MODELLING AND NUMERICAL ANALYSIS OF THE REINFORCED CONCRETE VIADUCT UNDER THE EUROCITY EC-114 TRAIN

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Abstract

The paper presents a methodology of finite element modelling and numerical simulation of symmetric vibrations of the reinforced concrete viaduct using advanced CAE systems. Two types of analysis were carried out – static analysis of the viaduct under dead load and dynamic analysis under a moving high speed train. The KNI 140070 viaduct of span length of 14.40 m with abutment zones was selected as a representative for the study. The serviced ballasted track was taken into consideration. The FE model of the track includes the main rails, fastening systems, sleepers and the ballast. The EuroCity EC 114, Polish high speed train, moving at 100–160 km/h was selected for the study. Components of the train FE model were considered as rigid bodies and were modelled using shell and beam elements. Cylindrical and revolute constrained joints were applied for kinematic connections and relations between relative components. Discrete springs and dampers were applied for modelling of the primary and the secondary suspension systems. The numerical analysis was performed using finite element code LS-DYNA. The stress distribution in selected components of the viaduct structure was presented as the results of the static analysis, whereas the selected time histories for displacements and stresses were shown as the results of the dynamic analysis. The results obtained from both analyses allowed to assess the actual displacement and stress states for the considered viaduct.

Keywords: *bridge – train interaction, moving train, reinforce concrete bridge, numerical analysis, dynamics*

1. Introduction

Nowadays, serious problems with durability protection of bridge superstructures, tracks and approach zones loaded by high-speed trains are observed. First of all, it results from complexity of bridge-track–moving train (BTT) systems, for which nonlinear models are described with a huge number of parameters. Many of these parameters, describing fastening systems, ballast, subsoil layers, suspensions of the railway vehicles, track irregularities, settlements etc., are difficult for identification. Therefore, some of them are mostly estimated. Producers and research institutions involved in modern high-speed trains do not bring to light structural details, values of parameters or their research results. These inconveniences make exact prediction of dynamic response of bridges subjected to moving trains very difficult.

In the 2nd half of the 20th century scientists mostly developed analytic – numerical methods in dynamics of railway bridges, summarized in [1, 2]. Simple problem-oriented computer codes were created and applied for simulations. At present, it can be observe various numerical approaches to dynamics of railway bridges using commercial CAE systems [3–7]. The BTT systems are generally three-dimensional, however they may be considered as simplified ones with respect to the vertical longitudinal plane of symmetry. Such approach was proposed by Authors in [8–11].

A methodology of physical and FE modelling and simulation of the BTT system has been developed with the use of commercial CAE systems. It is related to the composite (steel – concrete) bridge, ballasted track and high-speed train. Altair HyperMesh, LS-DYNA, LS-PrePost and Hyper View software was applied in the methodology.

The methodology is based on homogenization of reinforced concrete (RC) platform slab, RAIL_TRACK and RAIL_TRAIN LS-Dyna's modules for simulating the moving train – track interaction, non-linear modelling of rail fasteners and ballast, application of cylindrical and revolute constrained joints and discrete springs and dampers for modelling suspensions in railway vehicles. The KNI 140070 composite viaduct and the EuroCity EC-114 *Praha* train were selected as a representative for the study.

2. Description of considered objects

The composite (steel – concrete) viaduct No. KNI 140070, located at the Polish Central Main Line (PCM) No. 4–E 65, was selected for numerical modelling and simulation. The bridge span under track No. 1 was taken into considered (Fig. 1).

Fig. 1. The modernized viaduct No. KNI 140 070 composed of two separate spans, located at the PCM Line, Poland. Bottom view on the main beams and bracing

After recent modernization [12], the viaduct has the $k = +2$ rail-line classification coefficient. Track spacing equals 4.57 m, the spans are of 14.40 m theoretical length and 15.34 m total length. Four main beams are 0.77 m high and made of St3M steel. Bottom flanges have been reinforced with additional plates. The thickness of a new RC platform ranges from 0.29 m in the track axis to 0.25 m at the side wall. The platform is made of C35 concrete reinforced with AII/18G2-b steel rebars. The side wall is made of C30 concrete and has vertical dilatations at ¼, ½, and ¾ of the span length. The RESTON pot bearings (on the left support) can shift up to ±50 mm in the longitudinal direction. Bearings under the left inside main beam are unmovable in the lateral direction; the remaining bearings can displace in the lateral direction up to ±20 mm. A scheme of the longitudinal section of the KNI 140070 viaduct is depicted in Fig. 2

The ballasted track consists of UIC 60 rails, PS-94/SB/UIC-60 sleepers with SB3 fasteners, and the first class ballast. The ballast layer under sleepers is 0.35 m thick. The embankment in the approach zones contains cement-stabilized subsoil while outside the approach zones a 0.2 m thick sand-gravel mix top layer has been applied. The tracks over the viaduct are quasi-rectilinear. Schemes of the longitudinal and cross sections of the ballasted track are shown in Fig. 3.

The EC-114 *Praha* EuroCity train, moving over the bridge, has been taken into consideration. The trainset in selected configuration consists of 6 units – the EP-09 locomotive and five carriages. The train dimensions, centre pins' distances and wheel sets' distances are reflected in Fig. 4. All units are equipped with two independent two-axle bogies. The train was moving at the speed of 100–160 km/h during the analysis.

