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LIMIT STATES OF STEEL SUPPORTING STRUCTURE FOR BRIDGE CRANES

Summary. This paper describes a question of evaluation necessity of bridge cranes using the method of limit deformation state and oscillation damping. The solution was performed by means of theoretical analysis and an experimental verification at the selected bridge crane. The final result sounds that in the case of a correct strength computing of given bridge crane, it is not necessary to also check deformation and damping of oscillation as well.

Keywords: oscillation, damping, energy, experiment, deflection, time

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1. INTRODUCTION

The questions concerning vibrations of the bridge cranes are analysed in many professional works from various authors. For example, vibrations of the crane girder, which are occurring as a consequence of the crane travel on the crane track, are described by the authors in the works [1-3].

Scientific investigation of crane oscillations, together with the presentation of possibilities on how to eliminate these oscillations using a suitable form of the crane control strategy, is presented in [4-7].

Other methods, which are determined for the elimination of the crane oscillations by optimisation of the crane control components and mechanisms, are introduced in [8-10].

Different proposals of a dynamic model relating to the abovementioned problems are given in [11-14] and the corresponding experimental methods are in [15, 16].

Another interesting approach to the investigation of mechanical vibration, damage of transport machine parts and generation of failures is presented in [17-27].

Other experimental studies dealing with the mechanical properties of a layered beam, which is partially treated with a damping element based on a granular material, offers [28].

The author of [29], is focused on the assessment of vibration control performance using enhanced smart constrained layer damping treatment with edge elements. The non-linear dynamics of a crane is investigated in the contribution [30].

The commonly used or standard approach to the solution of a damping process represents the application of the logarithmic decrement, which enables estimation of the damping ratio from a time history of the oscillation process.

Analysis of this method and optimisation of the parameters during the processing and evaluation of the results is presented in [31, 32].

A new algorithm OMI (Optimization in Multiple Intervals), which is intended for computation of the logarithmic decrement concerning the exponentially damped harmonic oscillations, is described as well as compared with the classic computational methods in the publications [33, 34].

According to these articles, it is possible to say that the OMI algorithm is proved to be the best solution in the computation of the logarithmic decrement and the resonant frequency for high damping levels. Moreover, it is possible to take into consideration, a typical causal relation between the wear process and wear damage of constructional parts, which is introduced for example in [35].

The existing STN 27 0103 [36] informs the design of steel crane structures according to two groups of limit states.

For the first group of limit states, which lead to the loss of loading capacity or the loss of position stability, the following criteria are central to the design of steel supporting structure:

- strength and stability,
- fatigue strength,
- position stability.

For the second group of limit states, the following criteria are crucial:

- static deformation (deflection, displacement, and twisting),
- dynamic structure response (steel structure frequency, amplitude, and damping).

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The second group of limit states is outlined in the Article X of the standard only in section 66, which states as follows: “Deformation and oscillation damping must not affect operational safety and must not interfere with proper crane function with regard to its work specifications”. In Annex VIII, the standard recommends how to approach the bridge crane evaluation.

To a great extent, the STN 27 0103 [36] overlaps with the German DIN standard 15018 “Krane. Grundsätze für Stahlwerke. Berechnung“ [37] in the calculation procedures. This DIN, however, does not specify checking for the limit state of deformation and damping.

This article lays out the theoretical analysis which consequently ties into the experimental analysis on a particular bridge crane.

2. OSCILLATION DAMPING OF A STEEL SUPPORTING STRUCTURE IN A BRIDGE CRANE

A degree of freedom in a flexible system is conceived as a number of independent coordinates which determine the location of all the system’s masses.

If the continuous distribution of the flexible elements is small in comparison with the masses distributed in the individual points, then the flexible element mass (m) can be downplayed and only the coordinates of the masses distributed in the points examined.

Let us propose that the resistance of the surroundings is directly proportional to the speed in the following relation:

$$F_{resist} = -k \cdot v, \text{ where } k > 0, \quad (1)$$

k – the damping coefficient ($\text{kg} \cdot \text{s}^{-1}$).

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The resistance force counters the speed which is expressed by the minus sign in this formulation. If the acting restoring force is directly proportional to the displacement, the equation of motion is as follows:

$$m \frac{d^2 x}{dt^2} = -c \cdot x - k \frac{dx}{dt}, \quad (2)$$

c – spring stiffness ($\text{N} \cdot \text{m}^{-1}$).

Once the substitution is applied:

$$\omega_0 = \sqrt{c/m} \quad \text{and} \quad 2b = k/m, \quad (3)$$

where b is the damping coefficient and the ω_0 constant is the natural angular frequency, that is, the angular frequency of the undamped harmonic oscillator.

