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DETERMINING THE DYNAMIC RESISTANCE OF EXISTING STEEL INDUSTRIAL HALL STRUCTURES FOR AREAS WITH DIFFERENT SEISMIC ACTIVITY

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The paper presents the results of research concerning the assessment of dynamic resistance of existing industrial hall structures located in areas with different seismic activity. The basis for analyses was a three-nave industrial hall with a steel structure. Numerical calculations were performed using the finite element method (FEM), using the response spectrum method in dynamic analysis. The calculations were carried out in variants, using standard accelerated response spectra according to *Eurocode 8* and those determined for the *Upper Silesian Coal Basin* (USCB) and *Legnica-Glogow Copper District* (LGCD) area. Using the author's procedure for the assessment of the dynamic resistance, for each of the extortion analysed, the structure's response to the dynamic excitation was compared with the effects of load combinations adopted at the design stage, thus establishing the limit values of the design horizontal ground acceleration $a_{g,H}^{max}$ understood as the structure's resistance to tremors. This allowed to assess the impact of seismic activity from a specific area on the dynamic resistance of the subjected object. The article also discusses the way of interpretation and the scope of application of the obtained results and proposed procedure.

***Keywords*:** dynamic resistance; spectrum response analysis; mining tremors; steel industrial hall structure

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1. INTRODUCTION

Damage to building structures is an indispensable consequence of underground mining exploitation. Sometimes, the load-bearing capacity of the construction elements is exhausted, which can be a safety hazard, and requires expert assessment.

The field of engineering dealing with the assessment of threats to buildings associated with mining exploitation, includes the assessment of resistance of the existing objects to additional loads occurring during their use, but which were not included at the design stage. In mining areas, these impacts mostly include the effects of continuous surface deformation [1–3] and mining tremors [4–9]. The literature widely describes the methodology for assessing the resistance of building structures to the impacts of continuous deformation [10–13]. The problem of assessing the resistance of buildings to the dynamic impacts caused by mining tremors is much less frequently discussed [14–19,11].

The vast majority of the information contained in structural dynamics research concerns the design of new structures [20–22] or the analysis of the response of existing structures to a given dynamic extortion [23–27]. Guidelines for retrofit building structures in seismically active areas are also becoming increasingly common [28–31]. In this way, based on the absolute values of the material characteristics, the intensity of damage is reduced and the load-bearing capacity of the individual components is ensured. In addition to this deterministic approach, there are also descriptive guidelines in the form of scales for the assessment of mining tremor impacts in the context of damage to buildings [32]. Unfortunately, they do not have a universal character, because their levels of damage intensity result from symptomatic recognition of the effects of a given tremor, and can be applied to a rather limited group of objects [32–35]. Therefore, the field of research related to the assessment of resistance of the existing building structures to the dynamic impacts caused by mining tremors is still expanding.

In contrast, the actual assessment of the dynamic resistance of existing structures falls between the directives for the design stage of new buildings and the damage assessment of existing structures. In fact, it requires a thorough analysis of the structure, but this is difficult to carry out with absolute certainty. For existing structures, this difficulty results from the lack of information on the real strength characteristics of materials.

For this reason, a proprietary methodology has been developed to assess the dynamic resistance of building structures, including bridge structures [36] and industrial halls [37,38]. It is formulated in such a way that it does not require information about the yield strength of steel, the real degree of reinforcement of reinforced concrete elements, and the compressive and tensile strength of concrete.

In general, the proposed approach is, to a certain extent, consistent with the capacity spectrum method [39–42], but does not require analysis of the absolute limits of damage intensity or cross-sectional strength.

In practice, the so-called standard response spectra [43], which characterise the seismic activity of a specific area, are used for the design of structures located in seismic areas. They constitute a set of separate maximum values of acceleration, velocity and displacement of vibrations, assigned to the frequency domain. During the performed calculations, not one, but all maximum values of the analysed frequency range are taken into account. The contribution of each to the structure's response is determined by the resonance effect depending on the dynamic characteristics of the analysed object, i.e. the normal mode and the corresponding frequencies. The final response of the structure is, therefore, the sum of all normal modes subject to excitation. This results in the determination of the permissible characteristics of ground vibrations becoming complicated, due to the safety of the existing building structure.

