A LONG TERM OBSERVATION OF THE TECHNICAL STATE OF BUILDING POSITIONED IN THE AREA OF COAL MINE EXPLOITATION USING LASER SCANNING VIBROMETER

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Summary

In this paper a long term observation of the technical state of building positioned in the area of coal mine exploitation is presented. The investigated structure was induced to vibrate using impact excitation. The points of excitation were located on two different pillars supporting the building. In order to extract dynamic parameters of the structure, experimental modal analysis was applied. The parameters of vibration resulting from this excitation were measured by a scanning laser vibrometer. To introduce and compare classical and operational modal analysis, piezoelectric accelerometers were additionally utilized. Moreover, to investigate and explain the changes in the state of the building a finite element model (FE) was built. The analysed building is situated on the terrain of underground coal mine exploitation so the aim of the research was not only to observe the state of the building, but also to analyse possible changes during the process of excavation and after its completion. The building was observed from 2009 to 2012. Identification of the state of building using a laser scanning vibrometer proved feasible and relevant from a practical point of view.

Keywords: modal analysis, building structures, scanning vibrometer, non-destructive testing

DŁUGOOKRESOWA OBSERWACJA STANU TECHNICZNEGO BUDYNKU POŁOŻONEGO NA TERENIE, GDZIE PROWADZONA JEST EKSPLOATACJA GÓRNICZA, Z ZASTOSOWANIEM WIBROMETRII LASEROWEJ

Streszczenie

W artykule przedstawiono wyniki badań zrealizowanych w trakcie długookresowej obserwacji stanu technicznego budynku. Badany budynek był pobudzany do drgań z wykorzystaniem wymuszenia impulsowego. Punkty wymuszenia zlokalizowane były na dwóch filarach podtrzymujących kondygnację budynku. Parametry drgań mierzone były z zastosowaniem skanującego wibrometru laserowego. Aby umożliwić porównanie wyników uzyskanych dla klasycznej i eksploatacyjnej analizy modalnej dodatkowo na ścianie budynku zamocowano piezoelektryczne przetworniki drgań. Ponadto, aby umożliwić diagnozę i lokalizację zmian stanu technicznego utworzony został model elementów skończonych badanego obiektu. Badany budynek położony jest na terenie, na którym prowadzona jest podziemna eksploatacja górnicza. W związku z tym celem badań było nie tylko diagnozowanie stanu technicznego budynku ale również monitorowanie wpływu eksploatacji górniczej na badaną konstrukcję. Budynek badany był w latach 2009 – 2012. Badania z wykorzystaniem wibrometrii laserowej uznano za przydatne do określania zmian stanu technicznego obiektu jak i praktycznie uzasadnione

Słowa kluczowe: analiza modalna, struktura budowlana, wibrometr laserowy, badania nieniszczące

1. INTRODUCTION

Non-destructive testing (NDT) of building structures should be performed on the periodic or permanent basis. Not only does it ensure the control of technical state of such structures, but also allows to maintain safety conditions for people living or working there as well as passers-by. By applying non-destructive testing methods, it is possible to observe changes in a given structure's dynamic parameters and refer them to the initial point of inspection (before the possible damage). These changes affect modal parameters of an observed object, namely natural frequencies, damping factors and mode shapes (modal vectors). The comparison between experimental and reconciled theoretical models at each stage of observation seems to be valuable and informative. NDT may be realized using both classical modal analysis, where a source of vibration is controlled by a diagnostic crew, and operational modal analysis, where a source of vibration is induced by the environment.

2. MONITORING OF BUILDING DYNAMIC PARAMETERS

The object being considered in this paper is a four-storey building shown in Fig. 1 with its middle part investigated. As there are dilatations (red arrows) between adjacent parts of the whole building it was assumed that these parts behave fairly independently.

