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PROBABILISTIC SEISMIC ASSESSMENT OF NUCLEAR POWER PLANT SAFETY

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Abstract. *This paper presents the probabilistic seismic assessment of nuclear power plant (NPP) safety analysis in Slovakia. There are the experiences from the deterministic and probabilistic seismic analysis of the structure resistance. On the base of the geophysical and seismological monitoring of locality the peak ground acceleration and the uniform hazard spectrum of the acceleration was defined for the return period 10 000 years using the Monte Carlo simulations. There is showed summary of calculation models and calculation methods for the probability analysis of the structural safety considering load, material and model uncertainties. Earthquake resisting of the NPP building was considered taking a dynamic effect on soil-structures interaction into account. There are summarized the works performed by the IAEA in the areas of safety review. The generation of the seismic loads on the base of probabilistic seismic risk analysis is described. The calculation models and methods for generation floor response spectrum in the NPP buildings in accordance with international standard are presented. The synthetic spectrum compatible accelerograms generated in program COMPACEL (created by Kralik) are presented in comparison with requirements ASCE4/98 standard. The deterministic and probabilistic methodology of the floor response spectrum calculation is discussed. The upgraded conception of the NPP main building steel structures is demonstrated. The results from the reliability analysis of the NPP structures are discussed.*

1 INTRODUCTION

The IAEA (International Atomic Energy Agency) set up a program [4 - 6] to give guidance to its member states on the many aspects of the safety of nuclear power reactors. The risk of the NPP performance from the point of the safety must be calculated by consideration of the impact of the all effects during plant operation. The PSA (Probabilistic Safety Analysis) is one from the effective methods to analyze the safety and reliability of the NPP. The international standard NUREG-1150 [20] defines the principal steps for the calculation of the risk of the NPP performance by LHS probabilistic method

- Accident frequency (systems) analysis
- Accident progression analysis
- Radioactive material transport (source term) analysis
- Offsite consequence analysis
- Risk integration.

The accidents caused by the earthquake even are the critical emergencies from the point of the NPP performance. This paper gives the experiences from the seismic analysis of the operated NPP in Slovakia [8-16, 22 and 23]. The earthquake resistance analysis of NPP buildings in Slovakia were based on the recommends of international organization IAEA in Vienna to get international safety level of the nuclear power plants [5]. Seismic safety evaluation programs of the NPP structures should contain three important parts [12]:

• The assessment of the seismic hazard as an external event, specific to the seismic-tectonic and soil conditions of the site, and of the associated input motion;

• The safety analysis of the NPP resulting in an identification of the selected structures, systems and components appropriate for dealing with a seismic event with the objective of a safe shutdown;

• The evaluation of the plant specific seismic capacity to withstand the loads generated by such an event, possibly resulting in upgrading.

2 SEISMIC SAFETY METHODOLOGY

On the base of the experience from the reevaluation programs in the membership countries IAEA in Vienna the seismic safety standard No.28 was established at 2003 [6]. Seismic safety evaluation programs should contain three important parts

- The assessment of the seismic hazard as an external event, specific to the seismic-tectonic and soil conditions of the site, and of the associated input motion;
- The safety analysis of the NPP resulting in an identification of the SSSCs (Selected Structures, Systems and Components) appropriate for dealing with a seismic event with the objective of a safe shutdown;

The evaluation of the plant specific seismic capacity to withstand the loads generated by such an event, possibly resulting in upgrading [12].

2.1 Seismic Hazards

The assessment of the seismic hazards specific to the seismic-tectonic conditions at a site is performed on the following bases:

- IAEA Safety Guides [4-6] And NEA requirements [20 and 21],
- Use of current internationally recognized methods and criteria [1, 7, 12, 19, 22 and 23],
- New data [2 and 16].

