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SEISMIC FRAGILITY OF A REINFORCED CONCRETE STRUCTURE

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ABSTRACT

Structures can be exposed to seismic loading. For structures of major importance, extreme seismic loadings have to be considered. The proof of safety for such loadings requires a sophisticated analysis. This paper introduces an analysis method, which still includes simplifications, but yields a far more realistic estimation of the seismic load bearing capacity of reinforced concrete structures compared to common methods. It is based on the development of pushover curves and the application of time-histories for the dynamic model to a representative harmonic oscillator. Dynamic parameters of the oscillator, such as modal mass and damping are computed using a soil-structure-interaction analysis. Based on the pushover-curve nonlinear force-deformation-capacities are applied to the oscillator including hysteresis behavior characteristics. The oscillator is then exposed to time-histories of several earthquakes. Based on this computation the required ductility is computed. The ductility can be scaled based upon the scaling of the time-histories. Since both, the uncertainty of the earthquake by using different time-histories and the uncertainty of the structure by using characteristic and mean material values, are considered, the uncertainty of the structure under seismic loading can be explicitly represented by a fragility curve.

INTRODUCTION

Seismic loadings are common in regions with high and moderate seismicity. They endanger the critical infrastructures, such as bridges and nuclear power plants, and the lives of humans. Therefore new structures in such regions are designed against seismic loading. Especially for new nuclear power plants major care is undertaken to estimate the correct seismic hazard and to design the systems, structures and components in an appropriate way. Whereas in traditional building and bridge design usually earth-quakes with a 10 % exceedance probability in 50 years are considered (yielding to 475 years return period), for nuclear power structures earthquakes with a return period of up to 10 000 years have to be considered.

The specification of a return period is already an indication, that the uncertainty of the loading is considered by stochastic means. However, whereas current building codes use a semi-probabilistic safety concept, for extreme earthquake loads with high re-turn periods direct probabilistic concepts can be applied. This can be either done by full-probabilistic methods or by fragility analysis. In this paper we will discuss the development of a seismic fragility for an existing reinforced concrete structure. The investigation considers both, advanced structural and dynamic modeling and the sound modeling of the uncertainty by the calculation of the fragility.

The paper is divided into the following parts: First we introduce the concept of seismic fragilities. Subsequently the structural model used for the development of pushover-curves is discussed. In the third section we explain in detail the steps of the dynamic analysis. Finally some results are presented and discussed.

SEISMIC FRAGILITIES

The current safety concept in structural engineering is based on a probabilistic and statistic approach. Whereas other mathematical methods exist to deal with uncertainty (Proske 2011), stochasticity as umbrella term of probabilistic and statistic means is the mathematical tool most widely used, best understood and with the greatest variety of tools. However, the direct application of a full probabilistic analysis is limited to special cases, since modeling, computation and reporting are extremely time consuming. Simulations may run over weeks or even months, although major progress has been made in the last years, both computationally and methodically. Such computation times are not acceptable in daily business. However, if certain assumptions about the outcome of probabilistic computations are made, the required computation time can be significantly decreased. Fragilities belong to this group of simplified probabilistic analyses.

In general, fragilities are functions of the probability of failure of a certain structure, system or component depending on the intensity of the loading. For seismic fragilities the intensity of the loading is represented by the intensity of the earthquake. We have used the peak ground acceleration value as anchor point for the earthquake intensity scale. Other spectral acceleration values may be used as alternative anchor points.

The removal of the loading from the full probabilistic analysis can be seen as a first step of simplification. The next step is an assumption about the function type of the fragility. Usually here only a limited number of probability functions are considered, such as lognormal distribution (EPRI 1994), normal distribution or Weibull distribution. With the selection of the probability function based on recommendation documents, the need to prove this function in the specific case is eased. Some researchers assume that it is extremely difficult, if not impossible to prove the validity of a probability distribution. Indeed it requires high sample sizes or high simulation efforts when looking at the correct representation of the distribution tails. In contrast, by selecting a normal or lognormal distribution, the probability function and the fragility respectively can be fully developed by using only two supporting points. Since the supporting points do not have to be located in regions of extreme low or extreme high probabilities of failure, there is no need to carry out probabilistic computations in these regions. For sampling based probabilistic techniques this is an overwhelming advantage.

