TECHNICAL TRANSACTIONS | **CZASOPISMO TECHNICZNE**
CIVIL ENGINEERING | BUDOWNICTWO
3-B/2016

DOI: 10.4467/2353737XCT.16.221.5970

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NUMERICAL EVALUATION OF THE INFLUENCE OF THE MODERNISATION OF SELECTED APARTMENT BUILDINGS SUBJECTED TO MINING TREMORS ON THEIR DYNAMIC RESPONSE

NUMERYCZNA OCENA WPŁYWU MODERNIZACJI WYBRANYCH BUDYNKÓW MIESZKALNYCH PODDANYCH WSTRZĄSOM GÓRNICZYM NA ICH ODPOWIEDŹ DYNAMICZNĄ

A b s t r a c t

This paper presents the results of numerical analysis of the dynamic response of 12 storey precast concrete apartment buildings of prefabricated system WWP, located in a seismically active mining region in Poland – Legnica-Glogow Copperfield (LGC). The study involved typical buildings and structures after modernisation. The results allowed to assess the influence of the type of structural reinforcements of the buildings on their dynamic response.

Keywords: dynamic response, prefabricated buildings, mining tremors

S t r e s z c z e n i e

W pracy przedstawiono wyniki obliczeń numerycznych odpowiedzi dynamicznej dwunasto-kondygnacyjnych budynków wielkopłytowych systemu Wrocławska Wielka Płyta (WWP) zlokalizowanych na terenie LGOM, poddanych wstrząsom pochodzenia górniczego. Badania dotyczyły budynków typowych oraz po modernizacji. Uzyskane wyniki pozwoliły na ocenę wpływu rodzaju wzmocnienia konstrukcji budynku na jego odpowiedź dynamiczną.

Słowa kluczowe: odpowiedź dynamiczna, budynki prefabrykowane, wstrząsy górnicze

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1. Introduction

A lot of prefabricated buildings are located in mining areas and they are subjected to paraseismic excitations induced by mining tremors, which in some regions can be comparable to low intensity earthquakes [7, 9, 11, 14]. For instance, in the most seismically active mining region in Poland – Legnica-Glogow Copperfield (LGC), the tremors' energy even reaches 1E10 J.

Most of the structures have not been designed to carry this kind of load. So modernisation and reinforcement of the buildings becomes necessary to assure safe operation [3, 5].

Numerical analysis of the influence of structural modifications (reinforcement) on the natural vibration frequencies of selected actual apartment high buildings was presented in [6]. In this paper, dynamic responses of buildings before and after modification were calculated and compared. Typical accelerograms registered in LGC were applied as kinematic excitation.

2. Analysed buildings and their numerical models

In the paper, four (A0, A1, B0, B1) 12 storey precast concrete apartment buildings, constructed according to one of the Polish prefabricated WWP systems [3, 4], were analysed.

Buildings A0 and B0 are typical for the WWP system. A1 and B1 were reinforced because of mine-induced kinematic excitations [2, 12]. All of them consist of a solid reinforced concrete basement (wall thickness – 30 cm) and precast panel walls (thickness – 14 cm). Each of the buildings is divided into single-stair and double-stair segments. All of them are located in a seismically active mining region in Poland – Legnica-Glogow Copperfield (LGC). Examples of the analysed buildings are presented in Figure 1 [6].

Buildings A0 and A1 consist of three identical sections "XI-9". A1 was strengthened with the inner and exterior structural reinforcements. New monolithic concrete spans along

a) b)

Fig. 1. Analysed structures: a) building "A0" – typical for the WWP system; b) building "A1" with inner and exterior structural reinforcements [6]

both the gable wall as well as the new monolithic load-bearing longitudinal (“*y*” direction) walls and reinforcements of existing load-bearing transverse (“*x*” direction) walls were applied. Floor plans of analysed structures (A0, A1) are presented in Figure 2 [2, 6, 12].

