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PARASEISMIC RESISTANCE EVALUATION FOR EXISTING STEEL CONVEYOR BRIDGE SUBJECTED TO MINING TREMORS

The paper presents the author's approach to evaluating the dynamic resistance of existing building structures exposed to the action of paraseismic events. The idea of the approach was demonstrated in the example of an existing conveyor bridge, which is an important component of an industrial plant located in an area threatened by the occurrence of mining tremors. A scenario was analysed in which the object's structure was not adapted to absorb additional dynamic effects. Therefore, it was necessary to determine the load-bearing capacity reserve within which the dynamic effects induced by a mining tremor could be allowed. As part of the analysis, criteria for selecting the authoritative section of the analysed object for further dynamic calculations were established and described in detail. As a result of the implemented evaluation procedure, the limiting values of the ground acceleration components were obtained, which are understood as the resistance of the analysed object in the context of carrying additional dynamic actions induced by a tremor. The determined resistance is included in the ultimate limit state STR framework, which sets the level of strength of particular structures' components as a criterion. The limit values of the ground acceleration components were calibrated, taking into account other accompanying variable actions according to the Eurocodes guidelines. The study also justified using this approach and provides essential information about dynamic excitation's most sensitive structural components. Such information can direct the process of retrofit or necessary strengthening of the structure when the evaluated resistance will exceed the intensity of existing or predicted seismic events in the area.

Keywords: building structures; dynamic resistance; mining tremors; seismic event

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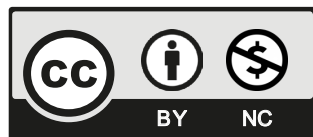
1. Introduction

Underground mining on the surface of the mining terrain results in static and dynamic impacts [1-4]. Static impacts cause additional strength to the load-bearing system, transferring the so-called continuous ground deformation strains onto the building structures [5-7]. They increase in time and usually cover a large area of terrain. The second type of negative impact is the so-called mining tremors [8,9]. They are dynamic and manifest on the ground surface in the form of ground vibrations induced by sudden relief of the rock mass. Due to the anthropogenic nature of such phenomena, mining tremors are treated as paraseismic tremors. As a result of the propagation of vibrations through the foundations in the building structure, additional inertia forces are generated, which cause an increase in the strength of its load-bearing structure members [4,10,11]. Both actions are taken into account at the design stage of new building structures, and the methods for analytical purposes are already sufficiently acknowledged and are the content of both European sector standards (Eurocodes) [12] and many dedicated guidelines and instructions of local scope [13-15].

The effects of mining tremors are similarly considered at the design stage as the effects of natural earthquakes [16-18]. However, due to their intensity and much more frequent occurrence, it is recommended to consider them exceptional loads [13]. Thus, the probability of an occurrence of a mining tremor with accompanying variable actions is higher than in the case of a seismic combination [12]. This, in turn, determines the resistance of existing structures and the effects of dynamic actions induced by mining tremors.

The problems of natural earthquake effects on building structures have been the subject of intensive scientific research for many years [19-21]. Generally, these studies are carried out in computational, laboratory and statistical terms. Analytical and computational studies verify the assumptions, models and calculation methods by means of which the response of a structure to a given seismic excitation is determined [22-24]. As a rule, laboratory research focuses on the mechanical simulation of dynamic actions and investigating their impact on the structure or its selected most sensitive components [25-28]. On the other hand, statistical research covers the analysis of consequences of the occurring seismic events and provides causal relations between the intensity of a given seismic event and the scale of observed damage [29,30]. This type of approach results from the damage risk models of building structures [31-33].

An analogous scope of research is also carried out in relation to issues concerning the impact of mining tremors on building structures. Here, however, numerical [4,10,34] and statistical research [17,35] is more prevalent.

The opposite situation refers to the existing building structures, whose load-bearing system was designed without taking into account the mining impact. Such structures still account for a significant part of development in mining areas where active underground mining is still in progress. In the further time horizon, the situation does not change even if the mines are closed, as the forecasted post-mining deformations may spread over a larger area than before, and the disturbance of the rock mass may manifest itself in the future in many more dynamic phenomena [42,43]. Thus, reliable assessment of the resistance of building structures to the impact of mining tremors will continue to be a serious problem of socio-economic nature related to ensuring safety and acceptable utility standards for building structures.

Apart from buildings, for which simplified methods of static and dynamic resistance assessment exist and are commonly used [35,44], in the group of endangered buildings, there are

also numerous cases of existing bridges, viaducts, flyovers, infrastructural networks and, among others, many industrial structures. Among industrial structures, numerous cases of conveyor bridges are very sensitive to static and dynamic mining impacts. Their role in the functioning of industrial plants is crucial, and difficulties related to their functioning pose a serious threat to the whole technological process.

Assessment of the dynamic resistance to mining tremors of bridge structures, including conveyor bridges, requires an individual computational approach and formulation of safety criteria each time. While the issue related to the numerical analysis of FEM (Finite Element Method) does not cause major problems nowadays, the formulation of resistance assessment criteria is a difficult issue. This difficulty is influenced by the fact that at the stage of establishing the resistance assessment criteria, the assumptions made at the design stage have to be taken into account and compared with the currently mandatory guidelines included in the Eurocodes [12,45]. It has to be considered that the design assumptions, including the statement of load combinations at the design stage, were governed by standards and regulations that are often already withdrawn. Therefore, every time an effort has to be made to reconstruct the design scenarios adopted for the design, which is often difficult when considering how the technical documentation is not properly archived. In the absence of data on the design assumptions, one may try to reconstruct them based on the standards used at the moment of design. This is particularly applicable in the case of special structures with a transparent static scheme. The group of building structures for which a reliable method of load assignment may be presumed include industrial halls, bridges, industrial chimneys or cold stores. Such an assumption makes it possible to determine the dynamic resistance assessment procedure for existing industrial structures, including conveyor bridges for which there is no complete information on design assumptions. It makes it possible to finally determine the load capacity margin within the scope of admissible strength of structural components resulting from the design stage, within which the dynamic impact of mining tremors may be allowed.

In the approach presented in this paper, the ultimate limit state STR [12] is analysed, which proves the safety of use for a particular structure in the context of the strength of its structural components. This type of approach has been verified so far for road viaducts [46,47] and industrial halls structures [10]. Despite the general assumptions of the presented approach, which are similar to those used up to now for industrial halls or road viaducts, the design of a conveyor bridge requires different assumptions for establishing the combination of normative loads assumed at the design stage and those occurring in the exceptional combination. The difference from typical bridge structures lies in the fact that there are no loads generated by car, train or pedestrian traffic. In addition, the wind, snow and temperature loads, as well as the combination factors assigned to them, will have to be revised given the wall enclosure of the conveyor belt itself.

A major difficulty in evaluating the dynamic resistance of existing structures is access to information on their strength characteristics and design assumptions. It is often caused by a lack of complete technical documentation, especially in the case of structures erected in the 70s of the previous century and earlier. Such circumstances enforce the assessor to verify the structure based on incomplete information. The methodology presented in this paper deals with just such situations. On the basis of many years of research, an approach has been established in which the basis for safety assessment of existing structures for additional dynamic excitation is the reference to the results of static calculations from the design stage. This can be regarded as an implicit (relative) approach in which the criteria indicating the possibility of additional distribution of

the mining shock loads is the maximum measure of the structure's strength, which was allowed during its design. Thus, the assumption that the decomposable exceptional combination, in which, according to [13], the effect of a mining tremor is included, cannot exceed the established levels of strength of individual structural components from the loads accepted at the design stage. Of course, in the case of complete technical documentation with an extract from static calculations, the dynamic resistance can be evaluated using ULS limit state criteria. This approach is treated as explicit (absolute). In such cases, however, it is necessary to have information on steel strength and assumed buckling characteristics for steel structures or on compressive and tensile strength of concrete, the actual degree of reinforcement of constricting elements and assumed rheological characteristics of concrete for reinforced concrete structures. The differences between these approaches and the required set of necessary material parameters needed for calculations for each of them are illustrated in Fig. 1.
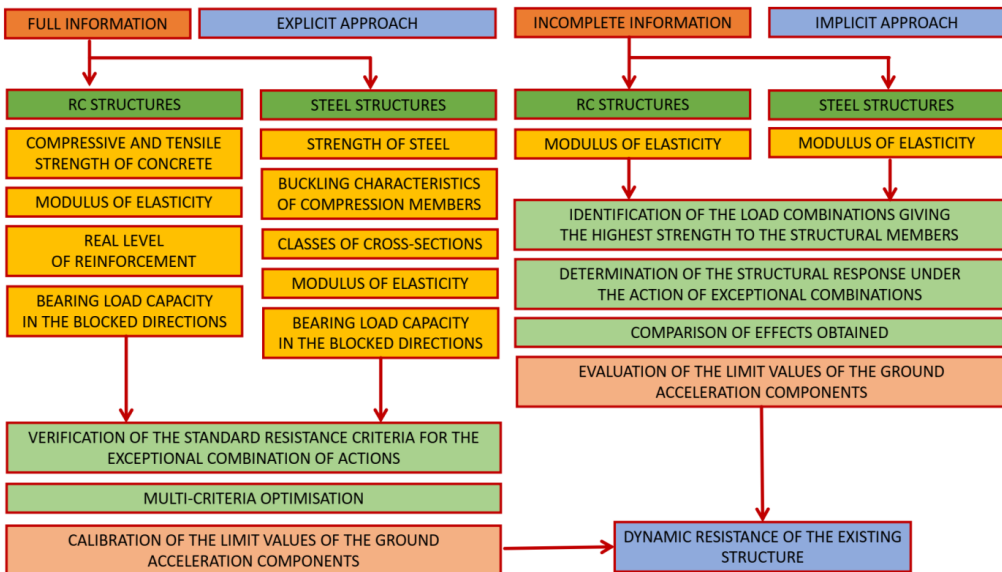

