

Static and modal analysis of Solidarity Bridge in Płock

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Abstract: Cable-stayed bridges are stunning structures, which thanks to their dimensions and shapes fascinate not a single observer. Designing such an object is awesome challenge for designers. These structures must transfer forces resulting from dead load, operational loads, temperature changes and coming from wind pressure. Nowadays, each cable-stayed bridge and suspension bridge must be subjected to special analysis to guarantee safe usage. Designers cannot allow the situation, which took place on November 7th, 1940 in north-western part of the United States of America near the town Tacoma, where the collapse of the suspension bridge occurred. The structure was destroyed as the result of coincidence of natural vibrations and the dynamic wind action. Authors of the paper undertake the task to provide dynamic analysis of Solidarity Bridge in Płock exposed to the wind action. The mentioned structure is the longest cable-stayed bridge in Poland and at the same time, the longest in the world with the arch suspended at one axis to the columnar pylon fixed to a platform. Additionally static analysis is made in order to verify horizontal and vertical displacements as the result of combined variable loads.

Keywords: FEM, modal analysis, natural frequency.

1. Introduction

The aim of this study is to analyse the Solidarity Bridge in Płock and to examine the response of the structure to various kinds of impacts, i.e. : dead load, weight of cars and wind impact. This types of analyses are very important issues from social and environmental perspectives. Each road user utilising the construction must feel safe and driving comfort. Cable-stayed and suspension bridges often are referred to as "icons" of the cities where they are situated or to which they give access. These structures attract tourists, enthusiasts, engineers or constructors from around the world and therefore must be incorporated into the surroundings and adapted to the existing environmental conditions. These huge constructions facilitate long distance travels that were once impossible to carry out. They are a very important point in the development of communication, overcoming ethnic, social, cultural barriers. They allow urban development in terms of economy, tourism, science. They also become an inherent, integral part of a city that is directly identified with it.

The study includes two analyses: static, through which stress values and values of construction displacement were received and modal which enables to gather ten frequencies and types of vibrations of natural frequencies. Using the analyses mentioned above, the right choice of cross-section parameters and dimensions of individual components of the bridge is possible. It has a very large impact on the optimal use of construction materials. Reducing the amount of materials used to build a bridge by even a few percent has a

positive effect on the environment. Moreover, the final aesthetic effect on the structure of the bridge is a direct result of the above analyses, which confirm that the safe usage of the bridge in this form is possible.

On the basis of the results, Ultimate and Serviceability Limit States depending on the combination of loads were checked. The analysis was carried out in Autodesk Simulation Multiphysics 2012. For the purpose of the analysis, six calculation scenarios, which are described in chapter 2.1.1., were created.

2. Description of the structure

The Solidarity Bridge in Płock is an engineering structure of cable-stayed construction. It was divided into two parts at the design stage: main bridge of a length of 615,00 m and access bridge over flooded area with a total length of 585,00 m. The following analysis concerns only the main bridge.

Cross-section of the bridge is a steel, three-chamber box with height fixed in relation to the lower and upper decks. The overall width of the cross-section is 27.50 m, 16.50 m of which is the upper part of the box and the two cantilevers with a total length of 5.50 m. The width of the bottom deck is 13.00 m. Following elements can be identified on the traveled way [2]:

- two carriageways separated by median strip, each with a width of 8.80 m,
- pavements along both sides of the road with a width of 2.50 m,
- median strip with a width of 2,50 m.

Upper strip is an orthotropic deck, reinforced with closed section longitudinal ribs in carriageways zones and open longitudinal ribs with flats in places where pavements and central reserve are located.

Pylons of column type are all made of steel. They have variable cross-section throughout its height - the widest where fastened to a span. The height of the pylons above the carriageway is 63,67 m. Ladder passageways are located inside the pylons, which enable continuous checking of the condition of the anchoring.