Current studies have shown again (Zentner et al. 2008) that the assumption yields reasonably accurate results. Therefore the following probability function and fragility respectively will be used within this study:

$$P_{f/a} = \Phi\left(\frac{\ln(a/A_m}{\beta}\right) \tag{1}$$

with P as probability of failure as function of the seismic intensity a, whereas a is spectral acceleration of an anchor point of the uniform hazard curve (we have used the PGA-value), Φ as Gauss-distribution (normal distribution), Am as value of the spectral acceleration yielding a median failure probability (50 % fractile) and ß as parameter of uncertainty (standard deviation). Furthermore, the uncertainty parameter is divided into aleatoric and epistemic uncertainty. The aleatoric uncertainty is the immanent uncertainty of the material and loading. In contrast, the epistemic uncertainty considers the limitation of knowledge. Whereas the epistemic uncertainty can be lowered by additional data, the aleatoric uncertainty is immutable.

It is common to consider the aleatoric uncertainty as a random variable, whereas the epistemic uncertainty is considered as uncertainty of the statistical parameters, usually known as confidence intervals. This yields the function corridor shown in fig. 1. The complete probability function and fragility respectively with explicit consideration of the types of uncertainty is

$$P_{f/a} = \Phi\left(\frac{\ln(a/A_m + \beta_u \Phi^{-1}(Q)}{\beta_R}\right) \tag{2}$$

with $ß_R$ as aleatoric uncertainty (standard deviation), $ß_U$ as epistemic uncertainty (standard deviation) and Φ^{-1} as inverse Gauss distribution. Furthermore often the composite-uncertainty is used, which is defined as:

$$\beta_C = \sqrt{\beta_U^2 + \beta_R^2} \tag{3}$$

An important point on the fragility curve is the HCLPF-value (High Confidence of Low Probability of Failure). This value can be directly related to the semi-probabilistic codes of practice. Modern codes of practice use so-called characteristic material values. The HCLPF value is defined as the 5 % fractile value with 95 % confidence interval. It is interesting to note, that with the application of nonlinear methods also structural characteristic values and safety factors have been developed (Cervenka et al. 2008), which can be interpreted as the HCLPF-value. Since maintenance of the distinction of the uncertainty parameters throughout the analysis is very difficult, the HCLPF-value is often defined as the 1 % fractile value of the fragility function assuming, that the confidence interval will be covered by the adjustment from the 5 % to the 1 % fractile value. This gives the HCLPF-value with

$$HCLPF = \frac{A_m}{\exp(2.33 \cdot \beta_c)} \tag{4}$$

The unit of the HCLPF-value, but also of the Am value and the uncertainty parameters depends on the selection of the earthquake intensity parameter. We have used the acceleration unit g.

For the computation of the fragility curve a sufficient structural model is required.

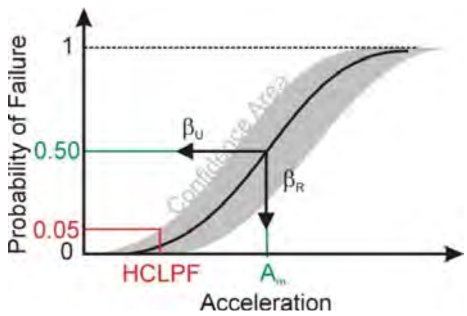

Figure 1. Seismic Fragility: Terms..

DETERMINISTIC MODEL

The structure, which is investigated, is built of reinforced concrete. The structure was erected with in-situ concrete at the end of the 1960s. The non-linear behaviour, which is expected during a strong earth-quake, has to be considered by a pushover-analysis to receive realistic earthquake loadings. With such an analysis the entire force and deformation capabilities of the structure can be determined as well as the energy dissipation capabilities. Furthermore the pushover-curve will identify local failure and therefore will indicate weak points in the structure. In general, a pushover-curve is a force-deformation-function for a structure regarding a one-sided horizontal loading (FEM 356 2000, Meskouris et al. 2007). In contrast to a full dynamic analysis, it is a static non-linear analysis.

Obviously, the results of such an investigation depend strongly on the load shape. Therefore the load shape has to be chosen carefully. In our case, we have chosen the load shape based on results of the probabilistic soil-structure interaction analysis.

