IOP Conf. Series: Materials Science and Engineering **245** (2017) 032020 doi:10.1088/1757-899X/245/3/032020

Proposed Assessment of Dynamic Resistance of the Existing Industrial Portal Frame Building Structures to the Impact of Mining Tremors

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Abstract. The article presents the method for assessing dynamic resistance of the existing industrial portal frame building structures subjected to mining tremors. The study was performed on two industrial halls of a reinforced concrete structure and a steel structure. In order to determine the dynamic resistances of these objects, static and dynamic numerical analysis in the FEA environment was carried out. The scope of numerical calculations was adapted to the guidelines contained in the former and current design standards. This allowed to formulate the criteria, on the basis of which the maximum permissible value of the horizontal ground acceleration was obtained, constituting resistance of the analyzed objects. The permissible range of structural behaviour was determined by comparing the effects of load combinations adopted at the design stage with a seismic combination recognized in Eurocode 8. The response spectrum method was used in the field of dynamic analysis, taking into account the guidelines contained in Eurocode 8 and the guidelines of National. Finally, in accordance with the established procedure, calculations were carried out and the results for the two model portal frame buildings of reinforced concrete and steel structures were presented. The results allowed for the comparison of the dynamic resistance of two different types of material and design, and a sensitivity analysis with respect to their constituent bearing elements. The conclusions drawn from these analyses helped to formulate the thesis for the next stage of the research, in which it is expected to analyze a greater number of objects using a parametric approach, in relation to the geometry and material properties.

1. Introduction

The buildings located in mining areas are at risk, which is often due to the impacts of loads that occur during mining exploitation, and which were not taken into account at the design stage. This problem also applies to industrial portal frame building structures located in the areas where paraseismic influences occur [1,2,3,4].

In the context of the paraseismic influence, the assessment of resistance must be consistent with the current guidelines for load combinations taken into account in the assessment of the ultimate limit state. It is therefore necessary to compare the assumptions made at the design stage, resulting from the directives of the obsolete standards, with the current criteria dictated by the Eurocodes.

Additionally, when determining the structure resistance to the influence of mining tremors, just as in the case of designing process, it is required to take into account the intensity of vibrations representing the seismicity of the land on which the building was erected. Therefore, according to the

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guidelines contained in [5] and [6], in such cases, it is recommended to use the response spectrum analysis in dynamic calculations [7,8]. In this approach, kinematic excitation of the supports is assumed in the form of standard response spectra [1]. This allows to define the extreme response of the structure to the excitation caused by vibrations of the ground. Therefore, in the case of the objects which were designed taking mining tremors into consideration, their resistance is equal to the design acceleration a_g, which is assumed as the scaling factor for the standard response spectra [9]. By adapting this approach to the analysis of the existing objects, their resistance is represented by the maximum value of the parameter a_g, characterizing ground vibrations which the structure may carry, without reducing the safety criteria to be met by the current design standards. Such resistance, therefore, determines the extent of a permissible response of a structure subjected to dynamic load.

An essential problem which occurs in all kinds of structural analyses of the existing structures, is the uncertainty regarding the material parameters adopted at the design stage. As far as steel structures are concerned, it refers to the strength of the steel used in load-bearing elements. On the other hand, regarding reinforced concrete structures, in addition to the uncertain information on the concrete compressive strength, there is an additional deficit of information on the degree of reinforcement of its load-bearing elements. The proposed methodology is based on a comparison of the effects of the combination of the design loads to the effects of the seismic combination [10]. Assuming, according to the design principles, that the basis of the selection of geometry for reliable cross-sections of structural elements and reinforcement are the values of the internal forces resulting from the static analyses for the load combinations adopted at the design stage, the procedure for the resistance assessment can be reduced to the comparison of these effects from the design stage to the effects generated by the loads, not included in the design. The proposed procedure does not require the structural analysis of the cross-sections, which calls for the exact value of the strength of steel or concrete, and the actual degree of reinforcement of reinforced concrete elements.

The paper presents the procedure for determining dynamic resistance of model industrial portal frame buildings of steel and reinforced concrete structures, subjected to additional paraseismic impacts. This approach to the subject may enable to determine dynamic resistance of the existing building structures, performed in the conditions of uncertainty.