Longitudinal profile consists of five spans: the main bridge span, suspended with a length of 375.00 m and four side spans with a length of 60.00 m each. The bridge has a longitudinal slope ratio of 0.50 % .

The Solidarity Bridge uses asymmetric suspension system, called the fan. The main cable-stayed span was suspended through the use of fourteen pairs of tendons, 7 coming out of each pylon. Suspending tendons were anchored in the main span in modular spacing ratio of 22.50 m, with the exception of the first one, 41.25 m far from the pylon. Side spans were suspended with the use of tendons at a spacing equal to 15.00 m, also with the exception of the first, which was fixed 30 00 m far from the pylon.

2.1. Description of the calculation model of the bridge

The calculation model was carried in Autodesk Simulation Multiphysics 2012 Student Version. Created model of bridge geometry was classified for ($e^1 + e^2$; p^3) class in accordance with [1]. The symbols e^1 and e^2 mean the dimension of the components and p^3 symbol indicates dimension of the space in which the model was made. At the creation stage of the model the following components were used:

- shell/plate,
- beam,
- truss.

The model consists of 893 694 elements, of which 643 257 are plate components, 250 381 beam components and 56 truss components. There are many components in the model that are not connected axially. In order to model such connections, the authors used an OFFSET option for the actual data mapping of the connection of components by shifting the axes which connect the centres of balance of cross-sections of the components, in the indicated location. The described model is presented below in figures 1 to 6.

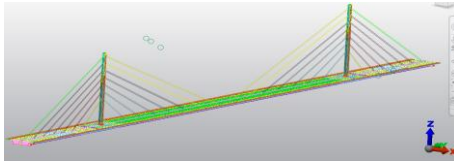


Fig. 1. Axonometric view of the calculation model

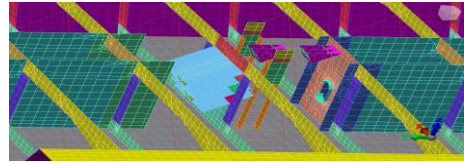


Fig. 2. Anchoring blocks view

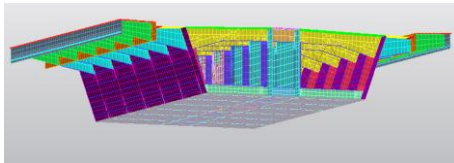


Fig. 3. One of the mounting units of the main bridge span

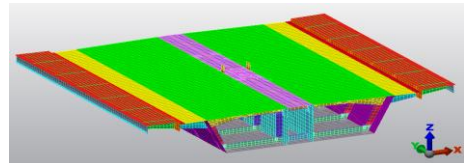


Fig. 4. View of the mounting unit of the main bridge span

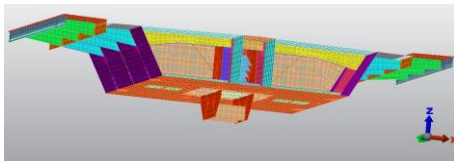


Fig. 5. View of the segment located above the support no. 2 with visible elements anchoring the span in the support by applying the tensioning tendons

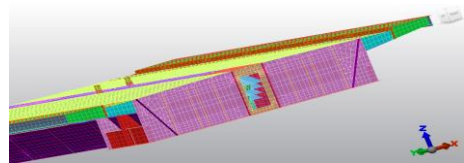


Fig. 6. Mounting unit no. 1 with the support specially added, anchoring the segment in the support

It is not possible to add pretension force to the tendons in the Autodesk Simulation Multiphysics at the stage of linear analysis (Static Stress with Linear Material Models). This operation is available only in the non-linear analysis (Static Stress with Non Linear Material Models). Unfortunately, due to the fact that the model was very complex, it was impossible to carry out a non-linear analysis. Pre-tensioning of the cable-stay in the linear analysis was possible by loading the cable-stay using temperature. Authors using Hooke's Law and thermal expansion law, saw a correlation between stress values and temperature. The following formulas describe the correlation mentioned above.

$$\sigma = E \cdot \varepsilon = E \cdot \alpha \cdot \Delta t, \quad (1)$$

$$\sigma = \frac{F}{A}, \quad (2)$$

$$\Delta t = \frac{F}{E \cdot A \cdot \alpha}. \quad (3)$$

where: E – The Young's modulus [MPa], α – coefficient of thermal expansion [1/K], F – value of the pulling tension on a tendon (kN), A – surface area of tendon's cross-section [m²].