Assessment of the resistance of the existing industrial hall structures must be consistent with the guidelines regarding standard load combinations. It is, therefore, necessary to refer to the assumptions adopted at the design stage and, subject to the directives of the obsolete standards, to the current criteria set out in the Eurocodes [44]. Conversely, the comparison of the effects of the load combinations acting on the structure, adopted at the design stage, with the effects of the accidental combination dictated by [44] and [11] enables the identification of reserves of load-bearing capacity, allowing for the structure to become additionally loaded with a mining tremor. Knowing this value, it is possible to define the limit values of the parameters describing tremors occurring in the specific area, which can be carried by a given object without any threat to its safety. A detailed description of the procedures for determining such resistance for existing industrial hall structures was presented in [37] and [38].

In the case of the existing structures, in the context of resistance, the standard response spectrum and the permissible value of parameter a_g are representative. Parameter a_g, which defines the design ground acceleration according to [22] for the newly designed structures for seismic impacts, is taken arbitrarily and expresses the predicted ground vibrations at the location of the object. In the case of the resistance of the existing building structures, which were not designed for seismic impacts, it may be determined according to the procedure described in [37,38]. Then, it becomes the scaling factor for the standard response spectrum adopted for the dynamic analysis describing the seismicity of the specific area. Determination of the limit value for parameter a_g with a predetermined shape of the standardised spectrum model, in fact, provides the shape of the permissible response spectrum, which the structure may carry without any threat to its security in the specific area. Here, there is a clear convergence with the capacity spectrum method, which was mentioned earlier.

The paper demonstrates the results of studies on the resistance of a typical three-nave steel hall structure for areas of varying seismic activity. For this purpose, standard acceleration response spectra contained in [22], spectra resulting from the adaptation of the provisions contained in Eurocode 8 to the design requirements for seismically active areas in Poland [7], and predetermined standard acceleration spectra for the Legnica-Glogow Copper District (LGCD) [6,45] and the Upper Silesian Coal Basin (USCB) [5] were used. In addition, analyses based on the non-linear standard response acceleration spectra for the LGCD mining area [7] were also performed. Finally, results were obtained for 14 areas of different seismic activities. Such a wide range of potential excitations adopted for the study aimed to illustrate the effect of the seismic intensity of a given area on the dynamic resistance of the structure located there.

In this study, for each of the analysed cases, the limit values of the design ground acceleration in the horizontal plane $a_{g,H}^{max}$ were determined, which the existing structure could carry without a safety hazard, were determined.

2. Research methodology

When formulating the criteria for the assessment of dynamic resistance, it was predetermined that the problem focuses on defining the additional extent of allowable effort of the existing structure in the event of a tremor. In order 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 [46–48] with the effects of the accidental load combination dictated by [44] and [11].

In general, this requires:

1. the assumption of the, at least, satisfactory technical condition of the structure, according to [49];
2. the assumption that the conditions for the load-bearing capacity of the building structure are met for all load combinations adopted at the design stage;
3. the summary of the predicted effects of the dynamic excitation, resulting from the accidental load combination with the equivalent (regarding individual elements and directions of impacts) effects of the combinations adopted at the design stage;

4. the formulation of the dependencies allowing to identify the limit value characterising 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 (1) according to [48], 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 [11], in addition to the dead load of the structure and the equipment, individual cases of loads in various combinations from the design stage are mutually exclusive from the loads included in the accidental load combination (2). This allows for the comparison of the effects of the combination of the design loads with the effects of the accidental combination. This comparison, in turn, allows to identify limit values characterising the vibrations of the ground, induced by a mining tremor at the location of the structure.

$$\sum_{i=1}^{m} \gamma_{fi} G_{ki} + \sum_{i=1}^{n} \Psi_{0i} \gamma_{fi} Q_{ki} \tag{1}$$

where:

