SEISMIC ROBUSTNESS
OF A FILTERED CONTAINMENT VENTING SYSTEM

Stephan Kranz¹, Andreas Strohm¹, Thoralf Gocht¹, Hamid Sadegh-Azar², and Thomas Schubert³

¹ EnBW Kernkraft GmbH, Germany
² Universität Kaiserslautern, Germany
³ HOCHTIEF IKS, Germany

ABSTRACT

After the Fukushima accident, stress tests were performed both on national and European level in Germany. One of the subjects subsequent to these stress tests is the performance of the filtered containment venting system (FCVS). This paper demonstrates the performed robustness analysis aiming at clarifying, whether the integrity of the FCVS is assured during and after an earthquake at an intensity of the Design Base Earthquake (DBE).

First, an overview of the given framework and the methodology for the assessment of seismic robustness of structures, systems, and components (SSCs) will be presented. The formal framework and the adaptation of the methodology to the FCVS are described in a general way. The first investigated subject deals with the general and overall seismic design process, wherein the various design steps are used as the basis for the subsequent derivation of safety margins. An exemplification of robustness will follow to contrast different methods for assessing robustness and to reveal their pros and cons. This provides the basis for the choice of the used method which will be explained in more detail. At the end of the general description, all relevant factors influencing the determination of safety margins will be discussed.

In the second part, the determination of the FCVS’s safety margins will be presented. Herein, the methodology to evaluate the relevant safety capacity factors will be specified. First of all, the characteristics of the underlying DBE will be shown. Then, the investigated key SSCs, e.g. piping systems, support constructions, anchorage, etc. will be introduced. Further, the verification of the analysis results, by comparison with empirical values and other investigations, as well as their interpretation, will be presented. Finally, the robustness of the FCVS against DBE will be shown.

INTRODUCTION

After the Fukushima accident, stress tests were performed both on national and European level in Germany. One of the subjects subsequent to these stress tests is the performance of the FCVS. By means of a robustness analysis the assuring of the FCVS’s integrity during and after an earthquake at an intensity of the DBE, which is used in Germany as the internationally known Safe Shutdown Earthquake (SSE), had to be verified.

FRAMEWORK AND METHODOLOGY FOR THE ASSESSMENT OF SEISMIC ROBUSTNESS OF SSCS

General And Overall Seismic Design Process

The full chain of analysis and design from the source of the earthquake through the soil into the structure and to the component is sub-divided into graduated calculation steps (Figure 1).
Each step of the calculation is performed separately. Possible variations, uncertainties, and results of each calculation step are enveloped. In Germany, the whole design and analysis process is officially regulated by the KTA (2012).

Earthquake design, with many interfaces in the full computation chain and the variations/uncertainties that need to be enveloped, covered, and assumed as worst-case, is by nature conservative. Furthermore, there are generally additional conservative assumptions and approaches in each individual calculation step, which are not explicitly quantified or considered in the conventional design process, but may be activated in a robustness analysis or seismic margin assessment (SMA). These conservative assumptions and approaches and the resulting safety reserves are summarized below.

- **Seismological Assumptions**: The engineering characteristics for earthquake excitation are usually based on statistical analysis or representative seismic registrations. Usually, in the design seismic load characteristics variations, scatter, uncertainty, and unknown soil and source parameters are covered broadly. This usually leads to an enveloped and broadened response spectrum.
- **Deconvolution**: Extremes, i.e. maxima and minima, of soil characteristics are included in the deconvolution. The results are enveloped conservatively.
- **Soil-structure interaction (SSI)**: When modal methods are used in SSI analysis, critical damping from soil hysteresis and energy dissipation (radiation damping) is limited to 15% (horizontal and rotary vibration) and 30% (vertical vibration), respectively. These reserves can only be activated and quantified by SSI-analysis in a frequency range. In addition, the soil characteristics are varied broadly. The results are enveloped and cover all extreme values.
- **For large rigid structures** (like those commonly found in NPPs) the excitation of the foundation varies significantly from that of the adjacent free field. Two basic effects can be distinguished and generally lead to a reduction of seismic excitation: foundation averaging through the rigid base slab and lateral bracing by embedding.
- **Analysis and design**: The sources of conservativeness in this part can be divided into three main categories:
  a) structures are able to absorb large amounts of seismic energy through nonlinear behaviour;
b) the design specifications themselves contain safety factors which are always applied;
c) design is done with the characteristic values of material properties. These correspond to 95% fractiles and are, therefore, conservative.