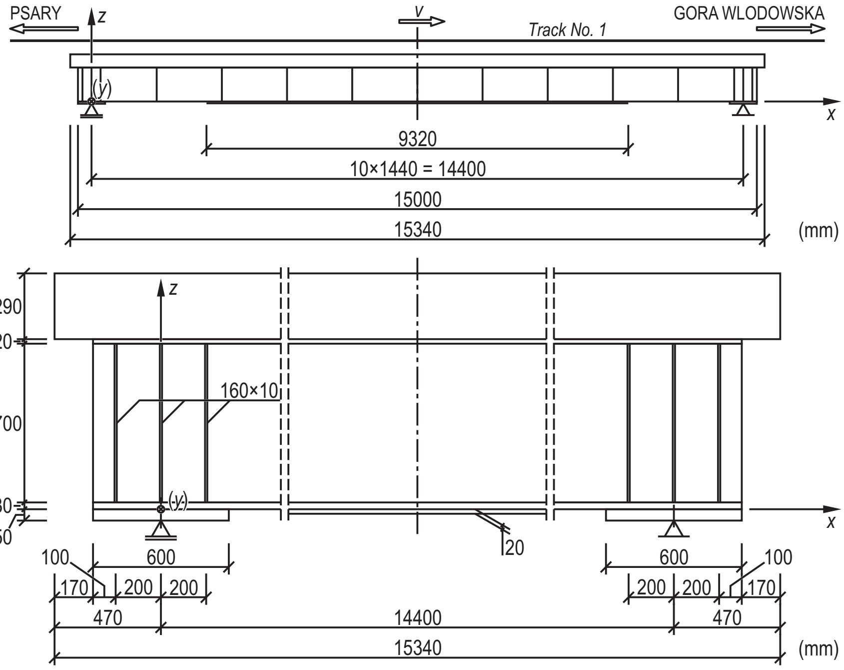

Fig. 2. The longitudinal section of the KNI 140070 viaduct in the xz plane

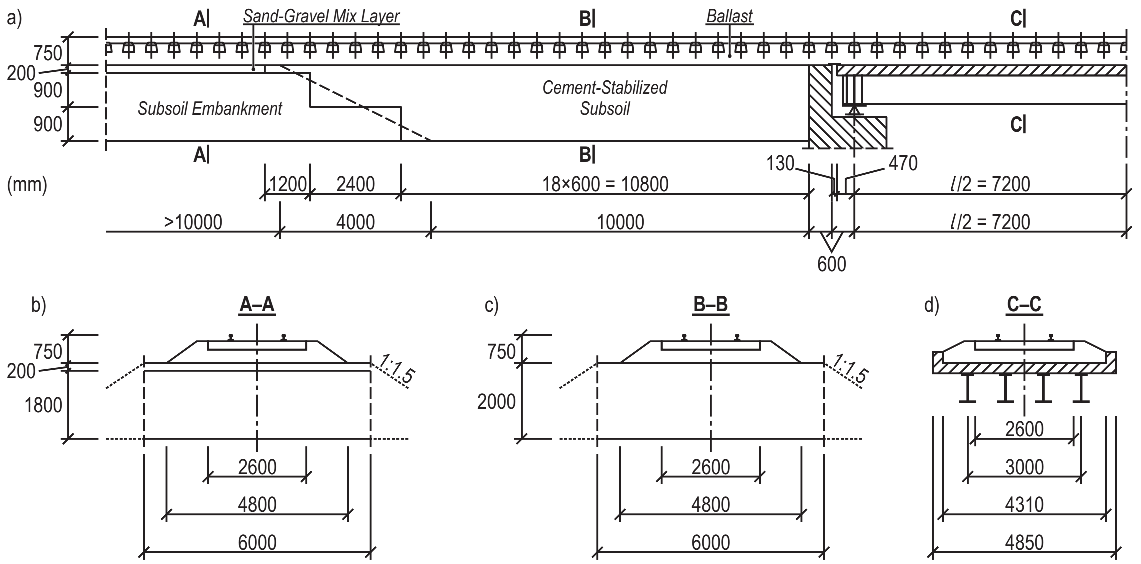

Fig. 3. The ballasted track in the KNI 140070 viaduct zone: the longitudinal section (a), the cross-section in the approach zone (b), the cross-section over the bridge span (c)