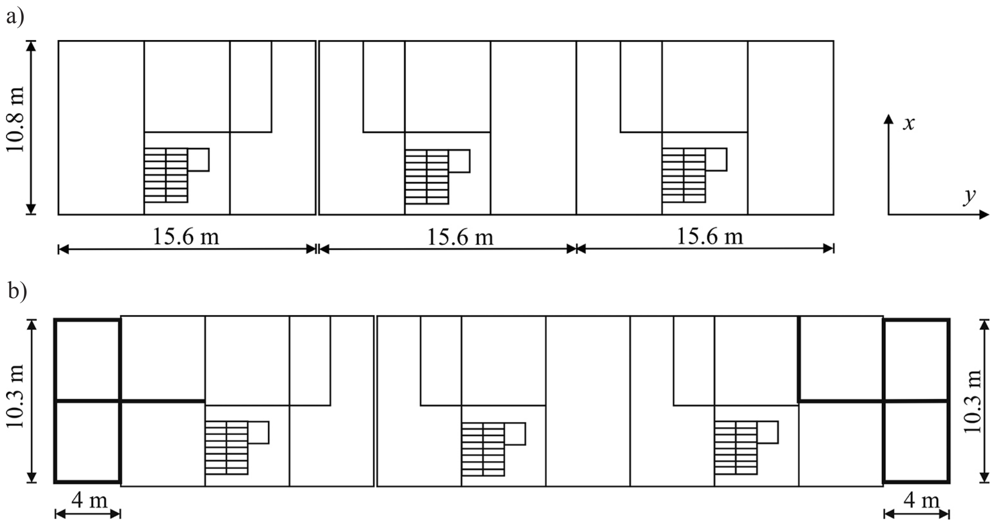

Fig. 2. Floor plans of analysed structures: a) building “A0”; b) building “A1” with reinforcement as lines marked in bold [6]

Buildings B0 and B1 consist of two different kinds of sections: “XI-9” and “XI-6”. In B1, only inner structural reinforcements (new monolithic load-bearing longitudinal walls and reinforcements of some of existing load-bearing walls) were applied. The floor plan of building “B1” with reinforcement as lines marked in bold is presented in Figure 3 [2, 6, 12].

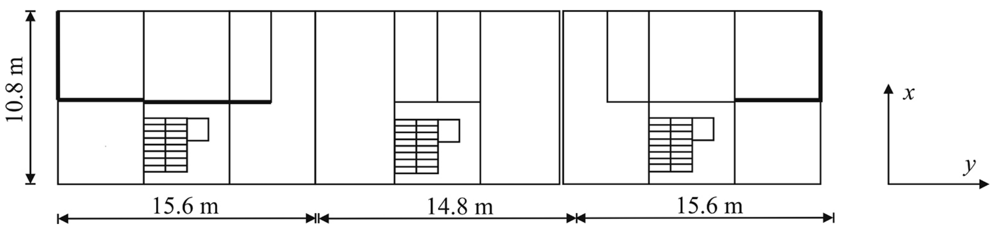

Fig. 3. Floor plan of building “B1” with reinforcement as lines marked in bold [6]

Numerical models of the analysed structures were built in finite element software Ansys [13]. All structural members were modelled using 4-node elastic shell elements with 6 degrees of freedom. The influence of panel joints by Young’s modulus reduction of load bearing elements was applied. Soil flexibility was taken into consideration by the application of spring elements with stiffness calculated for layered subsoil using the Savinov model

[8]. A dynamic soil parameter (Cz) equal to 55 MPa was considered. Finite element models of the analysed structures are presented in figure 4 and the applied material parameters of structural elements are shown in Table 1[6].

a)

b)