2. Research methodology

When formulating the criteria for the assessment of dynamic resistance, it was predetermined that the problem comes down to defining the additional extent of allowable effort of the existing structure in the event of a tremor. To define the formulated general thesis more precisely, the task involves finding a certain buffer included in the area of the combinations of the loads from the design stage, in which an additional load on the structure is allowed. Therefore, the proposed procedure for the resistance assessment involves the comparison of the effects of the combination of the loads from the design stage, acting on the structure, determined according to [11] with the effects of the seismic load combination dictated by [12].

In general, it requires:

1. the assumption of at least satisfactory technical condition of the structure, according to [13],
2. the assumption that the conditions for the load-bearing capacity of the building structure is met for all load combinations adopted at the design stage,
3. the identification of the potential scenarios of the structural response under dynamic load, which may increase the effort of its components or result in the work scheme which was not predicted during the design stage,
4. the summary of the predicted effects of the dynamic excitation, resulting from the seismic combination with the equivalent (regarding individual elements and directions of impacts) effects of the combinations adopted at the design stage,
5. the formulation of the dependencies allowing to identify the limit value characterizing the dynamic excitation (acceleration or velocity of ground vibration at the location of the structure).

In the design of hall structures without internal transport in the form of overhead cranes, in addition to the dead load, the dominant variable loads are wind load and snow load. Thus, the combinations adopted at the design stage according to [11], which take into account these mutual interactions, exhaust the scope of possible design situations where it is possible to search for the margin which would allow to carry additional dynamic impact. According to [12], in addition to the dead load of the structure and the equipment, individual cases of loads in the combinations from the design stage are mutually exclusive with the loads included in the seismic combination. This allows for the comparison of the effects of the combination of the design loads with the effects of the seismic combination. This comparison, in turn, allows to identify limit values characterizing the vibrations of the ground, induced by a mining tremor at the location of the structure.

According to [12] the criterion of the ultimate limit state STR, determining the scenario for structural damage due to the excessive effort of the cross-section or the strain of the load-bearing elements, is expressed by the following relationship:

$$\boldsymbol{E_d \leq R_d} \tag{1}$$

where:

E_d – design value of the impact effect,

R_d – design value of the load-bearing capacity

Assuming that the designed structure meets the assumptions made at the design stage, it can be concluded that the load-bearing capacity conditions are also met, which are considered for each required standardized combination of loads, according to [11]:

$$\boldsymbol{E_d^{PN} \leq R_d^{PN}} \tag{2}$$

where:

E_d^{PN} – design value of the impact effect for a given combination of loads, adopted at the design stage, according to [11]

R_d^{PN} – design value of the load-bearing capacity from the design stage, corresponding to a given combination of loads, according to [11].

It is well-known that the analysis of the effects of interactions for specific combinations of the loads allows for the design of reliable cross-sections of structural components of a given building structure. In the case of a structure being subjected to additional loads with inertial forces induced by ground vibrations, these cross-sections require that the conditions of the load-bearing capacity with the seismic combination are verified, according to [12]. Considering that the combinations of the loads are separate computational situations, and referring to the earlier assumption about meeting the capacity requirements for the design stage, the load-bearing capacity criterion for the analysis of the effects of the seismic combination can be formulated so that:

$$\boldsymbol{E_d^{SE} \leq R_d^{PN}} \tag{3}$$

where:

E_d^{SE} – design value of the effect of seismic impacts

R_d^{PN} – design value of the load-bearing capacity from the design stage, corresponding to a given combination of loads, according to [11].

On the other hand, according to (2), assuming the full use of the load-bearing capacity, taking into account the effects on a specific computational situation:

$$\boldsymbol{E_d^{PN} = R_d^{PN}} \tag{4}$$

the relationship (3) can be converted to the following form (5):

$$\boldsymbol{E_d^{SE} \leq E_d^{PN}} \tag{5}$$

Such formulation of the problem allows for the verification of the load-bearing capacity of the existing structure exclusively subject to the effects of the load combinations adopted for individual elements of the structure. This leads to a considerable simplification of the procedure for the assessment of dynamic resistance, as well as allows for the analysis of the structures where the information on the material properties or the actual degree of reinforcement of its load-bearing elements is uncertain.

