SEISMIC ANALYSIS OF THE SOIL-STRUCTURE INTERACTION CONSIDERING THE LOCAL SITE EFFECTS

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Abstract. *The paper presents the results of analysis of the soil-structure interaction effects considering the local site effects. The influences of the local site effects – masses, stiffness and thickness of the layered subsoil - can significantly modify the stresses and deflections of the structural system. Two important characteristics that distinguish the dynamic soil-structure interaction system from other general dynamic structural systems are the unbounded nature and the plastic strains of the soil medium. Generally, the influence of the layered subsoil will be considered to define of the seismic loads and the stiffness and damping of the complex system soil-base-structure behaviour.*

Keywords

Seismic, Safety, SSI, Nuclear Power Plants, ANSYS.

1. Introduction

After the accident of nuclear power plant (NPP) in Fukushima the IAEA in Vienna adopted a large-scale project "Stress Tests of NPP", which defines new requirements for the verification of the safety and reliability of NPP. Based on the recommendations of the ASCE standard and IAEA in Vienna [1- 8], the effective seismic resistance of objects is assessed in PGA sites up to 0.3g according to the "Seismic Margin Assessment" methodology (SMA) [2]. The required methodology was based on a reference earthquake (RLE) or a "Seismic Margin Earthquake" (SME) earthquake, which is an earthquake with seismological parameters of a given site and response spectrum at the free terrain level corresponding to 84.1% probability. The Peak Ground Acceleration (PGA) is defined for a probability of no-exceedance in 10^4 years. The design response spectra were prepared based on results of the PSHA (Probabilistic Seismic Hazard Analysis) study for the NPP J. Bohunice site developed by GFÚ SAV [7].

These design response spectra were generated based on the results of the seismotectonic evaluation of the site for the bedrock level. Based on the recommendations of the IAEA standard, it is necessary to take into account the effect of soil-structure interaction (SSI) and thus to define the seismic load on the free field level after taking into account the soil properties in the subsoil based on the geological investigation of this locality and at the level of the bedrock.

The influence of the layered subsoil on the seismic loading of the nuclear fuel storage (NFS) building was considered in the design project.

The analysis and the corresponding documentation must be carried out in 5 stages:

1. Evaluation of the input data from the point of view of complexity, reality and acceptation by EN and IAEA standard requirements for static and dynamic analysis of the safety and reliability of NPP structures.

2. Definition of the local response spectra and three-component synthetic accelerograms compatible with the response spectra at the foundation level and the bedrock under the building considering the strain - stress in the soil layers during earthquakes.

3. Dynamic analysis of the soil-structure interaction based on the previous geological and seismotectonic analysis of the locality.

4. Static analysis of the building settlement.

5. Design analysis of the seismic resistance of NFS structures in accordance with international standards and IAEA recommendations.

2. Nuclear fuel storage building

The NFS building will be located in the NPP areal in Jaslovske Bohunice. The building consists of a reinforced concrete block and steel hall (Fig. 1).