γ_{fi} – partial factor for action i

G_{ki} – characteristic value of permanent action i

Q_{ki} – characteristic value of variable action i

Ψ_{0i} – simultaneity factor of variable load

$$\sum_{J\geq 1} \gamma_{G,j} G_{k,j} + 0{,}8 \cdot \gamma_g Q_{g,k} + 0{,}8 \cdot \sum_{i>1} \gamma_{Q,i} Q_{k,i} + A_w \tag{2}$$

where:

$\gamma_{G,j}$ – partial factor for permanent action j

$G_{k,j}$ – characteristic value of permanent action j

γ_g – partial safety factor for mining impacts

$Q_{g,k}$ – characteristic value of mining impact caused by continuous deformation

$\gamma_{Q,i}$ – partial safety factor for variable action i

$Q_{k,i}$ – characteristic value of variable action i

A_w – accidental action induced by mining tremors

If the structure, however, is not affected by continuous soil deformation, an exceptional load combination according to [44], which is given in relationship (3), must be taken into account. Such a situation of impacts has been taken into account within the framework of the studies described in this article.

(3) $$\sum_{j\geq 1} G_{k,j} + (\Psi_{1,1} or\ \Psi_{2,1})Q_{k,1} + \sum_{i>1} \Psi_{2,i} Q_{k,i} + A_w$$

where:

$G_{k,j}$ – characteristic value of permanent action j

$Q_{k,i}$ – characteristic value of variable action i

$Q_{k,1}$ – characteristic value of leading variable action i

$\Psi_{1,1}$ – reduction factor for frequent leading variable action $Q_{k,1}$

$\Psi_{2,1}$ – reduction factor for quasi-permanent leading variable action $Q_{k,1}$

$\Psi_{2,i}$ – reduction factor for quasi-permanent variable action $Q_{k,i}$

A_w – accidental action induced by mining tremors

According to [44], the criterion of the Ultimate Limit State (ULS), determining the scenario for structural damage due to the excessive effort of the cross-section or the strain of the load-bearing elements (STR), is expressed by the following relationship:

(4) $$E_d \leq R_d$$

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 standardised combination of loads, according to [48]:

(5) $$E_d^{PN} \leq R_d^{PN}$$

where:

E_d^{PN} – design value of the impact effect for a given combination of loads, adopted at the design stage, according to [48]

R_d^{PN} – design value of the load-bearing capacity from the design stage, corresponding to a given combination of loads, according to [48]

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 accidental combination are verified according to [44]. 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 accidental combination can be formulated so that:

$$E_d^{ACC} \leq R_d^{PN} \tag{6}$$

where:

E_d^{ACC} – design value of the effect of accidental impacts

R_d^{PN} – design value of the load-bearing capacity from the design stage, corresponding to a given combination of loads, according to [48]

Conversely, according to (5), assuming the full use of the load-bearing capacity, taking into account the effects on a specific computational situation:

$$E_d^{PN} = R_d^{PN} \tag{7}$$

relationship (6) can be converted to the following form (8):

$$E_d^{ACC} \leq E_d^{PN} \tag{8}$$

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 allowing 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.

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 (relative to the length of the hall structure) – x, and the transverse direction – y were considered. The excitation effects on each direction were defined in accordance with the requirements contained in [22]. Ultimately, the least favourable effect of all possible combinations, dependent on the direction of a seismic wave, was taken into account for the calculations, according to [11]:

$$A_w = max \begin{cases} \sqrt{A_{w,x}{}^2 + A_{w,y}{}^2} \\ \pm A_{w,x} \pm 0{,}3 \cdot A_{w,y} \\ \pm 0{,}3 \cdot A_{w,x} \pm A_{w,y} \end{cases} \tag{9}$$

where:

A_w – the least favourable design value of the failure effect of tremor impacts

$A_{w,x}$, $A_{w,y}$ – failure effects of the structural components resulting from tremor impact in directions x and y respectively

At the same time, for directions x and y, the combinations of all 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 standard [46–48]. The accidental combination was determined according to the guidelines contained in [22,44]. The following were adopted as the component cases of the loads: the wind load on the vertical partitions and roof slopes according to [46], the snow load according to [47], and the dead load of the structure and fitting elements according to [48].

