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COMPARISON OF DETERMINISTIC AND PROBABILISTIC METHODS TO DETERMINE FLOOR RESPONSE SPECTRUM

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Summary: *This paper gives the results of the probabilistic seismic analysis of the site Mochovce. 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. The methodology of the seismic probabilistic risk assessment and seismic margin assessment is presented.*

1. Introduction

The earthquake resistance analysis of NPP buildings in Mochovce were based on the recommends of international organization IAEA in Vienna (ASCE 4/98-1998, IAEA-1994, Labák- 1998, NUREG 0800-2007) 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, Dobra Voda and alternative fault (Labák, 1998).

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_{GRS}=0,143g).

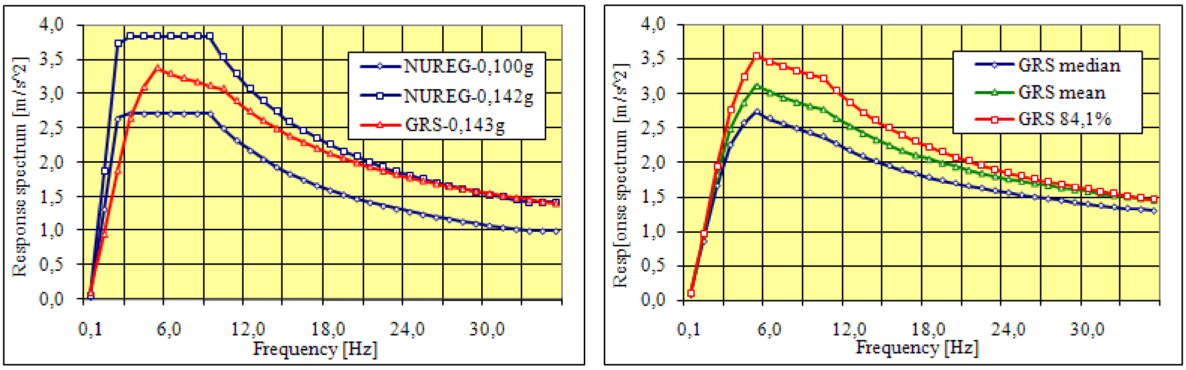
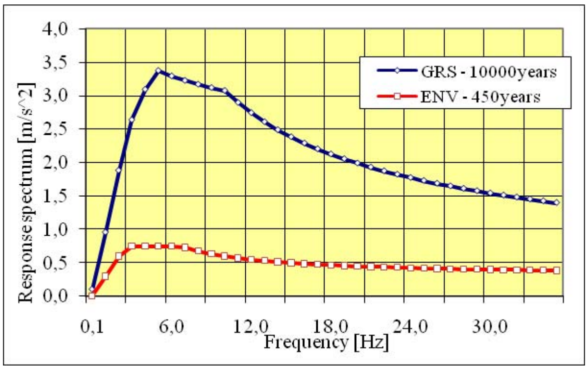


Fig. 1: Comparison of the horizontal acceleration response spectrum NUREG and GRS

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

*Assoc.prof. Ing. Juaj Králik, PhD.: Faculty of Civil Engineering, Slovak University of Technology, Radlinského 11, 813 68 Bratislava, Slovakia, tel./fax:+421 (2) 52 494 332; e-mail: juraj.kralik@stuba.sk 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 2 (Králik, 2004).

We can see than the seismic load is taken about 4-5 time higher than standard for the civil engineering buildings.



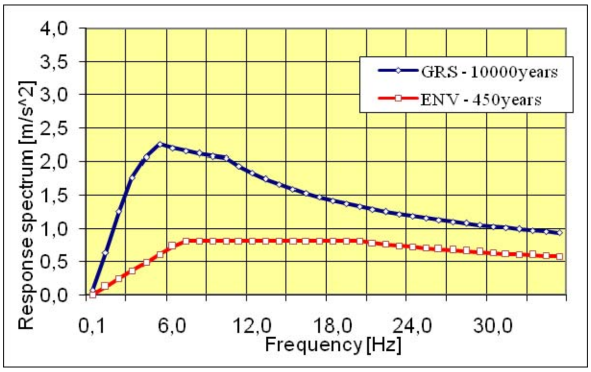


Fig. 2: Comparison of the GRS and ENV spectrum for return period 10000 and 450 years

The seismic response can be calculated in the frequency (spectrum response analysis) or time domain (transient analysis). The earthquake input must be specified in terms of free-field ground motion accelerograms for time-history dynamic analyses (Králik, 2006).

