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# A Comparative Aerodynamic Analysis of Two Cable Stayed Bridges Built in Poland

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## Abstract

The two longest cable-stayed bridges in Poland (230m and 250m main span lengths) have been subjected to the aerodynamic analyses. This paper presents the main topics in these calculation processes: modal analysis, estimation of internal forces generated by the static load action, analysis of vibrations excited by the atmospheric turbulence and feedbacks, determination of bridge vibrations generated by the vortex excitation, calculation of stresses produced with the dynamic wind action. The dynamic structure response has been calculated using the quasi-steady method, which allowed analysis of just a few representative mode shapes, with the results precision acceptable in the engineering practice. Finally, both bridges have been found safe and resistant to the dynamic wind action, however the Warsaw bridge is more susceptible to the aerodynamic influence, especially its deck.

## Introduction

Nowadays two cable-stayed bridges have been realised in Poland: Third Millenium John Paul II Bridge in Gdańsk (former called the Henryk Sucharski Bridge) and the Siekierkowski Bridge in Warsaw. Such bridges are susceptible to aerodynamic actions caused by wind, so the bridges contractors (Transprojekt Gdansk and Mostostal Warsaw) decided to performed wind tunnel tests and aerodynamic analysis. The tests for both bridges have been performed in the boundary layer wind tunnel in CSTB Nantes (France), while complex aerodynamic analyses for both bridges have been provided in Poland by the team of researchers from Technical Universities of Lublin and Cracow. The scope of the task performed for both bridges could be characterised as follows: wind field simulations- [1] and [2], wind tunnel investigations and analyses of the obtained results - [3], [4], [5] and [6], aerodynamic analyses - [7] and [8].

The paper presents the main problems of complex aerodynamic analysis such as modal analysis, calculation of internal forces generated by static load, evaluation of bridge vibrations due to atmospheric turbulence and aerodynamic feedbacks, evaluation of vortex induced bridge vibrations, calculation of internal forces in structure coming from wind action.

**Bridges characteristics**

The Henryk Sucharski Bridge over the Vistula river in Gdansk is a cable-stayed one with a single tower (Fig.1). The total length of the river crossing is 372m while the length of the suspended span is 230m and is located 8m above the water level of the Vistula river. The reinforced concrete tower has been shaped as the upturned letter Y. The total height of the tower is 99m. The bridge deck has been designed as a complex structure of longitudinal carrying beams and the 23cm thick reinforced concrete plate. The steel structure has been additionally reinforced with transverse beams every 4m in the suspended part of the bridge and every 4.33m elsewhere. The carrying system of the bridge is supplemented with suspenders of one to three cables which consist of 31 to 55 strands of 15.5mm diameter made of 7 wires. There are 15 points of suspension at each side of the bridge deck at the longer span, while 8 ones occur at the shorter span. The cable lengths vary from about 55m to 209m.

