Influence of earthquakes and mining related surface vibrations on RC skip tower

T. Tatara
Cracow University od Technology

F. Pachla
Cracow University od Technology

SUMMARY:
The study presents the results of the analyses performed on the dynamic 3D model of the RC skip tower which belongs to a coal mine located in Poland. The kinematic forcing was adopted in the form of the acceleration records caused by a rockburst and by the earthquake in Sitka in Alaska. Although Poland is not an active seismic region, there are regional seismic phenomena associated with mineral exploitation. Mining – related surface vibrations show some similarities but also differences when compared with natural earthquakes. Dynamic calculations were made using modal superposition in the time domain. The analysis of calculation results refers to the distribution of stresses and displacements which were compared to the limit values. The additional goal of the calculations was to check the possibility of the structure losing its stability and falling over. The reason for making the analysis was the fact that kinematic excitations of high structures had not been taken into account in the design procedure. Using an actual recorded surface vibrations caused by rock burst and earthquake as kinematics loads, led to the comparison of the dynamic responses of the structure to two different seismic events. The calculated dynamic results were also compared with those obtained for wind load which had been the base load at the design stage.

Keywords: Earthquake, rockburst, surface vibrations, industrial structures

1. INTRODUCTION

Kinematic loads are particularly essential among many loads which can act on buildings. Such loads can result from seismic shocks or paraseismic events. In Poland there are regions of large paraseismic activity caused by underground mining exploitation. Among particularly active regions are: the Upper Silesian Coal Basin (GZW), the Legnicko-Głogowski Copper Region (LGOM) and the Bełchatowski Brown Coal Region (the BOWB). In many cases, the structures situated in these areas were not designed and not even verified on additional seismic loads as the results of buildings vibration. This also concerns strategic objects for a mine with regard to its operation. Shaft towers, which transport output from a mine, are surely such objects. The study dealt with the dynamic analysis of the RC shaft tower located near a pit-coal at GZW. Wind load was the basic load comprised in the design process because of the height of the tower. Dynamic loads originating from mining operation were not included. Therefore, it seems justifiable to make dynamic calculations in order to check the dynamic resistance of the tower to additional inertia caused by mining shocks.

2. ANALYSED STRUCTURE

The tower discussed is made of reinforced concrete and is supported by four pylons (columns). Its dimensions are 6,7 m x 6,5 m and the thickness of walls is 30cm. The columns are founded by means of RC grillage. The foundation of the tower consists of RC plate with the thickness of about 1.5 m. The plan dimension of the rectangle plate is 25 x 34m with truncated corners (2.5 x 2.5 m) as well as 10 m diameter opening for a shaft pipe. The foundation is 7.5 m below the ground level. The foundation plate in the basement part is stiffened by the RC grillage with the 6-metre high walls. The beams hidden in the thickness of plate constitute the ribs of the grill. The ribs run under the walls of the tower. The basic data characterizing the construction of the tower and the technical devices [Technical description (1974), Static calculation (1974) & The technical drawings (1974)] are as follows:

- dimension of transverse section - 27,30 x 18,30 m,
- total height of the tower from foundation level - 101.77 m,
- height above the ground level - 95.77 m,
- level of axis of fly-wheels - 80,01 m,

Figure 1 shows a picture of the analyzed structure. A more detailed description of the structure can be found in [Tatara & Pachla 2011].

Figure 1. The analyzed RC skip tower

3. THEORETICAL MODEL

First of all, the principal stage of the dynamical study was focused on creating the numerical model of a discussed hoist tower. The model of the structure was built using the finite element code ALGOR and available project records [Technical description (1974), Static calculation (1974) & The technical drawings (1974)]. Taking into account the formulation of the dynamic problem concerning the 3D structure, the stiffness and the mass of the tower were thoroughly imitated in the spatial FEM model – Fig. 2. The global co-ordinates shown in Fig. 2 permit to univocally describe geometry, loads and interpretation of the received results of calculations.