2.1.1. Loading combinations

Four calculation scenarios, which, according to the authors, may have a significant impact on the value of the internal forces in the structure, were used in the analysis. Leading (fixed) load is dead load that occurs in each combination. In addition, variable loads caused by weight of the vehicles and wind as well as load from the weight of the pavement were included. Combination of loads in the event of persistent or transient calculation scenarios in accordance with chapter 6 of the standard [3] is determined by the formula $\sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \psi_{0,i} Q_{k,i}$ in case of checking the Ultimate Limit State (ULS) and the formula $\sum_{j \geq 1} G_{k,j} + \psi_{1,1} Q_{k,1} + \sum_{i > 1} \psi_{2,i} Q_{k,i}$ in case of checking the Serviceability Limit State (SLS).

Preferred combinations of Ultimate Limit States (ULT) are:

- dead load + live load according to PN-EN 1991-2:

$$\gamma_G G_k + \gamma_Q Q_k + \gamma_q q_k, \quad (4)$$

- dead load+ live load according to PN-85/S-10030:

$$\gamma_G G_k + \gamma_Q Q_k + \gamma_q q_k, \quad (5)$$

- dead load + wind load:

$$\gamma_G G_k + \gamma_w Q_{kw}, \quad (6)$$

- dead load + load of the weight of the pavement:

$$\gamma_G G_k + \gamma_G G_{kn},$$

Preferred combinations of the Ultimate Limit States (ULT) are:

- dead load + live load according to PN-EN 1991-2:

$$G_k + \Psi_{1,1} Q_k + \Psi_{1,2} q_k, \quad (7)$$

- dead load + live load according to PN-85/S-10030:

$$G_k + \Psi_{1,1} Q_k + \Psi_{1,2} q_k, \quad (8)$$

- dead load + wind load:

$$G_k + \Psi_{1,1} Q_{kw}, \quad (9)$$

- dead load + load of the weight of the pavement:

$$G_k + G_{kn}, \quad (10)$$

The accompanying variable impacts were not taken into account in the combinations above. Every combination consists of dead load and predominant variable impact.

γ_G – the partial coefficient for fixed effects taking into account the uncertainty of the model and changes in dimensions, γ_Q – the partial coefficient for variable effects taking into account the uncertainty of the model and dimensional deviation, γ_q – the partial coefficient for variable effects taking into account the uncertainty of the model and

dimensional deviation, γ_w – the partial coefficient for wind impact, G_k – the characteristic value of the fixed effect, G_{kn} – the characteristic value of the load of the pavement weight, Q_k – The characteristic value of the variable effect, $\Psi_{1,1}$ – the coefficient for the frequent variable impact 1, $\Psi_{1,2}$ – the coefficient for the frequent variable impact 2,

2.1.2. The load modelling

The static analyses with the following models were carried out:

- Load model of dead load,
- Load model of dead load and load of the pavement weight,
- Load model of the main span of variable load according to PN-EN 1991-2 (LM1 Model),
- Load model of the main span of variable load according to PN-85/S-10030 (LM1 Model),
- Load model of the main span of the weigh of thirty two Tatra T-815 vehicles,
- Load model of the wind load according to PN-EN 1991-1-4.

Each of the models and the results of the static analysis are described in more detail below.

1) Model 1 and 2 - dead load and dead load with the load of the pavement weight.