Fig. 1. Idea diagram to explain the adoption of implicit dynamic resistance assessment methodology

It should be noted that the author's dynamic resistance evaluation presented in this paper is an alternative to the methods used in seismic areas of the world [36-38]. This is because, as highlighted in the methodology chapter, this approach uses a relative measure derived from load information from the design stage. Therefore, it is in strict accordance with the combinations of actions adopted for ultimate limit state (ULS) verifications in terms of possible adjudication of safety risks of existing structures. On the contrary, the methods of determining the dynamic resistance used in seismically active areas use absolute measures as a criterion-related either to the displacement of the structure or to the strength of its load-bearing members without considering the influence of coexisting loads.

This study presents a procedure for assessing the resistance of the existing steel-structure conveyor bridge, a sensitive component of the mine's industrial plant complex. For this purpose,

after reviewing the structural design and assessing the technical condition of the bridge as part of the structural inventory, criteria have been formulated to determine the dynamic resistance of the subject structure. As part of the analyses, the intensity of the seismic phenomena that have occurred to date was also recognised. This made it possible to adopt standardised acceleration response spectra [39] that define a given terrain's seismic characteristics. The collected information made it possible to create a FE numerical model of the structure, simulate the static loads assumed at the design stage and carry out a dynamic analysis taking into account the seismic characteristics of the given terrain. All this made it possible to determine the permissible values of the ground acceleration components, understood as a measure of dynamic resistance, which the analysed structure can carry without safety hazards. The basis for the formulation of criterion conditions was the ultimate limit state STR (ULS – STR) dictated by the Eurocodes [12,45] and a collective work of the Building Research Institute [13] concerning the transfer of guidelines contained in the Eurocodes for design to the cases concerning the determination of load values in mining areas.

Having all the necessary information on the analysed object in disposal, additional calculations were performed to verify the load capacity of particular groups of components following the load capacity criterion for steel structures according to Eurocode 3. It consisted in generating the set of exceptional combinations taking into account not only the dynamic excitation but also the influence of accompanying actions (wind, temperature, snow). On this basis, as a result of multiple tuning of the dynamic excitation values, a set of acceptable values of the ground acceleration components was obtained. These provided a comparative basis for the results obtained by the implementation of the approach presented in this paper.

2. Overview of the applied methodology

Kinematic excitation of the building structure supports that are induced by a mining tremor causes the distribution of vibrations to the higher-lying elements of its load-bearing system. It results in the occurrence of additional mass inertia forces, which, in turn, contribute to the increase of the strength of individual elements or may cause the loss of their stability. Both of these phenomena may exceed the ultimate limit state of capacity determined by the strength, which according to European standards, is classified as STR.

The value of these forces depends on the seismic specifications of the terrain on which the analysed building structure is located. When evaluating the resistance of an existing object, the general seismic specifications of the site must be taken into account. Therefore, the influence of one strictly selected mining tremor is not analysed, but the predicted curves of standard response spectra are used. This determines the application of the Response Spectrum Method (RSM) in dynamic calculations. Most mining terrains have established seismic characteristics represented by well-defined, dedicated response spectrum standard curves. However, in case there is no specific terrain spectrum, based on geological data, a standard spectrum from among those in the European EC8 guidelines can be adopted for calculations.

The location of the additional inertia forces and their spatial orientation concerning the analysed building structure is crucial. For objects other than buildings, it may be necessary to consider the excitation in the horizontal plane as well as the vertical direction. This situation may occur within the framework of analysis of mutual interaction of dynamic excitations in the stability analysis of the whole structure system.

In this procedure, the issue of dynamic resistance of a structure to a given intensity of tremor requires each time formulation of an individual set of criteria, which allows for determining the acceptable strength of the structure in case of a seismic event. Finally, it determines the reserve of bearing capacity of the structure, resulting from the combination of loads taken into account at the design stage. Within this reserve, it is possible to allow for additional loading of the structure in an exceptional situation caused by a mining tremor with strictly defined values of the horizontal components of ground vibration acceleration (x, y, z). This procedure has been applied to many structure types, such as industrial halls with steel and reinforced concrete structures, road and railroad viaducts and bridges, industrial chimneys, culverts and power line masts. All the above-mentioned objects were characterised by high transparency of the structural system, thanks to which it was possible to confidently determine their static schemes and load sets assumed at the design stage.

In general, carrying out the assessment of dynamic resistance for a given structure requires:
– to create a numerical (or analytical) model being a representation of the static scheme of the analysed structure,
– determination of representative directions of dynamic excitation for the analysed structure,
– an indication of the most unfavourable effect of the dynamic excitation based on the analysis of spatial stiffness (vulnerability) of the analysed structure,
– to make assumptions about the static load combinations, analogous to those adopted at the design stage, acting in the plane (direction) consistent with the leading dynamic force,
– perform a static analysis for given load combinations,
– to create a numerical (analytical) model representing dynamic characteristics of the structure,
– to determine the set of eigenforms and frequencies,
– application of the response spectrum method in the plane (direction) of the leading dynamic effect,
– to compile the results of dynamic analysis in exceptional combination,
– identification of meaningful structural elements for further comparison,
– comparison of stress for selected (meaningful from the point of view of the character of dynamic excitation) structural elements for the comparison from the design stage and exceptional combination.

The results of analyses from the implementation of the above procedure for structures other than buildings were used, for example, in the paper [46].

2.1. Dynamic resistance evaluation criteria

The assessment of the dynamic resistance of the existing structure shall be determined on the basis of current guidelines at the time of such assessment. Current guidelines are the Eurocodes, which define ultimate limit states (ULS), dedicated to different forms of failure of the structural system, and serviceability limit states (SLS). With regard to structural safety, ultimate limit states (ULS) are considered, which are detailed in [12]:
– EQU: the limit state associated with the loss of static equilibrium of a structure or a rigid-body part of it, respectively,
– STR: limit state associated with internal collapse or excessive deformation of the structure (or its member), for which the strength of the structure's materials is decisive,

- GEO: limit state resulting from excessive strain or failure of the subsoil important for the load-bearing capacity of the structure or from deformation of the subsoil, which may significantly affect the structure's load-bearing capacity.

The procedure of dynamic resistance assessment of existing building structures described in this paper is based on general STR safety criterion analysis. It is dictated by the fact that in the case of existing building structures, the combinations dimensioned at the design stage were mostly related to the form of failure, which reflects the STR criterion. This, in turn, allows for qualitative and quantitative comparison of effects between the combination from the design stage and the exceptional combination taking into account mining tremors.

The basis for the formulation of the criteria for the presented approach is the general notation of the ultimate limit state condition of STR [12]:

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

Where:

E_d — design value of the effect of actions, such as internal force, moment, or a vector representing several internal forces or moments,

R_d — design value of the relevant load capacity.

Then, assuming it is valid for existing structures that the dimensioning effect of the individual components from the design stage can be considered as a quantitative measure of their actual bearing capacity, it can be written:

$$E_d^{Des} \approx R_d \tag{2}$$

Where: E_d^{Des} — effect of the individual components from the design stage

The above statement makes it possible to eliminate in the analyses the necessity to take into account detailed material data, which has been mentioned before. It comes to comparing only the effects, for which it is enough: to determine the static scheme of the structure, to determine the authoritative combinations for the design stage, to carry out the dynamic analysis and to create a set of exceptional combinations taking into account the mining tremors. Therefore, in order to determine the values of limit components of ground accelerations at the location of the analysed structure, it is only necessary to compare the effect in the structure from the loads from the design stage with the combination in which the influence of mining tremors was taken into account. In general form, this can be written as:

$$E_d^{Dyn}\left(a_{g,x}, a_{g,y}, a_{g,z}\right) = E_d^{Des} \rightarrow \left(a_{g,x}^{\lim}, a_{g,y}^{\lim}, a_{g,z}^{\lim}\right) \rightarrow \left(a_g^{\lim}, a_{gv}^{\lim}\right) \tag{3}$$

Where:

E_d^{Dyn} — design value of the influence effect from exceptional load combination including mining tremors,

E_d^{Des} — adequate (in terms of the response of the element under consideration) effect value from the combination of design stage loads.

In the following, the effects will be understood as extreme values of stresses and strains in the structural elements of the evaluated object.