The foundation of the reactor building is 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.

2. Response spectrum compatible accelerogram

To provide input excitations to structural models for sites with no strong ground motion data, it is necessary to generate artificial accelerogram (Králik, 2004). 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. Thus, the simulated earthquake records must have realistic duration, frequency content, and intensity, representing the physical conditions of the site. Due to the complex nature of the formation of seismic waves and their travel path before reaching the recording station, a stochastic approach may be most suitable for generating artificial accelerogram. Earlier, stationary white noise random models for modeling earthquake ground motions were developed (Clough, 1993).

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:

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

here ξ_g and ω_g are the site dominant damping coefficient and frequency, and S_0 is the constant power spectral intensity of the bed rock excitation. The Kanai-Tajimi power spectral density function may be interpreted as corresponding to an "ideal white noise" excitation at the bedrock level filtered through the over-laying soil deposit at site. The generalized no stationary Kanai-Tajimi model is represented by the following equation:

$$\ddot{u}_f+2\xi_g\omega_g\dot{u}_f+\omega_g^2u_f=y(t) \qquad \ddot{u}_g=-(2\xi_g\omega_g\dot{u}_f+\omega_g^2u_f).e(t), \tag{2}$$

where u_f is the filtered response, ω_g is dominant ground frequency, ξ_g is the effective ground damping coefficient, is the output ground damping acceleration, and $e(t)$ is the amplitude envelope function. After numerical integration of eq. (2) can be evaluate the ground damping acceleration $\ddot{u}_g$.

To generate a synthetic ground motion accelerogram $a(t)$ compatible with a response spectrum, the following steps can be used according to (Clough, 1993) :

1. A simple time function y(t) can be established from natural accelerogram or as Gaussian distribution with zero mean value and a variance of unity. The function y(t) is a stationary Gaussian white noise process - $E[y(t)] = 0$, $E[y(t_1),y(t_2)] = 2\pi S_0\delta(t_1-t_2)$
2. A no stationary function $z(t)$ can be obtained from the stationary-type waveform $y(t)$ and the deterministic time function $f(t)$ as follows

$$z(t)=y(t).f(t), \qquad f(t)=\begin{cases}(t/t_1)^2 & \forall\, t<t_1\\ 1 & \forall\, t_1<t<t_2\\ e^{-c(t-t_2)} & \forall\, t>t_2\end{cases}, \tag{3}$$

where the values t_1, t_2 and c depends on earthquake magnitude and epicenter distance.

3. Using Fast Fourier Transformation (FFT) we can to get $Z(i\varpi)$ from the wave form $z(t)$ and the complex function $A(i\varpi)$ after the filtration of the smaller frequency than ω_2 and lower frequency than ω_1

$$Z(i\omega)=\int_{-\infty}^{\infty}z(t).e^{-i\omega t}.dt, \qquad A(i\varpi)=Z(i\varpi).H(i\varpi), \tag{4}$$

where the function $H(i\varpi)$ is modified Kanai-Tajimi filter function in the form

$$H(i\varpi)=\frac{\varpi}{\omega_2}.\frac{\left(1+2i\xi_1\dfrac{\varpi}{\omega_1}\right)}{\left(1-\dfrac{\varpi^2}{\omega_1^2}+2.i.\xi_1.\dfrac{\varpi}{\omega_1}\right).\left(1-\dfrac{\varpi^2}{\omega_2^2}+2.i.\xi_2.\dfrac{\varpi}{\omega_2}\right)}, \tag{5}$$

4. The normalized accelerogram *a(t)* will be get from the complex function *A(iϖ)* in (2.4) after the inverse FFT transformation

$$a_n(t) = \frac{1}{a_{peak}} \int_{-\infty}^{\infty} A(i\omega) e^{i\omega t}.d\omega\,, \tag{6}$$

5. The accelerogram spectrum $S_{pv}^{a}(\xi_s, T)$ can be considered as maximum acceleration response of a single degree-of-freedom (SDOF) structure under the ground acceleration

$$\ddot{u} + 2\xi_s\omega_s\dot{u} + \omega_s^2 u = -\ddot{u}_g\,, \tag{7}$$

where ωs and ξs are the fundamental frequency and the damping coefficient of the SDOF. The accelerogram spectrum is defined as $S_{pv}^{a}(\xi_s, T) = \max\{\ddot{u}(t)\}$.