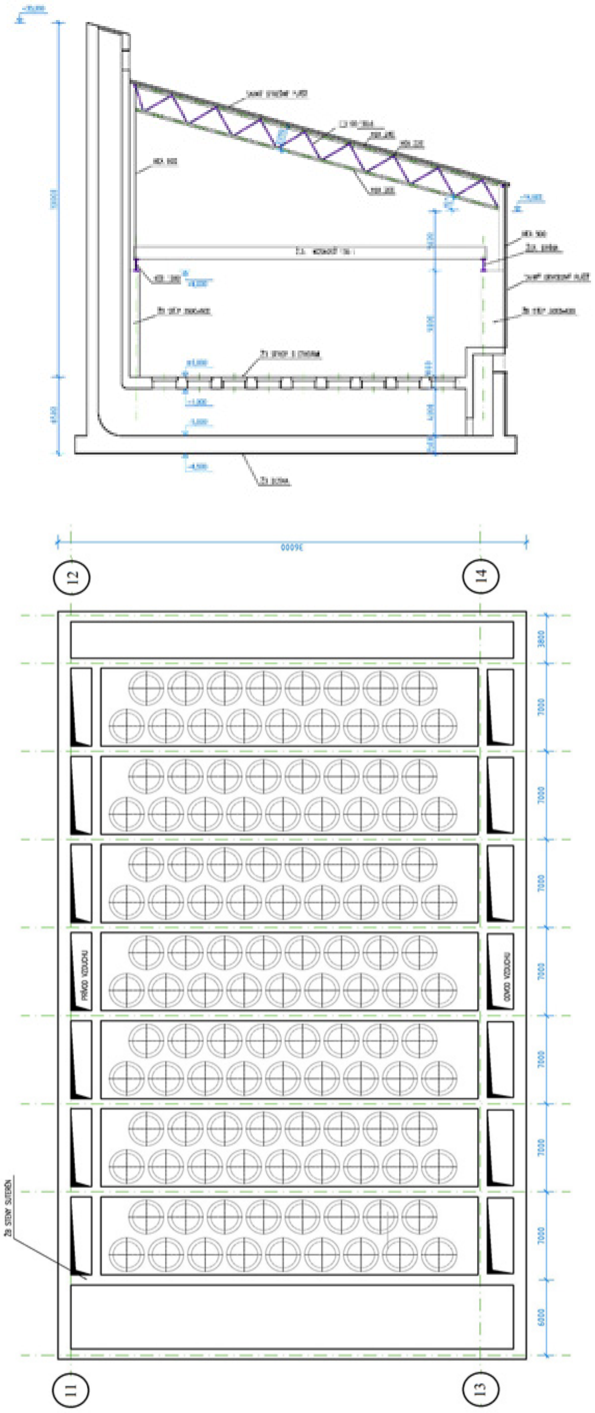

Fig. 1: The plane and cross section of NFS building.

Storage capacities are projected in 7 modules by 7 m. These e dimensions of the building in the floor plan are 58.8 m x 36 m. The height level of the hall is proposed asymmetrically from 14 m to 30 m. The building is based on a solid reinforced concrete plate of 1.5 m thick. The foundation level is at a depth of -6.5 m. The storage ceiling is at a level of ± 0.0 m. The total weight of the NFS structure with the dead load from the technology is 35487.64 t. The stress at the level of the foundation joint is 164.57 kPa. The center of gravity of the object has the following coordinates $Y_T = 14.76$ m and $X_T = 32.2$ m. The eccentricity of the location of the gravity center with respect to the gravity center of the foundation plate is $e_x = 3.24$ m and $e_y = 2.25$ m.

3. Seismic design loads

To provide input excitations to structural models for sites with no strong ground motion data, it is necessary to generate artificial accelerograms. 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 accelerograms. Earlier, stationary white noise random models for modeling earthquake ground motions were developed [9-12].

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:

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

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 nonstationary Kanai-Tajimi model is represented by the following equations:

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

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

The International Standard for Nuclear Power Plants ASCE 4/98 [1] defines the following requirements for a synthetic accelerogram compatible with the response spectrum as follows:

1. The mean acceleration value with zero period (ZPA) must be equal to or higher than the design acceleration at free field level.

2. In the frequency range 0.5 to 33 Hz, the average of the ratios of the middle spectrum to the design spectrum, where the ratios are calculated for each frequency band, must be equal to or greater than 1.

3. No point of the average spectrum guide (using multiple accelerograms) shall be more than 10% below the design spectrum value.

4. The three components of motion in orthogonal directions must be statistically independent (with a mean correlation less than 0.3) and the time periods must be different.

Two basic methods are used to generate a synthetic accelerogram compatible with the response spectrum [12]

- Decomposition into harmonic series,

- Fourier transform.

To generate a synthetic ground motion accelerogram $a(t)$ compatible with a response spectrum, the following steps can be used according to [4]:

1. A simple time function y(t) can be established from natural accelerograms or as gaussian distribution with zero mean value and a variance of unity. The function y(t) is a stationary Gaussian white noise process -

$$E[y(t)] = 0,\ E[y(t_1), y(t_2)] = 2\pi S_0 \delta(t_1 - t_2) \qquad (3)$$

2. A nonstationary function z(t) can be obtained from the stationary-type waveform y(t) and the deterministic time function f(t) as follows

$$z(t) = y(t).f(t), \qquad f(t) = \begin{cases} (t/t_1)^2 & \forall\, t < t_1 \\ 1 & \forall\, t_1 < t < t_2\,, \\ e^{-c(t-t_2)} & \forall\, t > t_2 \end{cases} \qquad (4)$$

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

3. Using Fast Fourier Transformation (FFT) we can 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

$$Z(i\omega) = \int_{-\infty}^{\infty} z(t).e^{-i\omega t}.dt\,, \quad A(i\varpi) = Z(i\varpi).H(i\varpi) \qquad (5)$$

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

$$H(i\varpi) = \frac{\varpi}{\omega_2}.\frac{\left(1 + 2i\xi_1 \dfrac{\varpi}{\omega_1}\right)}{\left(1 - \dfrac{\varpi^2}{\omega_1^2} + 2.i.\xi_1.\dfrac{\varpi}{\omega_1}\right).\left(1 - \dfrac{\varpi^2}{\omega_2^2} + 2.i.\xi_2.\dfrac{\varpi}{\omega_2}\right)}\,. \qquad (6)$$

4. The normalized accelerogram $a(t)$ will be got from the complex function $A(i\varpi)$ in (5) after the inverse FFT transformation

$$a_n(t) = \frac{1}{a_{\mathrm{peak}}} \int_{-\infty}^{\infty} A(i\omega) e^{i\omega t}.d\omega\,. \qquad (7)$$

