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Ivo Demjan, Michal Tomko
Technical University of Košice
Civil Engineering Faculty
Institute of Structural Engineering

CALCULATION OF THE DYNAMIC RESPONSE OF REINFORCED CONCRETE STRUCTURES SUBJECTED TO THE FFECTS OF HEAVY TRANSPORT

A 3D model of a reinforced concrete building was created using a dynamic analysis which focused on the spectral response of the object represented by a random excitation experiment found in records in the form of load spectra.

Key words: experiment, spectral analysis, modeling.

Створено 3D модель залізобетонної будівлі з використанням динамічного аналізу, спрямованого на спектральну характеристику об’єкта через запис випадкових збурень у вигляді змінних навантажень.

Ключові слова: експеримент, спектральний аналіз, моделювання.

Introduction

In order to create a model of a building structure for dynamic analysis, it is necessary to specify the analysis model in terms of dynamic properties of the object as the same methods cannot be consistently applied to the substructure as those used in the structural analysis of the superstructure. When conducting a diagnosis of the current condition of the structure, the stiffness of the whole support system is generally unknown, but by experimental analysis of natural frequencies, it can be re-expressed in terms of computational model debugging and confirmed numerically.

The conciseness of computational models should be verified based on experimental investigations of the structure. Verification is made possible by comparing measured and computational model characteristics. The model may be optimized by the results of the diagnostic methods (experimental model analysis). Experimentally determined model characteristics describe the current status, properties and spatial behaviour of the structure, respectively certain elements of the structure at the time of experimentation. Such a model facilitates the calculation of the response of time courses of tested specimens to dynamic effects.

Experimental Measurements

The propagation of micro-vibrations in the soil of experimental measurements revealed that the obtained time courses of vibration (oscillations) above and below the surface of the half-space maintain a random functions nature, and thus may be described and analysed based on stochastic methods.

According to records obtained by seismic motion, it was found that the subsoil exhibits a dominant frequency of seismic motion, which depends on the source of excitation and the properties of the environment in which the seismic movement propagates. Individual frequency components of the original seismic motion are amplified or inhibited in the transition environment of the subsoil depending on the characteristics of the transmission path, [2].

Model characteristics of real buildings can be obtained in two manners:

– The processing of the response of the structure with the aid of controlled excitation power (Forced Vibration Testing – FVT);

– The processing of structural vibrations (oscillations), which are caused by natural ambient forces (Ambient Vibration Testing – AVT).

The role of an experimental investigation was to determine the model characteristics of reinforced concrete buildings, which were obtained from a dynamic response induced by natural ambient forces (AVT), so that in this case, ambient vibration effects spread through the natural environment because of actively used railway lines located near the monitored object.

The result is the natural frequency of vibration of the building in the analysed frequency range, where obtained custom shapes of vibration from the records were suppressed by the interference of different shaped components.

The 13-storey reinforced concrete building has a height of 47,3 m above ground level and is fixed to a reinforced concrete slab at a depth of 4,2 m below ground level which in turn rests upon a clayey-sand to gravelly-sand subsoil. The supporting structure is made up of twenty reinforced concrete columns with dimensions of 0,5 x 0,5 m and reinforced core loadbearing walls with a thickness of 0,3 m. Floors are made of precast reinforced concrete 22 cm thick resting on five reinforced concrete beams with dimensions of 0,5 x 0,5 x 20,4 m capped with a reinforced concrete bond beam around the perimeter of the floor with dimensions of 0,5 x 0,5 m. The axial distances between the floors are 4,2 m for the I. and II. floor and

3,3 m for the other floors, Fig. 1.

The railway line was designed with a speed limit of 60 km/h for all trains. The distance between the building and the railway line is 43,3 m.