Fig.1. The object under test, a four-storey building

The building is situated on the terrain of underground coal mine exploitation. Accordingly, the aim of the research is not only to observe the changes of technical state of the building, but also to analyse the influence of the process of excavation which was finished in March 2010. The building was investigated between 2009 and 2012. The structure was excited to vibrate by means of a modal hammer designed for large objects type 086D50, PCB, also shown in Fig. 1. The excitation was realised in single points positioned on pillars supporting the building. The reaction of the structure was then recorded using Polytec PSV-400 laser scanning vibrometer at equally distributed points positioned on the front face. To identify the parameters of modal model, the transfer functions (inertances) recorded in the course of testing were imported into a TestLab LMS computation module and modal analysis was performed [1,2,3,4,5]

3. RESULTS OF MEASUREMENTS

To evaluate modal parameter estimates properly, the analysis was performed for different frequency range selections and different time windows. The hammer impact was repeated several times for each measurement point of observations (laser vibrometer measurement) to improve the results through averaging measured FRF curves; good repeatability was observed. The results were not distorted by other sources of vibration – the building is positioned in a quiet urban district (moderate traffic on local roads). Natural frequencies of the structure vibration identified by PSV-400 between 2009 and 2012 are collected in Tables 1 to 7. A distinct shift of these values may be observed in 2011, especially in the range below 15 Hz. In situ observations revealed that there were quite visible crevices in dilatations between main parts of the whole building. They were probably caused by the stress in foundations after a slight ground collapse. In 2012 the stress remission must have taken place and frequency shift became less evident, as presented in diagram, Figs. 2 and 3.

Table 1. Natural frequencies of vibration of the structure identified by PSV-400 in 2009 for the left and right pillars

	Left pillar 2009		Right pillar 2009	
No.	Natural Frequency [Hz]	Modal Damping Coefficient [%]	Natural Frequency [Hz]	Modal Damping Coefficient [%]
1	7.57	1.01	7.52	0.85
2	13.31	3.48	13.25	2.92
3	29.14	1.91	28.89	0.55

Table 2. Natural frequencies of vibration of the structure identified by PSV-400 in 2010 for the left and right pillars

	Left pillar 2010		Right pillar 2010	
No.	Natural Frequency [Hz]	Modal Damping Coefficient [%]	Natural Frequency [Hz]	Modal Damping Coefficient [%]
1	6.96	1,.48	7.01	1.53
2	12.52	0.81	12.55	0.58
3	29.17	0.37	29.31	0.75

Table 3. Natural frequencies of vibration of the structure identified by PSV-400 in 2011 for the left and right pillars

	Left pillar 2011		Right pillar 2011	
No.	Natural Frequency [Hz]	Modal Damping Coefficient [%]	Natural Frequency [Hz]	Modal Damping Coefficient [%]
1	9.39	0.52	9.43	0.55
2	10.73	1.21	10.76	0.73
3	14.51	1.93	14.64	1.10
4	28.26	1.32	28.08	0.68

Table 4. Natural frequencies of vibration of the structure identified by PSV-400 in 2012 for the left and right pillars

	Left pillar 2012		Right pillar 2012	
No.	Natural Frequency [Hz]	Modal Damping Coefficient [%]	Natural Frequency [Hz]	Modal Damping Coefficient [%]
1	6.80	2.08	6.56	3.43
2	7.50	9.98	7.40	2.06
3	12.08	4.55	12.24	3.10

4	22.52	3.11	22.48	2.71
5	33.41	1.04	33.37	0.97
6	44.89	1.49	45.09	1.26

Table 5. Comparison of natural frequencies of the structure identified by PSV-400 in 2009, 2010, 2011 and 2012

	Right pillar 2009	Right pillar 2010	Right pillar 2011	Right pillar 2012
No.	Natural Frequency [Hz]			
1	7.52	7.01	9.43	6.56
2	-	-	10.76	7.40
3	13.31	12.55	14.64	12.24
4	28.89	29.31	28.08	33.37

Table 6. Comparison of natural frequencies of the structure identified by PSV-400 in 2009, 2010, 2011 and 2012

	Left pillar 2009	Left pillar 2010	Left pillar 2011	Left pillar 2012
No.	Natural Frequency [Hz]			
1	7.57	6.96	9.39	6.80
2	-	-	10.73	7.50
3	13.31	12.52	14.51	12.08
4	-	-	-	22.52
5	29.14	29.17	28.26	33.41
	-	-	33.17	-

