TOWARDS DAMAGE-CONSISTENT PERFORMANCE-BASED DESIGN OF CRITICAL INFRASTRUCTURES

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ABSTRACT

The objective of earthquake-resistant design of critical infrastructures like nuclear power plants or lifelines is to ensure the prevention of catastrophic disasters. Experience from recent past like the earthquake of Amatrice (2016) or the Napa earthquake of 2014 have shown that traditional code requirements based on probabilistic seismic hazard maps are not able to prevent disasters. The purpose of probabilistic hazard assessment is to support risk analysis. The latter is used to separate tolerated residual risks from non-tolerable, more frequent risks. Therefore, these methods do not intend to provide protection against extreme events. Additionally, it is proven that the traditional hazard parameter used in probabilistic seismic hazard maps, peak ground acceleration (PGA), is not very suitable for the description of the physical impact of earthquakes on structures, systems and components. The only hazard parameter describing physical effects of earthquakes at least on macroseismic scale is intensity or in engineering units, intensity factors. The actual EMS-98 scale correlates reasonably well with the damage of structures classified into vulnerability classes.

The availability of large databases of registered earthquake time-histories covering a wide range of site intensity values allows to model earthquake impact directly using dynamic time-history analysis methods. On this basis a methodology was developed that allows to design critical infrastructures for certain levels of seismic intensity directly.

The methodology and some applications are presented.

Keywords: disaster prevention, earthquake engineering, performance-based design, seismic hazard analysis

1 INTRODUCTION

The big Japan earthquake of March 11, 2011, the most costly natural hazard ever recorded, but also the earthquakes of Amatrice (2016) and the Californian Napa earthquake (2014) demonstrated that modern seismic hazard analysis and earthquake engineering methods as applied today have significant limitations in preventing disasters. Civil code regulations in most countries of the world are based on probabilistic seismic hazard (PSHA) maps with only few exceptions. Performance-based design methods as developed by NEHRP in the U.S. [1,2] are based on these maps. These methods define a performance level for structures and systems for different seismic use groups ensuring a low probability of failure for a given hazard level. The latter is defined in terms of the probability of exceedance for a certain period of time, e.g. 10% in 50 years for ordinary buildings. Typically three service levels are defined [3]:

- Serviceability.
- Damage control.
- Collapse prevention.

With different corresponding performance criteria:

- Near-elastic response.
- Limited inelastic response.
- Large inelastic response.

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Different design procedures were developed to ensure the intended performance level as:

1. Linear static.
2. Linear dynamic.
3. Nonlinear static.
4. Nonlinear dynamic.

Similarly, for critical infrastructures, like nuclear power plants, an analogous approach was developed distinguishing different performance categories in dependence on the technological function of the structure [4], system or component (SSC) under consideration. This approach defines five different seismic design categories [5] and provides detailed design procedures for the top three categories with the highest functional requirements.

Nevertheless experience from past disasters questions whether this approach is able to guarantee the intended level of safety to the public. Although there are many debates related to seismic design procedures and structural dynamics the root cause of the problem has to be seen in the missing link between seismic hazard analyses and engineering design. The key issues known are:

1. The purpose of probabilistic hazard assessment is to support risk analysis. The latter is used to separate tolerated residual risks from non-tolerable, more frequent risks. Therefore, these methods do not intend to provide protection against extreme events and thus cannot prevent disasters.
2. The limit states of different SSCs have to be defined in terms of different engineering parameters. Both probabilistic and traditional deterministic seismic hazard analysis do not provide the full level of information needed by engineers as long as they are focussing on simple ground motion parameters [6].

This paper presents an approach and the associated methodology that allows to move towards a truly damage-consistent performance-based design with some applications for critical infrastructures. It can easily be expanded to other structures and it makes use of the recent developments in structural dynamics and computer sciences allowing for the application of advanced simulation technics.

2 DESCRIPTION OF METHOD

2.1 Main objectives and general requirements to performance-based methods

The main and primary objective of any seismic design method with respect to disaster prevention is to ensure a very low probability of critical functional failure of the object to be designed even in case of very extreme rare earthquakes.

