Response of a panel building to mining induced seismicity in Karvina area (Czech Republic)

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A dynamic behaviour of technological structures and buildings under a non-stationary dynamic loading is investigated by technical seismicity. The solution of this problem is based on field seismic measurements using a specific source of technical seismicity induced by the mining activity in the area under study (Karviná region in the Czech Republic). Finite element models are prepared for computer analyses of seismic responses of measured structures. The twelve-storey panel residential building was selected for the seismic analysis as a representative structure.

Key words: *mining seismicity, mathematical modeling, seismic response*

Introduction

Study of mining induced seismic activity in the Karvina region (Czech Republic) documents that a vibration effect of most intensive events and/or events with shallow foci can be observed on the surface. These vibrations are frequently reasons for discussions and disputes, especially in cases when people live in surroundings of undermined places. However, if failures occur on structures and buildings, the vibrations may not necessarily be the true initiator. Other effects of mining activities are often blamed, e.g. sinking and other deformations on surface or changes of ground water level. In addition, effects that are unrelated with mining activities have to be considered sometimes, e.g. a wrong basement of building, unloading or additional load of the structure after reconstruction, or redistribution of heavy masses.

The presented results are based on up to date geological and seismological conditions. The region of Karvina, a part of the Upper Silesian Basin, where intensive mining induced seismic events have been documented for a long time, is the area with an underground exploitation of hard coal. Mining induced seismic events can reach intensities with maximum amplitude values that can cause strong macro seismic effects on the surface. Therefore, solitaire seismic stations are operated to obtain a better information about the seismic loading. The main results from the registration in years 2000-2005 are presented in this contribution. The experimental investigation shows that the seismic velocity component of most intensive shocks exceeds the value of 10 $mm.s^{-1}$ (the acceleration component reaches 500 $mm.s^{-2}$). These values are of such a degree of intensity that there exist real possibilities of damaging buildings. The vibrations will evoke an unpleasant feeling of inhabitants.

The paper concerns the analysis of the procedure of seismic response determination of an actual building structure using the ANSYS program package. A seismic response of the twelve-storey panel residential building to the excitation described by a computed accelerogram from recorded velocity seismograms is shown here. These studies are very important for the evaluation of response spectra (Viskup et al., 2005

Recent mining induced seismicity in the Karvina region

A primary cause of seismicity in the Karvina region is the underground mining activity that has proceeded for more than one hundred years. Each mine (7 mine fields, area about 10 x 12 km) consists of several tectonic blocks, in which individual coal seams are gradually mined by longwall methods. In the mines here, only longwall retreat faces are used, either with caving and/or the low-pressure stowing

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method. In contrast to the distribution of earthquake hypocenters along tectonic faults, the analysis of the hypocenters of induced seismic these events has proved that the great majority of them were concentrated in areas of current mining activity. Only rarely were hypocenters located in the vicinity of major faults, and in those cases could the events be attributed to tectonic movements induced by mining (Holub, 1997). The mining works are situated in the depth of 700 – 1000 m. No natural seismic activity was detected here.

Very complicated induced stress fields were generated by complicated geological (Dopita, Kumpera, 1993) and tectonic patterns and previous mining activities (e.g. mined-out and buried spaces, protecting pillars). Concerning the seismicity effect on mine structures, rockbursts are the most dangerous events because visible permanent deformations and failures of structures' stability are generated. Rockbursts are obviously connected with geological and geomechanical conditions in which the mining activity proceeds. The intensity and success of continuously performed rockburst control measurement is important (Konečný et al., 2003). About 10 rockbursts have occurred in the last twenty years.