Two levels of the seismic load are defined in the standards [4]. SL-1 (First level) is coincident with the design earthquake and SL-2 (second level) corresponds to the maximum design earthquake. On the base of the IAEA requirements the NPP structures of the first category have been resistance due to seismic level SL-2. This seismic level [4] should be updated in accordance with the above bases in the event that a reason for this has appeared since the evaluation of the SL-2 design level and should be used in the evaluation. In particular, the PGA (Peak Ground Acceleration) of the RLE (Review Level of Earthquake) should not be less than 0.1g. The level of the seismic risk is characterized by the probability level (return period) and the peak ground accelerations values, which are the typical free field zero period acceleration values at the ground surface.

2.2 High Confidence Low Probability of Failure

The concept of the HCLPF (High Confidence Low Probability Failure) capacity is used in the SMA (Seismic Margin Assessment) reviews to quantify the seismic margins of NPPs [5]. In simple terms it corresponds to the earthquake level at which, with high confidence (≥ 95%) it is unlikely that failure of a system, structure or component required for safe shutdown of the plant will occur (< 5% probability).

The value of the HCLPF parameter depends on the equipment structure or component resistance (R) and the corresponding effect of action (E) using elastic or inelastic behavior.
Generally it follows

$$HCLPF = (R - E_{NS}) / (E_{Si} / k_D)^2 + (E_{Sa} \cdot k_D)^2]^{1/2} \cdot PGA_{\mathrm{RLE=SL\text{-}2}} > PGA. \tag{1}$$

where E_{NS} is a total response to all the coincidental non-seismic bearings in the given combinations, E_{Si}, or E_{Sa} is the seismic response to RLE (SL-2) inertial actions, k_D is ductility coefficient ($k_D \geq 1.0$).

2.3 Seismic Input Data

The seismic response can be calculated in the frequency (spectrum response analysis) or time domain (transient analysis) [12].

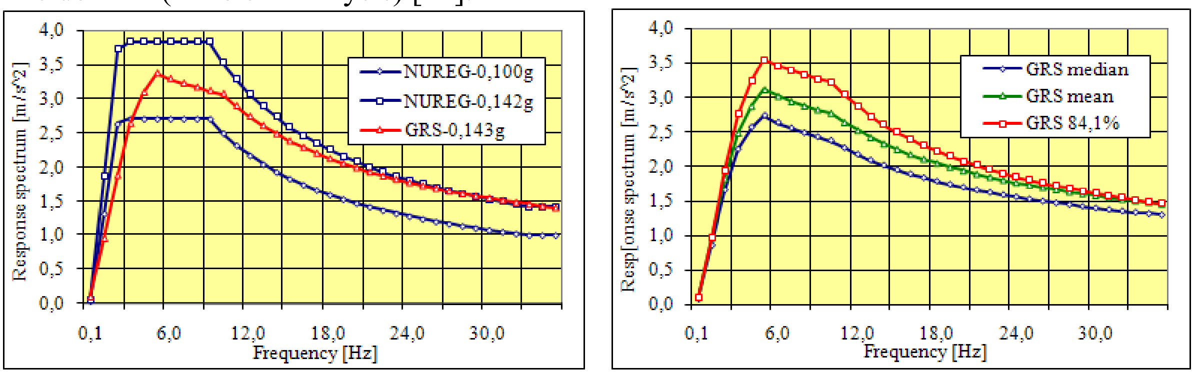

Figure 1: Comparison of the horizontal and vertical acceleration response spectrum NUREG and GRS.

Also, hence the earthquake input must be specified in terms of free-field ground motion accelerograms for time-history dynamic analyses [1]. The foundation of the reactor building NPP can be embedded into the subsoil. This embedment has generally two effects on the dynamic analysis of the building:

- In comparison to a surface foundation the dynamic behavior of the foundation is different. In the case of rock these differences are minimal. The impedance analysis results in stiffness parameters and damping ratios for the foundation soil system, which are higher than those for a surface foundation.

- The second effect is that the acceleration time histories at foundation level are different from the control motions specified at the surface of the free field.