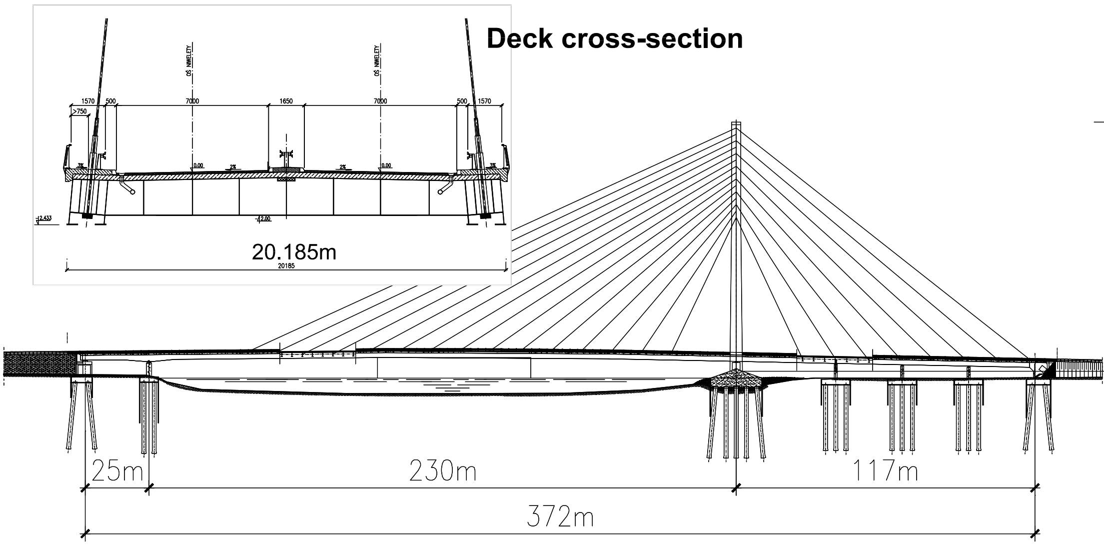



Fig.1.The scheme of the Sucharski Bridge in Gdańsk.

The Siekierkowski Bridge (Fig.2) is a part of the planned Siekierkowska Route in Warsaw. It has been designed as a structure of three separated parts: the cable-stayed bridge of 500m length, the bridge of 250m length at the right river bank and the bridge of 77m length situated at the left bank. The middle part of the river crossing has been subjected to the analysis. It is designed as a cable-stayed structure with two towers and 250m length of the suspended span. The towers have been designed as the same two structures made of reinforced concrete and steel with the shape close to the letter H. The tower height is about 90m. The bridge deck carrying structure consists of two steel girders, 2.07m high each, that are situated in the plane containing cables symmetry axes, and the transverse beams of the same height, which are spread every 4m. The steal structure is unified with the reinforced concrete plate. The distance between the main carrying beams varies from 26.1 to 33.1m. The total width of the deck in the main span is 33.8÷40.83m. The suspension cables consist of 7-wire strands with 15mm diameter. The number of the strands in a cable varies from 43 to 76. The bridge suspension is designed of total number of 56 cables. There are 14 cables in the main span at each side and 7 cables in each of bank sides at each side of the deck.