6. The accelerogram spectrum $S_{pv}^{a}(\xi_s, T)$ must be compared with design spectrum $S_{pv}(\xi_s, T)$. The correlation function can be getting for frequency band or for the value of discrete frequency using in FFT method. We can described the difference area of the accelerogram spectral function and the design spectral function for interval <*T₁, T₂*>

$$\Delta J = \int_{T_1}^{T_2} (S_{pv}(\xi_s, T) - S_{pv}^{a}(\xi_s, T)).dT\,, \qquad J = \int_{T_1}^{T_2} S_{pv}(\xi_s, T).dT \tag{8}$$

If $\Delta J / J > toller$ (where *toller* is a permissible deviation), we must to start the correction process. This iteration will be finished for condition $\Delta J / J \le toller$. The average of the ratios of design spectral value, $S_{pv}(\xi_s, T)$, to response spectral value $S_{pv}^{a}(\xi_s, T)$ over each frequency band were used to multiply the real and imaginary parts of *A(iϖ)*. After this reduction are calculated next steps to get the new modified accelerogram.

These iteration processes require the control system of the convergence. We can to use the implicit method (using three last correction parameters for each frequency band $\varpi = \omega_{n+1} - \omega_n$) to consider optimal control parameter for next step.

- Linear correction (9)

$$\lambda_{i+1}^{*}(\xi, \eta, \varpi) = \left|\ (1-\xi).(1-\eta)\lambda_{i-1}(\varpi) + (1-\xi)\eta\lambda_i(\varpi) + \xi.\lambda_{i+1}(\varpi)\ \right|, \forall\, 1 \le \xi \le 2,\ 1 \le \eta \le 2$$

- Arc length correction (10)

$$\begin{aligned} 2\lambda_{i+1}^{*}(\xi, \eta, \varpi) = \big|\ & (1-\eta)(1-\xi).\lambda_{i-1}(\varpi) + 2(1-\eta)(1-\xi^2)\lambda_i(\varpi) + (1-\eta)(1+\xi).\lambda_{i+1}(\varpi) + \\ & + \eta(1-\xi^2)\lambda_{ave}\ \big| \qquad \forall\, 1 \le \xi \le 2,\ 0 \le \eta < 1 \end{aligned}$$

where λ_i is the correction factor, for which is determined follow $\lambda_i(\varpi) = J_i(\varpi) / J_i^{a}(\varpi)$ (for $J^{a} = \int_{\omega_n}^{\omega_{n+1}} S_{pv}^{a}(\xi_s, \omega).d\omega$). The second possibilities of the correction synthetic accelerogram may be realized using various frequency bands $\varpi = \zeta.(\omega_{n+1} - \omega_n)$ for the response spectral value $S_{pv}^{a}(\xi_s, \omega)$. The choice of the parameters ξ, η in Eq. (9, 10) may be determined from the condition of minimization of the solution error. If the solution is diverged we can to go back step with correction these parameter. The FORTRAN program "COMPACEL" has been developed by Králik for generate synthetic ground motion accelerogram assuming the site effect and requirement of standard (Eurocode 8 (Europe), AFPS 90 (France), ASCE 4-86 (USA), NUREG /CR-0098 (USA), DIN 49 (Germany), JSCE 92 (Japan), STN 730036 (Slovak),) and the minimal square deviation method was incorporated for using.

The requirements for the synthetic ground motion accelerogram according to standard ASCE 4-98 are following:

1. The mean of the zero-period acceleration (ZPA) values shell equal or exceed the design ground acceleration,

2. In the frequency range 0,5 to 33 Hz, the average of the ratios of the mean spectrum to the design spectrum, where the ratios are calculated frequency by frequency, shall be equal to or greater then 1.

3. No one point of the mean spectrum (from the time histories) shall be more than 10% below the design spectrum.

4. The three components of motion in the orthogonal directions shall be statistically independent (with mean correlation smaller than 0,3), and the time histories shall be different.