3. Conditions of dynamic resistance of portal frame building structures

The conditions of dynamic resistance were formulated in relation to two directions of the potential kinematic excitation induced by a mining tremor. The longitudinal direction was considered (relative to the length of the hall structure) – x, and transverse direction - y. The excitation effects on each direction were defined in accordance with the requirements contained in [5]. Ultimately, the least favourable effect of all possible combinations dependent on the direction of a seismic wave was taken for the calculations, according to [5]:

$$E_d^{SE} = \max \begin{cases} \sqrt{E\left(a_{g,x}\right)^2 + E\left(a_{g,y}\right)^2} \\ \pm E\left(a_{g,x}\right) \pm 0,3 \cdot E\left(a_{g,y}\right) \\ \pm 0,3 \cdot E\left(a_{g,x}\right) \pm E\left(a_{g,y}\right) \end{cases} \tag{6}$$

where:

E_d^{SE} – the least favorable design value of the failure effect of seismic impacts,

$E(a_{g,x})$, $E(a_{g,y})$ – failure effects of the structural components resulting from tremor impact in the direction x and y, respectively,

$a_{g,x}$, $a_{g,y}$ – values of the horizontal ground acceleration in the longitudinal direction (x) and in the transverse direction (y), having the function of the scaling factors of the response spectrum curve taken for the calculations [9].

At the same time, for the directions x and y, the combinations of all the static loads from the design stage, which must be taken into account in designing portal frame structures, were compared. The values of the loads and the combinations from the design stage were determined pursuant to the standard [11,14,15]. The seismic combination was determined according to the guidelines contained in [6,12]. The following were adopted as the component cases of the loads: the wind load on the vertical partitions and roof slopes according to [15], the snow load according to [14] and the dead load of the structure and fitting elements according to [11].

Individual conditions allowing to determine dynamic resistance of a structure apply to all of its constituent structural components, i.e. posts, transoms, purlins and braces. Under each condition, depending on the form of the structure response, the values of normal and shear stresses were specified. These results formed the basis for determining the limit values of the dynamic excitation. Due to the fact that spatial models of the structures were used for the calculations, shear stresses in the cross-sections were also determined, which resulted from the torsional force originating from the seismic excitation or mutual interactions of the connected elements for the design stage combinations. The obtained results helped to specify whether the torsional effects generated by tremors did not exceed the effects of the loads adopted at the design stage. In most cases, portal frame building structures erected decades ago were designed with the use of the analytical models reduced to two-dimensional strut-and-tie models. Such analyses did not reveal torsional effects resulting from the interactions of the spatially connected structural components. Taking this into account, the calculated values of shear stresses from torsion accounted for only a complementary control set.

Considering the individual structural components, firstly these cross-sections were identified for which the stress values were extreme for all the combinations adopted at the design stage. Knowing that the stress values corresponding to these cross-sections form the basis for dimensioning a given structural component, it was predetermined that they would be the comparative base for the effects of the seismic combination acting on the structure. Then, the effects of the seismic combination were analyzed. Adopting the effects of the mutual interaction of the tremor on the relevant direction in accordance with [5], the values of the multiplier of the spectral curves a_g were calibrated according to [9] in such a way as to find the value above which the effects of the seismic combination would outweigh the extreme effects of the design stage combination. The criteria demonstrated in Table 1 relate to the assessment of the resistance of the posts, transoms, purlins and braces.

Table 1. Parameters adopted for the calculations to assess dynamic resistance for the conditions reducing the effort of individual structural components

<table>
<tr><td colspan="4">Combinations adopted at the design stage [11] - K_i^{PN}</td></tr>
<tr><td>Permanent loads - G_{ki}^{PN}</td><td colspan="3">Variable loads – environmental - Q_{ki}^{PN}</td></tr>
<tr><td></td><td>Denotation</td><td>γ_f</td><td>Description</td></tr>
<tr><td rowspan="3">Dead weight of the structure
Weight of the roofing
Weight of wall cladding</td><td>W_{Pk}^{PN}</td><td>1,3 [15]</td><td>characteristic value of the wind load applied to the vertical partitions</td></tr>
<tr><td>W_{Dk}^{PN}</td><td>1,3 [15]</td><td>characteristic value of the wind load applied to the roof</td></tr>
<tr><td>S_k^{PN}</td><td>1,4 [14]</td><td>characteristic value of the snow load</td></tr>
<tr><td colspan="4">Seismic combination according to [12] - K_{SEI}^{EN}</td></tr>
<tr><td colspan="2">General form of seismic combination</td><td colspan="2"></td></tr>
<tr><td colspan="2" rowspan="2">$\sum_{i=1}^{m} G_{k,i} + A_{ed}$</td><td>$\sum_{i=1}^{m} G_{k,i}$</td><td>dead loads of the structure and fitting elements</td></tr>
<tr><td>A_{ed}</td><td>design value of seismic impact according to (5)</td></tr>
<tr><td colspan="4">The final form of the condition in the analysis of each structural component</td></tr>
<tr><td colspan="4">$$\max\left\{\sigma,\tau\left(K_i^{PN}\right)\right\} \geq \max\left\{\sigma,\tau\left(K_{SEI}^{EN}\right)\right\}$$
$\sigma,\tau\left(K_i^{PN}\right)$ - values of normal and shear stresses for the design stage combination,
$\sigma,\tau\left(K_{SEI}^{EN}\right)$ - values of normal and shear stresses for the seismic combination,</td></tr>
</table>