In the case where structure and soil are idealized in only one Finite Element System or a consistent substructuring analysis the control motion is specified at the top of the surface and the effect of the embedment on both impedance and free field motion are automatically taken into account.

To provide input excitations to structural models for sites with no strong ground motion data, it is necessary to generate the synthetic accelerogram. It has long been established that due to parameters such as geological conditions of the site, distance from the source, fault mechanism, etc. different earthquake records show different characteristics. Based on Kanai's investigation regarding the frequency content of different earthquake records, Tajimi proposed the following relation for the spectral density function of the strong ground motion with a distinct dominant frequency [12]:

$$S(\omega) = \frac{\left[1 + 4\xi_g^2 \left(\omega/\omega_g\right)^2\right]}{\left[1 - \left(\omega/\omega_g\right)^2\right] + 4\xi_g^2 \left(\omega/\omega_g\right)^2} S_0 \tag{2}$$

Here ξ_g and ωg are the site dominant damping coefficient and frequency, and S_0 is the constant power spectral intensity of the bed rock excitation.

The program COMPACEL was created by J.Králik to generate synthetic spectrum compatible accelerograms. The comparison of the synthetic acceleration spectrum and GRS spectrum in the case of three and one accelerograms is showed in Figure 2 and 3.

Using three accelerograms for the calculation of spectrum response the calculation results is less conservative than for one accelerogram.

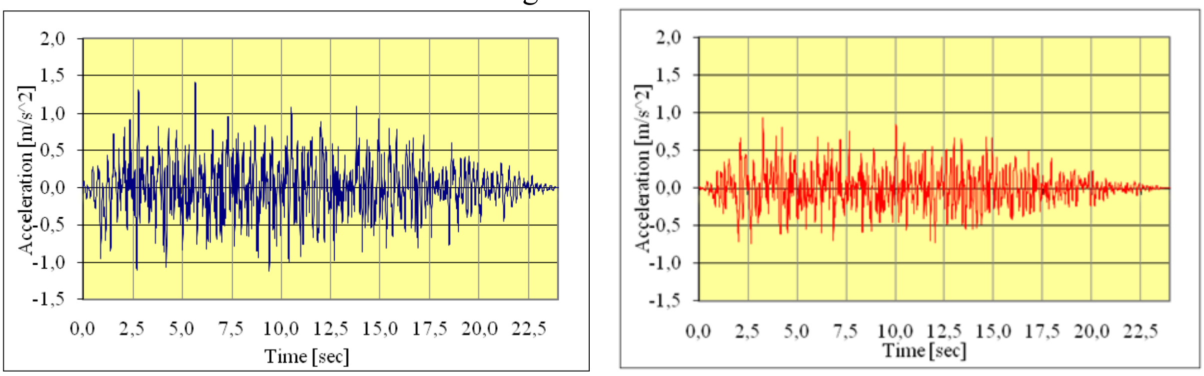

Figure 2: The spectrum compatible design horizontal and vertical accelerograms.

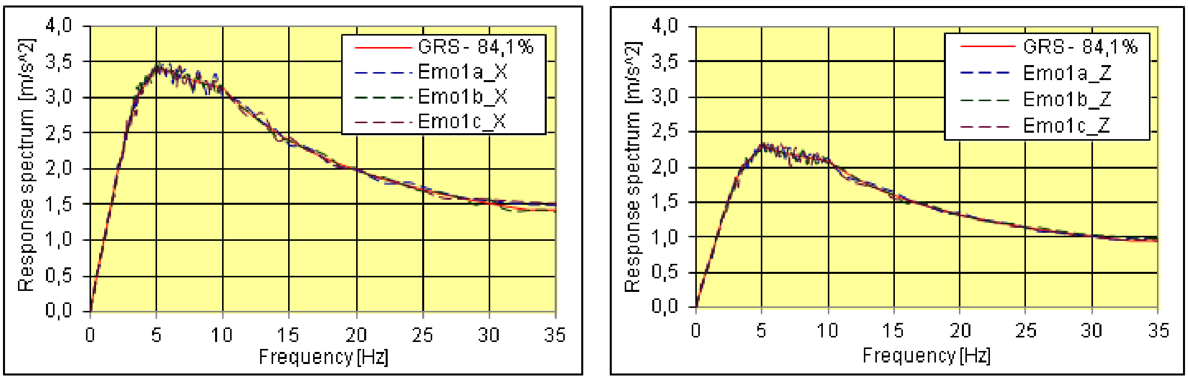

Figure 3: Comparison of the synthetic acceleration spectrum and GRS spectrum.