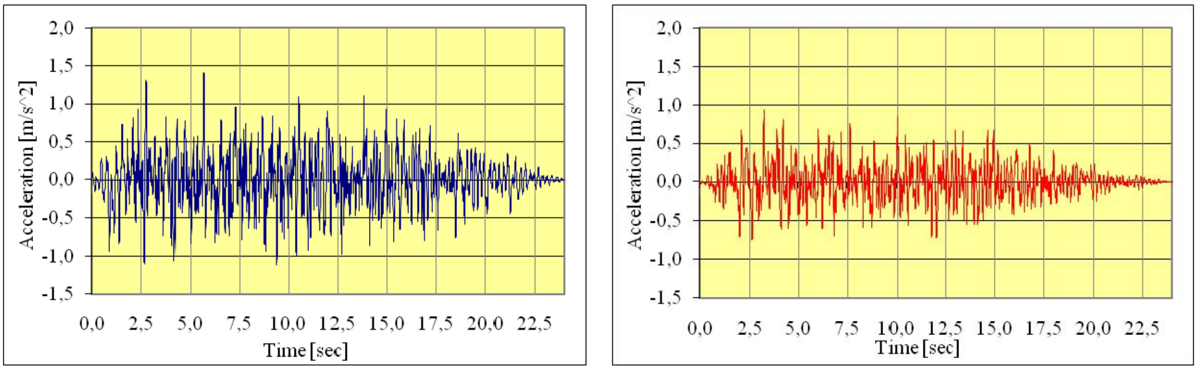


Fig. 3: 3D spectrum compatible design accelerogram Emo1c_X (Y) and Emo1c_Z

In Fig.3 the spectrum compatible synthetic accelerogram in the horizontal and vertical directions are presented (Králik, 2004). The correlation parameters between two accelerogram are equal 0,003 for Emo1c_X- Emo1c_Y, 0,004 for Emo1c_X - Emo1c_Z and 0,002 for Emo1c_Y - Emo1c_Z.

