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Research paper

Nonlinear analysis of a hoist tower for seismic loads

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Abstract: In recent years, the intensity of the loads caused by mining activity has increased in Poland. This exploitation is often carried out in urbanized areas, so their operation on structures is not only a social problem, but also a challenge for engineers. Many of the surface facilities safe use affects the failure-free operation of the mine. The paper presents the results of representative measurements of surface vibrations from mining areas in Poland and earthquakes and their comparison. Particular attention was paid to the values of PGA/PGV ratios and the most commonly used methods for dynamic calculation of the structure. The last part of the work presents an experimentally verified dynamical model of the selected RC skip tower. The forced vibrations of the model were analysed by taking representative earthquakes and mining origin tremors. Time history non-linear analysis and push over methods were used. The nonlinear concrete model was adopted in the analyses. The results show that pushover analysis is not able to capture the seismic demands imposed by far-field or near-fault ground motions, especially for short-period systems for which it can lead to significant errors in the estimation of the seismic demands. The results confirmed the qualitative results of the linear analysis. The carried out inventory of cracks to the skip tower also allowed their location in bearing elements of the skip tower. The results of non-linear numerical analyses allowed us to assess the safety of the structure.

Keywords: earthquakes, industrial structures, mining tremors, nonlinear analysis, surface vibrations, time history analysis

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1. Introduction

Linear and non-linear methods are used both in the design and in the analysis of structures subjected to earthquakes [1, 2]. This also applies to mine areas where mineral resources are exploited [3–10].

In recent years, the problem of the impact of dynamic loads is particularly important in Poland due to the increasing intensity of mining shocks associated with underground and ground mining exploitation [10–13]. The most intense underground mining shocks cause surface vibrations, which are characterized by values of the PGA (Peak Ground Acceleration) of horizontal components and the vertical component of vibrations with values reaching up to 0.3g (g – standard gravity) [10].

Predictions of surface vibrations are known and more and more accurate and design in these areas is based on these results. The velocity and acceleration of the vibrations are also used in the preparation of mine operation plans. Free-field vibration predictions are used in computational and approximate assessments of the resistance of surface structures to the influence of mining shocks [14–16].

High RC and steel structures are often found in seismic areas [17, 18]. Such structures also rise in the mine areas and an example of such high structures is, among others, water dams, industrial chimneys, residential buildings, cooling towers, steel towers and RC skip towers analysed at work [17]. Such structures are subjected to various types of static and operational loads, as well as to the movement of the ground. Among the kinematic loads, the most intense are seismic loads and, in Polish conditions, they are mining shocks.

The problem of designing structures in areas affected by surface vibrations of mining origin is not standardized in the national standard, and the results of calculations based on Eurocode 8 (EC-8) [19] give overestimated results [20]. This is due to different characteristics of earthquakes and mining shocks [10, 21]. The range of dominant frequencies in earthquakes is much lower than in the case of mining shocks. This is the main reason for not recommending the standard response spectrum from EC-8 for calculating and designing structures from mining areas [22]. In addition, local soil conditions in mining areas are the cause of differences in standard response spectra depending on the mining region [21–23].

Due to a large number of methods for solving these issues, a linear time history analysis (THA) [2, 24–26] was implemented in the previous paper and only the possibility of using a simplified nonlinear analysis [17], method was mentioned by using the modified the elastic standard response spectrum Sa and behaviour factor q according to EC-8 [19]. Using data from EC-8 the values of the q factor for RC industrial RC structures with load-bearing walls are in the range of 1.5–4.8 depending on the structure ductility [17].

Designing of building structures for the impact of earthquakes loads is carried out using non-linear analysis and, in justified cases, a linear analysis. In both cases, it is required to build a dynamic structural model. Such a model should be verified based on the in situ investigation results [17]. This verification consists of comparing the calculated and measured natural frequencies of the structure. It is also allowed to verify the model based on approximate formulas [14].

The earthquake resistant design of structures requires that structures should sustain, safely, any ground motions of an intensity that might occur during their construction or in their normal use. However, ground motions are unique in the effects they have on structural responses.

The article concerns the nonlinear analysis of the behaviour of an exemplary hoist tower due to seismic loads (seismic shocks, mining tremors). The failure-free operation of the hoist towers is necessary for the safe operation of the mine. The research dealt with the determination of the location of cracks in the tower's load-bearing elements and compare them with the damage occurring in the hoist tower in question. Furthermore, the results of the pushover analysis determine the base shear value at which the destruction of the supporting system begins, leading to loss of stability and collapse.

In this research, the applicability of the push-over method as an alternative means to the assessment of the response of the structure is examined. The second applied method is non-linear time history analysis. Overall push-over analysis can be non-conservative in estimating seismic demands of structures and it may lead to unsafe design. Of course, modal response spectrum analysis is a quite preferred analysis which is more useful for problems involving the structural design of newly constructed structures. On the other hand, a push-over analysis is more suitable for the analysis of the seismic vulnerability [27, 28].

One of the aims of the article is to compare computational methods used in earthquake engineering, to indicate their advantages and disadvantages on the example of the RC structure that has passed the history of loads associated with mining tremors. This structure was verified during in situ tests [17]. A detailed analysis of the damage with their location found in the structure will be carried out. The results of this analysis will be compared with the results of nonlinear analysis of the skip tower model and also with the results of the linear analysis performed at previous work [17].

1.1. Paper's novelty

The following novelty, primary and new contributions to the paper could be listed:

- Conduct in situ tests of the hoisting tower located in the area of the mined mine.
- Determining the location of damage in the structure model (which may be useful in terms of possible repairs) and their identification in the actual structure.
- Significant differences of surface vibrations due to earthquakes and mining tremors were found.
- The results of the implemented pushover analysis allow the use of the analysis to determine the dynamic limit of resistance of this type of structure in mining areas.
- The PGA parameter is considered to be the predominant quantity for determining the strain of the structure.

2. Models and methods included in seismic analyses

The methods used to solve seismic influence on structures can be divided into linear and non-linear. In these methods, we need to create a numerical finite element model. In the nonlinear model, we apply geometrical and material nonlinearities. The most popular methods used in dynamic calculations are Response Spectrum Analysis (linear and non-linear) [2, 29–31], Pushover static-nonlinear analysis [2, 32, 33] and Time History Analysis (linear and non-linear) [2, 24–26].

The most accurate analysis procedure for structures subjected to strong ground motions is time-history analysis [2, 24–26]. This analysis involves the integration of the equations of motion of a multidegree-of-freedom system, MDOF, in the time domain using a stepwise solution to represent the actual response of a structure. This method is time-consuming though for application in all practical purposes. The need for faster methods that would ensure a reliable structural assessment or design of structures subjected to seismic loading led to the pushover analysis.

The push-over analysis is the common name of a type of procedure that uses simplified nonlinear static analysis [2, 32, 33]. A modal response spectrum analysis is dynamic linear analysis. Modal response spectrum analysis is more suitable for problems involving the structural design of new structures, while push-over analysis is more indicated for assessing the seismic vulnerability of existing structures. Pushover analysis is based on the assumption that structures oscillate predominantly in the first or the lower modes of vibration during a seismic event. This leads to a reduction of the multidegree-of-freedom, MDOF system, to an equivalent singledegree-of-freedom, ESDOF system, with properties predicted by nonlinear static analysis of the MDOF system. The ESDOF system is then subsequently subjected to a nonlinear time history analysis or a response spectrum analysis with constant-ductility spectra or damped spectra. The seismic demands calculated for the ESDOF system are transformed through modal relationships to the seismic demands of the MDOF system. Pushover is a static-nonlinear analysis method where a structure is subjected to gravity loading and a monotonic displacement-controlled lateral load pattern which continuously increases through elastic and inelastic behaviour until an ultimate condition is reached [32, 33].

In the case of using the Response Spectrum Analysis (RSA) method, we have to have a standard response spectrum for the area in which the structure is located [23]. This is a problem to create individual spectra for each considered area because of the lack of a proper seismic network for the area [19]. Therefore, we created standard response spectra that are independent of soil properties.

Non-linear analysis of the structure consists of increasing load. It means that in the calculation loads are not taken into account in full, but they are gradually increasing and further states of equilibrium are solved. The non-linear behaviour of a structure may be related to a single element of construction (structural or material non-linearity) or may result from a non-linear relationship between forces and deformation in the entire structure (geometric non-linearity). Structural nonlinearity can be caused by nonlinear elements working only on compression or tension, cable components, and non-linear constraints (unilateral constraints or defined by the stiffness function: supports, releases, compatible nodes) occurring in the structure, material plasticity and nonlinear joints.