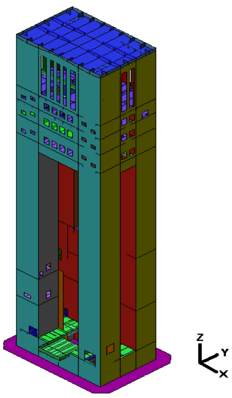


Figure 2. Dynamic model of the hoist tower with received global arrangement of co-ordinates

There is certain repeatability of geometry visible; however, there is no full symmetry of structure. Therefore, the 3D model of the tower was analyzed. All essential elements influencing the stiffness of the model were considered. The steel trunk serving transportation of output from the shaft was not considered in the model. Discretization of the model results in 82817 finite elements. The FEM in displacement version was used in analysis. It means that all qualities were calculated using approximation of displacement field inside each element on the basis of the displacement values in nodes. The bearing walls, ceilings, the foundation plate as well as the beam - wall elements were modelled by finite elements of the "plate / shell” type with five degrees of freedom in every node of the element. The beams and columns were modelled by finite elements of the ”beam” type " with six degrees of freedom in every node of the element. Additionally, for more exact imitation of the ceilings stiffness, centres of gravity of beam elements were modelled including their eccentricies. The values of coefficients characterizing the physical proprieties of materials were assumed according to the data from the archival project records, the data from codes valid during the design procedure in former time, as well as the data from literature [Ciesielski 1973, Korczyński & Bieczeniewa 1966]. Linear – spring mathematical models for all constructional materials were adopted. These values are listed in Table 3.1.

Table 3.1. Characteristics of assumed materials

Material	**Density ρ [kg/m3]**	**Young’s modulus E [GPa]**	**Poissons’s coefficient [-]**
1	*2*	*3*	*4*
Concrete „170”	2500	26	0,167
Concrete „200”	2500	29	0,167
Concrete „250”	2500	32	0,167
Steel St3S, 18G2a	7850	205	0,3

Young’s modulus of concrete both in static and dynamic analysis has been calculated according to Graff’s relation (3.1) [Ciesielski 1973, Korczyński & Bieczeniewa 1966)]:

$$E = \frac{1000000}{1{,}7 + \frac{360}{R_w}} \tag{3.1}$$

where:
R_w - mean compressive strength of the concrete

Different properties of materials for the bearing elements in the dynamic model of the tower were considered. The combination of load representation describes dependence (3.2) according to code [PN-85/B-02170]:

$$Q_k = Q_k^{'} + 0{,}6 \cdot Q_k^{''} \tag{3.2}$$

where:
Q_k’ – dead load
Q_k’’ – technological load

The influence of soil properties was considered by means of applying springs at the bottom of the foundation of the model. The springs were assumed in horizontal directions as well as the vertical one at the bottom level of the foundation. The values characterizing their properties correspond to code ones [PN-80/B-03040].

4. DYNAMIC ANALYSIS AND RESULTS OF CALCULATIONS

4.1. Natural frequencies of the model

The analysis of natural frequencies was conducted for a model of the tower. The natural frequencies and the corresponding mode shapes were appointed using the FEM model of the tower. The range of natural frequencies is very dense. The chosen natural frequencies and mode shapes are shown in Fig.

4. These modes correspond to vibrations of the model as a whole structure. The analysed model has a lot of natural frequencies and mode shapes corresponding to local vibrations of elements. They are taken into account in dynamic analysis using spectrum method (RSA). Shapes of modes corresponding to natural horizontal frequencies in y and x direction, as well as the first torsional frequency of the analysed model are shown in Fig. 3. The first 50 values of natural frequencies of the model are juxtapositioned in Table 4.1.

