FEM dynamic analysis of the hangers in a steel arch railway viaduct. Case study

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The article presents a dynamic analysis of a railway bridge located near Huta Zawadzka on the Central Rail Line. The analysis was mainly focused on the local behaviour of the hangers under the railway dynamic load.

Introduction

The bridge used in the following study is in Poland in km 26,541 of the Central Rail Line near Huta Zawadzka. The route stretches from Grodzisk Mazowiecki to Zawiercie and is a part of the European high-speed rail line E-65. The bridge consists of two separate structures under each truck of the rail line. The structure is a single span tied-arch bridge with vertical hangers and a deck as a strengthened cord. Each span has a length of 75 m and a total high of 14,92m. The bridge is made of steel and has a reinforced concrete ballasted deck. Basic materials used for construction are S355 steel, C40/50 concrete, and Y1880 steel for hangers. Both the arch girder and the tie beam have a box cross-section with dimensions of 1,53 x 0,73 m and 1,79 x 0,76 m. Cross-beams are double tee plate girders with a height of 0,7 m, placed each 5 m. The deck is a reinforced concrete slab with a thickness of 0,3 m connected with cross-beams and tie beams by shear connectors. The hangers are made of steel bars with ∅55 diameter.

Recently, several unfavourable phenomena related to the behaviour of arch spans under dynamic loading have been noticed [1–2]. The case concerns especially the vibrations of hangers and their influence on the load capacity. The analysis presented in the article shows the possibilities of predicting vibrations at the design stage.

Numerical model

To carry out analysis the numerical model was developed by using FEA software SOFiSTiK. The model was created with shell-beam elements in three-dimensional space (e1+e2, p3). Finite element types used in the model are quadrilateral plane elements for re-

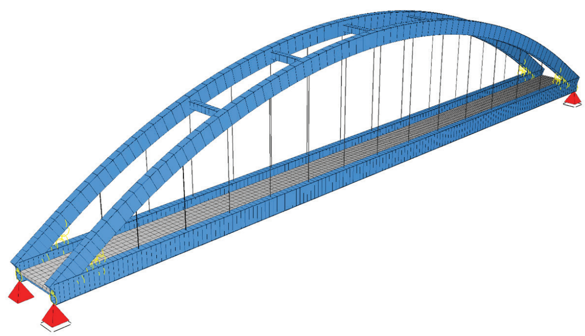

Fig. 1. Model visualisation

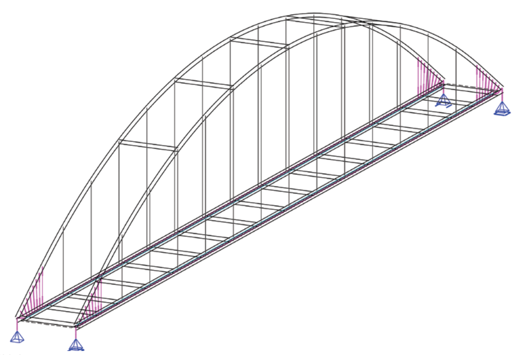

Fig. 2. Static scheme

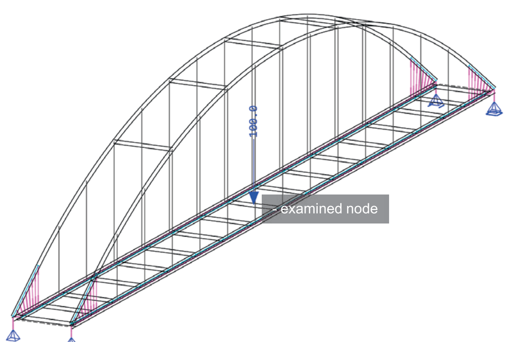

Fig. 3. Examined node during the impulse-based dynamic simulation.

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inforced concrete slab, Timoshenko beams for steel structure, and cable elements for hangers. Support conditions, arch girder-tie beam connection, and tie beam-stringer connection were modelled by using kinematic constrains of corresponding displacements and rotations of selected nodes. The analysis of the mesh convergence was performed and the size of the grid of 0.5 x 0.5 m was obtained for surface elements, 0.5 m for beam elements of the arch tie, cross-beams, and main girders, 1 m for beam elements of arch girder and top bracings, while hangers were made up as one cable element.

The computational model consists of 3906 nodes, 1248 quad elements, 1956 beam elements, and 26 cable elements. Fig. 1. shows model visualisation and fig. 2. shows static scheme. Support conditions were set as for simply supported beam.

