ENGINEERING MECHANICS 2009
National Conference with International Participation
Svratka, Czech Republic, May 11 – 14, 2009

pp. 703–714
Paper **#276**

DETERMINISTIC AND PROBABILISTIC ANALYSIS OF VENTILATING CONCRETE CHIMNEY RELIABILITY DURING EARTHQUAKE

Králik, J.*, Ivánková, O.**

Summary: *In this paper, the deterministic and probability analysis of the seismic resistance of the reinforced concrete structure of the nuclear power plants ventilating chimney, considering the nonlinear reserve of the structure under the seismic load of the reinforced concrete is presented. Simple and detailed solutions on the base of the „pipe" beam and shell elements are considered. The deterministic and the probability approaches are compared. Advantages and disadvantages of both methods are mentioned. The applicability of probabilistic approach to the solution of reliability of such structures is demonstrated, using the example of ventilating chimney of the nuclear power plant.*

1. Introduction

This paper deals with static and dynamic analysis of the ventilating chimney. In the frame of the upgrade project for elevation of the safety and reliability of NPP in Slovakia the seismic resistance of NPP buildings have been verified. The earthquake activities were monitored in this region 20 last years (Labák & Coman, 2006; Kralik, 2009). After this monitoring the new seismic load was defined for this locality. The earthquake resistance analysis of NPP buildings in Slovakia were based on the recommends of the international organization IAEA in Vienna (IAEA 1994, 2003) to get international safety level of nuclear power plants. The seismic response can be calculated in the frequency (spectrum response analysis) or time domain (transient analysis). The earthquake input can be specified in terms of free-field ground motion in the form of ground response spectrum for spectrum response analysis or the design spectrum compatible accelerograms for time-history dynamic analyses. The foundation of the chimney is embedded into the rock subsoil. The effect of the soil-structure interaction is neglected to seismic response. The ventilating chimney is aimed at structures with 2a. seismic category, what means, that it can be damaged but in the case of collapse it must not be threat the object of 1. seismic category (IAEA 1994, 2003).

Figure 1: Ventilating chimney

* Assoc. prof. Ing. Juraj Králik, PhD., Slovak University of Technology in Bratislava, Radlinského 11, 813 68 Bratislava, juraj.kralik@stuba.sk

**Assoc. prof. Ing. Oľga Ivánková, PhD., Slovak University of Technology in Bratislava, Radlinského 11, 813 68 Bratislava, olga.ivankova@stuba.sk

2. Seismic Re-evaluation Program

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.

Seismic safety evaluation programs should contain three important parts

- The assessment of the seismic hazard as an external event, specific to the seismotectonic 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 (SSSCs) 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.

The earthquake resistance analysis of NPP buildings in Mochovce was based on the recommends of international organization IAEA in Vienna (Safety Series 50-SG-S1, 50-SG-D15), ASCE 4/98, ASCE 7/95, NUREG/CR-0098, NUREG/CR-4334, ACI 349-90, ACI 318-92, EUROCODE 2, 7 and 8, CEB and Slovak National Standards.

Methodology of structure resistance verification is elaborately described by Kralik (2009) . There are illustrated the procedures, requirements and criterion of calculation models and methods for design of structure reliability. There are two principal methodology available for seismic design of NPP structures - deterministic (SMA- seismic margin assessment) and probabilistic (SPRA – seismic probabilistic risk assessments) in accordance with ASCE 4/98.

Table 1: Summary of Conservative Deterministic Failure Margin Approach - CDFM

Load combination	Normal + seismic (SME)
Ground response spectrum GRS	Conservative specified (84% Nonexceedance probability)
Damping	Conservative estimate of median damping
Structural model	Best estimate (median) + uncertainty variation in frequency
Soil-structure-interaction	Best estimate (median) + parameter variation
Material strength	Code specified minimum strength or 95% exceedance actual strength if test data available
Static capacity equation	Code ultimate strength (ACI), maximum strength (AISC). If test data are available, then use a value exceeded 84% of test data.
Inelastic energy absorption	For non-brittle failure modes and linear analysis, use 80% of computed seismic stress in capacity evaluation to account for ductility benefits (or perform nonlinear analysis and go to 95% excedance ductility levels)
In-structure (floor) spectra generation	Use frequency shifting rather than peak broadening to account for uncertainty plus use median damping

Table 1 provides a summary of the Conservative Deterministic Failure Margin (CDFM) approach by ASCE 4/98 (1998). This method is very similar to the design procedure followed in the industry, except that the parameter values have been liberalized. The objective of seismic margin assessment (SMA) is to determine for a nuclear power plant the high-confidence-of-a-low-probability-of-failure (HCLPF) capacity for a preselected seismic margin earthquake (SME), which is always chosen higher than the design basis input. In probabilistic terms, the HCLPF is expressed as approximately a 95% confidence of about a 5% or less probability of failure. The advantages and disadvantages associated with the application of SMA or SPRA methodology are compared in Table 2.

