DYNAMIC RESPONSE OF AN ARCH BRIDGE UNDER HSLM HIGH-SPEED TRAIN LOADING ACCORDING TO THE EUROPEAN STANDARDS (EN)

W. Radomski[1], R. Oleszek[2]

[1]Łódź University of Technology, Department of Geotechnics and Engineering Structures, Łódź, POLAND.
[2]Warsaw University of Technology, Institute of Roads and Bridges, Warsaw, POLAND.

e-mails: wojciech.radomski@p.lodz.pl, w.radomski@il.pw.edu.pl, r.oleszek@il.pw.edu.pl

SUMMARY

The modern regulations concerning railway bridges are based on the approach to structural dynamics applied in the European standards (EN). This paper presents the results of a theoretical dynamic analysis of the HSLM-A train set loading on the structure of a pre-stressed concrete arch bridge - the first railway bridge of that type built in Poland (completed in 1959). The recommendations of Eurocodes is followed and a modal analysis is carried out to define the sensitivity of the structure to chosen eigenforms. The paper presents also the course of calculations and the conclusions inferred from the analysis of displacements, accelerations and bending moments induced in the structure by simulated passage of a high-speed train in the context of the requirements of the European Standards. Reference is also made to the standards used at the time of bridge engineering.

Keywords: *Arch bridge, pre-stressed concrete, high-speed trains, dynamic analysis.*

1. INTRODUCTION

The contemporary railway engineering regulations [1÷3, 8] are based on Eurocodes [6, 7] in which the dynamic nature of loading is considered on the basis of the dynamic analysis of the structure's response to a simulated moving load. The analysis consists in determining the direct processes of deformation and the distributions of internal forces under loading imposed by high-speed trains simulated by High Speed Load Models (HSLM).

This paper presents the results of theoretical dynamic analysis of HSLM-A loading on the structure of a pre-stressed concrete arch bridge which is the first railway bridge of that type built in Poland (completed in 1959) against the requirements of the European standards. This computational approach is similar to the approaches used by other researches [10, 12, 17-20, 22, 23]. Due to its exploratory nature the results of this analysis can be useful in the design of the contemporary arch bridges or in evaluating existing bridges of that type in terms of their suitability for the new operating conditions.

2. BRIDGE DESIGN DESCRIPTION

The bridge ever the river Orz in Gucin (Fig. 1) is the first railway arch bridge in Poland with pre-stressed post-tensioned concrete structure, designed in the period 1955–1956 by the design team led by Prof. Zbigniew Wasiutyński, and constructed in the period 1956–1959 [21].

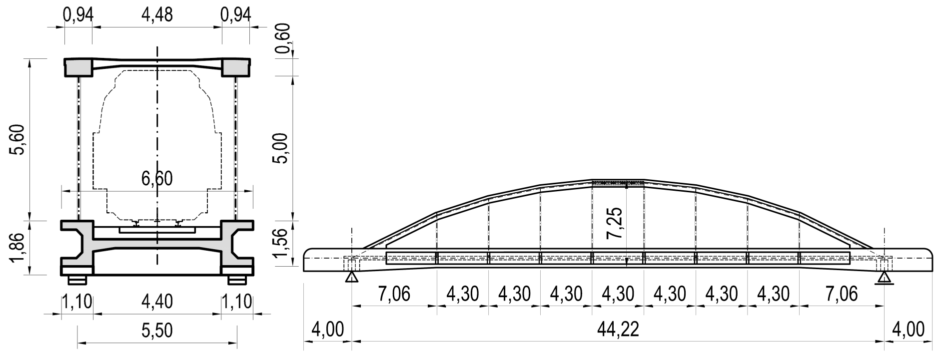


Fig. 1. *Railway bridge over the river Orz in Gucin, Poland: left - cross-section, right - side view.*