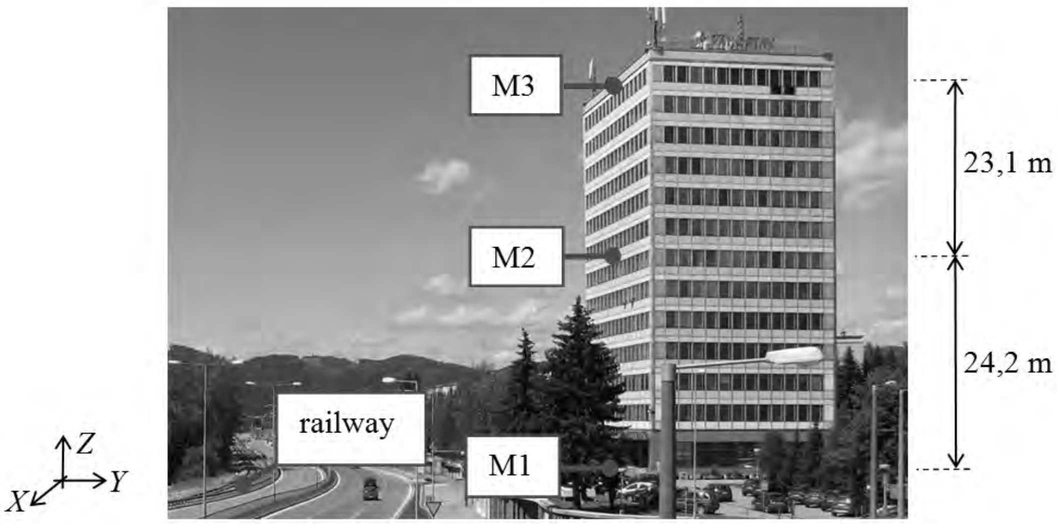


Fig. 1. Situation of reinforced concrete building and railway line,
Distribution of measuring points: M1 – basement; M2 – 7. floor; M3 – roof

Results of experimental measurements

The first natural frequency of oscillation for the pliable RC-wall skeleton – load bearing system can be expressed by the formula:

– for direction X: $$f_{0(1)}^{teor} = \frac{11{,}6\sqrt{B}}{H} = \frac{11{,}6\sqrt{25{,}5}}{48} = 1{,}22Hz \quad (1),$$

– for direction Y: $$f_{0(1)}^{teor} = \frac{11{,}6\sqrt{B}}{H} = \frac{11{,}6\sqrt{20{,}4}}{48} = 1{,}09Hz \quad (2),$$

where H is the height of the building, and B is a ground plan of the building in [m] considering the direction of the oscillation.

Dominant oscillation frequency bands of the subsoil at the base level of the reinforced concrete structure (point M1) which are induced by the effects of the rail, basically coincide in all directions. Dominant frequencies range from 9 Hz to 24 Hz, with peaks at 9,1 Hz, 11,2 ~ 11,9 Hz, 13,1 Hz to 13,8 Hz, and 15,1 Hz to 19 Hz, but also in the range of 21 Hz to 23,5 Hz. This confirms not only PSD but also CPSD (Fig. 2, Fig. 3, Fig. 4). Transfer functions other than those specified in the frequency regions also show frequency bands 2 ~ 3,5 Hz and 5,2 ~ 8 Hz. At the same time it can be concluded that these bands represent the linear energy transfer of vibrations (oscillations) from the source to the base of the structure, as is evidenced in the coherence function.

In terms of the frequency analysis of experimentally detected signals of vibration acceleration in the dynamic response of reinforced concrete buildings, it can be concluded that the dominant frequency occurred between 1,3 Hz, 4 ~ 4,8 Hz, with frequency bands ranging from 7 Hz to 9 Hz and above 10 Hz (fig. 6). Transmission characteristics of the building refer to the frequency bands 1,2 ~ 1,5 Hz, from 4,1 Hz to 4,7 Hz, and 7 Hz to 10,4 Hz and above 13 Hz (fig. 7).

Coherence functions show the linearity of oscillations in these frequency bands. In terms of the frequency response of the building, a good agreement of the first natural frequency of oscillation calculated on the basis of equation (1) and (2) $\mathrm{f}_{0(1)}^{\mathrm{teor}} = 1{,}09 \approx 1{,}22Hz$ and the first natural frequency of oscillation $\mathrm{f}_{0(1)}^{\mathrm{exp}} = 1{,}2 \approx 1{,}5Hz$ observed experimentally can also be obtained.

From the time histories, the vibration acceleration of the dynamic response of the building was calculated by logarithmic decrement of attenuation vexp = 0,0769.

Some experimentally observed power spectral density vibrations of the structure ranged in the area of 1,2 ~ 1,4 Hz which were more or less repressed and observed only in specific power spectral density oscillations. A decrease in performance vibration at a frequency that can be attributed to the observed dominant frequency domain in reference position (M1 – basement, fig. 3), demonstrates the dynamic characteristics of the transmission path (natural subsoil) and dynamic excitation effect (rail transport vehicles). It can be stated that the spectra of dynamic responses of reinforced concrete buildings from the effects of rail transport vehicles reveal the influence acting seismic motion and oscillating capabilities have on the structure.