The value of the dead load was automatically taken into account by the program on the basis of the given geometrical characteristics and material components included in the model. Due to lack of detailed information concerning thickness of the pavement layers, the authors have assumed that:

- main bridge span was loaded with mastic asphalt pavement 55 mm thick,
- side spans were loaded with a layer of mastic asphalt pavement 70 mm thick.

The bitmap showing the resultant displacements of the structure resulting from the dead load is shown in Fig. 7 below and the bitmap showing the displacements of the span resulting from the dead load with the load of the weight of the pavement is presented in Fig. 8. The results of the displacements in the direction of the axis X, Y and Z are presented in Tab. 1.

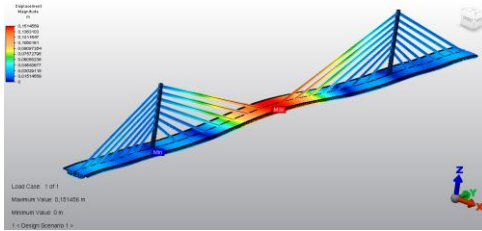


Fig. 7. Resultant displacements resulting from the dead load

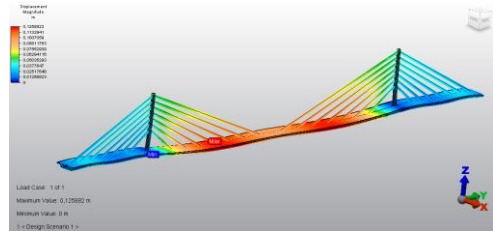


Fig. 8. Resultant displacements resulting from the dead load and load of the weight of the pavement

Table 1. The Results of the dead load (G) and dead load with load of the weight of the pavement (G + G_n)

	Maximum displacement in direction Y [m]	Maximum displacement in direction Z [m]	Maximum axial force in stay cables (kN)
G	0.020	0.151	4478.340
G + G _n	0.044	-0.126	5003.660

2) Load model of the main span of variable load according to PN-EN 1991-2 (LM1 Model).

The Standard [5] concerns the variable loads on bridges with a maximum span spread of 200 m. The main bridge span of the Solidarity Bridge is 187,5 % of this value. Primary load model in this standard is so-called the LM1 model. In view of the value of the bridge main span, it can be concluded that such long spans should not be loaded using this model. Nevertheless, the authors decided to apply forces corresponding to the LM1 model on the entire length of the main bridge span, in order to analyse the response of the structure under the influence of such an extreme load. The whole main span was loaded with the load originated from the UDL system (load evenly distributed, of 9 and 2,5 kN/m² - depending on the number of the lane) and the tandem system TS (load in the form of concentrated forces) in the middle of the span. The values of the resultant displacements of the structure under the influence of the applied load is shown in Fig. 9 and Fig. 10 presents the axial forces. Tab. 2 shows the maximum values of displacements of the structure in different directions.

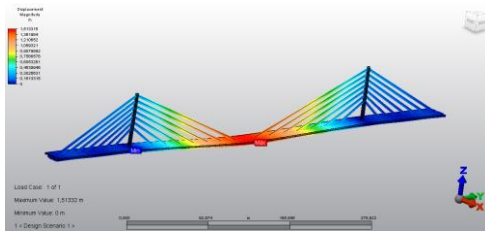


Fig. 9. Resultant displacements resulting from the dead load

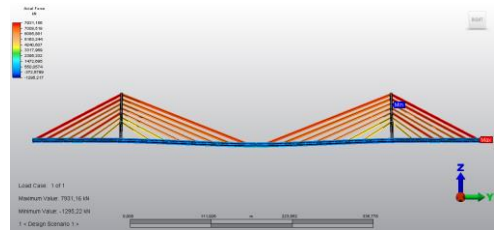


Fig. 10. Axial forces in the structure resulting from the load using LM1 model

Table 2. Results of the load using LM1 model

	Maximum displacement in direction Y [m]	Maximum displacement in direction Z [m]	Maximum axial force in stay cables (kN)
LM1 model	0.289	-1.513	7931.160

3) Load model of the main span of variable load according to PN-85/S-10030 (K+q),

The Solidarity Bridge in Plock was also loaded according to the standard [7]. The standard sets no rules concerning how such large bridge structures should be loaded. The authors decided to load the model analogically to LM1 model. Diagram of the application of force is shown on the diagram below (Fig. 11) and summary results of the displacements, in Tab. 3.