The erection of a network of seismic stations for the monitoring of the seismic activity induced by coal mining in the Karviná region was motivated by a frequent occurrence of intense seismic events, which were often of rockburst nature. The first monitoring station of Karviná part was built up on the surface of the Darkov colliery. Already the first results of operation of the station confirmed assumptions for the application of seismic monitoring method as well as provided the justification for a further gradual expanding of seismologic monitoring aimed at the objective assessment of particular mining induced events. Underground seismic stations have been gradually built up, equipped at first with an analogue instrumentation and later with the digital instruments which afterwards were linked up to local rnicroarrays with digital data transmission and central data processing at the ČSA colliery (Kaláb et al., 1994; Holub et al., 2002). Already during the initial phase of establishing the local network, a hitherto strongest rockburst (seismic energy about 10^{10} J) occurred at the ČSA mine in 1983. It turned out that the existing analogue systems did not comply in their indices with conditions for reliable recording and a subsequent proper evaluation of extraordinary geomechanical events. For this reason, a conceptual proposal of regional seismic network called Seismic Polygon was elaborated which should link up local and national seismic networks.

Thousands of weak mining induced seismic events (henceforth events) are annually recorded in the area under investigation (Holub, 1999; Kaláb, Knejzlík, 2002, 2006); however, there are only several tens of events with a higher value of irradiated seismic energy that can be detected as macro seismic effects. In the paper presented by Kaláb (2004), the main parameters affecting the intensity of seismic effects on the surface have been described. Many seismological, geological and construction factors must be taken into account. The variability in the given area (for the investigated event and building) is the result of local geological and hydrogeological conditions. This so-called site effects are often discussed and modeled (Bullen, Bolt, 1993; Viskup, Janotka, 1995; Ansal, 2004; Janotka et al., 2006).

Measurement of seismic effect in the building

Solitaire seismic stations in building structures are operated to study the seismic effect on the ground surface. All these stations are equipped with seismic apparatuses of the PCM3-EPC type developed in the Institute of Geonics (Knejzlík, Kaláb, 2002). The PCM3-EPC is built-up as a modular system consisting of self-standing functional blocks, which are designed as modules of EUROCARD standard. From different blocks, various configurations of recording apparatus can be assembled, such as the autonomous recorder, the different telemetric variants with the on-line digital data transmission by means of telephone line or eventually by means of radio link. A system of analog transmission of signals from remote seismometers by current a loop with a high resistance against the interference is developed. Amplifiers with the measuring range adjustable to 32, 16, 8, 4, 2 and l mm.s^{-1} and the frequency band 0.1-30 Hz are used. The sampling frequency is adjusted to 100 Hz (max. 250 Hz), the dynamic range of signal digitizing is 90 dB (MSB/LSB corresponds to 16-bit converter). The pre-trigger and post-event time of recording can be adjusted within range of 0 - 9 s. The recording capacity is about 30 hours with ZIP floppy-disc. Usually, three seismometers SM-3 were used in the geographical configuration.

Due to the variability of seismic noise level in buildings, it is ineffective to apply a more sophisticated algorithm requiring a long-term optimizing of parameters such as STA/LTA. Therefore, trigger levels during the discussed measurement were not unified because the anthropogenic seismic noise in the monitored places was very different. The trigger level in individual points was changed depending on the temporarily increasing noise (e.g. construction works in the surrounding). In tab. 1, the number most intensive events are presented in dependence on their seismic energy (energy classes) recorded during the period 2000 – 2006. The maximum value of velocity was recorded at the Doubrava seismic station – 13.2 mm.s^{-1} (May 21, 2005).

The present results of monitoring at the solitaire seismic stations provide the following conclusions:

- Maximum velocities were recorded in epicenter areas of the most intensive shocks.

- Using also new results from the monitoring it is possible to confirm that majority of recorded events do not reach the limit values specified for the least resistant buildings according to the Czech Technical Standard (CSN 73 0040).
- By contrast, the most intensive events can exceed the limit value fixed for small damages (e.g. cracks in backfilling) on buildings (CSN 73 0040). However, this situation occurs in a small epicenter area.

To present the influence of mining induced seismic effects on buildings, a typical record was used as a source of vibrations. The wave pattern of this event is presented in fig. 1 (recorded at the Orlová station, November 3, 2004, 04:57); its seismic energy is about 10^5 J.