- Building response spectra (floor response spectra): building response spectra are generally broadened, smoothed out and enveloped storey by storey.

Some of these safety reserves will be quantified in this paper using the FCVS as an example.

**Robustness Analysis Methodology**

Two methodologies have been proposed for evaluating the seismic safety of existing nuclear installations in IAEA (2009): the seismic margin assessment (SMA) and the seismic probabilistic risk assessment (SPRA) methodology. Variations of these approaches or alternative approaches may be demonstrated to be acceptable also. The methodologies are intended for evaluating and quantifying the seismic capacity of an existing installation according to the current as-built conditions. It is therefore important to use realistic and best estimate values for the as-built condition of the SSCs and not to introduce safety factors that may unnecessarily bias the results.

The purpose of a SPRA is to determine the probability distribution of the frequency of occurrence of exceeding various damage states or performance limits due to the potential effects of earthquakes. For the SPRA, fragility functions of the selected SSCs in the event trees and in the fault trees are needed. In contrast to a deterministic analysis that considers single-parameter values for seismic-induced forces and capacities, SPRA considers the total variability in seismic input, structure response, and material capacity variables. In simple terms, SPRA is the formal process in which the randomness and uncertainty in various physical variables are propagated through an engineering model leading to a probability distribution of frequency of occurrence of failure or other damage states. Seismic risk analysis, which is one of the facets of a SPRA, can be performed for many different reasons. It can be used to compute the frequency of occurrence of failure due to seismic effects in order to compare these to similar results for other hazards. It is a useful tool to identify weak links in a system or facility. In this context, it can guide the efficient allocation of funds to strengthen or modify an existing system. It also can be used as part of the design process to size members to comply with a given performance standard. A SPRA requires a probabilistic seismic hazard analysis. The seismic hazard analysis gives the relationship between seismic intensity (SI) and the corresponding probability of exceedance.

A Monte Carlo Analysis can be used to estimate the probabilistic floor response spectra and can also be applied to the whole SPRA. When a Monte Carlo Analysis is conducted for the development of response spectra or structural loads, all the important variables that affect the structural response have to be included. Statistics are then applied to the resulting response spectra in order to define median and e.g. the 84\textsuperscript{th}-percentile response spectra. The variation in spectral shape is simulated by utilizing a number of natural and synthetic time histories with median and 84\textsuperscript{th}-percentile response spectra ordinates that match the median and 84\textsuperscript{th}-percentile ground motion spectra. In developing the time histories, the response spectra may be first modified to incorporate ground motion incoherence (GMI) effects and high frequency spectral reduction to account for limited ductility of components. Other variables included in the probabilistic analysis are structural stiffness, structural damping, soil stiffness, and soil damping. The method delivers very precise fragility estimations, but requires complex analysis and an enormous effort.

On the other hand, the emphasis of IAEA (2009) is rather on pragmatic evaluations than using extensive complex analyses. E.g. only limited non-linear analyses of relatively simple structural models (provided that they are used with care) are recommended. Detailed, sophisticated non-linear analyses are generally not required in the usual practice.
In the simpler SMA capacity evaluations of SSCs are made in terms of so-called “high-confidence-of-low-probability-of-failure” (HCLPF) values. In probabilistic terms, the HCLPF capacity of a SSC is the earthquake motion level at which there is a high confidence (about 95%) of a low (5%) probability of failure. Although defined conceptually in a probabilistic sense, HCLPF values can also be calculated by a deterministic method. It is called the conservative deterministic failure margin (CDFM) method. This method is based on codes and guidelines developed in the US and will not be used here.

Relevant Factors Influencing The Determination Of Safety Margins

The seismic fragility of a SSC is defined as the conditional probability of its failure (or exceeding a given damage state) at a given seismic parameter (i.e., PGA or $S_a$ at different frequencies).

![Figure 2: Typical seismic fragility curves acc. to EPRI (2002)](image)

Typical seismic fragility curves are given in Figure 2. These are developed using plant design information and realistic response analysis. The databases used for fragility analysis include simulations, earthquake experience data, generic equipment ruggedness spectra, and fragility test results.