Table 2: Comparison of SMA and SPRA methodology

SMA	SPRA
Most important elements of seismic PRAs are retained (using date from plan or walkdown). The scope of components and systems that need to be reviewed is reduced	It provides a complete risk profile and can provide all the results obtained from the seismic margins methodology. Uncertainties are explicitly accounted for.
A measure of plant capacity is provided by engineers. It does not require fragility calculation.	It provides a more rigorous consideration of nonseismic failure and human actions
No direct risk insights are obtained	Decision-making can be based on plant-specific risk results
Results are not affected by seismic hazard issues	It can expand upon the even/fault frees developed for the internal events PRA analysis
Plan capacity estimates will be useful to judge the impact of design basis earthquake issues	It provides a more rigorous consideration of nonseismic failures and human actions
Ranking is based only on HCLPF capacities; thereby making it difficult to prioritize issues in the absence of a better risk-based ranking	Accident mitigation, accident management, and emergency planning can addressed more systematically and with greater detail
The level of effort required to implement is lower than that for a seismic PRA when both are done at the same level of detail	Ranking based on different indices are available, for instance, core melt, frequency, release
Correlations among failure can be identified and analyzed with the NRC event/fault tree method	It can expand upon the event/fault trees developed for the internal events PRA analysis

3. Calculation Model of Chimney Structure

The ventilating chimney consists of two basic parts: bulky bottom foundation structure and upper structure of chimney (Králik et al, 2006). Chimney height is 150m over ground. This object is founded on stepped circular foundation plate with Ø 12,0m and 20,5m diameter and with 4,50m thickness. The upper part (chimney) is created by chimney body with ring ground plane. The ring thickness is changed by the chimney height from 0,80m to 0,23m and the outer diameter is changed by the chimney height from Ø12,0m until Ø5,95m. Two principal FEM models were created in program ANSYS – simple 1-dimensional model (162 elements) from the pipe elements PIPE16 and detailed 3-dimensional model (5086 elements) from the shell elements SHELL43 and solid elements SOLID45.

The soil is modeled as rigid and flexible base. The shear velocity of the soil on the free field is equal to 1400-1600m/s. The soil with the shear velocity higher than 1000m/s can be modeled as rigid in accordance of standard ASCE 4/98 (1998). The flexible soil stiffness under circle foundation plate can be modeled by Pais-Kausel with considering the base depth (Králik, 2009).

$$K_u = \frac{8 G_p r_o}{2-\nu_p}\left(1+\frac{H_Z}{r_o}\right)\left(1+\frac{H_Z}{H_o}\right)\left(1+\frac{1}{2}\frac{r_o}{H_o}\right)$$

$$K_w = \frac{4 G_p r_o}{2-\nu_p}\left(1+0{,}54\frac{H_Z}{r_o}\right)\left(1+1{,}30\frac{r_o}{H_o}\right)\left[1+\left(0{,}85-0{,}28\frac{H_Z}{r_o}\right)\frac{H_Z}{H_o-H_Z}\right] \tag{1}$$

$$K_\varphi = \frac{8 G_p r_{xx}}{3\left(1-\nu_p\right)}\left(1+0{,}7\frac{H_Z}{H_o}\right)\left(1+\frac{1}{6}\frac{r_{xx}}{H_o}\right)\left[1+2{,}3\frac{H_Z}{r_{xx}}+0{,}58\left(\frac{H_Z}{r_{xx}}\right)^3\right]$$

where G_p is soil shear modulus, H_z is depth of foundation level, H_o is active depth, r_{xx} is equivalent radius $r_{xx} = \left(\ell_x \ell_y^3 / 3\pi\right)^{1/4}$.

3. Modal Analysis

The static and dynamic analysis were realized by 1D FEM model and 3D FEM model on the program ANSYS (Králik et al. 2006) for deterministic and probability solution in accordance with requirements ASCE 4/98 (1998).