The pushover-analysis was carried out with the program ATENA. This program can consider all important features of reinforced concrete behaviour such as concrete cracking, crushing, reinforcement yielding or rupture (Cervenka & Pappanikolaou 2008, Červenka et al. 2010). The finite element model considers four floors above a very massive foundation slab. It includes columns, beams and walls (fig. 2). For the model more than 3'000 three-dimensional higher-order-volume-elements were used. In the columns, the reinforcement was modeled with discrete reinforcement bars, whereas in the other structural elements the reinforcement was modeled as smeared reinforcement. The computation time for one pushover-curve was in the range of one day.

Both, element type and element size have been investigated by a sensitivity study of a column. It was shown that even with this detailed and laborious modeling, the load bearing capacity can still be considered as conservative.

Fig. 3 shows a deformation figure of the structure under horizontal load during a pushover-analysis. Fig. 4 shows an obtained pushover-curve. The considerable ductility of the structure becomes clearly recognizable in this figure. Furthermore a slight difference in the positive and negative horizontal loading behaviour is visible. This difference in the range of 20 % is caused by a non-symmetric structural behaviour.

The pushover-curve was computed either based on mean material properties (fig. 4) or based on characteristic material properties. Therefore two support points for the fragility curve have been set.

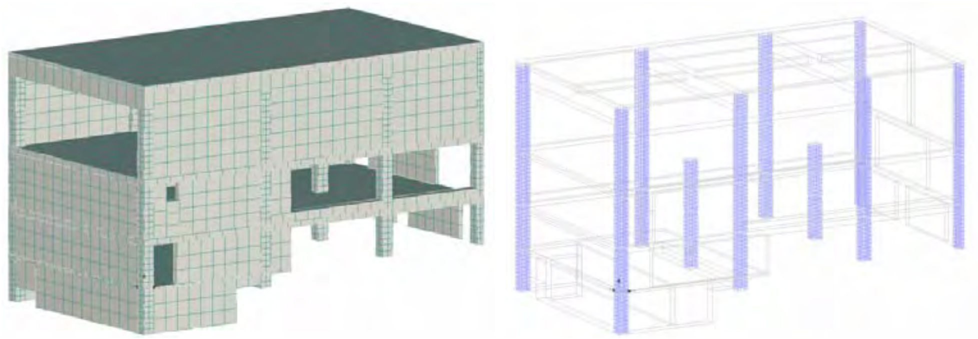

Figure 2. Mesh of the Finite-Element-Model used (above) and visualization of the reinforcement modeled in the columns (bottom).

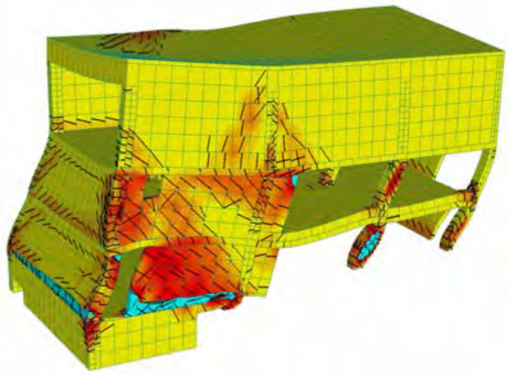

Figure 3. Deformation of the structure during a pushover-analysis.

DYNAMIC ANALYSIS

The pushover-curve is an essential part of the analysis introduced here. However it is only the first step, since it does not yet consider any dynamic properties. The curve is independent from the seismic dynamic loading and therefore it can be used even under changing seismic loading spectra. In contrast, the following steps are highly dependent upon the specific seismic loading.

Usually the seismic hazard is given as a uniform hazard spectrum for a certain ground level and a certain return period of the value (fig. 5, top). Such hazard spectra are related to hazard curves given for certain frequencies (fig. 5, bottom). In general, the uniform hazard spectra gives the spectral acceleration over the frequency range for a certain return period, whereas the hazard curve gives the spectral acceleration over the return period for a certain frequency. In fig. 5 the peak ground acceleration was used as anchor point of the hazard curve. Furthermore fig. 5 shows the relation between the hazard curve and the uniform hazard spectrum.