The most popular nonlinear material models are nonlinear elastic materials dedicated to isotropic materials; this material will not yield, which means that, however high the load will be, after taking that load away, the material will return to its initial state without any permanent deformations (e.g. [34]). It also does not show strain hardening (after several times loading-unloading cycles it acts the same); bilinear elasto-plastic material in which yield criterion may usually be applied as initial yield stress (yield point), cohesion and angle of internal friction, and hardening rule if we want to have perfectly plastic material. In the case of cyclic loads the following three possibilities of defining hardening are isotropic, kinematic and connection isotropic and kinematic; multi-linear plastic material is characterized by the same settings as for bilinear material and extra we have to define the multi-linear relation between stress and strain; rigid plastic material has the same settings as bilinear elastoplastic and multi-linear material and we have added to define plastic part of a stress-strain curve and assume that material is rigid in elastic range.

Taking into account the geometric nonlinearity allows to consider the real effects of a higher order and often leads to improving the convergence of the calculation process of a structure containing nonlinear elements. The included geometric non-linearity takes into account second or third-order effects for the entire structure. The second-order analysis contains i.e. "stress-stiffening" effect. This analysis results in a change in stress in the element and also takes into account the generation of moments from the action of vertical forces on the nodes displaced horizontally. The effects of the third order are associated with the P-delta analysis resulting in the additional transverse stiffness and stresses originating from the deformation. The third-order effects create additional forces in the deformed structure e.g. longitudinal forces appearing on both sides of the vertically loaded beam, and deflection decreases.

In a nonlinear analysis, iterative methods such as Newton–Raphson's full method, modified Newton–Raphson method, arc-length method, an improved arc-length method called the modified Crisfield-Ramm method [35], displacement control method, and constant stiffness method are used to solve equations describing nonlinear problems [36] due to the response cannot be calculated directly using a set of linear equations. The incremental method or the arc-length method is used to solve a system of nonlinear equations. In the incremental method, the load vector is divided into "*n*" equal parts called increments. Another load increase is applied to the structure when the equilibrium state has been reached for the previous increment. The norm for unbalanced forces is given for each step, which allows following the behaviour of the force-displacement relationship in the structure. The arc-length (displacement control) method should be used when incremental algorithms for solving equations by force control do not coincide.

3. Comparison of selected kinematic loadings

3.1. Introductory remarks

Earthquakes are the main loads that act on surface structures in seismic areas. Numerous seismic stations record surface vibrations due to seismic events. Records of these surface vibrations are the basis for preparing standard response spectra. The standard spectra are used in dynamic analyses of buildings as well as in the design process of buildings, e.g. [33, 38]. Seismic analyses are also carried out using full surface vibration component records using the time history analysis [17].

Poland is not in the seismic area but from time to time weak earthquakes occur like in 2004 in the Podhale region [39]. The magnitude of this earthquake was estimated to be about 4.7 on the Richter scale. The depth of the shock epicentre was determined to be about 10 km. The earthquake was felt in a radius of over 100 km from the epicentre that was located near the Czarny Dunajec. The evaluation of the intensity of this earthquake and its influence on structures have been discussed in [39].

In Poland, apart from sporadically occurring earthquakes, the most intense source of surface vibrations are mining shocks. Such shocks are caused by underground and ground exploitation of mineral resources. Standard response spectra are also determined in areas affected by mining tremors by the seismic area pattern of seismic areas [10].

Surface vibrations caused by earthquakes and mining shocks have many similarities, but also differ in several aspects. Earthquakes are phenomena that are independent of humans and they occur unpredictably. Earthquakes cover enormous areas and can cause damage and breakdowns of numerous buildings in these areas [40]. The maximum magnitudes of underground mining tremors are about 4.0–4.6. This magnitude value could be comparable to the earthquake.

Poland, especially the south of Poland, is exposed to mining tremors. There are many differences between earthquakes and mining tremors [10, 17]. The major differences concern the duration of the intensive vibration phase (mining tremors last shorter than earthquakes), values of Peak Ground Acceleration (PGA) [41], the content of predominant frequencies [1], frequency of occurring and depths of hypocentre [42].

In the article, the RC model of the tower is subjected to the action of selected representative earthquakes and mining tremors. The characteristics of surface vibration records caused by representative earthquakes and mining tremors are given below.

3.2. Example earthquakes

Horizontal recorded components from three representative earthquakes were selected for analysis. The intensity, values of PGA/PGV ratio and duration of the intensive vibration phase were the basis in the selection of representative loadings of seismic origin. Among other things, the earthquake in SITKA was selected for the analyses because the records of this shock constituted the forcing in the elastic dynamic analyses of the skip tower discussed here. Nonlinear analyses have been extended taking into account earthquakes from L'Aquila and El Centro.

The first earthquake concerns an earthquake that took place on April 6, 2009, in the city of L'Aquila (Italy) and its surroundings [43, 44]. Its intensity was 6.3 on the Richter scale. This earthquake had catastrophic consequences for the inhabitants (309 deaths, approximately 1,500 injuries, and approximately 40,000 people lost their homes) and caused damage to approximately 18,000 buildings in the epicentre area. 1975. Figure 1 shows example horizontal components (x and y) of the L'Aquila earthquake with the results of the FFT analysis. Table 1 presents the basic data characterizing these components. Strong motion duration for the horizontal components has been designed using the notion of the Husid plot [21, 45].

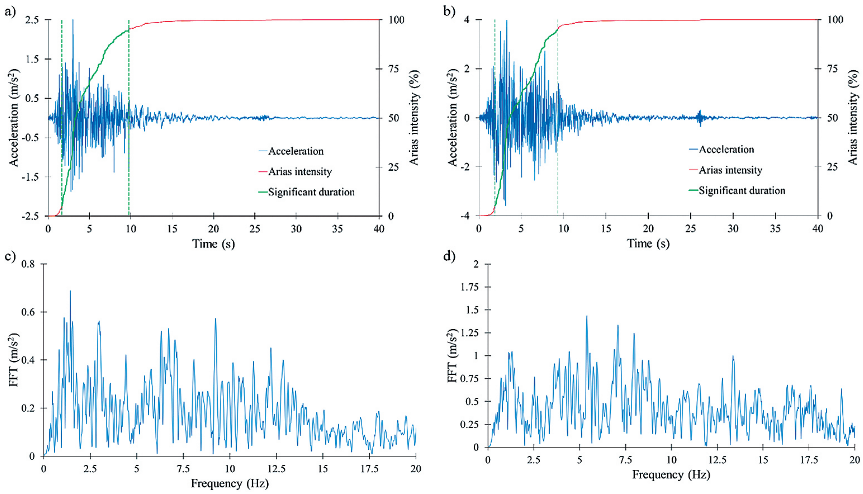

Fig. 1. The horizontal components of the L'Aquila earthquake, a) acceleration record in x direction, b) acceleration record in y direction, c) FFT of the acceleration record in the x direction, d) FFT of the acceleration record in the y direction

Table 1. Example characteristics of the analysed earthquakes

Earthquake	PGA_x	PGA_y	PGV_x	PGV_y	PGA_x/PGV_x	PGA_y/PGV_y	th_a 1)	th_v 2)
	(mm/s^2)	(mm/s^2)	(mm/s)	(mm/s)	(1/s)	(1/s)	(s)	(s)
L'Aquila	2677	3975	109	318	24.5	12.5	7.5	34.9
SITKA	765	894	74	67	10.3	13.3	27.1	28.9
El Centro	2099	3423	487	380	4.3	9.0	24.4	38.5

1) "strong-phase" duration corresponding to acceleration records,
2) "strong-phase" duration corresponding to velocity records

The second earthquake under the interesting phenomenon took place on 30th July 1972, 48 km from Sitka Alaska (US). This earthquake had a magnitude Mw of in the range of 7.3–8.1 [46]. The quake did not cause any loss of life. No injuries were reported. The Sitka's authorities stated that only minor damage had occurred. Light damage also occurred at Hoonah, Juneau, Pelican, and Yakutat [47, 48]. The origin of the Sitka shock is related to the existence of the Fairweather-Queen Charlotte fault, resulting in several very intense shakes in the years 1949–1972 [49–51]. Figure 2 presents an example of the horizontal components (x, y) of the recorded acceleration vibrations with Husid's graphs and the results of the FFT analysis of the records. The values of the ratio PGA/PGV are typical for earthquakes and remain in the band 10.3 to 13.3. The "strong phase" duration of this earthquake is almost 30 s. Table 1 presents data characterizing horizontal components of vibrations referring to the Sitka earthquake.