The ground acceleration capacity is a random variable that can be described completely by its probability distribution. However, there is uncertainty in the estimation of the parameters of this distribution, the exact shape of this distribution, and in the appropriate failure model for the structural or mechanical component. For any postulated failure mode and set of parameter values describing the ground acceleration capacity and shape of the probability distribution, a fragility curve depicting the conditional probability of failure as a function of PGA can be obtained (Figure 2).

At any acceleration value, the component fragility (i.e., conditional probability of failure) varies from 0 to 1; this variation is represented by a subjective probability distribution. On this distribution we can find a fragility value (say, 5%) that corresponds to the cumulative subjective probability of 95%. We have 95% cumulative subjective probability (confidence) that the fragility (failure or exceeding probability) is less than 5%. On the high confidence curve, we can locate the fragility value of 5%; the acceleration corresponding to this fragility on the high confidence curve is the so-called HCLPF capacity of the component. Development of the family of fragility curves using different failure models and parameters for a large number of components in a SPRRA is impractical if it is done as described above. Hence, a simple model for the fragility was proposed. In the following section this fragility model is described.

The entire family of fragility curves for an element corresponding to a particular failure mode can be expressed in terms of the best estimate of the median ground acceleration capacity, $A_m$, and two random variables. Thus, the ground acceleration capacity, $A$, is given by:

\[ A = A_m \cdot e_R \cdot e_U \] (1)
in which $e_R$ and $e_U$ are random variables with median values of 1.0, representing the inherent randomness about the median ($R$) and the uncertainty in the median value ($U$), respectively. The state of practice underlines that the probability distributions for capacities are assumed to be lognormal. Therefore, we assume that both $e_R$ and $e_U$ are lognormally distributed with logarithmic standard deviations, $\beta_R$ and $\beta_U$, respectively. The formulation for fragility given by Eq. (1) and the assumption of a lognormal distribution allow easy development of the family of fragility curves that appropriately represent fragility uncertainty.

With perfect knowledge of the failure mode and parameters describing the ground acceleration capacity (i.e., only accounting for the random variability, $\beta_R$), the conditional probability of failure, $f_0$, for a given PGA level, $a$, is given by:

$$f_0 = \Phi\left[\ln\left(\frac{a}{A_m}\right) \cdot \frac{1}{\beta_R}\right]$$  \hspace{1cm} (2)

where $\Phi[\cdot\cdot\cdot]$ is the standard Gaussian cumulative distribution of the term in brackets. The relationship between $f_0$ and $a$ is the median fragility curve plotted in Figure 2 for a component with a median ground acceleration capacity $A_m = 0.87g$ and $\beta_R = 0.25$. For the median conditional probability of failure range of 5% to 95%, the ground acceleration capacity would range from $A_m \cdot \exp(-1.65 \cdot \beta_R)$ to $A_m \cdot \exp(+1.65 \cdot \beta_R)$, i.e., 0.58g to 1.31g as shown in Figure 2.

When the modelling uncertainty $\beta_U$ is included, the fragility becomes a random variable (uncertain). At each acceleration value, the fragility $f$ can be represented by a subjective probability density function. The subjective probability, $Q$ (also known as “confidence”) of not exceeding a fragility $f'$ is related to $f$ by:

$$f' = \Phi\left[\left\{\ln\left(\frac{a}{A_m}\right) + \beta_U \cdot \Phi^{-1}(Q)\right\} \cdot \frac{1}{\beta_R}\right]$$  \hspace{1cm} (3)

where: $Q = P[f < f'|a]$; i.e., the subjective probability that the conditional probability of failure, $f$, is less than $f'$ for a PGA $a$, and $\Phi^{-1}[\cdot\cdot\cdot]$ is the inverse of the standard Gaussian cumulative distribution of the term in brackets.

In estimating fragility parameters, it is convenient to work in terms of an intermediate random variable called the factor of safety. The factor of safety, $F$, on ground acceleration capacity, $A$, above a reference level earthquake specified for design; e.g., the DBE level, $A_{DBE}$, is defined as follows:

$$A = F \cdot A_{DBE} \rightarrow F = \frac{Actual\ seismic\ capacity\ of\ element}{Actual\ response\ due\ to\ DBE}$$  \hspace{1cm} (4)

The median factor of safety, $F_m$, can be directly related to the median ground acceleration capacity, $A_m$, as:

$$F_m = \frac{A_m}{A_{DBE}}$$  \hspace{1cm} (5)

The logarithmic standard deviations of $F$, representing inherent randomness and uncertainty, are identical to those for the ground acceleration capacity $A$.

