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Safety assessment of the seismic resistance of nuclear power plant technology

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Abstract. This paper gives the methodology to get the floor response spectra of the buildings on the base of the results of the probabilistic assessment. 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 10^4 years. The methodology of the seismic resistance of the cable support structures is described.

1 Introduction

One from the most importance technology segments in nuclear power plant (NPP) are the cable way structures. The effective supported structures were the prefabricated steel structures of OBO Bettermann Company. This paper presents the methodology and the results of the numerical and experimental verification of the seismic safety of these structures used in Mochovce NPP buildings.

The earthquake resistance analysis of NPP buildings in Mochovce were based on the recommends of International Agency of Energy Atomic (IAEA) in Vienna [1, 2] to get international safety level of nuclear power plants. Three logical possibilities of the source zones were defined for Mochovce site – contact of Eastern Alps and Western Carpathians, Dobra Voda and alternative fault. The seismic load was defined by PGA (Peak Ground Acceleration) and local seismic spectrum in dependence on magnitude and distance from source zone of earthquake [2]. Two principal methods are appropriate for assessing the seismic safety of facilities, the seismic margin assessment (SMA) method and the Seismic Probabilistic Safety Assessment (SPSA) method. The review level earthquake (RLE) is very effective method to reevaluation of NPP safety then the new seismic hazard is defined [2, 3, 4]. The SMA methodology is based on the RLE determination. The first seismic reevaluation of the seismic hazard of the Mochovce site was established on the deterministic method and seismic-tectonical consideration of the Mochovce site was analyzed on the base of probabilistic method. The design response spectra were prepared based on results of the probabilistic seismic hazard analysis (PSHA) study for the NPP Mochovce site developed by GFÚ SAV [2, 5].

Basic parameters of the design ground response spectrum (GRS) are as follows:

- Spectral shape MDE (SL-2) (Maximum Design Earthquake);
- It corresponds to 84% NEP (Non Exceedence Probability) (i.e. median + 1 sigma);

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- Seismic event with annual recurrent frequency 10^{-4};
- PGA (Peak Ground Acceleration) for horizontal direction $PGA_H = 0.150g$;
- PGA for vertical direction $PGA_V = 0.100g$;
- Vertical response spectra defined as 2/3 of horizontal response spectra.

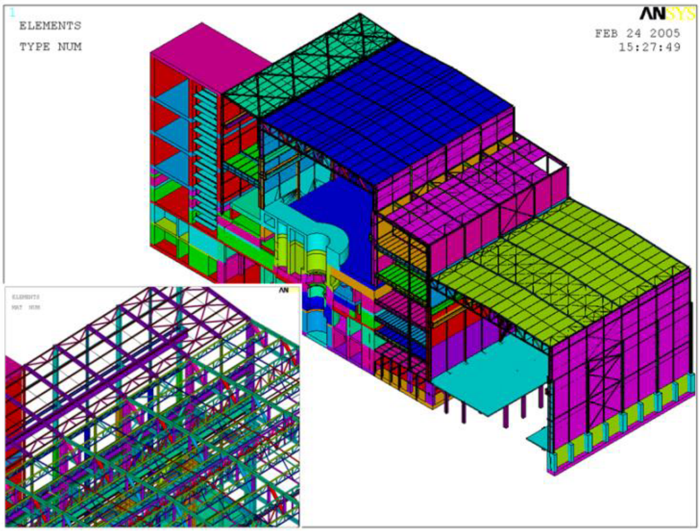

Fig. 1. Calculation model of NPP V2.

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 $PGA_{HS} = 0.143g$). The last PGA for horizontal direction was defined as $PGA_H = 0.150g$ on base of the seismic-tectonic investigation of Mochovce NPP site.

Mochovce NPP structures are typically the same as the NPP structures for reactor VVER 440. Around of the reinforced concrete structures of the box SG and bubbler tower are the steel structures of the reactor hall, longitude and transversal electrical buildings are built. The calculation model of NPP structures (Fig. 1) created by VUT Brno and modified by STU Bratislava has 161 856 elements and 135 629 nodes [2, 6]. The foundation of the reactor building is embedded into the rock subsoil

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. 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 [2, 3, 5 - 13].
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 [14]

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

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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:

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

where u_f is the filtered response, ω_g is dominant ground frequency, ξ_g is the effective ground damping coefficient, $\ddot{u}_g$ 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 $\ddot{u}_g$.

On the base of the methodology defined in [14] the software COMPACEL was created for the modified iteration processes and requirements of various standards [2].