4. Results and discussions

The subject of the study are the structures of two industrial portal frame buildings with a similar spatial geometry, of a steel structure and of a prefabricated reinforced concrete structure. The steel industrial hall has the dimensions of its horizontal projection of 20.0 m x 36.0 m, and the dimensions of the reinforced concrete industrial hall are 18.0 x 36.0 m. In the case of the steel hall, the main supporting system are transverse steel frames with the height equal to h = 10.0 m. The post and beam system, also with the height of h = 10.0 m, is responsible for the load bearing capacity of the reinforced concrete hall in the transverse plane. The geometry of the object and the parameters of its components were selected in accordance with the guidelines of the P-70 system [16]. In both building structures, the transverse systems are spaced at 6.0 m. The roofing of the steel industrial hall is a lightweight sandwich panel based on trapezoidal sheet, mounted on steel purlins. On the other hand, the roofing of the reinforced concrete structure is made of reinforced concrete ribbed roof panels based directly on the transoms. The steel hall has transverse and longitudinal roof braces, arranged at the external parts of the roof slopes. Moreover, vertical braces were used in each external part of the outer walls, and the longitudinal braces running at half the height of the walls along the entire length. In the case of the reinforced concrete hall, the longitudinal stiffness, pursuant to the guideline of the P-70 system, is ensured by the prefabricated reinforced concrete roof slabs. The lateral stiffness is ensured by the posts embedded in the footings. Numerical models of the Finite Element Method of the analyzed objects were demonstrated in Figure 1 and Figure 2.

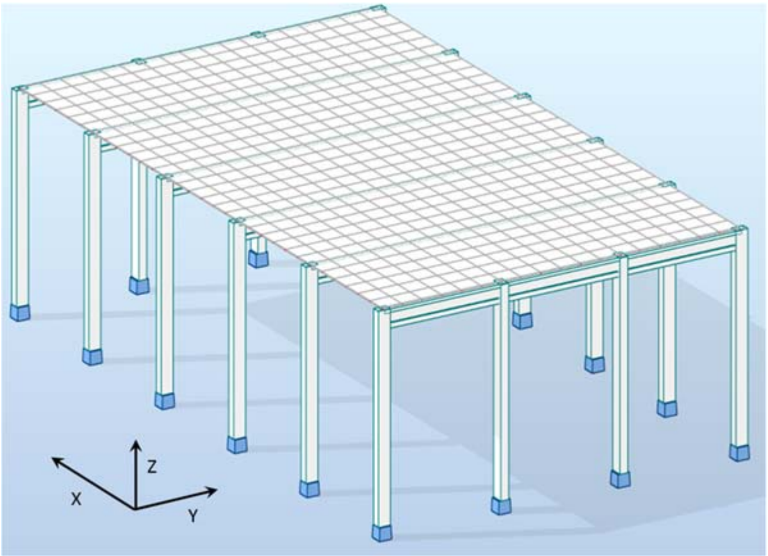

Figure 1. Numerical model of the reinforced concrete portal frame building structure

The dynamic analysis was performed by the response spectrum method [1,7,8,17]. The normalized elastic response spectrum for the ground B was adopted as the dynamic excitation [9]. Such an approach, according to the criteria of the resistance assessment, allowed for the identification of the permissible value of the horizontal ground acceleration $a_{H,dop}=a_{x,dop}=a_{y,dop}$. In the calculations, according to [18] the damping factor ξ = 0.05 was used for the steel structure hall, and ξ = 0.02 for the reinforced concrete hall. The summation of the contributions from the individual modes of vibrations was performed according to [6] using the CQC method (Complete Quadratic Combination [1]).