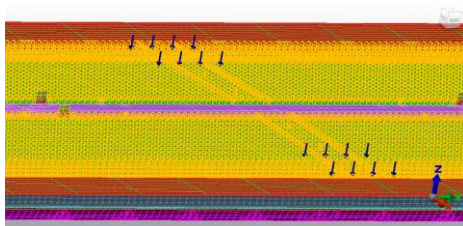


Fig. 11. Scheme of the load using K + Q model

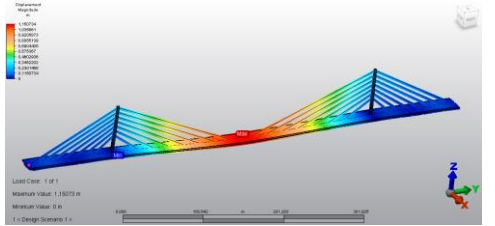


Fig. 12. Resultant displacements resulting from the load of scheme no. 2

Table 3. Results of the load using K+q model

	Maximum displacement in direction Y [m]	Maximum displacement in direction Z [m]	Maximum axial force in stay cables (kN)
K+q model	0.231	-1.151	7234.243

4) Load model of the main span of the weigh of thirty two Tatra T-815 vehicles,

According to the book [2], the model was loaded in the middle of a span, using additional thirty two Tatra T-815 vehicles. In the absence of data on the loads used to load the model created by the team of University of Gdańsk and lack of data on the exact location of application of the force, the authors adopted a method to load the model as it is shown in the following illustrations.

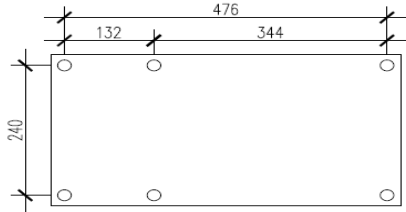


Fig. 13. Vehicle wheelbase of Tatra T-815

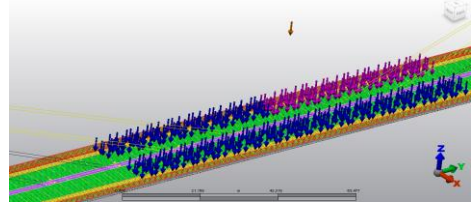


Fig. 14. Visualization of the load of the span of Solidarity Bridge

In the absence of any detailed data on loads, the model, presented in this paper, was loaded by a set of three pairs of concentrated forces with values calculated on the basis of the following formula:

$$F = \frac{m \cdot g}{6} = \frac{30000 \text{kg} \cdot 10 \frac{\text{m}}{\text{s}^2}}{6} = 50 \text{kN} \quad 11).$$

where: m – gross vehicle weight of Tatra T-815, g – approximate value of acceleration gravity. The values of displacements of the structure in various directions are presented in Tab. 4.

Table 4. Results of the load using load of the weight of the vehicles

	Maximum displacement in direction Y [m]	Maximum displacement in direction Z [m]	Maximum axial force in stay cables (kN)
32xTatra, T-815	0.287	-1.517	7947.690

5) Load model of the wind load according to PN-EN 1991-1-4.