Tab. 1. Number of events recorded by the mining network stations during the period 2000 – 2006 (according to the Annually Reports of OKD, DPB, a.s. Paskov - unpublished).

Number of seismic events								
Year		2000	2001	2002	2003	2004	2005	2006
Energy class (J)	10^4	186	178	282	294	330	312	278
	10^5	5	18	38	38	31	39	42
	10^6	3	1	4	2	2	5	6
	10^7	0	0	0	0	0	0	0
	10^8	0	0	1	0	0	0	0

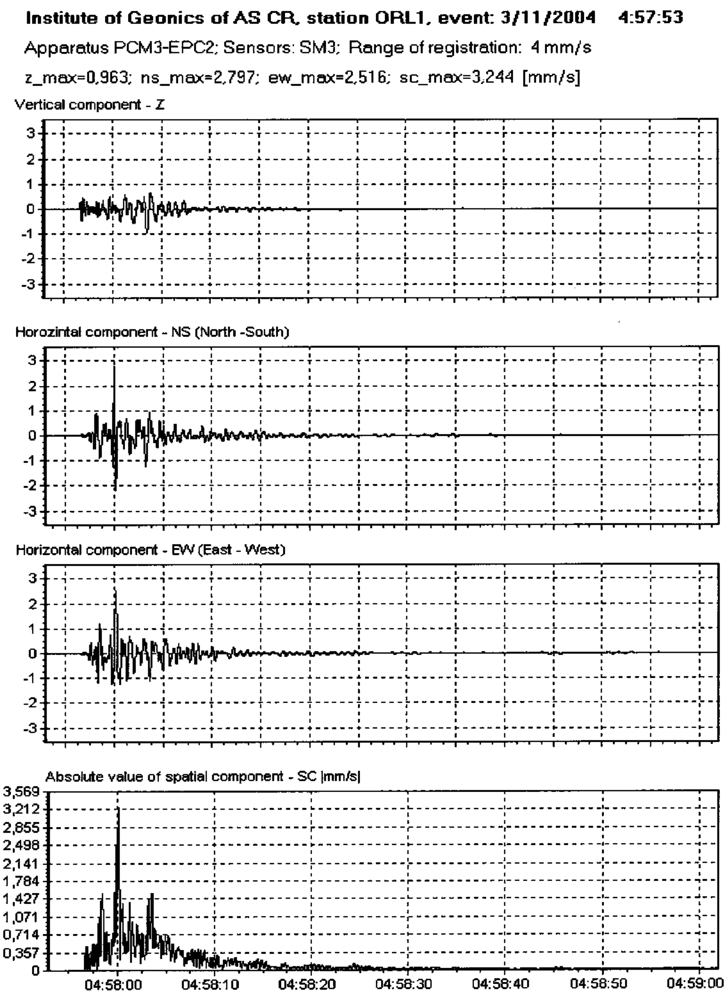

Fig. 1. Wave patterns of a mining induced seismic event from the Karviná region with the seismic energy about 10^5 J.

Representative structure – the panel building

Panel buildings have been widely built up in the Czech Republic, including regions endangered by the mining induced seismicity. The technical life of most panel buildings is not yet exhausted and various reconstructions are planned with the aim to prolong the service life respecting both present and future requirements on comfortable dwellings. The twelve-storey panel residential building shown in fig. 2 has been selected for the analysis as a representative structure constructed from large-sized precast reinforced concrete panels with complicated mutual constraints of parts.

Fig. 2. Twelve-storey panel residential building for the modeling.

The development of a correct computation model represents the most important step in the dynamic structural analysis (Kanický et al, 2005). The model has to introduce geometric and physical approximations in an extent, which is theoretically just acceptable for obtaining the information required in the description of dynamic behaviour of the modeled structure. The model has to allow a detailed description of inertial, elastic and dissipative properties of the modelled structure. The introduction of any simplification must be based on the corresponding sensitivity analysis. The model has to be structured so as to allow a solution of predictable supplementary problems (e.g. due to changes in substructures).