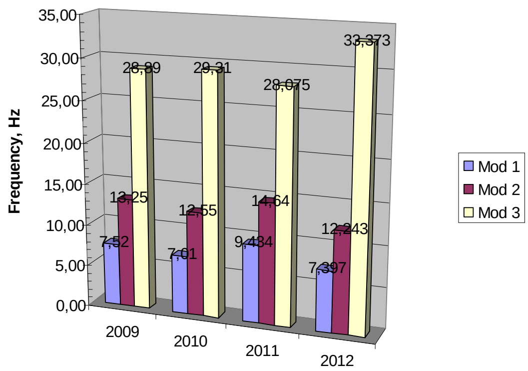

Fig. 2. Natural frequency shift for particular modes in years 2009 - 2012, right pillar

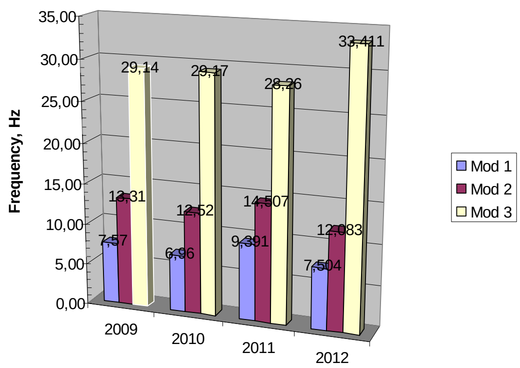

Fig. 3. Natural frequency shift for particular modes in years 2009 - 2012, left pillar

Figs. 4 and 5 show the examples of mode shapes of identified natural frequencies for impact excitation attached to the left and right pillars in 2012; the average FRF plots (waterfall) for investigated points are presented in Figs. 6 and 7, respectively.

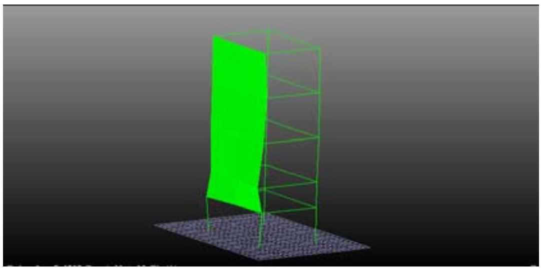

Fig. 4. The example of a mode shape of an identified natural frequency - 7,50 Hz for impact excitation attached to the left pillar in 2012

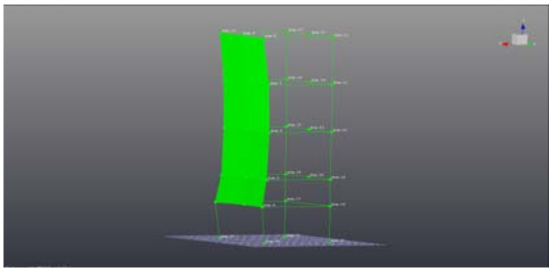

Fig. 5. The example of a mode shape of an identified natural frequency - 12,24 Hz for impact excitation attached to the left pillar in 2012

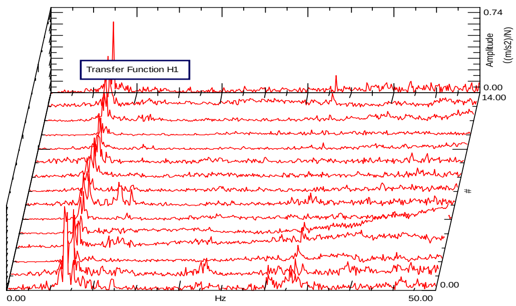

Fig. 6. FRF plots (waterfall) for investigated points 1-15, respectively for impact excitation attached to the left pillar in 2012

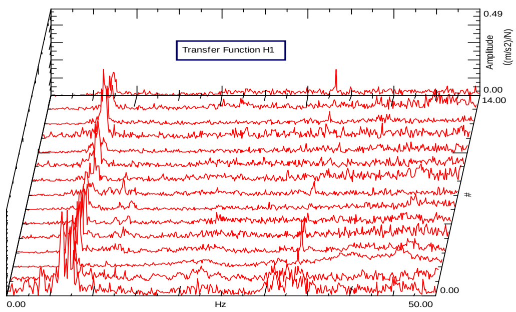

Fig. 7. FRF plots (waterfall) for investigated points 1-15, respectively for impact excitation attached to the right pillar in 2012

Table 7 contains comparison of results of classical versus operational modal analyses obtained in 2012. It must be stressed however, that excitation was induced by a modal hammer, not an environmental source of vibration, though the reference signal was taken from accelerometers placed on the right and left pillars, not a hammer head itself.