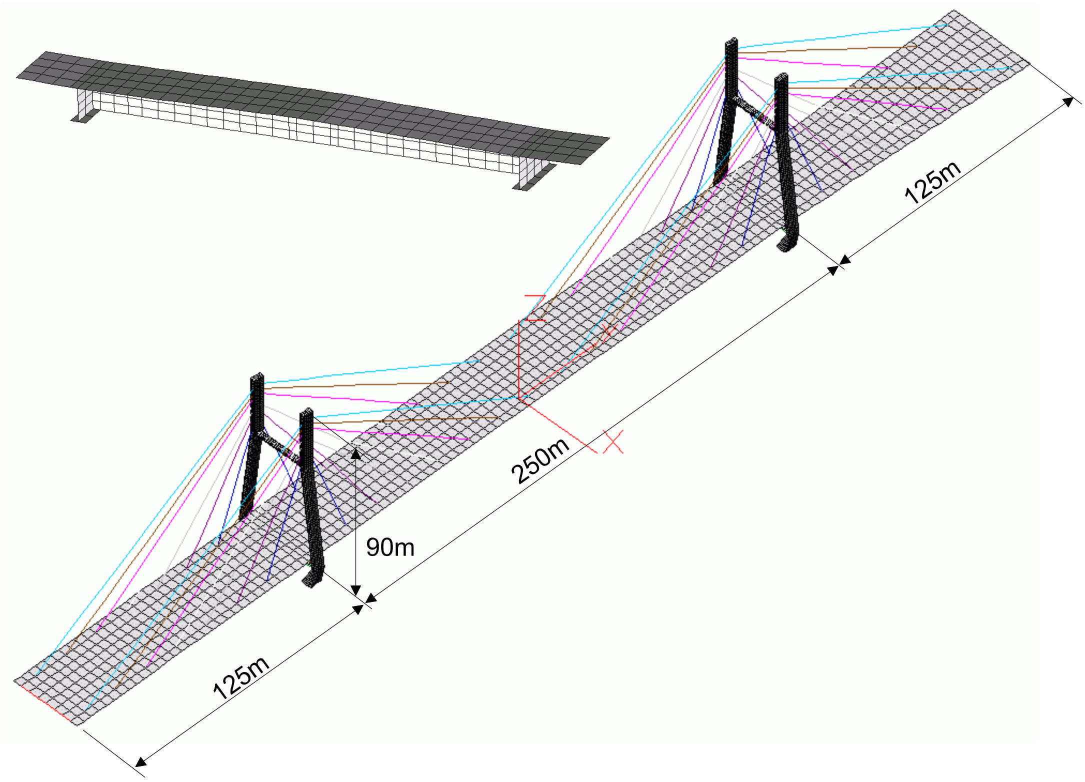


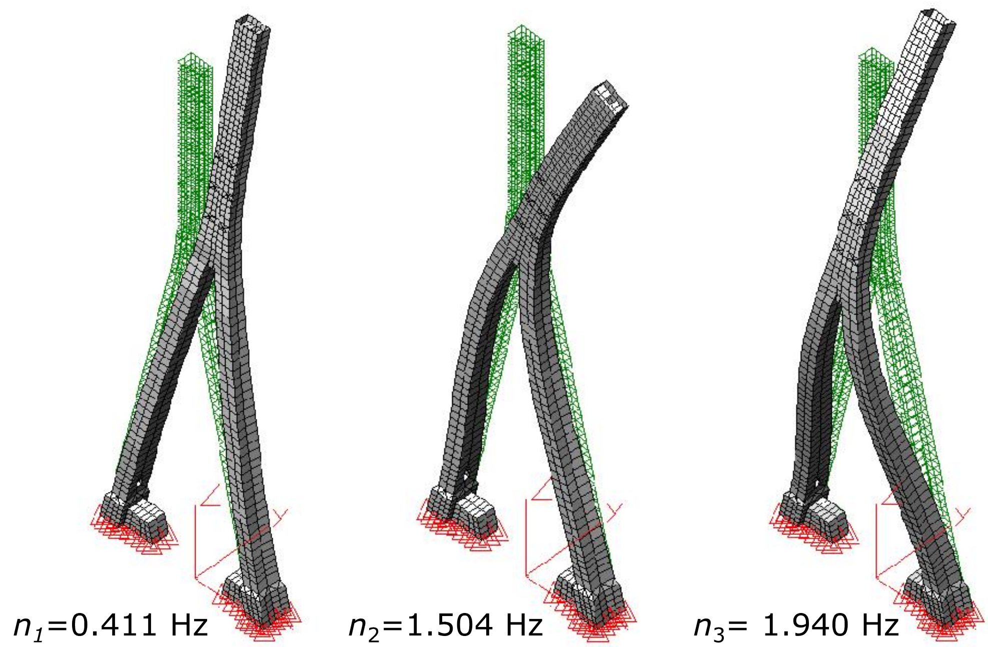

Fig.2.The static scheme of the Siekierkowski Bridge in Warsaw.

**Modal analysis**

The natural frequencies and mode shapes for both structures have been calculated with the help of the linear module of the **Algor** program, which is called SSAP1. It uses subspace iteration in order to obtain first natural frequencies and mode shapes. As for the bridge located in Gdańsk, there have been obtained ten mode shapes of the tower, 40 mode shapes for the whole bridge at the stage of erection, and 40 mode shapes of the whole bridge at the stage of operating. The following ones have been subjected to the further analysis:

- for the tower: the four first bending mode shapes with the respective natural frequencies 0.41Hz, 1.50Hz, 1.94Hz and 2.38Hz;
- for the whole bridge a the stage of erection: the two first bending mode shapes (with the respective natural frequencies of 0.24Hz and0.40Hz, the third torsional mode shape (0.65Hz), the fourth o (0.66Hz) and finally the 23rd one (0.71Hz) where bending deck vibrations and cables vibrations occurred;
- for the whole bridge at the stage of operating: the two first bending mode shapes (0.41Hz and 0.659Hz), the third torsional mode shape (0.66Hz) and finally the 35th one (0.79Hz) where torsional deck vibrations and cables vibrations occurred.

a)




b)