3 RELIABILITY ANALYSIS OF THE STRUCTURES

Most problems concerning the reliability of building structures [7, 12 and 19] are defined today as a comparison of two stochastic values, loading effects *E* and the resistance *R*, depending on the variable material and geometric characteristics of the structural element. In the case of a deterministic approach to a design, the deterministic (nominal) attributes of those parameters R_d and E_d are compared.

The deterministic definition of the reliability condition has the form

$$R_d \geq E_d \,, \tag{3}$$

and in the case of the probabilistic approach, it has the form

$$RF = R - E = 1 - E/R \geq 0 \,. \tag{4}$$

The reliability function *RF* can be expressed generally as a function of the stochastic parameters X_1, X_2 to *an*, used in the calculation of *R* and *E*.

$$RF = g(X_1, X_2, ..., X_n) \,. \tag{5}$$

The probability of failure can be defined by the simple expression

$$P_f = P\left[R < E\right] = P\left[\left(R - E\right) < 0\right] = P\left[\left(1 - E/R\right) < 0\right]. \tag{6}$$

In the case of simulation methods the failure probability is calculated from the evaluation of the statistical parameters and theoretical model of the probability distribution of the reliability function *g*(*X*). The failure probability is defined as the best estimation on the base of numerical simulations in the form [7]

$$P_f = \frac{1}{N}\sum_{i=1}^{N} I\left[g\left(X_i\right) \leq 0\right]. \tag{7}$$

where *N* in the number of simulations, *g(.)* is the failure function, *I[.]* is the function with value 1, if the condition in the square bracket is fulfilled, otherwise is equal 0.

3.1 LOAD AND LOAD COMBINATION

The load combination of the **deterministic calculation** is considered according to ENV 1990 [1] and IAEA [2, 3] for the ultimate limit state of the structure as follows:

➲ ***Deterministic method – extreme design situation***

$$E_d = G_k + Q_k + A_{Ed} \,. \tag{8}$$

where G_k is the characteristic value of the permanent dead loads, Q_k - the characteristic value of the permanent live loads, A_{Ed} - the design value of the extreme loads, $A_{Ed.k}$ - the characteristic design value of the extreme loads.

In the case of **probabilistic calculation** and the ultimate limit state of the structure the load combination [6] we take following:

➲ ***Probabilistic method – extreme design situation***

$$E = G + Q + A_E = g_{\mathrm{var}} G_k + q_{\mathrm{var}} Q_k + a_{\mathrm{var}} A_{E.k} \,. \tag{9}$$

where g_{var}, q_{var}, a_{var} are the variable parameters defined in the form of the histogram calibrated to the load combination in compliance with Eurocode [1] and JCSS requirements [4].

Seismic response was solved by linear response spectrum method. Spectral analysis results from linear behavior of structures and the appropriate damping due to structure plasticity is considered by proportional damping for the whole structure or separately by materials. The seismic response for each direction of excitation was calculated particularly by spectrum response method using combination rule SRSS

$$E_i = \sum_{m=1}^{N.\mathrm{mod}} E_{m.i} \ , \tag{10}$$

where "i" is excitation direction ($i = X, Y, Z$), "m" is the mode number from the modal analysis, "$N.mod$" is the total number of modes. Total seismic response was calculated by ASCE 4/98 [1] in the form

$$E_{tot}=E_X+0{,}4E_Y+0{,}4E_Z \quad \text{or} \quad E_{tot}=0{,}4E_X+0{,}4E_Y+E_Z \quad \text{or} \quad \mathrm{E}_{tot}=0{,}4E_X+E_Y+0{,}4E_Z \tag{11}$$

The maximum from all possibilities is taken to design structure.

