. This is taken into account by adjusting the characteristics of the seismic zonation model taken from the PEGASOS-project:

- by truncating the maximum magnitude distribution based on considerations from theory of records (Embrechts et al, 2003; it is very unlikely for a short-lived structure to be exposed to an earthquake with a magnitude breaking the "record" of historical observations performed during a significantly longer observation time) and the fault maps developed for the near site area;

This truncation led to an average reduction of the maximum magnitude values by 1 in the host zone of the site and by 0.5 to 0.8 for adjacent zones.

- and, by truncating the standard lognormal distribution for ground motion at a level of 1.28 sigma, which corresponds to the simulation results (from a total of 6'000 simulated time histories) obtained by using the stochastic source model;

As in the previous case only the inter-event variability was considered as relevant for the risk analysis. To apply the PSHA methodology it was necessary to develop an empirical ground motion prediction equation from synthetic data developed from simulations with the stochastic source model. A set of 795 pseudo response spectra was generated and used for the development of the model. The model equations are as follows:

$$\log(PSA) = b_1 + b_2 M + b_3 M^2 + (b_4 + b_5 M)\log\sqrt{R_{jb}^2 + b_6^2} + b_7\sqrt{R_{jb}^2 + b_6^2} \qquad \textbf{(6)}$$

The coefficients are given in Table 4.

Table 4 Coefficients for empirical ground motion prediction equation (6)

f,[Hz]	b1	b2	b3	b4	b5	b6	b7
100	1.36398354	0.40432286	-0.0596383	-3.6498402	0.33640925	20.444835	0.00363826
35	2.70562234	0.2352375	-0.0548534	-4.2420059	0.38569747	24.0445552	0.00454552
25	3.44389802	0.22058524	-0.0589992	-4.6390912	0.40681416	27.9156662	0.00517194
20	4.33231174	0.14703851	-0.0593994	-5.0620913	0.43780732	30.6418286	0.00575147
13.34	2.64977165	0.51247737	-0.0799856	-4.3640096	0.36237153	32.3542296	0.00420529
10	0.49086494	0.96729832	-0.1015626	-3.405888	0.24131381	34.2489846	0.00285855
6.67	-0.0511071	1.09366706	-0.1064819	-3.3135672	0.21643341	32.7449627	0.00389887
5	-0.8090296	1.14805333	-0.109433	-3.0608958	0.22511414	28.5850401	0.00342507
4	-1.7699494	1.14142813	-0.1042124	-2.5745769	0.22100427	20.6457281	0.00255374
2.5	-2.2572955	0.99804988	-0.0866903	-2.21077	0.22742684	11.8442726	0.00131145
2	-2.1855388	1.01596203	-0.0857795	-2.3154378	0.21942642	13.826147	0.00191829
1.334	-2.6493888	1.21575325	-0.0968272	-2.4350808	0.19040157	15.5408086	0.00336618
1	-3.8328239	1.46518616	-0.1068653	-2.0824447	0.13061632	15.3093195	0.00331016
0.667	-6.0632861	2.04720543	-0.1359375	-1.5148784	0.00620988	17.6865172	0.0039643
0.5	-7.8737128	2.47121918	-0.1585713	-1.0283136	-0.0714876	17.1899891	0.00399696

The inter-event variability was derived directly from the statistical data analysis of the time history simulations. Some dependency on magnitude and distance was found leading to σ values between 0.236 and 0.305. The lower value corresponds to higher magnitude values around magnitude 6.5. Due to the higher engineering importance of larger magnitude values the lower value of the inter-event variability was used in the case study. The computations were performed using the MATLAB based PSHA-code JKPSHA developed by the author. The code quantifies probabilistic hazard curves by using a Monte Carlo simulation approach.