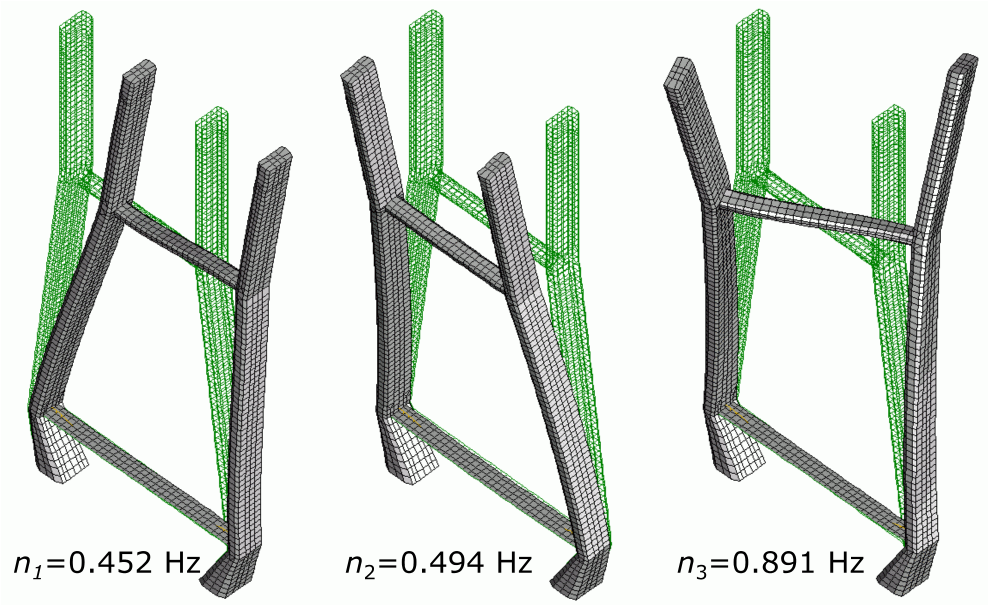
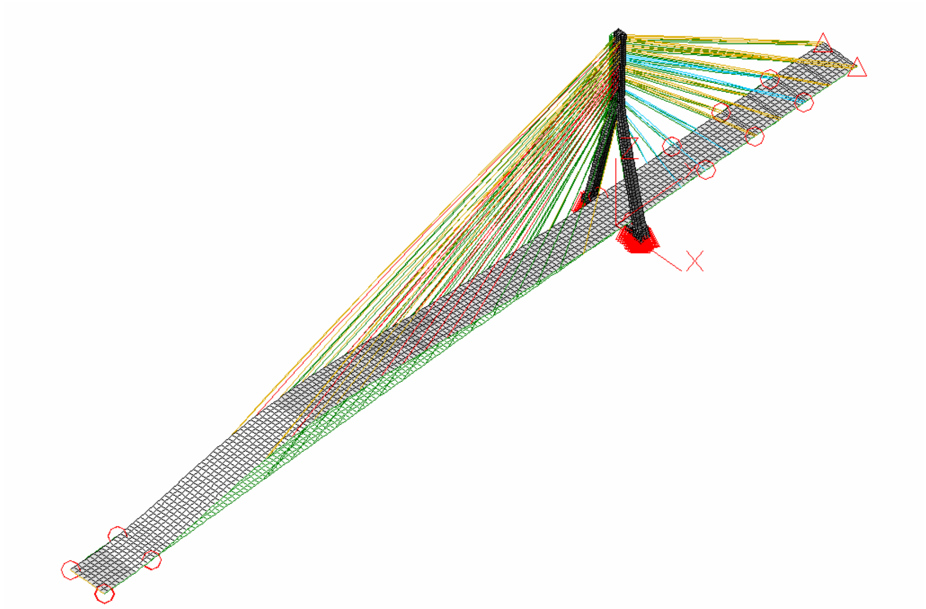



Fig.3. The exemplary mode shapes of the towers: a) for the bridge in Gdansk,
b) for the bridge in Warsaw.

For the bridge located in Warsaw, ten mode shapes of single tower have been calculated and 40 mode shapes for the whole bridge at the stage of operating. The six first ones for the bridge (0.45÷3.17Hz) and seven as for the whole bridge (0.43÷0.81Hz) have been used in the further analysis.