Due to the lack of transverse stiffeners and diaphragms in beam elements, self-weight and mass distribution were corrected to the values from the execution project. Loads were defined following the Eurocode [3] as railway track weight. All analyses presented were performed on characteristic values. Initial forces in hangers were calculated assuming no vertical displacements of the deck under the dead weight. by using a module CSM (Construction Stage Manager – SOFiSTiK). Forces in shortest hangers were additionally adjusted to the value like in the longer hangers.

Subsequently, modal analysis was performed to determine the natural mode shapes and frequencies of the structure. In the analysis, the weight of the ballast of the railway track was considered as a mass participating in vibrations. The results are shown in tab. 1.

Linear dynamic analysis

In the next step, the dynamic parameters were appointed. Damping was determined using Rayleigh's mass- and stiffness-proportional damping. The damping ratio has been assigned with the value of $\xi=0.5\%$ and then verified using an impulse-based dynamic simulation. In the middle of the span, the force of 100kN was applied (fig. 3.), operating in 0.2 s (fig. 4.). The free response of the structure was studied for 10 seconds. Based on the calculations, a displacement-time graph has been obtained (fig. 5.). A trendline was added to the chart from which the logarithmic decrement with value LD=3.7% was determined (equal to $\xi=0,59\%$). A time step for calculations was set as 0.01 s based on convergence analysis of Newmark procedure.

After calibrating the damping in numeric model, the dynamic analysis was performed.

The works [4–8] present the possibilities of modelling the rolling stock analyses with given examples. It is commonly assumed to replace damped moving masses systems with series of point loads. Load train in following study was composed of two ES64U4 type locomotives and four passenger cars 154A (fig. 6.) , used for field tests of the damping spans [9]. The analysis was carried out using the DYNA module in the SOFiSTiK software. The equations of motion were integrated with the implicit Newmark's numerical integration algorithm. The linear system was examined with the numerical damping of high frequencies [10].

Tab. 1. Natural frequencies of the structure

Mode	Natural frequency [Hz]	Description of mode shape
1	1,18	Horizontal bending
2	1,67	Vertical bending
3	1,93	Second horizontal bending
4	2,78	Torsional
5	2,91	Second vertical bending