The median safety reserve factor and the corresponding logarithmic standard deviations can be split into three parts,

$$F_m = F_C \cdot F_{RE} \cdot F_{RS}$$  \hspace{1cm} (6)
The three parts are

1. Capacity factor $F_C$: The capacity factor $F_C$ for the equipment is the ratio of the acceleration level at which the equipment ceases to perform its intended function to the seismic design level. This acceleration level could correspond to a breaker tripping in switchgear, excessive deflection of the control rod drive tubes, or failure of an equipment support.

2. Equipment response factor $F_{RE}$: The equipment response factor $F_{RE}$, is the ratio of equipment response calculated in the design to the realistic equipment response; both responses being calculated for design floor spectra. $F_{RE}$ is the factor of safety inherent in the computation of equipment response.

3. Response factor of the building $F_{RS}$: This factor describes the safety reserves and conservativity of the floor response spectra of the building.

These parameters are calculated for each respective component of the FCVS.

In seismic margin studies, an index of seismic margin is the HCLPF capacity of the component. This quantity considers both the uncertainty and randomness variabilities and is the acceleration value for which the analyst has 95% confidence that the failure probability is less than 5%. For example, Figure 2 shows a HCLPF value of 0.32$g$ for a fragility description of $A_m = 0.87g$, $\beta_R = 0.25$, $\beta_U = 0.35$. That is, it is an acceleration value for the component for which we are highly confident there is only a small chance of failure given this ground acceleration level:

\[
\text{HCLPF Capacity} = A_m \cdot \exp[-1.65 \cdot (\beta_R + \beta_U)]
\]  

DETERMINATION OF THE FCVS'S SAFETY MARGINS

Overview FCVS

To prevent a possible overpressure inside the containment during a severe accident scenario, a FCVS is installed. Substantially, the system consists of a piping, in which the containment atmosphere may be transported to an aerosol and iodine filter (Venturi scrubber). The containment atmosphere will be filtered in the scrubber and after cleaning released into the environment through the stack. The Venturi scrubber is fixed on a steel construction, to which it is anchored on the roof of the nuclear auxiliary building. The supports and bearings for the piping system and the Venturi scrubber are anchored in the building structures. A principle FCVS diagram is given in Figure 3.
Within the licensing procedure severe accident scenarios are defined and there are no direct causal requirements for FCVS to be resistant to external events. In this context it can be assumed, that an external hazard, e. g. an earthquake is not occurring at the same time as a core meltdown scenario. In the SPSA a seismically induced leak in the reactor cooling system could be excluded by reasons of the design basis. Due to the empirically proven high resistance of pipes, there are not arising seismic safety requirements for the FCVS. SPSAs confirm the design of the reactor cooling system. In the case of an earthquake up to the design level the integrity and intactness of the entire pressure boundary are assured. The use of the FCVS would not be required in this case.

Technical means exist at the fourth level of safety (severe accident condition) that exceed state-of-the-art precautions to mitigate the effects of extremely rare events. In this context, FCVS were installed in the plant on the basis of an RSK (German Reactor Safety Commission) recommendation for German light water reactors. The purpose of filtered containment venting is to achieve controlled release of containment air via the Venturi scrubber to the stack in case of a severe accident with long-time build-up of pressure. The intention of this measure is a sustained pressure reduction inside the containment, so that the containment pressure could be kept below the failure limit due to the integrity of the containment. Specific requirements for the design of the FCVS due to natural external hazards, e. g. DBE, were not included into the respective conception. Nevertheless, the existing level of seismic safety for the FCVS is being evaluated as part of a SMA. Current ground motion and floor response spectra are being used as earthquake excitation.

**Characteristics Of The Underlying DBE**

In the late nineteen seventies, the DBE for the plant licensing was set at a maximum peak ground acceleration (PGA) of 210 cm/s². Linear dynamic mathematical models were applied for the seismic design of the plant, using the response spectrum method. The spectral shape of the ground motion response spectrum corresponds to the USAEC-spectrum. In recent years, a probabilistic seismic hazard analysis has been performed. The results showed, that the occurrence probability of the DBE used for design is between $10^{-5}$ and $10^{-6}$ per year. It thus conforms to the present KTA rules (KTA (2012)).