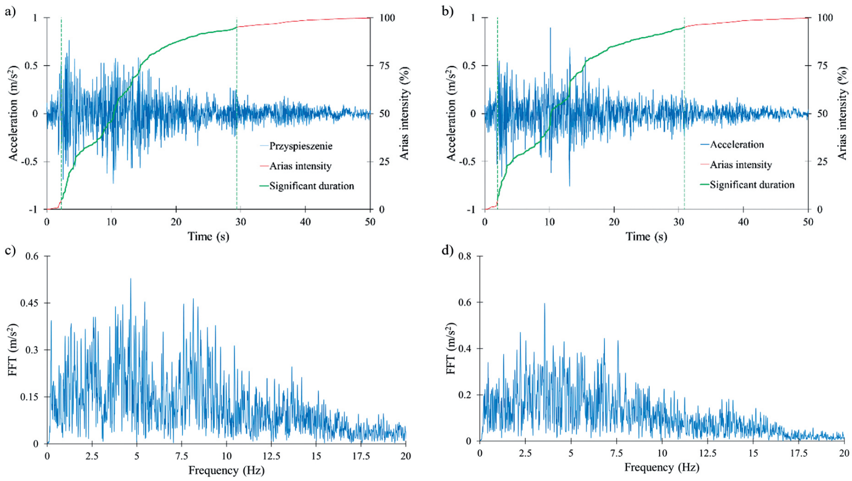

Fig. 2. The horizontal components of the Sitka earthquake, a) acceleration record in x direction, b) acceleration record in y direction, c) FFT of the acceleration record in the x direction, d) FFT of the acceleration record in the y direction

The next representative earthquake is El Centro (18.05.1940) earthquake which hit the Imperial Valey in California had a moment magnitude of 6.9. The earthquake caused extensive damage to irrigation systems and caused the deaths of nine people. Figure 3

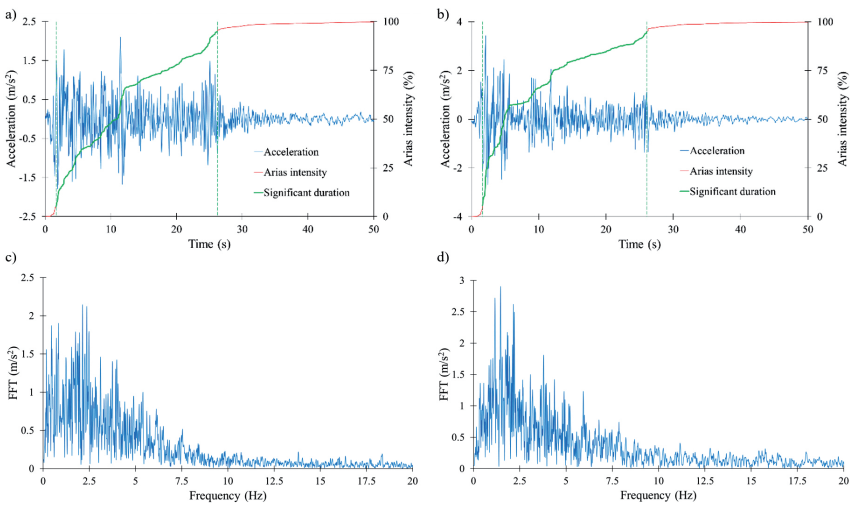

Fig. 3. The horizontal components of the El Centro earthquake, a) acceleration record in x direction, b) acceleration record in y direction, c) FFT of the acceleration record in the x direction, d) FFT of the acceleration record in the y direction

presents the horizontal components of vibration and corresponding Husid's plots. Frequency characteristics of the records from Figure 3a and Figure 3b are given using FFT and are presented in Figure 3c and Figure 3d.

The duration of the "strong phase" of this earthquake does not exceed 30 s. The values of the PGA/PGV ratio are low. In the case of the x component the value of the ratio does not exceed 5 and for the component y the value of 10. The characteristics of the horizontal components of the El Centro earthquake are summarized in Table 1.

3.3. Example mining shocks

The level of vibration acceleration from earthquakes is much higher than that from mining shocks. The dominant surface frequencies of vibrations from earthquakes are much lower than those of mining shocks. The most intense mining shocks in Poland are caused by the underground exploitation of mineral resources. Areas exposed to mining shocks include the Upper Silesian Basin (USCB), Legnicko-Głogowski Copper District (LGCD), and Beł chatowski District of Brown Coal (BDBC).

Mining tensions in the USCB are divided into two groups:

– Numerous shocks directly related to mining works; the foci of these shocks move along with the progress of the longwall fronts and the ancestors of the hollow corridors; the number of shocks is strongly dependent on the parameters of mining works and on local geological conditions; these shocks were called exploitation;
– few mining tremors, less related to currently conducted mining works; often occur at a large distance from mine workings; the long-term mining activity conducted and developed in a wider scope (sometimes even in several mines) has an impact on their formation; one can not exclude an important role as a factor triggering the shock of high-energy occurrence of faults and current exploitation, which violates the instability of the regional shock-absorbing zone; these shocks are called regional.

In the USCB area, the dominant frequencies of the horizontal components x, y of the ground vibrations remain in the 4.0–7.5 Hz band, and the vertical component in the range of 6.0–9.0 Hz. In the case of very intense vibrations (regional shocks), the vibration components from the narrow frequency band of 4.0–6.0 Hz dominate. Low- and high-intensity vibrations remain in the same frequency band of 4.0–9.0 Hz. It rarely happens that the dominant frequencies of vibrations from the USCB remain in the wider band 2.0–14.0 Hz. Figure 4 presents an example of the recorded vibration record of the ground acceleration with the corresponding result of the FFT analysis.

In the LGCD, mining tremors are of the exploitation type and result from underground mining of copper ore. Dominant frequencies of horizontal x and y surface records remain in the 1.0–7.0 Hz and 16.0-19.0 Hz bands. The vertical components of the vibrations contain only the highest dominant vibration frequencies in the band 14.0–18.0 Hz. Generally, it can be said that the bands of dominant surface vibration frequencies in this area are characterized by slightly higher values compared to the USCB area. A more detailed characterization of surface vibrations with LGCD is given in the paper (Maciag et al., 2016). Figure 5 presents an example of the recorded vibration record of ground acceleration with the corresponding result of the FFT analysis.

Lignite, besides hard coal, is a source of energy in power plants in Poland. This coal is extracted using the open-cast method. The exploitation of rich brown coal deposits in the Bechatów District of Brown Coal (BDBC) is a source of mining tremors resulting

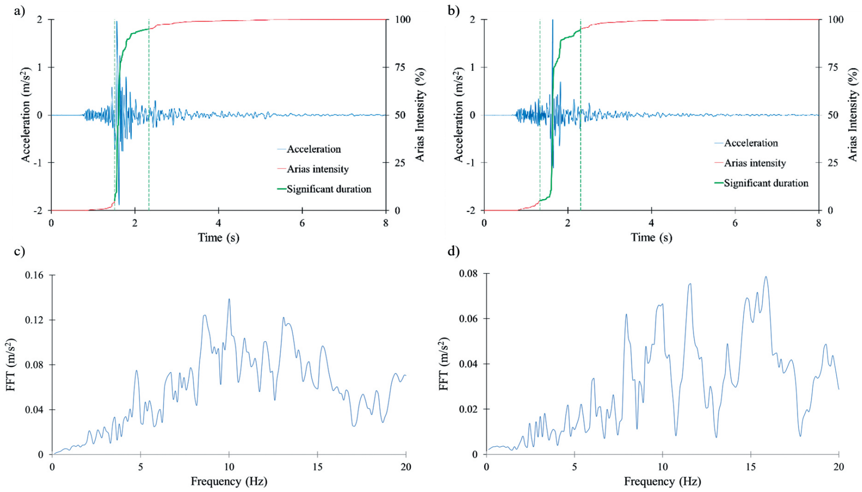

Fig. 4. The horizontal components of the USCB mining shock, a) acceleration record in x direction, b) acceleration record in y direction, c) FFT of the acceleration record in the x direction, d) FFT of the acceleration record in the y direction