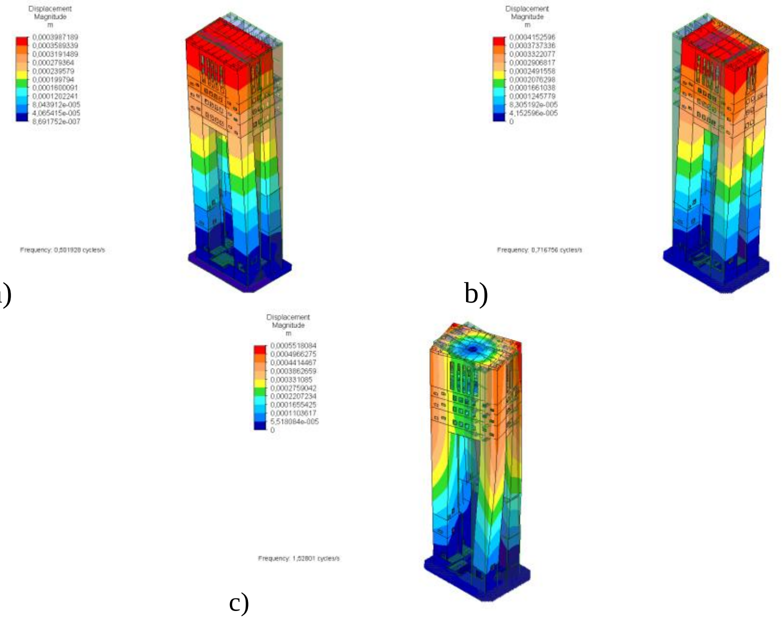


Figure 3. Chosen three first mode shapes of the model corresponding to natural frequencies

Table 4.1. The first 50 natural frequencies of the model

No of frequency	Natural circular frequency [rad/s]	No of frequency	Natural circular frequency [rad/s]
1	3,15	**26**	36,71
2	4,5	**27**	36,76
3	9,6	**28**	37,81
4	13,08	**29**	38,92
5	13,08	**30**	38,92
6	19,23	**31**	39,14
7	26,54	**32**	39,54
8	26,54	**33**	40,19
9	26,58	**34**	40,20
10	28,27	**35**	40,38
11	28,58	**36**	46,57
12	30,10	**37**	46,91
13	30,80	**38**	47,79
14	31,71	**39**	49,49
15	33,33	**40**	49,51
16	33,34	**41**	51,63
17	34,03	**42**	51,93
18	34,97	**43**	52,33
19	35,00	**44**	53,04
20	35,00	**45**	53,33
21	35,01	**46**	53,87
22	35,83	**47**	53,88
23	36,35	**48**	54,51
24	36,45	**49**	54,83
25	36,55	**50**	55,26

4.2. Assumed kinematic loads and dynamic analysis

The kinematic excitations were adopted in the form of the records of the horizontal components of ground acceleration vibrations caused by earthquakes and rockburst. The earthquake records were registered in the district of Sitka, Alaska (USA). The mining – related components of horizontal ground acceleration vibrations refer to one of the most intensive phenomena. The records of vibrations caused by the earthquake are shown in Figure 4. The parameters characterizing the records of vibrations are shown in Table 4.2.