Individual conditions allowing to determine the 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 generalised internal forces (M – bending moments, N – longitudinal force, Q – shear force) were specified. These results formed the basis for determining the limit values of the dynamic excitation.

Considering the individual structural components, firstly, these cross-sections for which the stress values were extreme for all combinations adopted at the design stage were identified. 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 constitute the comparative base for the effects of the accidental combination acting on the structure. Then, the effects of the accidental combination were analysed. Adopting the effects of the mutual interaction of the tremor on the relevant direction in accordance with [22], the values of the multiplier of the spectral curves a_g were calibrated according to [38] in such a way as to find the value above which the effects of the accidental combination would outweigh the extreme effects of the design stage combination. In general, the criteria demonstrated in Table 1 relate to the assessment of the resistance of the columns, 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

Load combination from design stage [48]– $\boldsymbol{Comb^{PN}}$					
$\sum_{i=1}^{m}\gamma_{fi}G_{ki}+\sum_{i=1}^{n}\Psi_{0i}\gamma_{fi}Q_{ki}$					
Permanent loads $\boldsymbol{G_{ki}}$	**Variable loads – environmental** Q_{ki}				
According to [48]	**Denotation**	γ_f	Ψ_{0i}	**Description**	
Dead weight of the structure	$Q_{k1}=W_{Pk}^{PN}$	1.3	1.0	characteristic value of the wind load applied to the vertical partitions	
Weight of the roofing	$Q_{k1}=W_{Dk}^{PN}$	1.3	1.0	characteristic value of the wind load applied to the roof	
Weight of wall cladding	$Q_{k2}=S_{k}^{PN}$	1.4	0.9	characteristic value of the snow load	
Accidental combination according to [44] – $\boldsymbol{Comb_{ACC}^{EN}}$					
$\sum_{j\geq1}G_{k,j}+(\Psi_{1,1}or\ \Psi_{2,1})Q_{k,1}+\sum_{i>1}\Psi_{2,i}Q_{k,i}+A_w$					
Permanent loads $\boldsymbol{G_{k,i}}$	**Variable loads – environmental** $Q_{k,i}$				
According to [50]	**Denotation**	$\Psi_{1,1}$	$\Psi_{2,1}$	$\Psi_{2,2}$	**Description**
Dead weight of the structure	$Q_{k,1}=W_{Pk}^{EC1}$	0.2	–	–	characteristic value of the wind load applied to the vertical partitions
Weight of the roofing	$Q_{k,1}=W_{Dk}^{EC1}$	0.2	–	–	characteristic value of the wind load applied to the roof
Weight of wall cladding	$Q_{k,2}=S_{k}^{EC1}$	–	–	0.2	characteristic value of the snow load
The final form of the condition in the analysis of each structural component					
$\boldsymbol{max\{M,N,Q\ (Comb^{PN})\}=max\{M,N,Q(Comb_{ACC}^{EN})\}}$					

3. Results

The subject of the research is a three-nave industrial hall structure (Fig. 1). The main nave has the following dimensions: 32.0 m × 42.0 m (width of main nave – 20.0, widths of two sides naves – 6.0 m) and height 8.35 m. The supporting structure, in the transverse plane, consists of fixed columns of I-section (HEB360) and a truss girder. The structural elements of the truss girder are made of rolled cross-sections (upper and bottom chord – RP rectangular hollow section 180 × 100 × 10 mm, crossbars and posts – C100 and C80 sections). On both sides of the main nave, in a symmetrical manner, are two side naves with a mean height of 3.10m. The main beams of the transverse support system are based on the outer columns of the side nave and the main columns of the main nave. The columns of the side naves are pinned to the ground. Both main beams and columns are made of rolled profiles (main beams – IPE300, columns – HEB320).