Additionally, it is reasonable to formulate a secondary objective. The methods and techniques applied shall allow to optimize the economic resources assigned to pursue the primary objective. The objective of optimization of economic resources puts a very strict requirement to the methods applied for the seismic design. They have to be as much as possible realistic and as such, in compliance with empirical information from earthquake recordings. The concept of performance-based design allows allocating resources in dependence on the functional importance of the design object and therefore, in principal has the capability to support an optimization of resources. Unfortunately the actual performance-based methods significantly deviate from the requirement of realism. They suffer from several deficiencies, which may even lead to a violation of the primary objective, to ensure a robust design.

The most relevant deficiencies are:

1. Performance-based criteria are expressed in terms of absolute values for the probability of exceedance [5]. This expresses the belief that the results of a PSHA reflect the recurrence of earthquakes in a realistic way. This belief is not justified. In a short cut this is not correct for the following main reasons:
   a. The stochastic process of occurrence of earthquakes is not ergodic and it is certainly not a homogeneous poisson process [7, 8]
   b. PSHA is based on uniform hazard spectra (UHS). The latter do not reflect the true ground motion response of single earthquakes. An UHS represents the weighted combination of many earthquakes with different damaging characteristics. As the result an UHS cannot be directly related to the physical effects of earthquakes.
   c. Modern PSHAs and their hazard curves consider two main sources of uncertainty: aleatory variability (reproducible in an experiment) and epistemic uncertainty (knowledge-based uncertainty), the latter being subjective. Thus results of PSHA by definition deviate from empirical observations due to the subjectivity associated with the treatment of uncertainty [9].
2. The main focus on acceleration spectra instead of (as for example) tripartite spectra in PSHA limits the applicability of hazard assessment results. The governing seismic failure modes for many SSCs are not controlled by peak values of accelerations.

As a consequence, the performance-based methods in use today are not related to the observed physical effects of earthquakes. Therefore, the quality of a seismic design based on these methods cannot be judged at all. The current methods cannot be recommended for the development of a graded approach to the seismic design of SSCs.

As general requirements for the development of an alternative approach to performance-based design one can define:

- Compliance with empirical information on the physical effects of earthquakes
- Flexibility to allow for the use of different engineering parameters suitable for the design of different structures.

The performance-based method presented below is meeting these requirements.

2.2 Selection of seismic hazard parameter

A realistic performance-based design is only possible if the seismic hazard assessment is performed in parameters that are closely linked to the physical damage in earthquakes. The only hazard parameter that is directly connected to the physical effects of earthquakes and that is used in seismic hazard assessment is intensity [6]. The intensity scale EMS-98 [10] is directly linked to the physical effects of earthquakes. It is very well calibrated against observed damage for buildings of different vulnerability classes. Therefore, intensity in EMS-98 provides the perfect basis for the development of a performance-based design.

Certainly, engineers pose the challenging question that they have not learned to perform calculations in intensities or in intensity factors (if integer site intensities are converted into intensity factors). Therefore, intensities have to be converted into engineering parameters.

The progress in the registration of time–histories from earthquakes as well as the large progress in computational simulations allows to solve this issue that for a long time has prevented the use of intensities as the basis of seismic design of SSC. Big databases similar to Ref. [11] that were prepared as part of the PEGASOS refinement Project (PRP) [12] contain a large number of time-histories that can be classified in terms of site intensity and site conditions. They can be used directly for structural analyses and the design of SSCs. For performance-based design it is mandatory that a set of time-histories is used to cope with the uncertainty associated with time-histories leading to the same site intensity and therefore to the same structural damage. Figure 1 illustrates the range of uncertainty of time-history recordings for intensity VIII (EMS-98) on the example of records from 11 different earthquakes.

In terms of PGA we observe a spread of data from 0.06 g (earthquake #8, Kerman, 22.02.2005, Derwood station (Iran)) to 0.37g (earthquake#1, Friuli, 6.05.1976, station Tolmezzo Centrale – Diega Ambesta 1). It is worth to note that this spread cannot be explained by site conditions lonely.