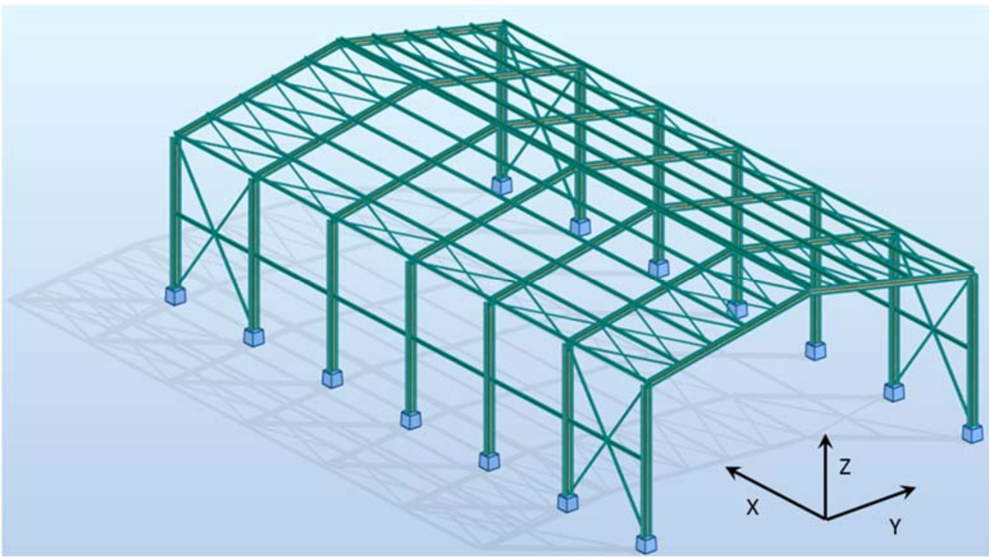

Figure 2. Numerical model of the steel portal frame building structure

IOP Conf. Series: Materials Science and Engineering **245** (2017) 032020 doi:10.1088/1757-899X/245/3/032020

The performed analyses resulted in the limit value of the ground acceleration $a_{H,dop}$. The obtained results were demonstrated in Tables 2 and 3. In addition to the ultimate limit value expressed by the permissible ground acceleration component, which can be produced by the object as a whole, also the results obtained for the individual structural components were included $a_{H,dop}^{Element}$. For each given element, it was defined which parameter characterizing its effort played the biggest role in the verification of its resistance. These parameters were the values of normal stresses (σ) and shear stresses (τ). The control values including the effects in the form of torsion of the individual components, expressed in the shear stress values (τ^R), were demonstrated in Tables 4 and 5.

These data demonstrate that the proposed methodology, in addition to the ultimate set of the limit values of the components of horizontal ground acceleration, also allows for the studied structure to be subjected to the simplified sensitivity analysis with respect to its structural components. Such information may contribute to the increased effectiveness of potential structural interference in the object because it identifies the most sensitive elements of its load-bearing structure.

Ultimately, the obtained results demonstrated that the dynamic resistance of the steel portal frame building is 5 times higher ($a_{H,dop}$ = 2,11 m/s^2) than that of a geometrically similar reinforced concrete portal frame building with $a_{H,dop}$ = 0,48 m/s^2. The significant difference in resistance for the analyzed objects obtained from the calculations confirms the results of the studies described in [3]. According to [3] it may be due to a greater mass of the reinforced concrete structure, and the resulting higher values of inertial forces which occur during the dynamic excitation.

In the case of the reinforced concrete structure, the most sensitive elements in terms of dynamic resistance were intermediate posts located in the plane of the gable wall (Table 2). On the other hand, the most sensitive components of the analyzed steel structure were vertical braces, arranged at the external parts of the industrial hall on the longitudinal direction (Table 3). The additional analysis of the effects which occurred as a result of the torsion of the individual load-bearing elements suggests that the reinforced concrete hall is characterized by a greater susceptibility to this type of excitation induced by ground vibrations than the steel hall (Tables 4 and 5).