After the adjustment, we get the following equation of motion:

$$\frac{d^2 x}{dt^2} + 2b \frac{dx}{dt} + \omega_0^2 x = 0. \quad (4)$$

The solution to the homogenous second order differential equation points to the following formulation:

$$x = e^{\lambda t}, \quad \dot{x} = \lambda \cdot e^{\lambda t}, \quad \ddot{x} = \lambda^2 \cdot e^{\lambda t}. \quad (5)$$

After we apply the function to Equation 4, we get the following characteristic equation:

$$\lambda^2 + 2b\lambda + \omega_0^2 = 0, \quad (6)$$

with this solution:

$$\lambda_{1,2} = -b \pm \sqrt{b^2 - \omega_0^2}. \quad (7)$$

The two λ values correspond to the general solution to Equation 4 in the linear combination:

$$x = C_1 e^{\lambda_1 t} + C_2 e^{\lambda_2 t} \quad (8)$$

provided that $\lambda_1 \neq \lambda_2$.

According to the extent of the damping, the following scenarios can play out:

- The damping is large and $b^2 - \omega_0^2 > 0$, then both solutions of the characteristic equation $\lambda_{1,2}$ are real numbers, and x has no periodical element. Theoretically, in time $t \rightarrow \infty$ the body gets back to the equilibrium position $x = 0$. If the damping is this large, the motion is called aperiodic. Oscillation does not occur at all.

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- The damping is such that $b^2 - \omega_0^2 = 0$. In such a case, mathematics informs the solution to Equation 4 as the function $x = e^{-bt}$ as well as the function $x = t \cdot e^{-bt}$. The general solution is their linear combination in the following form:

$$x = e^{-bt}(C_1 + C_2 \cdot t). \quad (9)$$

This motion is called critical aperiodic motion.

- Damped oscillatory motion occurs only when damping is small, if $b^2 - \omega_0^2 < 0$. Then:

$$\sqrt{b^2 - \omega_0^2} = \pm i \cdot \omega \quad \text{where} \quad \omega = \sqrt{\omega_0^2 - b^2}. \quad (10)$$

The general solution to the equation of motion is expressed as follows:

$$x = e^{-bt}(C_1 \cdot e^{i\omega t} + C_2 \cdot e^{-i\omega t}). \quad (11)$$

Along the lines of this procedure similar to the harmonic oscillator case, substitution and adjustment bring forward the real formulation of the general solution:

$$x = A_0 e^{-bt} \cos(\omega t + \varphi). \quad (12)$$

The angular frequency ω is smaller than the angular frequency at the undamped oscillation of the same system, and the amplitude which also changes, exponentially decays away over time:

$$A = A_0 e^{-bt}. \quad (13)$$

Damped oscillatory motion cannot be considered periodic because the oscillating point does not reach its original displacement. The motion here is quasiperiodic, and the T period can only be conceived as a time interval, past which a mass point passes the equilibrium position.

The damped oscillation period is:

$$T = \frac{2\pi}{\omega} = \frac{2\pi}{\sqrt{\omega_0^2 - b^2}}. \quad (14)$$

It is true that $T > T_0$, where T_0 is a period of natural oscillations. If the damping is small, the period practically equals the period of undamped oscillations. The period increases with growing damping.

The ratio of amplitudes of two consequent maximum displacements are denoted as λ and called damping.

$$\lambda = \frac{A(t)}{A(t+T)} = \frac{A_0 e^{-bt}}{A_0 e^{-b(t+T)}} = e^{bT}. \quad (15)$$

The natural damping logarithm is the logarithmic damping decrement δ

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$$\delta = \ln \lambda = bT. \quad (16)$$

From the dependence of amplitude on time (13), it is apparent that the oscillation amplitude decreases e -times over a time interval which equals $1/b$. The inverted value of the logarithmic damping decrement expresses the number of oscillations, during which the oscillation amplitude changes e -times. The greater the logarithmic damping decrement, the fewer the number of oscillations necessary for a particular decrease of the amplitude.

The total mechanical energy of the oscillating oscillator is proportional to the square of the amplitude. If the energy of the oscillator with damping in the point in time $t = 0$ equalled E_0 , then the mechanical energy of the oscillator decreases with increasing time according to the equation:

$$E = E_0 e^{-2bt}. \quad (17)$$

Friction causes energy dissipation; mechanical energy of the oscillatory motion changes to thermal energy and the motion gradually decays. For motion to be maintained in the oscillating system, then energy must be supplied to it in a suitable way. Experiments confirm that for most mechanical materials, the value of the dissipated energy during a single oscillation cycle does not depend on frequency but that it is just the function of the oscillation

amplitude. On the other hand, it is damping that is used in technical practices to eliminate undesirable vibrations.