Beam models or detailed finite element models were used for the dynamic analysis. The soil structure interaction and the influences of the subsoil were included in the analysis. Seismic loads on the structures (internal forces, acceleration) were evaluated using the response spectrum method, floor response spectra with time history method, and listed for the corresponding buildings models with different damping values. These local acceleration values were the input parameters for the design of systems and components.


**Fragility Analysis**

Hereafter the main analysis steps and results of the HCPLF analysis are summarised with regard to the main system components. Firstly, to identify the critical points and components of the FCVS, all available documents of structural stability and safety were listed and reviewed. With regard to these documents the HCPLF analysis was carried out for the most critical part. The following analysis of the main piping system illustrates the way the analysis was carried out.

**Piping systems**

The FCVS under review consists of pipes with nominal diameters between 15 and 300 mm. The stress analysis for the piping system from the containment to the Venturi scrubber is separated into two subsystems. The first section mainly (from the containment up to the relief valve) consists of pipes with nominal diameters 200 and 300 mm. The second section (from the relief valve to the Venturi scrubber) and the following piping system to the stack are considered in a second separate analysis. The Venturi scrubber tank is idealised in the piping calculation as a fixed point with spring stiffnesses.

In the piping calculations, the areas with the highest stress ratio were identified. The values of permissible median tensile strength were used to assess the ultimate capacity limit, taking 1.1 times the permissible stress as the limit. For the main piping, the assumed permissible value is

\[ \sigma_{ult} = 1.1 \cdot 225 = 247.5 \text{ MPa} \]  

(9)

In the piping calculations, the stress verification was done for load cases with (L2) and without (L1) earthquake excitation. The capacity reserve \( F_S \) was identified.

\[ F_S = \frac{\sigma_{ult} - \sigma_{L1}}{\sigma_{SSE}} = \frac{(\sigma_{ult} - \sigma_{L1})}{(\sigma_{L2} - \sigma_{L1})} \]  

(10)

The result for the main piping on the basis of the stress verification is a capacity reserve of \( F_S = 4.1 \) and for the small-diameter pipings of \( F_S = 1.3 \).

As the deviation from the experimental results according to EPRI (1994) is relatively small, a \( \beta_{R,S} = 0 \) is assumed. With an assumed variation of 0.12 in the compressive strength respectively tensile strength of the steel and an assumed variation of the calculating equation of 0.11 (EPRI (1994), Tab. 3-10), the result is:

\[ \beta_{U,S} = \sqrt{0.12^2 + 0.11^2} = 0.16 \]  

(11)

The critical failure mechanism of piping systems includes exceeding the permissible yield strength of the steel. For this reason, a ductility factor can be assumed in consequence of steel plasticity. This can be regarded with a median system ductility of \( \mu = 3 \) (EPRI (1994), Tab. 3-11), damping of 2% of the load spectra and the assumption of an elasto-plastic yielding range according to EPRI (1994):

\[ F_\mu = F_{\mu,med} = 1.92 \]  

(12)

Variation values are calculated as shown below. The logarithmic standard deviation is calculated to consider the uncertainty when using uncertainties of theductility factor according to EPRI (1994), Tab. 3-11:

\[ \beta_{U,\mu} = \ln \left( \frac{F_{\mu,med}}{F_{\mu,med-1}\sigma} \right) = \ln \left( \frac{1.92}{1.73} \right) = 0.10 \]  

(13)

\[ \beta_{R,\mu} = 0.4 \cdot (0.06 + 0.03 \cdot (F_{\mu,med} - 1)) = 0.035 \]
In summary, the result for the capacity factor $F_C$ is

$$F_C = F_S \cdot F_{\mu} = 4.3 \cdot 1.92 = 7.87 \quad \text{(Main piping)}$$

$$F_C = F_S \cdot F_{\mu} = 1.3 \cdot 1.92 = 2.50 \quad \text{(Small - diameter pipings)}$$

$$(14)$$

$$\beta_{R,C} = \sqrt{\beta_{R,S}^2 + \beta_{R,\mu}^2} = \sqrt{0^2 + 0.035^2} = 0.035$$

$$\beta_{U,C} = \sqrt{\beta_{U,S}^2 + \beta_{U,\mu}^2} = \sqrt{0.16^2 + 0.10^2} = 0.19$$

$$(15)$$

Support structure for main piping including anchors

The pipe hangers comprise various steel constructions that have to be verified in detail for the certified hanger loads. The same applies to the anchors, which are mainly concrete-embedded plates or dowels.

