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## SELECTED PROBLEMS RELATING TO STRUCTURAL CALCULATIONS OF BUILDINGS LOCATED IN MINING AREAS IN TERMS OF PREVENTIVE PROTECTION

### WYBRANE PROBLEMY PROJEKTOWANIA KONSTRUKCJI BUDYNKÓW NA TERENACH GÓRNICZYCH W ASPEKCIE ZABEZPIECZEŃ PROFILAKTYCZNYCH

#### Abstract

This paper discusses the effect of the cross-sectional dimensions of the main structural members of a frame building on the internal forces generated in it by mining-induced tremors and choice of a code combination of actions on the dimensioning of a building structure. A numerical analysis of a reinforced concrete building was carried out for different cross-sectional dimensions of its loadbearing system subjected to mining-induced seismicity occurring in the Legnica-Głogow Copper District (LGOM) area. Additionally, a simplified cost analysis for a selected column was performed.

*Keywords: mining damage, numerical analysis, dynamic loads*

#### Streszczenie

W artykule przedstawiono wpływ wielkości przekrojów układu nośnego budynku szkieletowego na siły wewnętrzne wywołane wstrząsem górniczym oraz wpływ doboru kombinacji normowej obciążeń na wymiarowanie konstrukcji. Przedstawiono analizę numeryczną żelbetowego budynku o zróżnicowanych wymiarach przekroju układu nośnego, który został poddany oddziaływaniom parasejsmicznym występującym na obszarze LGOM. Dodatkowo przeprowadzono uproszczoną analizę kosztową analizowanych wariantów obliczeń dla wybranego słupa.

*Słowa kluczowe: szkody górnicze, analiza numeryczna, obciążenia dynamiczne*

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## 1. Introduction

Protection of a building against mining damages is a very important and challenging problem for many areas in Poland [3]. Since one of the mining damage aspects are mining-induced tremors, it is crucial to know how different factors influence the costs of building protection.

The additional internal forces generated in a building structure by a mining-induced tremor are the result of several factors, such as: the loadbearing system's geometry and stiffness, the distribution of masses in the structure and the peculiarities of the mining-induced tremor [1, 2, 14]. Since the dynamic characteristics of the tremor are beyond the building designer's control, one primary task is to design the loadbearing system so as to minimise the costs involved in the additional protection of the building against seismically-induced loads. Besides the loadbearing system stiffening geometry, also the ratios of the stiffnesses of the particular structural members need to be determined. The analysis of a reinforced concrete building structure carried out in this paper shows that increasing structural stiffness by increasing the cross sections of the building's frame loadbearing system members does not always lead to a reduction in the overall costs of the construction project.

Also, the effect of a selected code combination of ultimate limit state loads on building structure dimensioning – a still unresolved problem in the technical literature – is analysed in this paper.

## 2. Computational model

A schematic of the computational model is shown in Figs. 1–2. The grid lines mark off 9 fields designated with the letters from *a* to *i*, respectively. In order to eliminate any effects due to computational system stiffness and mass distribution irregularity, the structure has the form of a simple regular solid. The building's loadbearing system consists of eight reinforced concrete columns braced together with reinforced concrete beams (described further as respectively "Columns 1" and "Beams"), located on the perimeter of inner field *e* and twelve outer reinforced concrete ("Columns 2") in the exterior corners of the other fields. The floors of the particular storeys are monolithic reinforced concrete slabs. The structure is settled on medium dense sand deposits, which "C" ground type according to code [12]. The columns on the perimeter of field *e* are fixed to 6×6 m foundation slab in the model they are elastically fixed in the subsoil, whose elasticity modulus is  $k_z = 500$  MN/m. Others columns are fixed 2.5×2.5 m foundations and the elastic modulus in model  $k_z = 225$  MN/m was assumed. The skeleton bracing the building against the action of horizontal forces is situated within axes 2–4/B–D.

Two structural models, in which the stiffness of the system was changed by modifying the stiffness of its individual members, were analysed. The geometric dimensions of the members in Model 1 and in reference Model 2, where the stiffness of selected linear elements was increased, are presented in Table 1. In Model 2, the dimensions of the geometric cross sections of the members belonging to the groups: Columns 1 and Beams were increased mainly to make the structure more rigid and to minimise second order effects.