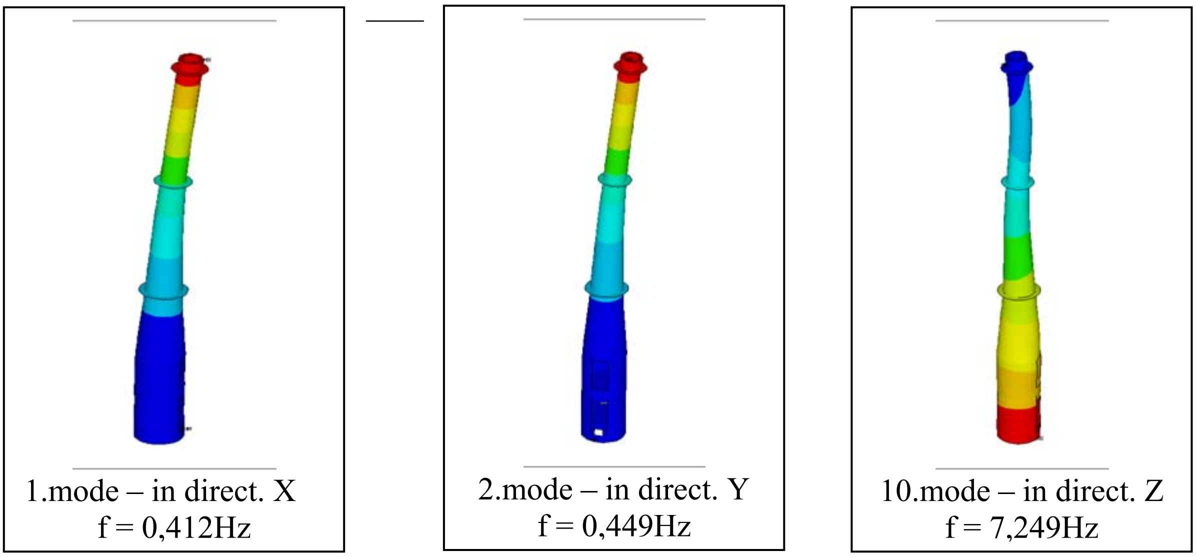

Figure 2: Dominant eigenmodes

Table 3: Eigenfrequencies of ventilating chimney

Model	Element	Soil	f_o [Hz] in direct. X	Effect.mas ratio %	f_o [Hz] in direct. Y	Effect.mas ratio %	f_o [Hz] in direct. Z	Effect.mas ratio %
1a	1D	R	1,471830	31,646	1,471830	31,646	7,49507	54,447
1b	1D	F	0,457301	33,685	0,457301	33,685	7,49496	54,448
2a	3D	R	0,411922	37,083	0,448855	35,783	7,24933	54,815
2b	3D	F	0,400256	38,098	0,434384	37,106	7,08394	56,770

Notes – R – rigid soil, F – flexible soil

The modal analysis was realized on four models two models (1a, 1b) were made from the PIPE16 elements a others (2a, 2b) were created from SHELL43 elements (see Tab.2). The eigenvalues and eigenvectors were calculated by block Lanczos iteration method, with 86 eigen values up to 35Hz frequency with effective modal mass 96% of total mass for horizontal vibration and 89% for vertical vibration. Dominant modes of vibration for particular directions are presented in Tab.3 and Fig.2.

5. Seismic Load

The seismic response can be calculated in the frequency (spectrum response analysis) or time domain (transient analysis). Also, hence the earthquake input must be specified in terms of free-field ground motion accelerograms for time-history dynamic analyses.

The earthquake resistance analysis of NPP buildings in Mochovce was based on the recommends of international organization IAEA in Vienna (ASCE 4/98, 1998, ASCE 7/95 , 1996; IAEA, 1994, 2003; Králik et al., 2006; Králik, 2009; Labák & Coman, 2006) to get international safety level of nuclear power plants.

Three logical possibilities of the source zones were defined – contact of Eastern Alps and Western Carpathians, Dobrá Voda and alternative fault (Labák & Coman, 2006).

The seismic load for the Mochovce site was defined by peak ground acceleration (*PGA*) and local seismic spectrum in dependence on magnitude and distance from source zone of earthquake. Firstly the value of *PGA* was defined at 1994 (PGA_{RLE}=0,1g) follow in accordance of the results of seismological monitoring this locality at 2003 (PGA_{UHS}=0,142g and PGA_{HS}=0,143g).

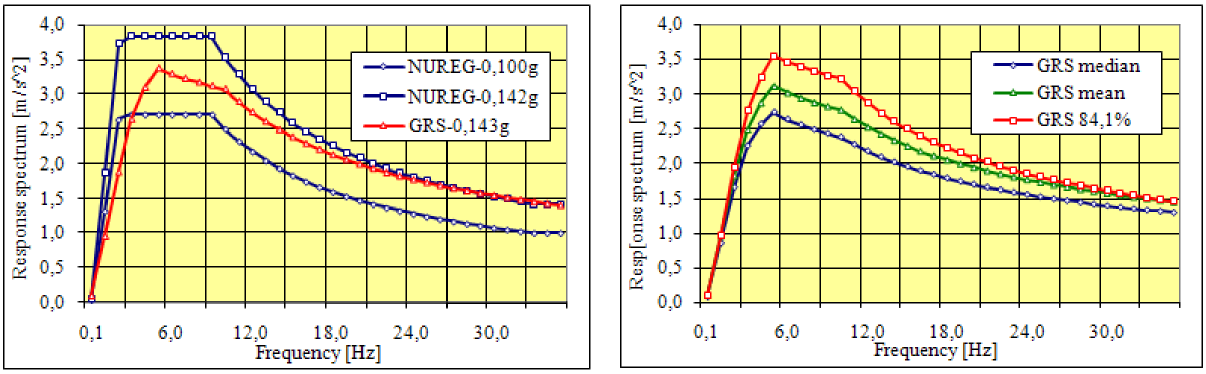

Figure 3: Comparison of the horizontal acceleration response spectrum NUREG and GRS