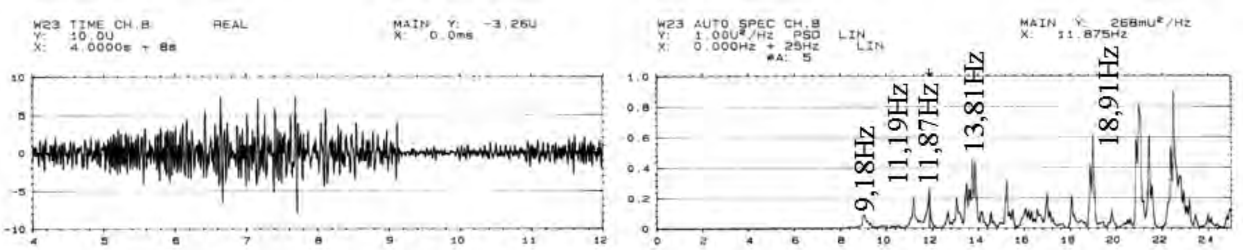


Fig. 2. Vibration acceleration time history at point M1,
power spectral density (GM1M1 (f)) at point M1 – direction Z

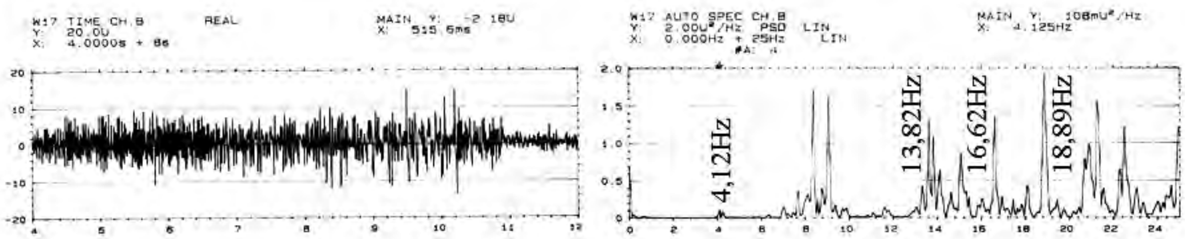


Fig. 3. Vibration acceleration time history at point M2,
power spectral density (GM2M2 (f)) at point M2 – direction Z

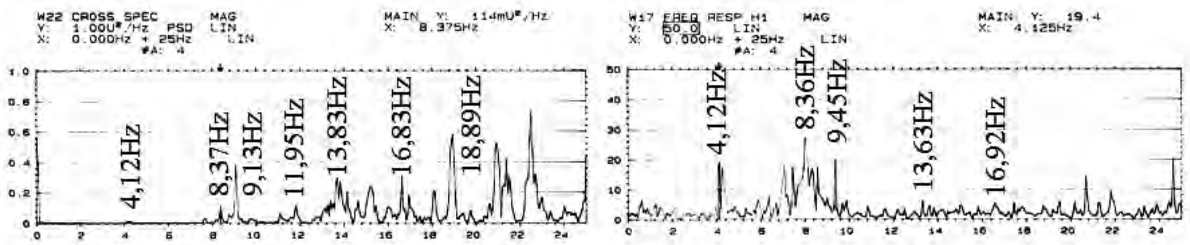


Fig. 4. Cross power spectral density (GM1M2 (f)), at points M1, M2,
transfer characteristic (HM1M2 (if)), at points M1, M2 – direction Z

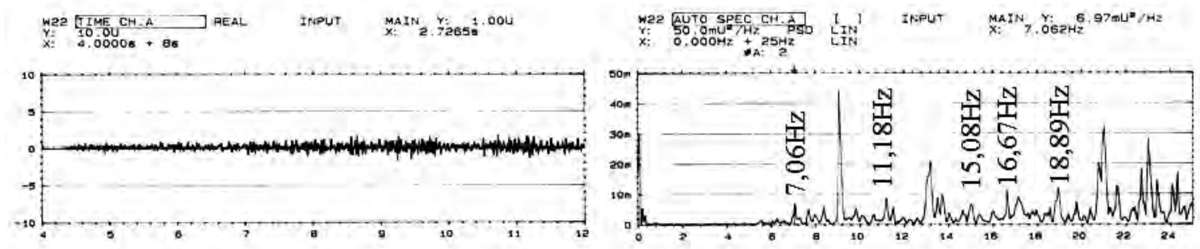


Fig. 5. Vibration acceleration time history at point M1,
power spectral density (GM1M1 (f)) at point M1 – direction X