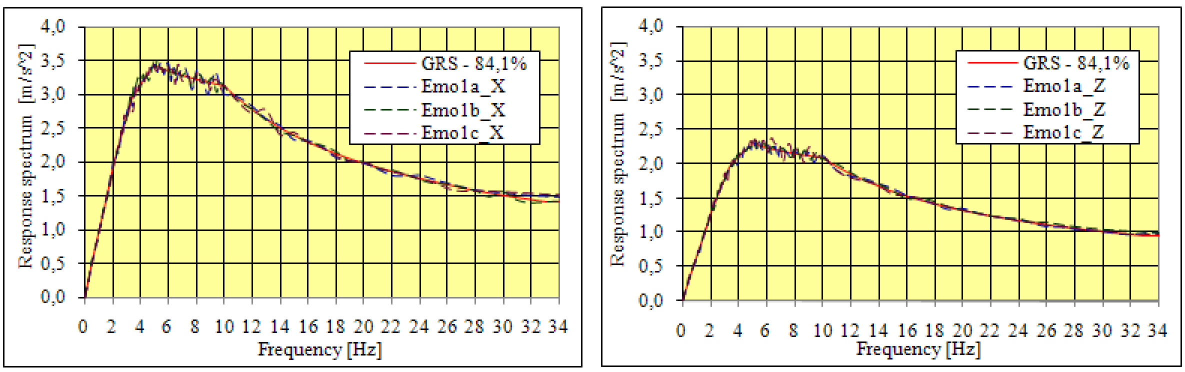


Fig. 4: Comparison of the 3 synthetic acceleration spectrums with GRS spectrum

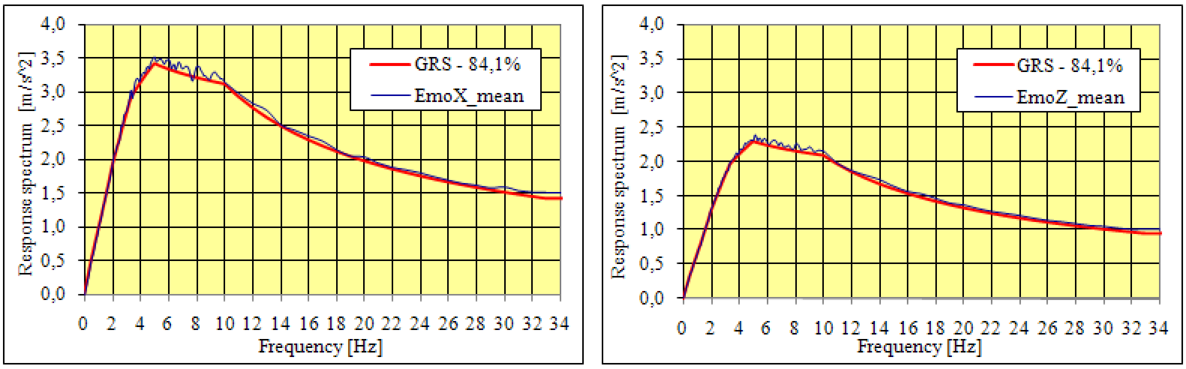


Fig. 5: Comparison of the mean synthetic acceleration spectrum with GRS spectrum

The comparison of the three synthetic accelerogram spectrum and GRS design response spectrum is demonstrated in fig.4 (Králik, 2004). All coordinates of the mean synthetic accelerogram spectrum are equal or higher as GRS design response spectrum (fig.5). The differences between mean response spectrum and design GRS spectrum are smaller than 5%.

3. Calculation model of nuclear power plant structure

The NPP (Power Block) building was discretized (Králik,J. et al., 2004) by the 3D finite elements model to obtain realistic behavior of structure. The model (TU 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.

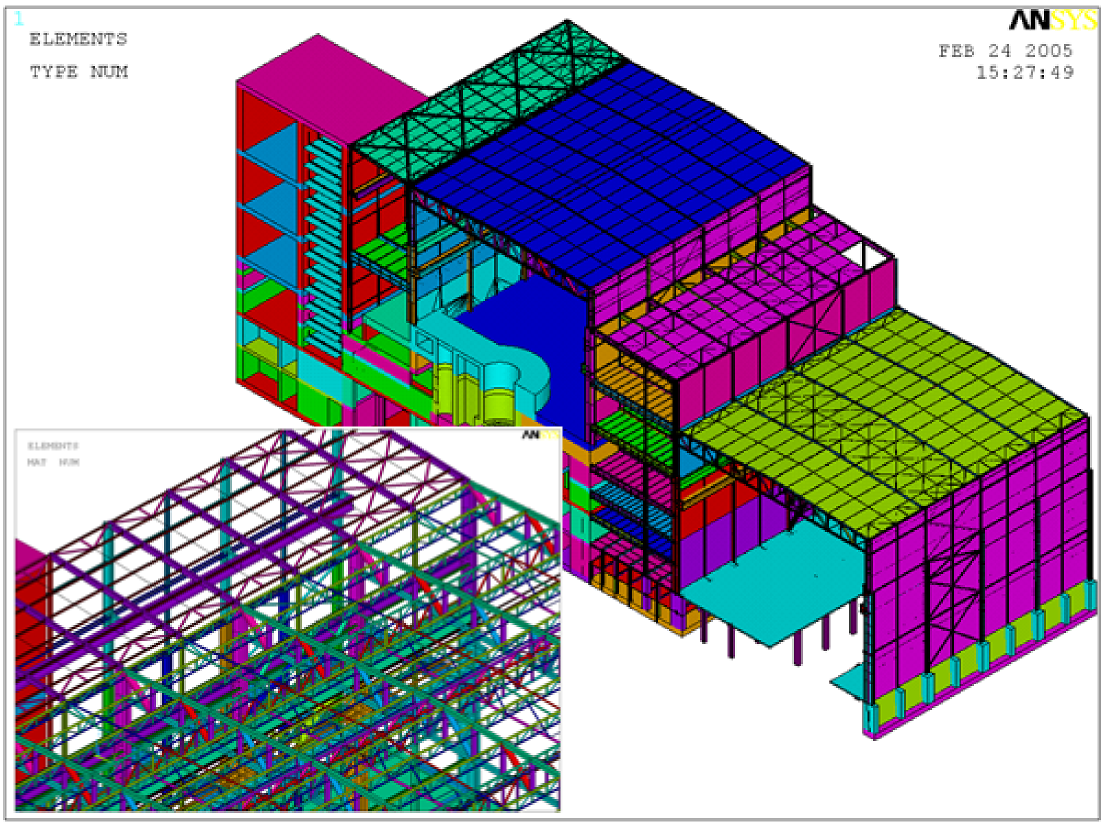


Fig.6: FEM model of NPP in Mochovce (TU Brno and STU Bratislava)

The seismic loading was considered by spectrum compatible 3D accelerograms at foundation level to response. The material damping occurring in the soil and the structure mainly involves a frictional loss of energy.