Local guidelines have already been established to design new building structures in which mining impacts can be considered within the combinations dedicated by the Eurocodes. In the collaborative study presented in the article [13], based on the current guidelines in Eurocodes [12,45], loads generated by continuous and discontinuous mining deformations and tremors were adapted to the load combinations dictated by [12]. According to the arrangements contained in the papers [10,46], it became possible to computationally account for the influence of mining exploitation on a building structure under the STR ultimate limit state analysis.

The presented example demonstrated the influence of only dynamic loads generated by mining tremors. The additional influence of other actions of a static character, such as the mentioned continuous ground deformations, etc., was not taken into account. According to the guidelines stated in [13], this makes it possible to consider mining tremors as actions of exceptional character. For this reason and further calculations, the combination according to [12] used to check the ultimate limit states STR and GEO is applied. The components of this combination for all actions taken into account are summarised in Table 1. Adoption of this combination was the basis for further analysis in which the effect from the impact of an exceptional set of loads was calculated, taking into account the mining tremor E_d^{Dyn}.

TABLE 1

Design values to be taken for exceptional actions for checking STR/GEO states according to A1.2B [12]

Design situation	Permanent actions		Leading variable actions	Accompanying variable actions	
	Unfavourable	Favourable		Main	Others
Accidental (*)	$G_{kj,\,sup}$	$G_{kj,\,inf}$	A_d	$\psi_{1,1}$ or $\psi_{2,1} \cdot Q_{k,1}$	$\psi_{2,i} \cdot Q_{k,i}$

According to the main idea of the procedure, compared to the effects from the combination of the design stage with the influence of mining tremors is taken into account (3), it is necessary to establish a form representing the loads from the design stage. For the subject structure, it was determined that it was designed in accordance with the current guidelines given in the Eurocodes. However, the calculations did not include the dynamic effects of mining tremors. This made it possible to assume a fundamental combination for further analyses, taking into account the persistent and transient design situations. Then, among the selected combinations, based on the values of extreme normal stresses, the representative ones for each group of structural components were selected. According to the proposed approach, this combination was the basis for selecting the reserve of bearing capacity in the structure, allowing it to carry the additional load of mining tremors. The components of the fundamental combination for all actions taken into account are summarised in Table 2. On this basis, the analysis proceeded to determine meaningful values of impact effects for the combination from the design stage, which was denoted as E_d^{Des}.

The research presented in this paper was extended by the stage when the limiting components of ground accelerations were extracted using the explicit approach. With all the necessary information from the technical documentation, calculations were performed in which the limit value of the ground acceleration components was calibrated for each group of the structural components in order to finally reach the ultimate condition with respect to stability in compression or compression with bending (4÷6) [48].

TABLE 2

Design values of actions for checking the STR/GEO state in permanent or transient design situations [12]

Permanent and transient design situations	Permanent actions		Leading variable actions	Accompanying variable actions	
	Favourable	Fnfavourable		Dominant	Others
(Formula 2)	$\gamma_{G_{j,\sup}} \cdot G_{kj,\sup}$	$\gamma_{G_{j,\inf}} \cdot G_{kj,\sup}$	$\gamma_{Q,1} \cdot Q_{k,1}$	—	$\gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i}$
(Formula 3a)	$\gamma_{G_{j,\sup}} \cdot G_{kj,\sup}$	$\gamma_{G_{j,\inf}} \cdot G_{kj,\inf}$	—	$\gamma_{Q,1} \cdot \psi_{0,1} \cdot Q_{k,1}$	$\gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i}$
(Formula 3b)	$\xi \cdot \gamma_{G_{j,\sup}} \cdot G_{kj,\sup}$	$\xi \cdot \gamma_{G_{j,\inf}} \cdot G_{kj,\inf}$	$\gamma_{Q,1} \cdot Q_{k,1}$	—	$\gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i}$

where:

G_{kj} — characteristic value of j-th permanent action,
A_w — design value of exceptional action,
$\gamma_{Q,1}$ — partial safety factor for the dominant variable action,
$Q_{k,1}$ — characteristic value of the dominant variable action,
$\psi_{1,1}$ — coefficient for the frequency value of the dominant variable action,
$\psi_{2,1}$ — coefficient for the quasi-permanent value of the dominant variable action,
$\psi_{2,i}$ — coefficient for the quasi-permanent value of the i-th variable action,
$Q_{k,i}$ — characteristic value of the i-th accompanying variable action.

Buckling capacity condition for a steel element with constant cross-section

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1,0 \tag{4}$$

Interaction capacity conditions for steel elements loaded simultaneously by a compressive force and a bending moment

$$\frac{N_{Ed}}{\dfrac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\dfrac{M_{y,Rk}}{\chi_{LT} \gamma_{M1}}} + k_{yz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\dfrac{M_{z,Rk}}{\gamma_{M1}}} \leq 1,0 \tag{5}$$

$$\frac{N_{Ed}}{\dfrac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\dfrac{M_{y,Rk}}{\chi_{LT} \gamma_{M1}}} + k_{zz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\dfrac{M_{z,Rk}}{\gamma_{M1}}} \leq 1,0 \tag{6}$$

where:

$N_{Ed}, M_{y,Ed}, M_{z,Ed}$ — design values of internal forces
$N_{Rk}, M_{y,Rk}, M_{z,Rk}$ — characteristic values of the load bearing capacity of cross-sections,
$\Delta M_{y,Ed}, \Delta M_{z,Ed}$ — possible bending moments due to the displacement of the centroid Class 4 section,
$\chi_y, \chi_z, \chi_{LT}$ — characteristic values of the load bearing capacity of cross-sections,
$k_{yy}, k_{yz}, k_{zy}, k_{zz}$ — characteristic values of the load bearing capacity of cross-sections,
$N_{b,Rd}$ — design buckling load bearing capacity of a compression element.

The next section discusses a detailed workflow dedicated to evaluating the resistance of an existing and operating conveyor bridge in a coal mining power plant.

3. Structure characteristics of the analysed conveyor bridge

The analysed object is a conveyor bridge (with an inclination of about 1.5‰), a basic component of the transport line for the excavated deposit to the main complex of the mine's industrial plant (Figs 2 and 3). The bridge is a supporting structure for the conveyor belt. The multi-span gallery is divided into three-span segments (the maximum span length is 37 m). The route of the structure was divided in the plan into three segments of length (Fig. 4): 300.0 m – the north-western part, 2600.0 m – the central section and 500.0 m – the south-eastern part. Spans are supported at the ends by fixed spatial supports (Fig. 5) and indirectly by swinging supports (Fig. 6). The spatial steel truss structure of the spans is connected by cross beams and braced at the floor level by steel wires and a reinforced concrete ceiling. The casing of the span structure

Fig. 2. View of the central segment (2600 m) from the north-east side

Fig. 3. View of the segment near the industrial plant (300 m) from the south side
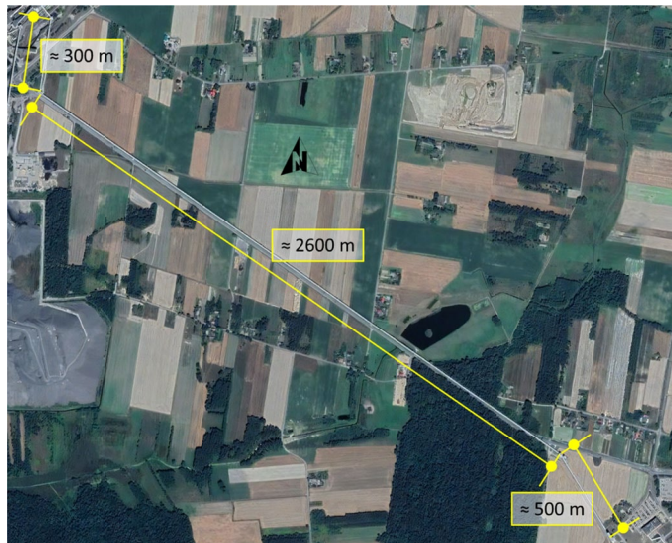

Fig. 4. Route of the conveyor bridge in the site plan

is made up of multilayer sandwich wall panels. Steel sections of the following types: HEA, HEB and IPE, were used to build the span structure and supports. The cross-sectional dimensions of the bridge span structure are a width of 4.20 m height of 3.25 m.

The expansion joints of the ceiling, roof and walls of the bridge span ensure free rotation on the articulated supports and free rotation and translation on the articulated-sliding supports. Two types of bearings were used: non-sliding spherical plain bearings in the vertical plane of the route with the possibility of limited horizontal rotation and sliding bearings allowing displacement and rotation in the vertical plane of the route as well as limited horizontal rotation. Two types of supports were used: swinging supports, which are planar supports braced in a plane perpendicular to the bridge axis, and spatial supports, which are braced in both planes. Both types of supports were made of steel HEA sections. The structure was founded using reinforced concrete footings except for the fixed supports, which were restrained around the perimeter and diagonally braced. The structure was designed for continuous deformations of the mining area, with the possibility of bridge support rectification within the range of 300÷900 mm.