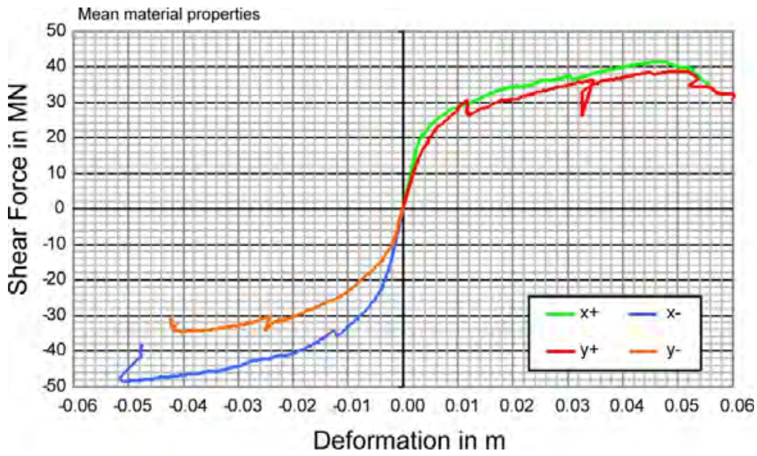

Figure 4. Pushover-curve based on mean material properties.

The uniform hazard spectrum and the hazard curve can be either taken from codes, as usually done for common building structures, or they are provided by experts. The development of site specific hazard curves for nuclear power plants can be a long-lasting process. For example, during the PEGASOS Project and the subsequent Pegasos Refinement Project the seismic loading for the nuclear power plant sites in Switzerland has been estimated (Renault 2011).

In a first step, for the given uniform hazard spectra acceleration time histories were developed. Detail about this time histories are given later. Based on this spectra-compatible acceleration time histories a dynamic soil-structure interaction (SSI) analysis was carried out using the program SASSI. Although not directly linked to this step of the analysis, one result of the SSI are floor response spectra, which can be used for the proof of components inside the buildings (APA Consulting 2011). Fig. 6 shows such a floor response spectrum. Furthermore, the SSI considers random input variables and provides statistical information about the results. For example, the spectral acceleration in floor response spectra can be given as median and 84 fractile values.

The SSI gives the fundamental frequency of the structure and the modal parameter such as mass and damping considering the coupled system (soil-structure) (Sadegh-Azar & Hartmann 2011). In contrast, the pushover curves are based on the assumption of a completely stiff soil. The damping considers material damping of the structure, material damping of the ground (strain-dependent damping) and emission damping of the ground (foundation and light embedment). To consider the high uncertainty of the different types of damping, variations were included in the analysis. Furthermore the SSI provided the effective height and the participation factors. These factors are required to scale the deformation from the top of the building computed with ATENA to the deformation at the effective height. With this information we are able to construct an equivalent oscillator with the dynamic properties of the full structure.

So far the pushover curve considers only monotonic increasing loads. However for the non-linear dynamic analysis cyclic structural behaviour has to be considered. This is defined by hysteresis functions. The representative oscillator model is expanded in the next step to include such hysteresis functions.

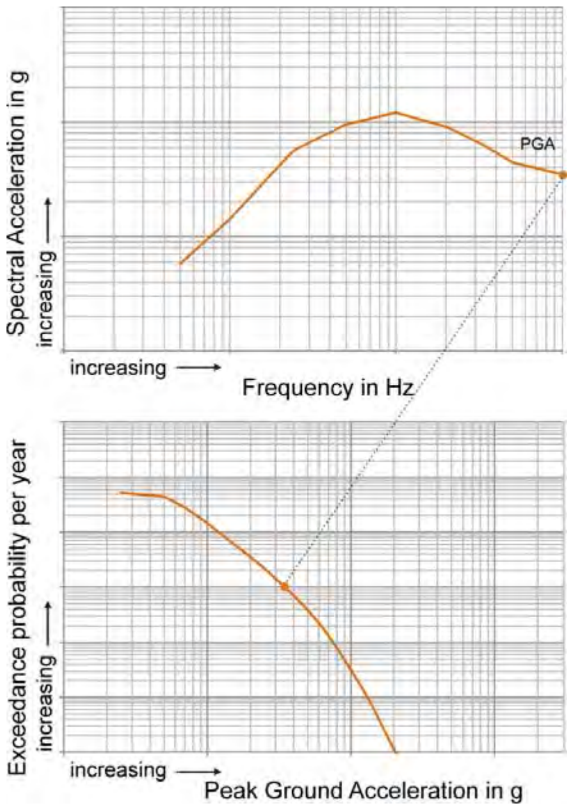

Figure 5. Uniform Hazard Spectrum and Hazard Curve (PGA: Peak Ground Acceleration).