Table 7. Comparison of natural frequencies of the structure identified by PSV-400 in 2012. Classical versus operational modal analysis for excitation located on right and left pillars

No	Left pillar 2012		Right pillar 2012	
	Classical	Operation.	Classical	Operation.
	Natural Frequency [Hz]			
1	6.80	6.87	6.80	6.63
2	7.50	7.25	7.50	7.32
3	-	10.92	-	11.00
4	12.08	12.28	12.08	12.08
5	22.52	-	22.52	-
6	33.41	33.29	33.41	33.51

The average Cross Power (CP) plots (waterfall) for investigated points, measurements realized in 2012, are presented in Figs. 8 and 9

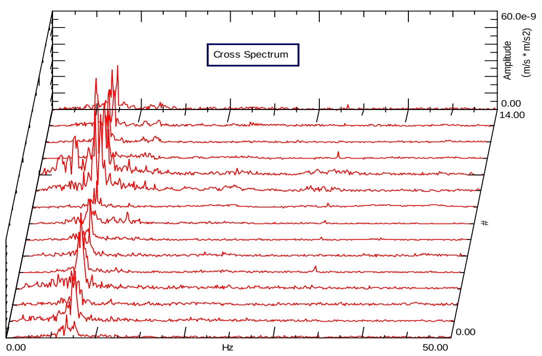

Fig. 8. CP plots (waterfall) for investigated points 1-15, respectively, operational modal analysis, impact excitation attached to the right pillar, in 2012

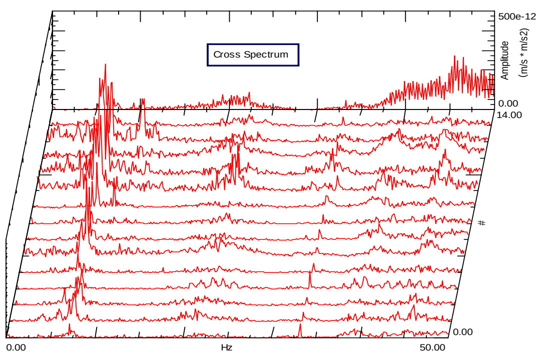

Fig. 9. CP plots (waterfall) for investigated points 1-15, respectively, operational modal analysis, impact excitation attached to the left pillar, in 2012

4. ANALYSIS OF FINITE ELEMENT MODELS

One of the crucial tasks of theoretical modal analysis is reconciliation of numerical models (i.e. finite element models) which represents an analytical illustration of the real examined object. Reconciliation is performed with support of the results of experimental modal analysis. All the modifications in a real structure may then be described by changes in a theoretical model [6].

After the reconciliation, the theoretical model may be used as a reference for further investigations. In the presented research the ANSYS Workbench environment was used to build the finite element model. The geometry of the examined structure was taken from its design project. Particular physical properties of materials such as density, Young Modulus, Poisson ratio etc. were taken from the pertinent literature. These parameters were crucial since mass and stiffness matrices were built in relation to them. Then loads and boundary conditions were introduced. Changing these parameters is one of the methods of reconciliation. The final phase involved performing structural modal analysis to obtain dynamic properties of the modelled object; calculations were also done in ANSYS. In Fig. 10 the FE model of the building structure under the test is presented. To reconcile theoretical and experimental modal models**,** optimization algorithms incorporated in ANSYS environment were utilized.

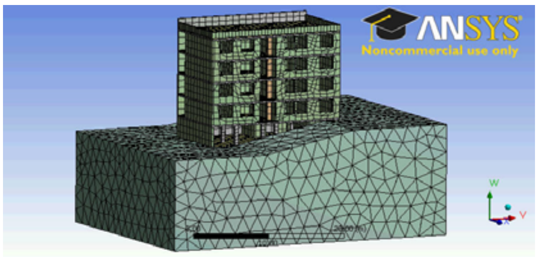

Fig. 10. FE model of the building structure under test

5. OPTIMIZATION

Goal Driven Optimization (GDO) is a set of constrained, multi-objective optimization techniques in which the "best" possible designs are obtained from a sample set given the goals you set for parameters [14]. There are three available optimization methods: **Screening**, Multi-Objective Genetic Algorithm **(MOGA)**, and Nonlinear Programming by Quadratic Lagrangian (**NLPQL)**. MOGA and NLPQL can only be used when all input parameters are continuous. The **Screening** approach is a non-iterative direct sampling method by a quasi-random number generator based on the Hammersley algorithm.