3.2 UNCERTAINTIES OF INPUT DATA

The uncertainties of the input data – action effect and resistance are for the case of the probabilistic calculation of the structure reliability defined in JCSS and Eurocode 1990 [3].

Name	Quantity	Charact. value	Variable paramet.	Histogram	Mean	Stand. deviation	Min. value	Max. value
Soil	Stiffness	k_{zk}	k_{var}	Normal	1	0,200	0,451	1,490
Material	Young's modulus	E_k	e_{var}	Normal	1	0,120	0,645	1,293
Load	Dead	G_k	g_{var}	Normal	1	0,010	0,755	1,282
	Live	Q_k	q_{var}	Gumbel	0,60	0,200	0	1
	Earthquake	$A_{E,.k}$	a_{var}	Gama(T.II)	0,67	0,142	0,419	1,032
Resistance	Steel strength f_{sk}	F_k	f_{var}	Lognormal	1	0,050	0,670	1,485
Model	Action uncertaint	M_E	m_{var}	Normal	1	0,100	0,875	1,135
	Resistance uncert.	M_R	r_{var}	Normal	1	0,100	0,875	1,135

Table 1: Probabilistic model of input parameters

The stiffness of the structure is determined with the characteristic value of Young's modulus E_k and variable factor e_{var}. A load is taken with characteristic values G_k, Q_k, $A_{E,k}$, $A_{W,k}$ and variable factors g_{var}, q_{var}, a_{var} and w_{var}. A soil stiffness variability in the vertical direction is defined by the characteristic stiffness value k_{zk} from the geological measurement and the variable factor k_{var}. The resistance of the steel is delimited by the characteristic values of the strength f_{sk} and the variable factor f_{var}. The uncertainties of the calculation model are considered by variable model factor M_R and variable load factor M_E for Gauss's normal distribution.

4 CALCULATION MODEL OF NPP STRUCTURE

The NPP WWER 440 building consists of six objects - reactor building, bubbler tower, air-conditioning centre, turbine building, and lengthwise side electrical building and cross side electrical building [12]. The foundation plate (75,0/43,0m) under building on part V-D/10-22 is on two levels -8.5m. The foundation plate (39,5m/27,0m) under bubbler tower on part D-E/10-17 is on level -8.5m too. The foundation strip and foot under columns are in the cross side electrical building and turbine building. The global geometry of the NPP structures in Jaslovské Bohunice and Mochovce is identical, but the bracing system and the section area of the steel elements are different.

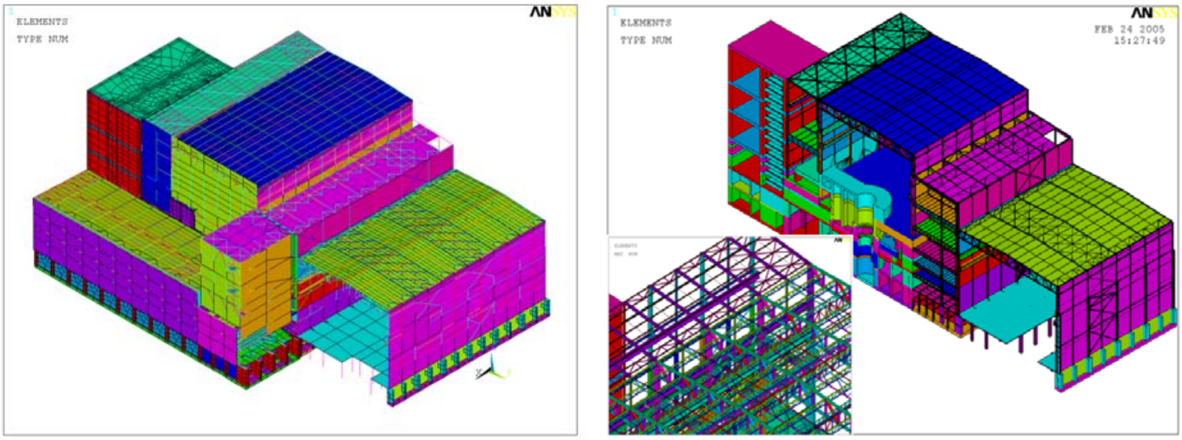

Figure 4: Calculation model of NPP.