The hall has been braced transversely at the level of the roof slope by three X-type bracing bands. The longitudinal bracing of the hall is realised in the form of X-type braces between the columns of the main nave and the side naves. This type of bracing is available in three bands. Additionally, the hall structure is braced in the plane of the ridge with a longitudinal truss. The roofing is realised as a layered, steel-based trapezoidal sheet, covered with mineral wool and galvanised sheet metal.

Static and dynamic analyses were performed in Autodesk Robot Structural Analysis Professional [51]. The numerical model of the structure is presented in Fig. 1. The numerical model uses beam and surface finite elements according to [51].

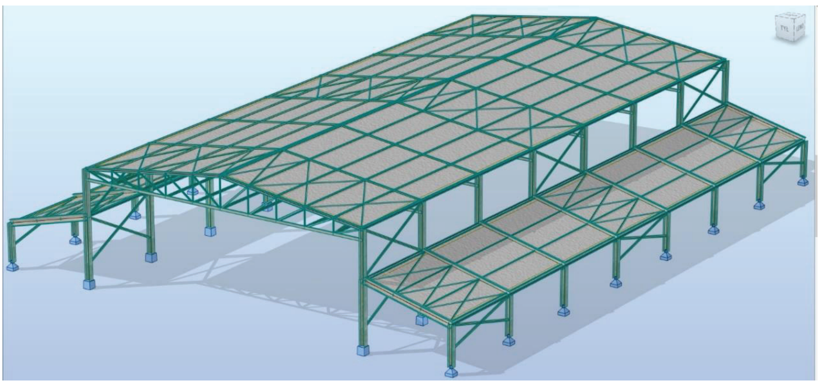

Fig. 1. Diagram demonstrating the geometry of the structure

The dynamic analysis was performed by the response spectrum method [39]. The normalised elastic response spectra for all ground types according to Eurocode 8 [22], as well as the regional standard acceleration response spectra for the USCB and the LGCD were adopted as a dynamic excitation (Table 2, Fig. 2 to Fig. 5). Additionally, Fig. 5 presents the proposed non-linear standard acceleration response spectra for the LGCD area (LGCD-A-INEL, LGCD-B-INEL and LGCD-C- INEL) [7]. This allowed the comparison of the established resistance between an elastic case and a situation where, due to the behaviour of the structure [52,53], energy dissipation is permitted.

The bracing system was selected on the basis of literature analysis in the field of designing steel structures in seismic areas [21,22].

All spectra representing the seismic activity of the mining areas of the USCB and the LGCD are the result of the performed research studies described in [22,5,6]. The spectra labelled as LGCD-A-EL, LGCD-B-EL and LGCD-C-EL (Fig. 4) constitute an adaptation of Eurocode 8 to the seismic conditions prevailing in the LGCD [7]. The curves denoted as LGCD-1, LGCD-2 and USCD (Fig. 3) are the results of research, which mainly involved the record of tremors occurring in the mining areas of the USCD and the LGCD [5,45,6]. A clear difference is the extent of these curves. In the case of the LGCD-1 and USCD curves, this range is twice narrower than in the case of all other spectra describing the seismicity of the LGCD ares (Fig. 3). This difference may have an effect on the other values of the parameters adopted for the description of the structure response. This, in turn, may lead to different limit values of ground acceleration a_g defining the resistance of the structure, as obtained from the calculations.

Spectra curves based on Eurocode 8 [22] are presented in Fig. 2. They constitute a comparative base for local spectra in the mining areas under consideration.

In the calculations, the damping ratio value of ξ=5% has been assumed for this type of structure [28]. The conditions of dynamic resistance were formulated in relation to the two directions of a potential kinematic excitation induced by a mining tremor. The longitudinal direction (relative to the length of the structure) – x, and the transverse direction – y were considered. The effects of excitation on individual directions in the horizontal plane were defined in accordance with the requirements contained in [11] and [22]. Ultimately, the least favourable effect of all possible combinations, dependent on the direction of a seismic wave, was taken into account for the calculations, according to (9).