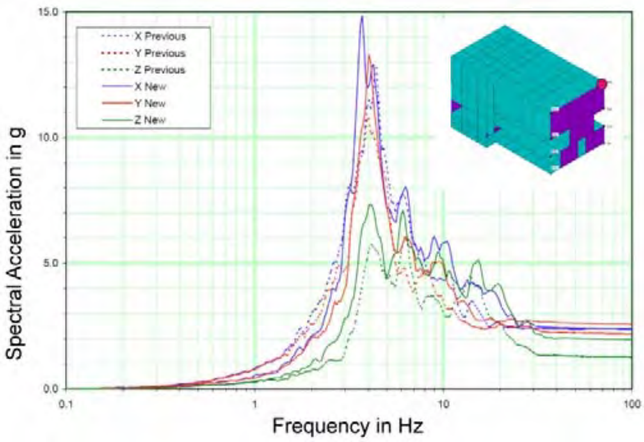

Figure 6: Floor Response Spectra in the upper part of the building

There exist different theoretical hysteresis models. We have used a hybrid model consisting of the Modified Takeda (55 % contribution) and the Origin Centred (45 % contribution). The methods are illustrated in fig. 7. The hybrid model shows lower hysteretic energy dissipation than the single Modified Takeda (Kurmann 2009). It should be noted, that dynamic experiments have already been reproduced by a ratio of 85% Modified Takeda and 15 % Origin Centred (Bimschas & Dazio 2008). Therefore the analysis of the existing building, using a mixture of the two chosen hysteretic models, is conservative. The experiments indicate a higher ductility.

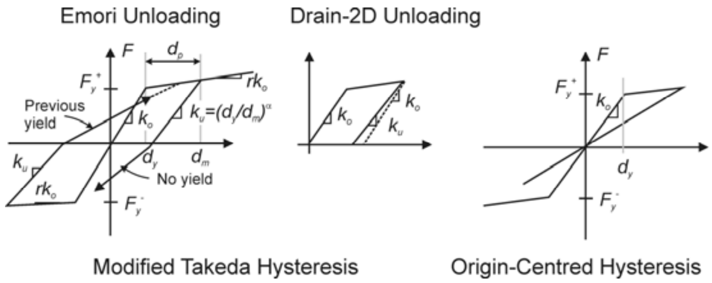

Figure 7 Used hysteresis rules for the non-linear dynamic analysis.

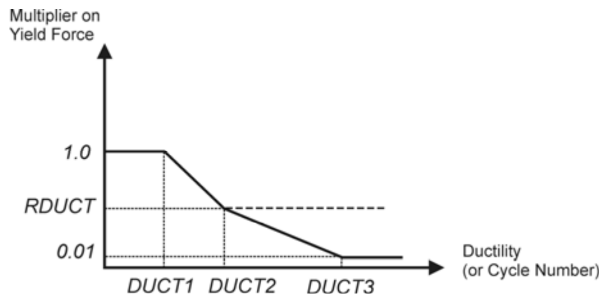

Figure 8. Model for ductility dependent on stiffness-drop according to Ruaumoko (Carr 2004).

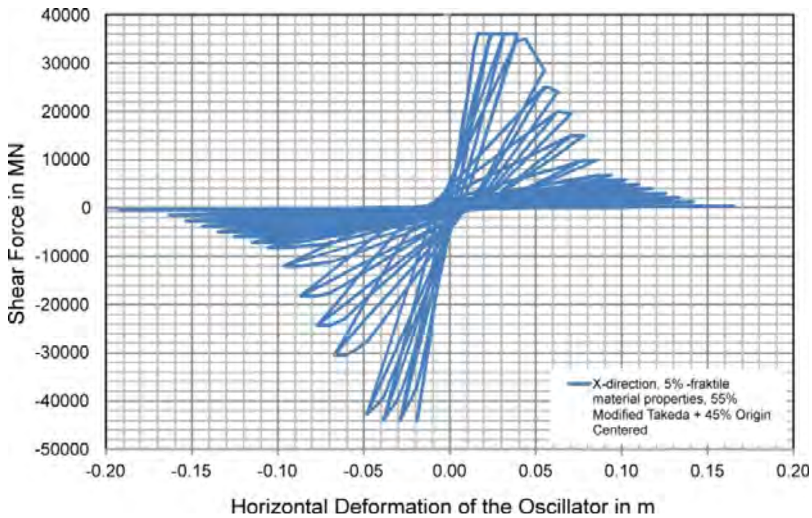

Figure 9. Hysteresis in X-direction using the 5 % fractile Value of Material Properties.