To adapt the procedure of assessing the dynamic resistance, it was necessary to study its load-bearing structure in detail. This was done based on available technical documentation and "in-situ" inventory measurements. Fig. 7 shows the general structural scheme of the analysed bridge, including directions of the adopted reference system for further analyses (x, y, z). On the other hand, Fig. 8 shows detailed structural characteristics of the bridge's steel supports and the span. On the other hand, Fig. 9 presents the features of the static scheme divided into XZ and YZ planes.

Fig. 5. The structural system of a fixed spatial support

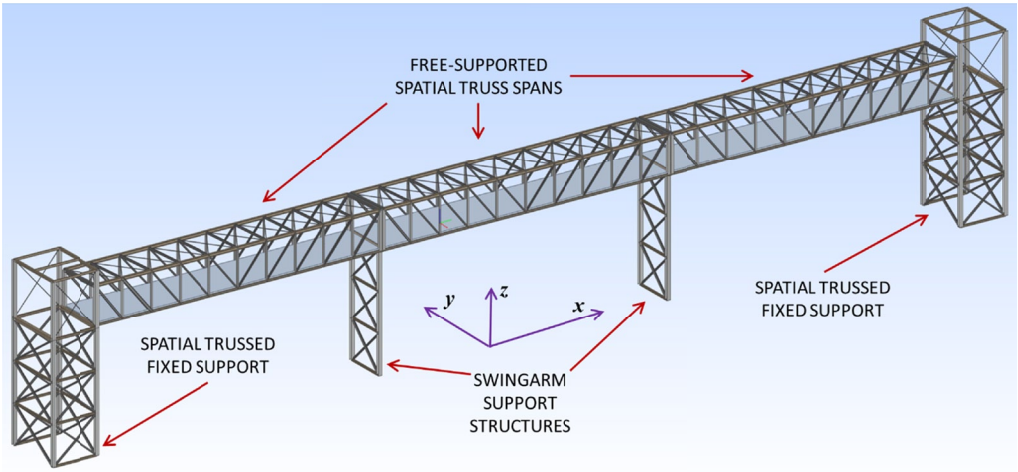Fig. 6. Swingarm support structure

Fig. 7. General structural scheme of the conveyor bridge being evaluated for dynamic resistance
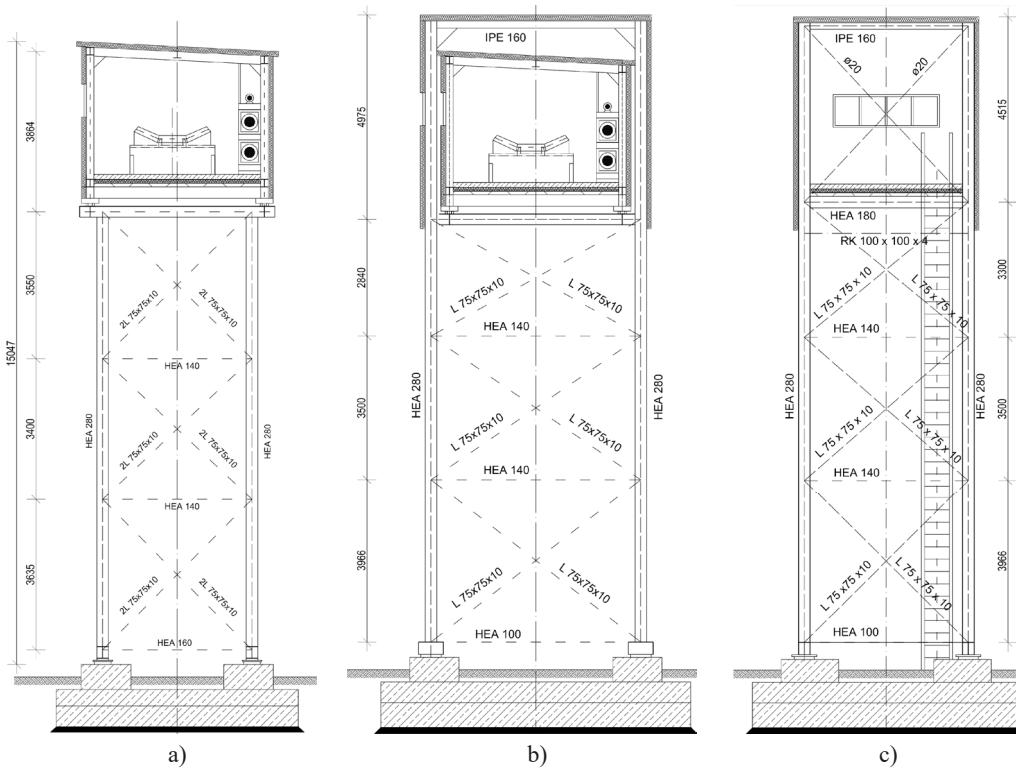

Fig. 8. A detailed structure of support members: a) intermediate swingarm support:
cross-section referencing the y axis, b) spatial trussed fixed support: cross-section referencing the y axis,
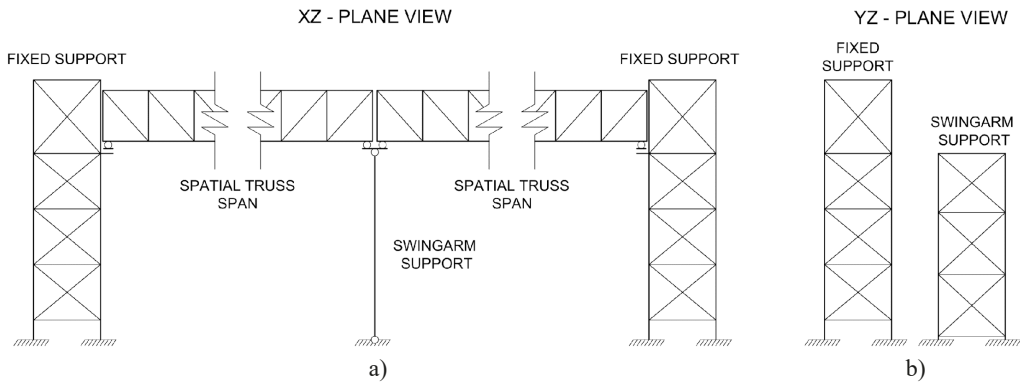c) spatial trussed fixed support: cross-section referencing the x axis

Fig. 9. Detailed presentation of static features of individual components of the spatial model
with division into planes: a) xz, b) yz,

4. Case study of the application of the proposed approach to determine the dynamic resistance of an existing conveyor bridge

The range of necessary calculations to determine the dynamic resistance of the analysed structure was divided into two stages. In the first stage, on the basis of dynamic characteristics and an assumed standard acceleration spectra, a meaningful segment of the conveyor bridge was selected for analysis. The assumptions made at this stage are described in chapter 4.1. The second stage was to adapt the proposed methodology to determine the limit values of component accelerations of ground motion. This stage is described in detail in chapter 4.2. Additional chapter 4.3 is dedicated to the comparison of the obtained results with the ones obtained for the explicit approach (cf. chapters 1 and 2).

4.1. Method for extracting a representative segment of the analysed conveyor bridge structure

Since the structure is multi-segmented (cf. Figs 2-4), in which spans of equal length are based on supports with height varying with the length of the structure, a meaningful selection of the segment for further dynamic calculations had to be made. In order to select a representative segment, a series of numerical FEM models were constructed, assuming the variation of bridge support heights. A set of the first significant eigenmodes was extracted. Typically, this was a set of the first ten eigenmodes and their corresponding frequencies (periods). The set of eigenmodes was divided in relation to the assumed dynamic excitation directions (x, y and z). The criterion allowing to indicate the dominant modes for particular directions was the value of modal masses in the analysis of natural vibration of the structural system.

In order to determine the response of the structure to dynamic excitation, the response spectrum method was applied [49]. For this purpose, the standard response spectrum for site category B, according to [39] (7), was adopted. This spectrum was used only for the dynamic

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excitation in the horizontal plane (H), which was used in determining the dynamic resistance for the (x, y) directions. Due to the necessity of considering vibrations also in the gravitational load plane (z), an additional response spectrum in the vertical plane (V) was adopted (3) according to the directives [39]. The curves of the adopted standard acceleration response spectra are illustrated in Fig. 10.