The database [11] contains time-histories that very well cover the range of intensities up to VIII, containing a few records of intensity IX. For critical infrastructures such as nuclear power plants or river dams the recommended range of seismic site conditions corresponds to this range of recordings. The direct use of time-histories provides the flexibility required for a performance-based design. If necessary they can be converted into response-spectra or tripartite spectra so that traditional response based engineering design methods can be applied. For critical infrastructures or for the design of aseismic foundations (base-isolated systems), the direct use of time histories is strongly recommended.

For high seismic areas there it may be necessary to cover higher intensity ranges the lack of recorded time-histories can be compensated by different types of waveform modelling, reaching from synthetic seismograms like in the neodeterministic method [13] to kinematic or dynamic modelling. A method successfully applied in high seismic regions is the 'Japanese recipe' used for the design of critical infrastructures [14] including nuclear

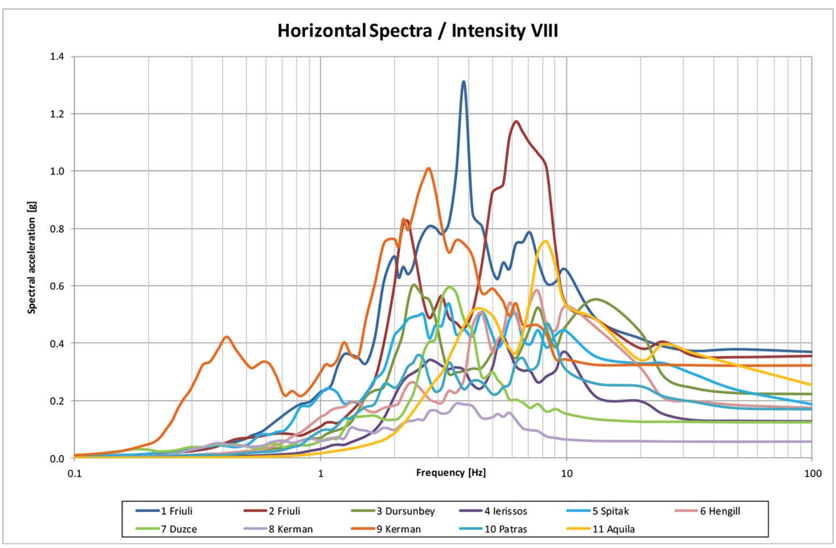

Figure 1: Uncertainty range of horizontal response spectra for intensity VIII.

power plants. Waveform modelling allows to develop all engineering parameters wanted by engineers including the development of tripartite spectra used in the more traditional design methods.

2.3 Adaptation of seismic hazard analysis results

National standards in many countries require the implementation of PSHA as the basis for the seismic design or the review of the design of structures, systems and components including critical infrastructures. Some countries require a comparison with deterministic hazard assessment results or even prefer a deterministic, physics-based approach. All types of hazard analysis can be performed directly in terms of intensity or can be converted into intensity scale. For example, in Switzerland the large scale PSHA-Study PEGASOS [15] was converted into site intensities (site intensity factors) using the results of hazard deaggregation [16]. Hazard deaggregation and conversion into intensity scale frequently allows to understand the true meaning of a study results in terms of the damaging effects expected. Because the uncertainty of hazard assessment results is frequently bounded within a single intensity unit the conversion into integer site intensity values is simple and the range of uncertainty can be maintained and propagated through the analysis.

2.4 Performance-based design goals for critical infrastructures

Due to the difficulties to calculate the frequency of earthquakes in a realistic way it is suggested to define performance criteria in terms of conditional probability of failure for a specified design earthquake. A review of the performance-based approach used in [5] shows that for many SSCs even for increased service levels (seismic design categories SDC 3 to SDC 5) the conditional probability of failure is rather high, reaching from 0.25 to 0.025. The requirements in [5] are significantly lower than in many other countries of the world. The approach suggested here is closer to the international practice. It is based on the traditional double earthquake concept distinguishing between an operability earthquake and a safety earthquake. The categorization of performance levels for SSCs follows the traditional approach in the industry by distinguishing between serviceability, damage control and collapse prevention. The seismic design categorization distinguishes between three categories corresponding to different levels of serviceability. The categories are:

- SDC 1 – damage control for the operational earthquake OE and collapse prevention for the safety earthquake SE are required
- SDC 2 – serviceability for the operational earthquake OE and damage control for the safety earthquake SE is required
- SDC3 – serviceability for the safety earthquake SE is required

The following table describes the performance goals suggested in terms of conditional probability of significant deviation from linear-elastic response.