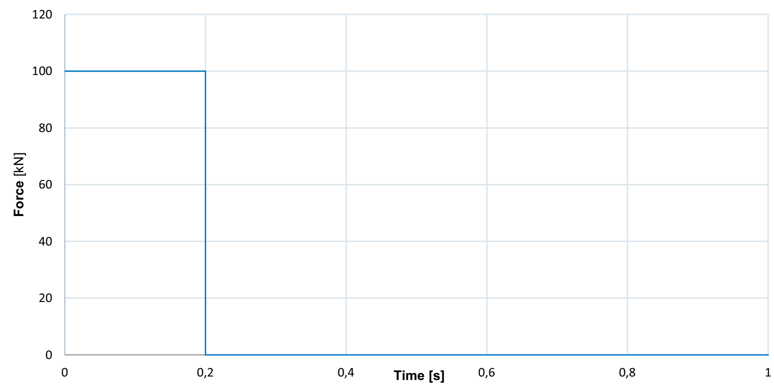

Fig. 4. Force-time graph

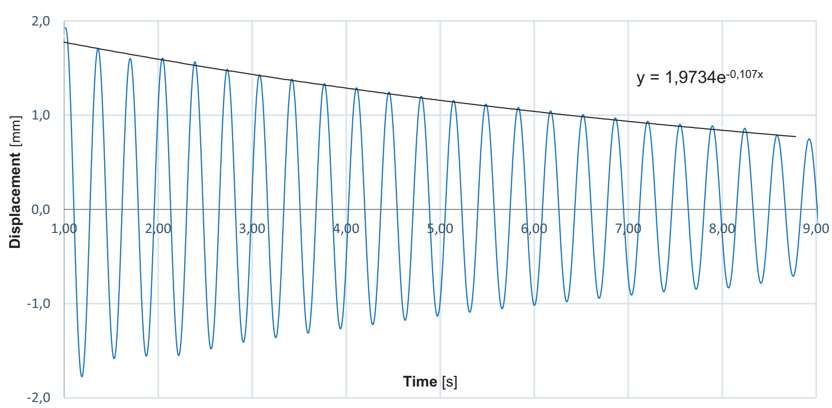

Fig. 5. Displacement-time graph from the impulse-based dynamic simulation

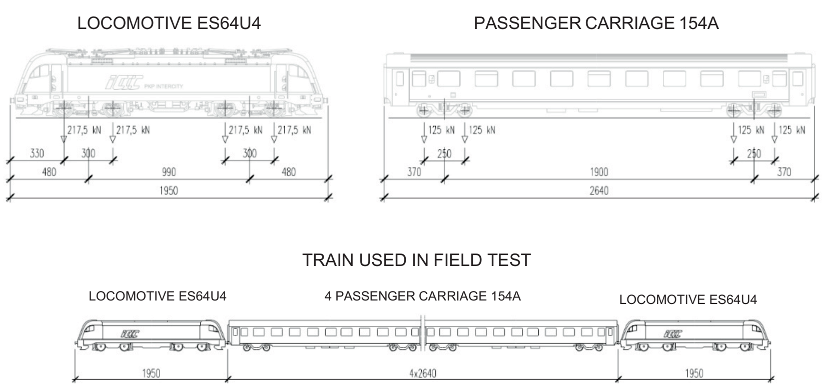

Fig. 6. Load train scheme use in field tests [9]

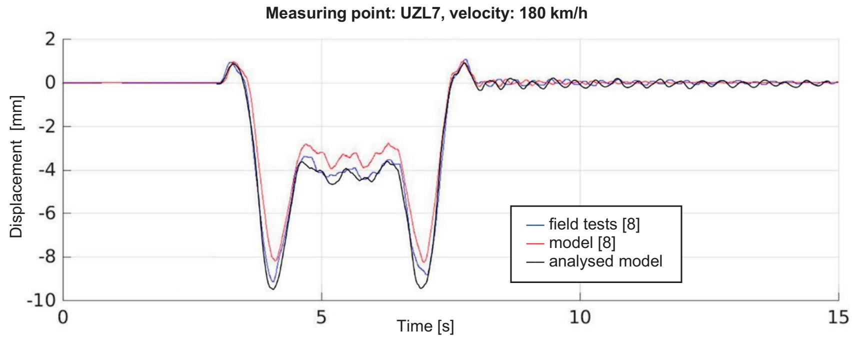

Fig. 7. Displacement-time graph for passing with a velocity of 180 km/h.

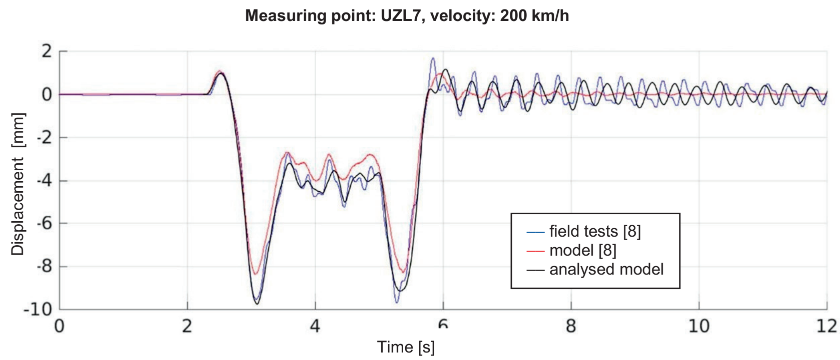

Fig. 8. Displacement-time graph for passing with a velocity of 200 km/h.

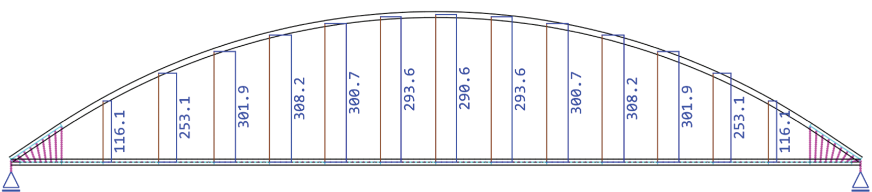

Fig. 9. Normal forces in the hangers without tension adjustment

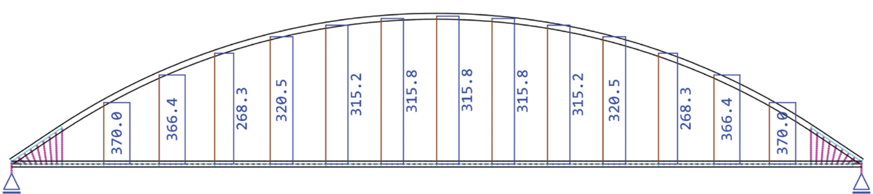

Fig. 10. Normal forces in the hangers with tension adjustment

Tab. 2. Comparison of the hanger natural frequencies

No of hanger	Natural frequency [Hz]			
	Without tension adjustment		With tension adjustment	
	Calc. by (4.1)	Numerical calc.	Calc. by (4.1)	Numerical calc.
W7	4,74	5,23	4,94	5,44
W6	4,83	5,35	5,01	5,53
W5	5,13	5,70	5,25	5,83
W4	5,67	6,35	5,78	6,47
W3	6,44	7,37	6,71	7,01
W2	7,33	8,85	8,82	10,30
W1	7,20	11,30	12,86	16,22