Consequently, the sophisticated computation model of the panel building has been deveveloped using the finite element method of discretization. The model (fig. 3) has been designated for the application of the ANSYS program package. The structuring of the model is extremely fine, the geometry of the model corresponds even in details with design drawings of the object. All wall cut-outs, windows, doors, floor cut-outs, floor substructures, internal partition walls, cellar partitions, foundation slab structure, etc. have been modeled precisely (fig. 4). Standard density values of construction materials have been considered. Permanent floor loads have been modeled explicitly introducing the equivalent distributed mass. Characteristic values of mechanical properties of used construction materials have been obtained from reliable sources. Dissipation properties of the structure have been approximated by using a corresponding standard value of modal damping (5 %). A more reliable value is expected to be obtained by field measurements.

The influence of subsoil on the dynamic behaviour of the structure has been modeled approximately. The physically correct soil - structure dynamic interaction was not considered. Several variants of the basic computation model were considered involving basically either the classical mat foundation model or the half-space foundation model.

Finite elements of the type SHELL43 were used in developing the model of the structure. Spatial elements SOLID45 have been used for the modeling of the subsoil massive. On the whole, up to 74725 elements with 279597 degrees of freedom were used.

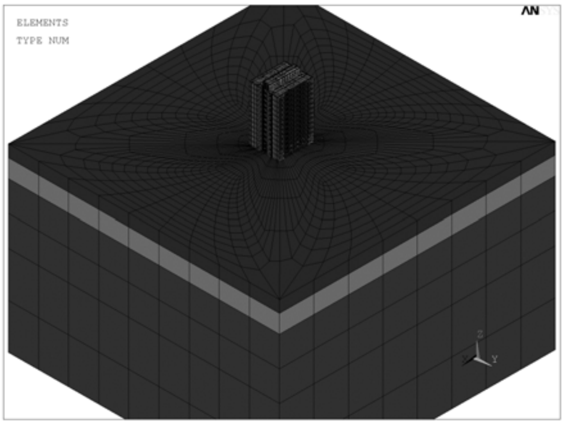

Fig. 3. *Model of the panel building - general view.*

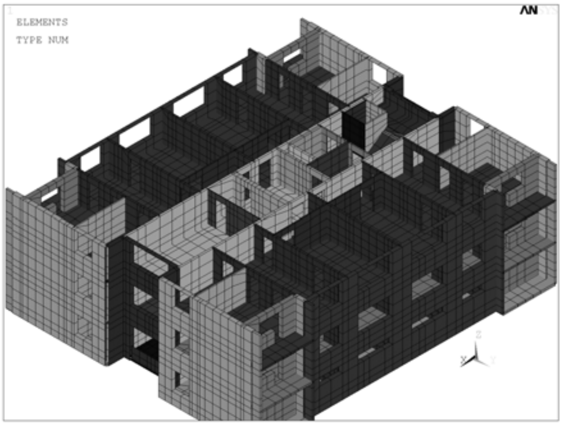

Fig. 4. *Model of the panel building – detail.*

Natural frequencies and normal modes of vibration

Natural frequencies and normal modes of vibration of the model were computed using the Lanczos method. The range of computed natural frequencies (up to 33 Hz) were selected in accordance with the results of seismic measurements. The number of computed frequencies satisfied the criteria for a correct structural response analysis. The ratios of cumulative effective modal masses to total mass of the structure were higher than 0,85 for three orthogonal excitation directions.

With respect to uncertainties of floor load specifications, the influence of permanent floor loading masses on free vibrations of the structure was analyzed. The importance of this analysis arises due to the fact that the relevant data cannot be reliably specified even for actual buildings. Consequently when analyzing a building structure the probable influence of permanent floor loading masses has always to be taken into account. Summing up the results of the analyses, it can be concluded that for actually appearing range of permanent floor loading mass values, the influence on free vibrations is not substantial (Kanický et al., 2005).