For the calculation of the conditional probability of significant deviation from linear-elastic response it is possible to use the well-established methods for developing fragility functions in terms of damage indices as needed by engineers [3]. This allows for a structure or component specific approach and ensures a high level of flexibility with respect to the engineering parameters selected for the design process.

Table 1: Performance goals for seismic design categories.

Seismic design category	Categorization criteria	Operational earthquake	Safety earthquake
SDC 1	SSCs needed to prevent losses of lives without active function	0.05	0.1
SDC 2	SSCs that shall maintain operability of active function during the operational earthquake or passive SSCs that shall not fail during the safety earth-quake	0.01	0.05
SDC 3	SSCs that shall maintain operability of active functions after the safety earthquake	Close to zero (0.005)	0.01

2.5 Summary of the procedure

The procedure that was developed can be subdivided into the following steps:

1. Convert existing seismic hazard assessment results into intensity scale if they are not yet readily available in this format.
2. Define the design site intensity level for the operational earthquake (OE) and for the safety earthquake (SE).
3. Select time-histories matching the site intensity levels and the site conditions from registered earthquakes. If needed complete the available number of records by wave modelling approaches (synthetic seismograms, kinematic models etc.).
4. Design calculations and verification
   a. Classify structures, systems and components (SSCs) to be designed against earthquakes into seismic design categories according to Table 1.
   b. Develop the design requirements according to the performance-based criteria in table 1.
   c. Construction design process aimed at meeting the requirements of step 4b.
   d. Verification of the design.

3 PRACTICAL EXAMPLES

The application of the procedure is demonstrated on the practical example of the new build of a nuclear power plant. It is assumed that a second unit for the nuclear power plant Goesgen shall be constructed at the same site. As the existing plant the new unit shall be a pressurized water reactor (PWR) from the same vendor. For simplicity it is assumed that the reactor building design will be the same as for the existing unit. Based on the available seismic hazard assessment study PEGASOS [15] it is required to develop the seismic design requirements for structures, systems and components that are categorized into the three different service levels SDC 1 to SDC 3. For simplicity, it is assumed that the design requirements shall be developed in terms of design level PGA and floor response spectral acceleration at 10 Hz (a typical value for the first natural frequency of many pieces of nuclear equipment).

3.1 Seismic hazard analysis results

In Switzerland the safe shutdown earthquake (the safety earthquake according to the nomenclature of the procedure) is defined as an earthquake with a mean annual frequency of exceedance of 10^{-4}/a. The operational earthquake is defined as 50% of the safe shutdown earthquake. Additionally, the seismic design shall envelope the largest historical event. For Goesgen this is the historical earthquake of Basel (1356) that would lead to a site intensity at the Goesgen site of VII–VIII. In [17] the detailed process of hazard deaggregation and conversion to the intensity scale is described. Here the final results are reproduced.

3.2 Design site intensity levels

Based on the results in Table 2 the safe shutdown earthquake (safety earthquake) has to be assigned to a site intensity of VIII and correspondingly, the operational earthquake to site intensity VII.

3.3 Selection of time-histories and conversion into engineering parameters

The seismological design parameters have to be converted into PGA based on empirical observations. Selecting the 11 records presented in Fig. 1 for site intensity VIII we obtain a mean PGA of 0.21g and a standard deviation of 0.1g. From the database [11] we also restore the corresponding time-histories. It is possible to approximate the hazard by a lognormal distribution as it is common for performance-based approaches [3]. Similarly, we proceed for the operational earthquake with intensity VII. For this intensity level we selected 17 records from [11] with a mean PGA of 0.08g and a standard deviation of 0.047g. The hazard assessment results in engineering parameters are shown below:

For the conversion of ground motion parameters into spectral accelerations a dynamic structural analysis was performed. For simplicity, it is assumed that the in-structure floor response at elevation 18.00 m of the reactor building can be regarded as bounding for the design of systems and components of the new nuclear power plant. Therefore, we present here only the results for this building level.