4. Comparison of the seismic evaluation methodologies

Two principal methodologies are proposed in ASCE 4-98 for seismic reevaluation of NPP structures. One method has evolved that provides a probabilistic assessment of risk due to the potential effects of earthquake. This methodology is called seismic probabilistic risk assessment (SPRA). The SPRA is an integrated process that includes consideration of the uncertainty and randomness of seismic hazard, structural response, and material capacity parameters to give a probabilistic assessment of risk.

Another methodology was specifically developed to assess the seismic margin of nuclear power plants. It is called the seismic margin assessment (SMA) methodology. It was designed to avoid frequency-of-occurrence arguments associated with the seismic hazard that have often proved highly contentious and unresolvedable. This method was designed to demonstrate margin over the design earthquake level to quantify plant safety. In contrast, the SPRA provides estimates of seismic risks of core damage and adverse public health effects.

The objective of seismic probabilistic risk assessment (SPRA) is to calculate the probability distribution of frequency of adverse consequences (i.e., core damage, containment release, and off-site consequences). In SPRA, a fragility analysis is performed to calculate the probability of failure as a function of a ground motion variable (e.g., peak ground acceleration) for the structures and components that contribute to the frequency of failure. The input to structures and equipments is median centered in SPRA, but with the corresponding valid appropriately incorporated into the analysis. In comparison, the input specified determines the NEP level in the Standard.

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 or a pre-selected 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 deterministic approach to defining the HCLPF of a component or structure is commonly referent to as the „Conservative Deterministic Failure Margin Approach“ (CDFM) and is fully explained in EPRI Report NP-6041. The CDFM approach to devote the HCLPF has three basic steps:

1. The SME will be conservatively defined so that in the frequency range of interest, in each direction, there is no more than approximately 16% probability that the response spectrum ordinate will be exceeded if the specified SME ground motion level occurs.
2. The calculation of structural and equipment response to the conservatively defined earthquake will be median centered with conservatism to cover only the uncertainties in response to maintain the 84% NEP level.
3. The assessment of capacity for the calculated response will be conservative, using approximately 95^{th} percentile exceeding material strengths, approximately 84th percentile exceeding strength prediction equations and incorporating conservative effects of structural system ductility.

Thus for SMA, the elastic computed seismic response of structures and components, i.e., Steps 1 and 2, is defined at the 84% nonexceding probability (NEP) level. The rest of the conservatism necessary to reach a HCLPF is included probably in the capacity, i.e., Step 3.

Hence, in actuality, the objectives of the two response analyses, i.e., SMA and the Standard, are essentially the same. In many respects the analysis procedures are the same; and any differences in conservatism are small.

Seismic Response Analysis – The development of models (i.e., masses, stiffness and general model arrangement) is the same for the Standard and for SMA.

In summary, the requirements in the Standard are only slightly more conservative then for SMA, which is consistent with the overall probability objectives for the two approaches as discussed above.

HCLPF parameters for structural elements							
Columns primary	Columns secondary	Vertical bracing	Horizontal bracing	Beams	Plane truss	Roof bracing	Anchors
SO 490 – Tools Hall							
0,184	0,201	0,240	-	-	0,468	0,243	-
SO 800 - Reactor Hall							
0,235	0,559	0,232	0,145	-	0,457	0,186	-
SO 800 – Ventilation Hall							
0,157	0,145	0,173	-	1,095	-	0,244	-
SO 805 Longitudal Gallery							
0,890	0,458	0,642	-	0,715	-	0,228	0,050*) 0,190
SO 806 Transversal Gallery							
0,368	-	0,235	-	0,264	-	1,008	0,190

Tab. 1: Recapitulation of the seismic resistance of the NPP structural elements

Notes - *) this value is equal to the state before upgrading

On the base of SMA methodology the seismic resistance of the NPP structure was calculated. The recapitulation of the HCLPF parameters of principal structure elements of the NPP buildings is demonstrated in tab.1. The seismic safety of NPP building after upgrading is determined by the seismic resistance of the gallery anchors and secondary columns of the ventilating hall.