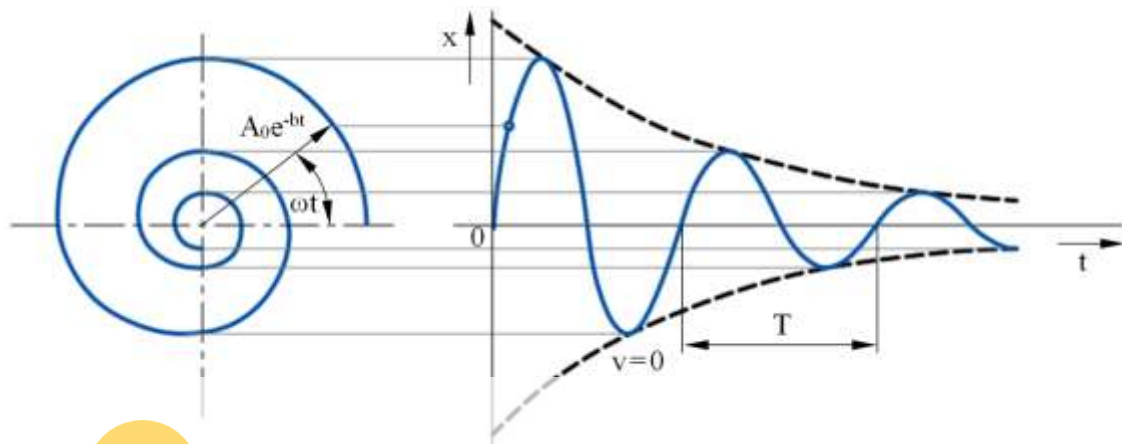


Fig. 1. Dependence of amplitude on time at damped oscillation

3. COMPUTER AND EXPERIMENTAL APPLICATION ON THE REAL BRIDGE CRANE

Basic data about the electrical bridge crane under examination:

| | |
|-------------------------|----------------------------------|
| Capacity | $Q = 500,000 \text{ N}$ |
| Span of the crane | $L = 28,200 \text{ mm}$ |
| Crane travel speed | $v_{travel} = 0.416 \text{ m/s}$ |
| Lifting speed | $v_{lift} = 0.0555 \text{ m/s}$ |
| Trolley mass | $m_{trolley} = 6,680 \text{ kg}$ |
| Main girder gravity | $G_1 = 294,800 \text{ N}$ |
| Main cross beam gravity | $G_2 = 41,240 \text{ N}$ |

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The calculated static values of the cross-section for the main girder without stiffeners and the rails are illustrated in Fig. 2.

3.1. Calculating damping for the steel supporting crane structure

According to STN 27 0103 titled “Design of crane steel structures” – Calculation by limit states, Article X – Steel supporting structure calculation based on the II. group of the serviceability limit states – deformation: deformation and oscillation damping must not affect operational safety and must not interfere with proper crane function in regard to its work specifications”. The recommended approach to the bridge crane evaluation is outlined in Annex VIII of the referenced standard.

The new shape of the girder caused by deformation is not detrimental to the operation if the deflections are smaller than the recommended values outlined in [28] and are caused by a random nominal loading, which includes as follows:

- rated loading,
- constant loading,

- loading brought about by natural mass, and the movable parts, which move in relation to the movement of the load.

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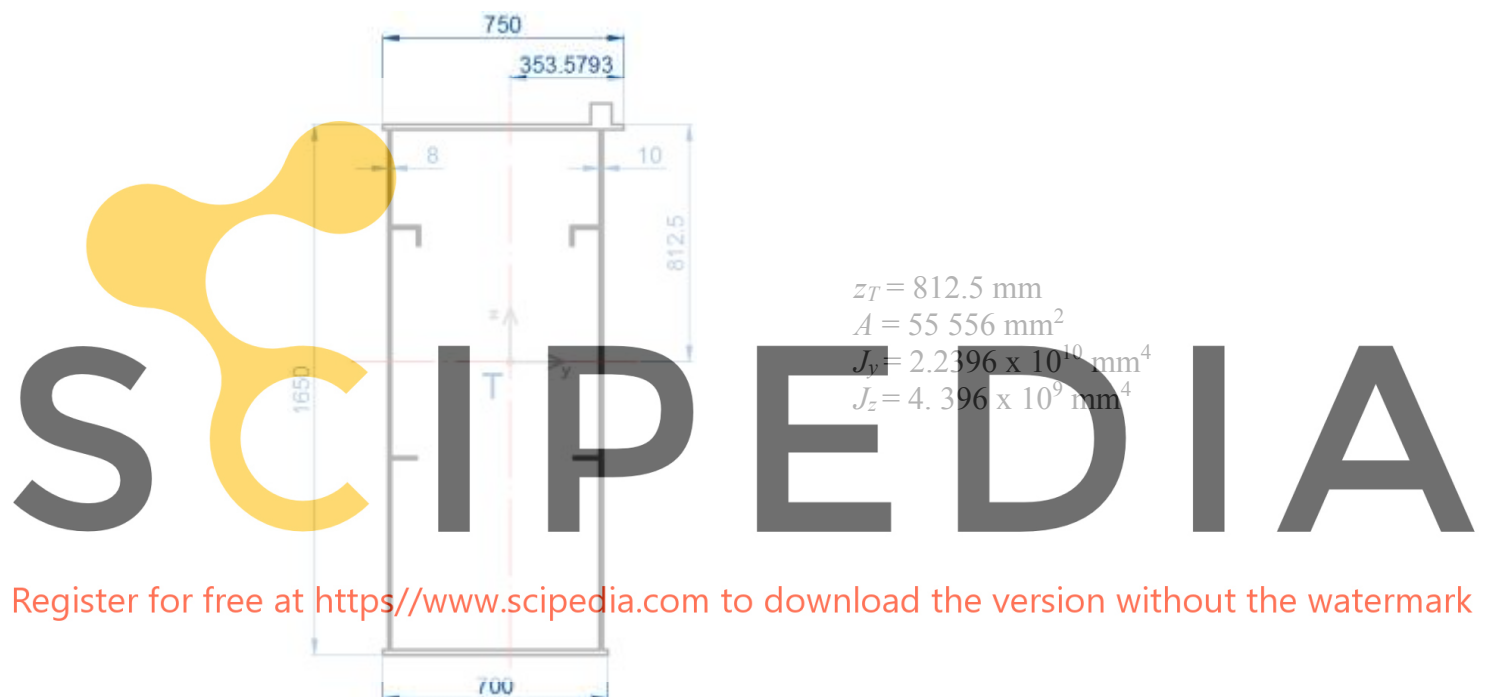