Fig.3 and Fig.4 show the exemplary mode shapes of the towers and the bridges.

a)



b)

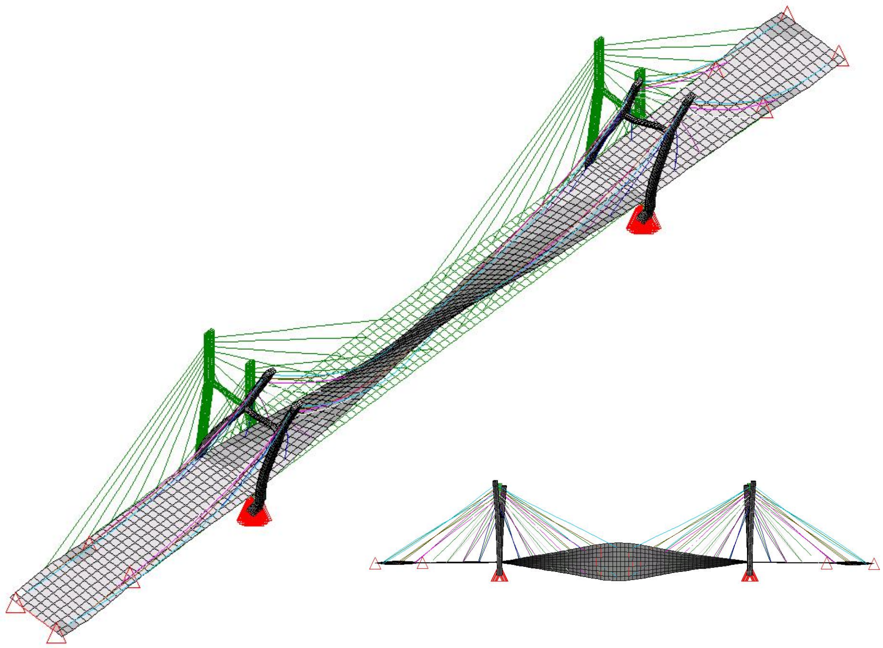


Fig.4. The exemplary mode shapes of the bridges: a) the first mode of the bridge in Gdańsk, $n_1$=0.4056 Hz; b) the second mode of the bridge in Warsaw, $n_2$=0.4856Hz.

**Bridge vibrations excited by atmospheric turbulence and feedbacks**

The calculation of the dynamic response of the structure exposed to atmospheric turbulence and feedbacks has been obtained with use of quasi-steady method see [9] and [10]). It has been assumed that it is possible to calculate the displacements with the accurate precision as the linear combination of three selected (representative) mode shapes. Several sets of mode shapes have been considered for each of the bridge construction stages.

The representative frequencies, which have been accepted for the analysis of the bridge located in Gdańsk, refer to the two bending and one torsional mode shapes. In the case of the bridge located in Warsaw, it decided that the six first mode shapes for the tower vibrations, and seven first modes shapes for the whole bridge vibrations are the representative ones.

As the result of the analysis, the following time-dependent multiplication factors for the equivalent generalised inertial forces (i.e. the ones which result with the same displacements as the displacements obtained with the analysis of the three DOF system) have been obtained. They allow to calculate the approximate displacements and stresses in the analysed bridges. For example, the obtained factors for the Gdansk bridge are:

- for the tower at the stage of erection: $\psi_1 = 1.071$; $\psi_2 = 0.011$; $\psi_3 = 0.035$;
- for the whole bridge at the stage of erection: $\psi_1 = 2.408$; $\psi_2 = 4.957$; $\psi_3 = 0.468$;
- for the whole bridge at the stage of operating: $\psi_1 = 5.122$; $\psi_2 = 0.441$; $\psi_3 = 0.305$.

The exemplary factors for the Warsaw bridge are:

- for the tower at the stage of erection with the wind direction perpendicular to the bridge deck axis: $\psi_1= 0.849$, $\psi_2= 0.043$, $\psi_3= 0.011$, $\psi_4= 0.064$, $\psi_5= 0.039$, $\psi_6=0.001$;
- for the tower at the stage of erection with the wind direction parallel to the bridge deck axis: $\psi_1= 0.397$, $\psi_2= 0.177$, $\psi_3= 0.084$, $\psi_4= 0.023$, $\psi_5=0.020$, $\psi_6= 0.003$;
- for the whole bridge: $\psi_1= 11.1978$ , $\psi_2= 2.91891$, $\psi_3= 1.64096$, $\psi_4= 1.82719$, $\psi_5= 2.71366$, $\psi_6= 1.53608$, $\psi_7= 3.89515$.