Finally, a function for the force drop has to be defined. We have chosen the force decrease related to ductility shown in fig. 8. Fig. 9 shows the force-deformation-function of the oscillator for monotone increasing ductility demand. The related hysteretic damping is shown in fig. 10.

In the next step, the representative oscillator is exposed to 30 acceleration time histories based on real earthquake measurements (fig. 11). Only earthquakes with magnitude 6.5 to 7.5 were considered. The earthquake duration was between 18.5 and 24 seconds. These 30 curves are selected and modified to meet the uniform hazard spectrum over a great frequency range.

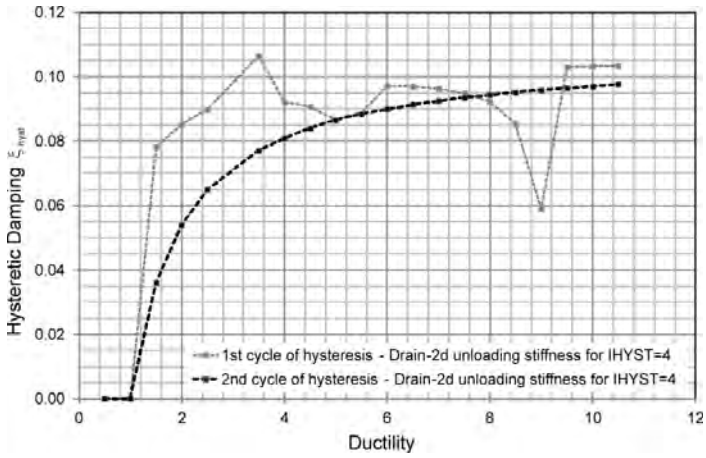

Figure 10. Hysteretic damping of the oscillator.

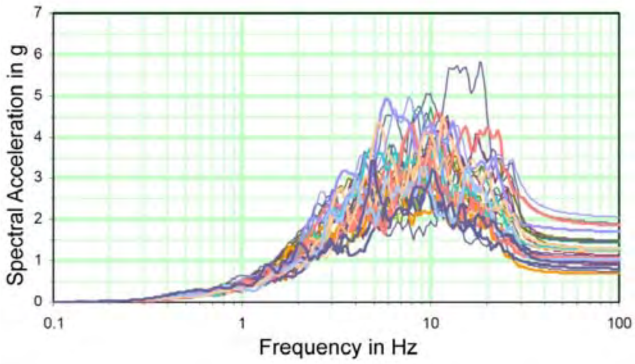

Figure 11. Individual Spectra of the different Acceleration-Time-Functions.

By using the developed model and the acceleration time histories, we can compute the ductility of the structure. However, the 30 applied acceleration time histories are related to one PGA-value only. Therefore these curves have to be varied to consider different earthquake intensities. In our case the time histories were incrementally up-scaled by 0.2 g based from 0.05 PGA. This method is called incremental dynamic analysis.

With this analysis the required ductility of the structure is determined for certain intensities (fig. 12). The structural failure is then defined by exceeding a maximum acceptable ductility or deformation. Alternatively an impact against neighboring structures can limit the deformation capacity.

Since the analysis was not only carried out using mean material properties but also characteristic material properties and furthermore using the 30 time histories, also the uncertainty is considered in the analysis. The uncertainty of exceeding the maximum acceptable ductility is given in terms of β_R and β_U.

Finally it should be mentioned, that the simultaneous seismic loading from the X-, Y- and Z-direction was included in the analysis by using the 100-40-40 rule. This rule states, that in one direction 100 % of the seismic loading is considered, whereas at the same time in the other directions only 40% of the loading is used.