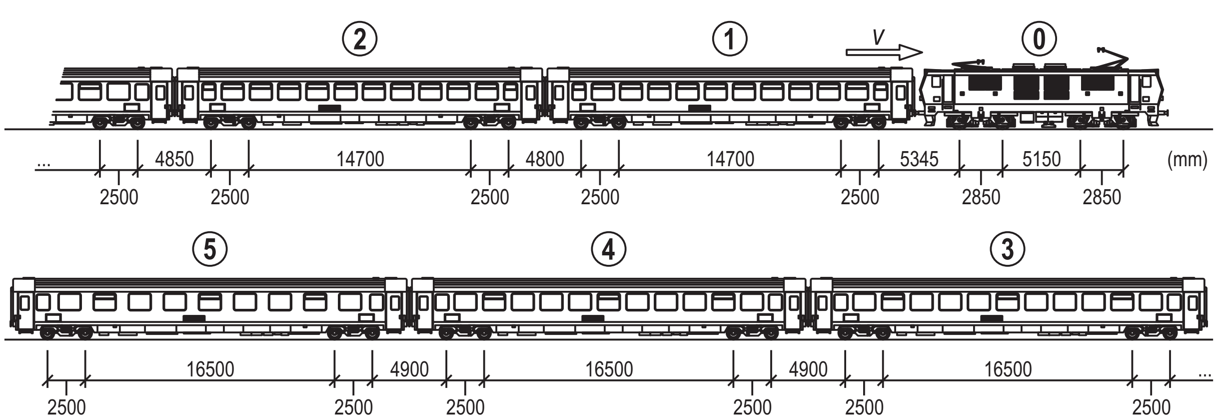

Fig. 4. A scheme of the EC 114 Praha EuroCity trainset

3. Physical and numerical modelling of the BTT system

The following methodology was developed in physical modelling of the viaduct. The slab and the side wall of the platform were homogenized since the reinforcement of the RC platform was distributed quasi-uniformly in the specified platform sections. After homogenization [13] the slab and the wall were reflected by linear viscoelastic orthotropic materials described by 3 Young's moduli, 3 Poisson's ratios, and 3 shear moduli in each subarea. A cross-section of the platform slab was approximated by a rectangle of 0.27 m height. The platform was symmetrised – single dilated wall was replaced by two smaller dilated walls on both sides of the platform slab. The original and approximate structures have the same values of mass and cross-section area. The vertical and horizontal bracings in the main beams' set were neglected. All bearings on the abutments are assumed to be unmovable in the lateral direction. Fig. 5 depicts the original and symmetrised cross-sections, as well as the longitudinal section of the viaduct.

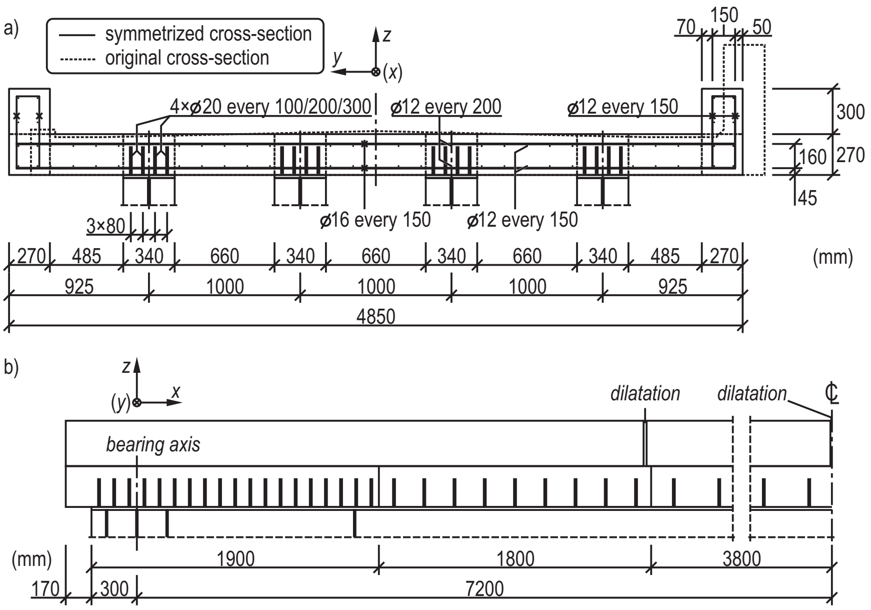

Fig. 5. The original and symmetrised cross-sections (a), and the longitudinal section (b) of the KNI 140070 viaduct

The FE model of the bridge superstructure was developed in Altair HyperMesh software (Fig. 6). The numerical model of the bridge superstructure consists of 3896 4-node shell elements (steel main beams) and 5568 8-node 48 DOF solid elements (the homogenized RC platform divided into orthotropic parts). Roller bearings on the left bridge abutment and pivot bearings on the right bridge abutment have been reflected by respective constraints.

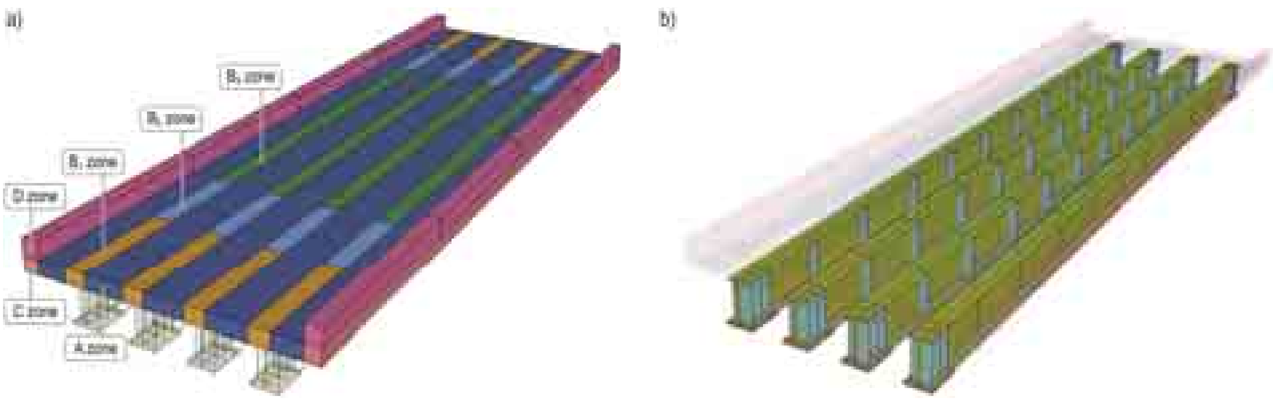

Fig. 6. The FE model of the KNI 140070 viaduct superstructure – FE parts and homogenization zones are marked