Wind load is one of the basic loads operating on cable-stayed bridges. In analysis described in this paper, the authors took note of the wind load directed perpendicular to the longitudinal profile (X axis) and perpendicular to the horizontal area of the span (Z axis). In order to calculate the forces that impacts the bridge, calculations of the following values were necessary [4]:

- basic wind speed v_b ,
- basic wind pressure q_b ,
- coefficient of land surface k_r ,
- roughness coefficient of land surface c_r ,
- average wind speed v_m ,
- turbulence intensity I_v ,
- peak wind pressure speed q_p ,
- adoption of the aerodynamic coefficients c_x, c_z ,
- calculation of the value of a design coefficient $c_s c_d$,
- calculation of the value of the wind pressure to the structure F_w .

$$F_w = c_s c_d c_f q_p(z_e) A_{ref} \quad 12).$$

where: A_{ref} – is a surface area influenced by the strength of the wind.

The location of force application on the deck and visualization of displacement of the top of the pylon under load wind, is shown on figures below.

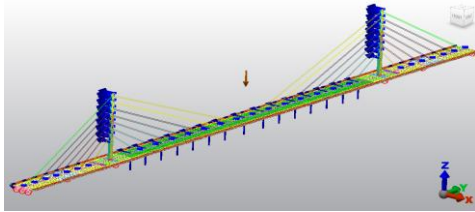


Fig. 15. Visualization of load wind force application

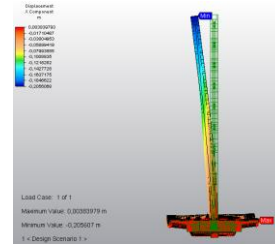


Fig. 16. Displacement of the structure

The cumulative results for the model with impacts of dead load, surface load and wind load are presented in Tab. 5.

Table 5. Results including the dead load and load of the weight of the pavement ($G+G_N$) and the load caused by the wind W_d

	Maximum displacement in direction X [m]	Maximum displacement in direction Y [m]	Maximum displacement in direction Z [m]	Maximum axial force in stay cables (kN)
$G+G_n + W_d$	0.206	0.021	-0.051	4658.260

2.1.3. Analysis of the results

The results obtained from each load model were compared with the limit values. Serviceability Limit State (SLS) was checked for main bridge span, and pylons and Ultimate Limit State (ULT) for stay cables. Table 6 lists the results and the analysis. The Serviceability Limit State was carried out.

Table 6. Serviceability Limit State – cumulative statistics

	Location	Permissible standard displacements [m]		Displacement value [m]
		Formula	Value (m)	
G	Middle of the span	L/300	1.250	0.151
	Top of the pylon	H/300	0.223	0.020
G + G_n	Middle of the span	L/300	1.250	0.126
	Top of the pylon	H/300	0.223	0.044
LM1	Middle of the span	L/300	1.250	1.513
	Top of the pylon	H/300	0.223	0.289
K + q	Middle of the span	L/300	1.250	1.151
	Top of the pylon	H/300	0.223	0.231
32xT-815	Middle of the span	L/300	1.250	1.517
	Top of the pylon	H/300	0.223	0.287
Wind	Middle of the span	L/300	1.250	-0.051
	Top of the pylon	H/300	0.223	0.206

Ultimate Limit State (ULT) was checked in accordance with the standard [6] concerning the design of tendon's structures. Tendons were qualified to group C. Bearing capacity for this group of tendons is as follows [6]:

$$\frac{F_{Ed}}{F_{Rd}} \leq 1, \quad (13)$$

where: F_{Ed} - declared design value of the longitudinal force in the component, F_{Rd} - declared design value of bearing capacity during stretch. The design value of bearing capacity during stretch F_{Rd} is determined in accordance with the following formula:

$$F_{Rd} = \min\left(\frac{F_{uk}}{1,5\gamma_R}, \frac{F_k}{\gamma_R}\right), \quad (14)$$

where: F_{UK} - specific value of the breaking strength, F_k - specific design value of bearing capacity during stretch, γ_R - partial factor. The force F_{UK} was calculated on the basis of the formula (15):

$$F_{uk} = A_m f_{uk}, \quad (15)$$

where: A_m - area of a metal cross-section, f_{uk} - specific value of tension strength of the bars, wires or splines according to appropriate standard of a product.