The developed model was subjected to comparative analysis with measurements from the field tests and FEM calculations presented in the study [9]. The compared model was developed in Abaqus software using beam elements (hangers, sleepers, and rails), shell elements (steel construction and hangers), surface elements (reinforcement), and solid elements (concrete slab, crushed stone). The total number of finite elements (and nodes) amounted to over 164 thousand.

The vertical displacement of the arc in the middle of the span was selected for the analysis. On fig. 7. and 8. a summary of the results obtained was presented. The model described in this article achieved the results almost coinciding with the values registered during field tests. In addition, the used model has achieved greater compliance with measurements than a more complicated model. Received charts indicate correct calibration of structural damping parameters.

Modal analysis of the hangers

In the following part of the article, attention was focused on the behaviour of hangers subjected to dynamic loads. To carry out the calculations, finite element types for hangers were changed, from a single cable element to 10 beam elements (including EJ). The boundary conditions were assigned as fixed support with an arch girder and a tie. Two types of construction technology were analysed. The first case assumes that normal forces in hangers are coming from dead load only, the second considers the addition of the tension adjustment. In the beginning, the differences between the normal forces occurring in hangers were examined. On fig. 9. and 10. the results are presented.

For both variants, a modal analysis was performed, considering pre-tension in the hangers.

Analytical calculations were made for comparison using equation (1). This equation is appropriate for the scheme of cable element with hinge support, with negligible bending stiffness, and with evenly distributed weight. The results are shown in tab. 2.

$$f_{si} = \frac{i}{2L} \cdot \sqrt{\frac{T}{m}} \quad (1)$$

where:

i – number of mode

L – length of the hanger [m]

T – force in the hanger [kN]

m – mass evenly distributed to length unit

In the table, the largest differences in the values occur in the shortest hangers where the main reason is the bending stiffness. Differences for the longest hangers are smaller and result mainly from different boundary conditions.

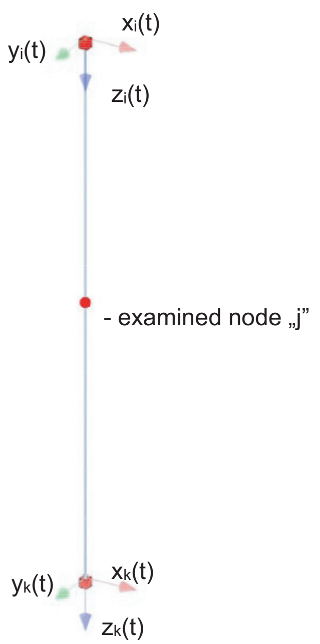

Fig. 12. Local model input loads

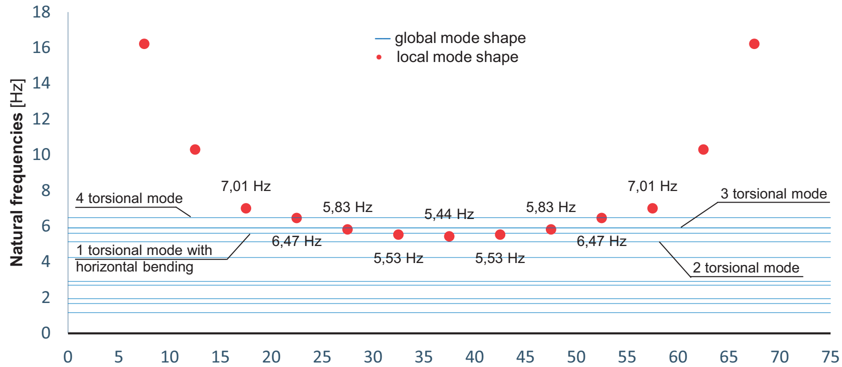

Fig. 11. Overlapping of the natural frequencies of the hangers and the structure from the global model