In the case of the NPP structures the requirements of the standard ASCE 4/86 [3] are acceptable by Slovak nuclear authority. These requirements 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 than 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).

To provide input excitations to structural models for sites with no strong ground motion data it is necessary to generate more than one response spectrum compatible accelerogram. On the base of the US standards [3] requirements three synthetic design accelerograms compatible with the GRS response spectrum were generated by software COMPACEL [2].

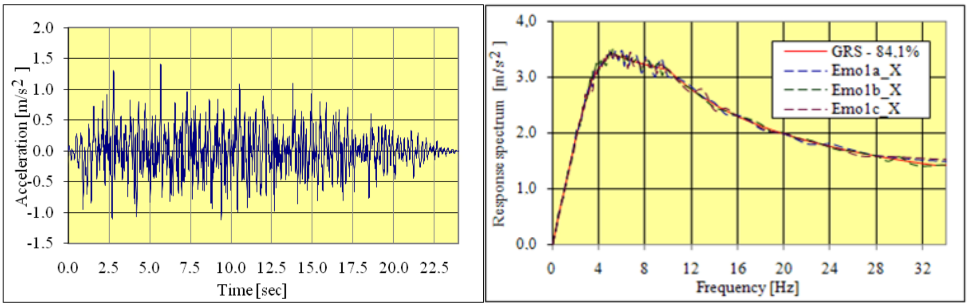


Fig. 2. The synthetic design accelerograms compatible with the response spectrum.

The comparison of the synthetic acceleration spectrum and GRS spectrum in the case of three synthetic accelerograms is showed in Fig. 2.

3 Seismic evaluation methodology

The methodology of structure resistance verification follows the requirements of IAEA [1] and acceptable US standards [3, 4]. This methodology is elaborately described in [2]. 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) and probabilistic (SPRA). The

CDFM 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 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.

4 Floor response spectrum

The equipment and interior structures of NPP are designed using floor response spectrum (FRS) as seismic loads. This spectrum is calculated from the time-history motions resulting from the dynamic analysis of the supporting structures in accordance of requirements of nuclear guide RG 1.122 [15]. The values of FRS are determined with the uncertainties of calculation methods and soil and structure properties. The FRS can be calculated from the in-structure spectra on the base of deterministic and semi-probabilistic methods [2]. 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 [2].

Deterministic method to generate FRS is defined in NUREG 0800 [16] and IAEA rep. No. 28 [1]. 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 [3] (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).

5 Acceptable evaluation methods

For seismic analyses of structures and technological equipment following methods are acceptable [15]:

- Response Spectrum Modal Analysis Method – RSMAM,
- Methods of direct time integration of the motion equations system (Linear Time History Method – LTH and Non-linear Time History Method – NTH are discerned),
- Equivalent Static Method – ESM (or also “approximate equivalent static seismic loads method”).

The equivalent static seismic loading method is used in cases when a structure or an equipment component is, in the viewpoint of the mechanics, a simple single-degree of freedom (SDOF) or 2-degree-of-freedom (2-DOF) system and when the intention of the analysis is to determine only the seismic forces and moments in the point of anchorage to the civil structure or to perform the stress analysis in some critical cross-section (when such a simple substitution is obviously sufficient).

The spectral analysis is based on linear behavior of the structures, while damping takes into account the plastic de-formations and the over thrust of friction-type connections. The response spectrum will be obtained by a combination of a response magnitudes for the individual shapes by the CQC method (considering the proximity) as follows [14]:

$$R_{CQC} = \left\{ \left| \sum_{i=1}^{N} \sum_{j=1}^{N} k.\varepsilon_{ij}.R_i.R_j \right| \right\}^2, \quad k = \begin{cases} 1\ ak\ i = j \\ 2\ ak\ i \neq j \end{cases}, \qquad r = \omega_j/\omega i, \tag{3}$$

where $\varepsilon_{ij} = \dfrac{8\left(\xi_i\,\xi_j\right)^{1/2}\left(\xi_i + r\xi_j\right).r^{3/2}}{\left[1-r^2\right]^2 + 4\xi_i\,\xi_j r\left(1+r^2\right) + 4\left(\xi_i^2 + r\xi_j^2\right).r^2}$ and ξ_i is the modal damping.