By analogy with the procedure for piping systems, the verification of the support structures and anchors along the FCVS started with the identification of the places with the highest stress ratio. For the support construction, a reserve factor of inelastic behaviour was assumed, whereas a brittle failure mode was assumed for the anchors. The capacity factor for the support structure was determined to $F_C = 2.41$ with the uncertainty coefficients $\beta_{R,C} = 0.028$, $\beta_{U,C} = 0.17$ and for the anchors to $F_C = 2.0$, $\beta_{R,C} = 0$, $\beta_{U,C} = 0.19$.

Venturi scrubber tank including support grating

The Venturi scrubber tank is fixed to a steel grating by support brackets. The support grating is permanently anchored in the wall of the building structure.

The tank including all the necessary support structures and anchors has been verified in the base design. The analysis of the verification showed that the concrete-embedded plates on the support grating are the dominating components for the entire "Venturi scrubber" system. In all, the controlling capacity values for the support construction were $F_C = 2.12$, $\beta_{R,C} = 0.031$ and $\beta_{U,C} = 0.18$.

Equipment and building response factors

The factor representing the reserve in the response of the component is assumed to be equal at all analysis points. Hereafter some important factors and uncertainties that are included in the HCLPF analysis are listed. The damping factor is assumed to be 2% since in the structural analysis the damping is set to 0.02. According to EPRI (1994), for piping systems a median damping factor of 5% can be used.

<table>
<thead>
<tr>
<th>Equipment response factors</th>
<th>$F_{RE}$</th>
<th>$\beta_{R,RE}$</th>
<th>$\beta_{U,RE}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Absorption</td>
<td>1.58</td>
<td>0</td>
<td>0.18</td>
</tr>
<tr>
<td>Modelling</td>
<td>1.0</td>
<td>0</td>
<td>0.18</td>
</tr>
<tr>
<td>Combined eigenmodes</td>
<td>1.0</td>
<td>0.1</td>
<td>0</td>
</tr>
<tr>
<td>Combined direction of excitation</td>
<td>1.0</td>
<td>0.18</td>
<td>0</td>
</tr>
<tr>
<td>Result</td>
<td><strong>1.58</strong></td>
<td><strong>0.21</strong></td>
<td><strong>0.25</strong></td>
</tr>
</tbody>
</table>
The building response factor is assumed as $F_{RS} = 1.0$. The standard deviations result from the spectral values for the 50th- and 84th-percentiles. The difference between these values determines the $\beta_{C,RS}$-value, from which the worst-case standard deviations were estimated according to EPRI (2002) as:

$$\beta_{U,RS} = \beta_{R,RS} = \frac{1}{\sqrt{2}} \cdot \beta_{C,RS} = 0.16$$

(16)

More precise probabilistic calculations of the standard deviations for other power station sites by the authors showed results on the same scale.

Analysis of all assessment points

The investigation of components of the FCVS leads to the following conservative HCLPF values on the basis of the aforementioned reserve and scatter factors:

- $H_{CFLP_{main pipeline}} = 0.96g$
- $H_{CFLP_{small diameter pipeline}} = 0.31g$
- $H_{CFLP_{support}} = 0.25g$
- $H_{CFLP_{Venturi scrubber tank}} = 0.26g$

Overall, the result is a minimal HCLPF value of 0.25g. This is determined by one support of the piping system. It demonstrates that the failure of the anchoring is the crucial point by comparison to critical component failure. As this minimal HCLPF value is greater than the design base PGA-value, adequate seismic robustness of the FCVS against the DBE is thus verified.

CONCLUSION

After the Fukushima accident, stress tests were performed both on national and European level in Germany. One of the subjects subsequent to these stress tests is the performance of the FCVS. By means of a robustness analysis the assuring of the FCVS’s integrity during and after an earthquake at an intensity of the DBE was clarified. The analysis was performed using a fragility analysis method. The following basic insights could be derived:

- The capacity check of the piping system and its fastening elements show adequate load capacity.
- The main piping system and its fixed supports show large capacity factors due to high ductility of materials and elements whereas fastening elements and anchor bolts are critical for the verification of robustness, due to their lower capacity factors.
- The fragility analysis of the FCVS gives a positive overall result and it could be verified that the FCVS’s integrity during and after an earthquake at an intensity of the DBE is secured.

REFERENCES

KTA (2012), "KTA 2201, Design of nuclear power plants against seismic events, Parts 1-6", 2012