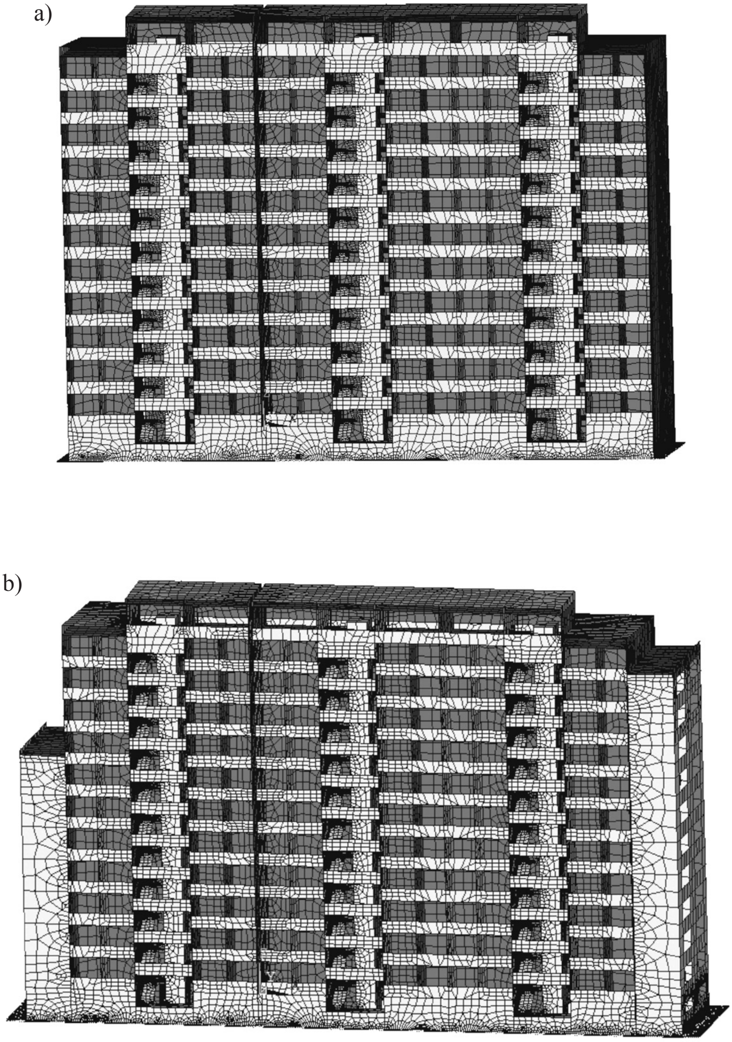

Fig. 4. Finite element models of the analysed structure: a) building "A0"; b) building "A1" [6]

Structural and material assumptions of finite element models of A0 and A1 buildings were verified by comparing the calculated natural frequencies of horizontal vibration with experimental ones obtained for actual structures [10]. Comparisons of the computed and experimental values are presented in Table 2 [6].

Table 1

Material parameters of structural elements [6]

Structure element	E [GPa]	ν	ρ [kg/m^3]
Foundation	23.1	0.2	2500
Basement	23.1	0.2	2500
Prefabricated elements	20	0.2	2500
Reinforced elements	23.1	0.2	2500
Cavity brick wall	0.72	0.25	1400

Table 2

Comparison of computed and experimental values of natural frequencies of building horizontal vibrations before and after modification [6]

Building	Natural frequencies of horizontal building vibrations [Hz]		
		computed	experimental [7]
A0	f_{1x}	1.59	1.60–1.63
	f_{1y}	1.73	1.71–1.76
A1	f_{1x}	1.79	1.64–1.71
	f_{1y}	2.47	2.17–2.28

3. Excitations

Two typical measured records of accelerations of foundation vibrations arising from mining shock in LGC in horizontal direction as a kinematic excitation were taken into consideration. Both of the mining tremors are high-energy, representative for the analysed area and could be dangerous for surface structures. However, they are different in the predominant frequencies band (Fig. 5b, 6b).

The acceleration profiles are presented in Table 3. The time history of component accelerations of vibrations in transverse (x) and longitudinal (y) direction as well as signal of fast Fourier transformation (FFT) is shown in Figure 5 and 6.