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

$$\ddot{u} + 2\xi_s \omega_s \dot{u} + \omega_s^2 u = -\ddot{u}_g\,, \qquad (8)$$

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

6. The accelerogram spectrum $S_{\mathrm{pv}}^{\mathrm{a}}(\xi_s, T)$ must be compared with design spectrum $S_{\mathrm{pv}}(\xi_s, T)$. The correlation function can be obtained for frequency band or for the value of discrete frequency using in FFT method. We can describe the difference area of the accelerogram spectral function and the design spectral function for interval <T_1, T_2>

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

$$J = \int_{T_1}^{T_2} S_{\mathrm{pv}}(\xi_s, T).dT \qquad (9)$$

If $\Delta J / J > toler$ (where "*toler*" is a permissible deviation), we must start the correction process. This iteration will be finished for condition $\Delta J / J \le toler$.

The average of the ratios of design spectral value $S_{\mathrm{pv}}(\xi_s, T)$ to response spectral value $S_{\mathrm{pv}}^{\mathrm{a}}(\xi_s, T)$ over each frequency band were used to multiply the real and imaginary parts of $A(i\varpi)$. The convergence of the solution is checked in the iteration process on base of the implicit solution method from the last three steps as follows

- Linearized method

$$\lambda_{i+1}^{*}(\alpha, \beta, \varpi) = \\ = \left| (1-\alpha)(1-\beta)\lambda_{i-1}(\varpi) + (1-\alpha)\beta\lambda_i(\varpi) + \alpha\lambda_{i+1}(\varpi) \right| \\ \forall\, 1 \le \alpha \le 2, \quad 1 \le \beta \le 2\,. \qquad (10)$$

- Arc-length method

$$2\lambda_{i+1}^{*}(\alpha, \beta, \varpi) = \\ = \left| \begin{matrix} (1-\alpha)(1-\beta)\lambda_{i-1}(\varpi) + 2(1-\alpha^2)(1-\beta)\lambda_i(\varpi) + \\ + (1+\alpha)(1-\beta)\lambda_{i+1}(\varpi) + (1-\alpha^2)\beta\lambda_{\mathrm{ave}}(\varpi) \end{matrix} \right| \\ \forall\, 1 \le \alpha \le 2, \quad 0 \le \beta \le 1\,, \qquad (11)$$

where λ_i is the correction factor that is defined in the form

$\lambda_i(\varpi) = J_i(\varpi)/J_i^a(\varpi)$ for $J_i^a(\varpi)\int_{\omega_n}^{\omega_{n+1}} S_{pv}^a(\xi_s,\omega)\,d\omega$.

Another way to correct a synthetic accelerogram compatible with the response spectrum is to modify the frequency band $\varpi = \delta(\omega_{n-1} - \omega_n)$ to calculate the spectral values $S_{pv}^a(\xi_s,\omega)$.

Presented iteration process is not explicit converged. We used the implicit method (using three last correction parameters for each frequency band) to consider optimal parameter for next step. If the solution is diverged, we can take a step back with correction of this parameter. The FORTRAN program "COMPACEL" has been created by author [6, 10] to generate synthetic ground motion accelerograms 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 applied.

4. Seismological evaluation of the site

The initial seismological data for the NPP EBO site were prepared in the years 1969 - 1970 in accordance with the standard ČSN 730036 - Seismic loading of buildings. Subsequently, a special study "Geological history, tectonic development and seismicity of J. Bohunice" (06/1970) was prepared, which specified the seismicity of NPP EBO site so that the most probable earthquake in J. Bohunice may be an earthquake with a degree of 6 - 6.5º MCS , corresponding to a value of 4.2 on the Richter scale (PGA = 0.025g). To design of NPP structures, the minimum value of PGA = 0.10g was considered in the design project in accordance with the IAEA recommendations.

As part of the seismic risk assessment by the Czecho-slovak government commission, the basic characteristics for the maximum computational earthquake with a probability of once every 10 000 years and an intensity of 8º MSK - 64 to PGA = 0.25g in the horizontal direction and PGA = 0.13g in the vertical direction were determined in 1995.

In the years 1994-1998, the GFÚ SAS Bratislava [7] implemented a project of probabilistic analysis of the seismic risk of NPP EBO site. Based on the probabilistic analysis of the seismic hazard of NPP EBO site, a new seismic assignment was determined for NPP buildings, which is based on the requirements of the SMA boundary seismic resistance methodology [4-6]. The seismic load is defined by the value of the acceleration peak at the PGA free field level for a return period of 10^4 years and the design acceleration response spectrum (GRS) for the SL-2 earthquake level (84% NEP) and 5% attenuation.

For comparison, we present the original and new PGAs for horizontal and vertical excitation in J. Bohunice locality for a return period of 10^4 years.