The following assumptions were made in physical modelling of the track. The rail-line as well as the rails axis – in the unloading state – is rectilinear. Vibrations of the track are small and symmetric with respect to the vertical *xz* plane. The rails were considered as prismatic beams deformable in flexure and shear, made of linearly viscoelastic material. Layers of the embankment were modelled as a linearly viscoelastic material continuum. Rail fastening systems were simulated using massless 1-D discrete non-linear spring and damper elements. The embankment was reflected approximately by a rectangular prism with unmovable side and bottom boundary surfaces and meshed using 8-node 24 DOF solid elements. Sleepers were modelled as elastic beams vibrating only vertically using finite beam elements and respective constraints. The ballast layer has been divided into cubicoid columns in coincidence with FE mesh of the parts under the ballast (9 ballast columns under each sleeper). Each ballast column was reflected by a vertical set of nonlinear spring and damper elements. The lumped mass distribution for the ballast has been put into the bottom set of the nodal points contacting the platform slab and the top subsoil layers. Values of geometrical and mechanical parameters of the ballasted track components were extracted from [1, 2, 14–16].

RAIL_TRACK and RAIL_TRAIN modules available in LS-DYNA [17] were applied for approximate modelling the train – track interaction (without simulation of wheels' rotation). Hughes-Liu beam elements were used for the rail FE model. In order to declare a set of integration points for the rail cross-section, the INTEGRATION card has been applied. For each rail an equivalent double-tee asymmetric cross-section was assumed. The actual values of the centre-of-gravity location, the area and the geometrical moment of inertia were saved with respect to the horizontal principal axis of the cross-section. The track FE model scheme was reflected in Fig. 7.

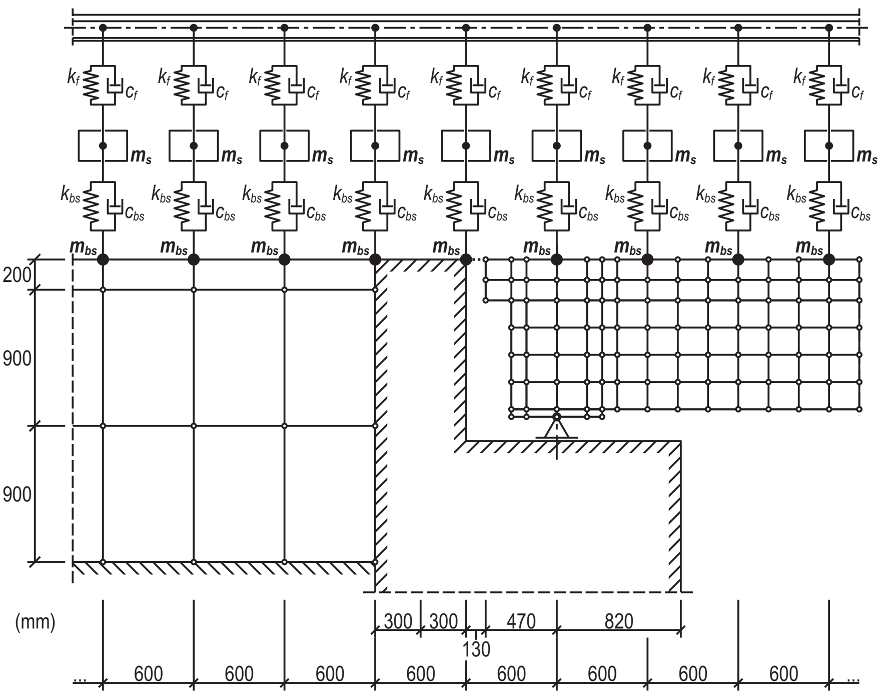

Fig. 7. The side view on the physical and FE model of the track in the left abutment zone of the KNI 140070 viaduct

Modelling of the EC 114 trainset was performed in LS-PrePost software. It was assumed that vibrations of the train units are symmetric with respect to the main longitudinal vertical plane of symmetry. A numerical model of the EC 114 trainset consists of the following components: carbodies, bogie frames, wheel sets – treated as rigid bodies – and vertical massless discrete linear viscoelastic elements reflecting the primary and secondary suspension systems. Respective constraints have been put into the system using translational and rotational elements [17]. A side-view scheme of the 3D model of the train units is shown in Fig. 8. Values of mechanical parameters of the EC 114 train units were determined based on [1, 18].