The exemplary maximum vertical and horizontal displacements of the Gdansk bridge endpoint at the stage of erection are equal to 23.74 cm (at 25.16 s) and 14.1 cm (at 35.5 s) respectively. The mean value of the endpoint horizontal displacement is equal to 6.13cm and the standard deviation – 4.04cm.

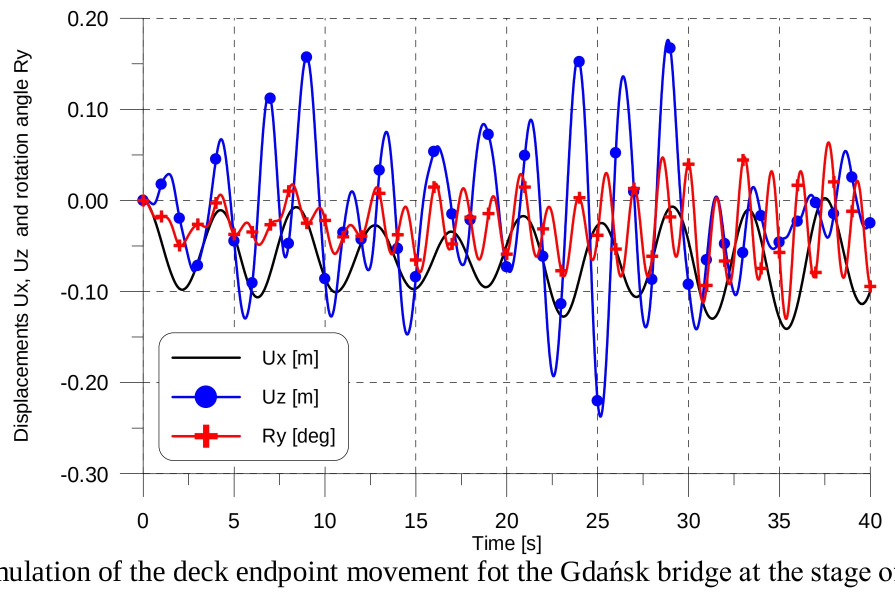



Fig.5. Simulation of the deck endpoint movement fot the Gdańsk bridge at the stage of erection

The amplitude of the rotational vibrations at the stage of erection is $\varphi$=0.13 deg (at 35.4 s). Such value of the angle results with the vertical displacement of each side of the bridge deck at the value of 2.0 cm. Exemplary bridges responses are presented in Fig. 5 and 6.