Table 3

Parameters of kinematic excitation

Excitation	Energy [J]	Epicentral distance [m]	Seismological coordinates [m]
Acceleration no. 1 (ACC1)	1.6E8	2684	$X = 29204$, $Y = 7846$
Acceleration no. 2 (ACC2)	1.3E8	650	$X = 30800$, $Y = 6392$

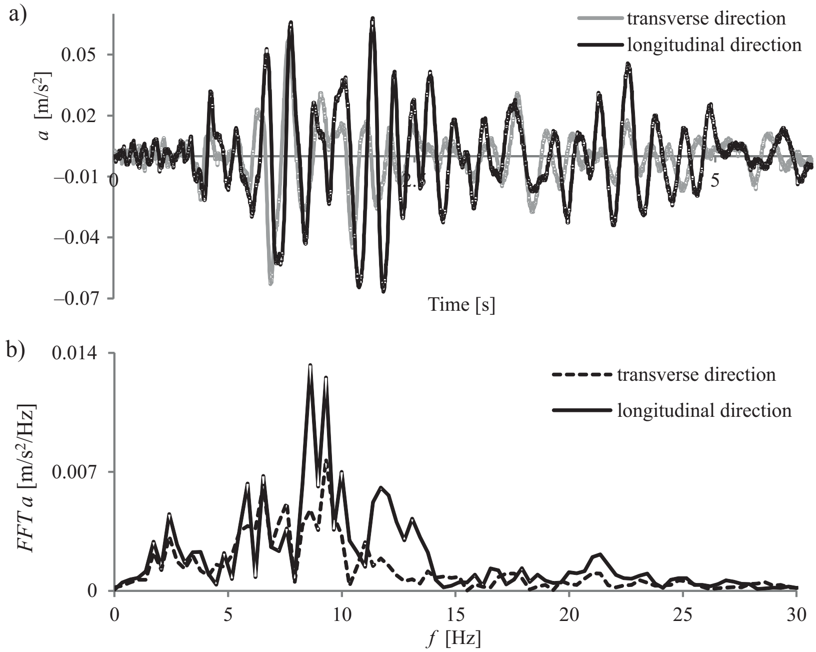

Fig. 5. Acceleration no. 1 of foundation vibrations: a) acceleration record, b) *FFT*

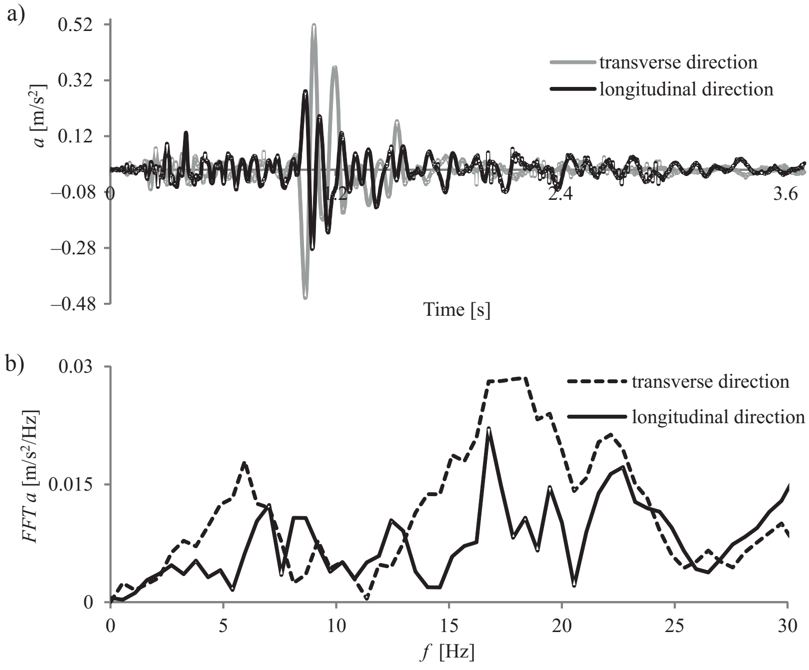

Fig. 6. Acceleration no. 2 of foundation vibrations: a) acceleration record, b) *FFT*

4. Results

Calculations of the dynamic response were carried out in Ansys software using transient analysis. The Newmark algorithm [13] with step 0.002s as well as the Rayleigh damping model [13] were applied. A damping ration equal 2% was taken into consideration [1]. As the kinematic excitation, two simultaneous mutually orthogonal horizontal components of the analysed accelerations of foundations were applied. In the paper, the influence of vertical component of acceleration has been omitted because the vertical inertia forces in typical buildings in the LGC region originating from mining rockbursts are smaller than the weight of these structures. This approach is typical in the case of vibration analysis from mining tremors [11].