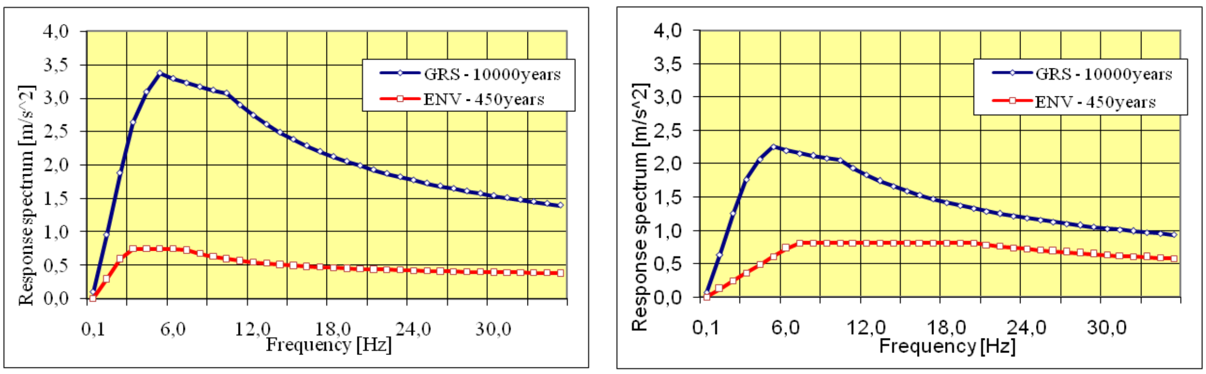

Figure 4: Comparison of the GRS and ENV spectrum for return period 10000 and 450 years

The seismic load for civil engineering buildings is defined for return period of 450 years but, on the other hand the safety of the nuclear power plants require the seismic loads defined for return period 10000 years. The comparison of the typical characteristics of the design acceleration spectrum according to a national standard ENV 1998 and ground response spectrum (*GRS*) for Mochovce NPP is showed in Figure 4. We can see since the seismic load is taken about 4-5 time higher than standard for the civil engineering buildings.

6. Seismic Response Analysis

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

$$E_i = \sum_{m=1}^{N.\mathrm{mod}} E_{m.i}\ , \tag{2}$$

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 in the form

$$E_{tot}=E_X+0{,}4E_Y+0{,}4E_Z \quad \text{or} \quad E_{tot}=0{,}4E_X+0{,}4E_Y+E_Z \quad \text{or} \quad E_{tot}=0{,}4E_X+E_Y+0{,}4E_Z \tag{3}$$

The maximum from all possibilities is taken to design structure.

7. Failure Function

The reinforced concrete chimney section is designed to the impact of bending moment and normal force as well the shear force for the failure function (Králik et al., 2006) in the form

$$g(N,M) = 1-\Phi_{id}\left(N_E, M_E\right)/\Phi_{id}\left(N_R, M_R\right) \geq 0\,, \qquad g(V) = 1 - V_E/V_R \geq 0 \tag{4}$$

where Φ_{id} is the function of the section interaction diagram, M_E, N_E, V_E are bending moment, normal and force of action and M_R, N_R, V_R are the resistance bending moment, normal and force per length.

In the case of the seismic event the maximum horizontal displacement of chimney is determined among the length *H*

$$g(d) = 1 - d_E/d_R \geq 0 \tag{5}$$

where d_E is the interstorey drift, d_R is limited value of interstorey drift defined in the form

$$d_R = 0{,}005.H/\nu \tag{6}$$

where ν is factor depend on type of object (ν = 0,4÷0,5).

8. High Confidence Low Probability 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. In simple terms it correspond to the earthquake level at which, with high confidence ($\geq$ 95%) it is unlikely that failure of a system, structure or component required for safe shutdown of the plant will occur (< 5% probability).

Estimating the HCLPF seismic capacity of a system, structure and component requires an estimation of the response, conditional on the occurrence of the RLE. Two candidate procedures to determine the HCLPF seismic capacities for NPP's structures and equipment components have been developed:

(1) the Fragility Analysis (FA), and

(2) the Conservative Deterministic Failure Margin (CDFM) method.

The HCLPF approach or an equivalent method may be used to verify the seismic capacity of Mochovce NPP. The general criteria for CDFM approach is contained in (ASCE 4/98, 1998).

A) Calculation procedure

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. The following equation follows for the strength and response (R/E) in respect to linear elasticity

$$(R/E)_{el} = R / [(E_{Si}^2 + E_{Sa}^2)^{1/2} + E_{NS}] \tag{7}$$

where E_{Si}, or E_{Sa} is seismic response to RLE (SL-2) inertial actions, or corresponding different seismic support movement, respectively, calculated according to linear elasticity. Then E_{NS} is a total response to all the co-incidental non-seismic bearings in the given combinations.