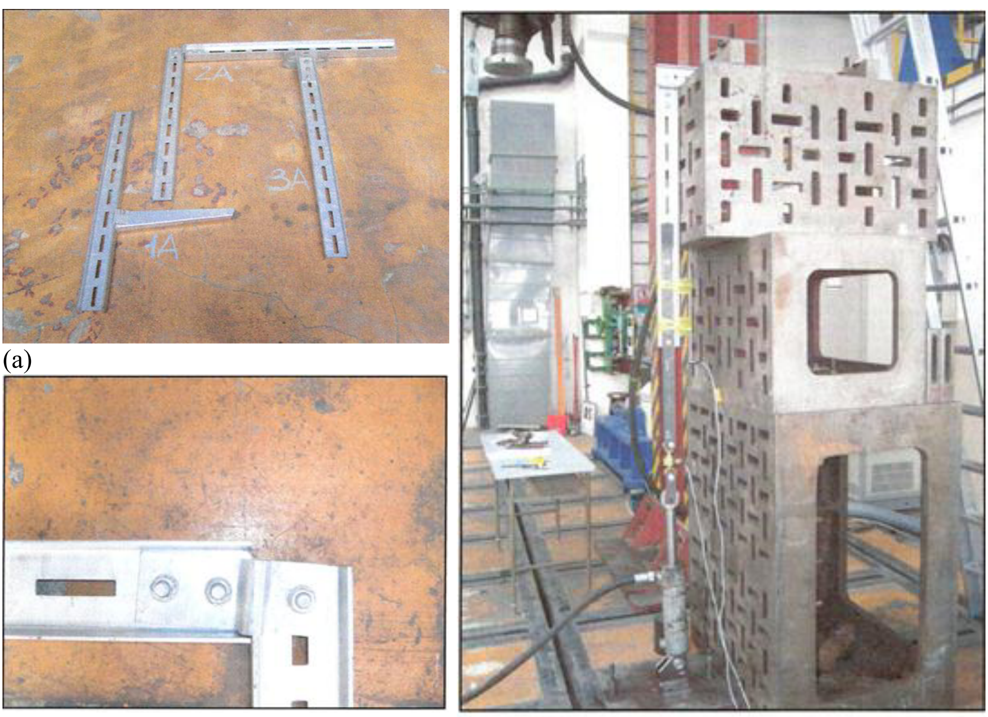
Overall response R_k in the cross-section k has been obtained by the envelope of combinations of responses along-side the individual excitation directions in the direction of the axes X, Y and Z [3].

$$R_k = \max\{R_{xk} + 0.4R_{yk} + 0.4R_{zk}\ ;\ R_{xk} + 0.4R_{yk} + 0.4R_{zk}\ ;\ R_{xk} + 0.4R_{yk} + 0.4R_{zk}\ \} \qquad (4)$$

where R_{xk}, R_{yk} and R_{zk} are the responses in the cross-section k from excitation by the acceleration spectra in the direction X, Y and Z. The combination rule in (4) is more conservative than in Eurocode 1998.

6 Experimental testing

The correct numerical model of the technology structures depends on the real behavior of the structures verified by the experimental tests [2, 9, 11, 17 - 19]. The static and dynamic resistance of the typical connections of the cable trays structures were tested in the certified laboratory VOP-026 Šternberk,s.p., division VTÚPV Vyškov [18].



(a) (b) (c)

Fig. 3. The photo-documentation - a) the tested segments 1A, 2A and 3A, b) the connection after test, c) the view of the test segmentˇ.

Three segments were tested (Fig. 3):

1. Segment 1A - steel column IS8 (No.6337252) of 3680mm with console of AS 30/41 (No. 6418813) for cable masses of 90 kg.
2. Segment 2A - steel column and beam IS8 (No.6337252) connected with joint AHIS 8 (No.6019064) and bolts FRS10×25 (No.6407560) for cable masses 270 kg.

3. Segment 3A - steel column US7/160 (No.6337252) connected with joint KU7 NOX (No. 6349056) for cable masses 600 kg.

Dynamic resistance of the tested segments was determined for the cyclic loads in horizontal and vertical direction (Table 1). All tested segments were satisfied for the static and dynamic loads defined. In the case of the tested connection OBO2A the plastic deformation of 9 mm occurred, but this connection was not damaged. This connection was not used in the final cable trays structures in NPP finally.

Table 1. Recapitulation of the experimental tests of OBO Bettermann segments.