5. Floor response spectrum

Interior constructions and technological components must be analyzed using decoupled model or a coupled structure-subsystem model. The analysis can be performed using time history analysis or response spectrum method (ASCE 4/98, NUREG 0800, 2007). The most popular is the response spectrum method using the FRS (floor response spectrum).

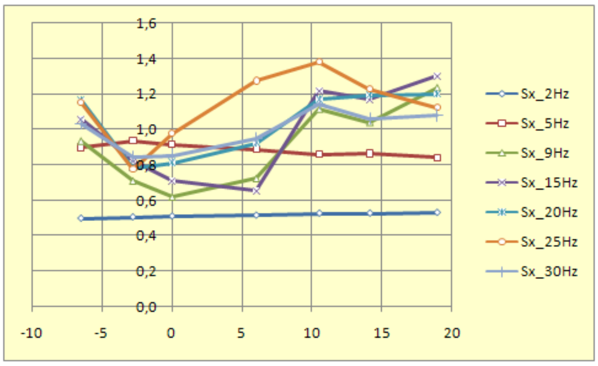
The FRS can be calculated from the in-structure spectra on the base of deterministic and semiprobabilistic methods (NUREG 0800, 2007). The results from the calculation of FRS in all NPP buildings show, that in the case of the floor with the higher variability of distributed masses and slab stiffness, the FRS values are more conservative as the envelope of maximum spectrum values in various points of floor

Deterministic method to generate FRS is defined in NUREG 0800 and IAEA rep.No28. The response spectrum in the points of floor is calculated using the transient analysis of the structure from synthetic 3D accelerograms (median+sigma). One or three 3D accelerograms can be used as input loads. The material properties are calculated with median values (best estimation). The damping values are requirement in ASCE 4/98 (max.7% for steel and 10% for concrete structures and rock soil). The floor response spectrum may be calculated as envelope of maximum of mean spectrum values in each typical point (minimum 5 points are recommended).

Probabilistic method to generate FRS is based on the statistical methods considering the uncertainties as a seismic risk, soil structure interaction, material properties, calculation model and other... The response spectrum in the points of floor is calculated using the transient analysis of the structure from group of synthetic 3D accelerograms. In the case of the rock soil the median and variation of the input accelerograms must be equivalent to values of seismic risk for this locality. The FRS may be calculated as statistical envelope (median+sigma) of the spectrum values in all typical point. The statistical characteristic of input and output parameters are investigated for lognormal distribution.

Semiprobabilistic method to generate FRS is based on the combination of deterministic and probabilistic methods. The total variation is calculated from the variation of the input loads, material properties and response in various typical points on the floor.

The uncertainties of the soil-structure interaction effects and calculation model can be considered by broadening and lowering of in-structure time history motion n accordance of requirements of standard ASCE 4/98 (1998).



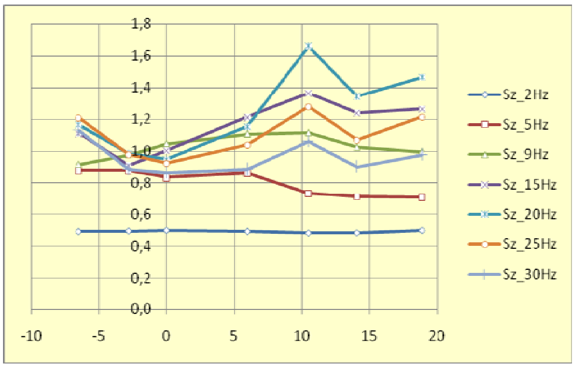


Fig.7 Comparison of ratio GRS/RLE spectrum in the Box PG NPP on level from -6,5m to 20m

6. Conclusion

This paper presented the deterministic and probabilistic methodology to analysis the seismic resistance of NPP in Mochovce site in Slovakia (Králik, 2003 and 2004). The generation of the seismic loads on the base of probabilistic seismic risk analysis was described. The paper presented the calculation methods for generation FRS in the NPP buildings in accordance of international standard NUREG 0800 (2007). The deterministic and probabilistic methods were discussed. The seismic load in the form of RLE and GRS acceleration spectrum was defined by GFU SAV (Labák, 1998). The synthetic spectrum compatible accelerogram was generated in program COMPACEL created by Králik (1999, 2003). The seismic reevaluation of the NPP structures was realized on the base of the SMA methodology. The results from this analysis present the level of the seismic resistance of the structures.

Acknowledgement

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