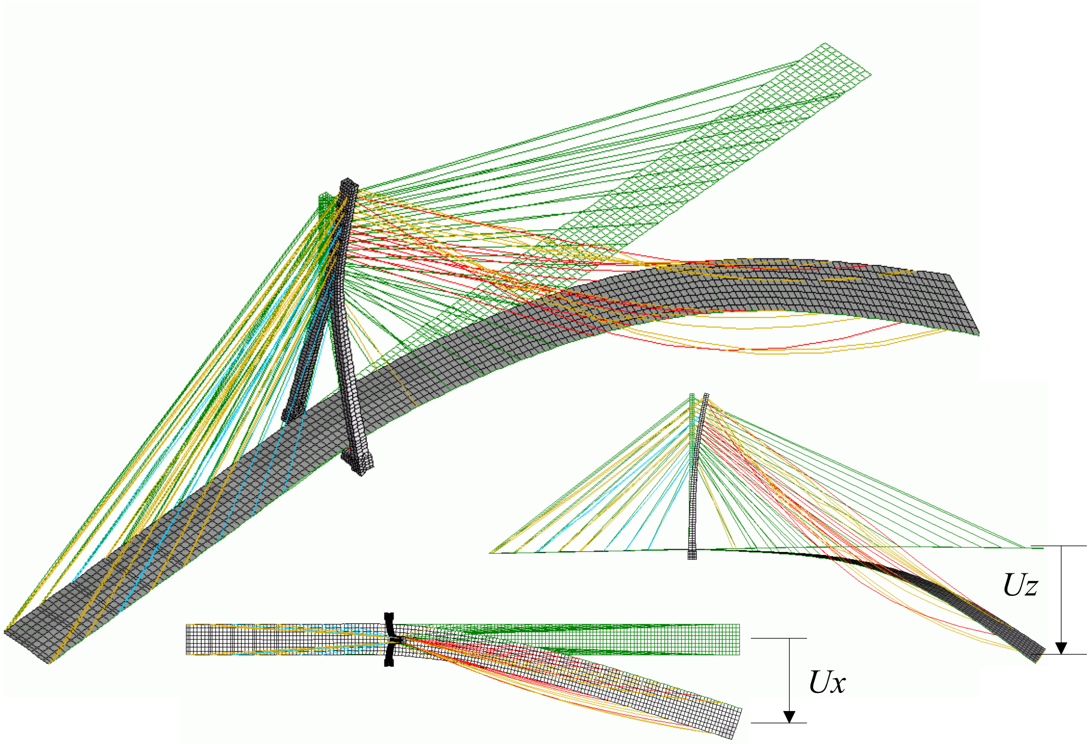



Fig.6. Snapshots (at 10.4 s) of the movement for the Gdańsk bridge at the stage of erection. Deck endpoint displacements: $U_X$ = -9.9 cm, $U_Z$ = -12.6 cm. Magnification scale 500 :1.

A level of deck acceleration was evaluated for the bridge in Warsaw [14]. The rms (root-mean-square) values of acceleration are equal to 0.95, 0.12, 1.07 or 0.18 m/s$^2$ at vibration frequencies 0.58, 0.49, 0.81 or 0.43 Hz, respectively. These vibrations may be perceptible to human but they are allowable.

The stresses that are produced with use of the following load combination: dead weight, static wind load and wind gusts, has been calculated using the following formula:

$$\sigma_{\max} = \sigma_{st} + \Delta\sigma_{wd}\ , \quad \Delta\sigma_{wd} = \sqrt{\sum_{i=1}^{3}\left(\psi_i \Delta\sigma_i\right)^2}\ , \tag{1}$$

where: $\sigma_{st}$ are the stresses in the structure produced by dead weight and static wind load, $\Delta\sigma_{wd}$ are the stresses produced by the dynamic wind action, $\Delta\sigma_i$ are the stresses coming from the inertial forces referring to the selected mode shape and $\psi_i$ are multiplication factors for the inertial forces modelling substitutional equivalent wind action.

Table 1 contains stresses in some elements of the bridge in operating stage in Gdansk and table 6 contains stresses in the Warsaw bridge. In table 6 there are stresses caused by dead weight. The presented critical stresses in concrete structures are calculated without consideration of reinforcement. Comparing values in tables 5 and 6, we may say that the stresses in Gdańsk bridge are significantly smaller than the stresses in the structure members of the Warsaw bridge.

Finally, the obtained with this method stresses values occurring in the towers, decks and cables have been found not exceeding the allowable values.

Table 1. Stresses in the bridge located in Gdańsk[7].

| | the stress type | $\sigma_{st}$ [Mpa] | $\sigma_{max}$ [MPa] | $\sigma_{st}/\sigma_{max}$ | $\sigma_{dop}$ [MPa] |
|---|---|---|---|---|---|
| the point near fixing of the longest cables | max. principal stress | 8.31 | 8.33 | 1.002 | 1.54 |
| | min. principal stress | -1.06 | -1.13 | 1.066 | -27.7 |
| the horizontal plate at „legs" connection | max. principal stress | 4.30 | 4.59 | 1.067 | 1.54 |
| | min. principal stress | -4.14 | -4.16 | 1.005 | -27.7 |
| the region near the door in the tower | max. principal stress | 4.72 | 5.31 | 1.125 | 1.54 |
| | min. principal stress | -15.22 | -15.43 | 1.014 | -27.7 |
| the reinforced concrete deck plate near fixing of the shortest cables | stress along the deck | -0.52 | -0.54 | 1.038 | 1.54 |
| | stress across the deck | -5.43 | -5.66 | 1.042 | -27.7 |
| the steel deck structure near fixing of the shortest cables | Huber-Mises stress | 26.96 | 29.12 | 1.080 | 295 |
| cables | | 492.7 | 520.57 | 1.057 | 1550 |