Fig. 2. Cross-section view of the main girder in the bridge crane

For the electrical bridge crane (50t x 28.2 m) with two main girders, along the top of which the trolley travels, the ratio between the maximum deflection from random nominal loading in relation to length, based on [37] can reach $L/700$ at most, that is, maximum deflection can reach:

$$w_{max} = \frac{L}{700} = \frac{28200}{700} = 40.28 \text{ mm} \quad (18)$$

Such a deflection is not detrimental to the crane operation. A steel supporting structure ought to meet the conditions of damping while oscillating. For cranes with box girders, it is recommended that the amplitude, after the nominal load of the oscillating bridge is set aside, sank in the middle of the bridge within 15 s to 0.5 mm at most. The damping time for a single-mass substitution system is determined by the following relation:

$$t_{ti} = \frac{\ln 2 w_{st}}{f \cdot \delta} \quad (19)$$

$$f = \frac{1}{2\pi} \sqrt{\frac{c_0}{m_{red}}} \quad (20)$$

where

- w_{st} is the maximum deflection of the girder from rated loading in (mm),
 f is the frequency of natural oscillations in the girder (s^{-1}),
 δ is the logarithmic decrement of oscillation damping which depends on the ratio between the girder height and length.

For the 50t x 28.2 m crane, the bridge girder height is $h = 1650$ mm (Fig. 2), hence, the ratio $h/L = 1650/28200 = 0.0585$.

Welded plate box girders are braced with a compression boom, with a height to length ratio greater than 1:20. Based on [36], the logarithmic damping decrement $\delta = 1$.

A girder spring constant can be determined by the following relation:

$$c_0 = \frac{48E J_y}{L^3} \times 10^3 \quad (Nm^{-1}) \quad (21)$$

where

$$E = 2.1 \times 10^5 \text{ MPa}$$

$$L = 28\,200 \text{ mm}$$

$$J_y = 2.2396 \times 10^{10} \text{ mm}^4$$

is the elastic modulus,

is the bridge length,

is the axial quadratic cross-sectional moment in relation to the neutral axis.

$$\text{Then } c_0 = 10066619.79 \text{ Nm}^{-1}.$$

The reduced mass of the girder, trolley, and all mass oscillating with the girder, once the

rated load is set aside, is determined by the following relation:

$$m_{red} = \frac{qL}{2} + \frac{m_k}{i} \quad (kg) \quad (22)$$

where

q – girder mass per length ($kg \cdot mm^{-1}$),

i – number of girders along which the trolley travels,

$m_{trolley}$ – the trolley's natural mass without load, including the mass which the trolley retains once the load is set aside.

To determine the reduced mass of the girders, the gravity of the individual crane components was relayed from the static calculation of the equipment, according to which the girder mass where there are no crane trolley units:

$$m_1 = \frac{294800}{2} \cdot \frac{1}{9.81} = 15025.5 \quad (kg) \quad (23)$$

The second girder's mass includes the mass of the crane travel units, of the switchgear, of the walkway, and of the trolley wire (relayed from the static calculation):

$$m_2 = \left(\frac{294800}{2} + 7840 + 3500 + 20110 + 1550 \right) \cdot \frac{1}{9.81} = 18399.4 \quad (kg) \quad (24)$$

If we consider that the drawing documentation for the trolley mass $m_{trolley} = 6680$ kg, then the reduced mass of the girder without the travel units and the walkway is:

$$m_{red}(1) = \frac{m_i}{2} + \frac{m_k}{2} = 10852.75 \quad (\text{kg}) \quad (25)$$

and for the girder with the travel units:

$$m_{red}(2) = \frac{m_2}{2} + \frac{m_k}{2} = 12539.7 \quad (\text{kg}) . \quad (26)$$

The relationships (20) and (21) are used to determine natural frequencies of the respective beams $f_1 = 4.847$ Hz, $f_2 = 3.142$ Hz.