Analogically, considering the elastic-plastic effect

$$(R/E)_{\mathrm{ep}} = R / \{[(E_{Si} / k_D)^2 + (E_{Sa} \cdot k_D)^2]^{1/2} + E_{NS})\} \tag{8}$$

where k_D is ductility coefficient ($k_D \geq 1.0$). The partial seismic response E_{Sa} in equation (8.2) is really multiplied, not divided, by the ductility coefficient. If SME is greater than RLE (SL-2), then $(R/E)_{\mathrm{ep}}$ is greater than 1.0 and vice-versa. However, the $(R/E)_{\mathrm{el}}$ and $(R/E)_{\mathrm{ep}}$ ratios do not define the multiplication factors for RLE (SL-2) to gain the HCLPF seismic margin value. These factors are calculated as follows:

$$(FS)_{\mathrm{el}} = (R - E_{NS}) / (E_{Si}^2 + E_{Sa}^2)^{1/2} \tag{9}$$

$$(FS)_{\mathrm{ep}} = (R - E_{NS}) / (E_{Si} / k_D)^2 + (E_{Sa} \cdot k_D)^2]^{1/2} \tag{10}$$

The equation (10) is valid provided that $(FS)_{\mathrm{ep}} > (FS)_{\mathrm{el}}$ and it can be significantly simplified if the E_{Sa} response to different seismic support movement as a result of RLE (SL-2) is negligible or it does not need to be considered. Then

$$(FS)_{\mathrm{ep}} = (FS)_{\mathrm{el}} \cdot k_D \tag{11}$$

Generally it follows

$$HCLPF\ (CDFM) = (FS)_{\mathrm{ep}} \cdot PGA_{RLE=\mathrm{SL\text{-}2}} \text{ (in horizontal direction)} \tag{12}$$

and this value must always be $HCLPF > ZPA$.

The HCLPF seismic margin value can also be determined via a non-linear elastic-plastic calculation (e.g. limit analysis defined in the ASME BPVC Section III – Mandatory Appendix XIII). Generally, such calculation needs to be repeated several times before the seismic margin value is reached. No ductility coefficient is used in these non-linear calculations, of course (ductility coefficients are used only in linear elastic calculations).

B) Non-Calculation Procedure

This procedure should be used when the HCLPF seismic margin values are determined from the results of seismic margin examinations or non-calculation determination via the GIP-WWER method. If there are the TRS seismic probe spectra for the damping of 5% available where seismic examinations in accordance with the IEC 980:1989 or IEEE Std 344-2004 were executed successfully, the usual praxis requires a seismic examination failure probability of these spectra to be less than 1%. Let us mark the floor seismic response spectra for the damping of 5% and given RLE (SL-2) as $FRS_{\mathrm{RLE\,=\,SL\text{-}2}}$ as a simple relation follows for the $(FS)_{\mathrm{ep}(el)}$ factor :

$$(FS)_{ep(el)} = \min\left[TRS / (FRS_{\mathrm{RLE\,=\,SL\text{-}2}} + \text{reserve})\right] \tag{13}$$

The question is what the value of the reserve is so that the determined seismic margin value would be the HCLPF value. It e.g. depends on the way the TRS spectra represent seismic actuating interpreted during seismic examinations; or how conservative the $FRS_{RLE=SL\text{-}2}$ seismic spectra are.

If the TRS in the (13) equation is substituted with a corresponding margin spectrum or its multiple of 1.5 (see GIP-WWER), this relation can be used also for the HCLPF seismic margin value determination for a component equipment with seismic resistance evaluated via the GIP-WWER method. However, this determination is valid only if all the other seismic resistance criteria are verified positively according to this method. The comparison in the (13) equation is executed from a conservatively determined free-oscillation frequency upwards. Thin peaks in the $FRS_{RLE=SL\text{-}2}$ might under certain circumstances outnumber the TRS, or BS, or its multiple of 1,5.

9. Deterministic analysis

In the case of deterministic analysis in accordance of methodology SMA (Králik et al., 2006) the structure element is designed to load combination (LC3, LC4) of the static and dynamic response in the form

$$\text{LC3: } E_d = 1{,}0G_k + 1{,}0Q_k + 1{,}0A_k.\text{SME}, \quad \text{resp.} \quad \text{LC4: } E_d = 1{,}0G_k + 1{,}0Q_k - 1{,}0A_k.\text{SME}, \tag{14}$$

where G_k is characteristic value of dead load, Q_k is characteristic value of live load, $A_{k,SME}$ is characteristic value of seismic load SME.

The comparison of the seismic resistance of the chimney is presented in the Figure 5 and 6 for load combination LC3 and LC4. The limited seismic resistance of the NPP buildings is defined by value HCLPF=0,143g. The seismic resistance of the ventilating chimney is determined by the resistance of the chimney section in the top part of it. It is consequence of the interaction of normal force and bending moment on the circle section of the chimney.