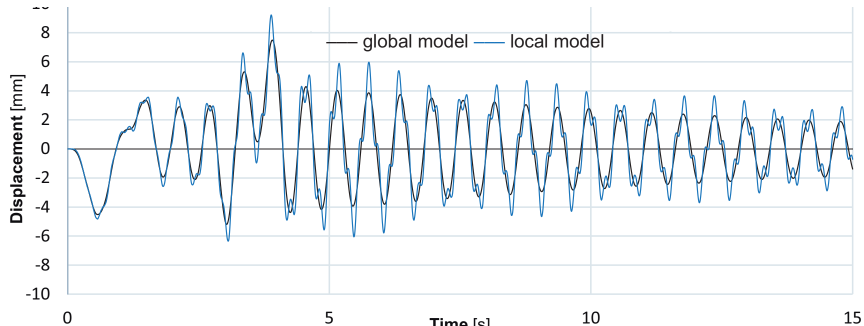

Fig. 13. Horizontal displacement-time graph for a central node of the hanger - passing with a velocity of 180 km/h

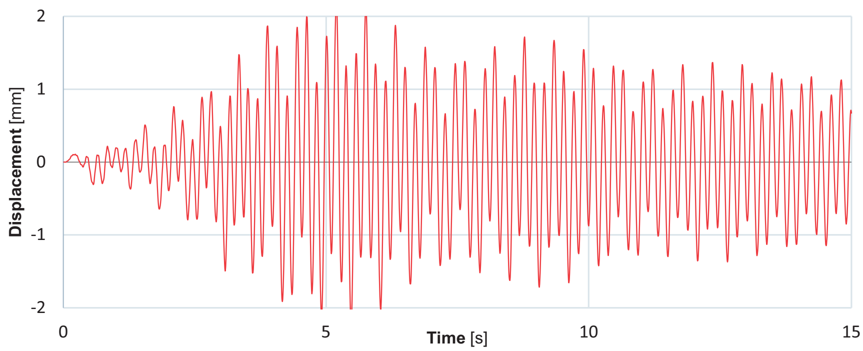

Fig. 14. Horizontal induced vibrations of the hanger - passing with a velocity of 180 km/h.

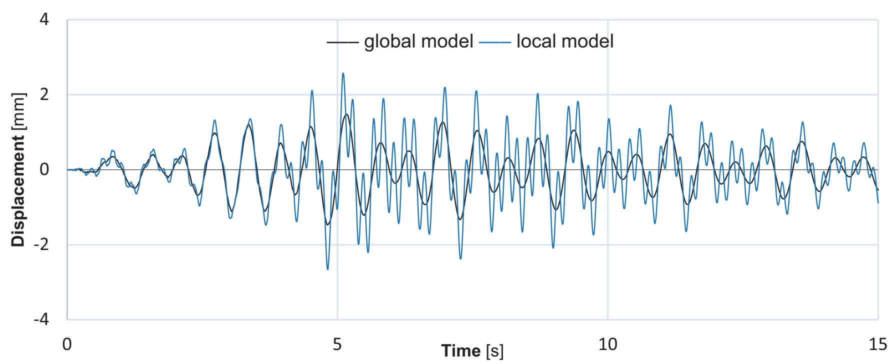

Fig. 15. Horizontal displacement-time graph for a central node of the hanger - passing with a velocity of 180 km/h.

For further studies, a model with hanger tension adjustment was used. On fig. 11. natural frequencies of hangers and structure from the global model are shown. The chart provides information on overlapping the frequencies of the central hangers with sensitive frequencies of the structure - regardless of the tension system used.

Nonlinear dynamic analysis

Due to the overlapping frequencies of the hangers and the structure, the impact of global vibrations of the structure on local vibrations of the hangers was examined. The research was carried out using non-linear dynamic analysis. A local hanger model was developed using updated geometry and forces from the global model (L=13,288m). The model consisted of 20 beam elements. The boundary conditions were assigned as fixed support at both ends. The damping ratio was set as ξ=0.2% which resulted in the value of logarithmic damping as LDT=0.94%.

The model was loaded with support displacements functions $x_{(i,k)}$ (t),$y_{(i,k)}$ (t),$z_{(i,k)}$ (t) obtained from linear rolling stock analysis on the global model (fig. 7., fig. 8.).The functions have been determined in each of the 3 directions in the global model with a time step of 0.01 s. The axle x is a longitudinal axis of the viaduct, the axle y is a transversal axis of the viaduct, while the axle z is a vertical axis.