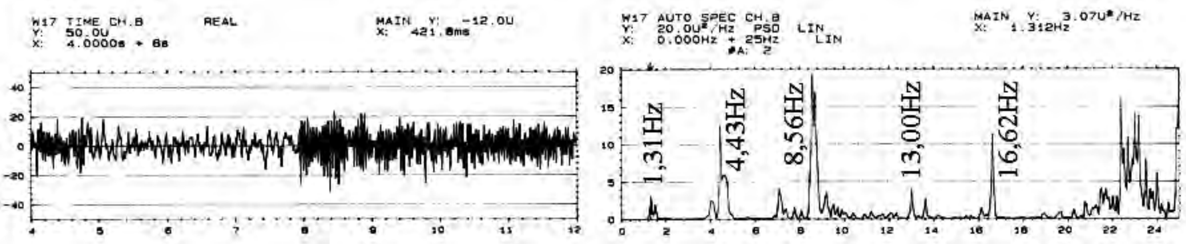


Fig. 6. Vibration acceleration time history at point M3,
power spectral density (GM3M3 (f)) at point M3 – direction X

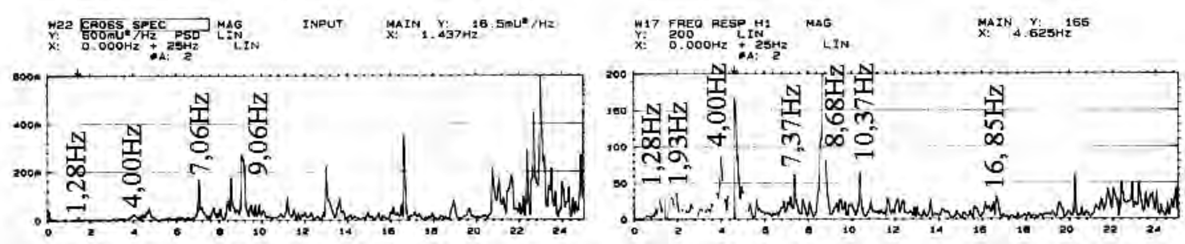


Fig. 7. Cross power spectral density (GM1M3 (f)), at points M1, M3
transfer characteristic (HM1M3 (if)), points M1, M3 – direction X

Calculation of internal oscillations

The spatial computation model of the structure modelled all floors (slabs), foundations, columns and load bearing walls. The weight of non-load bearing elements (walls, floors and floor load equivalent utility) was added to the weight of the floor slabs. The modulus of elasticity for reinforced concrete elements (concrete C 25/30) measured E = 31 000 MPa. Bulk density of the volume elements measured ρ = 2 400 kg/m^3. Thicknesses of the load-bearing columns and walls were also included using actual values. The element Brick – Solid 45 was used in the computational 3D model of the structure which had 118 380 nodes and 59 137 elements. The supporting computational model of the structure was considered as a perfect constraint.

The damping of the structure was derived from the experiment ($v = v_{exp}$ = 0,0769), creating a constant damping factor for the structure:

$$\xi = \alpha / 2\omega_{01} + \beta\omega_{01} / 2 = 0{,}0121 \quad (3),$$

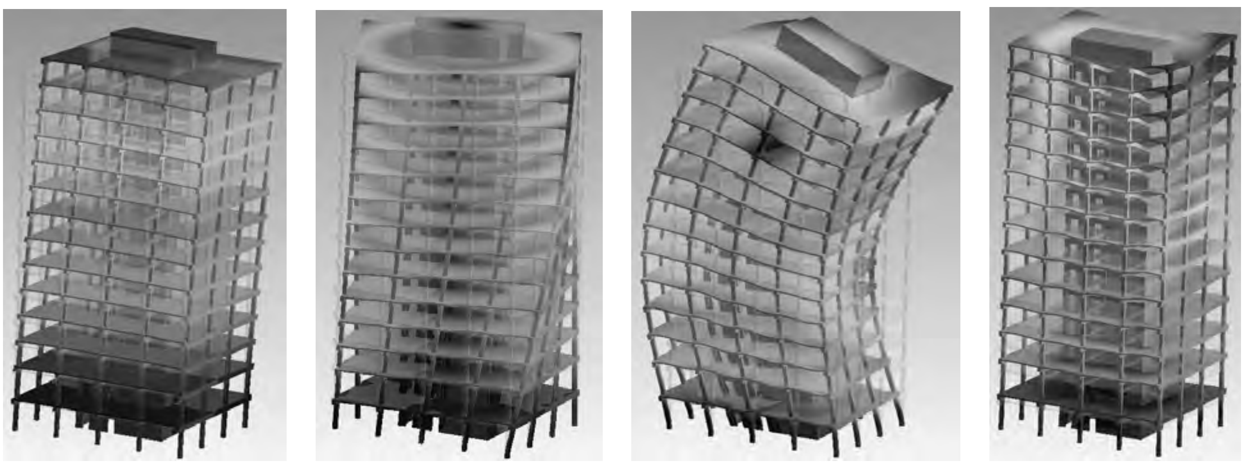
where $\alpha = \dfrac{\omega_{0(1)} . \nu}{\sqrt{\left(4\pi^2 + \nu^2\right)}}$ a $\beta = \dfrac{\nu}{\omega_{0(1)}\sqrt{\left(4\pi^2 + \nu^2\right)}}$.

Tab. 1 and fig. 8 lists some natural frequencies for the reinforced concrete building oscillation computational model.

Table 1

Natural oscillation frequency of the calculation model

Oscillation	Frequency [Hz]	Oscillation	Frequency [Hz]
1	1,3772	11	11,481
2	1,8382	12	11,801
3	2,3229	13	11,931
4	5,2224	14	13,064
5	6,8502	15	14,319
6	6,9759	16	14,554
7	8,7628	17	15,234
8	10,319	18	16,089
9	10,635	19	16,476
10	11,151	20	16,683



($f_{0(1)}$ = 1,3772 Hz) 1. Oscillation shape

($f_{0(3)}$ = 3,3229 Hz) 3. Oscillation shape

($f_{0(5)}$ = 6,8502 Hz) 5. Oscillation shape

($f_{0(7)}$ = 8,7928 Hz) 7. Oscillation shape

Fig. 8. Natural frequencies and vibration (oscillation) shapes of reinforced concrete buildings

The dynamic response of a structure based on a theoretical and experimental approach

The aim of the calculation is to determine the time course and amplitudes of dynamic stresses in typical cross sections i.e. the determination of the time course and amplitude deviations, velocity and vibration acceleration of structural parts of the building and its assessment by the limit state I. or II. group.

For lengthy time courses, random vibrations are statistically processed and transformed from the time domain to the frequency domain because of the difficulty in determining the characteristics and understanding the extent of their impact on system design. With the aid of the fast Fourier transform (FFT) random vibration time histories can be replaced by a deterministic sum of sinusoidal waveforms with appropriate amplitudes (oscillations), frequency and phase angle [5].

ANSYS – Workbench 12.0 was used in the study, so that the results were designed to compare the experimental and theoretical approach of the dynamic response of reinforced concrete buildings. Monitored points of the dynamic response of a computational model of the structure were consistent with areas in the experimental measurements.

Micro-seismic loads were represented by modified vibration (oscillation) acceleration time histories (accelerograms) which were measured at the reference position (M1) of the structure. The computational model of the structure is burdened by the performance of the power spectral density vibration acceleration (PSD – Power Spectral Density) of the experimentally observed accelerograms at point M1. In addressing the dynamic response calculation model of the structure, the actual effect of all three components of the PSD (direction X, Y, Z) are considered.

Traces of experimentally observed vibration acceleration time histories for each X and Z directions, calculated by PSD to individual vibration acceleration time histories and histograms of vibration acceleration time histories, describe coincidental effects exerted by the rail as is shown in fig. 19, fig. 10, fig. 11, fig. 13, fig. 14 and fig. 15. An approximation function describing the distribution function of the vibration acceleration time for a recorded random excitation effect of experimental measurements, which is suitable for analytical expressions of random excitation effect, fig. 12 and fig. 16 was also discovered.