With respect to uncertainties of subsoil specifications, the influence of the subsoil stiffness on free vibrations of the structure was analyzed. The importance of this analysis arises due to the fact, that engineering procedures converting reliably at least the known properties of the subsoil (homogenous or layered) to the effective stiffness value required for the simplified dynamic structural analysis using the elastic mat foundation model have not been known untill now. Thus, the relevant data cannot be reliably specified even for actual buildings with subsoil defined by tests. Consequently, when analysing a building structure, the probable influence of the subsoil stiffness value on free vibrations has to be always taken into account. Similar conclusions can be drawn for cases when the elastic half-space massless foundation model for the analysis is applied. The required values of both effective modulus of elasticity and effective shear modulus can be obtained only with very large dispersions. The problems related to the use of a correct subsoil massive model are out of scope of the presented paper. Summarizing the results of the series of analyses, it can be concluded that the influence of the subsoil effective stiffness value on free vibrations is decisive.

The results of analyses of five structure model variants (tab. 2) illustrate the above mentioned conclusions. The following variants have bee considered:

VAR. 1 Rigid subsoil, embedded foundation slab, floor loading mass 50 kg.m^{-2},
VAR. 2 Rigid subsoil, embedded foundation slab, floor loading mass 200 kg.m^{-2},
VAR. 3 Foundation slab on verticaly elastic mat (stiffness k = 5*10^6 N.m^{-3}),
VAR. 4 Foundation slab on verticaly elastic mat (stiffness k = 1.66*10^6 N.m^{-3}),
VAR. 5 Foundation slab on elastic subsoil, undrained Young's modulus of soil E = 58.5*10^6 N.m^{-2}.

Tab. 2. Summary of results of the analysis – variants of the model.

Variant	Floor loading mass [kg.m^{-2}]	Total mass [kg]	Vertical deflection [mm]	f_1 [Hz]	f_2 [Hz]	f_3 [Hz]	f_4 [Hz]
VAR. 1	50	6.6255*10^6	0.00	3.892	4.313	5.289	12.649
VAR. 2	200	7.2380*10^7	0.00	3.738	4.141	5.077	12.112
VAR. 3	50	6.6255*10^6	34.0	0.681	0.788	2.767	4.873
VAR. 4	50	6.6255*10^6	98.2	0.403	0.466	1.629	4.871
VAR. 5	50	6.6255*10^6	33.8	0.964	1.060	2.736	2.909

Response of the panel building to a seismic event

The problem of the determination of the response of a building to the ground motion due to an earthquake event is one of the most important issues in structural dynamics. With respect to the scope of the activities described above, the mining induced seismic event is exclusively regarded as a source of the foundation motion excitation. Compared to the low probability of intensive earthquake occurence, technical seismic events of lower intensity are often occuring in mining areas. However, long-lasting observations show also exceptional occurences of relatively high intensity seismic events due to the rockburst (as high as M = 3.5). Thus, a reliable design or reconstruction of panel buildings sited in mining areas should be based on the reliable seismic response analysis.

The presented results of seismic response analyses of two selected variants of the computation model of the panel building could illustrate the discussed problem (tab. 3). The first variant (A) of the model considers the subsoil massive as rigid. The second variant (B) is characterized by the subsoil with the undrained Young's modulus of soil increasing with the depth from E = 22 MPa up to 98 MPa (specified for the site). The specified Poisson's ratio is 0,35, the effective soil density value is 1850 kg.m^{-3}. First normal modes of vibration of models A and B are shown in fig. 5 and fig. 6, respectively.

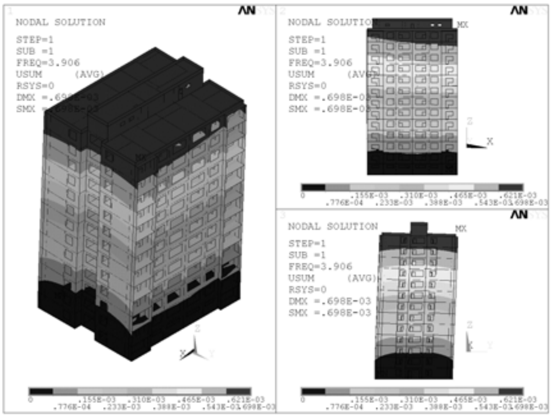

Fig. 5. 1st normal mode of vibration - model A.