$$0s \leq T \leq T_B : S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (2.5 \cdot \eta - 1)\right] = a_g \cdot \beta^H(T) \; T_B \leq T$$

$$\leq T_C : S_e(T) = a_g \cdot S \cdot 2.5 \cdot \eta = a_g \cdot \beta^H(T) \; T_C \leq T \leq T_D : S_e(T)$$

$$= a_g \cdot S \cdot 2.5 \cdot \eta \left[\frac{T_C}{T}\right] = a_g \cdot \beta^H(T) \; T_D \leq T \leq 4s : S_e(T)$$

$$= a_g \cdot S \cdot 2.5 \cdot \eta \left[\frac{T_C \cdot T_D}{T^2}\right] = a_g \cdot \beta^H(T) \tag{7}$$

$$S = 1.2; \; T_B = 0.15; \; T_C = 0.5; \; T_D = 2.0$$

where:

a_g — acceleration of ground vibrations,

S, T_B, T_C, T_D — parameters for a specific soil category according to [39]

$$\eta = \sqrt{10/(5+\xi)} \geq 0,55$$

ξ — viscous damping factor (unless otherwise specified, equal to 5%)

$$0s \leq T \leq T_B : S_{Ve}(T) = a_{vg} \cdot \left[1 + \frac{T}{T_B} \cdot (3.0 \cdot \eta - 1)\right] = a_{vg} \cdot \beta^V(T) \; T_B \leq T$$

$$\leq T_C : S_{Ve}(T) = a_{vg} \cdot 3.0 \cdot \eta = a_{vg} \cdot \beta^V(T) \; T_C \leq T \leq T_D : S_{Ve}(T)$$

$$= a_{vg} \cdot 3.0 \cdot \eta \left[\frac{T_C}{T}\right] = a_{vg} \cdot \beta^V(T) T_D \leq T \leq 4s : S_{Ve}(T)$$

$$= a_{vg} \cdot 3.0 \cdot \eta \left[\frac{T_C \cdot T_D}{T^2}\right] = a_{vg} \cdot \beta^V(T) \tag{8}$$

$$\frac{a_{vg}}{a_g} = 0.45; \; T_B = 0.05; \; T_C = 0.15; \; T_D = 1.0$$

where:

a_{vg} — acceleration of ground vibrations in the vertical direction,

S, T_B, T_C, T_D — parameters for a specific type of spectrum curve

$$\eta = \sqrt{10/(5+\xi)} \geq 0,55$$

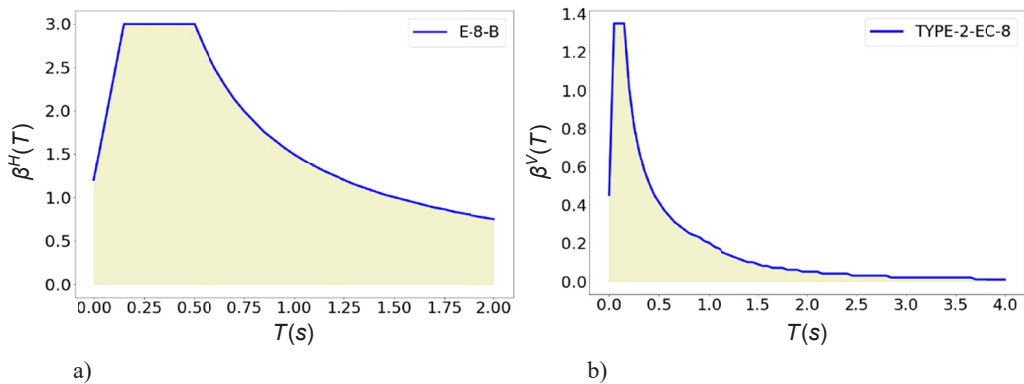ξ — viscous damping factor (unless otherwise specified, equal to 5%).

Fig. 10. Curves of standard acceleration response spectra adopted for analyses – a) curve of horizontal standard acceleration response spectrum for soil category B, – b) Curve of vertical standard acceleration response spectrum type 2

The response spectrum curve provides simplified information on the response of a single degree of dynamic freedom structure to dynamic excitation. The maximum accelerations of oscillators with one degree of dynamic freedom and different stiffness characteristics, are summarised in the frequency domain. In this way, a representation of the behaviour of the structure for different dynamic characteristics in periods or frequency domain is obtained. In the response spectrum analysis (RSA), on the other hand, the final vibration waveform of the system is composed of all of the selected forms of eigenmodes in varying proportions at the same time. To quantify the influence of the individual forms of eigenmodes generated for each segment of the structure, the method was applied based on counting the values of the spectrum curve for the set of calculated eigenmodes. In the next stage, for each segment of the analysed structure, according to the diagram illustrated in Fig. 11, the values of the spectra curves were determined for the periods of the preselected set of eigenmodes. This was based on previously separated vibration modes for the horizontal (H) and vertical (V) planes. Furthering the formula by the product of the values of the determined ordinates from the spectrum of individual periods of eigenmodes (at the determined division of vibration forms for individual excitation planes) gave the approximate value of the mass inertia force of the horizontal and vertical planes – Fig. 11. For this purpose, the product rule was used because it is sensitive to low values, which significantly affect the final result. By maximising the product of ordinates for the set of eigenmodes, simplified quantitative information was obtained for the response of the structure for the eigenmode periods in the individual segments. The set of these values, determined for each of the segments of the conveyor bridge under analysis, made it possible to indicate the segment with the most intense dynamic response for the assumed dynamic characteristics of the terrain in the form of adopted standard acceleration response spectra (7) and (8).

As a result of such analyses, the highest values of approximate mass inertia forces, both for horizontal and vertical planes, will be generated at dynamic excitation for the segment with the highest supports Figs 8 and 9.
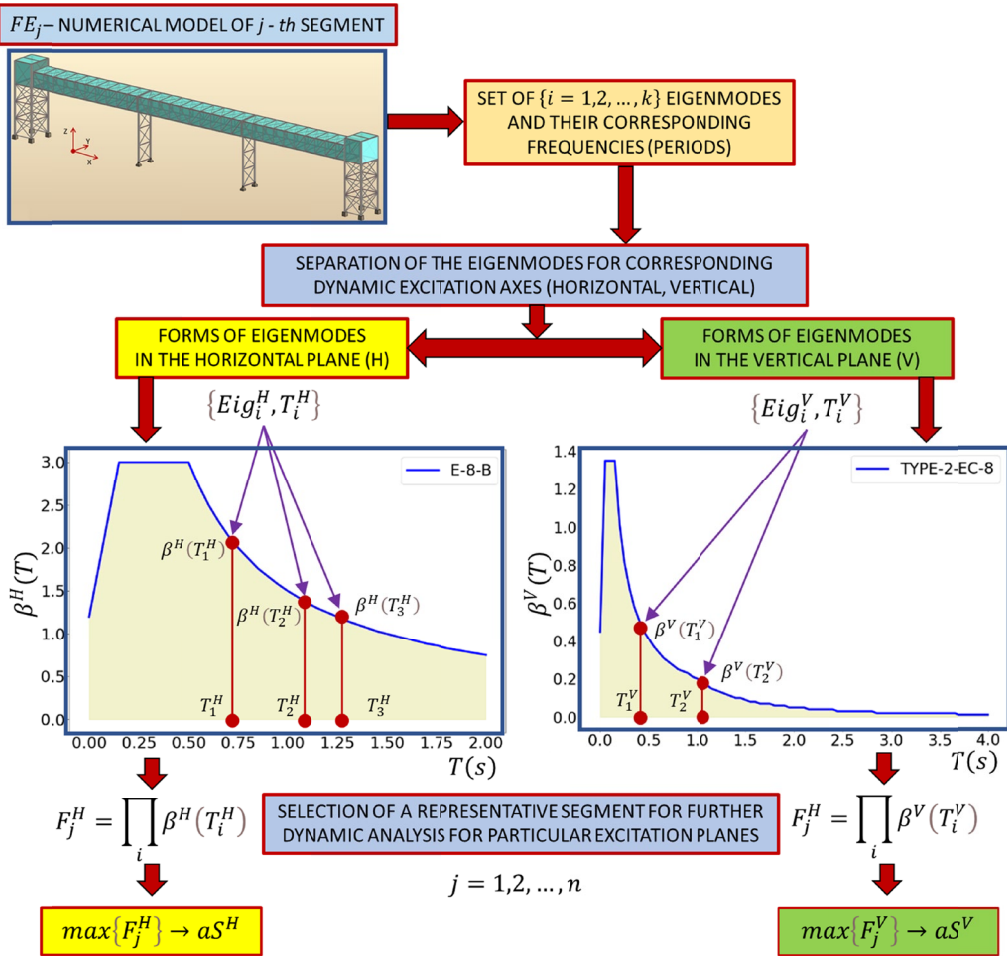
Fig. 11. Diagram of the adopted procedure for establishing criteria for selection of a representative segment of the analysed conveyor bridge structure: $\left(F_j^H, F_j^V\right)$ – approximated mass inertial force in the horizontal and vertical planes, $\left(aS^H, aS^V\right)$ – numerical models as representative for excitation in the horizontal and vertical plane

4.2. Detailed procedure description for determining dynamic resistance

After the selection of the authoritative segment of the bridge structure, the key stage of the procedure was proceeded, in which the permissible values of the ground vibration acceleration components $\left(a_{g,x}^{\lim}, a_{g,y}^{\lim}, a_{g,z}^{\lim}\right)$ were determined. For this purpose, a numerical model of the analysed segment was created, for which the set of the first ten eigenmodes was determined. These results are summarised in detail in Table 3. In addition, Fig. 12 shows an example of the first four eigenmodes for the created model.