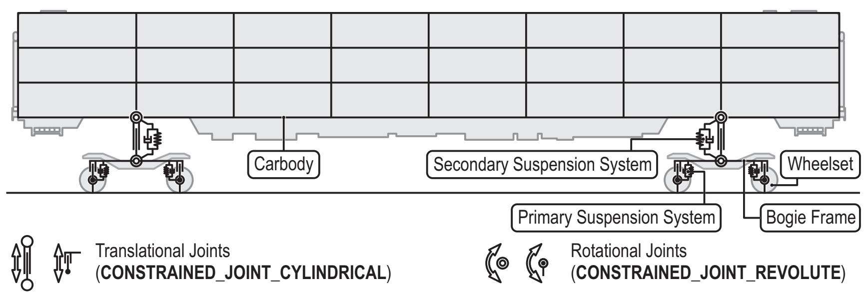

Fig. 8. A side-view scheme of the 3D model of the EC 114 train units – selected carriage

In the simulations, the DYNAMIC_RELAXATION option [17] has been replaced with loading the system by a set of vertical forces put in the moving vehicle – rail contact points according to the formula:

$$P(t) = \frac{P_0}{2}\left(1 - \cos\frac{\pi t}{t_0}\right), \tag{1}$$

where:
P_0 – static load of a single wheel on the rail head,
t_0 – time of increasing of the static load up to the full value, $t_0 = 2$ sec ($0 \le t \le 2$ sec).

A constant service velocity of the vehicle FE model was declared in two steps – with option INITIAL_VELOCITY for $t = 0$ and PRESCRIBED_MOTION_RIGID for $t > 0$ [17].

4. Numerical FE analyses

The numerical analyses were carried out using finite element code LS-DYNA. Two types of analysis were performed – static analysis of the viaduct under dead load and dynamic analyses under a moving high speed train. The dead load of the viaduct was not taken into account in dynamic analyses, therefore obtained values should be increased by the values from the static analysis. Measurement point was depicted in Fig. 9. Following values were registered at the midspan during the analysis:
D1, D2, D3 – deflection of the outer and inner main beam, and the rail, respectively,
S11, S12 – longitudinal normal stresses in respectively bottom and upper flange of the outer main beam,
S21, S22 – longitudinal normal stresses in respectively bottom and upper flange of the inner main beam.

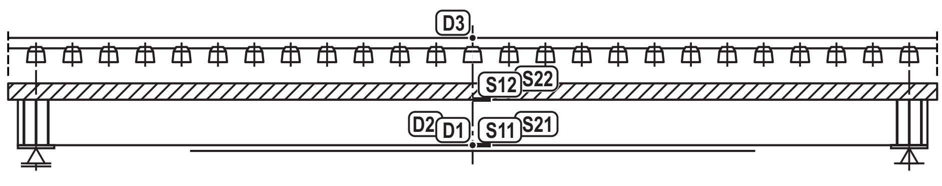

Fig. 9. Locations of the measurement points (nodes and elements) on the KNI140070 viaduct

Figure 10 shows contour of vertical displacements for the complete viaduct superstructure under the dead load. The contour of the longitudinal normal stresses in bottom and top flanges of the main beams is presented in Fig. 11. Selected obtained results were provided in Tab. 1 and 2.

Extreme values of registered deflections and stresses obtained from the dynamic analyses are presented in Fig. 12–13.

Contours of the vertical displacement and the longitudinal stresses for the viaduct under the train moving at the speed of 115 km/h were selected as representative and depicted in Fig. 14 and 15, respectively.

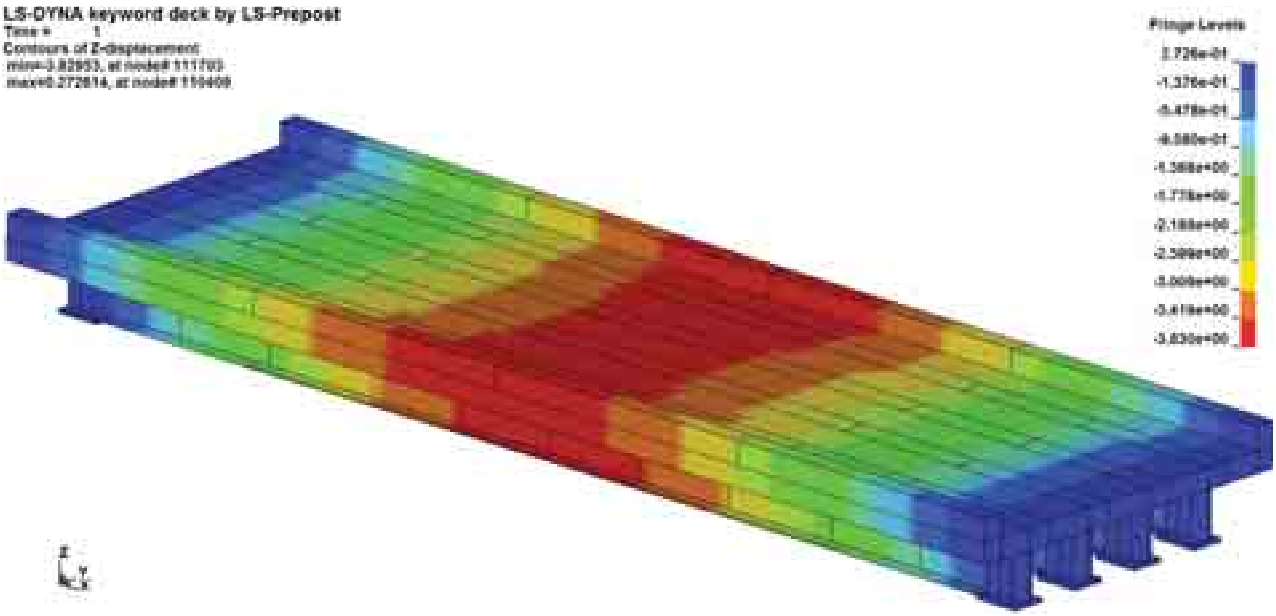

Fig. 10. The contour of the vertical displacement [mm] for the complete viaduct superstructure under the dead load