Damping period of the respective girders established from the value of the maximum static deformation from the rated load is as follows:

$$w_{st} = \frac{Q \cdot L^3}{48 E J_y} = \frac{25000 \cdot 9,81 \cdot 28200^3}{48 \cdot 2,1 \cdot 10^5 \cdot 2,2396 \cdot 10^{10}} = 24.36 \quad (\text{mm}) . \quad (27)$$

The damping period for the first and the second girder equals:

$$t_{tl}^1 = \frac{\ln 2 \cdot 24.36}{4.847 \times 0,1} = 8.02 \text{ s} , \quad t_{tl}^2 = \frac{\ln 2 \cdot 24 \times 36}{3.142 \times 0,1} = 12.36 \text{ s} . \quad (28)$$

Because the damping period in both cases is shorter than 15 s, the steel supporting crane structure meets the conditions for the oscillation damping according to [36].

3.2. Experimental evaluation of damping in a steel supporting crane structure

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Fig. 3 shows the strain-gauge measurement that was proposed and made on the electrical bridge crane with box girders. The proposed methodology enables us to determine the stresses and to identify the extent of the deflection caused by the suspended load, the amplitude of the oscillating bridge in the middle of its span after unloading or lifting the load, and the time of amplitude damping required for reaching a 0.5 mm oscillation. Furthermore, Fig. 3 shows where the sensors were applied in a diagram. Strain-gauge sensors were placed on the edges of the top beam flange along the axis of the bridge symmetry. To measure, we used HBM strain-gauge sensors, bonding cement (X60), measuring apparatus (Spider8) and the evaluation software (Catman) also from HBM.

Strain-gauge measurement was made under the following loading:

- loading of the crane with a trolley without load,
- loading with a 48,000 kg load.

The measuring apparatus was balanced in the empty trolley mode in the middle of the bridge length. The measured incremental values of relative deformation were used by the Catman software to evaluate and visualise time changes of normal stress increments in the points illustrated in Fig. 3.

The measured time sequences of the relative deformation were used to calculate normal stresses in the points of the single-axis stress of the main girder.

Fig. 4 illustrates the time changes of incremental stress during intermittent lifting of the 48,000 kg load in the points of measurements 1, 2, 3 and 4.

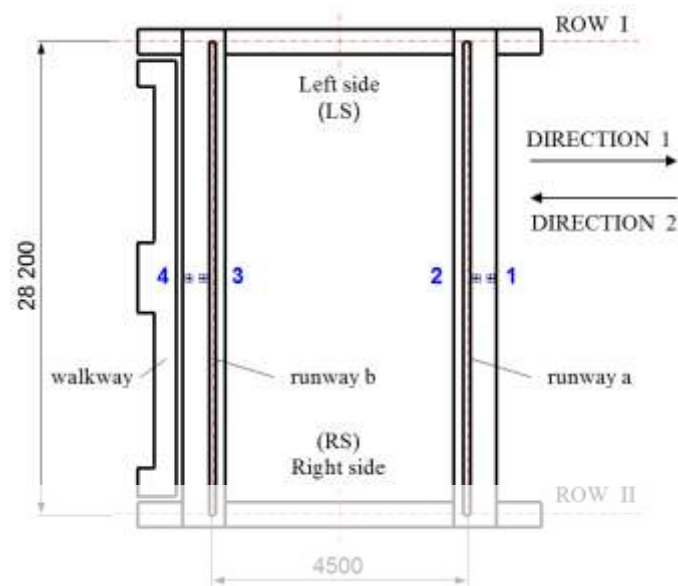


Fig. 3. Sensors in position on the crane

Because the load consisted of long slabs, which could touch the ground at the start of the lift, the dynamic behaviour was different from the repeated start of lifting if the load is suspended on a wire rope.

A detailed examination of the time sequences suggests that the extent of the oscillation amplitude at repeated lifts changes with the length of the wound wire rope.

The measurement mode ends when the load is lowered, during which the process of girder completing the oscillation does not align with the theory of damped oscillation.

It is caused by the fact that the load touched the ground again with some of its ends, and only then the load was completely lowered onto the ground.

Damping is also affected by the suppleness of the wire rope, which significantly reduces the time of damping to the required amplitude.

With regard to the evaluation of the oscillation amplitude and the damping time to the permissible 0.5 mm oscillation range amplitude specified in [36], it is necessary to transform the stress time sequence in the girders as shown in Fig. 4 into deforming the structure, that is, into deflection time sequences.