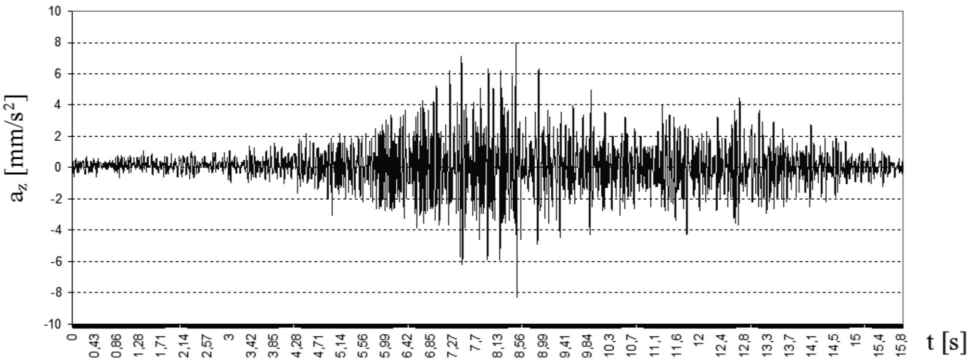


Fig. 9. Time record of vibration acceleration in M1 – Z, RMS = 1,3526

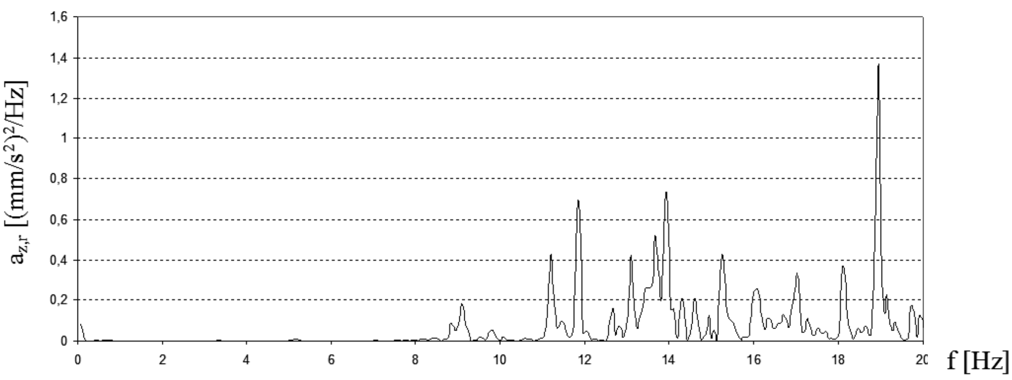


Fig. 10. PSD vibration acceleration (GM1M1 (f)) in M1 – Z

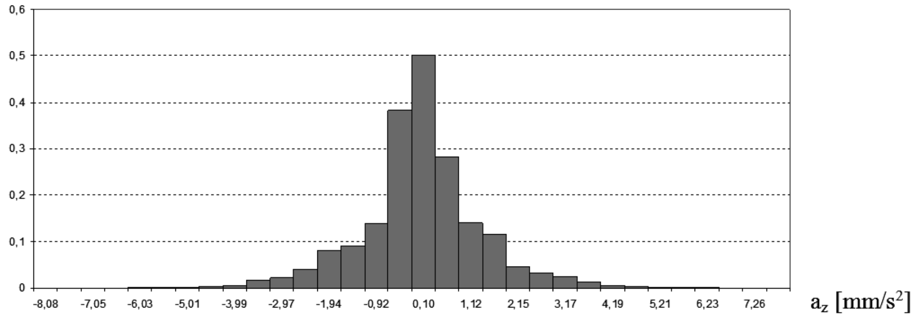
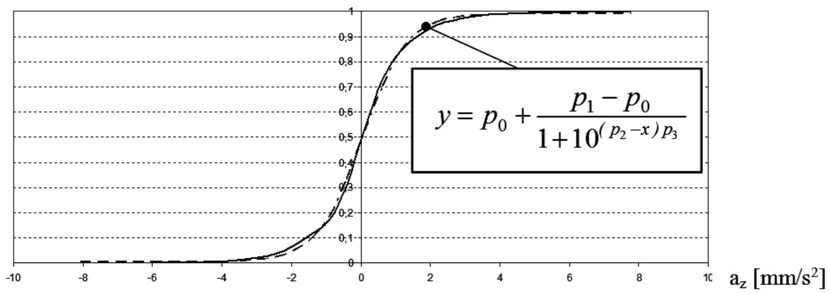


Fig. 11. Histogram time record of vibration acceleration in M1 – Z



$p_0 = 0{,}005218798372$, $p_1 = 0{,}9930346093$, $p_3 = 0{,}0451066486\,8$, $p_3 = 0{,}6754545482$