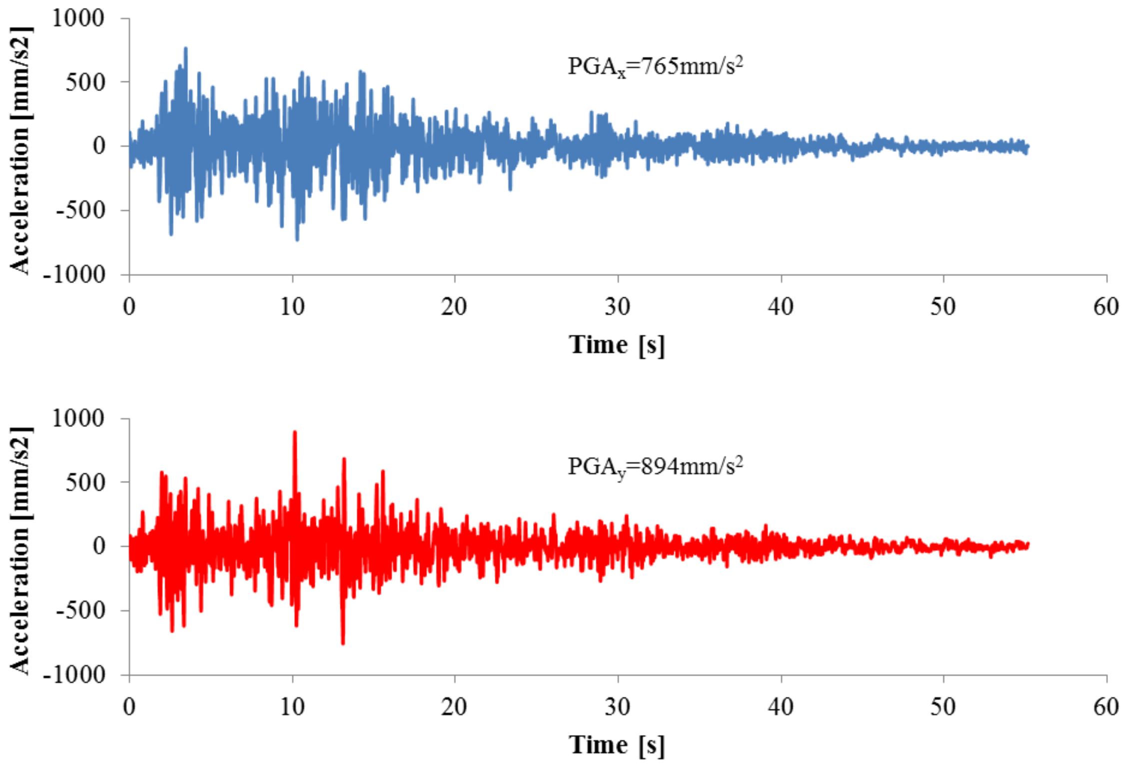


Figure 4. The horizontal components of vibration records of the analyzed earthquake

The calculations also assumed ground vibration records caused by the mining shock. Unlike earthquakes, which result simply from tectonic plates pushing against one another, mining tremors are a regional phenomenon, dependent on human activities. In Poland, their occurrence is related to the mining of mineral resources such as coal, lignite and copper ore. When you excavate a part of the deposit and take out some rocks bordering the selected space, the zones of increased pressure are formed in the rock mass surrounding the excavation zone, and the stress state seems to deviate from the normal one. As a result, elastic energy accumulates in the subsurface. It does not last for a long time and leads to a spontaneous process of the rocks regaining their balance. Elastic energy accumulated in rocks, after it exceeds the rock strength, is rapidly discharged in a manner similar to an explosion. The sudden relaxation of the rock mass at the hypocentral depths at a focus is connected with seismic waves that reach the surface due to the elastic energy release. In the surface layer surface waves are formed. They have the greatest impact on buildings, particularly their horizontal components. One of the most exploited areas in Poland is the Legnica-Glogowski Copper District (LGOM). Every year several strong shocks of energies - not less than 10^7 J are recorded. One of the most intense shocks that were registered in LGOM (since the time the continuous registration was introduced), took place on 21th May 2006. Horizontal components of acceleration vibrations for that shock registered on one of numerous seismic stations located in the LGOM were used as kinematic load forcing for the analyzed skip tower. Fig.5 depicts horizontal components of acceleration vibrations for this shock. Table 4.2 presents a comparison of basic parameters characterizing surface horizontal components of the vibrations analyzed. The duration of the intensive phase of acceleration for each horizontal component of vibration was calculated from the standard chart of Arias intensity. The summary of the parameters characterizing the analyzed components of vibrations shows some differences between earthquakes and mining tremors.