Test segment	Static resistance [N]	Dynamic resistance experimentaly determined				
		Horizontal force [N]		Vertical force [N]		Ratio [%]
		Min	Max	M in	Max	
1A	3 000	0	1040	450	810	35
2A	12 000	0	6370	2700	4864	53
3A	11 000	0	12530	5400	9730	88

7 Numerical analysis of the seismic resistance

The seismic resistance of the various typical cable support structures based on OBO Bettermann segments were numerically analyzed in software ANSYS (Fig. 4) [9]. The principal structures was made from the columns 5×I8/3680, beams 6×US7/1500, consoles 60×AS30/41/410 and the cable segments. These structures were used with the cable trays 48×WKSG140/1500 (OBO1Z) and the cable ladders 44×LG640N/1500 (OBO1R).

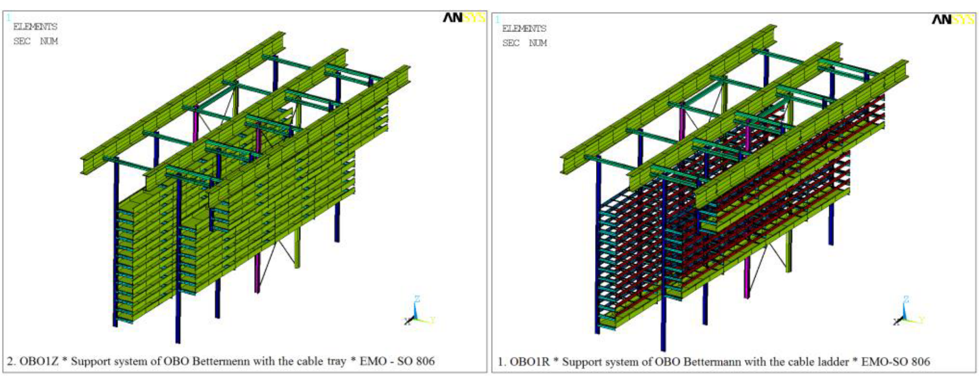


Fig. 4. Calculation models of the support structures with the cable tray EKS640 (OBO1Z) and the cable ladder LG640 (OBO1R).

The seismic response was solved by the method of the seismic response spectra from the floor spectra envelope in the horizontal direction as well as vertical direction at the level of the ceiling +5.4 m; upon considering 7% damping [2] (Fig. 5).

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

$$\text{HCLPF (CDFM)} = (\text{FS})_{\text{ep}} \cdot \text{PGA}_{\text{RLE}} = \text{SL-2} \tag{5}$$

The optimal configuration of the support systems of the cable trays was determined by the acceleration peaks of FRS, which are in intervals 5-10Hz [9]. The seismic effective structures are with the principal modes out from this inter-val. There are presented two different conceptions of the structures. The principal frequencies of model OBO1Z and

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OBO1R are out of this critical interval in direction X and Y (Table 2). The total masses of the structure OBO1Z is equal to 19 719 ton and OBO1R is equal to 22 176 ton.

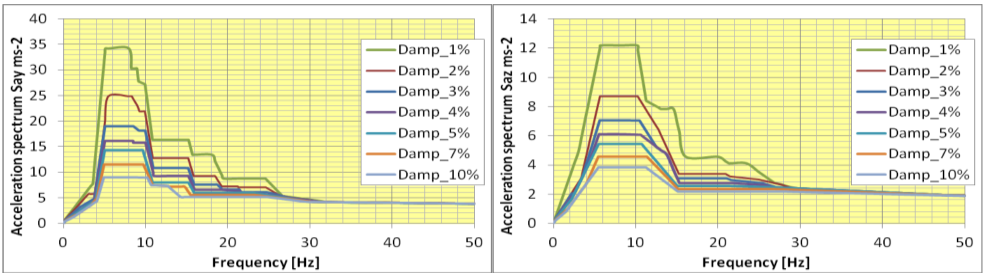

Fig. 5. FRS for horizontal and vertical direction at the level of +5.4m of SO805/806.

Table 2. Principal eigenvalues of the OBO Bettermann structures.

Model	f_0[Hz] in direct. X	Part. factor [%]	f_0[Hz] in direct. Y	Part. factor [%]	f_0[Hz] in direct. Z	Part. Factor [%]
OBO1Z	10.27	84.8	3.88	84.3	9.64	47.3
OBO1R	0.85	58.5	2.98	76.5	8.99	67.4

Table 3. Parameter HCLPF for the segments of OBO Bettermann structure.s

Model	Cable masses [kg/bm]	Column [g]	Beam [g]	Console [g]	Bracing [g]
OBO1Z	390	0.33	0.31	5.79	6.85
OBO1R	390	0.18	0.29	0.19	3.70

The seismic resistance of these support structures is determined by the lowest resistance of the structure segments (Table 3). The minimal HCLPF parameter of the structure OBO1Z is 0.31g and the structure OBO1R is 0.18g.