Section level	HCLPF [g] Model 1a	HCLPF [g] Model 1b	HCLPF [g] Model 2a	HCLPF [g] Model 2b
-2,7	0,411	0,418	0,447	0,463
5,0	0,361	0,370	0,395	0,407
15,0	0,372	0,380	0,396	0,401
18,0	0,372	0,381	0,395	0,398
30,0	0,336	0,344	0,347	0,345
50,0	0,253	0,258	0,264	0,260
56,0	0,194	0,198	0,207	0,203
75,0	0,197	0,201	0,209	0,205
100,0	0,175	0,177	0,187	0,186
125,0	0,212	0,214	0,222	0,221
146,0	2,237	2,247	2,704	2,099

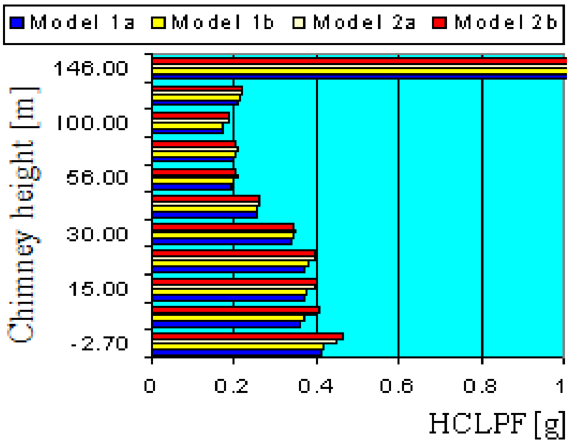

Figure 5: Comparison of HCLPF factor for various calculation models and load case LC3

Section level	HCLPF [g] Model 1a	HCLPF [g] Model 1b	HCLPF [g] Model 2a	HCLPF [g] Model 2b
-2,7	0,568	0,553	0,627	0,673
5,0	0,559	0,539	0,608	0,654
15,0	0,619	0,598	0,676	0,709
18,0	0,644	0,621	0,696	0,725
30,0	0,684	0,654	0,713	0,728
50,0	0,460	0,443	0,487	0,484
56,0	0,296	0,287	0,314	0,310
75,0	0,317	0,308	0,340	0,332
100,0	0,251	0,246	0,270	0,267
125,0	0,295	0,291	0,314	0, 309
146,0	6,620	6,533	7,736	7,534

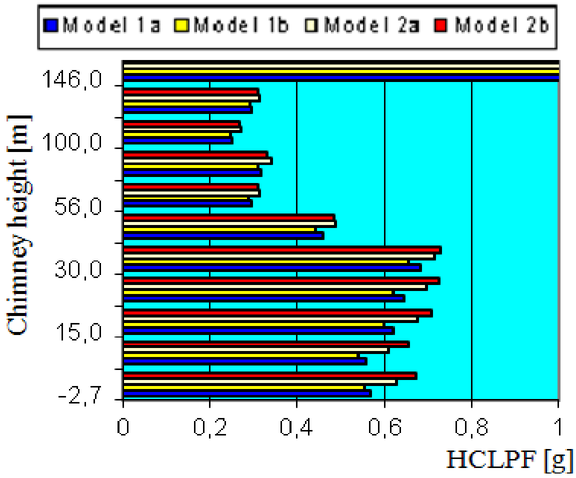

Figure 6: Comparison of HCLPF factor for various calculation models and load case LC4

10. Probabilistic Analysis

Recent advances and the general accessibility of information technologies and computing techniques give rise to assumptions concerning the wider use of the probabilistic assessment of the reliability of structures through the use of simulation methods in Czech Republic and Slovakia (Marek, Brozzetti, Guštar, 2001; Holický & Marková, 2005; Janas, Krejsa,M. Krejsa,V. 2006; Šejnoha & Novotná 2006; Teplý & Novák 2004; Králik, 2006, 2009). Much attention should be paid to using the probabilistic approach in an analysis of the reliability of structures (Ellingwood et al. 1990; Melchers 1999; Rosowsky 1995). The probabilistic analysis of the structure reliability on the point of view of ultimate limit state and serviceability limit state take the design forces into account from the load combination in the form

$$E = G + Q + A_{Ed} = g_{\mathrm{var}} G_k + q_{\mathrm{var}} Q_k + a_{\mathrm{var}} A_{Ed.k} \tag{15}$$

where g_{var}, q_{var}, a_{var} are a variable factors in the form of the histograms calibrated to action of variable loads by ASCE 7/95 (1996).

In the case of probability calculations of the structure reliability the uncertainties of load, soil stiffness and structure resistance can be effective considered by sensitivity analysis. The variability of soil stiffness is defined by variable factor $k_{z\cdot var}$ under the foundation plate. The uncertainties of calculation model are expressed by factor of model uncertainties θ_R and load uncertainties θ_E (Králik 2009).