Non-linear dynamic analysis was carried out using the ASE module [10] based on the classic Newmark method parameters. As a result, the imposed global and local vibrations of the hanger have been obtained. In the charts (fig. 13., fig. 15., fig. 17., fig. 19.) displacements of the middle node from the local model and the global model are shown. Next, both values were subtracted and charts of Induced vibrations in the hanger were obtained (fig. 14., fig. 16., fig. 18., fig. 20.). Calculations have shown a dynamic response in a direction perpendicular to the axis of the hanger. Vertical vibrations were not registered (in the direction of the hanger axis).

The presented charts show that the global vibrations of the structure from the train passage, cause the local vibrations of the 1–2 mm amplitude in the hangers.

Wind excitation of the hanger

The next phase of the work was to examine the vulnerability of hangers to the wind excitation. Assessment was made on the

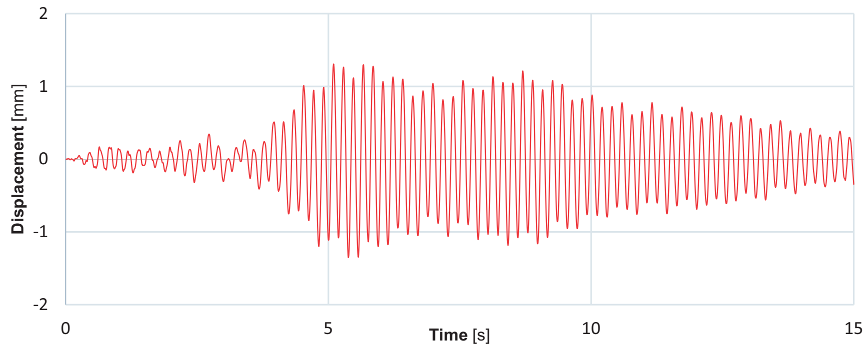

Fig. 16. Horizontal induced vibrations of the hanger - passing with a velocity of 180 km/h.

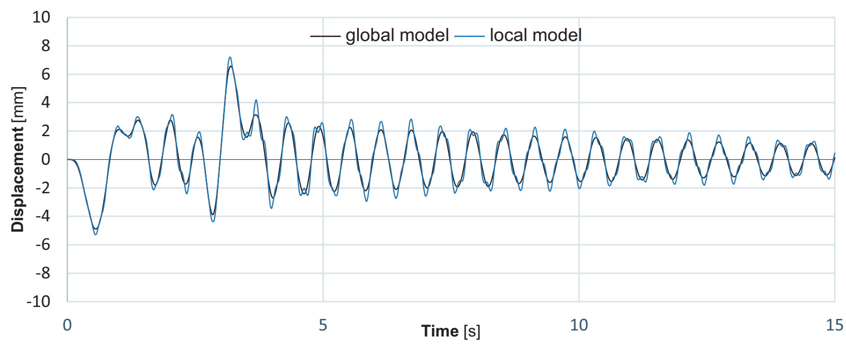

Fig. 17. Horizontal displacement-time graph for a central node of the hanger - passing with a velocity of 200 km/h.

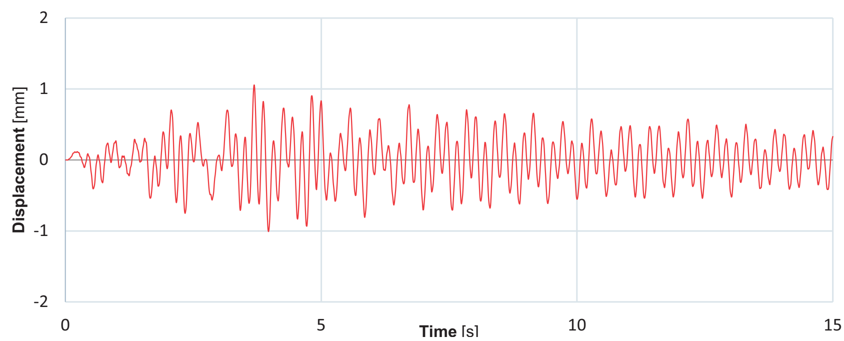

Fig. 18. Horizontal induced vibrations of the hanger - passing with a velocity of 200 km/h.