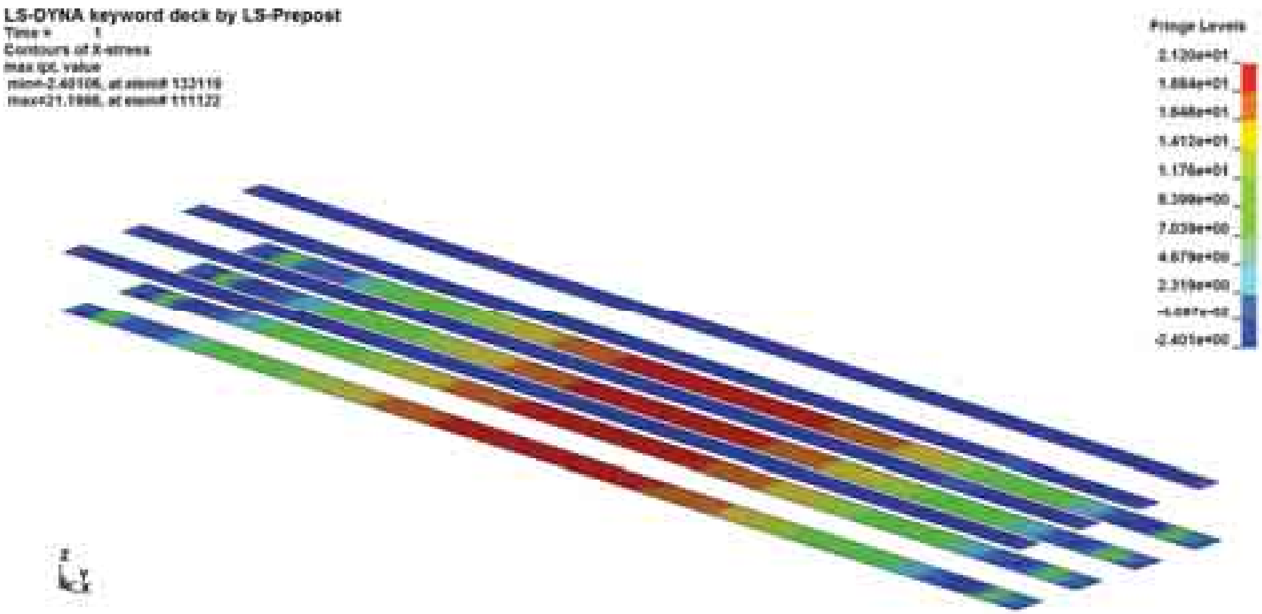

Fig. 11. The contour of the longitudinal normal stresses [MPa] in bottom and top flanges of the main beams under the dead load

Tab. 1. Vertical deflection of the viaduct – selected results (designations in accordance with the Fig. 9). Note that positive deflection is measured down

Measurement point		Dead Load	Top speed (160 km/h)		Selected speed (115 km/h)	
		Static	Dynamic	*Total*	Dynamic	*Total*
		Vertical deflection [mm]				
D1 (outer beam)	max	3.637	2.178	*5.815*	2.206	*5.843*
	min		–0.035	*3.602*	–0.184	*3.453*
D2 (inner beam)	max	3.557	2.216	*5.773*	2.242	*5.799*
	min		–0.038	*3.519*	–0.178	*3.379*
D3 (rail)	max	3.621	2.762	*6.383*	2.916	*6.537*
	min		–0.050	*3.571*	–0.190	*3.431*

Tab. 2. Longitudinal normal stresses in the flanges of the main beams – selected results (designations in accordance with the Fig. 9)

Measurement point		Dead Load	Top speed (160 km/h)		Selected speed (115 km/h)	
		Static	Dynamic	*Total*	Dynamic	*Total*
		Longitudinal normal stress [MPa]				
S11 (outer beam, bottom flange)	max	21.129	12.148	*33.277*	13.100	*34.229*
	min		–0.355	*20.774*	–1.218	*19.911*
S21 (inner beam, bottom flange)	max	20.304	12.250	*32.554*	13.144	*33.448*
	min		62	*20.042*	–1.111	*19.193*
S12 (outer beam, top flange)	max	–1.304	0.082–0.2	*–1.222*	0.155	*–1.149*
	min		–1.417	*–2.721*	–0.927	*–2.231*
S22 (inner beam, top flange)	max	–2.391	0.065	*–2.326*	0.147	*–2.244*
	min		–0.893	*–3.284*	–1.386	*–3.777*