Figures 18 to 21 allow a comparison of the results of the two diffferent approaches. A reasonable agreement (within 20%) of the results can be established up to hazard exceedance frequency of 10^{-4}/a. According to Swiss design practice for nuclear installations the mean hazard for the hazard exceedance frequency of 10^{-4}/a has to be selected as the (probabilistic) design basis. The spectra obtained by PSHA for this hazard frequency are in good agreement with the seismic design basis developed by the proposed procedure (anchor point at PGA=0.33g for design, PGA=0.348g from the noninformed PSHA approach and 0.397 g from the informed PSHA approach). This comparison shows that the physics based neodeterministic method despite its simplicity allows developing a robust seismic design basis.

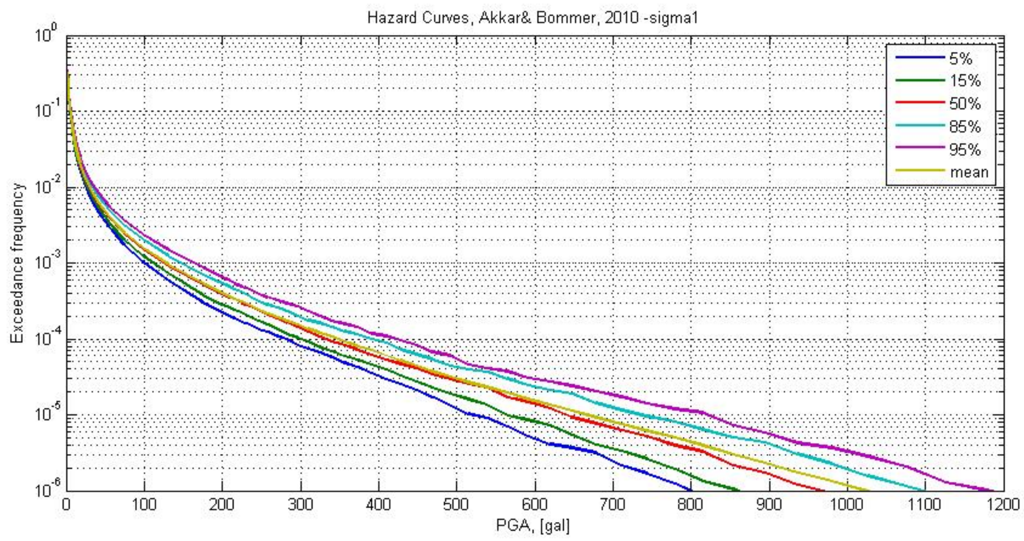

Figure 18 PSHA hazard curves – Method 1 Noninformed approach

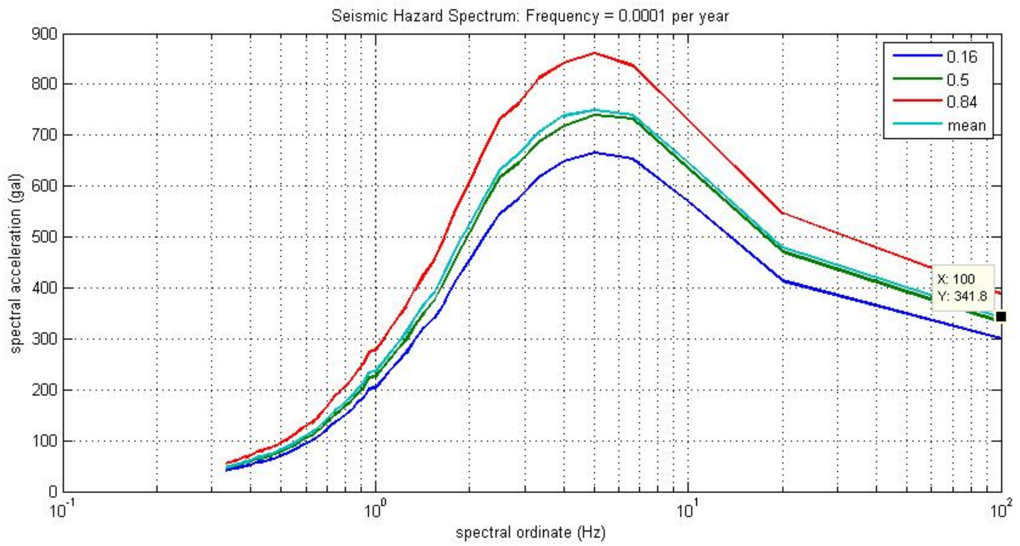

Figure 19 PSHA Uniform hazard spectrum (1E-4/a) – Method 1 Noninformed approach

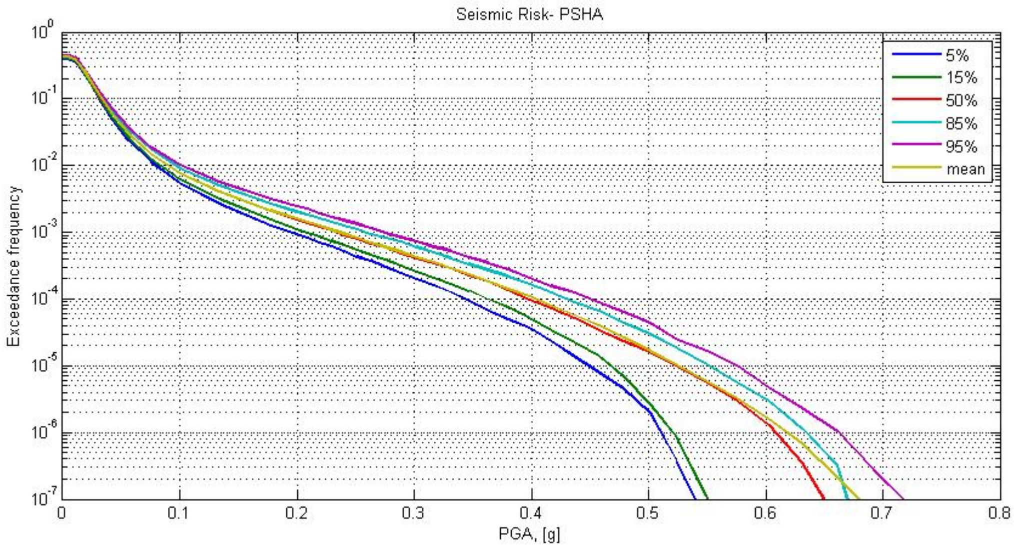

Figure 20 PSHA Hazard Curves – Method 2 Informed Approach

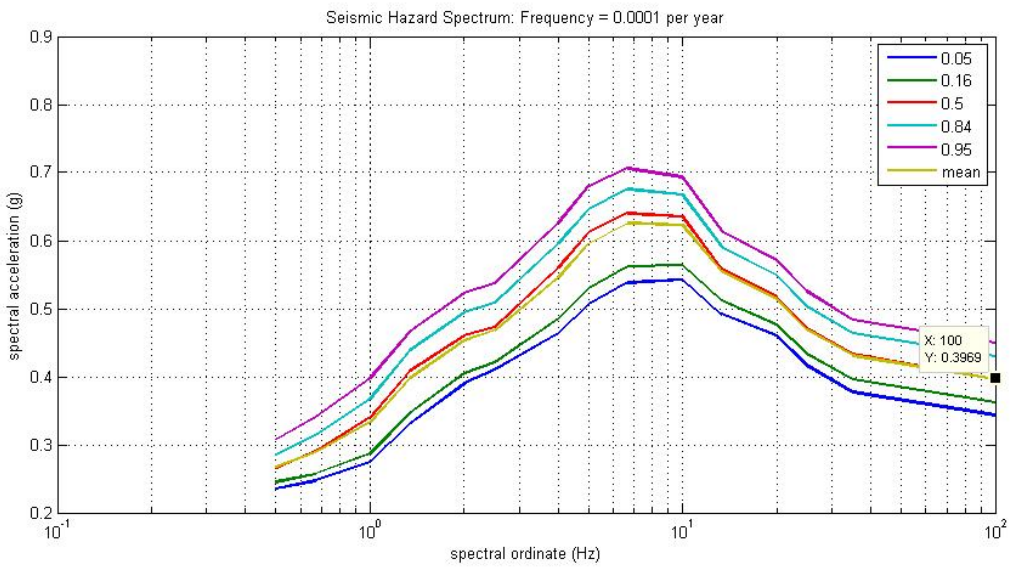

Figure 21 PSHA Uniform Hazard Spectrum – Method 2 Informed Approach

The reasonable agreement between the noninformed PSHA method and the scenario-based approach can partially be explained by the use of similar non-informative assumptions. In the design procedure all faults were considered as active and in the PSHA approach it is assumed that there exists a large amount