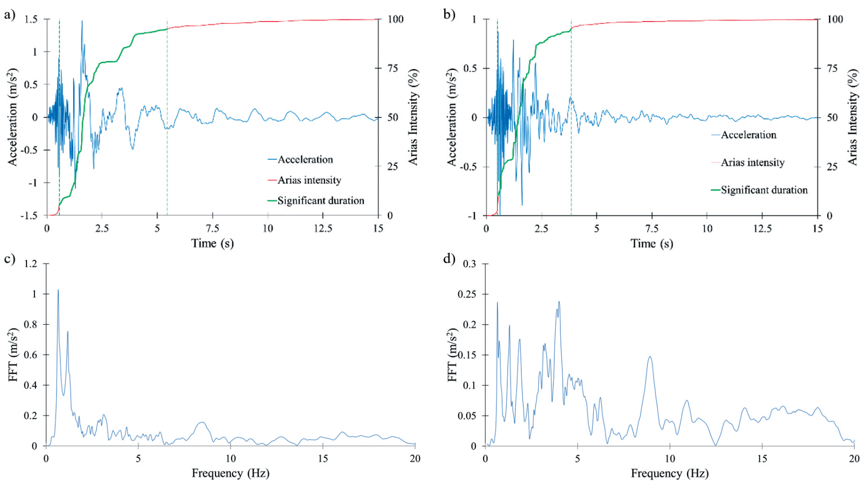

Fig. 5. The horizontal components of the LGCD mining shock, a) acceleration record in x direction, b) acceleration record in y direction, c) FFT of the acceleration record in the x direction, d) FFT of the acceleration record in the y direction

in surface vibrations. For example, in the years 1980–2012, there were seven rock mass shocks with the magnitude on the Richter scale in the range of 4.00–4.66. The energy of these tremors remained in the range of 3 · E9–1 · E11 J. Tremors of the greatest magnitude were accompanied by surface vibrations determined by acceleration and velocity exceeding 2 m/s^2 and 30 mm/s, respectively. Analyses of the FFT results of the horizontal surface vibration component records allow distinguishing the bands of the dominant vibration frequencies. The dominant frequencies are low and remain in the range of 1–5 Hz. The dominant frequencies of the vertical vibration component are lower than for horizontal components and remain in the range from 1 to 2 Hz. The mining tremors that occurred in the BDBC caused numerous architectural and structural damage to surface structures and did not pose a threat to the lives of residents. Figure 6 presents the example of the recorded vibration record of ground velocity with the corresponding result of the FFT analysis.

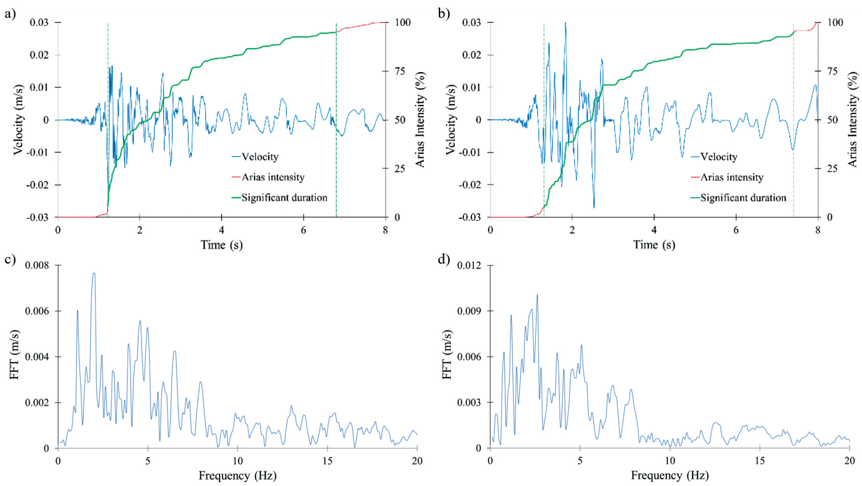

Fig. 6. The horizontal components of the BDBC mining shock, a) velocity record in the x direction, b) velocity record in the y direction, c) FFT of the velocity record in the x direction, d) FFT of the velocity record in the y direction

The duration of the intensive phase of surface vibrations is the main parameter that differentiates vibrations from mining tremors and earthquakes. The duration of vibrations from mining tremors in the USB, LGCD and the BDBC areas is shorter than vibrations from earthquakes. The duration of the intensive phase of surface vibrations from earthquakes on average is 20–30 s, but also more. From mining tremors, this duration is shorter and in most cases does not exceed 10 s.

Another parameter that differs from mining tremors and earthquakes is the ratio (PGA/PVG) of peak acceleration (PGA) to peak velocity vibration (PGV). In the case of earthquakes, this ratio is 5–30. In the USCB for mining tremors, the values of parameter PGA/PGV remain in a wide range. These values change depending on the origin of mining tremors. In the case of operational tremors, the values of this parameter remain in the range of 20–85; however, regional mining tremors associated with the occurrence of faults are characterized by lower values of PGA/PGV ratio of 18–46, with a predominance of values closer to 20 [52,53]. In the LGCD, the PGA/PGV parameter values are variable and their dependence on the mining tremors energy is visible. For mining tremors from the energy range of E6–E7, the PGA/PGV values are in the range of 20–106 and in most cases, the upper limit of this range does not exceed 90. In the case of high-energy mining tremors with energy E8–E9, PGA/PGV values remain in the range of 9 to 44 [14,54]. These values are close to the values describing earthquakes. In the BOWB area for the most intense tremors, the PGA/PGV ratio is slightly higher than the upper limit for earthquakes and does not exceed 40.

4. The numerical model of the analysed structure

The analysed structure is an RC hoist tower (see Fig. 7). It is a strategic building facility in one of the underground hard coal mines in Poland. This structure was the subject of previous preliminary research [17, 18, 55, 56]. The detailed description and dynamic

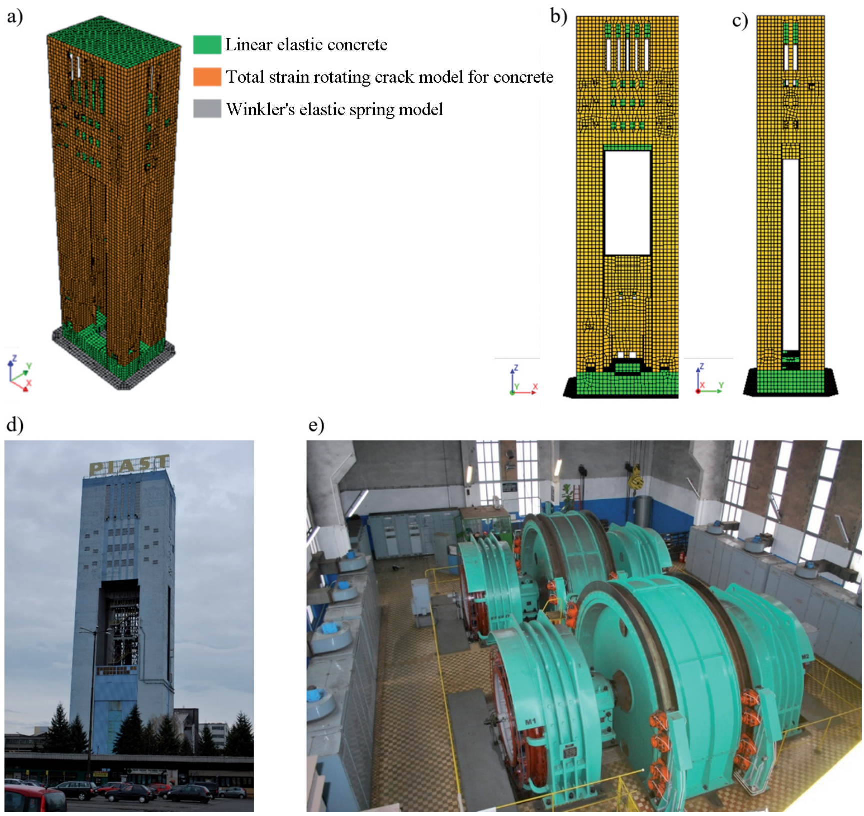

Fig. 7. Hoist tower and finite element model of the structure, a) 3D model of the tower, b) main load-bearing walls in the x-z plane, c) main load-bearing walls in the y-z plane, d) structure of the hoist tower, e) heavy weight machinery (level +80 m above the ground)

analysis of structure have been listed in [55]. The failure-free operation of this structure enables the transport of coal from underground and affects the continuous operation of the mine. In Figure 7a, the 3D model of the tower with the global coordinate system shown and a particular emphasis on the main load bearing walls is shown because we assume that damages will appear in these walls and develop in them due to affecting the excitations analysed. Besides, damage to these structural elements may affect the limit state and stability of the whole system. Figures 7b and 7c show the dimensions of the proportions of the dimensions of the load bearing wall. The model consists of shell elements modelling the foundation plate, the load-bearing walls, the floors and beams-wall finite elements. Beam elements are assumed to model beams and columns. In the tower model, eccentrics are also included. The subsoil for the foundation slab was implemented as an elastic with parameters corresponding to the local soil conditions (soil type B according to standard [19]. The experimentally verified natural frequencies of the tower confirmed the correctness of the assumption [17]. The damage total strain rotating crack model was used in the calculations for reinforced concrete (DIANA user manual 2017). The total number of finite elements of the model was equal to 82817. We applied the Finite Element DIANA code [57] in numerical analyses.

4.1. Natural frequencies of the model

Recalculation of the assumed 3-D model of the hoist tower was made due to natural frequencies. The values of the natural frequencies are the same or very close to those presented in the previous article and are confirmed based on the measurements results realised in situ [17]. It is worth emphasising that the first two calculated frequencies related to the lateral modes in two mutually perpendicular directions are the same. The values of the three first natural frequencies with values of effective modal mass vibrating in the respective directions are juxtapositioned in Table 2. The first three mode shapes corresponding to the natural frequencies of the model are shown in Figure 8.