- horizontal acceleration

$PGA_{HiRLE} = 0.250g$ and $PGA_{HRLE\text{-}SL\text{-}2} = 0.344g$

- vertical acceleration

$PGA_{ViRLE} = 0.130g$ and $PGA_{VRLE.SL\text{-}2} = 0.214g$

A comparison of PGA values shows that its value increased by about 38%.

Tab.1: Acceleration response spectrum RLE for 5% damping [7]

Frequency [Hz]	Horizontal spectrum [g]	Vertical spectrum [g]
0.50	0.127	0.054
0.67	0.182	0.073
1.00	0.287	0.107
1.33	0.387	0.141
2.00	0.562	0.196
3.33	0.761	0.312
5.00	0.800	0.423
10.00	0.621	0.503
33.30	0.344	0.214
50.00	0.344	0.214

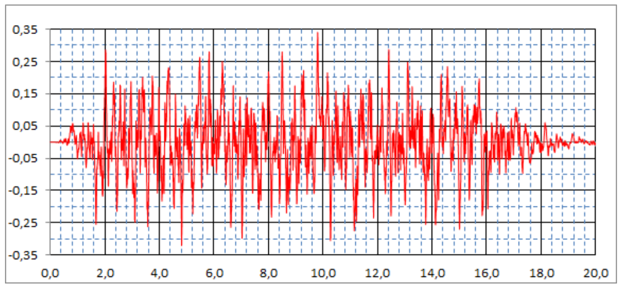

Fig. 2: The spectrum compatible synthetic accelerogram $a_x(t)$.

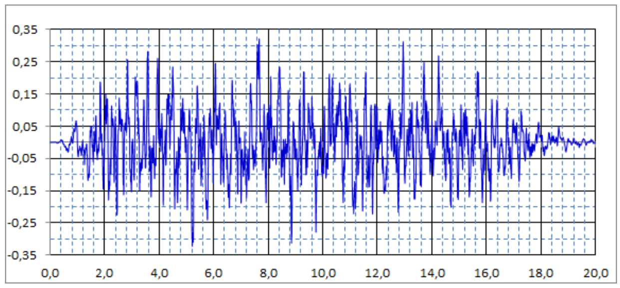

Fig. 3: The spectrum compatible synthetic accelerogram $a_y(t)$.

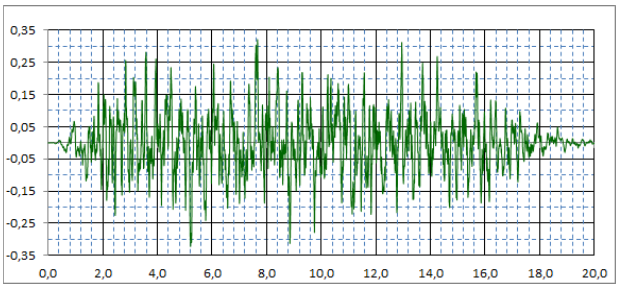

Fig. 4: The spectrum compatible synthetic accelerogram $a_z(t)$.

The values of seismic motion for SNF are taken from the report Labák & Moczo 1998 [7]. The horizontal and vertical response spectra for SL-2 are shown in the graphs below, and the values of these response spectra are summarized in the Tab.1. These spectra were used as input data for the generation of a synthetic three-component accelerogram (acceleration time course).

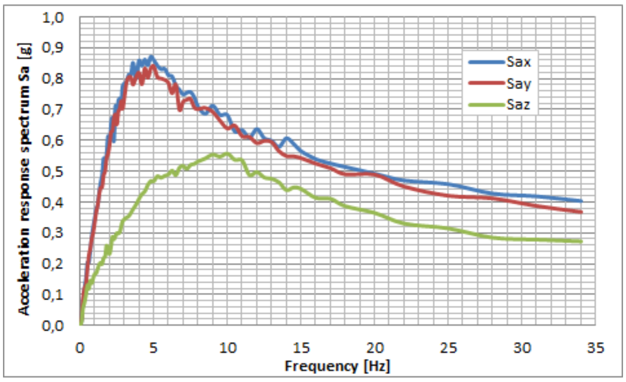

Fig. 5: The response spectrum from the synthetic accelerograms in direction *X*, *Y* and *Z* for 5% damping.

The RLE horizontal and vertical response spectra are presented in table 1. These spectra were used as inputs for generating synthetic three-component accelerograms. The values of the design response spectra were calculated from the generated synthetic three-component accelerograms (see Figs. 2-4) for the NFS building. The shape of the response spectra generated from the synthetic three-component accelero-grams is compatible with the RLE design spectrum considering the ASCE [1] and IAEA criteria [3].

5. Geophysical subsoil properties of the locality

For seismic analyses, it was recommended to use the seismic assignment for the nuclear fuel storage NFS at J. Bohunice site according to data in the feasibility study.