Fig. 12. Distribution function of the vibration acceleration time record in M1 – Z

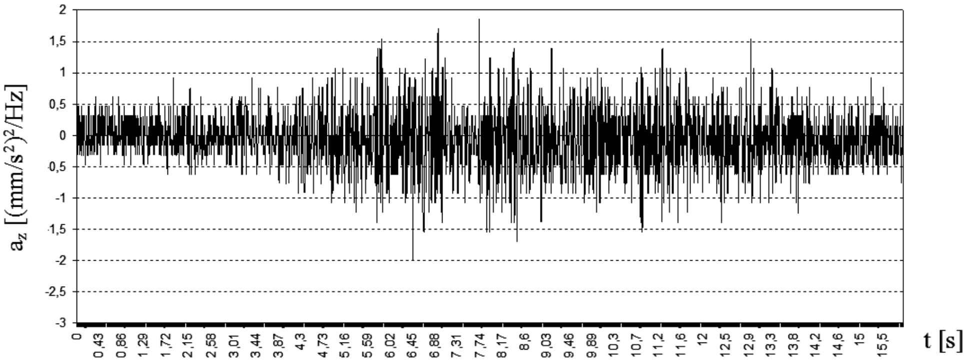


Fig. 13. Time record of vibration acceleration in M1 – X, RMS = 0,4341

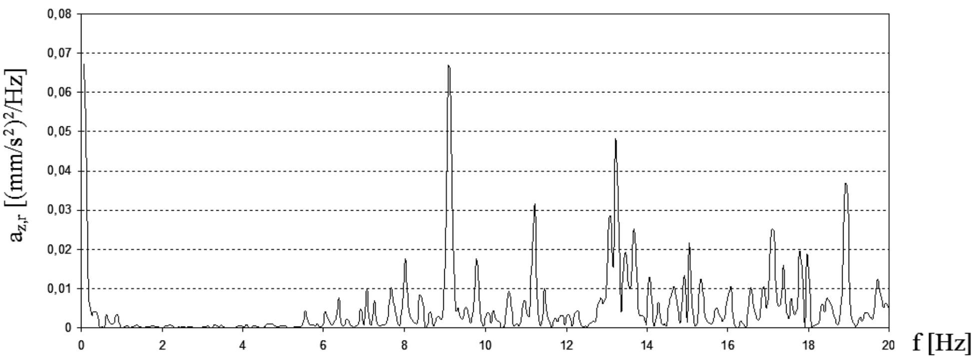


Fig. 14. PSD vibration acceleration (GM1M1 (f)) in M1 – X

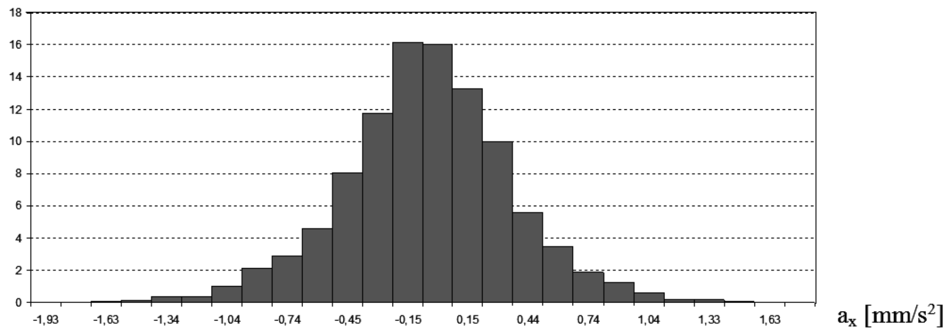
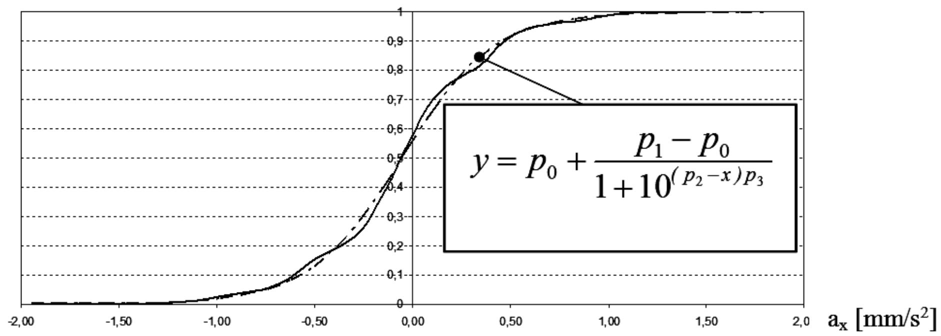