of unidentified hidden and active faults in the relevant area. Nevertheless the near coincidence of the hazards at an exceedance frequency of 10^{-4}/a is by chance and not a systematic feature of the methodology applied (the value of exceedance frequency where coincidence is observed depends on the characteristic recurrence period of larger events). The higher results of the second PSHA approach at the exceedance frequency of 10^{-4}/a are explained by the different ground motion attenuation model which reflects the local conditions more accurately then the generic models; this is shown in figure 22.

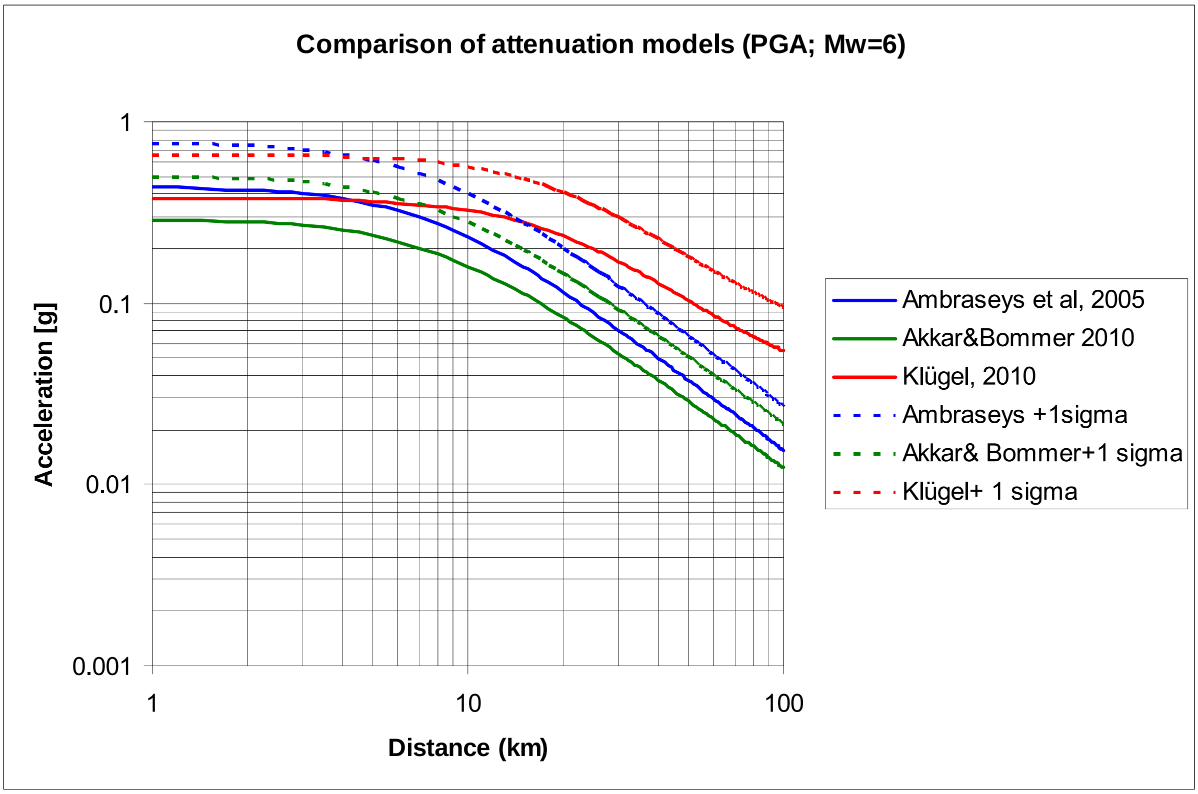

Figure 22 Comparison of attenuation models (Ambraseys et al (2005), Akkar and Bommer (2010), Klügel (this paper))