For the numerical analysis, the bar structure (columns and beams) and the surface structure (floors) were divided into finite elements. The computational models were subjected to analysis in the SAP2000 v. 17.2 program.

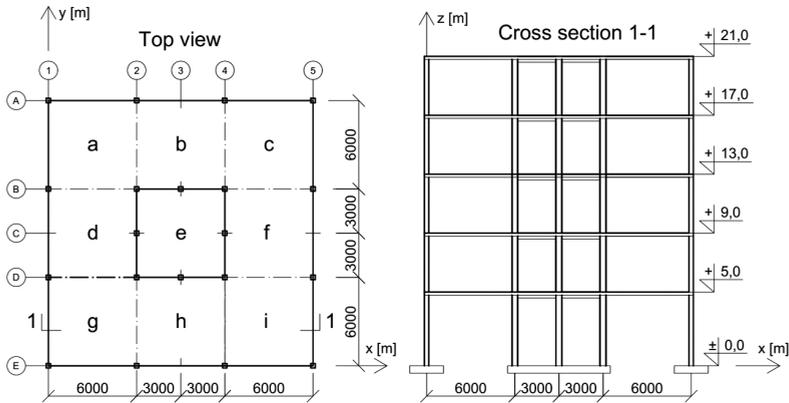


Fig. 1. Modular axes and cross section of basic model

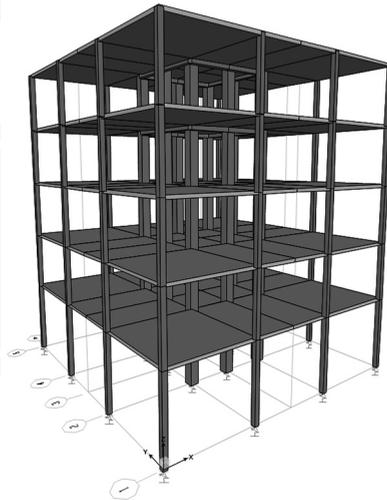


Fig. 2. 3D view of analysed basic model

Table 1

Dimensions of structural members in Model 1 and Model 2

	Column 1	Column 2	Beam	Slab
Model 1	40×40 cm	30×30 cm	20×40 cm	20 cm
Model 2	60×60 cm	30×30 cm	30×60 cm	20 cm

Exemplary computations of the internal forces changing along the height of the structure, the dimensioning of the members and a cost analysis were carried out for the corner columns of field  $e$ .

Loads' take down and dimensioning were performed in accordance with the procedures described in the current Eurocode [7–12]. The following assumptions concerning the loads were made:

- The snow load as for zone 1, with a characteristic value of  $0.56 \text{ kN/m}^2$ .
- The wind load on the walls as for wind zone 1, with a total characteristic value of  $0.73 \text{ kN/m}^2$ .
- The characteristic operational loading of the floors amounting to  $4.2 \text{ kN/m}^2$ .
- The characteristic operational loading of the roof slab amounting to  $0.4 \text{ kN/m}^2$ .
- The additional dead weight of the floors, including that of the finishing layers, amounting to  $1.75 \text{ kN/m}^2$ .
- The additional dead weight of the roof slab, including that of the finishing layers, amounting to  $1.78 \text{ kN/m}^2$ .

The following additional assumptions necessary for the computations were made:

- The response spectrum method defined for the LGOM (Legnica-Glogow Copper District) area by Zembaty [13] (terrain C with acceleration  $a = 0.6 \text{ m/s}^2$ ) was to be used.
- Grade C30/37 concrete used for all the structural members.
- The steel with yield point  $f_{yk} = 500 \text{ MPa}$ .
- The XC3 exposure class of the columns.
- In dimensioning, the second order effects were to be taken into account using the nominal curvature method.
- For the dynamic calculations, the stiffness of the members was assumed to be equal to half of the nominal stiffness, in accordance with pt. 4.3.1(6) of the EC8 code [12].

An important element of any structural analysis is the selection of a proper code combination of loads to ensure that the minimal and maximal internal forces will be obtained. The technical literature does not explicitly specify which combination of loads should be used when a seismically-induced tremor needs to be taken into account.