The time range of all the calculated dynamic responses corresponds to the time range of the applied excitations.

In Figures 7 and 8, the measured and computed dynamic response of building A1 was compared. Experimental vibrations were registered on the 11th floor.

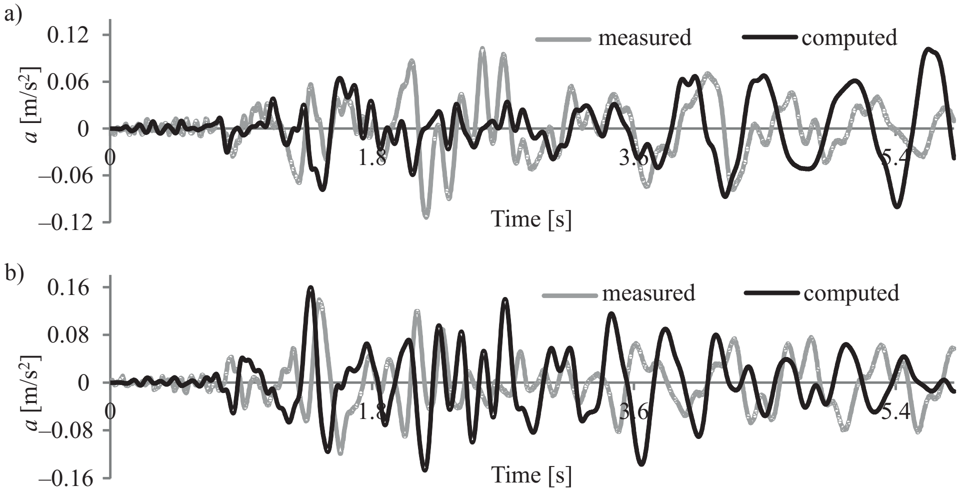

Fig. 7. Comparison of the measured and computed dynamic response of building A1 for excitation ACC1: a) transverse direction, b) longitudinal direction

A comparison of the computed dynamic response of buildings A0 (typical for system WWP) and A1 (reinforced) for excitation ACC1 and ACC2 is presented in figures 9 and 10.

Acceleration data are loaded in point of accelerometer position located on the 11th floor.

For building A1 with inner and exterior structural reinforcements (large structure modification), the difference between the dynamic responses of a typical and modified object is significant. However, the level of acceleration amplitudes' reduction strongly depends on the direction of vibrations.

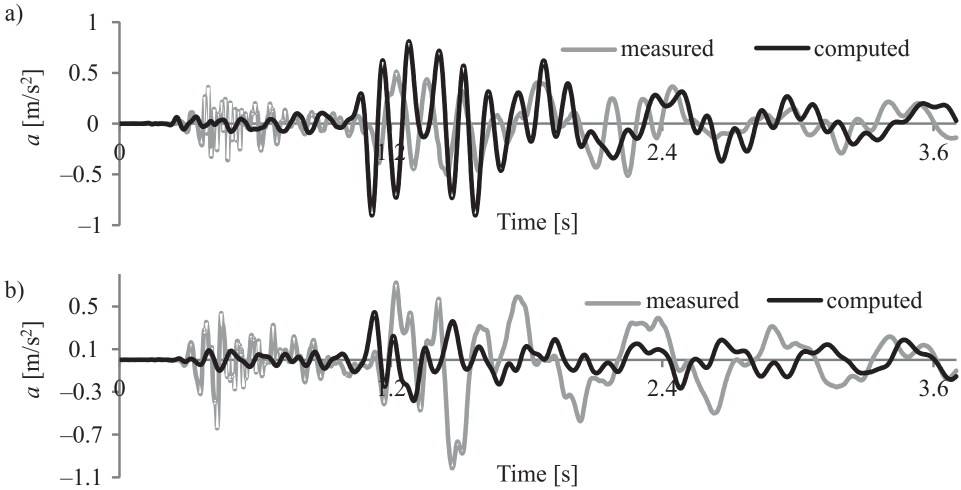

Fig. 8. Comparison of the measured and computed dynamic response of building A1 for excitation ACC2: a) transverse direction, b) longitudinal direction