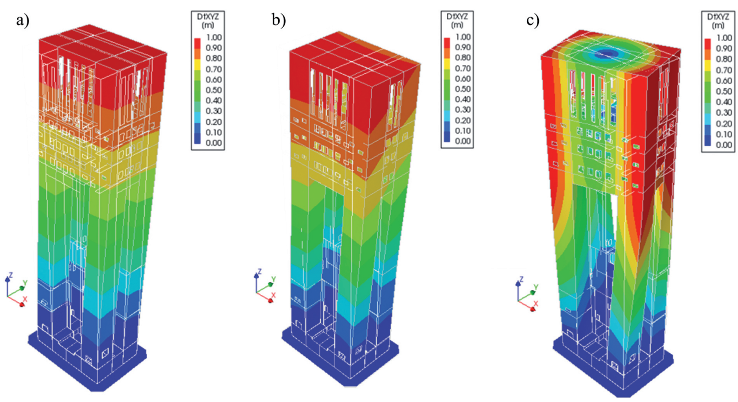

Fig. 8. First three mode shapes corresponding to the calculated natural frequencies of the finite element model, a) f_1 = 0.50 Hz, b) f_2 = 0.72 Hz, c) f_3 = 1.46 Hz

Table 2. First three natural frequencies of the model with values of effective modal mass

Mode	Frequency (Hz)	Participation factors in global directions (–)		
		X	*Y*	*Z*
1	0.50	0.33E+02	0.35E+04	0.22E+01
2	0.72	0.35E+04	0.36E+02	0.53E+01
3	1.46	0.29E+03	0.96E+01	0.28E+01

5. Comparison of selected kinematic loadings

Dynamic analysis of the hoist tower was performed using the records of horizontal components of earthquakes (comp. Figure 2 SITKA) and mining-related vibrations records (comp. Figure 5 LGCD). Records of the El Centro earthquake shown in Fig. 3 were also applied. Dynamic analysis was carried out in a non-linear range in two stages as described in detail in the article [13] using Finite Element Diana code [57]. A non-linear time history analysis (THA) was implemented in calculations using the Broyden-Fletcher-Goldfarb-Shanno (BFGS) method [58–62] belonging to quasi-Newton methods in which stiffness is updated with a constant time step which fulfils convergence criterion based on the relative norm of the last displacement increment vector [57].

5.1. Example results of calculations and discussion

The results of the numerical analyses are presented in the form of the magnitude of displacement envelopes and the envelopes of cracks in the directions of the main deformations (compare Figures 9 and 10). Envelopes of cracks refer to maximal values of crack width at the example point localised 80 m above ground level collected in each time

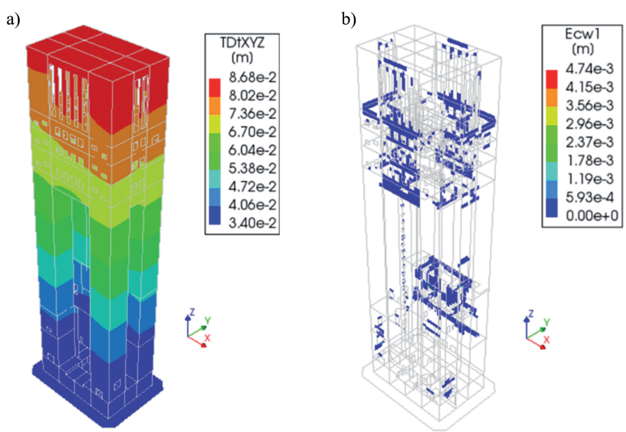

Fig. 9. Response of the tower due to the earthquake from Figure 2 – Sitka earthquake, a) envelope of the displacements, b) envelope of the cracks

step. The chosen example point lies in the corner of the opening in the outer load-bearing wall. The results of the comparative analysis of the change of crack opening in the same place for the accepted excitations are also presented.

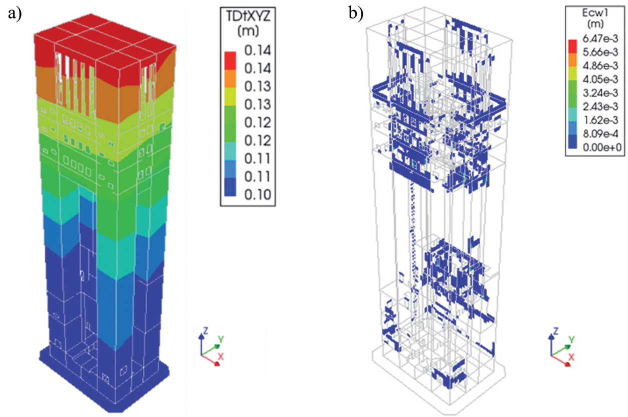

Fig. 10. Response of the tower due to the earthquake from Figure 2 – Sitka earthquake, a) envelope of the displacements, b) envelope of the cracks

The comparison of the calculated displacement envelopes from Figure 9 and Figure 10 shows that in both cases of kinematic loads the tower model response takes place at the first natural frequency. The behaviour of the structure corresponds to the cantilever work, which is mainly due to the ratio of dimensions and slenderness of the structure.

The mining shock from Figure 5 causes a greater response of the tower model than the earthquake from Figure 2 in the scope of, e.g. resulting displacements and cracks width. The maximum values of the resulting displacements of the tower are 8.68 cm and 14 cm respectively in the case of the SITKA earthquake and mining shock – comp. Figure 9a and Figure 10a. The increase in the resultant values of the maximum tower displacement calculated for the nonlinear state is over 94% compared to the result in the linear elastic state. This increase is more than twice smaller and amounts to about 40% in the case of mining shock load. Maximum crack opening values are higher for mining-related vibrations than for the Sitka earthquake. The difference is equal to 36.5%. With earthquake forces, the maximum width of the cracks in load-bearing walls do not exceed 4.8 mm – see Figure 9b. Crack width is larger and does not exceed 6.5 mm in case of mining-related excitation – comp. Figure 10b. In the tower model subjected to mining shock, more areas with cracks are observed in load-bearing walls are observed, which may be due to similar values of the natural frequencies of the tower and the dominant frequency of assumed excitation.

Examples of crack width opening vs time at a selected point in the load-bearing element are shown in Figure 11. At the selected point in the level where the machinery of the hoist is located (80m above ground level), there was the largest crack opening values for both assumed earthquake and rockburst. The dominant frequencies of the calculated crack opening records of crack openings for mining-related vibrations and earthquakes are equal to 0.65 and 0.56 Hz, respectively. These frequency values are very close to the values of the natural lateral frequencies of the tower. The duration of a significant phase of the Sitka earthquake (comp. Figure 2) is more than five times greater compared with mining-related vibrations (comp. Figure 5). It means that the duration of a significant phase of vibrations is not the reason for the difference in the results in the dynamic response for the tower. This may be the maximum value of the vibration acceleration (PGA) of the substrate and the assumed nonlinear material model of the load-bearing elements. The maximum PGA value in the case of mining shock is over 60% higher than in the case of the analysed earthquake.

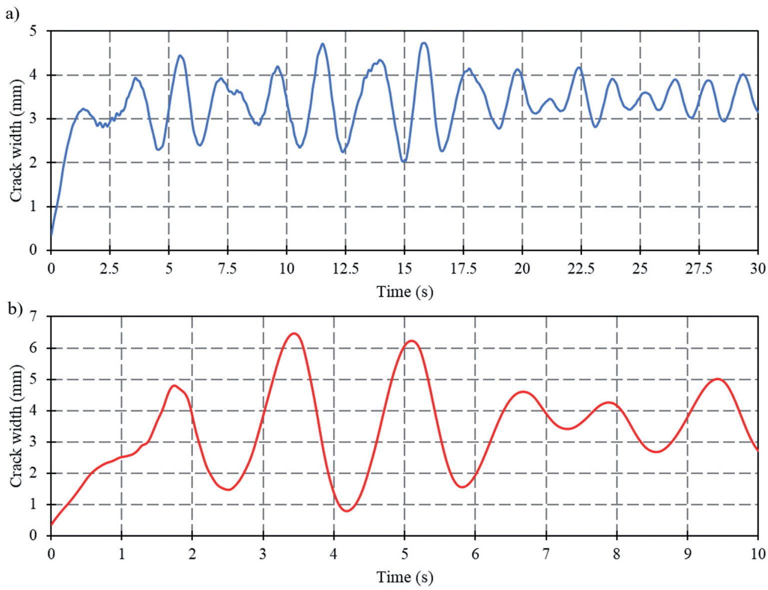

Fig. 11. Calculated time records of crack opening, a) crack opening – Sitka, b) crack opening – LGCD

5.2. Example results of the push-over analysis

Figure 12 and Figure 13 present the results of the push-over analysis. At the end of calculations, the value of resultant displacement reaches 2.84m and crack opening is equal to 10 cm – comp. Figures 12a and 12b. Analysing the envelope of the cracks opening in the main deformation directions it follows that cracks will be created practically along with the entire height of the load-bearing elements in the areas of stiffness changes after height – comp. Figure 12.