The single steps of the described analysis are visualized in fig. 13. In the first step the non-linear load bearing behaviour was investigated by a pushover-curve. Then a representative oscillator was modeled considering dynamic properties of the structure. To prepare these properties a soil-structure-interaction analysis was carried out. Finally the oscillator was exposed to several acceleration time histories. Since the entire analysis was carried out for different material properties and different time histories, both central tendency estimators as well as uncertainty estimators were computed. With this data the fragility can be constructed.

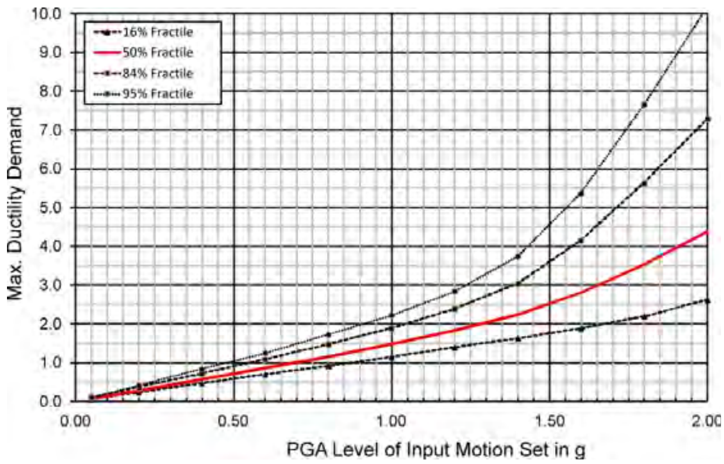

Figure 11. : Incremental dynamic analysis for representative oscillator with characteristic material values.

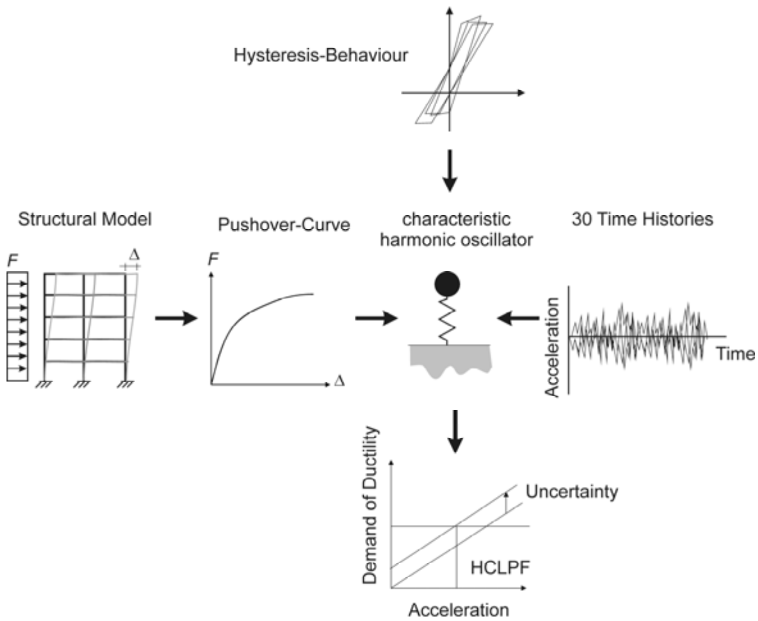

Figure 12: Analysis flow chart

RESULTS

The dynamic analysis shows, that the load bearing behaviour of the investigated structure is nearly identical for both horizontal directions. The small difference is based on the different torsion behaviour in the main directions. Therefore the fragility of the weaker direction is used. The HCLPF value is the anchor point of the fragility curve based on the maximum ductility. It is in the range of 0.82 g PGA, which is a considerable value. It is about double the acceleration, when the yielding of the reinforcement in the structure begins. This fact proves that non-linear behavior has to be considered in the analysis of reinforced concrete structures under seismic loading to achieve realistic results.

The HCLPF value for impact against neighboring structures is in the range of the yielding of the reinforcement. However impacts will occur at the top of the structure and therefore will not yield to a total collapse of the structure. Furthermore the impact will only cause local damage. A suitable damage grade has to be selected carefully.