Tab.2: Geological profile under the building NFS.

Depth [m]	v_s[m/s]	ρ [t/m3]	G [kPa]
-6.5	151.27	2.00	45772.57
-6.9	219.19	1.98	95109.63
-15.0	296.20	2.06	180520.81
-18.5	393.88	2.10	326353.47
-24.9	450.94	2.18	443026.98
-28.7	485.80	2.00	471246.48
-34.0	532.33	2.21	626168.07
-41.8	616.59	2.24	851092.88
-50.0	668.83	2.06	923062.10
-60.0	700.99	2.09	1025237.97
-70.0	736.32	2.11	1144888.59
-80.0	771.66	2.14	1272439.32
-90.0	807.00	2.16	1408079.24
-100.0	842.33	2.19	1551997.45
-110.0	860.00	2.20	1627120.00

Dynamic soil characteristics were obtained with sufficient accuracy from the refractive and reflexive survey of a given site [4]. Depending on the propagation rates of the longitudinal and transverse waves in the soil, we can determine its physical characteristics.

The basic rigid parameter characterizing the earth body for dynamic calculations is the dynamic G_{dyn}

$$G_{dyn} = v_s^2\rho,\quad v_{dyn} = \left(v_p^2 - 2v_s^2\right)/\left[2\left(v_p^2 - v_s^2\right)\right] \qquad (12)$$

where ρ is the soil density, v_s - the velocity of the shear waves propagation in the respective earth (layer), v_p is the velocity of the longitudinal waves.

6. Stiffness and damping soil parameters in the subsoil

In the case of earthquakes, there is a large movement of the soil, and because of plastic deformation, the value of the dynamic soil module also drops [13-16]. According to IAEA recommendations, this reduction will maximally reach 65% of the dynamic module measured for small seismic events. The process of the shear modulus and the damping can be seen in Fig. 6 and 7 depending on the shear strain [4].

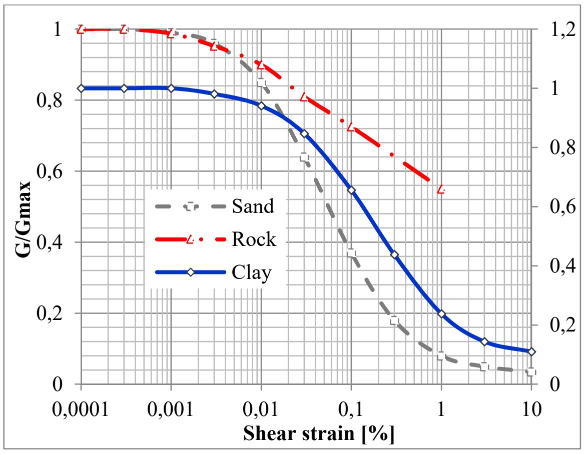

Fig. 6: Shear modulus *G* dependence on the shear strain γ.

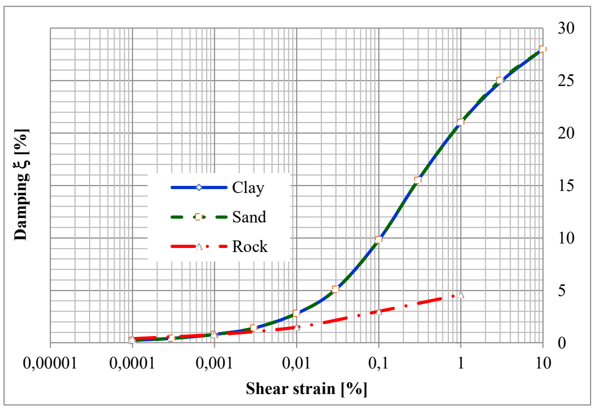

Fig. 7: Damping values ξ dependence on the shear strain γ.

Tab.3: Relative damping and shear velocity depended on PGA values.

Acceleration	Relative values of the physical quantities		
PGA [ms^{-2}]	Prop.damping ξ	Shear velocity $v_s/v_{s.max}$	Shear modulus G_{dyn}/G_{max}
0.1g	0.03	0.9(±0.07)	0.80(±0.10)
0.2g	0.06	0.7(±0.15)	0.50(±0.20)
0.3g	0.10	0.6(±0.15)	0.33(±0.20)

7. Seismic hazard considering site effects

The methodology for analyzing the influence of the layered subsoil of type 3 (for v_s < 300 m/s) according to the requirements of the IAEA NS-G-3.6 was used to define the seismic load for the nuclear fuel storage. The seismic load RLE was defined assuming that seismic waves are transformed from the source to the site in a rock bed (for v_s > 1100m/s).

Local design acceleration spectra were calculated considering the SSI effects used the SHAKESI program in accordance with the recommendations of the standards IAEA [2] and U.S. NRC.