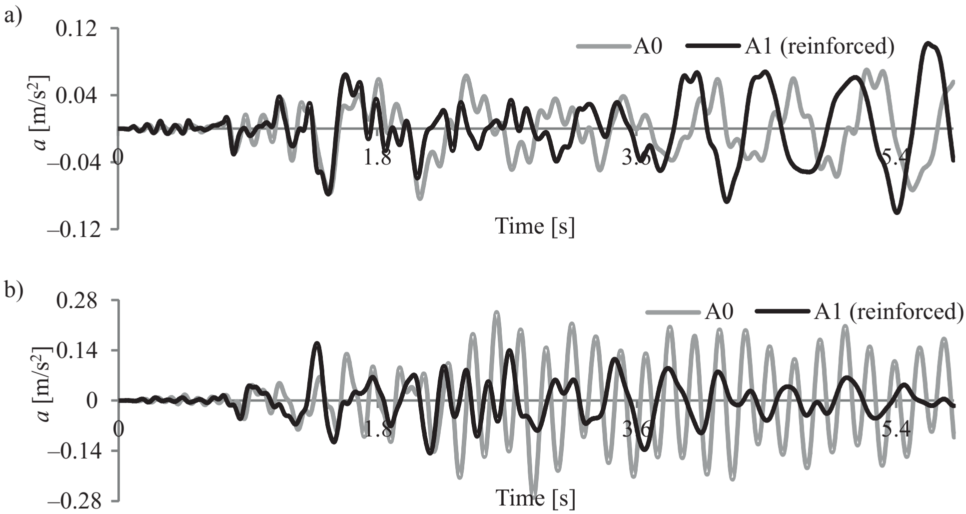

Fig. 9. Comparison of the computed dynamic response of building A0 and A1 for excitation ACC1: a) transverse direction, b) longitudinal direction

In case of the ACC1 excitation, a significant reduction of the amplitude (45% for maximal values of acceleration) is visible for the longitudinal direction of vibration (the main direction of the applied structural reinforcement). For the ACC2 excitation, greater reduction of acceleration amplitudes (28% for maximal values of acceleration) is connected to vibration along the transverse direction.

A comparison of the computed dynamic response of buildings B0 (typical for system WWP) and B1 (reinforced) for excitation ACC1 and ACC2 is presented in Figures 11 and 12.

In case of B1, where only the inner structural reinforcements were applied (small modification), changes of the dynamic response are not as visible as in A1, in spite of the increase of natural frequencies of horizontal vibration between B0 and B1 shown in Table 2.

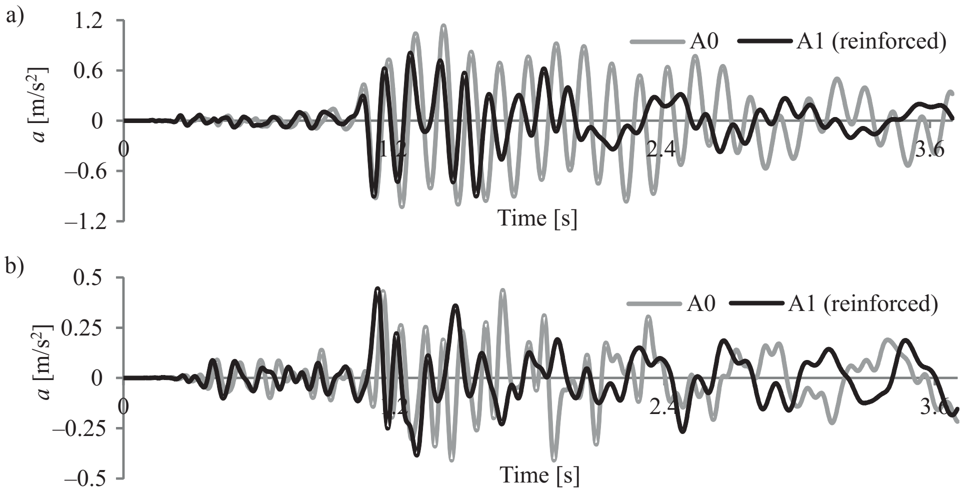

Fig. 10. Comparison of the computed dynamic response of building A0 and A1 for excitation ACC2: a) transverse direction, b) longitudinal direction