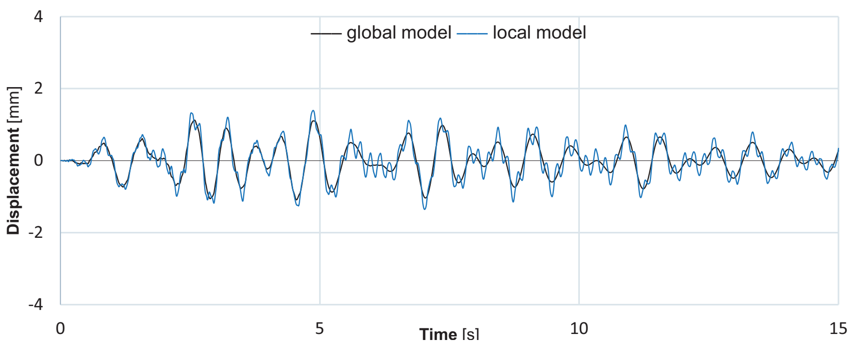

Fig. 19. Horizontal displacement-time graph for a central node of the hanger - passing with a velocity of 200 km/h.

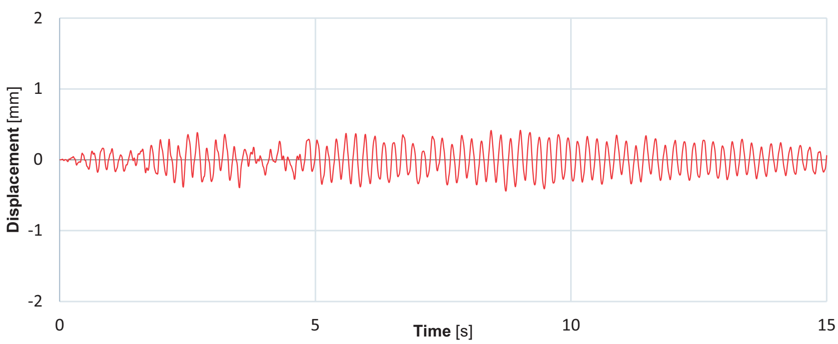

Fig. 20. Horizontal induced vibrations of the hanger - passing with a velocity of 200 km/h.

base of von Kármán's vortices-induced vibrations. The analysis was carried out according to the DIN standard [11]. Based on the procedure (equations (2) and (3)), the load was determined, which was used in the FEM calculations.

$$(t) = 0{,}7 \cdot \mathrm{D} \cdot \frac{v_{crit,i}^{2}}{1600} \cdot \sin(2 \cdot \pi \cdot f_i \cdot t) \cdot k_{F,i} \quad (2)$$

$$v_{crit,i} = \frac{f_i \cdot \mathrm{D}}{St} = \frac{5{,}4 \cdot 0{,}055}{0{,}2} = 1{,}47\,\frac{m}{s} \quad (3)$$

where:

0,7 – excitation coefficient for round hangers

D – diameter of the hanger [m]

$v_{(crit,i)}$ – critical wind speed for a given frequency [m/s]

f_i – natural frequency of the hanger [Hz]

$k_{(F,i)}$ – coefficient considering the decrease in exciting force at higher frequencies ($k_{(F,i)}$=1,0 for f_i<7 Hz)

St – Strouhal number (St=0,2 for round hangers)

The calculations were carried out for the first natural frequency of f_i=5,40 Hz, the diameter of the hanger was taken according to the project as D=0,055 m, and the designated critical speed was $v_{(crit,i)}$ = 1,47 m/s. The load width was set following the DIN standard [11] as L=24D=1,305 m. In fig. 21. a load diagram with the maximum amplitude is presented. The purpose of the calculation was to find the maximum amplitude of the vibrations of the hanger. Computation was processed on the base of theory and mechanical assumptions specified in part Nonlinear dynamic analysis.

The maximum displacement of the hanger was 0,043 mm and occurred after a period of 7 s of induced vibrations. The net force of the wind was 0,067 N. Presented analysis shows that the rolling stock induced vibrations are significantly bigger than the vortices-induced vibrations. As a result of vibrations, there was a slight change in the normal force in the hanger reaching 0,6 kN.

Conclusions

The presented article shows the effectiveness of non-linear dynamic analysis in the design process or at the stage of detailed expertise. Properly calibrated simple FEM models allows to obtain detailed information on the behaviour of structures without a lot of time commitment to dynamic analysis. A more detailed analysis of the interaction between the global and local model can be a basis for further scientific considerations. However, the simplifications resulting from the applied simulation of a train running over the bridge describe the dynamic vertical response of the structure sufficiently (from the technical point of view). The undoubted challenge is to develop equivalent loads representing horizontal dynamic loads transverse to the direction of traffic.