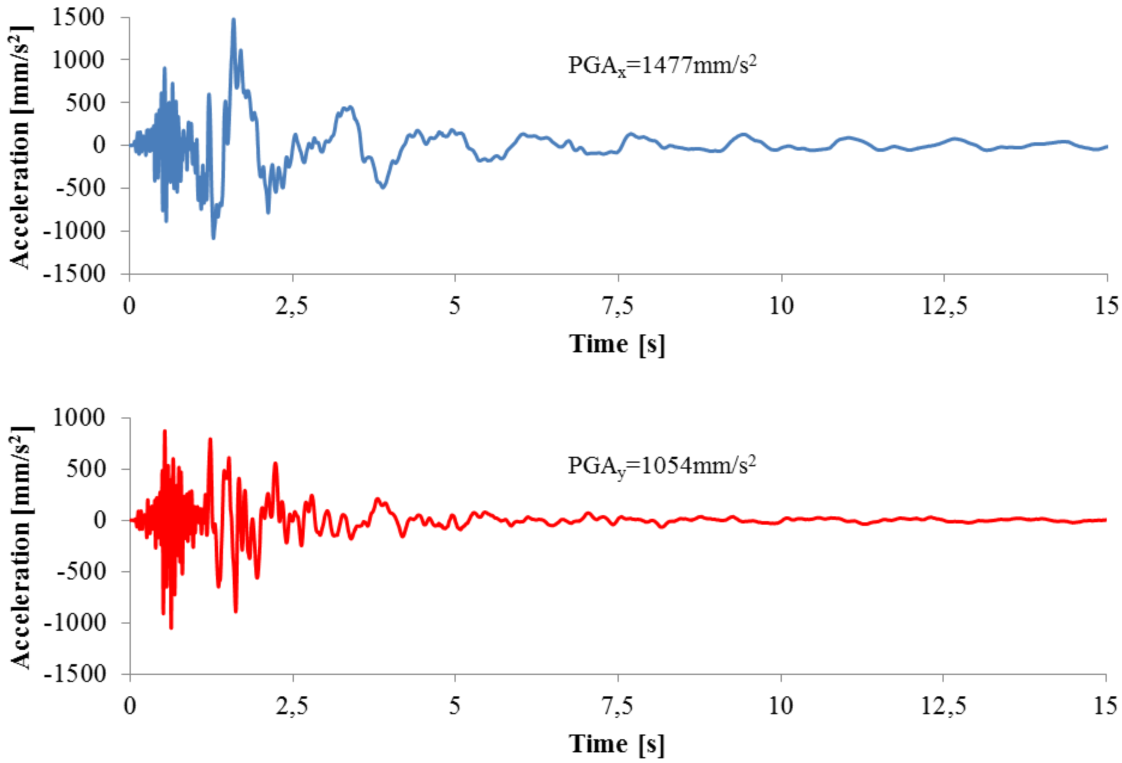


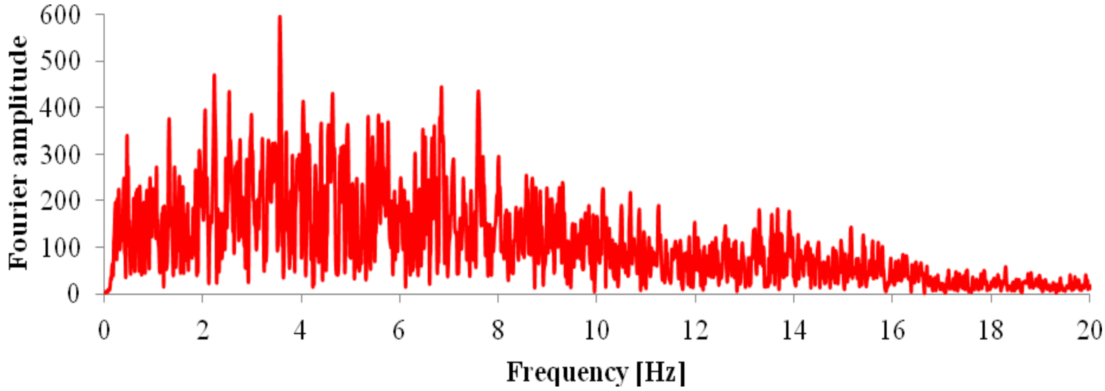
Figure 5. The horizontal components of vibration records of the analysed mining tremor

Table 4.2. Characteristics of the analyzed components of vibrations

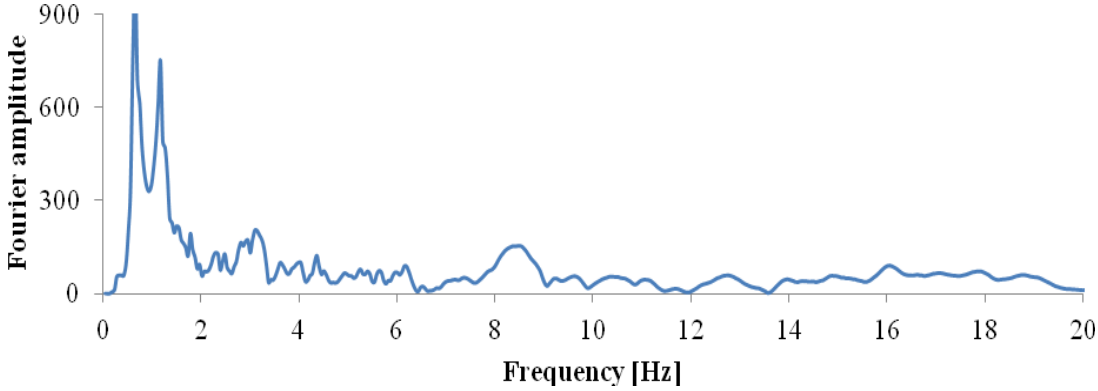
	PGA_x, mm/s^2	PGA_y, mm/s^2	PGV_x, mm/s	PGV_y, mm/s	PGA_x/PGV_x,	PGA_y/PGV_y, s	Significant duration in x,s	Significant duration in y, s
1	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*
SITKA	765	894	74,21	67,02	10,31	13,34	27,1	28,88
LGOM	1477	1054	180,21	48,16	8,20	21,89	5,13	3,41