The MOGA approach is an iterative algorithm, which can optimize problems with continuous input parameters. NLPQL is a gradient based single objective optimizer which is based on quasi-Newton methods. Problems with mixed parameter types (i.e., discrete, continuous with Manufacturable Values, or scenario parameters with continuous parameters) or discrete ones cannot currently be handled by the MOGA or NLPQL techniques, and in these cases the Screening technique could only be used.

The GDO framework uses a Decision Support Process (DSP) based on satisficing criteria as applied to the parameter attributes using a weighted aggregate method. In effect, the DSP can be viewed as a postprocessing action on the Pareto fronts as generated from the results of the MOGA, NLPQL, or Screening process. The GDO process allows you to determine the effect of given output parameters with certain objectives on the input ones. For example, in a structural damage detection problem, we may want to determine which set of parameters (in terms of geometric problem dimensions and material properties) best satisfies coherence of natural frequencies and mode shapes for experimental and finite element models. In the given approach the Screening technique was utilized. In Fig. 11 sensitivities of input material parameters in the aspect of model refinement are presented.

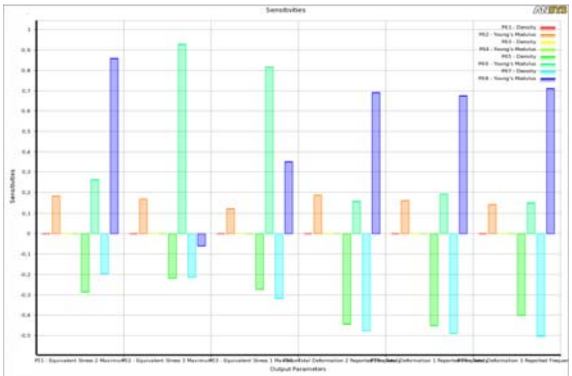

Fig. 11. Sensitivities of input material parameters in the aspect of model refinement

6. RESULTS OF RECONCILIATION OF EXPERIMENTAL AND FINITE ELEMENT MODELS

As a result of reconciliation of experimental and analytical models the set of material properties was optimized. These changes in material properties describe (mainly Young modulus and density) the increase or loosening of stiffness of relevant parts of the building structure. In Tables 8 to 11 optimized material properties - Young modulus and density in relevance to the obtained results are shown.

Table 8. Optimized material properties - Young modulus and density in relevance to results obtained in 2009

Material	Density	Young's Modulus	Natural Frequency	
			Experiment	Optimized
	[kg/m^3]	[Pa]	[Hz]	[Hz]
ground	1590.4	1.0622E+09	-	7.03
foundation	1247.4	2.1276E+10	7.57	7.99
cement	1400.6	1.9398E+10	13.31	11.69
brick	1181.0	6.8131E+09	29.14	-

Table 9. Optimized material properties - Young modulus and density in relevance to results obtained in 2010

Material	Density	Young's Modulus	Natural Frequency	
			Experiment	Optimized
	[kg/m^3]	[Pa]	[Hz]	[Hz]
ground	1590.4	1.0622E+09	6.96	7.03
foundation	1247.4	2.1276E+10	-	7.99
cement	1400.6	1.9398E+10	12.52	11.69
brick	1181.0	6.8131E+09	29.17	-

Table 10. Optimized material properties - Young modulus and density in relevance to results obtained in 2011

Material	Density	Young's Modulus	Natural Frequency	
			Experiment	Optimized
	[kg/m^3]	[Pa]	[Hz]	[Hz]
ground	1621	1.000E+09	9.43	7.17
foundation	1353	2.350E+10	10.76	10.47
cement	1300	1.837E+10	14.64	14.00
brick	1282	5.289E+09	28.08	28.43

Table 11. Optimized material properties - Young modulus and density in relevance to results obtained in 2012

Material	Density	Young's Modulus	Natural Frequency	
			Experiment	Optimized
	[kg/m^3]	[Pa]	[Hz]	[Hz]
ground	1599.50	1.038E+09	6.80	6.82
foundation	1330.76	2.407E+10	7.50	7.75
cement	1385.30	1.772E+10	12.08	11.27
brick	1224.29	6.381E+09	22.52	-

As results of optimization should be consistent for mass properties, the proposed by algorithm values were also critically inspected from that point. The maximum difference in material density for referred models in years 2009, 2010 and 2012 did not exceed 2.4% except for foundation density that differed from an average by 4.4%. Concerning Young Modulus the difference was not greater than 8.4%. It seems that in the process of optimization it was possible to adjust experimental and theoretical models for the first two or even three natural frequencies, Fig. 12.