The authors assumed that if the load bearing capacity of a tendon with the smallest cross-section for the maximum force is not exceeded, then Ultimate Limit State is met for each tendon. Tab. 7, presented below, presents a summary of the ULT analysis for suspension tendons, depending on the loading model.

Table 7. Checking the Ultimate Limit State of suspension tendons - summary

Model	No of tendon	F_k [kN]	F_{uk} [kN]	F_{Rd} [kN]	F_{Edmax} [kN]	ULT
G + Gn					5004	62%
LM1					7931	99%
K + q	W7G/D	10331	12009	8006	7234	90%
32xT-815					7945	99%
Wind					4716	59%

According to [1] stress level in the suspension tendons need to meet an additional condition for bearing capacity, i.e.:

$$\sigma_{ck} \leq 0,45R_{pk}, \quad (16)$$

where: σ_{ck} – stress in the tendon of characteristic loads, R_{pk} – characteristic strength of the tendons. Well-designed suspension system should meet the condition described by the formula (17):

$$\Delta\sigma_{ck} \leq 200MPa \div 300MPa, \quad (17)$$

Table 8 contains checking of the ULS in view of the stress present in tendons for different load models.

Table 8. Checking the Ultimate Limit State of suspension tendons - summary

Model	σ_{ek} [MPa]	$0,45 \cdot R_{pk}$ [MPa]	ULT	$\Delta \sigma_{ek}$ [MPa]
G + G _n	473.0	837	57%	364.0
LM1	749.8		90%	87.2
K + q	592.3		71%	244.7
32xT-815	751.3		90%	85.7
Wind	440.0		53%	397.0

2.2. Modal analysis

The modal analysis was carried out in order to obtain the ten frequencies of natural frequencies of Solidarity Bridge in Płock. With the analysis carried out, it was possible to identify the necessary values of the coefficients on basis of which, the set values reflecting the wind load were determined. Table 9, presented below, compares the cumulative value of ten natural frequencies of the bridge and selected forms of natural frequencies are shown on Fig. 17-22.

Table 9. Types of vibrations and the values of natural frequencies of Solidarity Bridge in Płock

No.	The type of vibration	Frequency value [Hz]
1.	First vertical flapwise frequency of the bridge deck	0.445
2.	First flapwise frequency of the pylon	0.487
3.	Second flapwise frequency of the pylon	0.487
4.	First horizontal flapwise frequency of the bridge deck	0.581
5.	Second vertical flapwise frequency of the bridge deck	0.658
6.	First torsional frequency of the deck	0.979
7.	Third vertical flapwise frequency of the bridge deck	0.988
8.	Fourth vertical flapwise frequency of the bridge deck	1.417
9.	Second torsional frequency of the deck	1.592
10.	Third torsional frequency of the deck	1.959

On the basis of natural frequency an assessment of the susceptibility of the bridge structure on the aerodynamic impact, hereinafter referred to as flutter, can be performed. Danger of flutter phenomena is low when the ratio of the first flapwise natural frequency to the first torsional natural frequency is greater than or equal to 1.5. In the analysed example of the bridge, the first flapwise natural frequency is 0,979 Hz and the first torsional natural frequency is 0,445 Hz. The ratio of the two values is 2.2, which means that the probability of occurrence of the flutter on the bridge is very small.

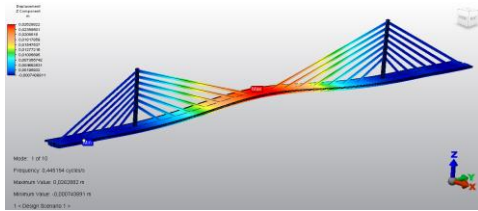


Fig. 17. First mode of natural frequencies of the Solidarity Bridge - flapwise frequency, frequency equals 0, 445 Hz