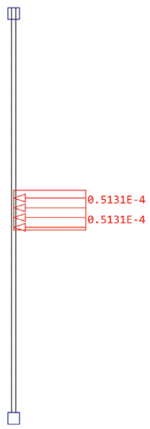

Fig. 21. Load scheme [kN/m]

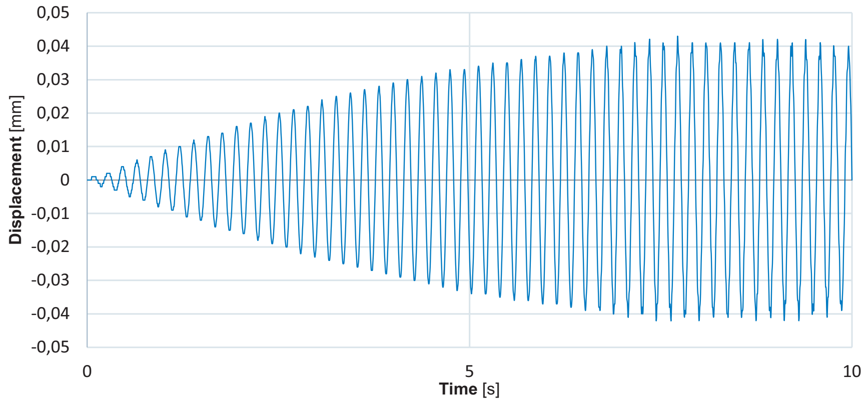

Fig. 22. Displacement-time graph of the vortices-induced vibrations

As a result of the analyses, the following conclusions can be formulated:

The technology of the hangers application theoretically has little influence on their dynamic properties. Nevertheless, welded rod hangers are susceptible to assembly errors that can be eliminated using pin connected, pre-tensioned systems.

In the case of short hangers, the bending stiffness and boundary conditions significantly affect the natural frequencies and their shape.

In the studied case, with the correct tension of the hanger, no significant transverse vibrations caused by the passing train should occur.

In the studied case, assuming correct tension in the hangers, a significant transverse vibration caused by wind should not occur,

Negative dynamic behaviour of the hangers in a tie arch bridge may result locally from insufficient tension under dead load. Such a situation may occur because of objective assembly imperfections. In case pre-tensioned hangers, this effect should not occur. However, dynamic simulations of the effect of transverse loads on the dynamic response of the structure may provide new information on the dynamics of hangers in an arch bridge.

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CORRECT METHOD OF QUOTATION

Bleja Marek, Żółtowski Krzysztof, 2021, FEM dynamic analysis of the hangers in a steel arch railway viaduct. Case study, „Builder" 11 (292). DOI: 10.5604/01.3001.0015.4166

Abstract: FEM dynamic analysis of the hangers in a steel arch railway viaduct. Case study The article presents a dynamic analysis of a railway bridge located near Huta Zawadzka on the Central Rail Line. The analysis was mainly focused on the local behaviour of the hangers under the railway dynamic load. The calculations were performed on the FEM model. The efficiency of the model was assessed by comparing it with the measured dynamic response of the structure. The nonlinear dynamic analysis was conducted to develop the behaviour of the hanger (tension element). Afterwards, the dynamic response of the hanger ware evaluated during the passing train and wind excitation. Following study was a part of the master thesis.

Keywords: dynamic analysis, FEM analysis, Cable dynamic properties, Newmark method

Streszczenie: NUMERYCZNA ANALIZA DYNAMICZNA WIESZAKÓW W ŁUKOWYM WIADUKCIE KOLEJOWYM. Analiza przypadku W artykule przedstawiono analizę dynamiczną wiaduktu kolejowego w ciągu Centralnej Magistrali Kolejowej zlokalizowanego koło Huty Zawadzkiej. W analizie zwrócono szczególną uwagę na lokalną odpowiedź wieszaków na obciążenie przejeżdżającym pociągiem oraz wiatrem. Obliczenia przeprowadzono na podstawie wykonanego modelu MES. Zweryfikowano poprawność modelu poprzez porównanie uzyskanych wyników z wartościami pomierzonymi. Przeprowadzono nieliniową analizę dynamiczną badania zachowania wieszaków. Następnie porównano amplitudy drgań wieszaka podczas przejazdu taboru oraz w sytuacji wzbudzenia wirami von Kármána. Analizy zostały przeprowadzone przy okazji realizacji pracy magisterskiej.

Słowa kluczowe: analiza dynamiczna, analiza MES, drgania wieszaków, metoda Newmarka