A probabilistic concept for the evaluation of anchor displacements under earthquake excitation

Torsten Luther*, Hamid Sadegh-Azar

HOCHTIEF Solutions AG, Consult IKS Energy, Frankfurt am Main

Abstract

Anchor plates with undercut anchors are frequently used, e.g. in fossil or nuclear power plants, to fasten safety-related systems and components. The anchor plates have to resist significant loads under earthquake excitation without loss of safety or function. On the other hand, earthquakes can cause large cracks in concrete structures at the anchor locations, which will result in large displacements of the affected anchors. Anchor displacements have to be minimized to ensure the functionality and safety of attached systems and components. A concept is presented to numerically determine realistic anchor displacements of installations and components under earthquake excitation and to specify exceeding probabilities for design limits using the software optiSLang. All relevant variables with influence on the resulting displacements are quantified by stochastic variables. The computation and evaluation of anchor displacements under earthquake excitation is demonstrated for a typical building of a nuclear power plant. Realistic anchor displacements under earthquake excitation are calculated by comprehensive dynamic computations in time domain within the framework of a Monte-Carlo-Simulation and applying variance reduction techniques. The probabilistic simulation yields a stochastic distribution of seismic anchor displacements resulting from realistic boundary conditions defined by stochastic parameters. Based on the resulting distribution of anchor displacements, exceeding probabilities for displacement design limits can be calculated and evaluated considering the stochastic variation of the design earthquake itself.

Keywords: Stochastic, Seismic Analysis, Anchor Displacements, Time History Analysis

* Contact: Dr.-Ing. Torsten Luther, HOCHTIEF Solutions AG, Consult IKS Energy, Lyoner Str. 25, D-60528 Frankfurt am Main, E-Mail: torsten.luther [@] hochtief.de

1 Introduction

Anchor plates with undercut anchors are frequently used, e.g. in nuclear power plants, to fasten safety-related systems and components. These include for instance, mechanical components, power piping, steel structures, and cable racks. The anchor plates have to resist significant loads under earthquake excitation without loss of safety or function. Also, earthquakes can cause large cracks in concrete structures at anchor locations, which will result in large displacements of the affected anchors.

Anchor displacements have to be minimized to ensure the functionality and safety of attached systems and components. In case of unlimited anchor displacements knocking effects can occur, which amplifies the pull out of concerned anchors and affects the attached components. Moreover, nonlinearities, which are coming along with large anchor displacements, cannot be captured within the design process. In Germany, the qualification procedure of anchor systems under earthquake excitation in nuclear power plants requires experiments to evaluate anchor displacements under most unfavorable conditions with regard to anchor forces, crack widths and the number of crack opening/closing sequences during earthquake. The corresponding tests are specified in the DIBt-guideline (2010). Based on a deterministic safety concept, these experiments should verify the safety and functionality of anchor systems in case that all aforementioned unfavorable boundary conditions occur at the same time and in the same direction. Thus, the conservatively measured worst case anchor displacements clearly overestimate the expected real displacements of installations and components. For a realistic calculation of expected anchor displacements under real earthquake excitation, all variables, which significantly influence the anchor displacements, have to be quantified and evaluated in a realistic and non-conservative manner.

A concept is presented to numerically determine anchor displacements of realistic installations under earthquake excitation and to specify exceeding probabilities for displacement design limits by a reliability analysis using the software optiSLang. Therefore, influencing variables (e.g. soil characteristics, eigenfrequencies of structural elements and plant components, utilization factors of anchors, and crack widths) are defined by stochastic variables. The whole analysis process is simulated numerically, including earthquake excitation acting on the building, crack opening time-histories in structural elements, and loading time-histories at anchors. The realistic anchor displacements under earthquake excitation are calculated by comprehensive dynamic computations in the time domain within the framework of a Monte-Carlo-Simulation and applying variance reduction techniques. The probabilistic simulation yields a stochastic distribution of anchor displacements resulting from realistic boundary conditions defined by stochastic variables. The probabilistic concept is applied to undercut anchors in a typical building of a nuclear power plant. Based on the resulting distribution of anchor displacements, exceeding probabilities for design limits can be calculated and evaluated considering the stochastic variation of the design earthquake itself.

2 Determination of anchor displacements under earthquake excitation

2.1 General

In Germany the design of anchor systems under earthquake excitation in nuclear power plants is based on a qualification procedure. This procedure includes experimental tests according to recommendations of the DIBt-guideline (2010). The guideline assumes that cracks arise in concrete structures under earthquake excitation and that these cracks show cyclic opening and closure during an earthquake. Furthermore, the reaction forces in anchor systems underlie cyclic changes in case of earthquake excitation, in which the dynamic behavior of the fastened components significantly affects the time response of anchor reaction forces.

Assuming that under earthquake excitation a crack runs exactly through the anchor hole, the DIBt-guideline (2010) recommends different cyclic experiments to evaluate anchor displacements, which correspond to the design earthquake. On the one hand an anchor displacement is experimentally determined for an opened crack in the anchor hole and under cyclic tension with 10 load cycles. On the other hand an anchor displacement is measured with constant tensile load and 5 cycles of crack opening and closure. Using a deterministic safety concept, these experimental boundary conditions are very conservative, even for lack of more precise insights.

The paper suggests a probabilistic concept to evaluate realistic anchor displacements under earthquake excitation. The concept is based on a detailed quantification of all parameters, which have a significant influence on these displacements. With it, the very conservative assumptions in the experimental procedure shall be replaced by realistic boundary conditions in numerical simulations of anchor displacements under earthquake excitation.

Fundamental conservatives in the deterministic concept and within the corresponding experiments according to the DIBt-guideline (2010), which are in contrast to real situations of installed anchor systems, are listed below:

- Typically, anchor systems consist of multi-anchor plates. Due to the distribution of forces multi-anchor plates show different displacements compared to single anchors under most unfavorable design loads and crack width. The effect of plates is not yet considered in the DIBt-guideline (2010).
- The deterministic concept assumes maximal crack width in all anchor holes of a multi-anchor plate at the same time. In real concrete structures cracks with maximal crack width are in larger distance to each other, because of stress relaxation in cracked reinforced concrete.
- In case of components, which are fastened with multiple anchor plates, the deterministic concept implies maximal crack width in all anchor holes of all anchor plates. This is a highly conservative assumption.

- The experiments are carried out with the value of maximal crack width. This value is equal to the 95%-fractile of the crack width distribution in a concrete structure, which is fully utilized under design earthquake loads. The real distribution of maximal crack width is not considered. Furthermore, design reserves exist in most cases and the reinforced concrete sections are not fully utilized.
- In experiments according to the DIBt-guideline (2010) anchors are loaded with maximal loads. By contrast, anchor systems in real situations typically have design reserves.
- In real situations the maximal crack width and maximal anchor tensile force do not occur always at the same time and for all crack opening and load cycles.

Realistic boundary conditions for numerical simulations of anchor displacements under earthquake excitation are specified in Section 4 for a typical stiff building of a nuclear power plant.

2.2 Dynamic analysis of buildings under earthquake excitation

Anchor displacements under earthquake excitation mainly depend on two types of internal forces. On the one hand they depend on the tension force N which acts on the anchor. On the other hand they are significantly affected by the bending moment M of the associated structural part to which the anchor is fastened and in which the quantity of M primarily correlates with the quantity of crack opening. The determination of the response time-history of internal force variables is realized in the time domain by linear dynamic analyses of the primary structure with attached component fastenings in the model. The complete simulation is performed with one global model, including the excitation, the calculation of bending moments and correlated crack openings, and the tension force at anchors. The excitation of the building is realized by acceleration time series $\ddot{u}_n(t)$ representing the specific design earthquake.

Figure 1 shows a scheme of the analysis process of a primary structure with components. The building is modeled either as beam or as shell structure with additional mass points, if necessary. The soil-structure interaction is covered by spring elements with stiffness derived from soil properties. The anchor systems are also modeled by spring elements and attached mass points, which represent fastened components. The corresponding spring stiffness and mass are adjusted to cover the basic dynamic behavior of appropriate real component fastenings. In the model, components can be assigned to walls and slabs. The response time-histories of bending moments $M_j(t)$ are determined at anchorage points j in walls or slabs, respectively. The response time-histories of anchor forces $N_j(t)$ at these anchorage points are identical to forces in the corresponding spring elements.

The example of Section 4 refers to a typical stiff building of a nuclear power plant. In Germany, the design earthquakes of nuclear power plants are defined by a wide-band frequency spectrums, which according to the standard KTA 2201.1

(2010) represent the various characteristics of different acceleration time-histories of possible ground motions. Based on a specified wide-band frequency spectrum it is possible to generate artificial acceleration time-histories, which are applied as earthquake excitation to the model and which all of them cover the initial spectrum. As a consequence these artificial acceleration time-histories come along with higher energies and a higher number of large amplitudes compared to natural ones. For this reason natural acceleration time-histories of earthquake excitation are used in the present framework to evaluate anchor displacements. The natural time-histories are selected for the power plant location with respect to soil consistency, geological conditions, and the maximal ground acceleration and the ground response spectrum of the design earthquake. Typically, an earthquake analysis is performed with 10 natural time-histories per structural model. In case of linear analyses the results of all calculation runs can be averaged.

Within the dynamic analyses two ground models with different parameters of soil material are applied to consider soil variations. On the one hand the "*MIN*"-model with reduced values of ground stiffness is used and on the other hand the "*MAX*"-model with increased values is applied. Following the recommendations of the standard KTA 2201.1 (2010) and based on the average ground stiffness, which is typically specified in the soil investigation report, the reduction of ground stiffness is realized by factor 2/3 and the increase by factor 3/2. The dynamic calculation of internal force time-histories is performed for both soil models and each with all natural time-histories of earthquake excitation. The internal design forces are defined by the relevant extreme values of the finally derived time-response functions, which are averaged functions of the time-response calculations of internal forces for the "*MIN*"- and "*MAX*"-model, respectively.

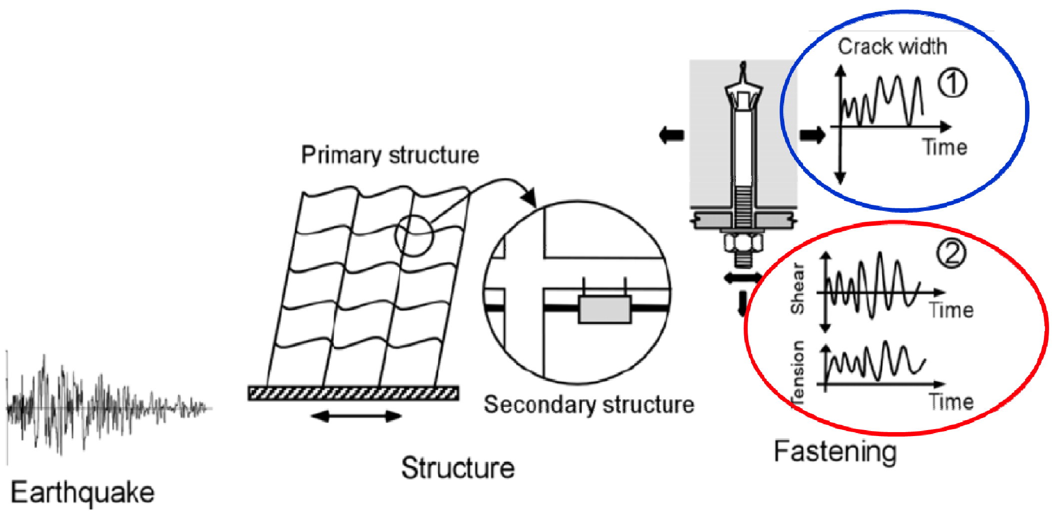


Figure 1: Scheme of the analysis process of a primary structure with components: simulation of response time-histories at fastenings due to earthquake excitation.

2.3 Determination of anchor displacements from time-histories

The present analysis investigates displacements of undercut anchors which occur under earthquake excitation in direction of the fastened components. The computational model is based on the assumption that these anchor displacements accumulate in case of an opened crack in the anchor hole and a tensile force acting on the fastening at the same time. In the serviceability limit state the crack width in reinforced concrete primarily calculates from the appropriate bending moments. The relevant bending moment M_{max} at the anchor location is determined from the maximum values of the "*MIN*"- and "*MAX*"-models as follows:

$$M_{max} = \max\left(\widetilde{M}_{max}^{MAX}, \widetilde{M}_{max}^{MIN}\right) . \qquad (1)$$

The maximum values of bending moments of the "*MIN*"- and "*MAX*"-models are median values derived from m simulations with different ground acceleration functions.

$$\widetilde{M}_{max}^{MIN} = \text{median}_{n=1}^{m}\left(M_{n,\,max}^{MIN}\right) \quad ; \quad \widetilde{M}_{max}^{MAX} = \text{median}_{n=1}^{m}\left(M_{n,\,max}^{MAX}\right) \qquad (2)$$

In each case $M_{n,\,max}^{MIN}$ and $M_{n,\,max}^{MAX}$ at the anchor location are maximum bending moments in the "*MIN*"- and "*MAX*"-model, respectively, caused by the n-th excitation function.

It is assumed that the crack width in a reinforced concrete part behaves proportional to the bending moment at the same location. Consequently, the time response of crack width $w_n(t)$ caused by excitation n calculates from the time response of the associated bending moment by normalization with the relevant maximum moment M_{max} and scaling with a corresponding crack width w_{max}.

$$w_n(t) \cong w_{max}\ \frac{M_n(t)}{M_{max}} \qquad (3)$$

Analogously to the relevant bending moment, the design relevant tensile force in a fastening calculates as follows:

$$N_{max} = \frac{1}{\Omega} \cdot \max\left(\widetilde{N}_{max}^{MAX}, \widetilde{N}_{max}^{MIN}\right) \quad ; \quad 0 \le \Omega \le 1 \qquad (4)$$

$$\widetilde{N}_{max}^{MIN} = \text{median}_{n=1}^{m}\left(N_{n,\,max}^{MIN}\right) \quad ; \quad \widetilde{N}_{max}^{MAX} = \text{median}_{n=1}^{m}\left(N_{n,\,max}^{MAX}\right) . \qquad (5)$$

In that Equation $N_{n,\,max}^{MIN}$ and $N_{n,\,max}^{MAX}$ are the maximum tensile forces in the fastening caused by excitation n of the "*MIN*"- and "*MAX*"-model, respectively. The factor $1/\Omega$ captures the partition of the total tensile force N_{max} in a first part $\Omega \cdot N_{max}$ due to earthquake excitation and a second part $(1\text{-}\Omega) \cdot N_{max}$ due to quasi constant loading. The time response of anchor force $F_n(t)$ caused by excitation n calculates from the time response of the force $N_n(t)$ in the fastening added to the quasi constant anchor load, and by normalization of this summed force with N_{max} and scaling with the maximal allowed anchor force F_p.

$$F_n(t) = F_p \; \frac{N_n(t) + (1-\Omega) \cdot N_{max}}{N_{max}} \tag{6}$$

The anchor displacements can be determined based on the time response of crack width $w_n(t)$ and the time response of anchor froce $F_n(t)$ for the "*MIN*"- and "*MAX*"-model and in each case for all excitations *n*. For that purpose the two time series are overlaid and evaluated at discrete points in time (Figure 2). The possible anchor displacement $D_{n,z}(t_i)$ at time t_i due to excitation *n* calculates:

$$D_{n,z}(t_i) = \begin{Bmatrix} 0 & \text{wenn} \;\; w_n(t_i) \le 0 \;\; \text{oder} \;\; F_n(t_i) \le 0 \\ \mathrm{C} \cdot w_n(t_i)^{\alpha} \cdot F_n(t_i)^{\beta} & \text{wenn} \;\; w_n(t_i) > 0 \;\; \text{und} \;\; F_n(t_i) > 0 \end{Bmatrix} . \tag{7}$$

This formula is derived from evaluated cyclic pull out tests of anchors, which are carried out according to the DIBt-guideline (2010) as explained in Section 2.1. From these pull out tests the constant parameter C as well as exponents $\propto$ and β can be derived for various types of undercut anchors.

The relevant anchor displacement $D_{n,z}$ of a crack opening cycle *z* is defined by the maximum value of all possible anchor displacements within that cycle.

$$D_{n,z} = \max{}_{i=1}^{n} \left(D_{n,z}(t_i) \right) \tag{8}$$

The total anchor displacement D_n caused by earthquake excitation *n* results from the summation of relevant displacements $D_{n,z}$ over all crack opening cycles *z*.

$$D_n = \textstyle\sum_{z=1}^{m} D_{n,z} \tag{9}$$

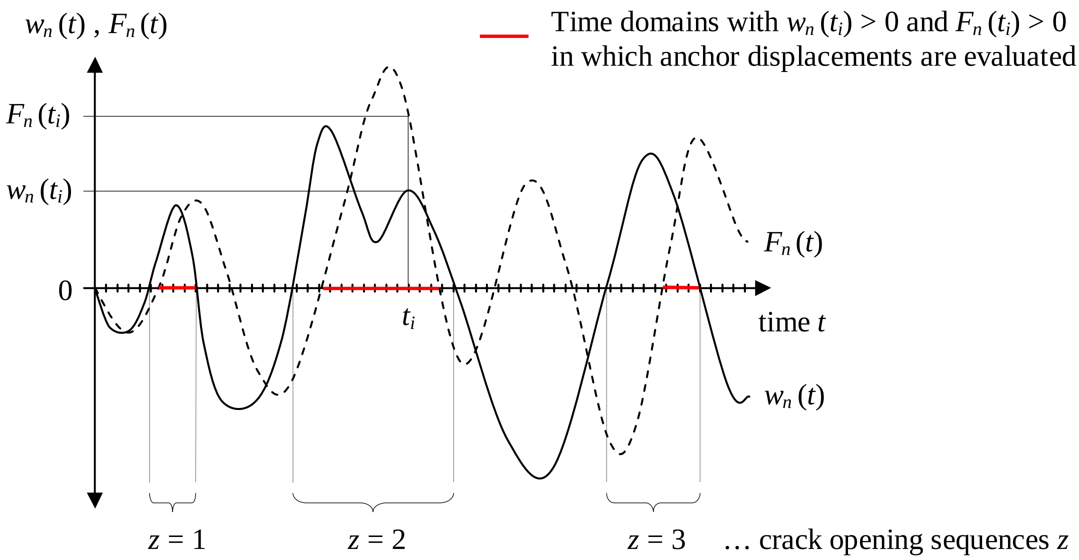


Figure 2: Overlay of crack opening time-history $w_n(t)$ and anchor loading time-history $F_n(t)$.

The final calculative value of anchor displacement under earthquake excitation is the maximum value of the medians of the "*MIN*"- and "*MAX*"-model added to the initial slip D_0 of the anchor.

$$D = D_0 + \max\left(\text{median}_{n=1}^{m}\left(D_n^{MIN}\right), \text{median}_{n=1}^{m}\left(D_n^{MAX}\right)\right) \tag{10}$$

3 Probabilistic concept

3.1 General

In the context of a probabilistic analysis, those variables, which mainly influence the anchor displacements, are defined stochastically. First of all, the relevant influencing variables are determined from the entire set of input variables by a sensitivity analysis. This leads to a sensible limited set of variables for the subsequent stochastic analysis of anchor displacements. Then, probability distribution functions are assigned to these variables and the functions are parameterized based on available data records.

Within a Monte-Carlo-Simulation the relevant influencing variables are varied independently of each other and according to their predetermined realistic distribution. The corresponding anchor displacements are simulated for each parameter set of the variation. At last, the Monte-Carlo-Simulation yields a distribution of simulated anchor displacements depending on the variation of input variables. The simulated distribution of the output variable enables for a probabilistic evaluation of anchor displacements concerning exceeding probabilities of design limits.

3.2 Influencing variables

In the following all influencing variables are specified, which directly or indirectly affect the calculation of anchor displacements. From this entire set of variables the relevant influencing variables are determined by a sensitivity analysis.

- The soil consistency is important for the soil-structure interaction and therewith it influences the internal force variables of the dynamic analysis.
- The eigenfrequency of the construction part (wall, slab) is a decisive property concerning its response in case of dynamic excitation.
- The eingenfrequency of the assembled component characterizes its dynamic behavior and affects the anchor force in case of dynamic excitation.
- The crack width in concrete structures depends on the maximal crack width in the construction part and its utilization level within the serviceability limit state. The anchor displacements increase with increasing crack width in the anchor hole.
- The tensile force in an anchor depends on the approved strength of the anchor and its utilization level. Consequently, the anchor type indirectly influences the resulting anchor displacement under earthquake excitation.

The anchor displacement increases with increasing anchor tensile force in case of an opened crack in the anchor hole.

- The ratio between anchor load due to earthquake excitation and entire anchor load influences the calculated anchor displacement according to Equations (4) and (6).
- The concrete strength affects the resistance of the construction part against the pullout of anchors. The concrete strength is considered indirectly in Equation (7) by the experimentally determined constants C, α and β.

The number of anchors per anchor plate makes an impact on the anchor displacements as explained in Section 2.1. However, this effect is not yet considered within the presented probabilistic concept.

3.3 Monte-Carlo-Simulation

Plain Monte-Carlo-Simulations require an enormous number of parameter sets, so-called samples, to ensure sufficient quality in the subsequent probabilistic evaluation. The number of necessary samples increases vastly with increasing number of stochastic input variables. For this reason variance reduction techniques are often applied in stochastic analyses to ensure a high quality in the probabilistic evaluation with a limited number of simulated samples. An efficient variance reduction technique is the Latin Hypercube Sampling, which is applied for the Monte-Carlo-Simulation in the example of Section 4.

The Latin Hypercube Sampling is based on a subdivision of the range of each stochastic variable in N intervals of equal probability. In doing so, N is the number of samples which have to be generated. From each interval one value of the stochastic variable is extracted randomly so that the range of each variable is entirely covered considering the corresponding probability distribution. The extracted values of each stochastic variable are randomly combined with the values of all other variables. A detailed description of the Latin Hypercube Sampling can be read, for instance, in the documentation of the software optiSLang (2011), which was applied for stochastic simulations within the following example.

4 Example

4.1 Model description

The probabilistic concept for the determination and evaluation of anchor displacements under earthquake excitation is demonstrated on the example of a typical stiff building of a nuclear power plant. The beam model, which is applied for the dynamic analysis of the building, is illustrated in Figure 3. The equivalent stiffness of braced structural parts is assigned story-wise to beams in the model. Masses of stories are concentrated in positions of floor slabs. The model is discretized by finite 2D beam elements with nodal degrees of freedom in the vertical and in the resulting horizontal direction. Exemplary, the probabilistic concept is applied to evaluate anchor displacements in walls. If required, the simulation can be extended also to evaluate anchor displacements in slabs.

Basically, the dynamic behavior of walls correlates with the dynamic behavior of the building in horizontal direction. Since the simulated time response of internal forces is normalized, the bending moments can be determined from walls with story-equivalent stiffness. Thus, the fastening of a wall component can be directly applied as a single degree of freedom system attached to an existing beam in the building model. The stiffness and masses of the building model and the attached single degree of freedom system are adjusted and varied to cover the typical spectrum of realistic eigenfrequencies.

The translational and rotational stiffness of base springs are determined from a static analysis independent on frequencies but considering the particular soil properties at the power plant location. This proceeding is based on recommendations of the guideline DGGT (2002).

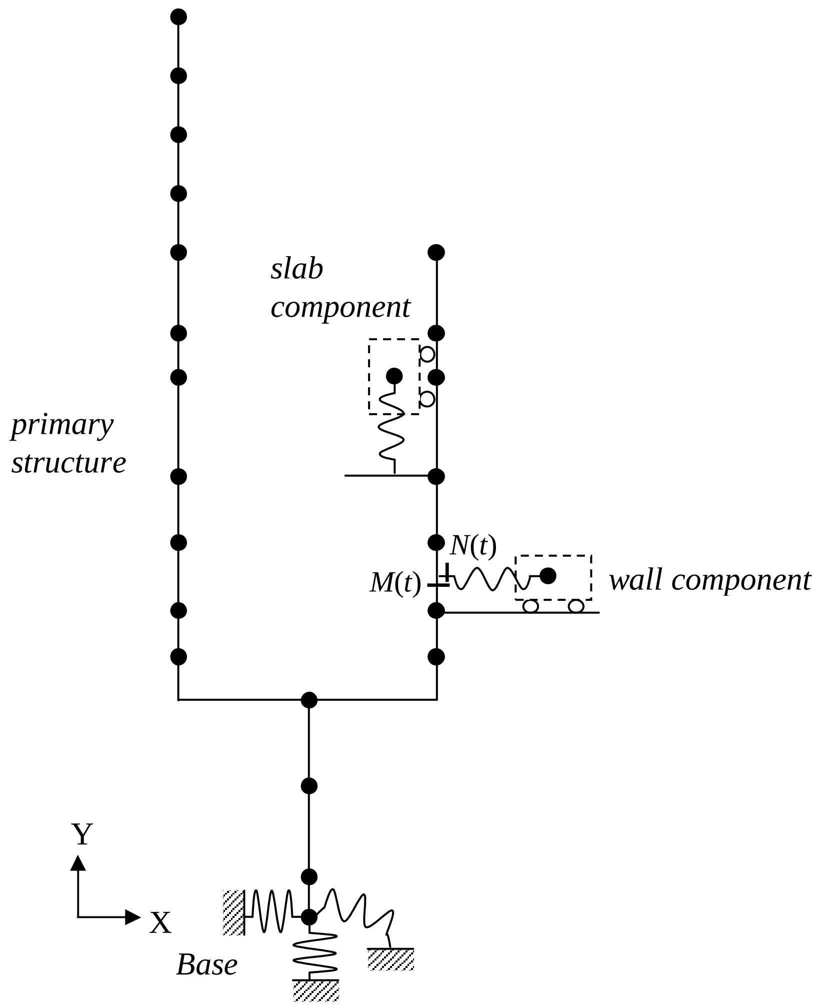


Figure 3: Typical stiff building of a nuclear power plant: structural model with fastened components at a wall and a slab respectively, and evaluation points of anchor loading time-histories.

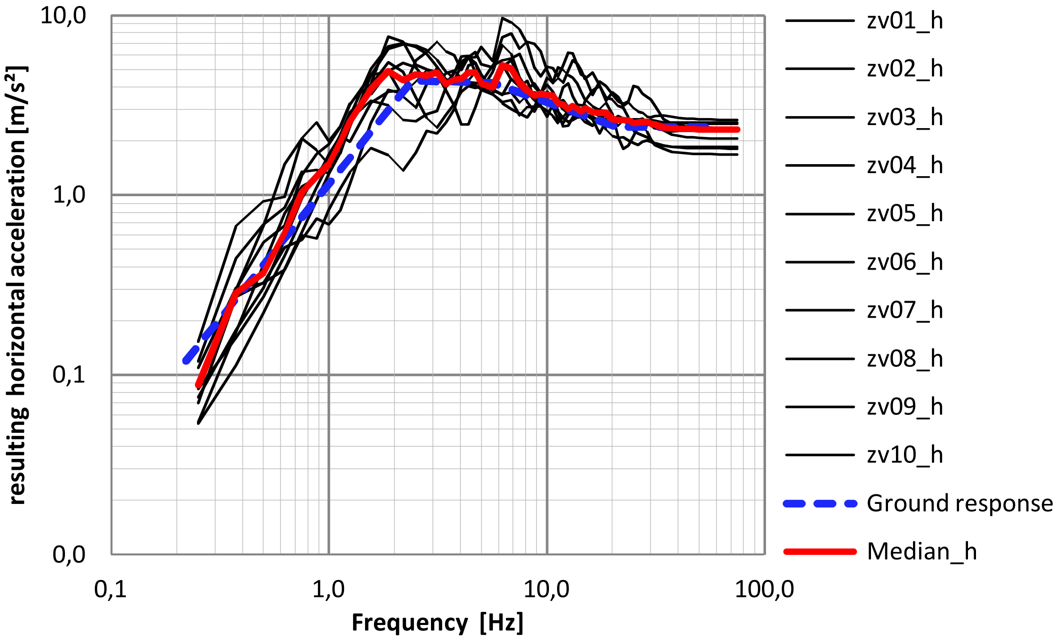


Figure 4: Ground response spectrum of the resulting horizontal acceleration compared to the median spectrum of 10 scaled natural acceleration time histories.

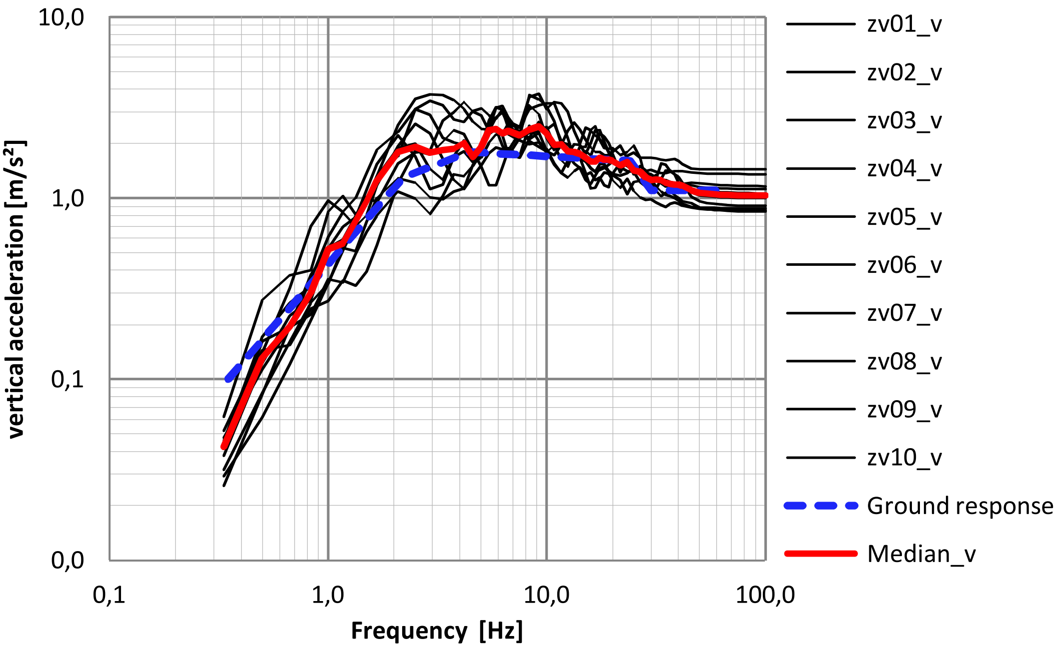


Figure 5: Ground response spectrum of the vertical acceleration compared to the median spectrum of 10 scaled natural acceleration time histories.

4.2 Time response of excitation

The simulations are realized with 10 independent natural time series applied for the horizontal excitation of the building and 10 independent natural time series applied for the vertical excitation. These time series are chosen in accordance with the geological characteristics of the ground, the maximum ground acceleration, and the ground response spectrum of the design earthquake. The natural time series are adjusted by scaling factors to cover the design spectrums of horizontal and vertical excitation of the building. Figures 4 and 5 show the computed response spectrums of the adjusted natural time series in comparison with the design spectrums of a certain place of location in Germany. These ground response spectrums define accelerations at ground level and can be used approximately for the excitation of the building.

4.3 Influencing variables and sensitivity analysis

The probabilistic analysis considers four types of undercut anchors and their real installation frequency. These four anchor types are representative for the exemplary investigated building. The considered anchor types are: Fischer FZA-K M12, Hilti HDA-T M10, Hilti HDA-T M12, and Hilti HDA-T M16.

The probabilistic analysis of anchor displacements under earthquake excitation is based on the stochastic characterization of influencing variables. In the example, the stochastic variation of influencing variables is approximately described by normal or lognormal distribution functions, respectively. Mean values, standard deviations and bounds of the stochastic variables are defined by experience or by analyzing existing data of the building. The stochastic description of the variation of maximal crack width is based on experimental data, which are documented in the bulletin for crack formation DBV (2006). Among others, these experimental data are the basis of the crack width concept of the standard DIN 1045-1. Detailed comments on this topic are also written in the bulletin DAfStb 525 (2010). According to the standard the maximal crack width in concrete structures is defined as the 95%-fractile of experimentally measured maximal crack width distributions.

Equations (3) and (6) refer to an entire utilization of the allowable maximal crack width and the allowable anchor force, respectively. In reality, the design of structural parts and anchors is usually carried out with considerably reserves. For this reason a factor is introduced in the mentioned equations to consider the utilization of structural parts and anchors. The distribution of utilization factors A_W of the allowable crack width in walls is determined from crack width calculations on the considered building. The stochastic characterization of utilization factors of the allowable anchor force is also estimated from available data of the building. Accordingly, Equations (3) and (6) are modified by the consideration of utilization factors as follows:

$$w_{n,\mathrm{W}}(t) \cong A_{\mathrm{W}}\, w_{max}\, \frac{M_{n,\mathrm{W}}(t)}{M_{max,\mathrm{W}}} \tag{11}$$

Table 1: Influencing variables and stochastic characterization

Influencing variables		Unit	Type of Distribution
factor of ground stiffness	g	[-]	normal
factor of wall stiffness	s_W	[-]	lognormal
eigenfrequency of wall component	f_{WK}	[Hz]	lognormal
maximal crack width	w_{max}	[mm]	lognormal
utilization factor of walls	A_W	[-]	lognormal
utilization factor of anchors	A_F	[-]	lognormal
fraction of anchor load caused by earthquake excitation	Ω_W	[-]	normal
compressive strength of concrete	f_c	not varied in the example: f_c = 38 N/mm²	
number of anchors per anchor plate	n_{anchor}	not considered in the example: n_{anchor} = 1	

$$F_{n,\mathrm{W}}(t) = A_\mathrm{F}\, F_p\, \frac{N_{n,\mathrm{W}}(t) + (1-\Omega_\mathrm{W}) \cdot N_{max,\mathrm{W}}}{N_{max,\mathrm{W}}} \,. \tag{12}$$

Table 1 contains an overview of all influencing variables in the probabilistic analysis and the defined distribution types. Until now, the present example does not include the variation of concrete strength and the number of anchors per anchor plate. Their stochastic parameterization is planned in prospective simulations.

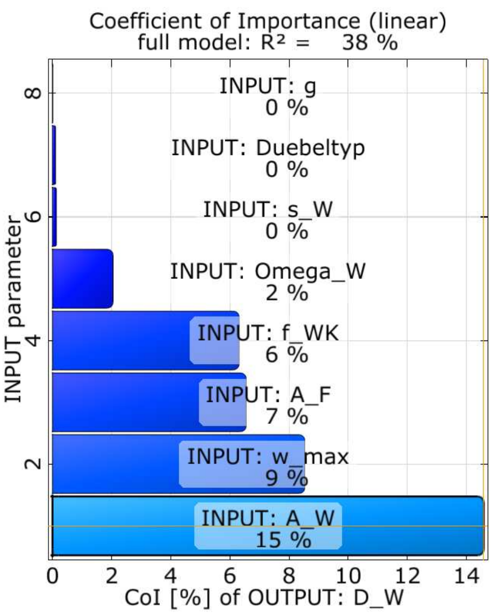
A first stochastic simulation is performed to investigate the influence of input variables on the output quantity, which is the anchor displacement in walls. Therefore, a sensitivity analysis with 100 samples is performed using the software optiSLang (2011). The samples are generated according to the Latin Hypercube Method as explained in Section 3.3. On the one hand the sensitivity analysis is aimed at the understanding and control of the computational model. On the other hand a reasonable interpretation of the sensitivity analysis helps to reduce the number of stochastic input variables to a limited set of relevant influencing variables, which are stochastically described in a subsequent reliability analysis. Defining only the relevant influencing variables as stochastic variables leads to a reduced number of required samples in a reliability analysis to gain acceptable quality of probabilistic results.

Figure 6 shows the findings of the sensitivity analysis in optiSLang. Based on the “Coefficient of Importance” only 38 % of the variation in the output quantity is explainable by variations in the input variables. The low value attributes to the high level of nonlinearities in the computational model. An improved interpretation of the sensitivity analysis is possible by introducing a “Moving Least Squares” approximation of the response surface. With that and on the basis of a metamodel the sensitivity of simulated anchor displacements is estimable by the “Coefficient of Prognosis”. By doing so, 93 % of the variation in the output quantity is explainable. As a result of the sensitivity analysis the following influencing variables are found to be of high importance concerning anchor displacements in walls:

- Utilization factor of walls: A_W
- Maximal crack width in fully utilized walls: w_{max}
- Utilization factor of anchors in walls: A_F
- Eigenfrequency of components attached to walls: f_{WK}
- Fraction of earthquake based anchor load on the total anchor load in walls: Ω_W.

The anchor type is not relevant in the event of actually considered undercut anchors. The reason is that the anchor type depending mathematical term $C \cdot F_n(t_i)^\beta$ in Equation (7) leads to similar results for all considered undercut anchors.

Those influencing variables, which define the crack width in anchor holes, have the most relevant influence on the simulated anchor displacements. Namely, these influencing parameters are the maximal crack width in fully utilized walls and their utilization factor. These two variables mainly affect the crack width $w_n(t_i)$ in Equation (7).



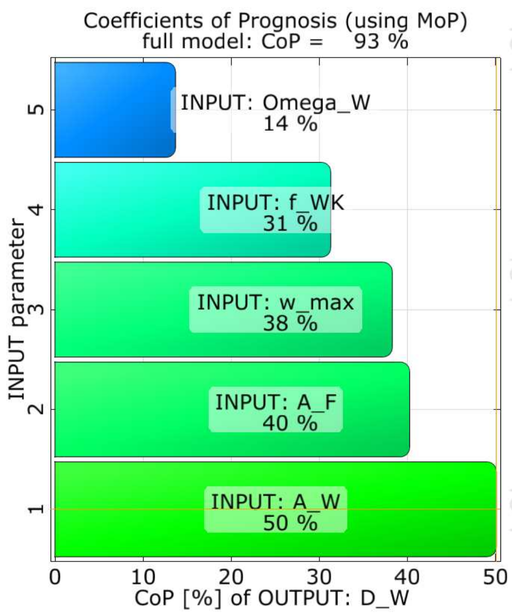


Figure 6: Evaluation of the importance of influencing variables.
Left: based on simulated displacements using the "Coefficient of Importance".
Right: based on approximated metamodel using the "Coefficient of Prognosis".

4.4 Probabilistic evaluation of anchor displacements

The evaluation of anchor displacements in walls of the exemplary investigated building is carried out on the basis of stochastic simulations with 10’000 samples using the software optiSLang (2011). The distribution of simulated anchor displacements is illustrated in the histogram of Figure 7. The mean value of all computed anchor displacements is 0.07 mm. The estimation of exceeding probabilities can be performed using an approximated Weibull distribution function, which is fitted to the discrete simulation results.

The simulated anchor displacements under earthquake excitation have to be added to a displacement caused by initial slip. The resulting anchor displacements have to be limited to 3 mm for German nuclear power plants. Based on experimental data the initial slip is appreciated to a value of D_0 = 1.0 – 1.5 mm. Considering the range of initial slip the limit value of the simulated anchor displacement under earthquake excitation reduces to G = 1.5 – 2.0 mm. The appropriate exceeding limits are determined based on the simulated discrete displacements as well as the fitted Weibull distribution function. In the example both proceedings lead to similar results. Considering the occurrence probability of the design earthquake, which is for German nuclear power plants 10^{-5} a^{-1}, the probability of undercut anchor displacements, which exceeds the explained limit range G, is very low.

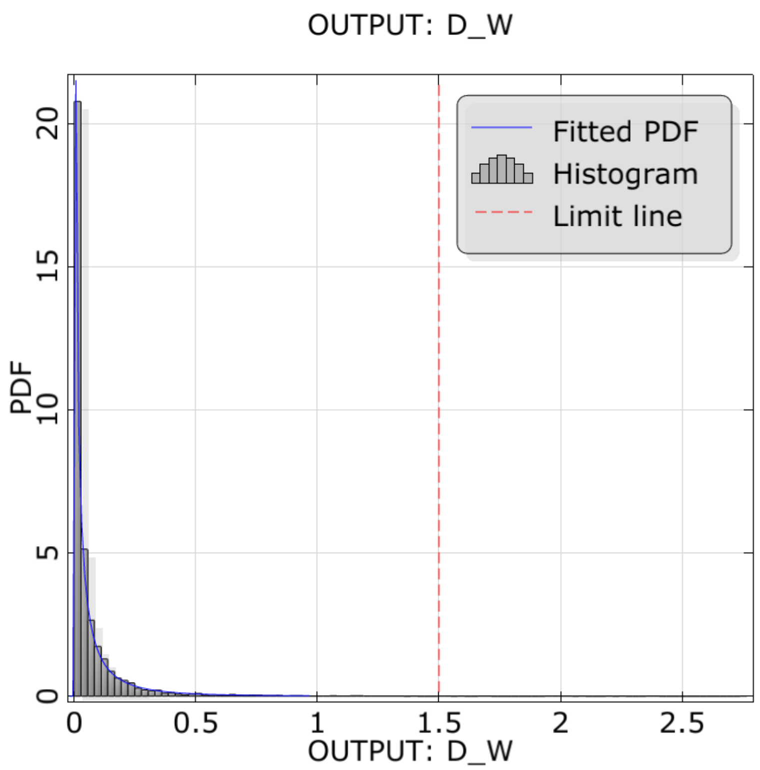


Figure 7: Histogram and fitted probability distribution function (PDF) of simulated anchor displacements in walls (D_W). The limit value is 1.5 mm.

5 Conclusions and Outlook

The probabilistic concept offers a possibility to evaluate expected anchor displacements under earthquake excitation by the definition of realistic boundary conditions. For that reason this concept is a promising alternative to the conservative determination of anchor displacements according to the deterministic concept of the DIBt-guideline (2010).

On the example of a typical stiff building of a nuclear power plant it has been shown, that the expected anchor displacements in a wall due to the design earthquake are very small. First stochastic simulations yield anchor displacements with a mean value of 0.07 mm caused by earthquake excitation added to the initial slip of about 1 mm. Consequently, the exceeding probability for a limit value of 3 mm including the initial slip is marginal. The presented results have to be considered as first estimations, since they are calculated by the application of a simplified building model. Thus, the results have to be verified by further simulations.

In prospective probabilistic analyses, the stochastic variables, which were presently parameterized by experience, have to be quantified more precisely. Furthermore, it is planned to consider variations in the concrete strength as well as the effect of force distribution in anchor plates concerning their influence on anchor displacements under earthquake excitation.

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