The bridge has the effective design span of 44.22 m and overall length of 52.22 m. The C-shaped longitudinal girders have a varying height of 1.56-1.76 m, the width of 1.10 m, 0.38 m thick web with 0.30 m top flange and 0.28 m bottom flange thickness. On 4.30 m length at supports the channel section changes linearly to a 1.10×1.86 m box section. The main girders are suspended from the concrete arches of 0.92-0.94×0.60 m using 120x40 mm flat irons. The arches are braced at their crowns with a 0.20 m tie plate. The pre-stressed concrete deck slab is 0.30 m thick, The concrete members are made of concrete with R_w = 400 kG/cm^2 and Freyssinet prestressing steel tendons. The structure is designed for NC load class (standard heavy train) according to [4, 5].

3. STRUCTURAL DYNAMIC CALCULATIONS

3.1. Computational models used in the analysis

The following are the computational models of the structure differing in terms of mesh refinement (Fig. 2), built in SOFiSTiK FEM software [15, 16, 19, 22].

PB is a spatial beam and shell model of the structure (e^1+e^2, p^3). The longitudinal girders, arches and the supporting cross-beams are modelled with Timoshenko beam finite elements. The bridge deck is represented by Mindlin-Reissner plate elements. The hangers are modelled with lattice bars. This model was used to investigate torsional mode shapes and to calibrate the remaining representations.

R3D-1 is a fully detailed spatial bar frame (e^1, p^3). The longitudinal girders, arches, cross-beams and deck (grillage built of longitudinal girders and cross beams) are modelled with Timoshenko beam finite elements and the hangers – with lattice bars. This model was used to investigate the torsional and bending mode shapes.

R3D-2 is a modified spatial bar frame (e^1, p^3). This provides a simplified representation of the structure in which the deck slab is represented by a single longitudinal beam connected by kinematic couplings with the edge girders. The hangers are represented by beam elements. This model was used to investigate the vibrations of hangers.

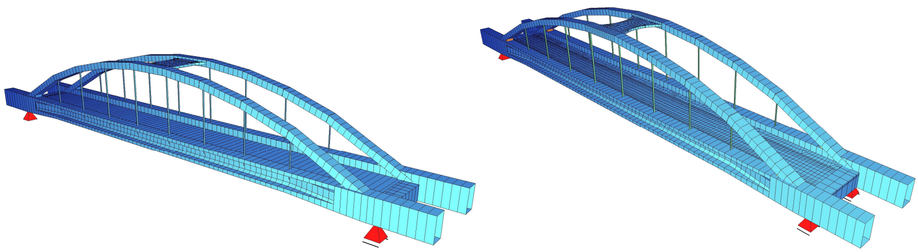


Fig. 2. Graphical presentation of the respective numerical models: a) spatial frame model – R3D-3, (e^1, p^3), b) beam and shell model, PB, (e^1+e^2, p^3).

R3D-3 is a simplified spatial frame model (e^1, p^3). The hangers are modelled with lattice bars. For the deck slab a simplified discretization method was used giving a model with a single longitudinal beam connected with the edge girders by kinematic couplings. This model was used for verification of the main bending mode shapes of the structure (skipping secondary elements) and for the time-history analysis (integration of the equations of motion needing to reduce the number of the dynamic degrees of freedom and the size of the numerical problem).

3.2. Dynamic analysis

The modal analysis (solving of the eigenvalue problem) was carried out in ASE module of the SOFiSTiK software. The natural vibration computations consider also the weights of non-structural components (ballast, sleepers, rails) which are converted to equivalent node weights.

The calculations of the global (that is ignoring the vibration of hangers) dynamic response of the structure (time-history analysis) were carried out in the DYNA module of SOFiSTiK. The implicit time integration method based on the Newmark algorithm was used for the equations of motion integration [11-13, 14, 16, 24].