8 Conclusion

This paper presented the methodology of the seismic reevaluation of NPP in Mochovce due to new results from the geological and seismic-tectonic monitoring of this site. The deterministic methodology of seismic design based on CFDM methodology is presented. The synthetic spectrum compatible accelerograms generated in program COMPACEL (created by Králik) are presented in comparison with requirements IAEA and US standards [1, 3, 4, 15 and 16]. The methodology of the seismic resistance of the cable support structures is described. This methodology was used for the optimal design of the cable support systems from the structural elements of OBO Bettermann experimentally tested.

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References

1. IAEA No.28. *Seismic Evaluation of Existing Nuclear Power Plants* (IAEA Vienna, 2003)
2. J. Králik, *Safety and Reliability of Nuclear Power Buildings in Slovakia. Earthquake-Impact-Explosion*. (Ed. STU Bratislava, 307 pp., 2009)

3. ASCE 4/98. *Standard, Seismic Analysis of Safety - Related Nuclear Structures and Commentary on Standard for SASRNS - approved* 1998 (New York, ASCE 1998)
4. NUREG/CR-6926, *Evaluation of the Seismic Design Criteria in ASCE/SEI Standard 43-05 for Application to Nuclear Power Plants* (U.S. NRC Washington, DC 20555-0001, 2007)
5. P. Labák, P. Moczo, *Determination of the Review Level Earthquake (RLE) Characteristics for the Mochovce Nuclear Power Plant Site* (Contract No.370/96, Geophysical Institute SAV Bratislava, 1998)
6. V. Salajka, P. Hradil, J. Kala, Assess of the nuclear power plant structures residual life and earthquake resistance, In: Proc. *The Second International Conference on Engineering and Technology Innovation* (*ICETI* 2012), Kaohsiung, Taiwan, November 02-06, pp.4 (2012)
7. J. Králik, M. Šimonovič, Earthquake response analysis of nuclear power plant buildings with soil-structural interaction. *Mathematics and Computers in Simulation 50*. IMACS/Elsevier Science B.V. pp. 227-236 (1999)
8. J. Králik, Comparison of Probabilistic and Deterministic Assessments for Evaluation of Seismic Safety of Nuclear Power Plants in Slovakia. In: *First European Conference on Earthquake Engineering and Seismology*. 3-8 September, Geneve, p. 422 (2006).
9. J. Králik, *Design of the Seismic Resistance of the Connections. OBO Bettermann's Supporting Structures for the Electric Line in EMO34* (Rep. 04-130-088/OBO1a-2011, FCE STU Bratislava, 2011)
10. J. Králik, Risk-Based Safety Analysis of the Seismic Resistance of the NPP Structures, In: Proc. of the *8th International Conference on Structural Dynamics, EURODYN 2011*, Leuven, Belgium, 4-6 July, Vol.2, p. 292-299, ISBN 978-90-760-1931-4 (2011)
11. K. Kotrasová, E. Kormaníková, A case study on seismic behavior of rectangular tanks considering fluid - Structure interaction. In: International Journal of Mechanics, **10**, p. 242-252 (2016)
12. W. T. Lin, M. H. Hsieh, Y. C. Wu, C. C. Huang, Seismic Analysis of the Condensate Storage Tank in a Nuclear Power Plant. Journal of Vibroengineering, **14** (3), pp. 1021-1030 (2012)
13. A. Tesár, J. Melcer, Dynamic identification of fractal structures. In: Int. J. Numer. Meth. Eng. **71**, p. 1321-1341 (2007)
14. R. W. Clough, J. Penzien, *Dynamics of Structures* (Mc Graw-Hill, Inc. 1993)
15. RG 1.122. *Development of FRS for seismic design of floor-supported equipment or components* (U. S. NRC. Washington DC, Revision 1, February, 1978)
16. NUREG-0800. *Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants, LWR Edition* (U.S. NRC Washington, DC 20555-0001, 2007)
17. J. Protivínsky M. Krejsa, Using scaled physical model for assessment of mechanical damping of power plant boiler structure, In: *Perspectives in Science* 7, p. 287-291, Ed. Elsevier, (2016)
18. J. Štuchal, et al. *Protocol of the experimental tests. Force measurements. Assembled joints segments of OBO Bettermann*. (No 6430-81/2008. VOP-026 Šternberk, s.p., divize VTÚPV Vyškov, June, 2008)