From the perspective of probabilistic risk assessment the difference between the noninformed PSHA approach (method 1) and the informed approach (method 2) is significant. The assumption of a nearly unconstrained hazard (the hazard was actually truncated at 3 σ) leads to significantly higher hazard values at low frequencies of exceedance. As mentioned, the data analysis for single earthquake events based on stochastic simulations does not support the existence of such heavy upper tails in the ground motion distribution and the physics based design procedure presented in the previous sections does not justify such high ground motion values, either. Analyses performed by authors of empirical ground motion prediction equations indicating the applicability of the unconstrained (or with an upper limit set to theoretical maximum ground motion values) lognormal distribution model (Ambraseys et al, 2005, Akkar and Bommer, 2010) are biased by data pooling effects resulting from the combination of datasets from different seismic sources and frequently even different seismotectonic settings. Their analysis results simply reproduce the effect of the central limit theorem applied in logspace.

Because a probabilistic risk assessment has to be realistic (without undue conservatism) to meet modern requirements of an risk-informed reactor oversight process the results of method 2 were applied for the probabilistic risk assessment of the planned (fictive) new nuclear power plant at the Goesgen site.

For the probabilistic risk assessment (PRA) it was assumed that the new plant will meet the seismic design developed by the procedure presented in this paper. That means that the plant will have a generic fragility function (lognormal function) with (at least) an HCLPF (High Confidence of Low Probability of Failure) value of at least 1.5 times (this relationship is known from engineering experience) the seismic design basis, which makes up 0.5g. It is assumed that the new plant will have two independent, seismically hardened paths for safe shutdown each of them with an HCLPF value of 0.5g. Here the usual fragility function approach which is based on the double logarithmetic distribution model (EPRI 1994) is applied. The reliability of plant components (to account for independent failure modes that may coincide in time with seismically induced failures) is assumed to correspond to the reliability of components of the existing Goesgen nuclear power plant.

The quantification of the probabilistic risk model was performed using RISKMAN© 12.0. The calculated seismic core damage frequency for the fictive new nuclear power plant located at the Goesgen site is 1.09×10^{-6}/a. According to current international guidelines this is an acceptable value of residual risk. Furthermore, it has to be mentioned that a more detailed probabilistic risk assessment based on deaggregated hazard scenarios (Klügel, 2009) in countries of low to moderate seismicity would lead to even lower values of core damage frequency. This demonstrates that the seismic design basis developed by the procedure presented in the paper is also robust under the perspective of probabilistic risk assessment.

5 Summary and Conclusions

A procedure was developed how to structure the decision making process with respect to the seismic design basis from the perspective of the designer of a critical infrastructure. The procedure assures a robust and time-invariant decision-making, leading to economically acceptable design solutions and plant designs characterized by a low contribution of seismic hazard to the plant operational risk. The core of the procedure consists in the systematic application of physics based deterministic and neo-deterministic analysis methods taking into account the available information on historical seismicity and on seismo-tectonic features presented by a detailed fault map. Due to the iterative character of the procedure relevant design decisions can be performed in the early stage of the planning process of a new critical infrastructure. The application of the procedure was demonstrated for the design of a fictive new nuclear power plant to be located at the current site of the nuclear power plant Goesgen in Switzerland. It was demonstrated that the seismic design approach proposed leads to a robust design solution also under the perspective of probabilistic risk assessment. The procedure can be applied in a similar way for any other type of short-lived critical infrastructure.

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