The integration time step of Δt=0.005 sec. was determined from the requirement of $\Delta t \leq 0.1/f_{max}$ [9, 12, 13, 16]. Mass- and stiffness-proportional (Reylaigh) damping model was applied. The damping matrix **C** is made by combining the mass matrix **M** and the stiffness matrix **K** according to the following formula: $\mathbf{C} = \alpha\mathbf{M}+\beta\mathbf{K}$. The coefficients of proportionality of internal and external damping (α and β) were calculated according to the percentages of the respective eigenforms and eigenfrequencies and the value of damping ratio ζ=1.0 % as calculated according to EN [7]. With the span length of 44.22 m i.e. more than the limit 30.0 m the additional damping to account for the train-bridge interaction (indirectly i.e. in a simplified way) is $\Delta\zeta$ = 0 %.

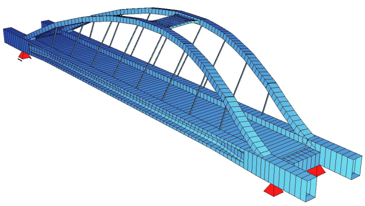
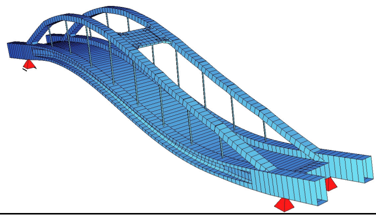
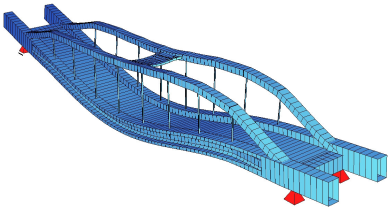
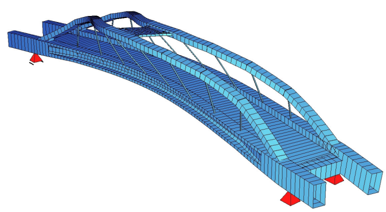
The loading was modelled by the sets of concentrated loads simulating the action of HSLM train models A1–A10 [8]. In this method the load which is constant in time is converted the equivalent set of nodal forces of with a defined phase shift corresponding to the analysed vehicle speed. The analysis was carried out for speed range 160-360 km/h at 10 km/h intervals. Static "train passages" were also considered. This enabled comparing the response to static and dynamic loading and determination of the dynamic amplification factor as a function of deflections and running speed of HSLM train sets $\varphi=\varphi(v,u_z)$.

4. RESULTS OF CALCULATIONS

4.1. Modal analysis

Table 1 provides graphical presentations of the selected bending eigenform frequencies. The first two torsional eigenforms of the bridge deck are presented in Table 2.

Table 1. *Graphical presentation of selected bending eigenforms and eigenfrequencies of the analysed bridge.*

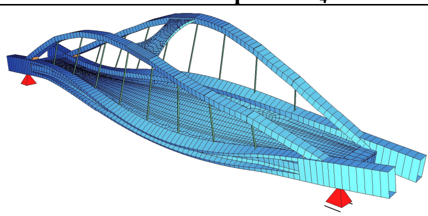
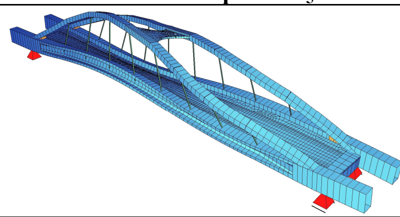
1	**$f_1 = 1.56$ Hz**	2	**$f_2 = 3.74$ Hz**
	vibration of arches in the horizontal plane, half-sine wave 		first eigenform for bending vibration in the vertical plane, two half-sine wave 
3	**$f_3 = 4.60$ Hz**	**7**	**$f_7 = 7.41$ Hz**
	second eigenform for bending vibration in the vertical plane, half-sine wave 		bending vibration of the bridge deck and arches, half-sine wave 

The first eigenfrequency of f_1=1.56 Hz concerns vibration of arches in the horizontal plane. The first (and the second among all) eigenfrequency for bending vibration of the bridge deck in vertical plane, identified in the simplest model (R3D-1) is f_2=3.74 Hz. It is an anti-symmetric mode shape (two half-sine wave). The value of 3.74 Hz slightly exceeds the lower frequency limit according to the graph in EN [7]. The remaining frequencies are within the upper and lower envelopes of 9.35 Hz and 3.77 Hz respectively, thus not requiring time-history analysis for running speeds below 200 km/h.