Table 4: Probabilistic model of the basic parameters

Definition	Material		Load		Resistance	Model uncertainties	
	Soil stiffness	Modulus concrete	Permanent Load	Seismic load	Shear Resistance	Action	Resistance
Characteristic value	$k_{z,k}$	E_k	G_k	A_{k1}	V_{uk}	θ_{Ek}	θ_{Rk}
Variable	$k_{z\cdot var}$	$e_{\_var}$	$g_{\_var}$	$a_{\_var}$	v_{u_var}	t_{e_var}	t_{r_var}
Histogram	Normal	Beta(T.I)	Beta(T.I)	Beta(T.I)	LN	Normal	Normal
Mean value	1,00	1,05	1,00	0,67	1,00	1,00	1,00
Coef.variation	0,20	0,09	0,10	0,14	0,10	0,10	0,10
Min. value	0,15	0,90	0,57	0,40	0,68	0,70	0,70
Max. value	1,87	1,50	1,38	1,20	1,32	1,30	1,30

The variability of stiffness and masse of structure produce the variability of the model eigenfrequencies. The modal analysis of the probabilistic model show, that the dominant frequencies in the direction of X (e.g. Z) are in the interval from 0,317HZ to 0,570Hz (e.g. from 5,940Hz to 8,889Hz). These intervals have considerable influence to seismic load intensity of the design response spectrum below 2Hz.

11. Sensitivity Analysis

The sensitivity analysis of the influence of the variable input parameters to the seismic response is based on the statistically dependency between the input and output parameters.

Matrix of correlation coefficients of the input and output parameters is defined by Spearman in the form

$$r_s = \frac{\sum_{i=1}^{n} \left(R_i - \bar{R}\right)\left(E_i - \bar{E}\right)}{\sqrt{\sum_{i=1}^{n} \left(R_i - \bar{R}\right)^2} \sqrt{\sum_{i=1}^{n} \left(E_i - \bar{E}\right)^2}} \tag{16}$$

where E_i is rank of input parameters within the set of observations $[x_i]^T$, R_i is rank of output parameters within the set of observations $[y_i]^T$, $\bar{R}, \bar{E}$ are average ranks of the parameters R_i and E_i respectively.

Fig. 5 shows the results of the sensitivity analysis for first frequency and maximum horizontal displacement. This analysis demonstrate that first frequency is sensitive to variable of concrete stiffness on 76% and soil stiffness on 26%. The horizontal displacement depends on the seismic load and model uncertainties.

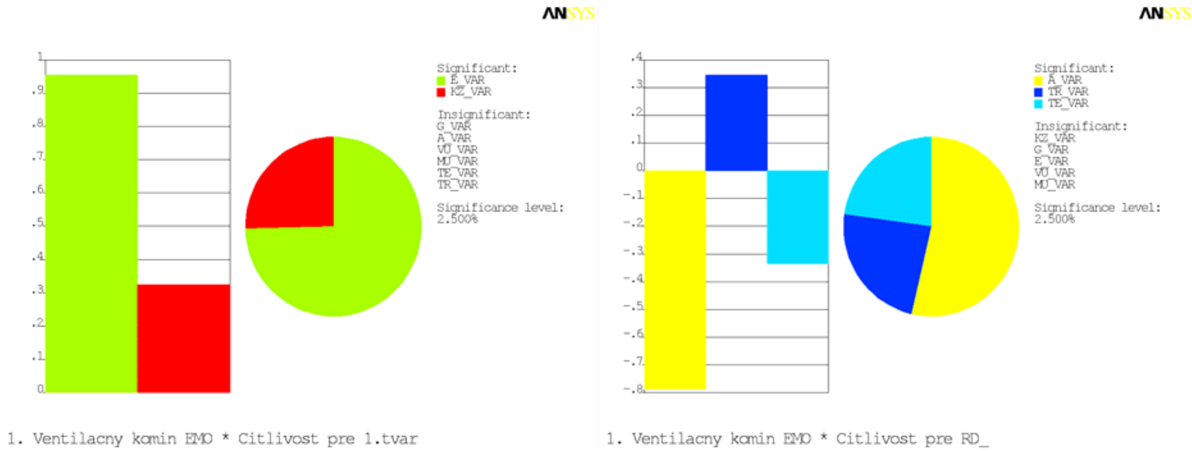

Figure 7: Sensitivity analysis of first chimney eigenfrequency and maximum horizontal displacement

12. Comparison of Deterministic and Probabilistic Calculation

Deterministic and probabilistic calculation was realized on the 3D FEM calculation model with variability of soil stiffness and all parameters in the Table 3.

Table 5: Comparison of shear forces and interstorey drift of chimney

	Extreme interstorey drift [mm]				Extreme shear force [MN]			
Method	Min	Max	Mean	St.dev	Min	Max	Mean	St.dev
Determin	-	-	1,08950	-	-	-	47,782	-
Probabilit	0,33932	1,3877	0,74968	0,18061	16,415	63,160	32,646	7,6927

The results from these analyses are presented in the Table 5. In consequence with these results we can declare, that the deterministic analysis is more conservative than probabilistic. This fact is related to load combination and variability of structure and soil stiffness. The probability of shear failure is $0{,}5036.10^{-4}$.