The mode shape of the first natural frequency is generally front to back movement. The second one may be described as side to side bend with the third mode shape as torsion movement.

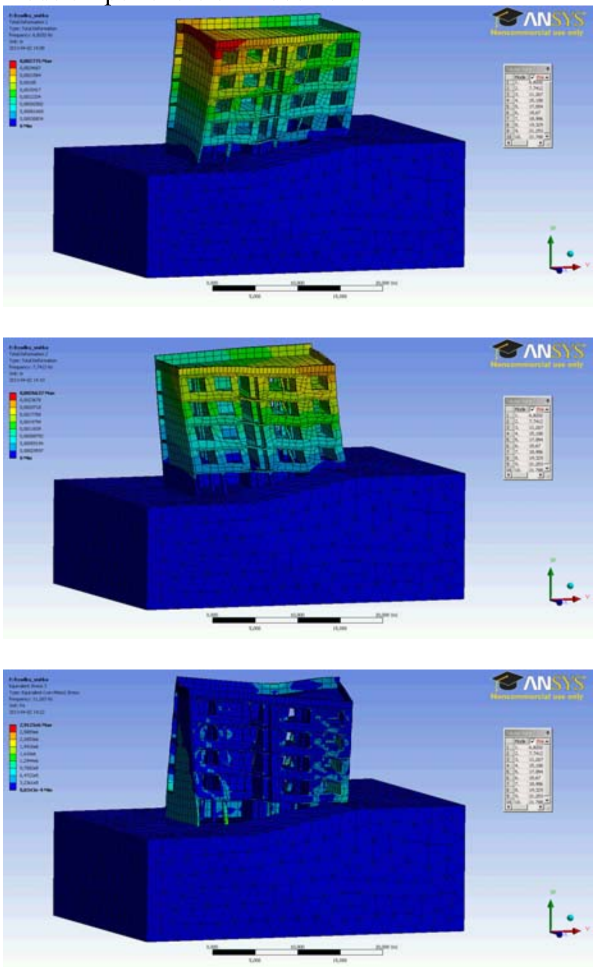

Fig. 12. FE model of the building structure under test. The first three natural frequencies for optimized model

Refinement and adjustment of FE model to the experimental one through optimization did not succeed for the results obtained in 2011 experiment. In order to fit these models additional stiffness in the foundation was assumed. Furthermore, evaluation of possible cracks in the tested structure using specific algorithm was realized. The procedure is as follows, to form a possible crack**,** the elements of the finite element model with the highest value of stress are removed from the model, which describes the crack.

The subsequent calculation of the modal parameters gives information whether the changes are adequate to those observed and is valuable at next monitoring stages.

After that process the identified natural frequencies were compared with the experimental ones, presented in Table 10. FE model with localized cracks is shown in Fig. 13.

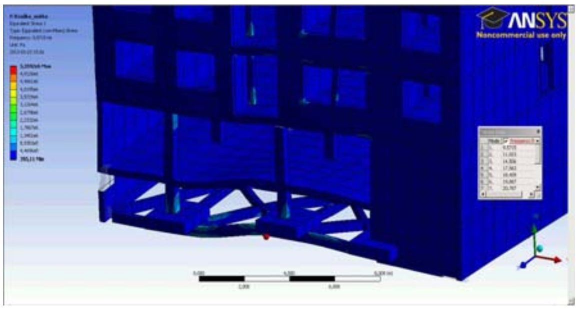

Fig. 13. FE model of the building structure, visualisation of cracks in the foundation

The mode shapes of FE model changed. Though a natural frequency of about 14 Hz was evaluated using theoretical modal analysis for each refined model, it was diagnosed experimentally only in 2011. The shape of that mode changed from vertical movement of the whole structure to a more local one around the first floor. The natural frequency of about 11 Hz - 12 Hz which was also evident at each observation stage changed its shape from a torsion-like to a vertical one and became more local too, see Fig. 14. The frequencies of about 7 Hz disappeared and were shifted to higher values around 9 Hz. There is no certainty ,of course, that those modifications of FE model are sufficient to explain real mechanical changes, but the results seem to be quite reasonable. It is also probable that in 2012 after pressure decrease in the foundation the construction turned back to its previous state.