The most significant difference between earthquakes and mining tremors is the duration of the intensive phase of the vibration, which in case of the analyzed earthquake lasted 30 seconds. In case of a mining tremor, it slightly exceeded 5 seconds. The frequency characteristics of the assumed seismic occurrences are also different. Fig. 6 shows Fourier spectrum of more intensive components of the examined vibrations. The values of PGA/PGV ratio are similar for both events. Higher values of maximal acceleration are recorded for a mining shock than for an earthquake. It should be emphasized that the mining shock analysed in the study was one of the most intensive events in this area, but normally maximal values of acceleration do not exceed 0,2g (g – acceleration of gravity).
Dynamic calculations were performed in time domain using modal superposition method, assuming that the phenomena discussed above are kinematic excitations. In each case of the calculations we assumed a dominant horizontal component of vibration in the direction of the lower rigidity of the model. The first one hundred mode shapes of the skip tower were taken into account. The value of

critical damping ratio ξ = 2,5% was adopted. Table 4.3 presents a comparison of selected results for the two adopted kinematic excitations. The calculation does not include any other loads, but only kinematic forcing. Calculated stresses and displacements obtained in this way allow us to analyze the dynamic response of the model due to kinematic loads originating from the earthquake and the mining shock.



a)



b)

Figure 6. Fourier spectrum for the most intensive components of vibrations

Table 4.3. Chosen results for analyzed kinematic loads

	SITKA	LGOM
1	*2*	*3*
Displacement "*x*" level at +80,01	1,43 cm	1,63 cm
Displacement "*y*" level at +80,01	3,06 cm	8,42 cm
Resultant displacement "$(x^2+y^2)^{0,5}$" level at +80,01	3,22 cm	8,70 cm
Displacement "*x*" level at +95,77	1,64 cm	1,82 cm
Displacement "*y*" level at +95,77	3,59 cm	9,74 cm
Resultant displacement "$(x^2+y^2)^{0,5}$" level at +95,77	3,77 cm	9,95 cm
Stress σ_{zz} (compression) in pillars	6,57 MPa	16,35 MPa
Stress σ_{zz} (tension) in pillars	5,97 MPa	15,84 MPa

Comparing the results shown in Table 4.3, it can be stated that the level of displacement in case of a mining shock is larger than for the analyzed earthquake. In particular, it refers to the component "y", which dominates distinctly in the case of a mining shock. Due to the fact that most of the energy of a mining shock is transferred in low-frequency band closer to the natural frequency of the tower model, this results in higher values of displacement and stress (both tensile and compressive). The answer of the model to both dynamic excitations is qualitatively very similar. Figure 7a shows the character of deformation at the maximum deflection of the tower to dynamic load described by the record in Figure 4 (SITKA earthquake). It may be noted that the model response corresponds to the first bending mode shape (see Figure 3a). This mode is accompanied by the largest vertical stress σzz. They concentrate at the place of stiffness between the underground RC rigid foundation box and the part with less rigid RC pylons. Figure 7b shows the vertical stress distribution and location of the maximum and minimum values listed in Table 4.3, col. 2. Transparently, in Figure 7 the non -deformation model is shown. Such deformation as this one - shown in Figure 7, is a threat to the stability of a high structure due to the possibility of overturning it. Figure 8a shows the distribution of displacement vector under the foundation slab, resulting from the Sitka earthquake activity, and Figure 8b shows the simultaneous distribution of deformation, taking the weight of the tower into account. Figure 8b shows that there are no tensile stresses under the slab. It means that the value of the moment related to the weight of the structure is higher than the value of the overturning moment caused by kinematic excitation.