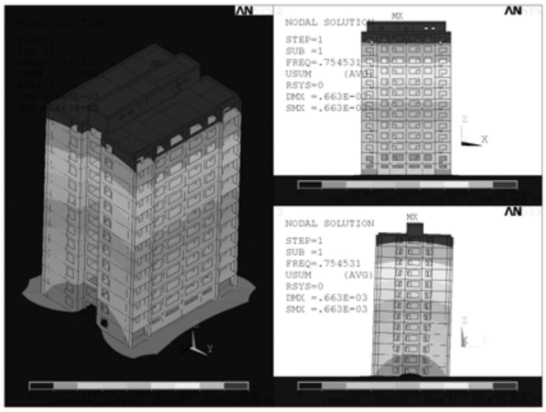

Fig. 6. 1st normal mode of vibration - model B.

The response analyses have been carried out using both the direct integration procedure and the linear response spectrum method. The base acceleration time histories shown in fig. 7, fig. 9 and fig. 11 were derived from the given records of the typical seismic event (fig. 1). The velocity values were increased in order to cover extreme events. The linear response spectra shown in fig. 8, fig. 10 and fig. 12 were generated using the acceleration time histories. Floor response spectra were generated for selected height levels. The set of damping ratios 0.00, 0.02, 0.05 and 0.07 was been considered. The spectra for the height level h = 33.6 m computed by using themodels A and B are shown in fig. 13 and fig. 14, respectively. The fields of response displacement components u_y, which have been obtained by the linear response spectrum method with the use of models A and B are shown in fig. 15 and fig. 16, respectively.

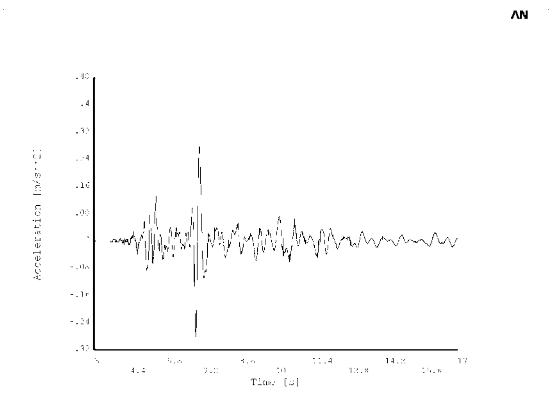

Fig. 7. Derived ground acceleration response spectrum – dir. x.

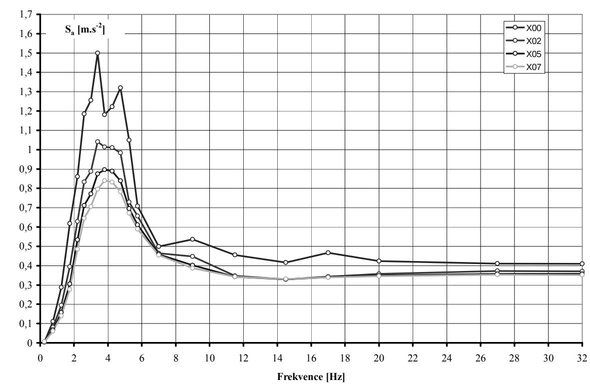

Fig. 8. Generated design acceleration response spectrum – dir. x.