Due to the small 1,390 mm trolley wheelbase and the 28,200 mm bridge length, concentration of load mass into a point in the middle of the bridge length.

Deflection in the middle of the girder w :

$$w = \frac{F \cdot L^3}{48 E \cdot J_y} = \frac{L^2}{12 \cdot z} \cdot \varepsilon = \frac{L^2 \cdot \sigma_0}{12 \cdot E \cdot z} \quad (29)$$

where:

F – weight of half a load $Q = 470,880$ N,

L – bridge length 28,200 mm,

z – distance from the neutral axis to the top beam flange,

ε – measured relative deformation in the point where the strain-gauge sensor is located,

E – elastic modulus for the girder material,

σ_0 – stress in the point where the strain-gauge sensor is located is equal.

$$\sigma_0 = \frac{F.L}{4 E.J_y} \cdot z \quad (\text{MPa}). \quad (30)$$

Fig. 5 shows the time sequence of the deflection in the interval between 26 and 91 s for the sensor no. 2 in the time sequence of the crane girder deflections brought about by the 48,000 kg load established from the measured values in the measurement points 1, 2, 3 and 4 corresponding to the stress increments.

The relation (29) considers the cross-section according to Fig. 2, where the cross-sectional characteristic similarly considers the rail, the top and the bottom longitudinal stiffening.