The presented results of the push-over analysis correspond to the first form of natural vibrations and just before the destruction of the structure the maximum value of (PGA) is equal to 0.13g ≈ 1.28 m/s^2. At this stage, the base shear achieved 17.28 MN and the corresponding maximal resultant displacement is equal to 0.3 m. After reaching the critical value of the base shear force, the destruction process begins, leading to the loss of stability of the tower and its collapse.

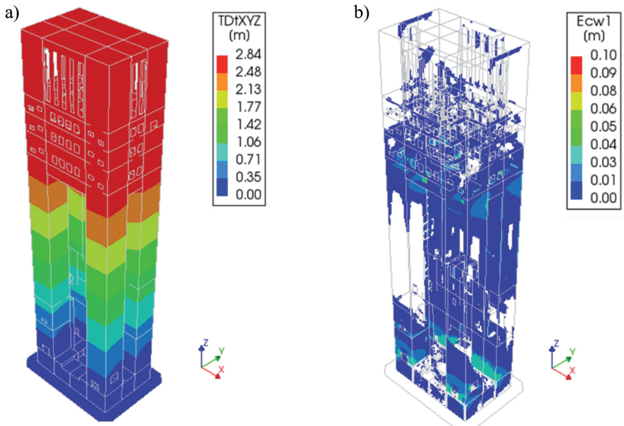

Fig. 12. Mode of destruction of the tower in push-over analysis, a) envelope of the displacements, b) envelope of the cracks

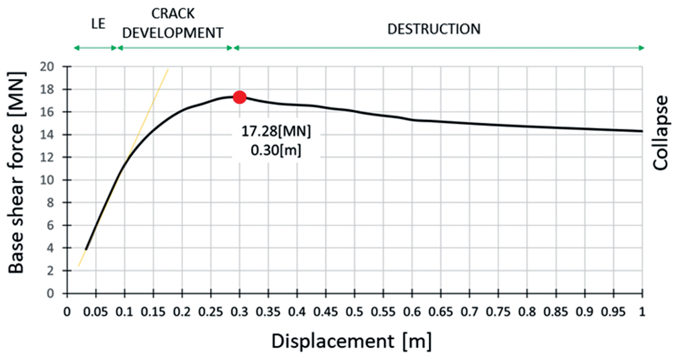

Fig. 13. The result of push-over analysis for first mode shape

6. Conclusions

The construction of a reinforced concrete hoist tower was analysed in the underground hard coal mine. These are structures that ensure safe operation of the mine and transport coal to the surface. The analyses were carried out assuming an experimentally verified spatial (3-D) structure model in the field of dynamic characteristics. The tower model includes all elements relevant to the rigidity of the system. Soil flexibility was also taken into account. The accuracy of the adopted model in comparison to measurements of vibrations in a real structure. In situ research required the necessary approval of the mine operator due to the strategic nature of the lifting tower.

The paper also compares selected representative earthquakes and mining tremors. Attention was paid to similarities and differences in surface vibrations from these sources. Significant differences in surface vibrations due to earthquakes and mining tremors were found. It was proven, using long-term monitoring of mine-induced vibrations, that the significant relationship between mining activity in the regions and the parameters characterizes surface vibrations subjected to mining-related tremors.

Dynamic analysis of the tower in the time domain was performed in a nonlinear range using representative records of surface vibrations from selected earthquakes and rockbursts. Nonlinear dynamic numerical analyses were performed using the Diana code. Two different approaches were used and compared to determine the behaviour of the construction: direct integration of the equation of motion and pushover analysis. In both cases, the nonlinear constitutive model of reinforced concrete was adopted. The iterative quasi-Newton method based on the stiffness update method (BFGS) was used. The results of dynamic analyses allowed us to determine, among others, values of the envelope of displacements and crack opening. It has been shown that the duration of the vibration records assumed as kinematic loads is not a significant factor affecting the dynamic response of the model. The parameter determining the level of dynamic response is the PGA value. Two areas where the highest crack density of structural elements occur are indicated. These are areas where there is a significant change in the rigidity of load-bearing elements along with the height of the structure. The results of observation of the location of the cracks in the structural elements of the structural elements of the hoist tower structural elements were confirmed by analysing the results of calculations.

The duration of a significant phase of the analysed records is not the reason for the difference in the results in the dynamic response for the tower. The main factor influencing the level of the dynamic response of the structure is the value of the PGA.

In addition, a pushover analysis was also performed for the adopted tower model, the results of which determined the base shear value at which the destruction of the tower support system begins, leading to the loss of stability and collapse.

References

[1] R.W. Clough, J. Penzien, *Dynamics of structures.* New York: Mc Graw-Hill Book, 1975.

[2] M.A. Khan, *Earthquake-Resistant Structures: Design, Build, and Retrofit.* Waltham: Elsevier, 2013, DOI: 10.1016/C2009-0-19143-2.

[3] O. Deck, M. Al Heib, F. Homand, "Taking the soil–structure interaction into account in assessing the loading of a structure in a mining subsidence area", *Engineering Structures*, 2003, vol. 25, no. 4, pp. 435–448, DOI: 10.1016/S0141-0296(02)00184-0.

[4] F. Khosravikia, M. Mahsuli, M.A. Ghannad, "The effect of soil–structure interaction on the seismic risk to buildings", *Bulletin of Earthquake Engineering*, 2018, vol. 16, no. 9, pp. 3653–3673, DOI: 10.1007/s10518-018-0314-z.

[5] P.K. Singh, M.P. Roy, "Damage to surface structures due to blast vibration", *International Journal of Rock Mechanics and Mining Sciences*, 2010, vol. 47, no. 6, pp. 949–961, DOI: 10.1016/j.ijrmms.2010.06.010.

[6] C.P. Lu, L.M. Dou, B. Liu, Y.S. Xie, H.S. Liu, "Microseismic low-frequency precursor effect of bursting failure of coal and rock", *Journal of Applied Geophysics*, 2012, vol. 79, pp. 55–63, DOI: 10.1016/J.JAPPGEO.2011.12.013.

[7] R.A. Stewart, W.U. Reimold, E.G. Charlesworth, W.D. Ortlepp, "The nature of a deformation zone and fault rock related to a recent rockburst at Western Deep Levels Gold Mine, Witwatersrand Basin, South Africa", *Tectonophysics*, 2001, vol. 337, no. 3-4, pp. 173–190, DOI: 10.1016/S0040-1951(01)00028-2.

[8] P. Konicek, K. Soucek, L. Stas, R. Singh, "Long-hole destress blasting for rockburst control during deep underground coal mining", *International Journal of Rock Mechanics and Mining Sciences*, 2013, vol. 61, pp. 141–153, DOI: 10.1016/j.ijrmms.2013.02.001.

[9] A.V. Lovchikov, "Review of the strongest rockbursts and mining-induced earthquakes in Russia", *Journal of Mining Science*, 2013, vol. 49, no. 4, pp. 572–575, DOI: 10.1134/S1062739149040072.

[10] E. Maciag, K. Kuzniar, T. Tatara, "Response spectra of ground motions and building foundation vibrations excited by Rockbursts in the LGC region", *Earthquake Spectra*, 2016, vol. 32, no. 3, DOI: 10.1193/020515 EQS022M.

[11] D. Olszewska, "The acceleration response spectra for Legnica-Glogow copper district", *Technical Transactions*, 2017, vol. 9, pp. 99–115, 2017, DOI: 10.4467/2353737XCT.17.151.7163.

[12] Z. Zembaty, et al., "A system to mitigate deep mine tremor effects in the design of civil infrastructure", *International Journal of Rock Mechanics and Mining Sciences*, 2015, vol. 74, pp. 81-90, DOI: 10.1016/J.IJRMMS.2015.01.004.

[13] F. Pachla, A. Kowalska-Koczwara, T. Tatara, K. Stypuła, "The influence of vibration duration on the structure of irregular RC buildings", *Bulletin of Earthquake Engineering*, 2019, vol. 17, no. 6, pp. 3119–3138, DOI: 10.1007/s10518-018-00546-4.

[14] T. Tatara, *Odporność dynamiczna obiektów budowlanych w warunkach wstrząsów górniczych. Dynamic resistance of buildings in mining tremors*. Kraków: Wydawnictwo Politechniki Krakowskiej im. Tadeusza Kościuszki, 2012 (in Polish).