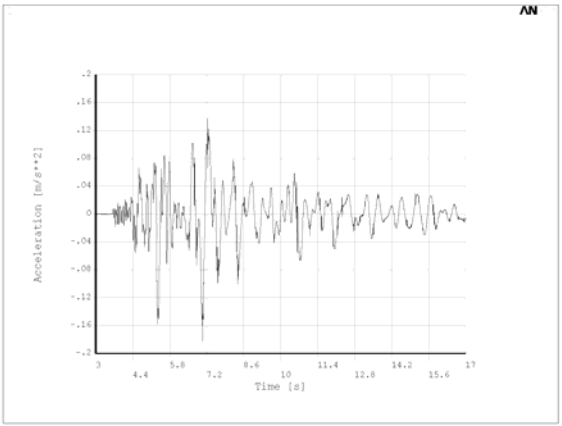

Fig. 9. Derived ground acceleration response spectrum – dir. y.

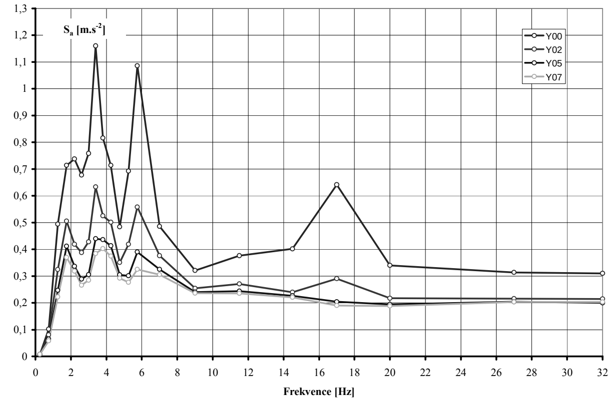

Fig. 10. Generated design acceleration response spectrum – dir. y.

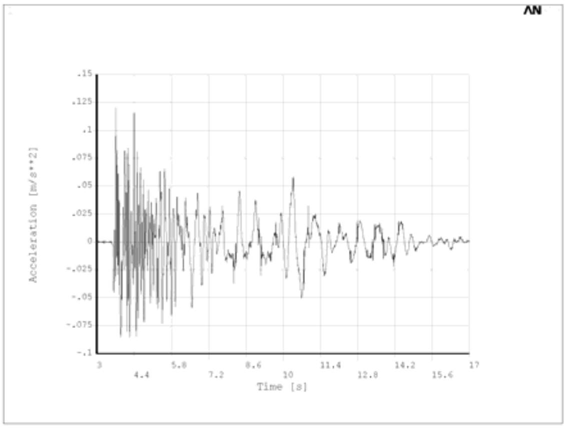

Fig. 11. Derived ground acceleration response spectrum – dir. z.

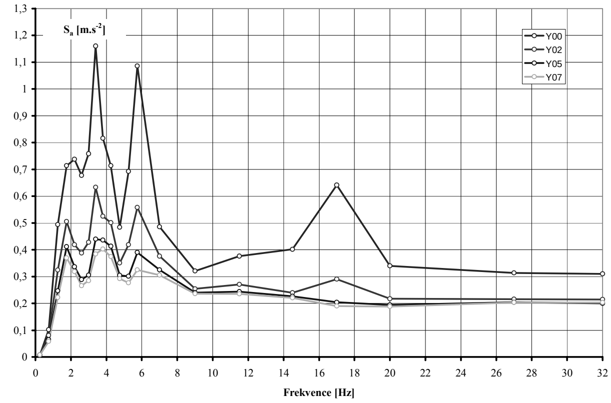

Fig. 12. Generated design acceleration response spectrum – dir. z.

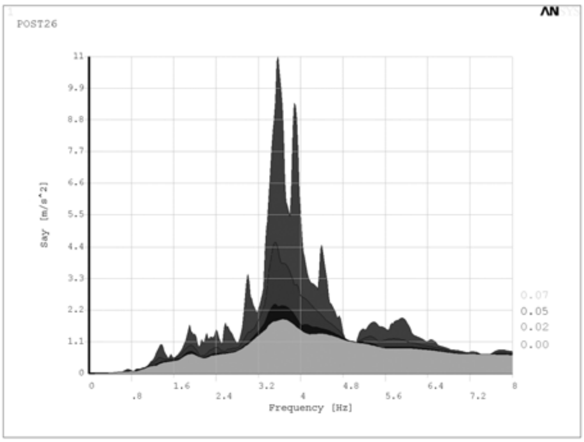

Fig. 13. Generated floor response spectrum $S_{ay}(f)$ - model A.