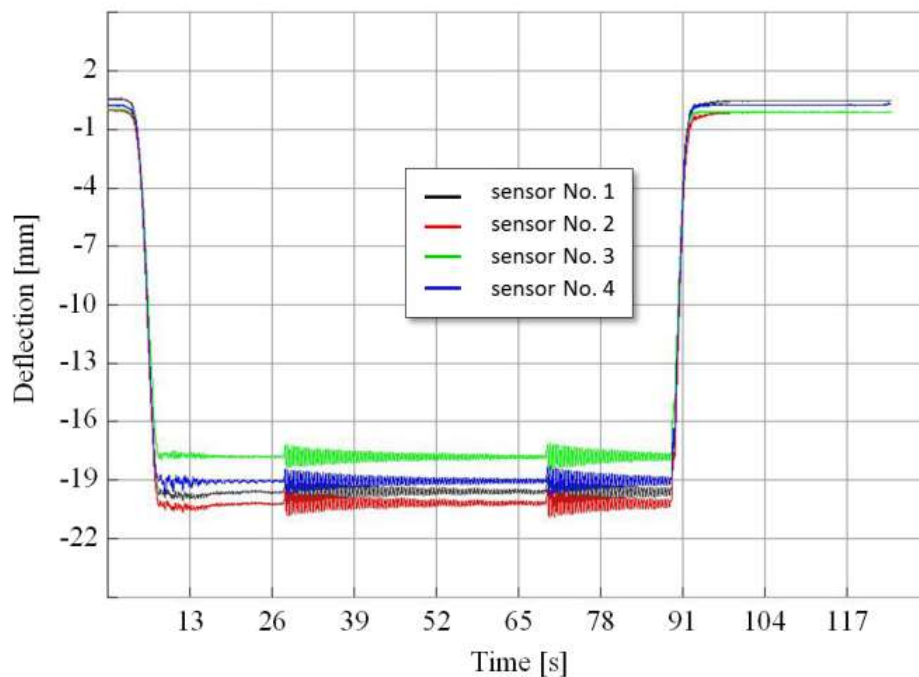


Fig. 4. Time dependence of deflections

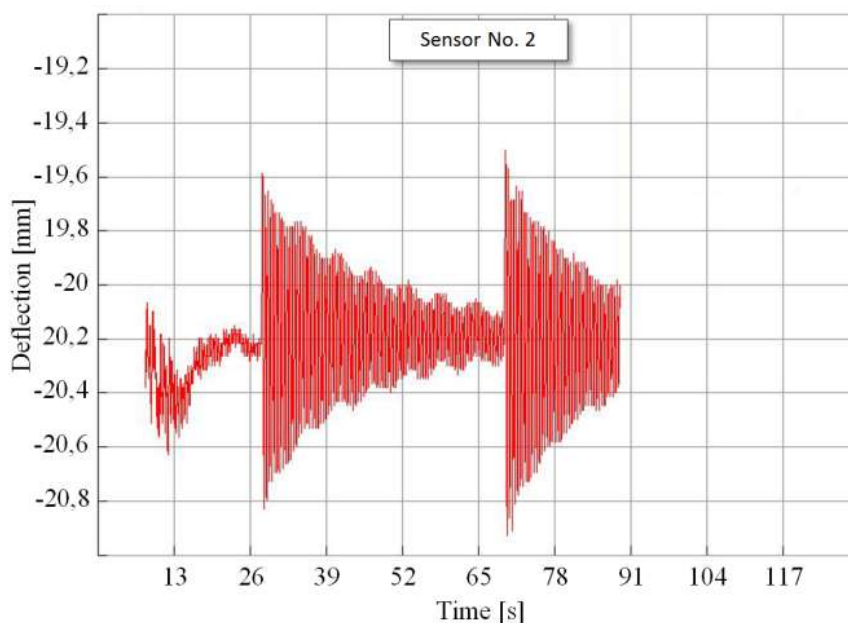


Fig. 5. Measurement interval for sensor No. 2

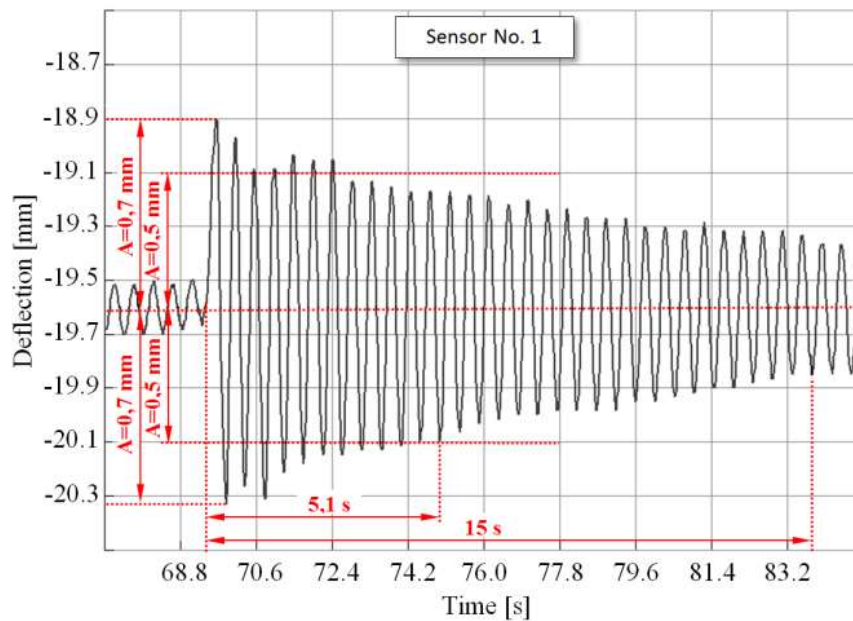


Fig. 6. Time sequence of deflections (Sensor No. 1)

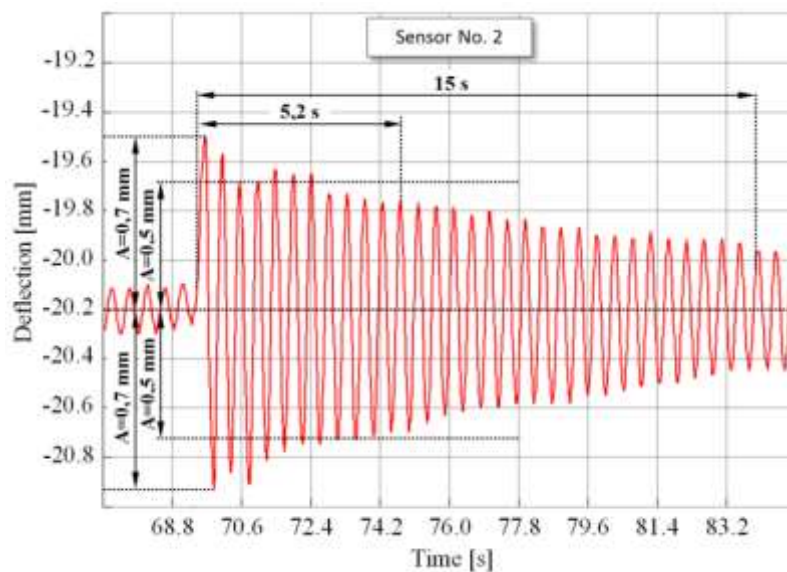


Fig. 7. Time sequence of deflection (Sensor No. 2)

3.3. Strain-gauge measurement evaluation

Figs. 6 and 9 show selected intervals of deflection time sequences established from stresses at lifting from Fig. 4, which can be used to determine the system dynamic characteristics. It involves determining the natural angular frequency with respect to damping, natural frequency, and natural oscillation period while considering the real damping in the girders and in the mutual relations. The diagrams shown in Figs. 6 to 9 clearly depicts the following periods of natural girder oscillation for the individual girders: $T_1 = 0.4s$ and $T_2 = 0.47s$. The amplitude sequence of the consecutive cycles can be used to establish the logarithmic damping decrements, which is, however, contingent on the potential influence of the oscillation amplitude. It is necessary to state, however, that the values established by

the strain-gauge measurement are affected by the measure of suppleness, or conversely, the stiffness of the wire rope, which translates into these quantities.

The values of cycle period established experimentally are greater than the values acquired by the analytical calculation. Other quantities are tied to the value by a precisely defined dependence, and therefore, do not require further commentary.