The **R3D-1** model (a fully detailed spatial frame) yielded the following torsional vibration frequencies given in the increasing order: 5.47 Hz, 6.44 Hz, 7.24 Hz, 8.13 Hz. In the PB model, the most accurate of all, the torsional mode shapes were obtained at the following frequencies: 5.52 Hz, 6.33 Hz, 7.30 Hz, 8.15 Hz. The differences between these results do not exceed 2%.

R3D-2 model (with hangers modelled by beam elements) was used to determine the eigenfrequencies of the structure and of the suspension bars. The total of 100 mode shapes were determined. The last significant bending mode shape of free vibration of the whole structure was noted at the frequency of f_{12}=12.30 Hz. The subsequent composite mode shapes (vibration of hangers, arches, crown tie plate) including the main girders bending mode shape occur at the frequencies higher than f_{50} = 23.64 Hz (the fiftieth mode shape). Such high vibration mode shapes are, however, physically impracticable, i.e. hardly possible to occur in field. Moreover, they exceed the limits given in the regulations: f_{max}=20 Hz according to [2] and f_{max}=30 Hz according to [6, 7, 12]). Generally it is practicable to identify only the first mode shapes of the main superstructure components. The investigation of the torsional and bending vibrations is required by EN [7].

Table 2. *Graphical presentation of selected torsional eigenforms and eigenfrequencies of the analysed bridge.*

1st torsional mode shape	**f_4 = 5.52 Hz**	**2nd torsional mode shape**	**f_5 = 6.33 Hz**
			

The first torsional mode shape, determined on the basis of the fully detailed shell and beam model (PB, e^1+e^2, p^3) occurs at the minimum frequency of n_T = f_{min} = 5.52 Hz. In the simplified, spatial frame model (R3D-2, e^1, p^3) the first torsional mode shape occurs at the frequency of 5.47 Hz. The results obtained with the two models differ by about 1% only.

In the analysed arch structure the ratio of the lowest frequency of torsional vibration to the first frequency of the vertical bending vibration is n_T/n_0 = 5.52/ 3.74 = 1.47 > 1.2. Therefore, according to EN [7] the analysed bridge is insensitive to torsional vibration and for the purpose of time-history analysis these mode shapes are of minor importance.

The minimum frequency of lateral (horizontal bending) vibration obtained with R3D-3 model is $f_{h\text{-}min}$ = 7.41 Hz which is more than the value of f_{h0} = 1.2 Hz. Thus, according to EN [7] the structure is insensitive to lateral vibration, as may be caused, for example, by sideway.

4.2. Time-history analysis (dynamic response of the system)

The time-history analysis considered bending moments M_y, vertical displacements u_z and accelerations a_z at the points located on the main girders at about 1/4.5 ($0.22 \cdot L$) and 1/2 ($0.50 \cdot L$) of the span length (Figs. 3 and 4). The relationships between the analysed parameters (a_z, u_z, M_y) and the running speed and HSLM-A load model are presented in Figs. 5-8.

From the graphs (Figs. 5-8) we see that higher dynamic effects occur in the maximum bending moments section (ca. $0.22 \cdot L$) at the train speed of about 230-280 km/h. At that point certain types of HSLM train sets cause vertical span accelerations a_z exceeding the limit value a_{zdop} = 3.5 m/s^2 defined due to the risk of ballast instability (Fig. 6). The greatest dynamic effects of high-speed trains were noted at the speeds higher than 320 km/h. It transpires from the graph in Fig. 5 that the value of acceleration a_z at the midspan is below a_{zdop} = 3.5 m/s^2 for all the considered train speeds (they do not exceed 3.0 m/s^2).