Summary of characteristics of the determined eigenmodes and modal masses with respect
to the directions (x, y, z)

Eigenmode number	Frequency [Hz]	Period [sec]	Modal mass x direction [kg]	Modal mass y direction [kg]	Modal mass z direction [kg]
1	1.97	0.508	−14.0	13.03	198621.84
2	2.66	0.376	0.24	71060.96	121.55
3	3.72	0.269	0.02	78552.93	293.22
4	5.20	0.192	418.22	28.46	482.91
5	5.40	0.185	34.29	44.3	361.08
6	5.71	0.175	47.65	66.12	312.32
7	7.26	0.138	2.34	0	3144.88
8	7.28	0.137	−0.74	0	3147.72
9	7.38	0.136	0	0.34	3202.39
10	7.94	0.13	25684.46	669.81	442.48
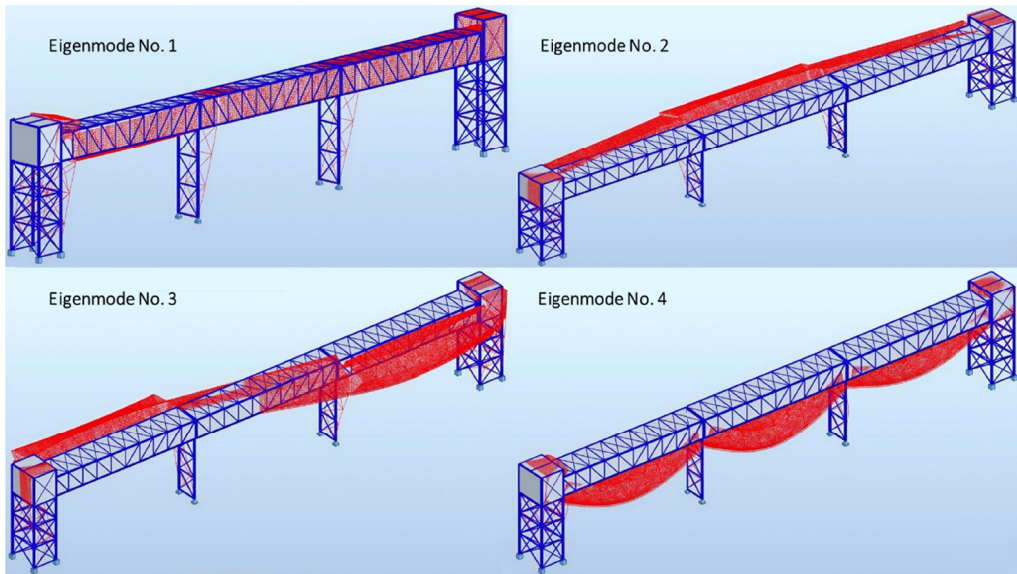
Fig. 12. Graphical representation of the first four eigenmodes for the conveyor bridge under analysis

Further from the design stage, the design effects from the load combination and the exception combination for the STR limit state were determined. For this purpose, the provisions of the standard [12] were adapted so that the respective design effects can be expressed:
- for the combination from the design stage:

$$E_d^{Des} = E\left(\sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \gamma_p P + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \right) \tag{9}$$

- for the exception combination with consideration of a mine tremor:

$$E_d^{Dyn} = E\left(\sum_{j\geq 1} G_{k,j} + P + A_w + \left(\psi_{1,1} \text{ or } \psi_{2,1}\right)Q_{k,1} + \sum_{i>1}\psi_{2,i}Q_{k,i}\right) \qquad (10)$$

In the next stage, representative load combinations from the design stage were listed for each analysed dynamic excitation direction (x, y and z). This finally allowed a comparison of the effects from the design stage load combinations and the exceptional combination, which included the effect of the mining tremor.

The effect of action in an exceptional situation caused by a mining tremor was checked for all component groups of structural elements. For this purpose, three directions of forcing were analysed along the assumed x, y and z axes. Therefore, it was necessary to establish an adequate set of impacts from the design stage. The determination of the authoritative combinations from the design stage was carried out by considering the set of all combinations according to [12] for the ultimate limit state STR. Following Eurocode 0, permanent and transient design situations were considered and summarised in Table 5. Finally, for each given group of structural elements (cf. Table 4), those combinations of actions were selected that caused the highest level of strength in their cross-sections. Depending on the purpose of a given group of elements, their performance was considered due to: compression, tension and compression with bidirectional bending effects. The determined combinations, together with assigning to the appropriate group of structural elements and specifying the considered loading condition, are summarised in the Table 5.

By having the established combination from the design stage, the variable interactions in the exceptional combinations were also taken into account. Due to the repeated occurrence of the same combinations of loads summarised in Table 5, they were reduced to a group of 6 independent combinations – Table 6. According to the scheme in Fig. 13, a set of unique exceptional combinations representative of particular directions of dynamic forcing, was established. In this
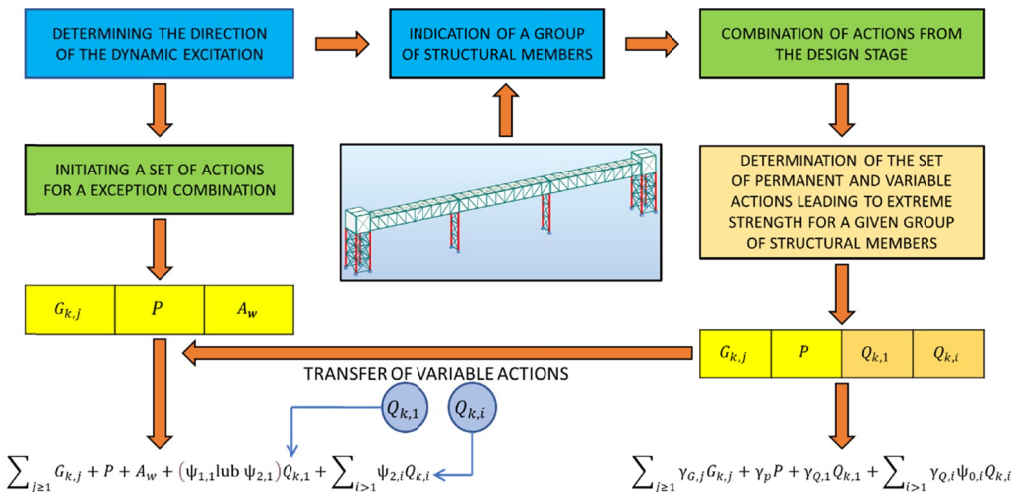

Fig. 13. Illustrative scheme for determining coherent load combinations for comparing exception and design situations

TABLE 4

List of all considered groups of structural members with their associated labels

Characteristics of the separated group of structural members	Assigned name for group
Supporting structure columns	Col
Upper chord of the truss girder	Ch_U
Lower chord of the truss girder	Ch_L
Support structure braces in the XZ-plane	Br_1
Support structure braces in the YZ-plane	Br_2
Pillars of span truss girder in the XZ plane	Trg_1
Cross-braces of span truss girder in the XZ plane	Trg_2
Lower and upper bracing along the truss girder in the XY plane	Br_3

TABLE 5

Summary of all authoritative load combinations from the design stage for each group of analysed structural elements together with their assigned structural performance characteristics [12,45,50-52]

| Group of structural elements and the type of their load performance in the structure under assessment | | Considered combinations of actions from the design stage | | | | | | | | | | | |
| | | Dead load | | | Wind action ($W^{(x)}$ or $W^{(y)}$) | | | Temperature action ($T^{(+)}$ or $T^{(-)}$) | | | Snow action | | |
		$\gamma_{G_{sup}}$	$\gamma_{G_{inf}}$	ξ	$\gamma_{Q,1}$	$\gamma_{Q,i}$	$\psi_{0,i}$	$\gamma_{Q,1}$	$\gamma_{Q,i}$	$\psi_{0,i}$	$\gamma_{Q,1}$	$\gamma_{Q,i}$	$\psi_{0,i}$
Col	Compression with bi-directional bending	1.35	—	—	—	1,5	0,75	—	1,5	0,6	—	1,5	0,2
Cho_U	compression	—	1,00	—	—	1,5	0,75	1,5	—	—	—	—	—
	tension	1,35	—	0,85	—	1,5	0,75	1,5	—	—	—	1,5	0,2
Cho_L	compression	1,35	—	0,85	—	1,5	0,75	1,5	—	—	—	1,5	0,2
	tension	—	1,00	—	—	1,5	0,75	1,5	—	—	—	—	—
Br_1	compression	1,35	—	0,85	—	1,5	0,75	1,5	—	—	—	1,5	0,2
	tension	—	1,00	—	1,5	—	—	—	1,5	0,6	—	—	—
Br_2	compression	1,35	—	0,85	—	1,5	0,75	1,5	—	—	—	1,5	0,2
	tension	—	1,00	—	—	1,5	0,75	1,5	—	—	—	—	—
Trg_1	compression	1,35	—	0,85	—	1,5	0,75	1,5	—	—	—	1,5	0,2
	tension	1,35	—	0,85	—	1,5	0,75	1,5	—	—	—	—	—
Trg_2	compression	—	1,00	—	—	1,5	0,75	1,5	—	—	—	—	—
	tension	1.35	—	—	—	1,5	0,75	—	1,5	0,6	—	1,5	0,2
Br_3	compression	—	1,00	—	—	1,5	0,75	1,5	—	—	—	—	—
	tension	—	1,00	—	—	1,5	0,75	1,5	—	—	—	—	—

set, similarly to the case of combinations from the design stage, the way of transfer of loads by elements of particular groups was taken into consideration. The final set of accompaniment load patterns was shown in Table 7. In turn, Table 8 shows the ready-made combination sets prepared from the design stage and the extracted exceptional combinations. The dynamic forces complementing the extracted patterns of accompanying loads to the full exceptional combination, were determined concerning the way the loads were transferred by the individual structural elements.