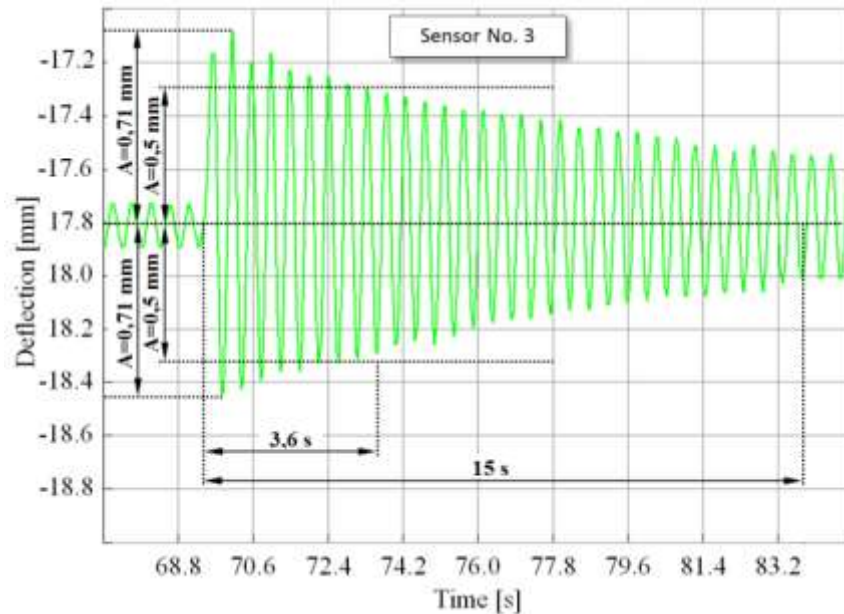


Fig. 8. Time sequence of deflections (Sensor No. 3)

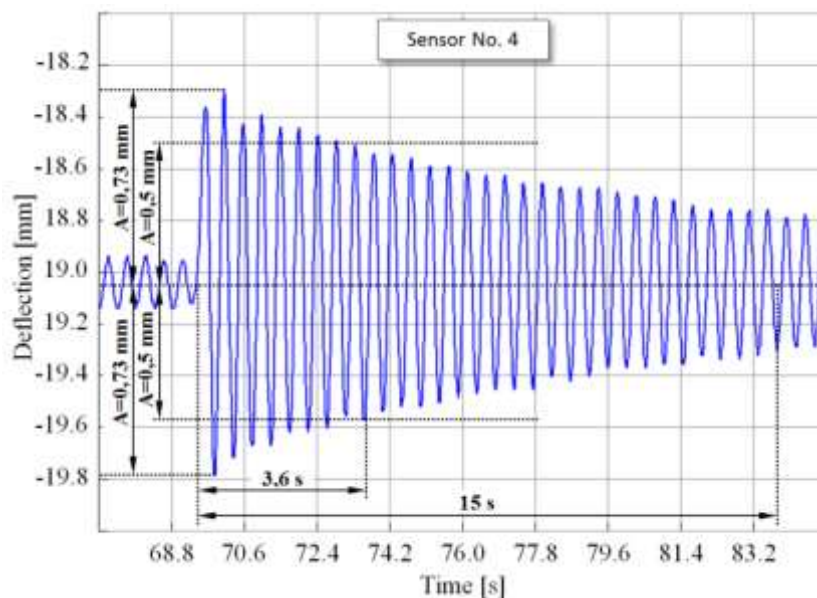


Fig. 9. Time sequence of deflections (Sensor No. 4)

3.4. Strain-gauge measurement evaluation

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4. SUMMARY AND CONCLUSION

When the structures are evaluated by serviceability limit states and they are accessible by the operators or the public, they must be designed to prevent personal discomfort (ex. natural frequencies of the internal organs that are identical to the frequency of the structure) caused by the dynamic effects of loading expressed by acceleration (or speed) and frequency.

The crucial natural frequencies of the structure and its parts ought to be sufficiently different from the frequencies of the actuating forces to prevent resonance, which is also especially important from the perspective of structural fatigue loading.

The standard [36] lists that there are two limit values of natural frequency and specifically 3 and 5 Hz. It also states that for the girders with a span $L \leq 10m$, the criteria could be deemed met if the specified extent of the deflection does not exceed 10 or 28 mm. It also points out that in special cases, a dynamic calculation is needed to show that the resulting acceleration and the frequency do not cause significant personal discomfort or failure of the equipment or its parts. Furthermore, the value of relative internal resistance is affected in no small measure by the losses of overcoming passive resistance in relations between structural elements, which the experimental measurement confirmed as well.

It follows that in fact, these relative quantities of internal resistance are greater; hence, the greater values of the logarithmic damping decrement. These are the actual reasons why it is impossible not to take into consideration Section X and Annex No. VII from the standard [36]. It can be concluded that provided the steel supporting structure of the bridge crane complies strength-wise, it complies also in terms of the second limit state of deformation and damping.

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