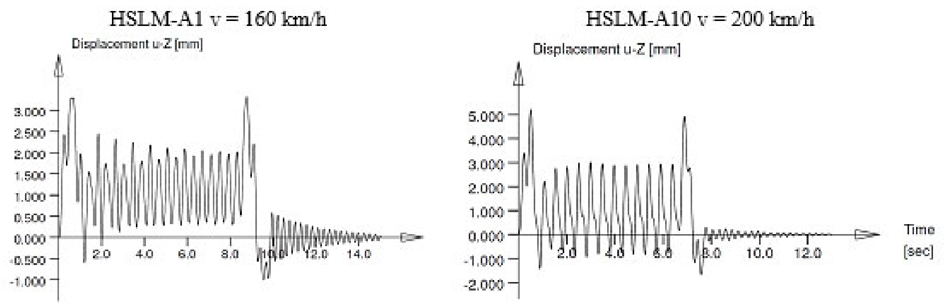


Fig. 3. *Selected time-history diagrams of the main girder vertical displacements u_z at the node located at $0.22 \cdot L$ from the support, imposed by HSLM train set.*

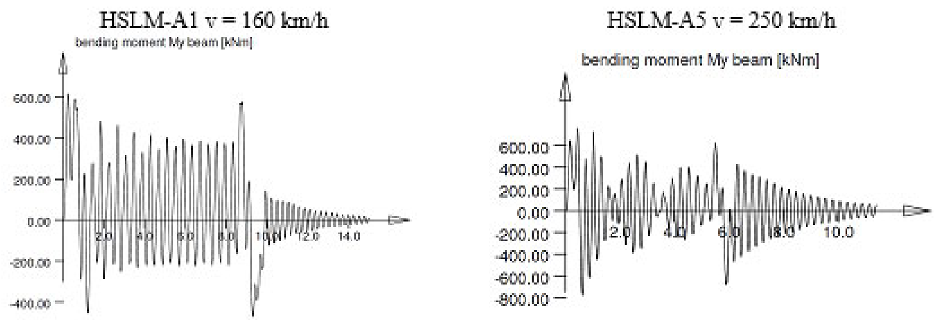


Fig. 4. *Selected time-history diagrams of the main girder bending moments M_y at the node located at $0.22 \cdot L$ from the support, imposed by HSLM train set.*

The dynamic effects are the greatest when the trains enters into or leaves from the bridge, as represented by the peaks of u_z amplitudes (Fig. 3 and Fig. 4). Between these points in time the vibrations at that point oscillate about the point of equilibrium. As soon as the load leaves the bridge the vibrations fade out.

The maximum moments in static load cases using HSLM-A are ca. 36 % for point 0.22·L and ca. 45 % for point 0.50·L of the design moment from NC class trains according to [4, 5]. The maximum dynamic moments from HSLM-A do not exceed the design moments from NC class trains at the running speeds of about 230-280 km/h and v > 320 km/h (Fig. 7).

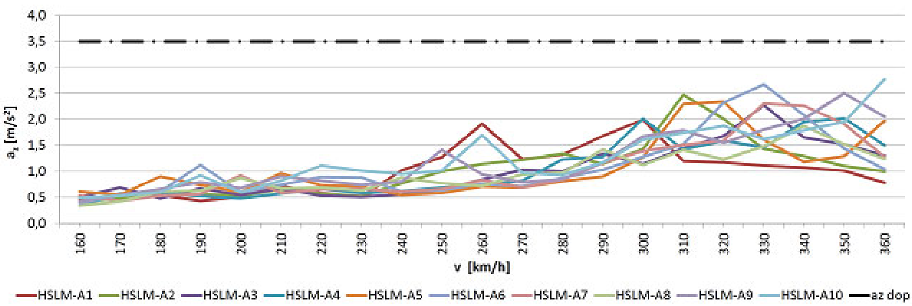


Fig. 5. *The main girder vertical acceleration a_z [m/s^2] at midspan (0.50·L) vs. running speed v [km/h] for HSLM-A1-A10.*