$$E_d^{SE} = max\begin{cases}\sqrt{E\left(a_{g,x}\right)^2 + E\left(a_{g,y}\right)^2}\\ \pm E\left(a_{g,x}\right) \pm 0{,}3 \cdot E\left(a_{g,y}\right)\\ \pm 0{,}3 \cdot E\left(a_{g,x}\right) \pm E\left(a_{g,y}\right)\end{cases} \tag{10}$$

where:

E_d^{SE} – the least favourable effect from tremor impacts

$E\left(a_{g,x}\right), E\left(a_{g,y}\right)$– effects resulting from tremor impact in directions x and y respectively

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

At the beginning of the dynamic analysis, 10 forms of eigenmodes were extracted. They were in the frequency range from 2.56 to 9.33 Hz. The summation of the contributions from the individual modes of vibrations was performed according to [22] using the CQC method (Complete Quadratic Combination – [43]). The performed analyses, taking into account the resistance assessment criteria defined in [38], resulted in the limit value of the horizontal component of design ground acceleration $a_{g,H}^r = max\{a_{g,x}^{max}, a_{g,y}^{max}\}$.

According to [22], the structure was treated as a moment resisting frame combined with concentric bracings. This allowed the narrowing of the range of analysed elements to: pillars and bracings. In addition, the frame's main girder was also analysed, taking into account the side naves. The obtained results of the resistance for all adopted tremor excitations are contained in Table 2 and illustrated in Fig. 6.

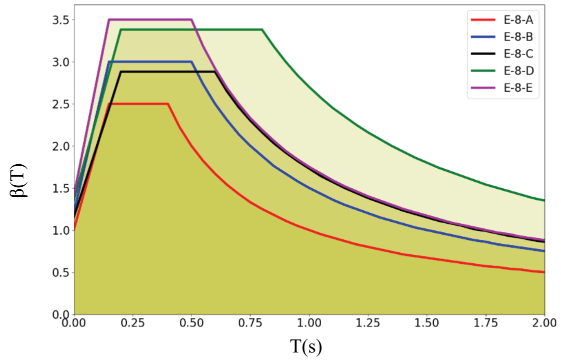

Fig. 2. Curves of the standardized acceleration response spectra adopted for the dynamic calculations according to EC-8 [22]

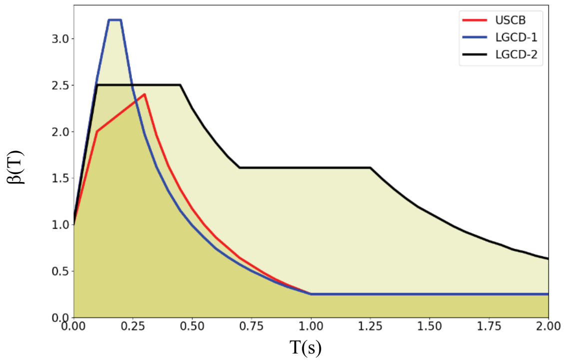

Fig. 3. Curves of the regional standardized acceleration response spectra for the areas of LGCD and USCB according to [5,45,6]

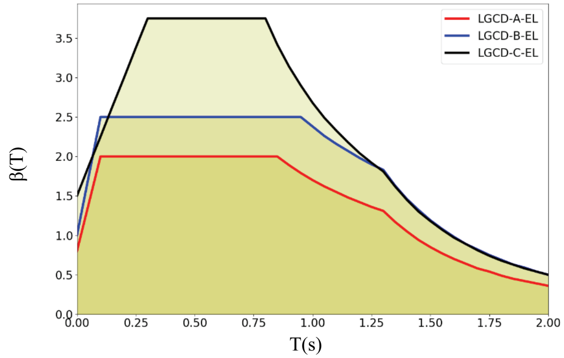

Fig. 4. Curves of the standardized elastic acceleration response spectra from EC-8 adopted to the LGCD according to [7]