Formulas (1–6) define the above problem and the different load combination variants:

$$F_d = \sum_1^m \gamma_{fi} G_{ki} + 0.8 \sum_1^n \gamma_{fi} Q_{ki} + F_a \quad (1)$$

$$E_d = E \{ G_{k,j}; \Psi_{2,i} Q_{k,i}; A_{Ed} \} j \geq 1, i \geq 1 \quad (2)$$

$$E_d = E \{ G_{k,j}; (\Psi_{1,1} \text{ or } \Psi_{2,1}) Q_{k,1}; \Psi_{2,i} Q_{k,i}; A_d \} j \geq 1, i \geq 1 \quad (3)$$

$$E_d = E \{ \gamma_{G,j} G_{k,j}; \gamma_g \Psi_{0,1} Q_{G,k}; \gamma_{Q,i} \Psi_{0,i} Q_{k,i}; A_w \} j \geq 1, i \geq 1 \quad (4)$$

$$E_d = E \{ \gamma_{G,j} G_{k,j}; \gamma_{Q,i} \Psi_{0,i} Q_{k,i}; (A_w \text{ or } A_g) \} j \geq 1, i \geq 1 \quad (5)$$

$$E_d = E \{ \gamma_{G,j} G_{k,j}; \gamma_g Q_{G,k}; \gamma_{Q,i} \Psi_{0,i} Q_{k,i} \} j \geq 1, i \geq 1 \quad (6)$$

where:

- $F_d, E_d$  – design action effect,  
 $\gamma_{fi}$  – safety factor according to [6],

- $G_k$  – characteristic permanent action according to [6] (eq. (1)) or [7] (eq. (2–6)),
- $Q_k$  – characteristic variable action according to [6] (eq. (1)) or [7] (eq. (2–6)),
- $F_a$  – accidental action according to [6],
- $\Psi_2$  – factor for quasi-permanent value of a variable action according to [7],
- $A_{Ed}$  – design value of seismic action according to [7],
- $\Psi_1$  – factor for frequent value of a variable action according to [7],
- $A_d$  – design value of an accidental action,
- $\gamma_G$  – partial factor for permanent actions according to [7],
- $\gamma_g$  – partial factor for continuous deformations according to [4, 13],
- $\Psi_0$  – factor for combination value of variable action according to [4, 13],
- $\gamma_Q$  – partial factor for variable actions according to [7],
- $A_w$  – design value of an accidental action from mining tremors according to [4, 13],
- $A_g$  – design value of an accidental action from non-continuous mining deformations according to [4, 13].

Problems relating to the adoption of a combination of loads are discussed in, e.g., [4, 5, 13]. According to the former Polish standard [6], formula (1) should be used (as for accidental actions). Code EC0 [7] distinguishes a seismic combination according to formula (2) and an extraordinary combination according to formula (3). Variable loads in the new code are treated as characteristic loads (without the safety factor). In [4, 13] a synthesis of the two codes, in the form modified consistently with formulas (4) and (5) was proposed. Factors  $\Psi_{0,1}$  and  $\Psi_{0,r}$  are equal to 0.8. Different guidelines for defining load combinations can be found in [14], where it is recommended to use relations consistent with formula (3), as for the sustained loads. A completely different, controversial proposal would be to treat the seismic load not as an accidental load, which (acc. to standard [7]) “is unlikely to occur on a given structure during the design working life”, but as a variable load, consistently with formula (6).

One should also note that as the safety factors change, so do the shares of the masses having a bearing on the vibration parameters, which results in a change in the natural vibration period and in a change in the values of the inertial forces arising from the accelerations of the system masses.

The internal forces according to combination (3) (Model 1a) and the ones according to combination (5) (Model 1b) are compared in this paper.

### 3. Comparative analysis

The natural vibration periods for the analysed models are presented in Table 2. Five presented modes are needed by [12] to achieve 90% of the total mass of the structure by the sum of the effective modal masses. Mode shapes of vibrations for “Model 1a” are presented on (Fig. 3) for other models they are similar. Since calculations showed that the first mode of vibration for the two models was torsional, the second mode of vibration was adopted as the basic one and marked on the response spectrum diagram (Fig. 4).