TABLE 6

List of 6 independent combinations from the design stage

Labels of the identified combinations from the design stage	Considered combinations of actions from the design stage											
	Dead load			Wind action $(W^{(x)}$ or $W^{(y)})$			Temperature action $(T^{(+)}$ or $T^{(-)})$			Snow action		
	$\gamma_{G_{sup}}$	$\gamma_{G_{inf}}$	ξ	$\gamma_{Q,1}$	$\gamma_{Q,i}$	$\psi_{0,i}$	$\gamma_{Q,1}$	$\gamma_{Q,i}$	$\psi_{0,i}$	$\gamma_{Q,1}$	$\gamma_{Q,i}$	$\psi_{0,i}$
I	1.35	—	—	—	1,5	0,75	—	1,5	0,6	—	1,5	0,2
II	—	1,00	—	—	1,5	0,75	1,5	—	—	—	—	—
III	1,35	—	0,85	—	1,5	0,75	1,5	—	—	—	1,5	0,2
IV	—	1,00	—	1,5	—	—	—	1,5	0,6	—	—	—
V	1,35	—	0,85	—	1,5	0,75	1,5	—	—	—	—	—
VI	1.35	—	—	—	1,5	0,75	—	1,5	0,6	—	1,5	0,2

TABLE 7

Summary of accompanying loads patterns for the exceptional combinations

Name of pattern	Accompanying load pattern									
	Dead load	Wind action $(W^{(x)}$ or $W^{(y)})$			Temperature action $(T^{(+)}$ or $T^{(-)})$			Snow action		
	$G_{kj,sup}/G_{kj,inf}$	$\psi_{1,1}$	$\psi_{2,1}$	$\psi_{2,i}$	$\psi_{1,1}$	$\psi_{2,1}$	$\psi_{2,i}$	$\psi_{1,1}$	$\psi_{2,1}$	$\psi_{2,i}$
A_1	1,00	0,5	—	—	—	—	0,5	—	—	0,2
A_2	1,00	—	—	—	0,6	—	—	—	—	0,2
A_3	1,00	—	—	—	—	—	0,5	0,5	—	—
A_4	1,00	—	—	—	0,6	—	—	—	—	—
A_5	1,00	0,5	—	—	—	—	0,5	—	—	—

TABLE 8

Summary of the authoritative combinations from the design stage and the corresponding exceptional combinations for each group of structural elements with respect to the dynamic excitation of the tremor

Labels of the element group and the considered state of their load		Combination labels from the design stage		Accompanying load pattern					Tremor load		
				A_1	A_2	A_3	A_4	A_5	a_x	a_y	a_z
Col	Compression with bi-directional bending	I	=	■	■	■			■	■	■
Cho_U	compression	II	=				■	■			■
	tension	III		■	■	■					■
Cho_L	compression	III	=	■	■	■					■
	tension	II					■	■			■
Br_1	compression	III	=	■	■	■				■	
	tension	IV					■	■	■		
Br_2	compression	III	=	■	■	■				■	
	tension	II					■	■		■	

TABLE 8. Continued

Trg$_1$	compression	III	=	■	■	■					■
	tension	V					■	■			■
Trg$_2$	compression	II	=			■	■				■
	tension	VI				■	■				■
Br$_3$	tension	II	=			■	■	■	■		■

Finally, the authoritative combinations from the design stage and the exceptional combinations, with consideration of mining tremors, were assigned for each group of structural elements. At this stage, the dynamic forces in the x, y and z directions were taken into account, concerning the load transfer behaviour of the individual members.

Having a fixed set of combinations from the design stage and the corresponding exceptional combinations (cf. Table 8), the final stage proceeded with the computational determination of the limit components of the ground acceleration. By referring, in turn, to particular groups of structural members, these combinations were the basis for determining the design effects E_d^{Des}, E_d^{Dyn}. In the present analysis, the extremal normal stresses in the analysed cross-sections were taken as representative and are summarised in Table 9 for all computational situations. Additionally, these values were distributed into the effect from the action of the combination from the design stage ($\sigma_{e,Des}^t$, $\sigma_{e,Des}^c$), the effect from all variable loads occurring in the exceptional combination ($\sigma_{e,Dyn}^t$, $\sigma_{e,Dyn}^c$) and the effect from the dynamic forcing in the given direction ($\sigma_{m,Res}^x$, $\sigma_{m,Res}^y$, $\sigma_{m,Res}^z$). To make the calculations more precise, the type of action was also taken into account, distinguishing between compression and tension.

TABLE 9

Extracted values of design effects as extreme normal stresses in cross-sections of analysed groups of structural members

Assigned labels name of a group of structural members		Obtained values of applied strength measures for particular groups of structural members						
		$\sigma_{e,Des}^c$	$\sigma_{e,Des}^t$	$\sigma_{e,Dyn}^c$	$\sigma_{e,Dyn}^t$	$\sigma_{m,Res}^x$	$\sigma_{m,Res}^y$	$\sigma_{m,Res}^z$
Col	Compression with bi-directional bending	189.24	—	129.80	—	47.55	—	—
		189.24	—	129.80	—	—	108.07	—
		189.24	—	129.80	—	—	—	54.04
Cho$_U$	Compression	104.20	—	82.34	—	—	—	78.70
	Tension		76.44	—	45.67	—		78.70
Cho$_L$	Compression	99.67	—	67.56	—	—	—	65.53
	Tension		63.50	—	14.49	—		65.53
Br$_1$	Compression	57,13	—	35,96	—	18,25	—	—
	Tension		63,12	—	16,50	18,25	—	—
Br$_2$	Compression	132,31	—	117,34	—	—	74.85	—
	Tension		87,57	—	46,70	—	74.85	—
Trg$_1$	Compression	72.19	—	44.22	—	—	—	15.12
	Tension	—	82.30	—	48.70			15.12

TABLE 9. Continued

		57.84	—	40.22	—	—	—	10.01
Trg$_2$	Compression	57.84	—	40.22	—	—	—	10.01
	Tension	—	51.59	—	32.56	—	—	10.20
Br$_3$	Tension	—	84.18	—	47.41	12.06	—	—
		—	78.16	—	57.41	—	9.30	—
		—	13.45	—	5.54	—	—	6.08

$\sigma_{e,Des}^{t}$ [MPa] – extreme tensile principal stresses for the considered variant of actions from the design stage,

$\sigma_{e,Des}^{c}$ [MPa] – extreme compression principal stresses for the considered variant of actions from the design stage,

$\sigma_{e,Dyn}^{t}$ [MPa] – extreme compressive principal stresses from accompanying actions for the considered variant in the exceptional combination (without considering the impact of the tremor),

$\sigma_{e,Dyn}^{c}$ [MPa] – extreme compressive principal stresses from accompanying actions for the considered variant in the exceptional combination (without considering the impact of the tremor),

$\sigma_{m,Res}^{x}$, $\sigma_{m,Res}^{y}$, $\sigma_{m,Res}^{z}$ [MPa] – maximum absolute value of normal stresses induced by the dynamic action of the shock on x, y and z directions.

The stress values given in Table 9 allowed the determination of the limiting values of the ground acceleration components for each of the considered planes. In order to do this, the superposition principle was applied to the effects from the design stage and the exceptional combination in the form:

$$E_d^{Dyn} = E\left(G_{kj,\sup} \text{ or } G_{kj,\inf}\right) + E\left(A_d\right) + E\left(\psi_{1,1} \text{ or } \psi_{2,1} \cdot Q_{k,1}\right) + E\left(\psi_{2,i} \cdot Q_{k,i}\right) \tag{11}$$

$$E_d^{Des} = E\left(\gamma_{G_{j,\sup}} \cdot G_{kj,\sup} \text{ or } \gamma_{G_{j,\inf}} \cdot G_{kj,\sup}\right) +$$
$$+ E\left(\gamma_{Q,1} \cdot Q_{k,1}\right) + E\left(\psi_{1,1} \text{ or } \psi_{2,1} \cdot Q_{k,1}\right) + E\left(\gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i}\right) \tag{12}$$

The above notation was made possible by assuming a linear response range of the structure, which was already taken into account at the stage of selecting the spectrum curves (7) and (8) and Fig. 10.