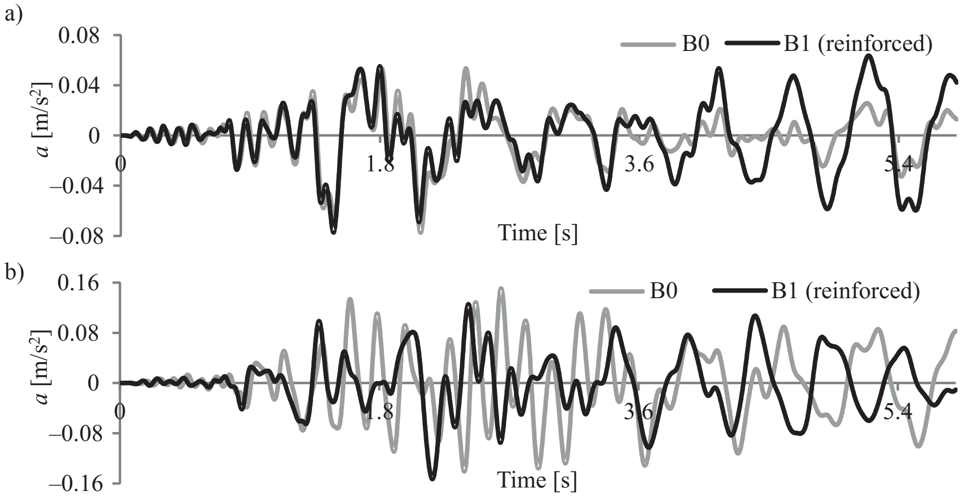

Fig. 11. Comparison of the computed dynamic response of building B0 and B1 for excitation ACC1: a) transverse direction, b) longitudinal direction

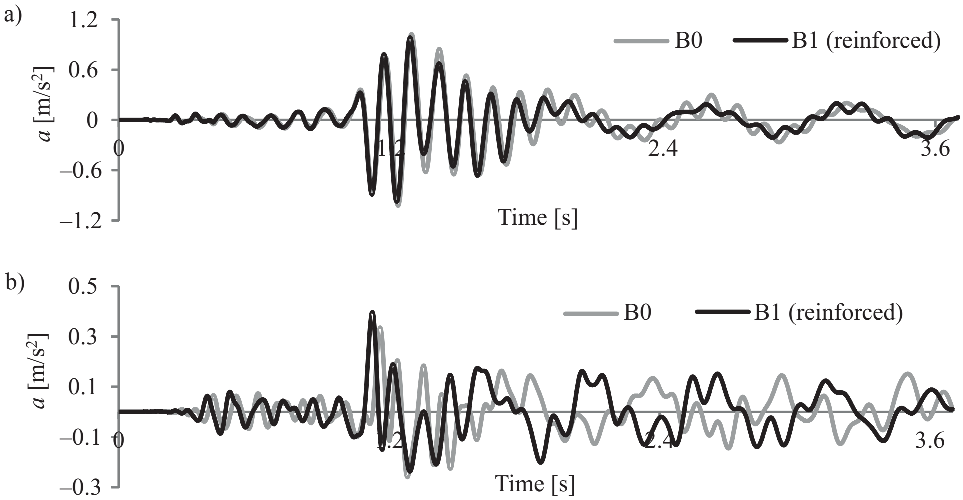

Fig. 12. Comparison of the computed dynamic response of building B0 and B1 for excitation ACC2: a) transverse direction, b) longitudinal direction

5. Conclusions

Calculations of the dynamic response of the analysed buildings show that in case of large modification (inner and outer reinforcement) significant change in the acceleration record (amplitude reduction) is visible.

For a small modification (only inner reinforcement), no amplitude reduction is observed in spite of the increase of natural frequencies of horizontal vibrations.

Despite the good coincidence between measured and computed natural frequencies of vibrations, differences in the dynamic responses are visible. In some cases, they concern both the amplitude as well as phase shift.

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