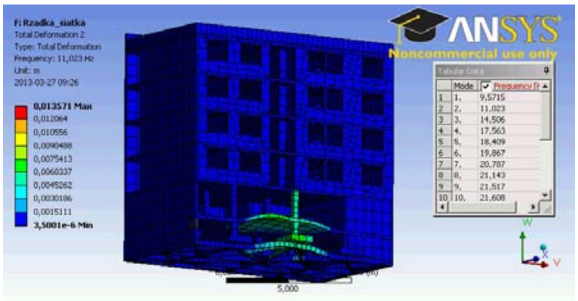

Fig. 14. FE model of the building structure, visualisation of the mod shape for frequency 11,02 Hz

7. DAMAGE DETECTION

As a general method for damage detection it is proposed to refine theoretical and experimental modal models. If necessary, the task should be performed for each identified consecutive natural frequency separately and then the stress distribution for each refined model is calculated. Consequently, the sum of these particular stresses, as shown in Fig. 15, conveys information for building engineers, enables comparison with their in situ observations and may be treated as indicative of a possible damage [10, 13].

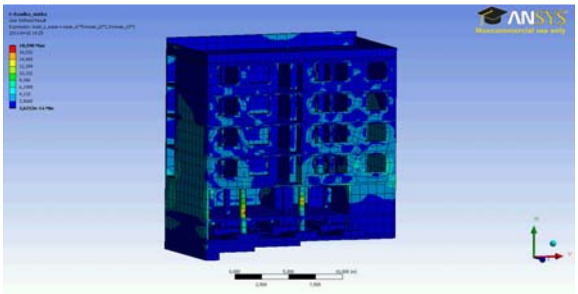

Fig. 15. Distribution of the sum of particular stresses for FE model

8. INFLUENCE OF ENVIRONMENTAL CONDITIONS

In order to assess the influence of environmental conditions on building structure mechanical properties the temperature values on a day of measurement are discussed. They are presented in Table 12. Between 2010 and 2012 the environmental temperatures did not vary markedly. In 2009 the temperature was lower, which probably caused a slight increase in stiffness of the tested structure and as a consequence a shift of the first two natural frequencies to the highest values, see Tables 5 and 6. Naturally, it cannot be assumed that it was the only reason for those changes, but partially they may be attributed to weather conditions .
The experiments were performed in spring and autumn at hours when most citizens were not present (before 3 pm), so observed changes, in author's opinion, should not be related to the different mass distribution of the analysed structure or fluctuations of mechanical parameters.

Table 12. Comparison of environmental conditions – temperature – in 2009, 2010, 2011 and 2012

Year	2009	2010	2011	2012
Date	27.11	07.07	01.04	28.06
Temperature [ºC]	10.9	20.9	18.0	21.8

9. CONCLUSIONS

The results presented above provide the basis for drawing the following conclusions:

1. A reliable identification of the state of building structures using a laser scanning vibrometer is useful from a practical point of view.
2. For damage detection it is applicable to refine theoretical and experimental modal models for each natural frequency. The distribution of stresses for each refined model is then calculated and consequently the sum of these particular stresses obtained . That proves informative for building engineers and enables comparison with their in situ observations [13].
3. The comparison of classical modal analysis versus operational one [11,12] gives very coherent results.
4. Evaluation of possible cracks in the tested structure using specific algorithm can be realized. To form a possible crack, the elements of the finite element model with the highest value of stress are removed from the model, which describes the crack. The subsequent calculation of the modal parameters can give information on possible changes of the modal model properties and may be valuable at next monitoring stages.
5. In a structural damage detection problem, we may want to determine which set of parameters (in terms of geometric problem dimensions and material properties) best satisfies coherence of natural frequencies and mode shapes for experimental and finite element models. It is relevant to utilize optimization methods, the Screening Algorithm for example.

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