Fig. 15. Histogram time record vibration acceleration in M1 – X



$$p_0 = 0{,}00288310603, p_1 = 0{,}9976950656, p_3 = -0{,}05179398032, p_3 = 1{,}874737908$$

Fig. 16. Distribution function of the vibration acceleration time record in M1 – X

Random vibration analyses enable the determination of the response of the system to excitation vibrations, which are not deterministic in nature. Such excitation effects can be described statistically (middle quadratic variation, root mean square, etc.). Excitation is applied in the form of power spectral density PSD, which determines the spectral values with respect to frequencies, thus capturing the frequency range. The resulting output is statistical in nature and represents that 99,737 % of the time response, produce values less than the standard deviation, [5].

Fig. 17 illustrates the calculated maximum deflection, velocity and acceleration at the point corresponding to experimental measurements at point M3 (roof).

deformation [mm] velocity [mm.s^{-1}] acceleration [mm.s^{-2}]

Fig. 17. Deformation, velocity and acceleration in M3

Confronting theoretically calculated RPSD (RPSD – Response Power Spectral Density) with experimentally obtained RPSD, the observed point – M3 (roof) is shown in fig. 18. By comparing the RPSD it can be said that this method of calculating the response of the structure from the experimentally observed load effects observed in RPSD theory showed some dominant frequency bands that corresponded to the experimental RPSD. In this case it involves a frequency at 8,76 Hz, then at 11,8 Hz, respectively in the 13 Hz to 14 Hz, and also in the 16,5 ~ 17,1 Hz range.

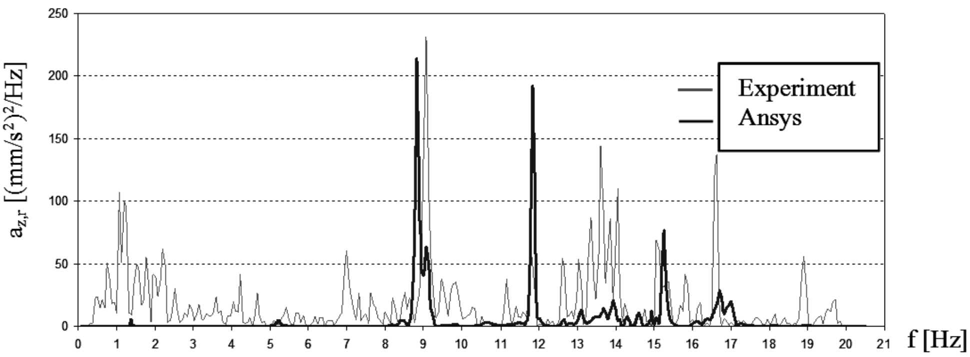


Fig. 18. Comparison of response spectra power spectral density vibration acceleration at point M3 – direction Z

Comparison of the results of the dynamic response based on the theoretical and experimental approach can be implemented on the basis of the observed RMS vibration acceleration (RMS = x_{ef}) at point M3:

RMS_{exper} = 6,81261 mm.s^{-2};

RMS_{teor} = 9,9726 mm.s^{-2}.

Conclusion

This case involves a calculation of the dynamic response of the structure by a randomly acting ballast effect represented by the experimentally observed PSD load vibration acceleration of the reference object position. The application of this method confirms that the presented random frequency analysis is a suitable method for modelling the effects of random excitations and dynamic responses of the structure. Those procedures can also disclose results at any point of the structure (deformation, velocity, acceleration, internal forces and stress) including places where dynamic responses of the structure were not experimentally observed. The presented method facilitates the determination of responses during the design phase in order to reveal actual excitation of dynamic effects not only by rail, which could previously only be obtained from experimental measurements during on-site construction.

Acknowledgements

The research has been carried out within the project NFP 26220220051 Development of progressive technologies for utilization of selected waste materials in road construction engineering, supported by the European Union Structural Funds.

The paper is carried out within the project No. 1/0321/12, partially founded by the Science Grant Agency of the Ministry of Education of Slovak Republic and the Slovak Academy of Sciences.

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