The time-histories selected from the database [11] were used for a dynamic linear analysis of the reactor building using a three-dimensional model in SASSI 2010. Figure 2 shows the resulting linear-elastic floor response spectra and the quantiles of the associated discrete probability distributions for the reactor building level 18m for site intensity VIII.

3.4 Development of performance-based design requirements for SSCs

For the development of performance-based design requirements in the example we consider two typical buildings and three different groups of components. Regarding buildings, we consider the reactor building with the containment and the turbine building. Due to its high

Table 2: Conversion of the PEGASOS hazard to site intensity (EMS-98) – Goesgen site.

Frequency of exceedance, [1/a]	Mean site intensity factor (from PEGASOS hazard deaggregation) (Intensity (EMS-98)
10^{-3}	VI.6 (VII)
10^{-4}	VII.5 (VII–VIII)
10^{-5}	VIII.5 (VIII–IX)
10^{-6}	VIII.9 (IX)

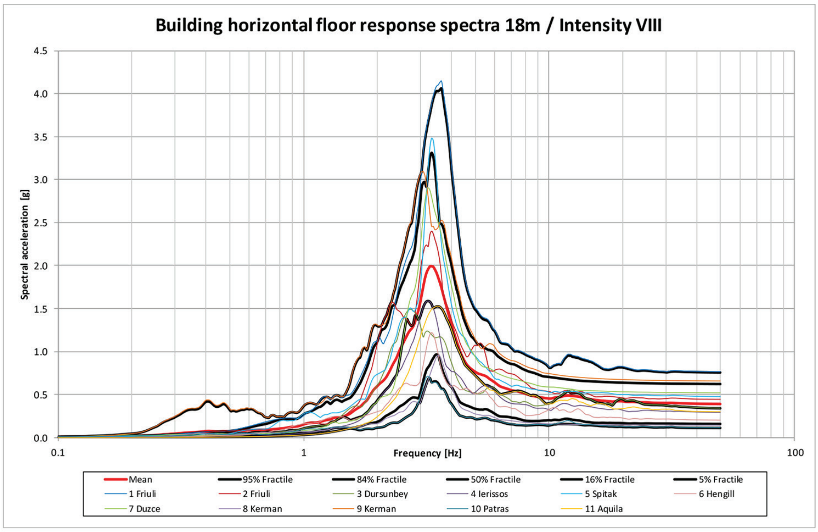

Figure 2: In-structure floor response spectra, intensity VIII, 18m, reactor building.

importance for the safety of the plant, especially with respect to the retainment of radioactivity after an accident, the reactor building is assigned to seismic design category SDC3. The turbine building of a PWR has significantly lower importance (for the vendor AREVA (KWU Germany)). Therefore, it is assigned to seismic design category 2. As additional components tanks, pumps and motor-operated valves (MOVs) are selected for this example. For these groups of components, the performance-based design requirements for all three seismic design categories are developed.

The capacity functions for SSCs are frequently described in the form of double lognormal distributions as [18]:

$$A = A_m \varepsilon_R \varepsilon_U . \tag{1}$$

Here ε_R and ε_U are log-normally distributed with unit medians and standard deviations β_R and β_U respectively. They represent the inherent randomness (aleatory variability) about the median and the epistemic uncertainty of the median value respectively. In some cases, the composite variability is used, defined by:

$$\beta_c = \sqrt{\beta_R^2 + \beta_U^2} \tag{2}$$

A is the seismic capacity and A_m is the unknown median capacity for the component. This parameter is to be defined based on the performance-based requirements in Table 1. The capacity of a component is frequently characterized by its HCLPF (high confidence of low probability of failure) value, which is defined as:

$$HCLPF = A_m e^{-1.645(\beta_R + \beta_U)} . \tag{3}$$

This value approximately corresponds to a probability of failure of 1%.

To calculate the performance objectives given as conditional probabilities of significant deviation the seismic hazard is represented in form of a lognormal distribution with mean and standard deviation as shown in Table 3 and Table 4.

Table 3: Seismic design basis, PGA.

Earthquake	Mean PGA, [g]	Standard deviation, [g]
Safety earthquake	0.21	0.10
Operational earthquake	0.08	0.047

Table 4: Seismic design basis for components, SA (10Hz), 18m.