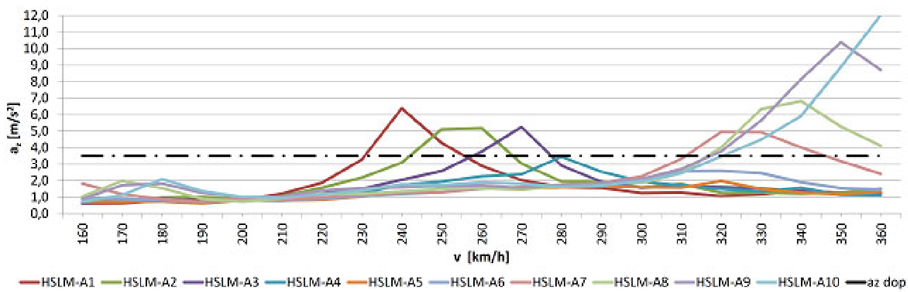


Fig. 6. *The main girder vertical acceleration a_z [m/s^2] at 0.22·L from support vs. running speed v [km/h] for HSLM-A1-A10.*

Fig. 8 presents the relationship between the dynamic amplification factor vs. midspan deflection and train speed $\varphi = \varphi(v,u_z)$. They are compared with the design values required at the time of bridge engineering [4, 5] $\varphi_{proj} = 1.212$ and by the Eurocode [7] that is $\Phi_2 = 1.140$ for careful maintenance and $\Phi_3 = 1.210$ for standard maintenance of the track.

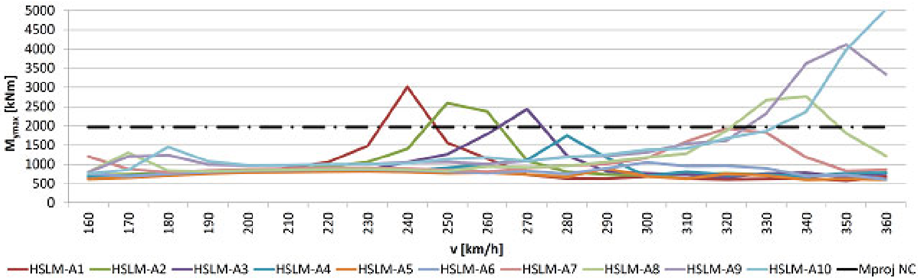


Fig. 7. *The main girder bending moments M_y [kNm] at 0.22·L from support vs. running speed v [km/h] for HSLM-A1-A10.*

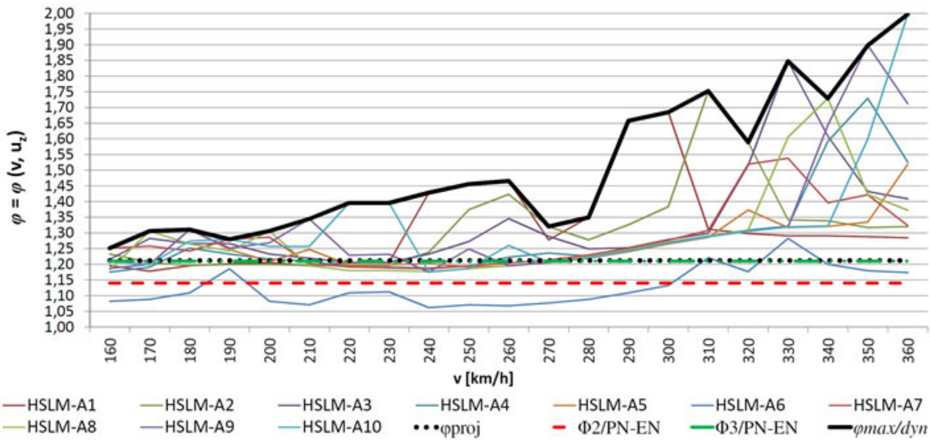


Fig. 8. *Dynamic amplification factor $\varphi = \varphi(v,u_z)$ vs. deflections and running speed of HSLM A1-A10 trains.*