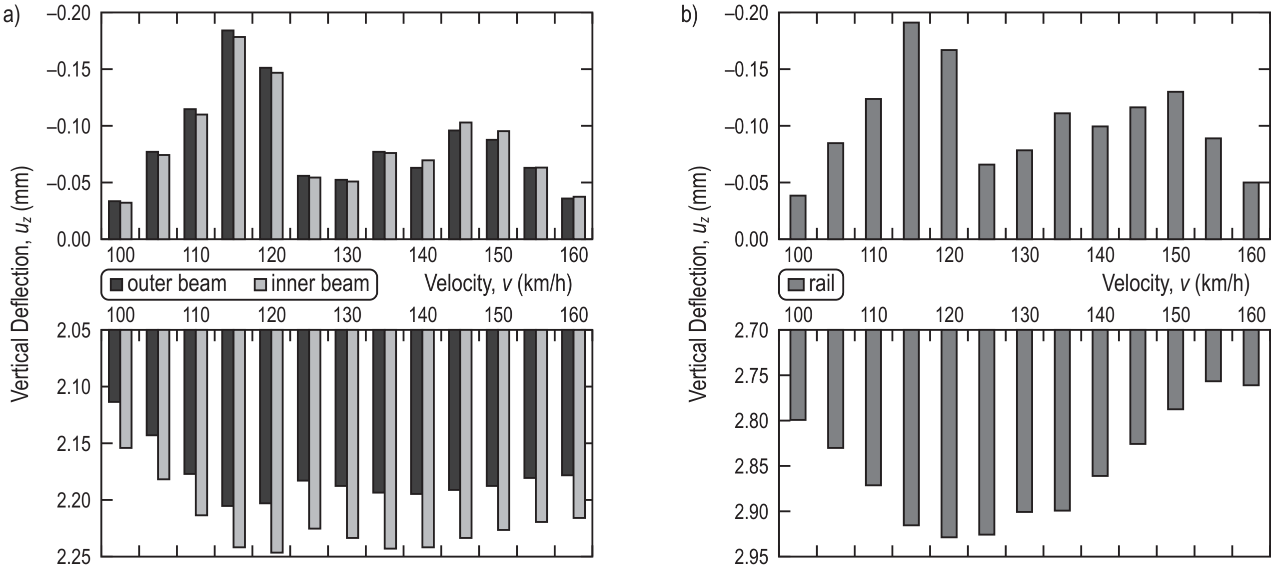

Fig. 12. Extreme values of the vertical deflection of the outer and inner main beam (a), and the rail (b) at the midspan

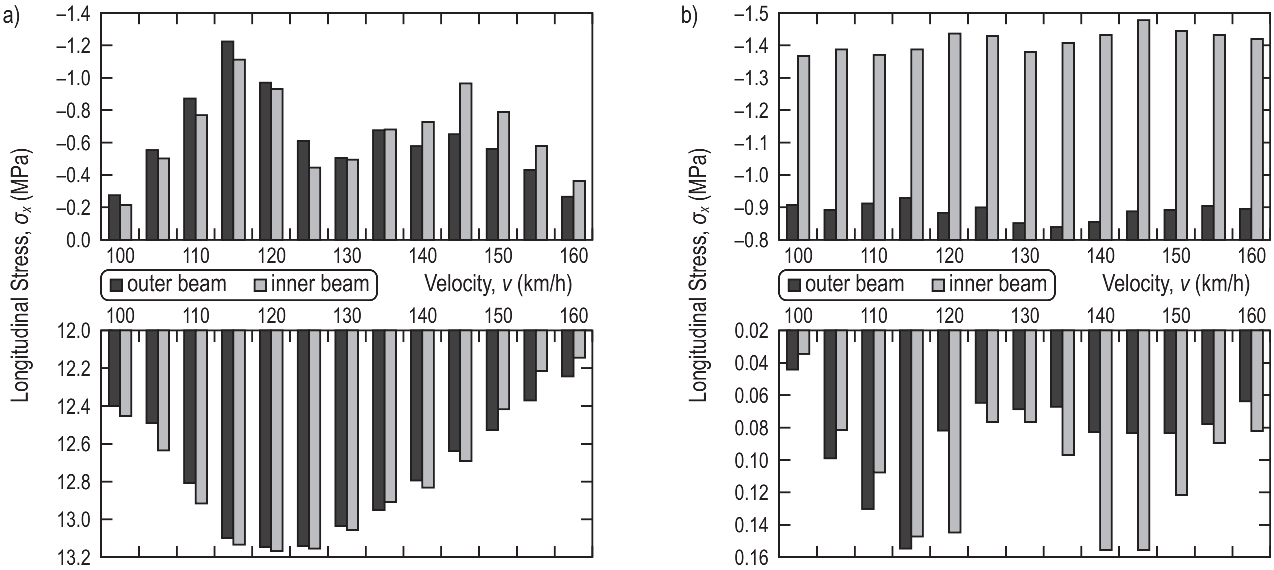

Fig. 13. Extreme values of the longitudinal normal stresses in the bottom (a) and the top (b) flanges of the main beams at the midspan