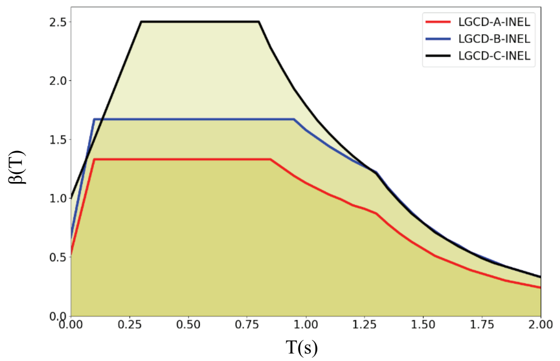

Fig. 5. Curves of the standardized inelastic acceleration response spectra from EC-8 adopted to the LGCD according to [7]

Table 2. Summary of the results of the resistance assessment of the analysed structure for specific areas with different seismic activity (the elements determining the final dynamic resistance value are marked in grey)

Spectrum curve	**Results for individual elements of the superstructure $a^r_{g,H,i}$ [m/s²]**				**Dynamic resistance of the structure $a^r_{g,H}$ [m/s²]**
	pillars (main nave)	pillars (side naves)	main girder (side naves)	wall braces	
E_8_A	**0.34**	**0.37**	**0.42**	**0.28**	**0.28**
E_8_B	**0.28**	**0.31**	**0.35**	**0.23**	**0.23**
E_8_C	**0.29**	**0.32**	**0.37**	**0.25**	**0.25**
E_8_D	**0.25**	**0.27**	**0.31**	**0.22**	**0.22**
E_8_E	**0.24**	**0.26**	**0.30**	**0.20**	**0.20**
SCUB	**0.35**	**0.38**	**0.44**	**0.32**	**0.32**
LGCD-1	**0.38**	**0.47**	**0.54**	**0.22**	**0.22**
LGCD-2	**0.34**	**0.37**	**0.42**	**0.28**	**0.28**
LGCD-A-EL	**0.42**	**0.46**	**0.53**	**0.35**	**0.35**
LGCD-B-EL	**0.34**	**0.37**	**0.42**	**0.28**	**0.28**
LGCD-C-EL	**0.23**	**0.25**	**0.28**	**0.23**	**0.23**
LGCD-A-INEL	**0.64**	**0.69**	**0.80**	**0.52**	**0.52**
LGCD-B-INEL	**0.51**	**0.55**	**0.63**	**0.41**	**0.41**
LGCD-C-INEL	**0.34**	**0.37**	**0.42**	**0.34**	**0.34**

The research carried out shows that:

– the assessment of dynamic resistance, due to the frequency of the analysed phenomenon, is strictly dependent on the seismic characteristics of the area

– an identical building structure, analysed against a different spectrum curve, results in a different dynamic resistance (this is very important because it shows the difference from resistance to static loads, which is not dependent on frequency)

– in relation to the mining areas of LGCD and USCB, there is a similarity between the results determined according to the LGCD-2 and USCB curves
– resistance values determined according to curves: E-8-B, E-8-D, LGCD-1 and LGCD-C-EL, are similar
– resistance values determined according to curves: E-8-A, LGCD-2 and LGCD-B-EL are similar
– the dynamic resistance values determined when energy dissipation is allowed results in higher reserves of safety and, thus, higher values of allowable ground acceleration (LGCD-A-INEL, LGCD-B-INEL, LGCD-C-INEL)
– in most cases, the element determining the final value of dynamic resistance was the wall braces (except for the results obtained for the LGCD-A-INEL and LGCD-C-INEL spectrum curves, where apart from the wall braces, the main nave columns were a dimensioning element)

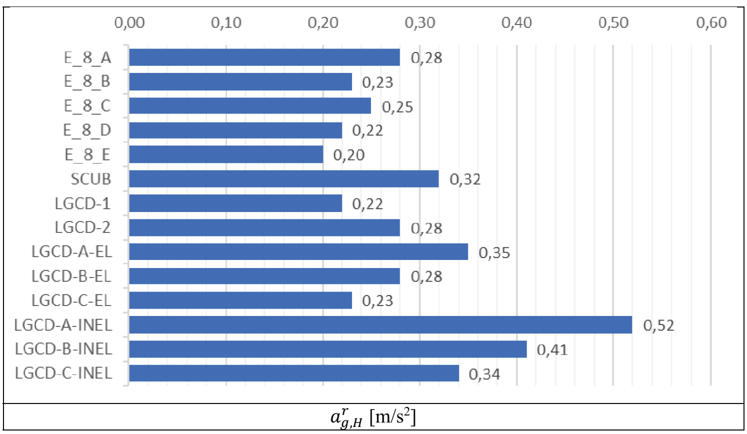

Fig. 6. Distribution of the values of dynamic resistance of the analysed structure