Table 2. Stresses in the bridge located in Warsaw without baffles [13].

| | the stress type | $\sigma_c$ [MPa] | $\sigma_{st}$ [MPa] | $\sigma_{max}$ [MPa] | $\frac{\sigma_{\max}-\sigma_c}{\sigma_{st}-\sigma_c}$ | $\sigma_{st}/\sigma_{max}$ | $\sigma_{dop}$ [MPa] |
|---|---|---|---|---|---|---|---|
| the windward top of the tower next to fixing cables | max. principal stress | 6.7 | 31.2 | 32.3 | 1.049 | 1.035 | – |
| | min. principal stress | -5.8 | -41.8 | -42.8 | 1.028 | 1.024 | – |
| the tower above lower horizontal beam | max. principal stress | 14.9 | 16.6 | 25.6 | 6.294 | 1.54 | 1.93 |
| the tower under lower horizontal beam | min. principal stress | -37.7 | -39.8 | -49.4 | 5.571 | 1.24 | -33.3 |
| the reinforced concrete deck plate in the middle of the span | stress along the deck | -0.6 | -0.8 | -0.9 | 1.5 | 1.125 | -23.3 |
| | stress across the deck | -5.3 | -6.4 | -7.6 | 2.091 | 1.188 | -23.3 |
| the steel deck structure in the middle of the span | Huber-Mises stress | 197.3 | 256.9 | 292.9 | 1.604 | 1.140 | 295 |
| cables | | 655.9 | 658.9 | 733.6 | 25.9 | 1.113 | 1770 |

$\sigma_c$ – stress caused by dead weight, other notation as in equation (1).

**Vortex induced bridge vibrations**

Vortex induced vibrations problem with reference to spans, towers and cables need to be described separately [11], [12]. Vortex influence on the deck is completely different from the influence on tower or cable. This comes from the fact that the bridge deck shape is elongated in the direction of the wind flow and may be influenced by more vortices generating transverse, rotational or transverse-rotational vibrations. The most important conclusions coming from the analysis of the Gdansk bridge are:

- the deck ant tower load generated with vortex shedding at cables is small in comparison to the cables dead weight or the static wind load;
- the cable force variation produced by vortices are negligibly small;

- the estimated cable vibration number of cycles coming from vortices is at the lower limit of allowable number of cycles;
- the effects of vortices action on deck estimated with use of the accepted computational model are not dangeres for the structure, if the wind cornishes are in use;
- the maximum value of tower load coming from vortices is small in comparison to static wind load, so the vortices influence on tower behaviour is not significant.

The most important conclusions coming from the Warsaw bridge analysis are:

- the tower and deck loads coming from vortex shedding at cables is small in comparison to the cables dead load and static wind action;
- the cable forces change generated with the static wind load is small;
- the estimated number of cycles of cable vibrations excited by vortices is very big
- the values of deck displacements coming from vortices are not exceeding the allowable displacements for operating stage;
- the values of tower load coming from vortices is small in comparison to the static wind load (of about 20%) and the vortices influence on the towers behaviour is not significant.

## Concluding remarks

The complex aerodynamic analysis of the cable-stayed Third Millenium John Paul II Bridge in Gdańsk has shown that the main structure members of the bridge are safe and resistant to the dynamic wind action. The aerodynamic analysis of the cable-stayed Siekierkowski Bridge in Warsaw has also confirmed that the bridge structure with additional baffles, which had been proposed during the CSTB wind tunnel tests is resistant to the dynamic wind action. However there has been found a possibility of bridge deck motion with high acceleration.

The quasi-steady method used to determine the dynamic wind influence onto structure has been found very efficient. It allowed the reduction of numerical task size with selection of a few representative mode shapes subjected to the analysis. The obtained results are precise enough to be used in engineering practice.

Such complex aerodynamic analyses of cable-stayed bridges are the first ones performed in Poland.

Displacements obtained in the wind tunnel experiments in CSTB for both bridges are of the same order as obtained in aerodynamic calculations presented in this paper.

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