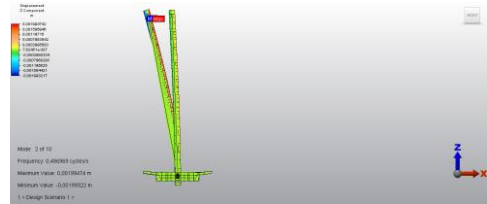


Fig. 18. Second mode of natural frequencies of the Solidarity Bridge - flapwise frequency of the pylon, frequency equals 0, 0.487 Hz

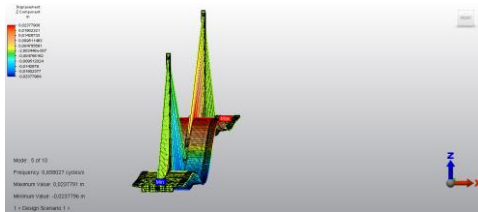


Fig. 19. Fifth mode of natural frequencies of the Solidarity Bridge - vertical flapwise frequency of the span, frequency equals 0, 0.658 Hz

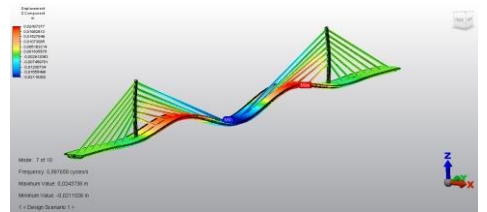


Fig. 20. Seventh mode of natural frequencies of the Solidarity Bridge - vertical flapwise frequency of the span, frequency equals 0, 0.988 Hz

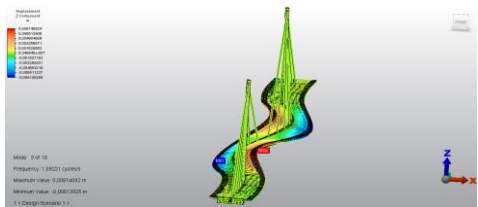


Fig. 21. Ninth mode of natural frequencies of the Solidarity Bridge - horizontal torsional frequency of the span, frequency equals 0, 1.592 Hz

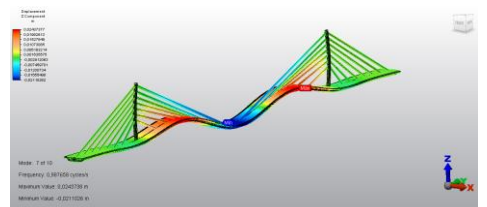


Fig. 22. Tenth mode of natural frequencies of the Solidarity Bridge - horizontal torsional frequency of the span, frequency equals 0, 1.959 Hz

3. Conclusions

Thanks to the analysis of the Solidarity Bridge in Płock, results on the response of the structure in the different load models were achieved. The job was difficult because no standard specifies the detailed way of loading such structures. By creating several loading models, it was possible to compare the results. The Serviceability Limit State of the span was not met for the load model 3 and 5. Model 3 reflects the LM1 model in accordance with [5]. It should be noted that the load was evenly distributed on the whole length of the main bridge span. The likelihood of occurrence of such a load on a bridge is very small, but the main aim of the authors in this case was to analyse the response of the structure under the influence of such an extreme load. Special attention should also be paid to similar values of the displacements obtained in model 3 and 5. Unfortunately due to lack of detailed information in the analyses carried out by the team of the University of Gdańsk about modes and values of application of the forces reflecting the load of the weight of

Tatra T-815 vehicle, it was not possible to carry out a comparative analysis of the results, mentioned in the summary of this paper.

On the basis of the results, it can be concluded that loads according to the old Polish standard [7], are less strict in relation to the new ones proposed by eurocodes [5].

Designed suspension system showed a sufficient load-bearing capacity of the tendons. The Serviceability Limit State (SLS) and Ultimate Limit State (ULS) was met for each calculation model.

The Serviceability Limit State for model 6 (wind load) was met. Deviation of the top pylon was within a limit values [4].

The types of vibrations and values of natural frequencies of the structure showed that the probability of occurrence of a flutter phenomenon on the Solidarity Bridge in Płock is low.

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