Therefore, the methodology for calculating local design spectra is based on the following assumptions:

- PGA values for RLE seismicity were determined for the free field assuming the rigidity of the bed of the corresponding to the rock subsoil (for v_s > 1100m/s).
- The response spectrum acceleration for SL-2 [3] were defined based on a probabilistic analysis of the site effects.
- Synthetic 3D accelerograms compatible with response spectra were generated in accordance with the requirements [3].

Based on these input data, the calculation of local design spectra, considering the real geological composition at the location of the SVP object, is carried out in the following steps:

- Calculation of the synthetic accelerograms on the base at level -100m from the free level in accordance with IAEA [3] standards.
- Calculation of the local synthetic accelerograms and the design response spectra at level of foundation (-6.5m) and at level of the pile foundations (-18.5m) from the excitation synthetic accelerations using the program SHAKESI for original and modified geological conditions.
- Calculation of the smoothed design spectra at foundation level (-6.5m) and pile level (-18.5m) than the median values and the statistical envelope for 84.5% probability of failure is based on previous analyses for characteristic excitation frequencies.

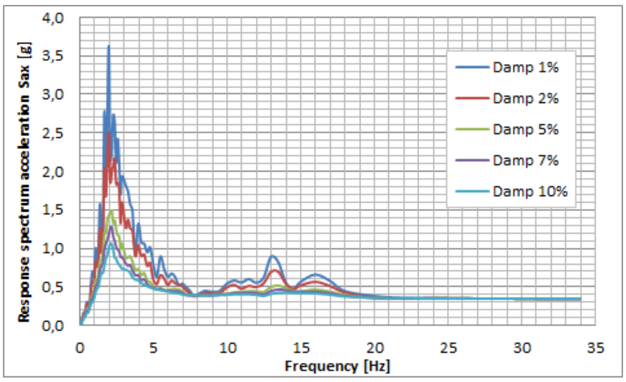

Fig. 8: The acceleration response spectrum S_{ax} at level -6.5m.

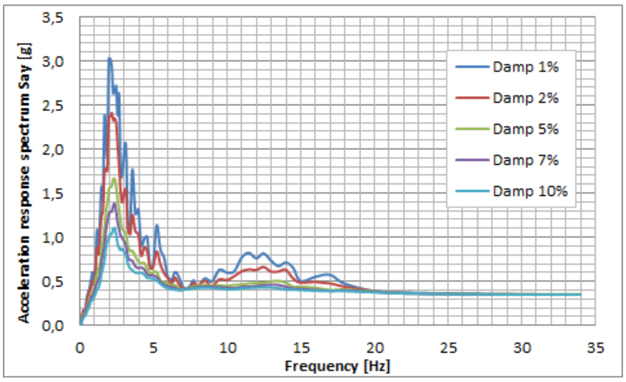

Fig. 9: The acceleration response spectrums S_{ay} at level -6.5m.

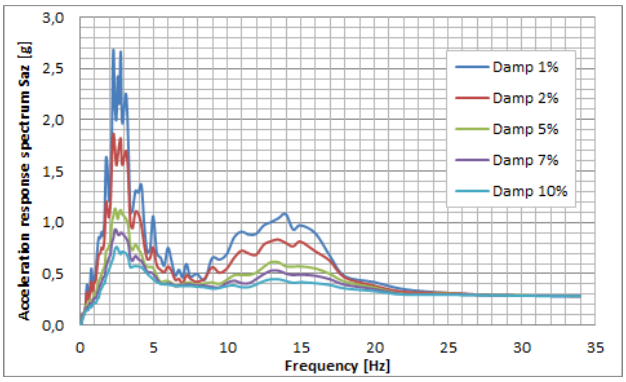

Fig. 10: The acceleration response spectrum S_{az} at level -6.5m.

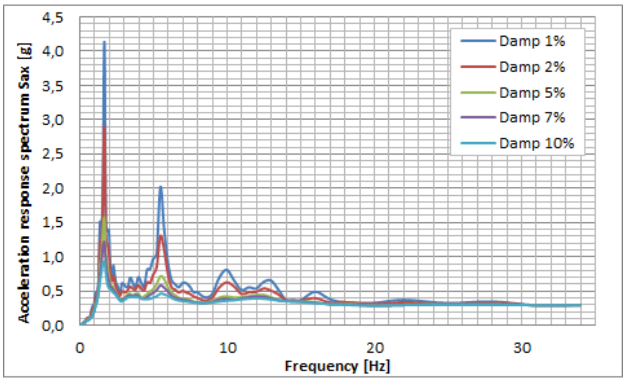

Fig. 11: The acceleration response spectrum S_{ax} at level -18.5m.