Assuming all the expressions used so far, i.e. (7) and (8), the effect from the tremor action in relation (11), divided into horizontal and vertical planes, was written as:

$$E\left(A_d\right)_H = E\left[S_e\left(T\right)\right] = E\left[a_g \cdot \beta^H\left(T\right)\right] = a_g \cdot E\left[\beta^H\left(T\right)\right] \tag{13}$$

$$E\left(A_d\right)_V = E\left[S_{ve}\left(T\right)\right] = E\left[a_{vg} \cdot \beta^V\left(T\right)\right] = a_{vg} \cdot E\left[\beta^V\left(T\right)\right] \tag{14}$$

Further, the criterion value was taken as the limit of the condition:

$$E_d^{Dyn} \leq E_d^{Des} \rightarrow E_{d,\lim}^{Dyn} = E_d^{Des} \tag{15}$$

This allowed the deriving of a relationship leading to the determination of the limiting values of ground accelerations in the form:

$$\left\{ a_g^{\lim}, a_{vg}^{\lim} \right\} \leftarrow | a_g$$

$$= \frac{E_d^{Des} - E\left(G_{kj,\sup} \text{ or } G_{kj,\inf}\right) - E\left(\psi_{1,1} \text{ or } \psi_{2,1} \cdot Q_{k,1}\right) - E\left(\psi_{2,i} \cdot Q_{k,i}\right)}{E\left[\beta^H(T)\right]} a_{vg}$$

$$= \frac{E_d^{Des} - E\left(G_{kj,\sup} \text{ or } G_{kj,\inf}\right) - E\left(\psi_{1,1} \text{ or } \psi_{2,1} \cdot Q_{k,1}\right) - E\left(\psi_{2,i} \cdot Q_{k,i}\right)}{E\left[\beta^V(T)\right]} \qquad (16)$$

After considering a determined set of combinations at a given dynamic excitation plane (Table 8) and substituting the assumed as the effect measure of extreme normal stresses (compressive and tensile – Table 9), as well as taking into account the sign variation of stresses being the effect of dynamic loading, the relation (3) can be reduced to the form (16).

$$\left\{ a_g^{\lim}, a_{vg}^{\lim} \right\} \leftarrow | a_g$$

$$= \min \left\{ \begin{array}{l} \min\left(\dfrac{\sigma_{e,Des}^c - \sigma_{e,Dyn}^c}{\sigma_{m,\mathrm{Re}s}^x}, \dfrac{\sigma_{e,Des}^c - \sigma_{e,Dyn}^c}{\sigma_{m,\mathrm{Re}s}^y} \right) \\[4mm] \min\left(\dfrac{\sigma_{e,Des}^t - \sigma_{e,Dyn}^t}{\sigma_{m,\mathrm{Re}s}^x}, \dfrac{\sigma_{e,Des}^t - \sigma_{e,Dyn}^t}{\sigma_{m,\mathrm{Re}s}^y} \right) \end{array} \right\} a_{vg}$$

$$= \min\left(\frac{\sigma_{e,Des}^c - \sigma_{e,Dyn}^c}{\sigma_{m,\mathrm{Re}s}^z}, \frac{\sigma_{e,Des}^t - \sigma_{e,Dyn}^t}{\sigma_{m,\mathrm{Re}s}^y} \right) \qquad (17)$$

Expression (17) was used directly to determine the limiting values of the ground acceleration components for particular groups of analysed components of the conveyor bridge. The results of these calculations are listed in Table 10.

When generalising, the following set of components of ground accelerations in the horizontal and vertical directions was considered the dynamic resistance, which was determined according to the relation (13) in the analysed case:

$$a_g^{\lim} = \min\left(a_{x,g}^{\lim}, a_{y,g}^{\lim} \right) = 0.20 \left[\frac{m}{s^2} \right], \quad a_{vg}^{\lim} = a_{z,g}^{\lim} = 0.28 \left[\frac{m}{s^2} \right] \qquad (18)$$

With the applied approach, this means that the strength level of individual elements (cf. Table 4) of the structure obtained for the leading combinations from the design stage will not be exceeded also in the case of the exceptional effect of a mining tremor. In this case, the difference of resistance for a given element obtained for the combination from the design stage and the separated exceptional combination is taken as the resistance reserve. All components of the ground vibration acceleration were calibrated in such a way that they would not exceed the ultimate limit state STR.

Computed limiting values of the ground acceleration components for particular groups of members of the analysed conveyor bridge

Labels of the element group and the considered state of their load		Obtained limit values of the ground acceleration components for particular groups of structural members		
		$a_{g,x}^{lim}$ [m/s²]	$a_{g,y}^{lim}$ [m/s²]	$a_{g,z}^{lim}$ [m/s²]
Col	Compression with bi-directional bending	1.25	0.55	1.10
Cho_U	compression	—	—	0.28
	tension	—	—	0.39
Cho_L	compression	—		0.49
	tension	—	—	0.75
Br_1	compression	1.16	—	—
	tension	2.55	—	—
Br_2	compression	—	0.20	—
	tension	—	0.55	—
Trg_1	compression	—	—	1.85
	tension	—	—	2.22
Trg_2	compression	—	—	1.76
	tension	—	—	1.87
Br_3	tension	3.05	2.23	1.30

4.3. Verification of the results with the dynamic resistance of the structure determined by the explicit method

As was mentioned in the introduction, to verify the applied approach, limit values of the ground acceleration components were also determined using the explicit approach. This consisted in conducting the verification of the bearing capacity of cross-sections of particular groups of structural elements to the final criterion of bearing capacity of steel sections due to compression and compression with bidirectional bending (4), (5), (6) [48]. For this purpose, a set of 55 exceptional combinations was generated automatically, in which the dynamic load caused by the tremor was taken into account in addition to the accompanying loads. The results were obtained by calibrating the values of dynamic excitation for particular directions (x, y and z) so that the given limit criterion was satisfied to the degree of unity.

In this sense, the results obtained represent the so-called true bearing capacity of the structure for dynamic excitation generated by a mining tremor (cf. Table 11). However, to carry out such calculations, it is necessary to know all the information concerning the design assumptions, strength characteristics of the materials, etc. Such circumstances, though, are difficult to achieve in the case of existing structures where due to limited information, it is possible to apply an approximate implicit approach, as presented in this paper.

The obtained results indicate that the actual resistance of individual structural elements determined by the explicit method is higher than the implicit approach proposed in the paper. Nevertheless, the example analysed in the paper confirms that the implicit method leads to the determination of dynamic resistance, which remains on the safe side.

TABLE 11

Calculated limiting values of the components of ground acceleration for individual groups
of members of the analysed structure obtained by the explicit method according
to the ultimate limit state STR according to [48]

Labels of the element group and the considered state of their load	Obtained limit values of the ground acceleration components for particular groups of structural members		
	$a_{g,x}^{lim}$ [m/s²]	$a_{g,y}^{lim}$ [m/s²]	$a_{g,z}^{lim}$ [m/s²]
Col	1.8	1.4	2.2
Cho$_U$	—	—	1.3
Cho$_L$	—	—	1.8
Br$_1$	1.35	—	—
Br$_2$	—	0.66	—
Trg$_1$	—	—	>3.0
Trg$_2$	—	—	>3.0
Br$_3$	>3.0	>3.0	>3.0

5. Summary and conclusion

The paper demonstrates the applicability of the author's methodology for evaluating dynamic resistance to the effects of mining tremors concerning the existing structure of a conveyor bridge. A developed methodology is an implicit approach concerning design assumptions. This procedure can provide a solution to the problem of determining the dynamic resistance of existing structures, for which there is insufficient information to carry out a full design according to the ultimate limit state assumptions of the Eurocodes. By treating the exceptional combination as acting separately from the other load combinations, a comparative criterion was defined to determine such a reserve of load-bearing capacity and adapt it to carrying additional forces generated by a mine tremor. The solution to this problem is presented in this paper.

Calculations were carried out as part of the research, having at the disposal the exemplary conveyor bridge object and a full set of information on its structure, strength characteristics of materials, etc. The verification of the conditions of bearing capacity of individual structural components was carried out [48]. It was shown that the proposed method leads to lower values of limit values of component accelerations of ground vibrations than the full (explicit) approach. It concludes the proposed method can be used in case of the necessity to assess the building structures for which there is no full information to verify the complete conditions of STR limit states. The fact that it leads to lower results concerning the explicit approach permits additionally to state that it gives the dynamic resistance of the structure being on the safe side.

In summary, the proposed methodology allows:
- assessment of dynamic resistance of a structure in case of incomplete information on its strength characteristics, which very often happens in case of necessity to assess existing objects,
- makes it possible to indicate the most sensitive elements of structure to dynamic excitation, which can be applied to increase the efficiency of repair and renovation planning. In the case studied, the elements most sensitive to dynamic excitations in the relevant

planes are the upper chord of the truss girder (zx plane) and the in-plane braces (yz plane) – cf. Table 10,

- as it results from the additional comparative analysis with calculations performed for the situation, in which the full information on strength parameters was available, the proposed method leads to the reduction of resistance characterised by the limit values of components of ground accelerations, which puts it on the safe side.

The ultimate limit state STR criterion was used as the base criterion in this study. As a part of planned research, it is foreseen to extend the spectrum of considered criteria by those resulting, for example, from the analysis of interacting soil or fatigue effects of multiple dynamic actions.

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