[15] K. Kuźniar, T. Tatara, "Drgania pochodzenia górniczego gruntu i fundamentu budynku w ocenie ich szkodliwości. Vibrations of mining origin of the ground and building foundation in the assessment of their harmfulness", *Zeszyty Naukowe Instytutu Gospodarki Surowcami Mineralnymi i Energią Polskiej Akademii Nauk*, 2016, vol. 94, pp. 5–14 (in Polish).

[16] F. Pachla, T. Tatara, "Odporność zabudowy mieszkalnej i gospodarczej na wpływy eksploatacji górniczej w obszarze górniczym KWK Janina. Resistance of residential and rural buildings due to mining exploitation in the coal mining area", *Zeszyty Naukowe Instytutu Gospodarki Surowcami Mineralnymi i Energią Polskiej Akademii Nauk*, 2017, vol. 101, pp. 45–60 (in Polish).

[17] T. Tatara, F. Pachla, P. Kuboń, "Experimental and numerical analysis of an industrial RC tower", *Bulletin of Earthquake Engineering*, 2017, vol. 15, no. 5, pp. 2149–2171, DOI: 10.1007/s10518-016-0053-y .

[18] T. Tatara, P. Kuboń, "Theoretical and experimental investigation of the steel hoist tower", in *Proceedings of the 2009: 5th International Conference Concrete and Concrete Structures, C&CS 2009, 15–16 October 2009, Zilina, Slovakia.* EDIS – University of Zilina, 2009, pp. 393–400.

[19] *EN 1998-1:2004 Eurocode 8 Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings*, 2004.

[20] T. Tatara, F. Pachla, "Analysis of the dynamic response of masonry buildings with irregularities of localization of bearing elements due to mining shocks" in *Seismic Behaviour and Design of Irregular and Complex Civil Structures II. Geotechnical, Geological and Earthquake Engineering*, vol 40, Z. Zembaty, M. De Stefano, Eds. Cham: Springer International Publishing, 2016, pp. 275–287, DOI: 10.1007/978-3-319-14246-3_25.

[21] Z. Zembaty, "Rockburst induced ground motion-a comparative study", *Soil Dynamics and Earthquake Engineering*, 2004, vol. 24, no. 1, pp. 11–23, DOI: 10.1016/j.soildyn.2003.10.001.

[22] W. Kowalski, E. Maciąg, T. Tatara, "Normalized response spectra for mining tremors and their application", in *ECEE 1998: Proceedings of the: Eleventh European Conference on Earthquake Engineering 1998, 96–11 September 1998, Paris, France*, P. Bish, et al., Eds. Rotterdam: Balkema, 1998.

[23] L. Czerwionka, T. Tatara, "Wzorcowe spektra odpowiedzi z wybranych obszarów GZW. Standard response spectra from chosen mining regions at Upper Silesian Coalfield", *Czasopismo Techniczne. Budownictwo*, 2007, vol. 104, no. 2–B, pp. 11–18 (in Polish).

[24] D. Vamvatsikos, C.A. Cornell, "Incremental dynamic analysis", *Earthquake Engineering and Structural Dynamics*, 2002, vol. 31, no. 3, pp. 491–514, DOI: 10.1002/EQE.141.

[25] C.W. Lin, "Time History Input Development for the Seismic Analysis of Piping Systems", *Journal of Pressure Vessel Technology*, 1980, vol. 102, no. 2, pp. 212–218, DOI: 10.1115/1.3263322.

[26] F. Naeim, Ed., *The Seismic Design Handbook*. Boston: Kluwer, 2001, DOI: 10.1007/978-1-4615-1693-4.

[27] G. Harsha, S.A. Raza, "Seismic Vulnerability of RC Building With and Without Soft Storey Effect Using Pushover Analysis", *International Journal of Modern Engineering Research (IJMER)*, 2014, vol. 4, no. 9, pp. 32–47.

[28] F.C. Day, A.W.C. Oretaa, "Seismic Vulnerability Assessment of Soft Story Irregular Buildings Using Pushover Analysis", *Procedia Engineering*, 2015, vol. 125, pp. 925–932, DOI: 10.1016/J.PROENG.2015.11.103.

[29] M.N. Aydinoglu, "A response spectrum-based nonlinear assessment tool for practice: Incremental Response Spectrum Analysis (IRSA)", *Journal of Earthquake Technology*, 2007, vol. 44, no. 1, pp. 169–192.

[30] K. Gupta, *Response Spectrum Method in Seismic Analysis and Design of Structures.* London, England: Routledge, 1990, DOI: 10.1201/9780203740781.

[31] M. Berrah, E. Kausel, "Response spectrum analysis of structures subjected to spatially varying motions", *Earthquake Engineering and Structural Dynamics*, 1992, vol. 21, no. 6, pp. 461–470, DOI: 10.1002/EQE.4290210601.

[32] P. Fajfar, "A Nonlinear Analysis Method for Performance-Based Seismic Design", *Earthquake Spectra*, 2000, vol. 16, no. 3, pp. 573–592, DOI: 10.1193/1.1586128.

[33] A.K. Chopra, R.K. Goel, "A modal pushover analysis procedure for estimating seismic demands for buildings", *Earthquake Engineering and Structural Dynamics*, 2002, vol. 31, no. 3, pp. 561–582, DOI: 10.1002/EQE.144.

[34] E. Thorenfeldt, A. Tomazewic, J.J. Jensen, "Mechanical properties of high-strength concrete and application in design", in *Proceeding of the 1987: Symposium on Utilization of High-Strength Concrete*. 1987, pp. 149–159.

[35] M. Fafard, B. Massicotte, "Geometrical interpretation of the arc-length method", *Computers and Structures*, 1993, vol. 46, no. 4, pp. 603–615, DOI: 10.1016/0045-7949(93)90389-U.

[36] S.P. Venkateshan, P. Swaminathan, "Numerical Integration", in *Computational Methods in Engineering*. Elsevier, 2014, pp. 317–373, DOI: 10.1016/B978-0-12-416702-5.50009-0.

[37] M.D. Trifunac, "Earthquake response spectra for performance based design—A critical review", *Soil Dynamics and Earthquake Engineering*, 2012, vol. 37, pp. 73–83, DOI: 10.1016/J.SOILDYN.2012.01.019.

[38] O.A. López, J.J. Hernández, R. Bonilla, A. Fernández, "Response Spectra for Multicomponent Structural Analysis", *Earthquake Spectra*, 2019, vol. 22, no. 1, pp. 85–113, DOI: 10.1193/1.2162898.

[39] Z. Zembaty, R. Jankowski, A. Cholewicki, J. Szulc, "Trzęsienia ziemi w Polsce w 2004 roku. Earhquakes in Poland in 2004", *Czasopismo Techniczne. Budownictwo*, 2007, vol. 104, no. 2-B, pp. 115–126 (in Polish).

[40] E. Juhásová, P. Pezzoli, E.M. Da Rin, R. Sofronie, Z. Zembaty, "Resistance of brick building model with arches before and after retrofitting", in *ECEE 1998: Proceedings of the : Eleventh European Conference on Earthquake Engineering 1998, 96–11 September 1998, Paris, France,* P. Bish, et al., Eds. Rotterdam: Balkema, 1998.

[41] T. Furumura, S. Takemura, S. Noguchi, et al., "Strong ground motions from the 2011 off-the Pacific-Coast-of-Tohoku, Japan (Mw=9.0) earthquake obtained from a dense nationwide seismic network", *Landslides*, 2011, vol. 8, pp. 333–338, DOI: 10.1007/s10346-011-0279-3.

[42] S.K. Kufner, et al., "Zooming into the Hindu Kush slab break-off: A rare glimpse on the terminal stage of subduction", *Earth and Planetary Science Letters*, 2017, vol. 461, pp. 127–140, DOI: 10.1016/j.epsl.2016.12.043.

[43] A. Akinci, L. Malagnini, F. Sabetta, "Characteristics of the strong ground motions from the 6 April 2009 L'Aquila earthquake, Italy", *Soil Dynamics and Earthquake Engineering*, 2010, vol. 30, no. 5, pp. 320–335, DOI: 10.1016/J.SOILDYN.2009.12.006.

[44] E. Chioccarelli, I. Iervolino, "Near-source seismic demand and pulse-like records: A discussion for L'Aquila earthquake", *Earthquake Engineering and Structural Dynamics*, 2010, vol. 39, no. 9, pp. 1039–1062, DOI: 10.1002/eqe.987.

[45] M.D. Trifunac, A.G. Brady, "A study on the duration of strong earthquake ground motion", *Bulletin of the Seismological Society of America*, 1975, vol. 65, no. 3, pp. 581–626, DOI: 10.1785/BSSA0650030581.

