CHALLENGE JOURNAL OF STRUCTURAL MECHANICS 2 (4) (2016) 196–204

Assessment of aerodynamic response of the Nissibi cable-stayed bridge using three-dimensional computational fluid dynamics

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ABSTRACT

Aerodynamic behavior has the greatest impact on long-span bridges and is the most important factor in the design of cable stayed bridges, which should not be overlooked. CFD (Computational Fluid Dynamics) is the most widely used technique, among bridge engineers, to predict wind speed, direction and vortex-shedding form before conducting wind tunnel tests. In this study, a bi-directional CFD analysis with the wind flow parallel and perpendicular to Nissibi Bridge's, which has a main span of 400 m and claimed the spot of Turkey's 3rd largest bridge, deck cross-section has been performed by approximate modelling of the bridge and the surrounding structures. The study is done by using CFD++ software/computer program. The results showed that the effect of wind acting on *x* direction of impact with 30 m/s has caused turbulence and vortex on conjugation area of the tower and it is observed that the upside down Y shape of the tower breaks down the balance of wind flow. However, bridge deck is not exposed to serious amount of vortex influence due to the wind on *y* direction. In addition, the analysis revealed that maximum pressure distribution occurred on vertical surface of the tower and it increases in direct proportion to the height of the tower.

ARTICLE INFO

Article history:
Received 8 November 2016
Accepted 17 December 2016

Keywords:
Aerodynamic behaviour
CFD
Cable-stayed bridge
Wind

1. Introduction

Nissibi Bridge, a part of Adiyaman-Kahta-Siverek-Diyarbakır highway/motorway, got drowned due to the lake formed as a result of the construction of Ataturk Dam which came into operation in 1992, thus resulting in the disconnection of highway link between Adıyaman and Diyarbakır. At that time, the government divided the highway into two as a consequence of which the distance between Adıyaman and Diyarbakır increased by 150 km. To rectify this problem, construction of a new bridge between Kahta and Siverek came into consideration. The materialization of the project in question started in 2012 with the commencement of work on cable stayed Nissibi Bridge that was later inaugurated in the first half of 2015.

The cable stayed bridge has a main span of 400 m and claimed the spot of Turkey's 3rd largest bridge. Due to the complications and cost involved in positioning the bridge pier inside the reservoir/lake of the dam, it was deemed necessary to cross the reservoir/lake with a long span bridge. All types of bridges are subjected to wind effects but the long span bridges are the ones which are more sensitive to dynamic responses/effects as compared to others. Dynamic wind load/force, despite of not being very influential from loading viewpoint, is important due to the vibrations it brings about. Tacoma Narrows Bridge is the most important example of aerodynamic properties' influence/effect on the design. The bridge got demolished due to the aeroelastic flutter arising from the wind flowing at a velocity of only 64 m/s. This incident brought to light the importance of aerodynamic effects over bridge (Arioli and Gazzalo, 2013).

Bridge design should be conducted in such a way that it would not encounter any kind of instability and sudden wind impulses/excitations would remain within the acceptable limits. An example of it is Messina Strait Bridge, which would have overtaken Akashi Kaikyo

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ISSN: 2149-8024 / DOI: http://dx.doi.org/10.20528/cjsmec.2016.12.037

Bridge as the longest suspension bridge, with the central span length of 60% more than the Akashi Kaikyo Bridge's. To check the aerodynamic design properties of the bridge, a comprehensive study involving wind tunnel test simulations has been carried out (Diana et al., 2013).

Aerodynamic behavior has the greatest impact on long-span bridges and is the most important factor in the design of cable stayed bridges, which should not be overlooked. In previous studies, analysis of bridges to wind-induced motions were performed by various researchers (Ge and Tanaka, 2000; Xu et al., 2004; Yongle et al., 2004; Matsumoto et al., 2001 and Ren et al., 2005). However, previous studies have been limited to analysing two dimensional sections only, and three dimensional analyses of the bridges have not yet been studies.