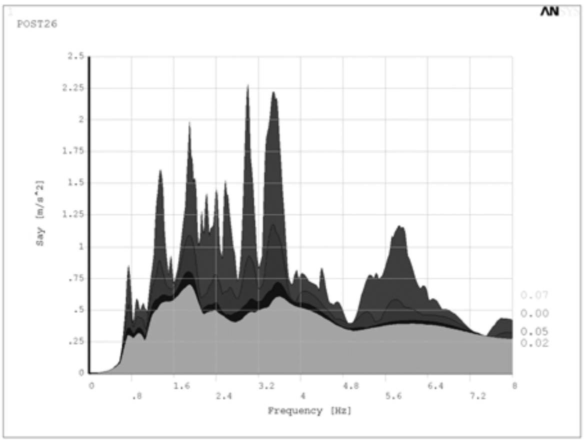

Fig. 14. Generated floor response spectrum $S_{ay}(f)$ - model B.

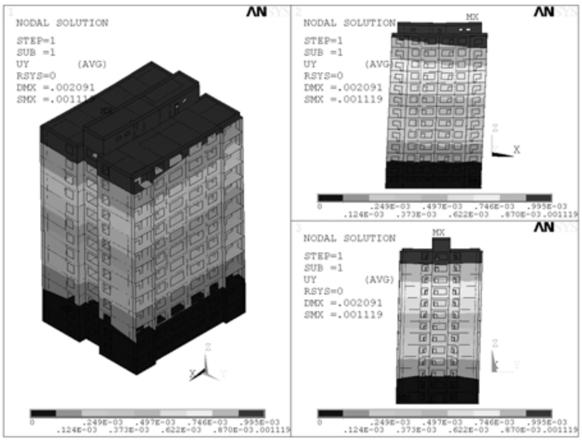

Fig. 15. Field of maximum response displacements |uymax| – model A.

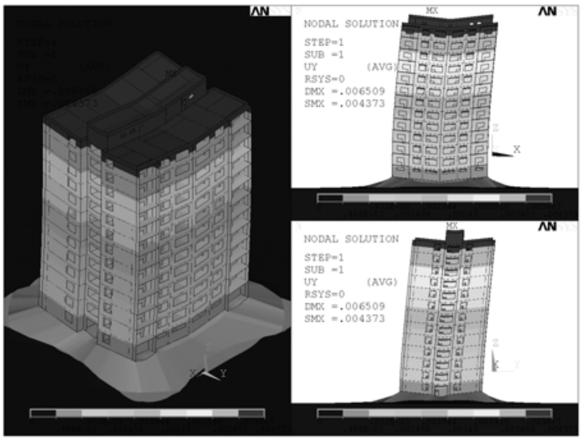

Fig. 16. Field of maximum response displacements |uymax| – model B.

Tab. 3. Summary of results of the response analysis – models A and B.

Displacement	Computation model A			Computation model B		
	Seismic motion described by			Seismic motion described by		
	Acceleration time history		Response spectrum	Acceleration time history		Response spectrum
	Maximum response [m]	Minimum response [m]	Maximum response [m]	Maximum response [m]	Minimum response [m]	Maximum response [m]
u_x	+0,001500	-0,001774	0,001752	+0,003698	-0,003328	0,004350
u_y	+0,000875	-0,000970	0,001119	+0,003475	-0,003642	0,004373
u_z	+0,000402	-0,000401	0,000515	+0,002038	-0,001683	0,003044

Conclusion

Seismic responses of a residential panel building structure to the technical seismicity caused by mining industry activities have been analyzed using a sophisticated computation model. Some problems encountered during the development of the model are described here. It has been shown that during the computation of the structure response many related problems arise. These problems have to be analyzed in order to get reliable results.

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For the version of the paper with colored figures, ask kalab@ugn.cas.cz.

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