[46] R.D. Koehler, G.A. Carver, Alaska Seismic Hazards Safety Commission, *Active faults and seismic hazards in Alaska*. Alaska Division of Geological & Geophysical Surveys, 2018, DOI: 10.14509/29705.

[47] M. Wyss, "A search for precursors to the Sitka, 1972, earthquake: Sea level, magnetic field, andP-residuals", *Pure and Applied Geophysics*, 1975, vol. 113, no. 1, pp. 297–309, DOI: 10.1007/BF01592919.

[48] M.M. Schell, L.J. Ruff, "Rupture of a seismic gap in southeastern Alaska: the 1972 Sitka earthquake (Ms 7.6)", *Physics of the Earth and Planetary Interiors*, 1989, vol. 54, no. 3-4, pp. 241–257, DOI: 10.1016/0031-9201(89)90246-X.

[49] L.R. Sykes, "Aftershock zones of great earthquakes, seismicity gaps, and earthquake prediction for Alaska and the Aleutians", *Journal of Geophysical Research*, 1971, vol. 76, no. 32, pp. 8021–8041, DOI: 10.1029/JB076i032p08021.

[50] R.A. Page, "The Sitka, Alaska, earthquake of 1972: An expected visitor", *Earthquake Information Bulletin*, 1973, vol. 5, no. 5, pp. 4–9.

[51] D. Tocher, "The Alaska earthquake of July 10, 1958: Movement on the Fairweather fault and field investigation of southern epicentral region", *Bulletin of the Seismological Society of America*, 1960, vol. 50, no. 2, pp. 267-292, DOI: 10.1785/BSSA0500020267.

[52] G. Mutke, J. Chodacki, "Charakterystyka parametrów drgań od najsilniejszych wstrząsów regionalnych w GZW w aspekcie kryteriów oceny intensywności w oparciu o skalę GSIGZW. Characteristics of vibration parameters from the strongest regional shocks in the GZW in terms of intensity assessment criteria based on the GSIGZW scale", *Scientific Papers of Central Mining Institute, Mining and Environment*, 2010, no. 4, pp. 65–80 (in Polish).

[53] K. Stec, A. Barański, "Charakterystyka i ocena skutków drgań wstrząsów w kopalniach Kompanii Węglowej SA o pikowym przyspieszeniu drgań gruntu powyżej 0.5 m/s^2. Characteristics and evaluation of the shock vibration effects in the mines of Kompania Węglowa SA with the peak acceleration of ground vibrations above 0.5 m/s^2", *Scientific Papers of Central Mining Institute, Mining and Environment*, 2010, no. 4, pp. 81–100 (in Polish).

[54] K. Kuzniar, T. Tatara, "The ratio of response spectra from seismic-type free-field and building foundation vibrations: the influence of rockburst parameters and simple models of kinematic soil-structure interaction", *Bulletin of Earthquake Engineering*, 2020, vol. 18, no. 3, pp. 907–924, DOI: 10.1007/s10518-019-00734-w.

[55] T. Tatara, F. Pachla, "Influence of mining tremors on industrial structures", in *Proceedings of 9th International Conference on New Trends in Statics and Dynamics of Buildings, 20-21 October, Bratislava, Slovakia*, N. Jendzelovsky, M. Nagyova, Eds. Bratislava: Slovak University of Technology in Bratislawa, 2011, pp. 153–156.

[56] T. Tatara, F. Pachla, "Influence of earthquakes and mining related surface vibrations on RC skip tower", in WCEE 2012: Proceedings of the 15th World Conference on Earthquake Engineering, WCEE 2012, 24–28 September 2012, Lisbon, Portugal, Eds. London: Oxford, 2012.

[57] "DIANA (2017) Finite Element Analysis User's Manual release 10.2 – Bing". [Online]. Available: https://manuals.dianafea.com/d102/Analys/Analys.html. [Accessed: 19. May 2022].

[58] C.G. Broyden, "The Convergence of a Class of Double-rank Minimization Algorithms 1. General Considerations", *IMA Journal of Applied Mathematics*, 1970, vol. 6, no. 1, pp. 76–90, DOI: 10.1093/IMAMAT/6.1.76.

[59] R. Fletcher, "A new approach to variable metric algorithms", *The Computer Journal*, 1970, vol. 13, no. 3, pp. 317–322, DOI: 10.1093/COMJNL/13.3.317.

[60] D. Goldfarb, "A family of variable-metric methods derived by variational means", *Mathematics of Computation*, 1970, vol. 24, no. 109, pp. 23–26, DOI: 10.1090/S0025-5718-1970-0258249-6.

[61] D.F. Shanno, "Conditioning of quasi-Newton methods for function minimization", *Mathematics of Computation*, 1970, vol. 24, no. 111, pp. 647–656, DOI: 10.1090/S0025-5718-1970-0274029-X.

[62] R. Fletcher, *Practical Methods of Optimization*. New York: John Wiley and Sons, 2000.

Analiza nieliniowa wieży wyciągowej na obciążenia sejsmiczne

Słowa kluczowe: trzęsienia ziemi, konstrukcje przemysłowe, wstrząsy górnicze, analiza nieliniowa, drgania powierzchni, analiza w dziedzinie czasu

Streszczenie:

Dodatkowym obciążeniem działającym na konstrukcje budowlane są drgania przenoszone przez grunt. Z reguły takie konstrukcje, gdy znajdują się poza obszarami trzęsienia ziemi, nie są przystosowane do takich dodatkowych obciążeń. Na terenach dotkniętych wstrząsami górniczymi konstrukcje nie były projektowane na takie obciążenia. W projekcie uwzględniono tylko obciążenia stałe i ciężar własny, obciążenia technologiczne oraz obciążenia od podmuchów wiatru. W ostatnich latach w Polsce wzrosła intensywność obciążeń powodowanych działalnością górniczą. Eksploatacja ta często prowadzona jest na terenach zurbanizowanych, dlatego ich eksploatacja i wpływ na budynki staje się nie tylko problemem społecznym, ale także wyzwaniem dla inżynierów. Ich zadaniem jest zapewnienie bezpieczeństwa konstrukcji i osób przebywających w tych budynkach. Wiele obiektów naziemnych znajduje się bezpośrednio na terenach kopalni. Ich bezpieczne użytkowanie wpływa na bezawaryjną pracę kopalni. Awarie obiektów powierzchniowej infrastruktury budowlanej kopalni prowadzą do dużych strat finansowych i powodują problemy społeczne. W pracy przedstawiono wyniki reprezentatywnych pomiarów drgań powierzchniowych z terenów górniczych w Polsce. Drgania te były spowodowane najintensywniejszymi wstrząsami górniczymi, jakie wystąpiły podczas podziemnej eksploatacji kopani węgla kamiennego i rud miedzi w Polsce. Wyniki pomiarów in-situ tych drgań porównano z zapisami drgań z wybranych trzęsień ziemi. Szczególną uwagę zwrócono na wartości wskaźnika PGA/PGV, a także różnice charakteryzujące drgania powierzchniowe indukowane podziemną eksploatacją górniczą i wstrząsy sejsmiczne oraz czas trwania intensywnej fazy drgań. Następnie przedstawiono najczęściej stosowane metody obliczeń dynamicznych konstrukcji. W ostatniej części pracy zaprezentowano numeryczny model dynamiczny wybranej żelbetowej konstrukcji wieży wyciągowej. Drgania wymuszone przyjętego, zweryfikowanego eksperymentalnie modelu wybranej konstrukcji wieżowej zostały przeanalizowane, przyjmując wymuszenia kinematyczne w postaci reprezentatywne przebiegów drgań powierzchniowych od trzęsień ziemi i wstrząsów pochodzenia górniczego. Wykorzystano nieliniową analizę w dziedzinie czasu oraz metodę push-over. W analizach przyjęto nieliniowy model betonu. Wyniki pokazują, że analiza metodą push-over nie jest w stanie uchwycić wymagań sejsmicznych narzuconych przez ruchy gruntu w polu dalekim lub bliskim, szczególnie w przypadku układów o krótkim okresie drgań; w takich przypadkach analiza push-over może prowadzić do znacznych błędów w oszacowaniu wymagań sejsmicznych. Porównano również wyniki analiz nieliniowych z wynikami wcześniej wykonanych obliczeń w zakresie liniowym. Wyniki potwierdziły jakościowe wyniki analizy liniowej. Przeprowadzona inwentaryzacja uszkodzeń wieży wyciągowej pozwoliła również na ich lokalizację w elementach nośnych. Lokalizacje tych uszkodzeń porównano z wynikami analiz numerycznych. Wyniki nieliniowych analiz numerycznych pozwoliły ocenić bezpieczeństwo konstrukcji.

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