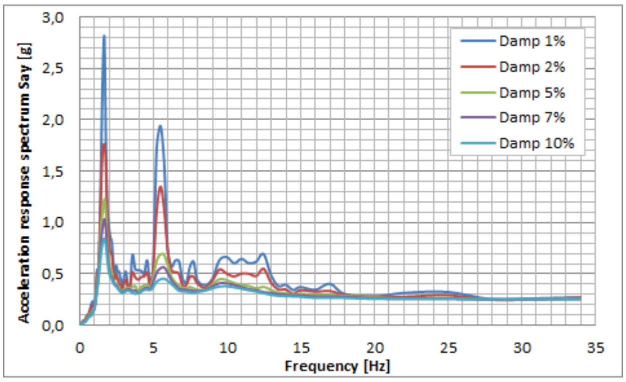

Fig. 12: The acceleration response spectrum S_{ay} at level -18.5m.

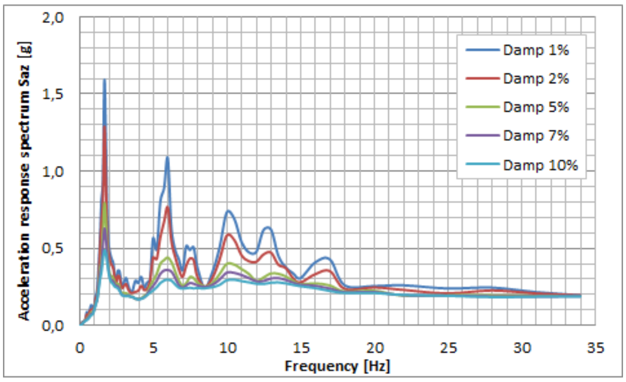

Fig. 13: The acceleration response spectrum S_{az} at level -18.5m.

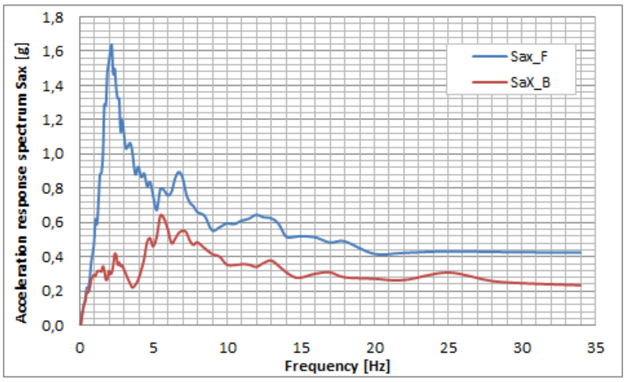

Fig. 14: The comparison of the response spectrum S_{ax} at level free field and bedrock in *X*-direction at level -100m.

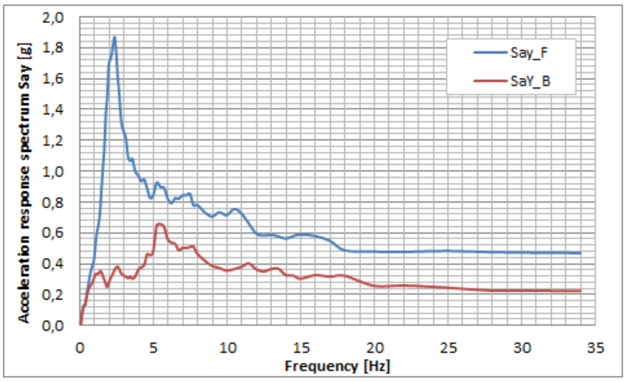

Fig. 15: The comparison of the response spectrum S_{ay} at level free field and bedrock in *Y*-direction at level -100m.

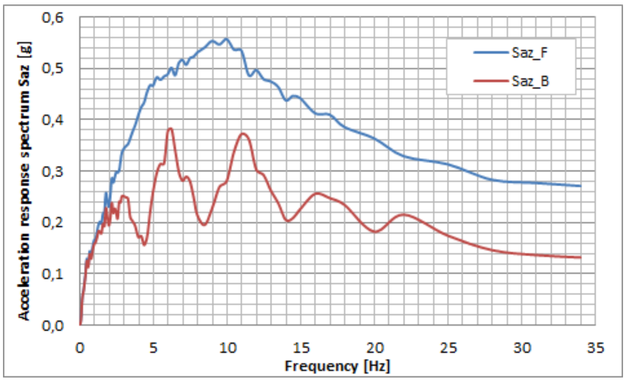

Fig. 16: The comparison of the response spectrum S_{az} at level free field and bedrock in *Z*-direction at level -100m.

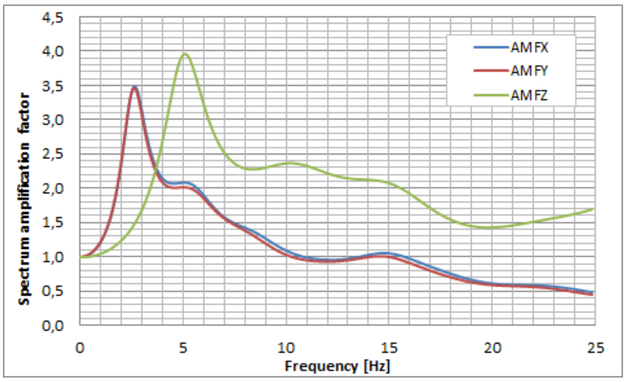

Fig. 17: The amplification factors *AMFX*, *AMFY* and *AMFZ* between the base and free field – SHAKESI.