Earthquake	Mean SA(10Hz), [g]	Standard deviation, [g]
Safety earthquake	0.450	0.216
Operational earthquake	0.146	0.089

$$H = H_m \gamma_R \gamma_U. \tag{4}$$

Here γ_R and γ_U are log-normally distributed with unit medians and standard deviations ϑ_R and ϑ_U similarly as defined for the capacity functions. The combined variability is denoted as ϑ_c.

With capacity and hazard expressed by lognormal probability distributions (using the combined variability as in eqn (2)) the probability of significant deviation from linear elastic behaviour can be calculated analytically from the standard normal distribution.

$$P = 1 - \int_{-z}^{\infty} \varphi(z)\,dz. \tag{5}$$

Here z is defined as

$$z = -\frac{\ln(A_m) - \ln(H_m)}{\sqrt{\beta_c^2 + \vartheta_c^2}}. \tag{6}$$

Using standard values for the component capacity variabilities and for the hazard parameters (converted to logarithmic space) we obtain the results shown in Table 5. The results are expressed in terms of the median design capacity and of the HCLPF value following the standardized approach to fragility analysis for nuclear power plants [18]. Where applicable, the results are presented in terms of PGA and of SA (10 Hz) at the location of the component. The design requirement expressed in spectral acceleration is of practical interest because it defines the boundary conditions for qualification tests. Such tests are required for safety classified components of nuclear power plants.

The analysis of the results shows that the safety earthquake controls the requirements for the design of the pant for all seismic design categories. Therefore, it is feasible to use a single earthquake design for nuclear power plants. Note, that for other industries the situation might be different. It is also worth to note that the application of performance-based methods for nuclear power plants as in use in the USA [4,5] today would lead to a significantly weaker seismic design due to the permissible higher probabilities of failure.

Table 5: Performance-based seismic design requirements for NPP SSCs.

SSC	Performance level	Capacity variability parameters		OE median PGA (SA), [g]	SE median PGA (SA), [g]	Final design, [g]	
		β_R	β_U			Median (A_m)	HCLPF
Reactor Building	SDC3	0.21	0.41	0.80 (-)	0.95 (-)	0.95 (-)	0.34 (-)
Turbine Building	SDC2	0.21	0.41	0.37 (-)	0.61(-)	0.61 (-)	0.22 (-)
Pump	SDC1	0.3	0.23	0.22 (0.41)	0.44 (0.94)	0.44 (0.94)	0.18 (0.37)
Pump	SDC2	0.3	0.23	0.35 (0.64)	0.54 (1.16)	0.54 (1.16)	0.23 (0.45)
Pump	SDC3	0.3	0.23	0.66 (1.21)	0.80 (1.71)	0.80 (1.71)	0.33 (0.67)
Tank	SDC1	0.23	0.35	0.23 (0.43)	0.45 (0.98)	0.45(0.98)	0.18 (0.34)
Tank	SDC2	0.23	0.35	0.38 (0.70)	0.57 (1.22)	0.57 (1.22)	0.22 (0.43)
Tank	SDC3	0.23	0.35	0.75 (1.37)	0.9 (1.93)	0.9 (1.93)	0.35 (0.68)
MOV	SDC1	0.3	0.23	0.22 (0.41)	0.44 (0.94)	0.44 (0.94)	0.18(0.37)
MOV	SDC2	0.3	0.23	0.35 (0.64)	0.54 (1.16)	0.54 (1.16)	0.23 (0.45)
MOV	SDC3	0.3	0.23	0.66 (1.21)	0.80 (1.71)	0.80 (1.71)	0.33 (0.67)

4 SUMMARY AND CONCLUSIONS

A procedure for damage-consistent performance-based seismic design of critical infrastructures was developed. The method allows to achieve the main objective of disaster prevention, a robust seismic design of critical infrastructures, in combination with an optimization of economic resources due to the definition of graded performance goals. The performance goals are defined in terms of conditional probabilities of significant deviation from linear-elastic response of SSCs. The implementation of the procedure is demonstrated for the postulated new build of a nuclear power plant unit. The procedure is applicable for all types of critical infrastructures. The categorization of SSCs into seismic design categories can be adjusted in dependence on the problem and the performance objectives to be addressed.

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