The NPP building was discredited [12] by the 3D finite elements model to obtain realistic behavior of structure. The model (VUT Brno and STU Bratislava) consists of 161.856 elements with 440.531 degrees of freedom. The drawbars are modeled by bilinear elements and contact between bubbler tower and air-conditioning center by gap elements [17].

5 SEISMIC RESISTANCE OF NPP BUILDING

On the base of SMA methodology the seismic resistance of the NPP structures in Slovakia was calculated. The seismic load for the NPP site was defined by peak ground acceleration (PGA) and local seismic spectrum in dependence on magnitude and distance from source zone of earthquake.

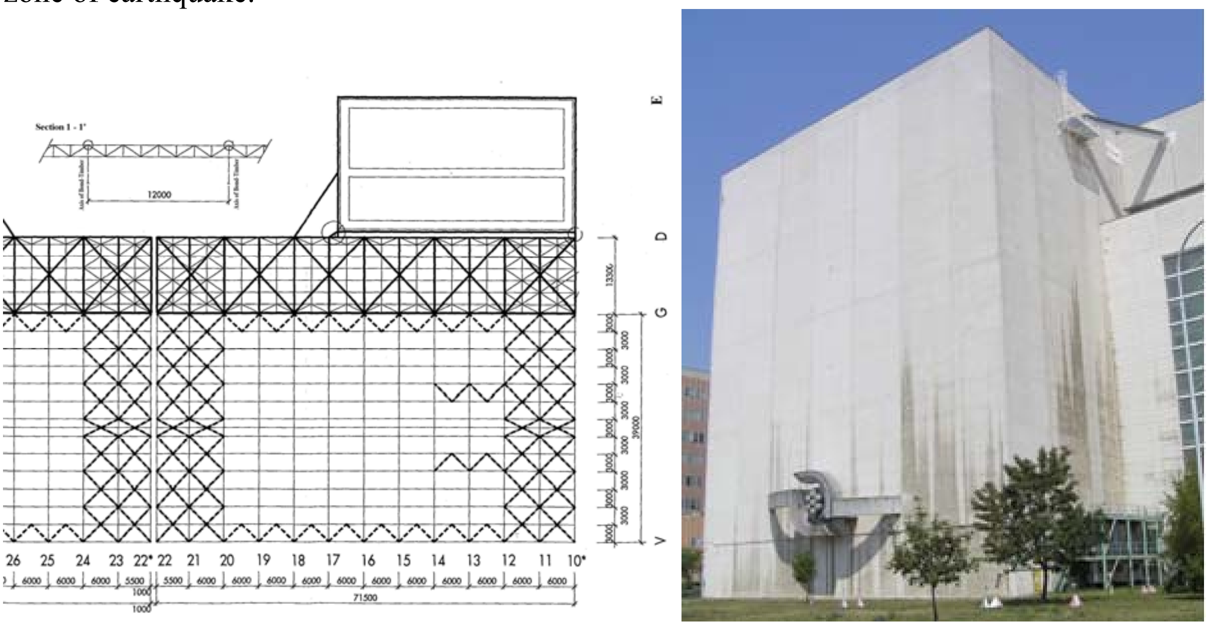

Figure 5: Upgrade steel structure of reactor hall and concrete structure of bubbler tower.