13. Conclusion

The aim of this paper was to propose the methodology of the deterministic (ASCE 4/98, 1998; György and Radnay, 2005) and probabilistic analysis (ASCE 4-98, 1998; Králik 2009; Šejnoha & Novotná 2006) of the nuclear power plant buildings in accordance of international requirements (IAEA, 1994, 2003; NUREG/CR-6926, 2007). There are presented the advantages and disadvantages of the various calculation methods (Králik 2009). The influence of the soil and structure stiffness and load uncertainties was considered in the example of the ventilating chimney. The seismic resistance of chimney body is represented by ASCE 4/98 (1998) requirements by parameter HCPLF = 0,175g (minimum value of HCLPF is 0,143g). We can affirm that the probabilistic analysis of structure give us the effective tool to consider the importance of uncertainties of various parameters (geometric and material) to the reliability of chimney structure.

Hence we can establish, that the ventilating chimney satisfies the safety condition by the most unfavorable load combinations.

14. Acknowledgement

This project was realized with financial support of Grant agency of Slovak Republic. Registration number of project is 1/0849/08.

15. References

ASCE 4-98, (1998) “ Seismic Analysis of Safety Related Nuclear Structures ”, *ASCE Standard*.

ASCE 7-95, (1996) *Minimum Design Loads for Buildings and Other Structures*, American Society of Civil Enginneers, New York.

Ellingwood, B. (1990) Validation studies of seismic PRAs. *Ncl. Engng.* Des. 1990; 123(2): 189-96.

Györgyi J., Radnay L. (2005) Chimneys Between the Earthquake and Wind, Between the Different Regulations. in: *New Trends in Statics and Dynamics of Buildings*, October 20 - 21, 2005, SVF STU Bratislava, pp. 99 - 102. ISBN 80-227-2116-6.

Holický,M., Marková,J. (2005) *Základy teorie spolehlivosti a hodnocení rizik*. Skriptum ČVUT Praha.

IAEA, (1994) *“ Technical Specifications for The Seismic Reevaluation Programme of Mochovce NPP”*, SLR/9/002 IAEA.

IAEA, (2003) *Safety Guide No.28*, *Seismic Evaluation of Existing Nuclear Power Plants,* Vienna.

Janas, P., Krejsa, M. a Krejsa, V. (2006) Structural Reliability Assessment Using Direct Determined Fully Probabilistic Calculation, proceedings of *3rd International ASRANet Colloquium 2006*, (abstract p.8, full paper on CD), 10 – 12th July 2006, Glasgow, UK.

Králik, J., Ivánková, O., Varga, T. (2006) Seismic Analysis of the Resistance of the Ventilating Chimney SO 803/1-01 for probabilistic seismic risk. *Report No. 04-196-04/EMO-803,* SvF STU Bratislava, 2006. (in Slovak)

Králik, J. (2009) *Safety and Reliability of Nuclear Power Buildings in Slovakia. Earthquake-Impact-Explosion.* Edition STU Bratislava, 2009, 305 p.

Labák, P., Coman, O. (2006) Sensitivity Tests of Seismic Hazard Computation for the Mochovce NPP Site. Stevenson and Associates, Bucharest.

Novotný,J-Kanický,V.-Salajka,V.-Štepánek,P. (2006) Seismic Analysis of selected structures of the NPP Dukovany- Influence of Modeling on the Corectness of Results.In proc. *Sborník prednášek z 1. konference s medzinárodní účastí. Dynamicky namáhané konstrukce – DYNA 2006*, VUT Brno, UAM Brno, ČSM VEDA Brno. pp.229-237, ISBN 80/214/3164-4.

Marek,P., Brozzetti,J., Guštar,M.: *Probabilistic Assessment of Structures using Monte Carlo Simulation, Background, Exercises and Software,* ITAM Academy of Sciences of the Czech Republic, GLOS s.r.o., Semily Czech Republic, Praha 2001.

Melcer, J. (2002) Posibility to Use the Approximate Relations in Dynamic Calculations. In Proc. VII. Science Conference with International participation, Section No.9, Building mechanics*, Košice*, 22.-24. 5. 2002, TU Košice, 2002, p. 182-185. (in Slovak)

Melchers,R.E. (1999) *Structural Reliability: Analysis and Prediction,* John Wiley & Sons, Chichester, U.K.

NUREG/CR-6926 (2007) *Evaluation of the Seismic Design Criteria in ASCE/SEI Standard 43-05 for Application to Nuclear Power Plants, BNL-NUREG-77569-2007,* U.S. Nuclear Regulatory Commission Washington, DC 20555-0001, Job Code N6112.

Rosovsky, D. V. (1995) Structural Reliability. Part of publ. W.Chen, *The Civil Engineering Handbook*, CRC Press, W.F.Chen Ed., Clenson University, SC.

Šejnoha,J., Novotná,E. (2006) Naplňování představ o pravděpodobnostním posuzování spolehlivosti systémů na základě simulace. In proc. *VII.Celostátní konfernce se zahranižní účastí „Spolehlivost konstrukcí“*, 5.dubna 2006, Praha, edit. ÚTAM AV ČR, pp.177-182.

Teplý,B., Novák,D. (2004) *Spolehlivost stavebných konstrukcí. Teórie, numerické metódy, navrhování, software.* Skriptum VUT Brno.