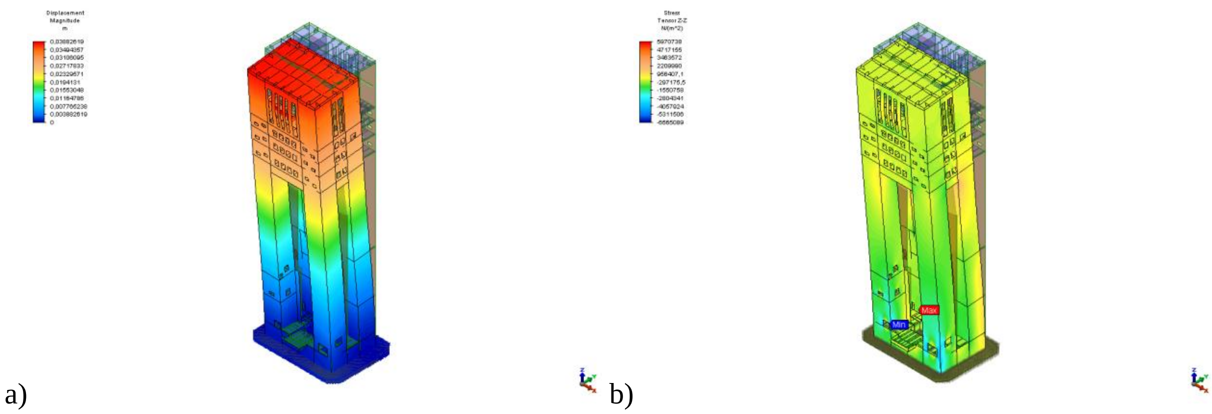


Figure 7. Deformation mode corresponding to the maximal inclination of the tower due to an earthquake

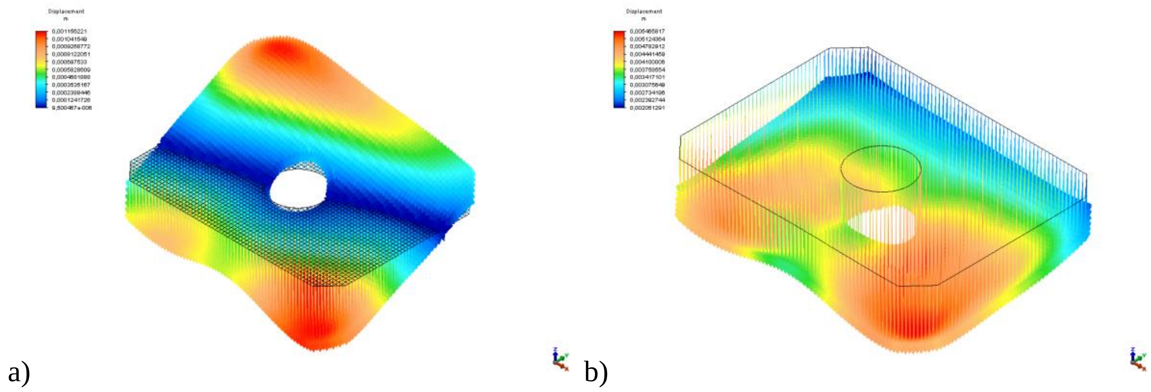


Figure 8. Displacement distribution of the foundation slab due to an earthquake: a) without dead load, b) assuming dead load

4.3. Assumed kinematic loads and dynamic analysis

The computed values of dynamic displacements, stresses and distribution of the foundation slab displacements presented in Section 4.2 were compared with the results obtained for the code wind loads. The wind load was determined on the basis of the standard [EN 1991-1-4] for the location of buildings in Poland. This load was the basic load in the design procedure, apart from the technology ones related to the use of this structure. Values of coefficients of aerodynamic resistance were defined on the basis of the work [Riera & Davenport 1998, Hania & Sakamoto 1987], mainly because of the unusual shape of the tower that consists of four pillars located at a close distance from one another. Table 4.4 presents the selected results for wind loads, which can be compared with the values obtained for the analyzed kinematic excitations. Greater values of calculated displacements and stresses are obtained for selected kinematic excitations than for wind load. In particular, the displacement and stress-induced mining shock is the greatest. In the worst case, the level of maximum displacement and stress is almost four times higher than for wind loads. In the case of wind loads, there is no danger of the tower losing its stability that protects it from falling over.