The alternative conception was used in NPP in Paks [12]. There was built the steel bridge structure between two concrete structures of the bubbler towers. The level of the seismic risk is lower in Mochovce site as in J. Bohunice site. The seismic resistance of the NPP structures is satisfying in accordance of the IAEA requirements. The recapitulation of the HCLPF parameters of principal structure elements of the NPP buildings in Mochovce is demonstrated in [12].

Columns primary	Vertical bracing	Beams	Plane truss	Roof bracing	Anchors
SO 490 Tools Hall					
0,184	0,244	-	0,364	0,243	-
SO 800 Reactor Hall					
0,235	0,231	-	0,457	0,186	-
SO 800 Ventilation Hall					
0,172	0,173	1,095	-	0,244	-
SO 805 Longitude Gallery					
0,890	0,642	0,715	-	0,228	0,190
SO 806 Transversal Gallery					
0,368	0,235	0,264	-	1,008	0,190

Table 2: HCLPF parameters for structural elements

The seismic safety of NPP building, after strengthening of the steel structures of gallery building floors to the concrete structure of the reactor building, is determined by the seismic resistance of the gallery anchors and secondary columns of the ventilating hall. The elements of the NPP steel structure were designed in accordance of the Eurocode requirements described below. The results from the design check of the deterministic analysis are shown in Table 2. There are described the safety level of the critical elements of the steel structures in accordance with the IAEA requirements [5]. The probabilistic analysis was realized using 10^6 Monte Carlo simulations in program FReeT [19]. The uncertainties of the input data was considered in the form of the histograms (see Table 1). The density of the probability of the failure (Figure 6) presents the reliability function in the form of the equation (5). The probability of the structure failure was equal to $P_f = 9{,}5262.10^{-7} < 10^{-6}$, the reliability index is equal to $\beta = 4{,}7632 > 4{,}7$.

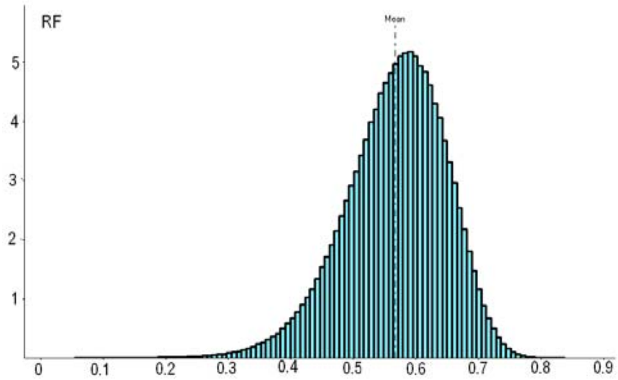

Figure 6: Reliability function of the failure of the critical column

6 CONCLUSIONS

This paper presented the deterministic and probabilistic methodology to analysis the seismic resistance of NPP in Slovakia. There were summarized the works performed by the IAEA in the areas of safety review. The methodology of the seismic reevaluation of NPP in Slovakia is based on the new results from the geological and seismic-tectonic monitoring of this site. There were summarized the works performed by the IAEA in the areas of safety review. The generation of the seismic loads on the base of probabilistic seismic risk analysis was described. The calculation models and methods for generation FRS in the NPP buildings in accordance with international standard were presented. The synthetic spectrum compatible accelerograms generated in program COMPACEL (created by Kralik) were presented in comparison with requirements ASCE4/98 standard. The upgraded conception of the NPP main building steel structures was presented in Slovakia. The seismic safety of NPP building, after strengthening of the steel structures of gallery building floors to the concrete structure of the reactor building, is determined by the seismic resistance of the gallery anchors and secondary columns of the ventilating hall. The seismic resistance of NPP buildings is defined as HCLPF = 0,172g > PGA = 0,15g. The PGA=0,15g is determined for the earthquake SL-2. The probability analysis was accomplished from the 10^6 Monte Carlo simulations in program FReET. The probability of the structure failure was equal to $P_f = 9{,}5262.10^{-7} < 10^{-6}$, the reliability index is equal to $\beta = 4{,}7632 > 4{,}7$.

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