In the whole range of the analysed running speeds of HSLM-A load models the dynamic amplification factors determined on the basis of the time-history analysis are in the range of $\varphi(v,u_z)$ = 1.25-2.0. These values are higher than calculated according to the standard equations (φ_{proj} = 1.212, Φ_2 = 1.140, Φ_3 = 1.210). Already at v = 160 km/h the dynamic amplification factor exceeds the design value of φ_{proj} = 1,212 as per [4, 5].

The peaks of dynamic amplification factors $\varphi(v,u_z)$ occur at the speeds of v = 180 km/h (φ = 1.31), v = 260 km/h (φ = 1.47), v = 310 km/h (φ = 1.75), v = 330 km/h (φ = 1.85) and v = 360 km/h (φ = 2.0). Note that the maximum values of $\varphi(v,u_z)$ are generated at the given speeds by different HSLM-A trains. The dynamic heights are the greatest in the case of HSLM-A6 and the lowest in the case of HSLM A1, A2, A3, A9 and A10. The load testing before putting the bridge in service in 1959 [21] was carried out by loading the bridge with locomotives travelling with much lower speeds of v_1 = 27 km/h, v_2 = 60 km/h, v_3 = 69 km/h, v_4 = 75 km/h and the following dynamic amplification factors were obtained at that time: φ_1 = 1.059, φ_2 = 1.125, φ_3 = 1.258, φ_4 = 1.194 ($\varphi_{max} = \varphi_3 = 1.258 > \varphi_{proj} = 1.212$).

5. FINAL CONCLUSIONS

The results of our analysis of the arch bridge Gucin allow us to conclude as follows:

- according to the EN recommendations the structure is relatively insensitive to the torsional mode shapes (n_T/n_0 = 1.47 > 1.2) and horizontal deck vibration (f_h = 7.41 Hz > 1.2 Hz) and these mode shapes are not relevant to the dynamic response analysis (HSLM load model consists of vertical forces without any load eccentricity),
- the HSLM-A load models generated unacceptable increase of accelerations a_z, displacements u_z and bending moments M_y at the speed range of about 230-280 km/h and over 320 km/h,
- passages of HLSM-A trains generate at ca. 1/4.5 span length (0.22·L) maximum vertical accelerations of $a_z > a_{zdop} = 3.5$ m/s^2 at the speeds of v > 230 km/h and this value is unacceptable because of the risk of destabilising of the ballast,
- in the speed range considered in the analysis, i.e. v = 160-360 km/h the dynamic amplification factor determined on the basis of dynamic response analysis of the structure under HSLM A1-A10 loading exceeds the design value of φ_{proj} = 1.212 and the values determined with the empirical relationships defined by EN: Φ_2 = 1. 140 for careful maintenance and Φ_3 = 1.210 for standard maintenance of the track.

In general, it can be concluded on the basis of the simulations with the standard load models HSLM-A1-A10 at running speeds of v = 160-360 km/h that the analysed arch bridge structure complies with the dynamic requirements of EN for trains travelling at the speeds of v < 230 km/h. The dynamic values measured in field are usually smaller than the values estimated in the theoretical analyses according to Eurocodes which feature a number of simplifications in modelling the train-track-bridge interaction. More sophisticated calculation methods are presented, for example, in [9, 11, 12, 14, 19, 24, 25]. The procedure presented herein can be applicable to estimate the behaviour of old bridge structures subjected to high-speed passage of the trains.

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