4. Conclusion

The study results presented in this work confirm the noticeable effect of the seismic intensity of a given area on determining the dynamic resistance of building structures. This is due to the adoption of the response spectrum method for the calculations, and thus the representative standard spectral curves, defining the seismicity of the area.

Although the subject of the study was an industrial hall structure, the relativity of assessing the resistance of existing structures to dynamic impacts caused by mining tremors applies to all types of structures. The obtained results are not representative for the entire hall building, because its diversity both in terms of geometry and material is too large to formulate general conclusions. Therefore, it is planned to build a database of dynamic resistance of this type of structures to the impacts of mining tremors. The analysis of such a database will allow for the generalisation of knowledge regarding the assessment of resistance of industrial hall structures and the sensitivity of individual structural components to dynamic excitation induced by mining tremors.

This type of approach was used by the authors in the case of bridge structures, where a database of dynamic resistance to mining tremors was created, which was analysed using machine learning methods [54,55]. This approach allows the combination of multiple FEM simulations with advanced statistical analysis methods. It also allows for Monte-Carlo simulations, in order to identify a wider spectrum of analysed cases.

From a utilitarian point of view, the applied methodology for the assessment of dynamic resistance may significantly increase the effectiveness of the assessment of existing structures for which there is no detailed information on the strength of steel or concrete, and the actual degree of reinforcement of reinforced concrete elements.

One has to be aware that such situations are common in the construction field. With a view to the development of BIM technology and maintenance management, the proposed methodology may contribute to a faster and more efficient update of data on existing buildings that relate to the development of mining areas affected by seismic activity.

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LIST OF FIGURES AND TABLES:

WYZNACZANIE ODPORNOŚCI DYNAMICZNEJ ISTNIEJĄCYCH HAL PRZEMYSŁOWYCH O KONSTRUKCJI STALOWEJ DLA OBSZARÓW O RÓŻNEJ AKTYWNOŚCI SEJSMICZNEJ

Streszczenie

W pracy przedstawiono wyniki badań dotyczących oceny odporności dynamicznej istniejących konstrukcji hal przemysłowych zlokalizowanych na terenach o różnej aktywności sejsmicznej. Podstawą do analiz była trójnawowa hala przemysłowa o konstrukcji stalowej. Przeprowadzono obliczenia numeryczne metodą elementów skończonych (MES), wykorzystując w analizie dynamicznej metodę spektrum odpowiedzi. Obliczenia przeprowadzono wariantowo, stosując wzorcowe przyspieszeniowe spektra odpowiedzi według Eurokodu 8 oraz te, wyznaczone dla obszaru Górnośląskiego Zagłębia Węglowego (GZW) i Legnicko-Głogowskiego Okręgu Miedziowego (LGOM). Stosując autorską procedurę oceny odporności dynamicznej, dla każdego analizowanego wymuszenia porównano reakcję konstrukcji na wzbudzenie dynamiczne z efektami od kombinacji obciążeń przyjętych na etapie projektowania, ustalając tym samym wartości graniczne projektowego przyspieszenia poziomego gruntu $a_{g,H}^{max}$ rozumianego jako odporność konstrukcji na wstrząsy. Pozwoliło to na ocenę wpływu warunków sejsmicznych z określonego obszaru na odporność dynamiczną badanego obiektu. W artykule przedstawiono sposób interpretacji zaproponowanej procedury i wskazano potencjalne możliwości jej zastosowania.

***Słowa kluczowe*:** odporność dynamiczna; analiza spectrum odpowiedzi; wstrząsy górnicze; stalowe hale przemysłowe

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