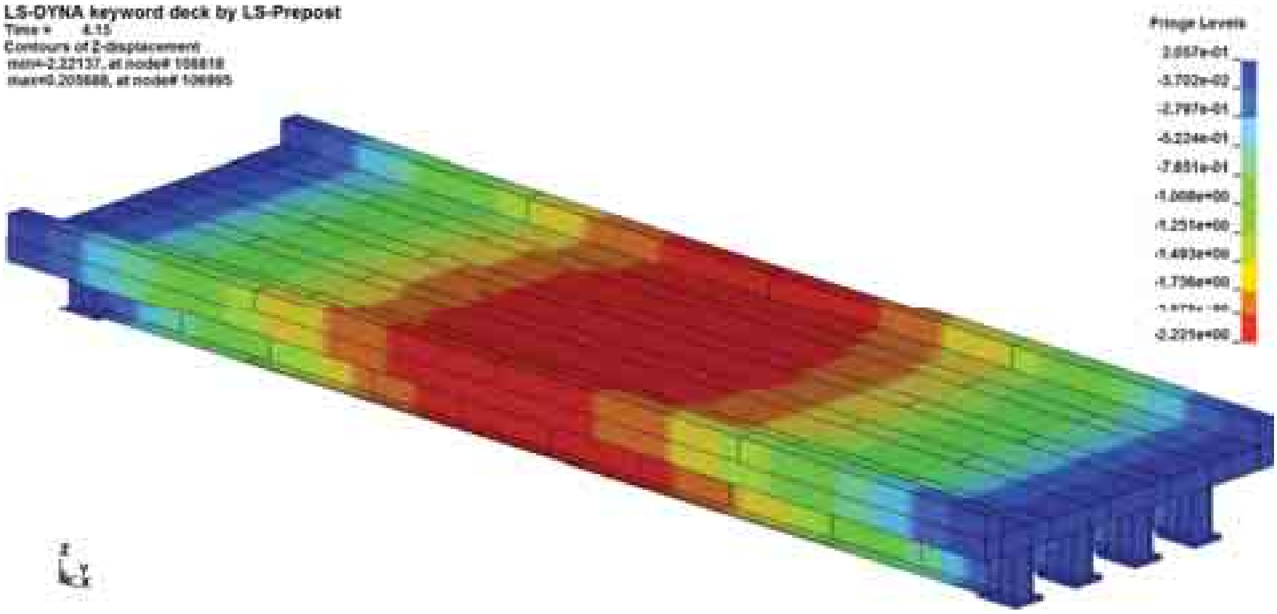

Fig. 14. The contour of the vertical displacement [mm] for the complete viaduct superstructure under the train moving at the speed of 115 km/h

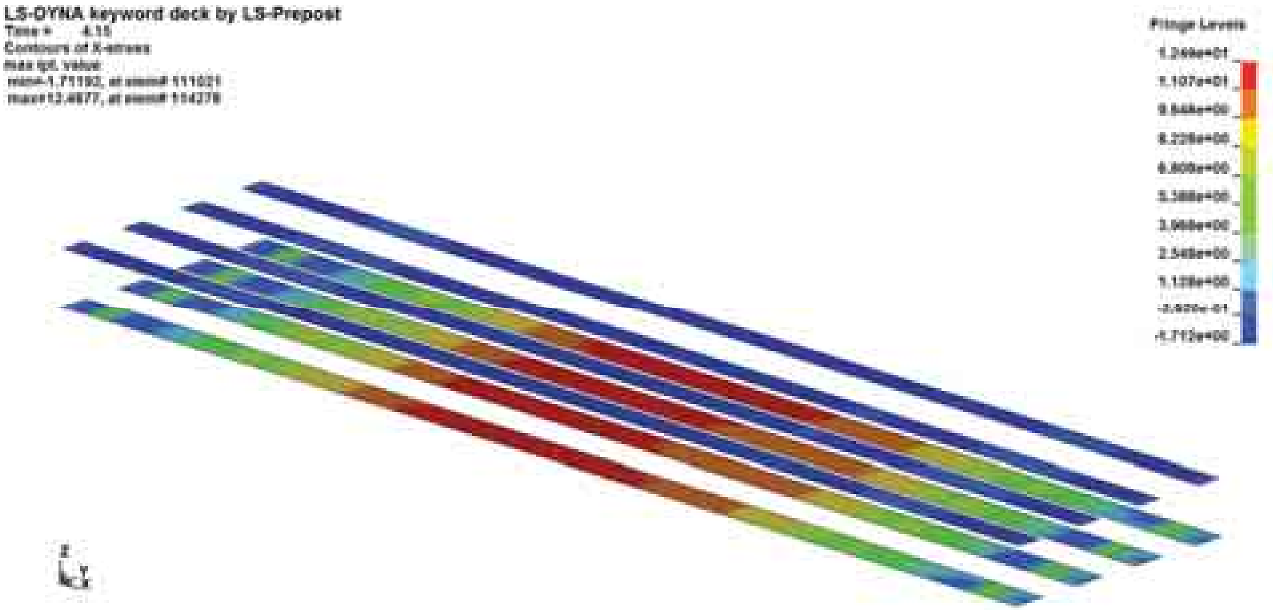

Fig. 15. The contour of the longitudinal normal stresses [MPa] in bottom and top flanges of the main beams under the train moving at the speed of 115 km/h

5. Conclusions

Selected results of the numerical analyzes presented in this study are related to the KNI 140070 viaduct on the Polish Central Main Line and the EC-114 *Praha* train, hence the conclusions cannot be generalized. Based on the simulation results, the following main conclusions can be formulated:

1) The dynamic response of the viaduct has the beam character with dominant influence of the first modal system.
2) There are no significant peaks in the extreme values of the deflection and stresses for the considered velocity range. Obtained results do not clearly indicate the occurrence of the resonant velocity.
3) Under the dead load, higher values of the longitudinal stresses in the bottom flanges appear in outer beams due to non-even weight distribution on the main beams. Load of the side walls of the RC platform slightly increase deflections and stresses in outer beams.

4) During the dynamic analyses, higher values of the longitudinal stresses in the bottom flanges appear in the inner beams since they are located almost directly under the rails. Therefore, the train load is distributed more on the inner beams.

Acknowledgments

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