Table 2. Dynamic resistance of the steel portal frame building structure taking into account the resistance of its components

	Resistance of the component determined relative to the normal stress failure criterion	Resistance of the component determined relative to the shear stress failure criterion	Resistance of the component $a_{H,dop}^{Element}$ [m/s^2]	Resistance of the object $a_{H,dop}$ [m/s^2]
Structural components	$a_{H,dop}^{Element,\sigma}$ [m/s^2]	$a_{H,dop}^{Element,\tau}$ [m/s^2]	$\min\left\{a_{H,dop}^{Element,\sigma}, a_{H,dop}^{Element,\tau}\right\}$	$\min\left\{a_{H,dop}^{Element,i}\right\}$
Purlins	6.98	NO LIMIT	6.98	2.11
Transoms	6.58	12.15	6.58	
Posts	3.54	6.50	3.54	
Roof slope braces	2.32	N/A	2.32	
Vertical braces	2.11	N/A	2.11	
Longitudinal braces	8.16	N/A	8.16	

However, the obtained results of the torsion are not binding as they result from the adoption of the three-dimensional model for the calculations, while the models most frequently adopted at the design stage for the analyzed structures were two-dimensional static schemes, where the torsional effect was not taken into account while dimensioning the individual load-bearing elements. Therefore, the search for the margin of the load-bearing capacity resulting from the original design assumptions for such a case is unfeasible, and the results are only control data and they do not determine the resistance of the analyzed structures.

Table 3. Parameters adopted for the calculations to assess dynamic resistance for the conditions reducing the effort of individual structural components

	Resistance of the component determined relative to the normal stress failure criterion	Resistance of the component determined relative to the shear stress failure criterion	Resistance of the component $a_{H,dop}^{Element}$ [m/s²]	Resistance of the object $a_{H,dop}$ [m/s²]
Structural components	$a_{H,dop}^{Element,\sigma}$ [m/s²]	$a_{H,dop}^{Element,\tau}$ [m/s²]	$\min\left\{a_{H,dop}^{Element,\sigma}, a_{H,dop}^{Element,\tau}\right\}$	$\min\left\{a_{H,dop}^{Element,\mathrm{i}}\right\}$
Purlins	0.58	1.45	0.58	0.48
Transoms	0.60	1.09	0.60	
Posts	0.48	1.25	0.48	

Table 4. Dynamic resistance of the steel portal frame building structure taking into account the resistance of its components - control data due to torsion τ^R

	Resistance of the component determined relative to the shear stress failure criterion from torsion τ^R
Structural components	$a_{H,dop}^{Element,\sigma}$ [m/s²]
Purlins	2.06
Transoms	2.14
Posts	4.31

Table 5. Dynamic resistance of the reinforced concrete portal frame building structure taking into account the resistance of its components - control data due to torsion τ^R

	Resistance of the component determined relative to the shear stress failure criterion from torsion τ^R
Structural components	$a_{H,dop}^{Element,\sigma}$ [m/s²]
Purlins	0.33
Transoms	0.34
Posts	1.00

The ultimate resistance of the analyzed structures is therefore determined by the limit values of the horizontal ground acceleration summarized in Tables 2 and 3. It should be noted that the sensitivity of the structure is closely linked to its geometry and the material parameters of its load-bearing components. Thus, when analysing the resistance of a structure of other geometric and material features, it should be expected that these values will change.

5. Conclusions

The methodology for the assessment of dynamic resistance of portal frame buildings proposed in this paper allows to determine a permissible safety margin for the existing structure in the case of the occurrence of mining tremors. This procedure allows to determine the maximum permissible value of the horizontal ground acceleration ($a_{H,dop}$),even if there is no exact information on the reinforcement of load-bearing elements or the parameters describing the resistance of the materials used.

The analysis of the obtained results also provides a possibility to assess the sensitivity of individual structural components to the dynamic load induced by mining tremors. Such additional information may prove useful in the decision-making process regarding potential structural interference in the object, aimed to adapt the structure to carry additional loads induced by mining tremors in the specific area.

It should be noted that the adopted approach based on the response spectrum method depends on the shape of the spectrum used. Thus, the determined resistance is not an absolute measure but depends on the seismicity of the area where the object is located.

Acknowledgement
This study was supported by the AGH University of Science and Technology statutory research No.11.11.150.005

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