Table 2

Natural vibration period of computational models [s]

Mode of vibration	Model 1a	Model 1b	Model 2
1	2.801	3.548	1.964
2	2.563	3.247	1.520
3	2.547	3.227	1.502
4	0.883	1.116	0.627
5	0.806	1.019	0.463

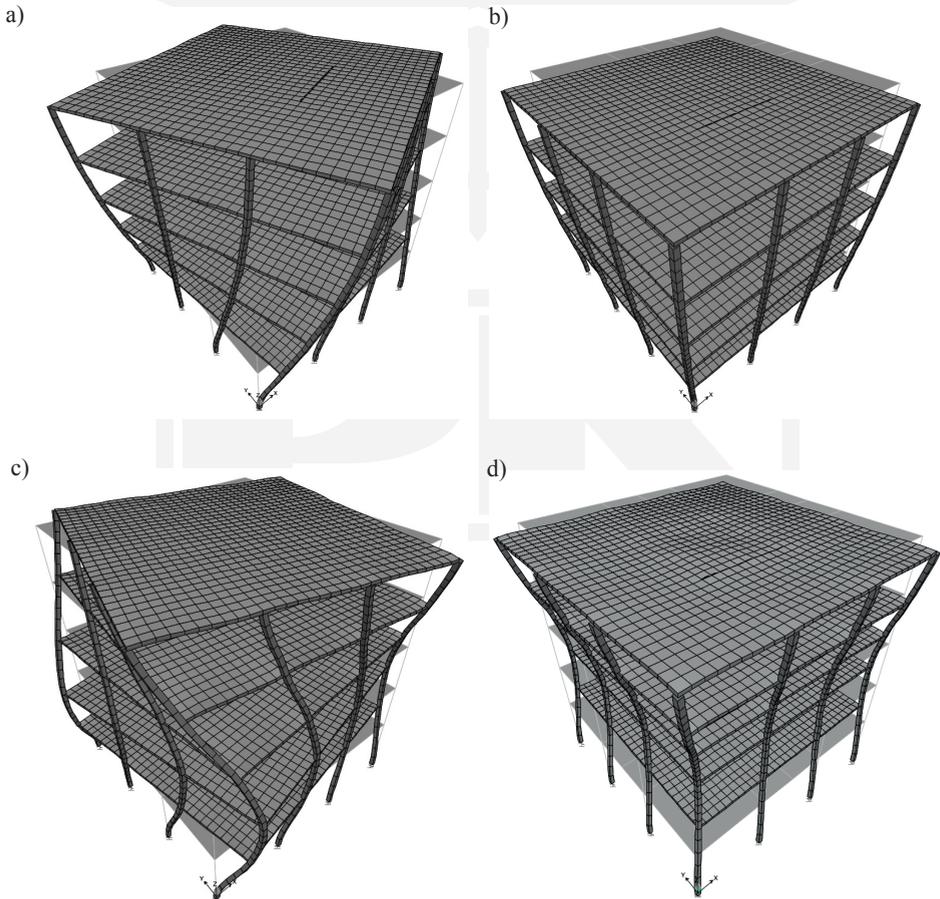


Fig. 3. Mode shapes of vibrations for “Model 1a”: a) 1<sup>st</sup> mode, b) 2<sup>nd</sup> mode, 3<sup>rd</sup> mode similar, c) 4<sup>th</sup> mode, d) 5<sup>th</sup> mode

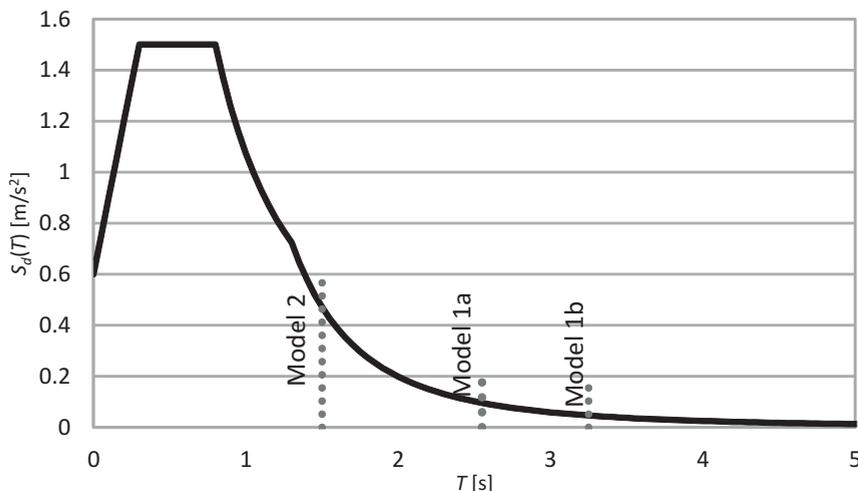


Fig. 4. Acceleration response spectrum acc. to Zembaty [9] for the LGOM area. Second form of natural period was marked with red lines

The successive forms of vibration can be taken into account in structural calculations in several ways [2]. The main two ways are: the SRSS combination (7) and the CQC combination (8). The SRSS combination can be used when the vibration periods of the particular forms do not differ by more than 10%. In the CQC combination, the interdependence between the vibrations forms is defined by a correlation coefficient, which depends on the structure's damping and on the ratio between the particular vibration frequencies. Considering that the differences between the vibration periods of the analysed models are small and that the CQC combination can be easily implemented in modern computer programs for calculating building structures in seismic areas, this combination was used in the analysis presented below.

$$r_o = \sqrt{\sum_{i=1}^N r_{io}^2} \quad (7)$$

$$r_o = \sqrt{\sum_{i=1}^N \sum_{n=1}^N \rho_{in} r_{io} r_{no}} \quad (8)$$

where:

- $r_o$  – the total effect of the impacts,
- $\rho_{in}$  – the coefficient of the correlation between the vibration forms,
- $r_{io}$  – the effect of the action of the  $i$ -th form of vibration,
- $r_{no}$  – the effect of the action of the  $n$ -th form of vibration.

The envelopes of the combinations of internal forces in the column situated in node 4D are shown (along the column height) in the diagrams in Figs. 5–7. In order to better interpret the results, the wind load along the 0Y direction was neglected. The combinations, which do not take into account mining-induced tremors, are referred to as “static 1a” and “static 2” for

model 1 and model 2 respectively, while the combinations, which take into account dynamic impacts, are designated as respectively “dynamic 1a”, “dynamic 1b” and “dynamic 2”.

The required bar area along the column height in node 4D is shown in Fig. 8. Symmetric reinforcement in both directions  $x$  and  $y$  of the column cross-sectional area was assumed. The computational criterion was to minimise the amount of reinforcement by, e.g., bundling (as far as possible) rebars in the column’s corners and using the commercially available rebar cross sections. The computed amount of reinforcement is only approximate since auxiliary

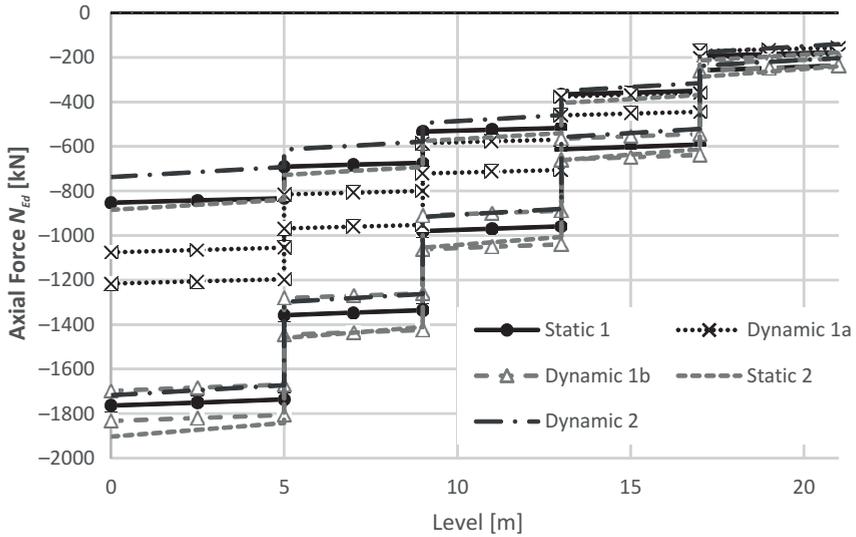


Fig. 5. Envelope of axial forces  $N_{Ed}$  in column in node 4D

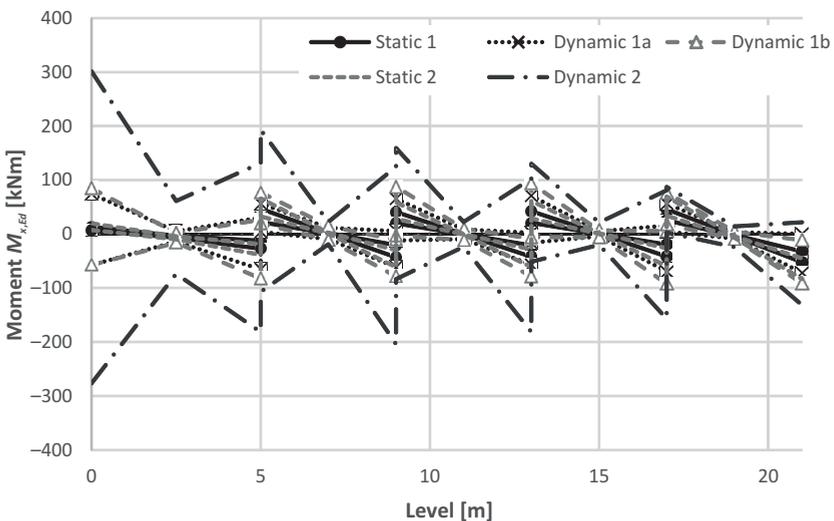


Fig. 6. Envelope of moments  $M_{x,Ed}$  in column in node 4D

and excess reinforcement (laps, anchorages, etc.) needed to properly construct a structural member were not included.

Reinforcement costs were compared for the reinforcement amount necessary from the point of view of the structure's loadbearing capacity. The additional amount of reinforcement required for laps and reinforcement anchorages was not taken into account.

The following average prices of the materials were assumed: reinforcement steel – 2.00 PLN/kg and grade C30/37 concrete – 280 PLN/m<sup>3</sup>.

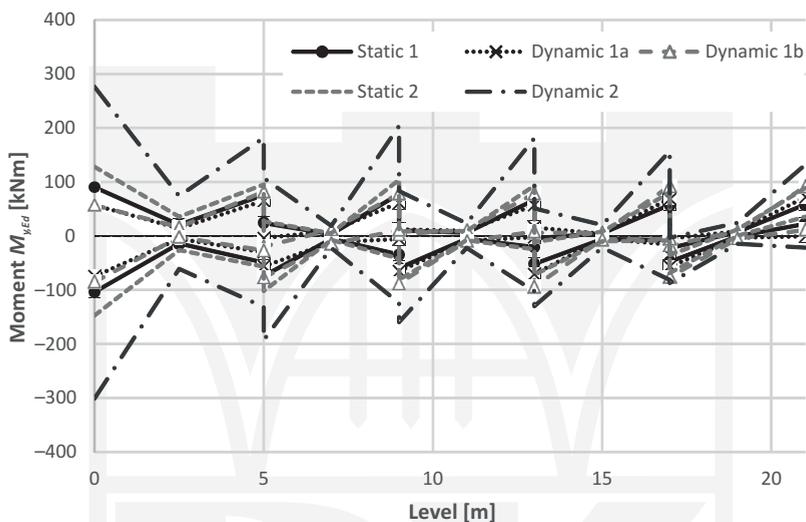


Fig. 7. Envelope of moments  $M_{y,Ed}$  in column in node 4D

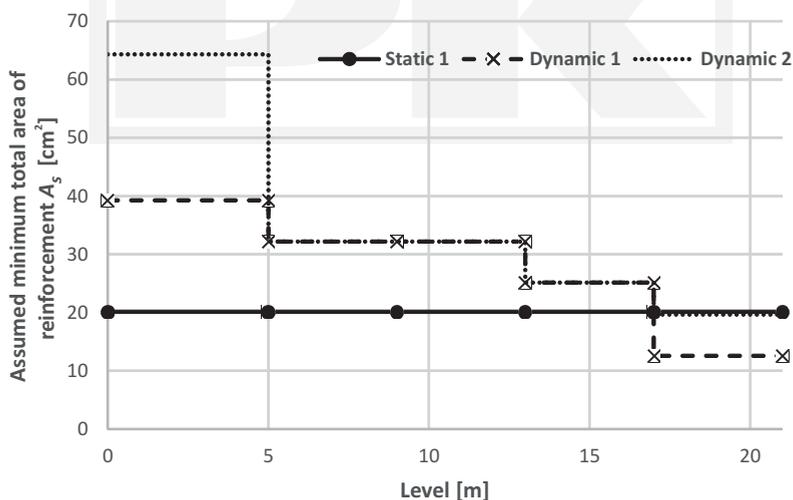


Fig. 8. Total required reinforcement area. In level interval of 5–17 dynamic 1 graph coincides with reinforcement value of dynamic 2

**Approximate costs of materials necessary to build column in node 4D**

	Static 1	Dynamic 1a	Dynamic 1b	Dynamic 2
Reinforcement weight	332 kg	431 kg	470 kg	595 kg
Reinforcement cost	664 PLN	862 PLN	940 PLN	1190 PLN
Concrete volume	3.36 m <sup>3</sup>	3.36 m <sup>3</sup>	3.36 m <sup>3</sup>	7.56 m <sup>3</sup>
Concrete cost	941 PLN	941 PLN	941 PLN	2117 PLN
Total cost	1605 PLN	1803 PLN	1881 PLN	3307 PLN

#### 4. Conclusion

The analysis has indicated several important factors having a bearing on the optimum design of building structures in mining areas.

The obvious conclusion is that when the dimensions of the cross sections are increased, the stiffness of the structure also increases and consequently the natural vibration period decreases (Tab. 2). However, the structural engineer must be aware that this change may have a highly adverse effect on the internal forces produced in the structure by seismically-induced loads. A comparison of Model 1 and Model 2 shows that when the cross sections of the columns in Model 2 were increased, the natural vibration period decreased by about 40%, but the  $S_d$  value increased several times in comparison with that in Model 1 (Fig. 2). An analysis of the graphs of the internal forces produced by the static load (Figs. 5–7) shows that the change in the stiffness of the interior columns did not result in a substantial redistribution of the internal forces. Therefore, one can assume that if the cross sections were increased, the steel consumption would significantly decrease. However, when designing a building structure in an area where mining damage occurs, one should take into account the increased dynamic loads. If the designer incorrectly assumed that by increasing the concrete cross section the amount of reinforcement would be reduced, this would result in 25% higher expenditures on reinforcing rods (Fig. 8, Tab. 3), not to mention the higher cost due to the increased consumption of concrete.

A mining-induced tremor has a more adverse effect on reinforced concrete columns than a static load (e.g. a wind load). In the analysed model, the moments generated by the dynamic combinations were at best close to the static ones, and at worst they were over twice higher. Moreover, during a tremor the column is bent in two directions. One should also note that in dynamic combinations the beneficial effect of the compressive axial force is much smaller.

The choice of a proper code combination of loads is not obvious. As shown in the example, when the more conservative approach [4–6, 13] than the one strictly complying with [14] is adopted, larger masses of the structure are taken into account, whereby the natural vibration periods are considerably lowered (Tab. 2). In the LGOM area, this may entail a reduction in the resultant acceleration acting on the loadbearing structure masses (Fig. 2). Due to all the above factors, the internal forces in the building's geometric system (Figs. 5–7) do not

differ significantly, regardless of the adopted combination, and consequently the cost of the preventive protection of the structure against seismically-induced impacts does not change significantly (Fig. 8, Tab. 3).

It should be added that because of the ongoing research into the forms of response spectra in mining areas in Poland and the implementation of code EC8 [12] in dynamic design, problems have arisen, which affect analytical results due to interpretation difficulties involved in mainly the unambiguous definition of calculation assumptions modelling the dynamics of a building structure. One can mention here problems connected with the choice of a proper response spectrum and the definition of a proper code combination.

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