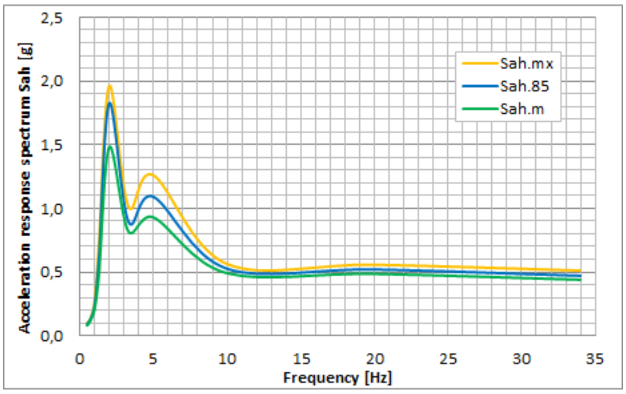

Fig. 18: The smoothing horizontal response spectra at level -6.5m.

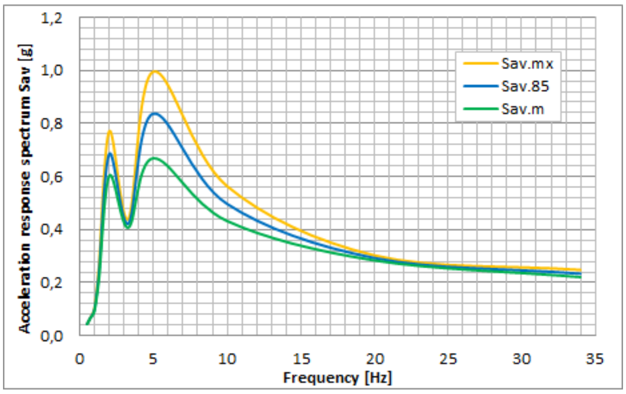

Fig. 19: The smoothing vertical response spectra at level -6.5m.

For these analyses, the modified SHAKESI [4] program was used to determine a best estimate of the transformation of ground motion from free field to base level for the 1D model of the subsoil.

The smoothing response spectra were calculated in program SHAKESI at defined frequencies recommended by IAEA standards.

The acceleration response spectra S_{ax} S_{ay} and S_{az} for direction *x*, *y* and *z* at level of foundation plate (-6.5m) are presented in figures 8-10 and at level of the pile foundations (-18.5m) are presented in figures 11-13 for various level of damping. The comparison of the response spectra at level free field and bedrock at level -100m are presented in figures14-16.

Tab.4: Comparison of the global and local response spectrum on original subsoil.

Acceleration response spectrum for 5% damping [m/s^2]				
Horizontal accelerations				
	Base level		Free field level	
Frequency [Hz]	RLE	Local	RLE	Local
0.5	0.025	0.059	0.050	0.087
2	0.173	0.159	0.364	1.082
5	0.319	0.293	0.837	1.285
10	0.229	0.240	0.780	0.782
33	0.140	0.136	0.367	0.574

We can see that the spectral value in the horizontal directions increase significantly at frequency intervals 2 – 10Hz, which are the dominant frequencies of the NFS structure. The amplification factors between the base level and free field calculated in software SHAKESI are presented in fig. 17. The peaks of the amplification factors are in the interval 2-3Hz. The results of the dynamic analysis of the influences of the local site effects are documented in tab. 4 and 5. The value of the accelerations increase about 3 times in comparison with RLE spectrum.

Tab.5: Comparison of the global and local response spectrum on original subsoil.

Acceleration response spectrum for 5% damping [m/s^2]				
Vertical accelerations				
	Base level		Free field level	
Frequency [Hz]	RLE	Local	RLE	Local
0.5	0.015	0.043	0.050	0.086
2	0.085	0.061	0.364	1.048
5	0.199	0.167	0.837	0.815
10	0.215	0.242	0.780	0.599
33	0.103	0.096	0.367	0.531

8. Conclusions

This paper describes the soil-structure interaction effects in the case of the nuclear fuel storage building during earthquake excitation. The methodology of the calculation of the soil-structure interaction effects was presented. The local design acceleration spectra were calculated considering the SSI effects used the SHAKESI program in accordance with the recommendations of the standards IAEA. This paper presented that the consideration of the local site effects based on the experimental investigation of the subsoil properties is very important from the point of view of the safety and reliability NPP structures. On base of this analysis the project of NFS building was recalculated and modified.

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