Since the uncertainty of the loading and the resistance was considered, the analysis does not only provide characteristic value, such as the HCLPF value or mean values, but also variation values. Therefore the full fragility including these uncertainty parameters was developed. The final results are given with $ß_R = 0.19$, $ß_U = 0.34$ und $A_m = 1.94$ g. These values can be used consecutively in a Seismic Probabilistic Safety Assessment (SPSA).

By the application of advanced static and dynamic models, major computational reserves were utilized and proof of the load bearing capacity of the structure was even successful for extreme rare seismic events. We would classify the analysis according to the system by Sadegh-Azar & Hartmann (2011) as the most sophisticated method. An application to other structure types, such as bridges, is also possible (Kurmann 2009).

In general, the robustness of carefully designed and built reinforced concrete structures has been observed during some heavy seismic loadings in recent events (Japan, Chile, New Zealand). The damage patterns of these types of structures indicate great safety margins, which can be directly incorporated in probabilistic investigations such as the one presented here. Furthermore the results can be included in probabilistic risk assessments considering not only the possibility of the structural failure, but also express the consequences. In general, the analysis shows that civil engineers have learned from some very tragic seismic events and are now able to design and build structures that resist heavy seismic loadings. This yields a considerable decrease of the hazard for humans and structures in seismic active regions.

REFERENCES

Proske, D. (2011): Zur Zukunft der Sicherheitskonzepte im Bauwesen, *Bautechnik*, V. 88, 4, pp. 217-224

EPRI (1994): *Methodology for developing seismic fragilities*, prepared by J. R. Benjamin and Associates, Inc and RPK Structural Mechanics Consulting, TR-103959, Project 2722-23

Zentner, A. Nadjarian, N. Humbert & Viallet, E. (2008): Estimation of fragility curves for seismic probabilistic risk assess-ment by means of numerical experiments. C.-A. Graubner, H. Schmidt & D. Proske (Eds.): *6th International Probabilistic Workshop*, 26–27 November 2008, Darmstadt, Germany 2008, Technische Universität Darmstadt, pp. 305–316

Cervenka, J., Cervenka, V., Pukl. R & Janda, Z. (2008): Assessment of remaining structural capacity by computer simulation, *Tailor Made Concrete Structures* – Walraven & Stoelhorst (eds), 2008 Taylor & Francis Group, London, ISBN 978-0-415-47535-8, pp 665-671

FEMA 356 (2000): *Prestandard and commentary for the seismic rehabilitation of buildings*. Federal emergency management agency, American Society of Civil Engineers, Reston, Virginia

Meskouris, K., Hinzen, K.-G., Butenweg, C., Mistler, M. (2007): *Bauwerke und Erdbeben: Grundlagen - Anwendung – Beispiele*, Vieweg + Teubner Verlag; 2. Auflage

Cervenka, J., Pappanikolaou, V. (2008): Three dimensional combined fracture-plastic material model for concrete, *International Journal of Plasticity*, Vol. 24, 12, pp. 2192-2220

Červenka, J., Jendele, L., Červenka, V. (2010): *ATENA Program documentation*, Cervenka Consulting

Renault, P.: PEGASOS Refinement Project (2011): New Findings and Challenges from a PSHA for Swiss Nuclear Power Plants. In: Dutta, B. K., Singh, R. K. & Durgaprasad, O. V. (editors), *SMIRT 21 - International Conference on Structural Mechanics in Reactor Technology*, New Delhi, India

APA Consulting (2011): *Development of Acceleration Time Histories for the Intermediate PRP Uniform Hazard Spectra*, Walnut Creek

Sadegh-Azar, H., Hartmann, H.-G. (2011): Grundlagen der seismischen Auslegung von Kernkraftwerken und Einfluss der Boden-Bauwerk Wechselwirkung, *Bauingenieur*, Band 86, 3, pp. S3-S10

Kurmann, D. (2009): *Seismic Analysis of Existing Bridges with Detailing Deficiencies*, ROSE School Master Thesis, Istituto Universitario di Studi Superiori di Pavia, Università degli Studi di Pavia

Bimschas, M. & Dazio A. (2008): Large Scale Quasi-Static Cy-clic Tests of Existing Bridge Piers, *Proceedings 14th World Conference on Earthquake Engineering,* Beijing, China

Carr, A. J. (2004): RUAUMOKO-Volume 2; *User Manual for the 2D-Version, Ruaumoko 2D*, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand