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Seismic Risk Analysis for the Fast Breeder Prototype SNR-300 in Kalkar (FRG)

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SUMMARY

This paper summarizes the seismic part of the SNR-300 Risk Oriented Analysis [1,2]. Two different approaches were used for the seismic hazard description. In the first one, similar to the German Risk Study for PWR [3,4], the seismic input was given by a site-independent mean acceleration response spectrum and duration of strong motion prescribed for the design of the plant; the spectrum was scaled with the peak ground acceleration the probability of exceedance of which at the site Kalkar had been calculated in a former seismic hazard study[5]. For the second approach, site-and intensity- dependent mean acceleration response spectra and duration of strong motion were derived and the probability of exceedance of the site intensity was evaluated in a probabilistic seismic hazard analysis [6,7].

The seismic responses of safety related and other important buildings were calculated by time-history analyses using artificial acceleration time-histories with the given frequency content and duration of strong motion. The influence of uncertainties in dynamic soil parameters and structural modelling was assessed in parametric studies. Some important structural elements within the buildings were investigated in more detail. Their seismic performance was evaluated using ultimate limit state definitions according to the respective design codes or rotation limits for nonlinear dynamic calculations.

Conditional probabilities of failure for seismic excitation with varying peak acceleration level (first approach) or site-intensity level (second approach) were computed by the help of the well known first order reliability method. The random variations of the seismic responses due to parameter uncertainties were evaluated to be in the order of 60 % (C.O.V. , assuming lognormal distribution); the scatter of the structural performance (C.O.V. < 20 %) is a minor contributor to the overall uncertainty. Convolution of the conditional probabilities of failure of the probability of peak acceleration or site intensity falling in different ranges yields total probabilities of failure of the investigated structures.

1. Introduction

The main aim of the SNR-300 Risk Oriented Analysis [1] (see related paper by K. Köberlein [2]) was to perform a comparative safety evaluation of the SNR-300 fast breeder prototype reactor being under construction at Kalkar and a modern pressurized water reactor of the Biblis B type as investigated in the German Risk Study for PWR [3,4]. As a contribution to [1] the seismic risk due to structural failures was evaluated in [8]. Like in the German Risk Study for PWR [3,4], the following steps had to be performed:

i. Seismic hazard analysis for the site Kalkar with evaluation of the most important earthquake groupings.
ii. Definition of the seismic input parameters representative for the seismic excitation due to the earthquake groupings from i. and assessment of the random variations due to parameter uncertainties
iii. Choice of buildings which are important for the reactor safety, evaluation of appropriate vibration models including soil-structure interaction and calculation of the building and floor responses due to the excitation from ii.
iv. Choice of critical and representative structural elements within the buildings and evaluation of their responses to the floor excitations from iii.
v. Definition of ultimate limit states for the structural elements and assessment of the random variations of stress, strain or rotation limits
vi. Evaluation of conditional probabilities of failure of the structural elements for varying seismic excitation and of total probabilities of failure by convolution with the probability of the seismic excitation falling in different ranges.

The investigations according to i. and a part of ii. were performed in a separate study by Ahorner [6] (see related paper by. L. Ahorner/W. Rosenhauer [7]). The results from vi. were used in [1] as input for event tree analyses yielding the frequencies of seismic induced system failures (see paper [2]).

2. Seismic input

Two different approaches were used to define seismic input parameters for the study. The first one was chosen to be directly comparable with the German Risk Study for PWR [3,4]. The frequency content of the free field excitation was given by the median site-independent acceleration response spectrum (Housner-spectrum) prescribed in 1972 for the aseismic design of the plant buildings [9]; also the conservatively estimated duration of strong motion was taken from [9]. The probability distribution of peak ground accelerations a_o - used as scaling parameter for the spectrum shape - at the site Kalkar has been evaluated by Ahorner [5] in a former seismic risk study for the northern Rhine area. Finally, in accordance with [3,4], the coefficient of variation of the seismic responses of structures was assumed to be in the order of $V_a \cong 0.6$.

The second approach is described in more detail in [7]. Here site intensity I was the leading parameter for the seismic input description. It includes all seismological information on historic or potential sources, the respective magnitudes and focal depths and the attenuation from the source to the site. For the intensity range of most influence upon the seismic risk at Kalkar, site-dependent free field response spectra (median and standard deviation) were assessed based on a statistical evaluation of near-field data of the Friauli earthquake. In the same manner the duration of strong motion was calculated with its median and standard deviation. The standard deviation of the spectrum shapes including random variations of souce and soil was found to agree very well with the abovementioned C.O.V. $V_a \cong 0.6$.

As final seismic input for the dynamic analyses of plant buildings artificial acceleration time histories were generated to fulfill the given frequency content and duration of strong motion requirements for the horizontal and vertical excitation. Generally, one resulting horizontal and the vertical component of excitation were assumed acting simultaneously.

3. Dynamic analyses

The following buildings were chosen to be representative for the investigation of seismic induced structural failures:

- reactor building with inner and outer containment
- emergency cooling chimneys
- steam generator buildings
- switch-gear building.

These are not only safety related buildings in the official sense but - in the cases of outer containment, steam generator buildings and switch-gear building - also buildings which positively influence accident sequence and/or release category after core destruction [1,2].

Because of sufficiently stiffening concrete walls and floor slabs all buildings could be modelled as beams with equivalent sections and lumped masses. Torsional vibrations due to excentric masses were taken into account. Soil-structure interaction could be considered approximately by the help of frequency-independent damped springs representing some 100 m of relatively homogenous horizontally layered dense sands. The influence of possible unfavourable deviations of the mean soil stiffnes was accounted for by variation of the estimated mean dynamic shear modulus in the range of ± 40 %.

The dynamic analyses with the SAP IV code yielded the overall responses (vertical forces, shear forces and overturning moments) of the buildings for checking the possibility of sliding or overturning as well as the floor responses (floor time histories or floor response spectra) as input for the dynamic analyses of structural elements and components (Fig. 1). Generally, both seismic input sets from section 2 were used for different excitation levels. Due to the linear-elastic overall behaviour of the buildings, only a small influence of the different strong motion durations was found.

Detailed dynamic analyses were performed for the following structural elements:

- supporting structures of primary sodium pump, reactor tank and heat exchanger in the reactor building
- roof construction of the outer containment
- stiffening walls of the steam generator buildings
- stiffening walls of the switch-gear building.

Only the supporting structures of the heavy components in the reactor building are safety related in the official sense. They have been analysed for design by the help of linear-elastic dynamic calculations with the prescribed seismic input set (approach 1 in section 2) and a peak acceleration $a_o = 1.2$ m/s^2 of the SSE. Generally, the SSE responses were lower than the responses due to internal initiating events (primary sodium pump and heat exchanger) or due to airplane crash (reactor tank). Because of the similar eigen frequency range and safety margin the design calculations were checked only for the primary sodium pump; some separate dynamic analyses were performed using a simplified vibration model (Fig. 2a) and exciting it with the floor time histories for the supporting reactor building floor (Fig. 1). Given the same peak ground acceleration level and dynamic soil shear modulus, the load-effects at the restraint of the supporting structure agreed well. Therefore the load-effects from the design calculations for the other supporting structures fixed nearly at the same level could be modified by factors derived for the pump to take account of different seismic input sets. The mean bearing capacities of the supporting structures were evaluated to be higher than the load effects due to the defined SSE by a factor of 1.6 (pump) up to 3.1 (heat exchanger).

The roof of the outer containment is supported by the containment walls and two rows of compact r.c. columns. One row of columns has fixed bearings at the top, the other as well as the containment walls have deformable bearings. Therefore, given a seismic excitation of the roof, the total horizontal load has to be taken up primarily by the one row of columns with fixed bearrings. Besides, the roof construction has only been designed for the lower OBE. The consequence are plastic deformations at the restraint of the columns if the seismic excitation exceeds the design level by more than the safety margin. Therefore nonlinear dynamic analyses of the columns seemed to be appropriate. Because of the very simple system of the columns, a single degree of freedom model could be chosen (Fig. 3a). A horizontal force-deformation relationship was evaluated with failure of the shear reinforcement near the restraint being the most critical ultimate limit state. The floor response time histories of the respective reactor building floor (both approaches with varying amplitudes) served as seismic excitation. The mean bearing capacity of the columns was evaluated to be - in terms of peak ground acceleration - about $a_o = 0.8\ m/s^2$ in the linear-elastic case and about $a_o = 1.4\ m/s^2$ in the nonlinear case.

The stiffening walls of the steam generator buildings and the switchgear building were analysed as examples for global building failure. The ultimate limit states (tension failure of the reinforcement or compression failure of concrete) were defined according to the respective design code. Starting with the free field time history from section 2 the linear-elastic dynamic analyses resulted in the following mean bearing capacities: $a_o = 1.3\ m/s^2$ (approach 1) and $a_o = 1.85\ m/s^2$ (approach 2) for the steam generator buildings; $a_o = 2.9\ m/s^2$ and $a_o = 2.7\ m/s^2$ for the switch-gear building. In the first case, the capacity increases clearly with the smaller duration of strong motion (approach 2); in the second case the more unfavourable amplification of the free field spectrum for approach 2 dominates in the frequency range of most influence.

4. Probabilistic analyses

The probabilistic analyses by the help of the first-order second-moment reliability theory were performed analogously to [3,4]. Generally, a randomly variating global resistance R (bearing capacity) had to be convolved with a global loading S (load-effect or given ground acceleration level) which takes account of seismic as well as normal operating loads. The distribution parameters of the global resistance R were assumed to agree approximately with those of the dominating material strength (e.g. reinforcing steel or concrete. They are well known from quality assurance tests and have only to be somewhat increased because of resistance model uncertainties. Lognormal distributions were presupposed for physical reasons. The random variations of the structural responses given an earthquake of peak acceleration a_o at the site or site intensity I, include contributions from 1) uncertain focal and attenuation parameters, 2) soil parameters and uncertainties in soil-structure interaction, 3) uncertainties in structural modelling from which the first group is clearly dominating. The overall coefficient of variation has been estimated in section 2 - on the basis of the experiences in [3,4] and of the statistical evaluation of site-dependent spectra - to be in order of $V_a = 0.6$ (for acceleration response); here also lognormal distribution was assumed. This C.O.V seems to be acceptable in the case of proper modelling of structure and soil-structure interaction; otherwise it should be increased. Compared with the variations of the seismic portion of the global loading S, the scatter of the other portions, primarily self weight or quasistationary live loads, is approximateley neglectable, exept for very small seismic load portions (see paper by G. König/D. Hosser [11]).

Given the lognormal distributions of R and S, simple closed form solutions for the conditional probability of failure of a structural element or system p_{f,a_o} (given an earthquake with peak acceleration a_o - approach 1) and $p_{f,I}$ (given site intensity I - approach 2) were possible. The conditional probabilities of failure (fragility curves) for the support of sodium pump and the r.c. columns of the reactor containment roof are shown in the Figures 2b and 3b. They take only account of the failures with dominating seismic portions of loading S; therefore the lower part of the curves (possible failure due to operating and live loads) is dotted. The different curves belong to different seismic input sets (approach 1 or 2) and linear-elastic or nonlinear analysis ; for approach 2 the intensity I has been converted into the zero-period acceleration of the respective site-dependent spectrum (e.g. $I = 7 \rightarrow a_o = 1.2\ m/s^2$). The difference between site-independent and site-dependent seismic input is relatively small, but there is a clear probability decrease for nonlinear compared with linear-elastic dynamic analysis.

Finally, the convolution of the conditional probabilities of failure p_{f,a_o} or $p_{f,I}$ and the probabilities $P(a_o)$ of peak ground acceleration falling in the range $a_o \pm \Delta$ respectively $P(I)$ of site intensity falling in the range $I \pm \Delta$ results in total probabilities of failure p_f due to earthquake for the investigated structural elements. All (best estimate) results are summarized in Tab. 1. The results of approach 2 are generally more favourable, except for linear-elastically analysed containment roof columns and the stiffening wall of the switch-gear building where only the very low excitation level ($a_o \leq 0.8\ m/s^2$) is of interest for p_f.

5. Discussion and conclusions

Within the scope of the SNR-300 Risk Oriented Analyses [1,2] the procedure of seismic risk evaluation [8] had to be comparable to that of the German Risk Study for PWR [3,4]. Therefore like in [3,4] a seismic input set according to the design analyses [5] was assumed and the peak ground acceleration a_o was used as the only measure for the seismic excitation level; frequency content and duration of strong shaking were taken as constant and site-independent (approach 1). Probabilities of peak acceleration exceeding a_o at the site Kalkar were taken from a former seismic risk analysis for the region [5]. Beside this, a site-dependent method (approach 2) started with a detailed probabilistic seismic hazard analysis [7,8] resulting in annual frequencies of different earthquake groupings (magnitude/focal distance combinations) contributing to the site intensity I. For the grouping of most influence more realistic site-dependent free field response spectra and durations of strong motion were evaluated on the basis of the Friauli near field data. Besides, approach 2 was considered as a test of feasibility for the beginning Phase B of the German Risk Study [3]. This test is felt to be successfully finished with reasonable results and -perhaps- a bit narrower error bounds.

In evaluating the ultimate limit states, structural elements were generally taken as designed, built in and loaded during plant operation. "Failure" was defined as exceeding strain limits (generally those defined in design codes) or rotation limits (for nonlinear dynamic analysis). The seismic excitation level - peak ground acceleration a_o or site intensity I - was increased until the design limits were exceeded for the first time; the excitation level just reached was the seismic load bearing capacity. The distribution assumptions describing the random variations of relevant parameters were also based on the experience in [3,4] but - referring to the overall seismic response - with new confirmation from [7,8].

Finally, the method of calculating the conditional probabilities of failure by the help of simple closed form convolution of lognormal distributions seems further to be justified because of two parameters (seismic load portion and material strength) clearly dominate within the overall uncertainty and both can, by physical reasons, be assumed lognormally distributed. The conditional probabilities of failure of the structural elements served in [1,2] as input for event tree analyses of possible seismic induced releases resulting in a 50 percent contribution to the most severe release category.

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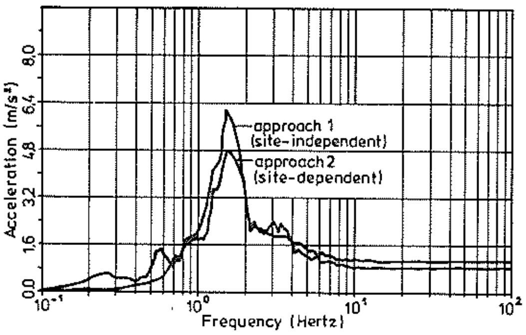

Fig. 1 Floor response spectra for the primary sodium pump supporting structure with two seismic input sets (damping D = 3 %)

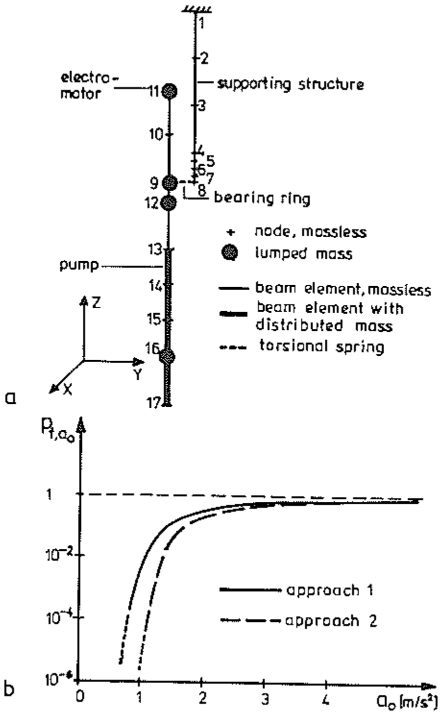

Fig. 3 Simplified vibration model of the containment roof columns and conditional probability of failure of the columns for two seismic input sets

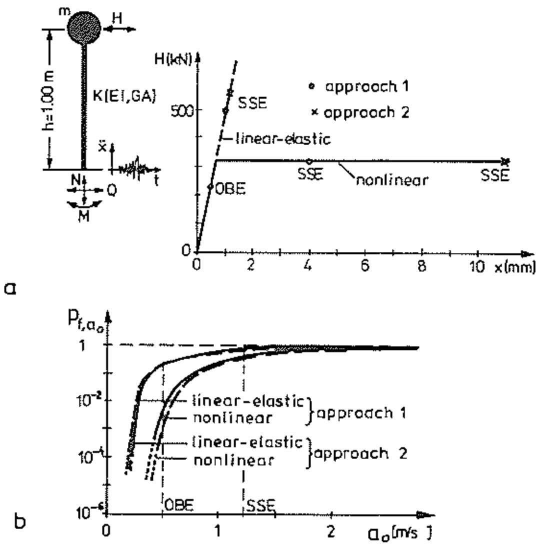

Fig. 2 Simplified vibration model of the primary sodium pump and conditional probability of failure of the supporting structure for two seismic input sets

Tab. 1 Total probabilities of failure due to earthquake for the investigated structural elements and two seismic input sets

Structural element	seismic input	
	approach 1	approach 2
primary sodium pump support	$2.4 \cdot 10^{-6}$	$1.9 \cdot 10^{-7}$
reactor tank horizontal support	$5.5 \cdot 10^{-7}$	-
heat exchanger support	$3.0 \cdot 10^{-7}$	-
containment roof columns		
linear-elastic	$1.8 \cdot 10^{-4}$	$2.4 \cdot 10^{-4}$
nonlinear	$2.9 \cdot 10^{-5}$	$9.3 \cdot 10^{-6}$
steam generator building wall	$5.6 \cdot 10^{-5}$	$3.9 \cdot 10^{-6}$
switch-gear building wall	$9.2 \cdot 10^{-6}$	$1.1 \cdot 10^{-5}$