Table 4.4. Chosen calculated results for wind load

	Wind load
1	*2*
Displacement *"x"* at level +80,01	0,48 cm
Displacement *"y"* at level +80,01	2,12 cm
Displacement *"x"* at level +95,77	0,55 cm
Displacement *"y"* at level +95,77	2,45 cm
Stress σ_{zz} (compression) in pillars	3,53 MPa
Stress σ_{zz} (tensile) in pillars	5,24 MPa

5. CONCLUSIONS

The study presents the influence of one of the most intensive mining shocks and earthquakes on the skip tower. This structure is a strategic facility for the functioning of a coal mine, and its exclusion from the use in the event of an accident or damage, would generate a great loss for the mine. Dynamic analyses were performed using 3D model FEM and modal superposition method. An analysis was also conducted for wind loads. All results were compared and commented and were the base for conclusions.

If the tower were located in the seismically active area, the appropriate standards [Eurocode 8, UBC, 1997] would have to be used in the design procedure. Therefore, in the design process, additional stresses (cross – sectional forces) resulting from an earthquake would be considered. However, in Poland, at the time of that skip tower design, no standards or instructions for the design of structures in the mining shock areas existed. The primary loads for such structure were wind and technology load.

The calculation results indicate that the kinematic load of the base of the structure due to a mining shock can be a basic design load, as it is for earthquakes in the seismically active areas. In this particular case, the mining – related load caused greater displacements than an earthquake or wind. Similarly, the stress in the main structural components was higher.

In Poland there are a number of industrial buildings that belong to the mines, and were not designed for seismic impact caused by mining tremors.

The approach presented here, connected with creating a representative FEM model and performing calculations using the recorded components of vibrations, is a good method for predicting negative effects of such loads.

AKCNOWLEDGEMENT

This paper has been supported by the Ministry of Science and Higher Education (grant No L-4/95-DS/2012).

REFERENCES

Tatara, T. Pachla F (2011) *Influence of minng tremors on industrial structures*. 9th International conference on new trends in statics and dynamics of buildings, October 20-21, 2011 Bratislava.

Ciesielski, R. (1973) *Evaluation of harmfulness of dynamic influences in building*. Arkady, Warszawa (in Polish).

Haniu, H. Sakamoto, H. (1987) *Fluctuating forces acting on two rectangular prisms placed vertically in a turbulent boundary layer*. Journal of Wind Engineering and Industrial Aerodynamics, Vol 26, Issue 1, Pages 85-103.

Korczyński, J.L. Bieczeniewa, G.W. (1966) *Procznost stroitielnych materiałowpri dinamiczeskich nagrużenijach*. Striojizdat, Moskwa (in Russian).

Riera, J.D. Davenport A. G. (1998) *Wind effects on buildings and structures: proceedings of the Jubilee* Conference on wind effects on Buildings and structures, Balkema, Rotterdam.

Technical description (1974) KWK "Piast" - Shaft tower II . Office of Projects for Coal Industry branch in Cracow (in Polish).

Static calculation (1974) Shaft tower No. II, building part, Office of Projects for Coal Industry, branch in Cracow, 1974 (in Polish).
The technical drawings (1974) KWK “Piast” - The shaft tower. RC construction, Office of Projects for Coal Industry branch in Cracow, 1974 (in Polish).
PN-85/B-02170, Evaluation of the harmfulness of building vibrations due to ground motion (in Polish).
PN-80/B-03040, Machine foundations and supporting structures. Calculation and design (in Polish).
Eurocode 8, (2003) Design of structures for earthquake resistance. Draft No 6. January 2003.
PN-EN 1991-1-4, Eurocode 1- Influence on constructions, part 1-4: General influence – Wind influence.
Uniform Building Code (1997), vol. 2. California: International Conference of Building Officials.