All these studies underline that the high level of flexibility associated with the cable stayed bridges make them more sensitive to wind flow as compared to other structures. The reasons for this susceptibility/sensitivity are:

- Aerodynamic instability; incompatibility of dynamic effect with static effect or vibrations in the bridge deck arising from the dynamic effects may be sufficient to cause destruction of the bridge.
- Buffeting; the effect arising from the turbulences generated by the wind flow. The bridge deck, when subjected to these turbulences for a long time, may experience fatigue effect depending upon the movement of the deck.
- Vortex-shedding realization results in periodic vortex vibrations; bridge deck may experience forced vibrations (ICE Manual of Bridge Engineering, 2008).

Investigating wind-structure interaction with wind tunnel tests is a quite compelling and comprehensive engineering problem/study. With the developments in wind engineering after the tragedy of Tacoma Narrows Bridge, wind tunnel testing has been made compulsory for the design of long span bridges (Diana et al., 2013). However, wind tunnel test calculations become very difficult/complex due to different profiles/cross-sections involved in the test and are very costly, indeed. Due to the aforementioned reasons, Numerical simulations became the center of attention for aerodynamic and aeroelastic analyses. CFD (Computational Fluid Dynamics) is a computer modeling technique to simulate fluids flow. CFD is the most widely used technique, among bridge engineers, to predict wind speed, direction and vortex-shedding form before conducting wind tunnel tests. 2D numerical simulations, related to aerodynamic of bridge deck, performed over Great Belt East Bridge in Denmark showed very close results to the ones obtained from wind tunnel tests and progress was recorded in the vortex formation (Bruno and Chris, 2003). In a study, on U-shaped Bridge, involving 2D and 3D CFD models, a comparison of results of 2D and 3D CFD with the wind tunnel test results showed that 3D CFD simulations presented more close results to wind tunnel test results as compared to 2D CFD simulations. For 3D turbulent incompressible viscous flow, 3D numerical methods alongside Detached-Eddy simulations have been effectively used for bridges (Bai et al., 2010).

In this study, a bi-directional CFD analysis with the wind flow parallel and perpendicular to the bridge deck cross-section has been performed by approximate modeling of the bridge and the surrounding structures. The goal of the study is to determine the wind load effects on the bridge and its elements, by considering the wind flow characteristics of the region where the bridge is situated. The study is done by using CFD++ software/computer program. CFD++ is an advanced level general purpose computational fluid dynamics software, based on integrated element modeling, integrated physics and integrated calculation methodology, within advanced numerical discretization and solution system.

2. Methodology

2.1. Construction stages of the Nissibi Bridge

Nissibi Bridge is a single span bridge with the main central span of 400m and total span of 610m (Fig. 1). The height of each pylon is approximately one-fourth of the main span length i.e. 96.8 m and is in λ form/type. The cross-sections of pylons are in the form of reinforced concrete box sections. Despite having a self-weight much more than their steel counterparts, they support a variety of cable configurations. Box sections are preferred to provide safety against buckling with the minimum amount of material. Limestone type rock units related to Gaziantep formation, the idealized soil type which is observed from the surface, are generated/created. The bridge foundation has been designed as shallow foundation/spread footing to sit on the limestone formations at the base. For the static load case, the allowable bearing capacity of the soil has been calculated as 1000 kPa. The foundation of pylon is designed to have dimensions of 40m x 7.3m x 5m as shown in Fig. 2.

In cable-stayed bridges, straight cables radiating from the pylon are connected to the deck. In case of multiple cable arrangements the system can be fan type (i.e. cables radiating from the top) or harp type (i.e. cables are positioned parallel and are connected to pylon at different heights). A combination of these two types is often used, known as modified fan system, and has been used in Nissibi Bridge. Construction/erection of hollow pylon shaft, in the design, provides access for supervision or for new placements. The horizontal component of the forces in the cables will be withstood by the vertical and transverse prestressing of the concrete pylon.

The main span of Nissibi Bridge is made up of structural steel sections incorporating an orthotropic road deck while prestressed reinforced concrete box sections have been used in the edge spans. Box section has been preferred to provide a substantial amount of torsional stiffness. In box girders, top flange/cap not only serves as pavement deck but also transfer loads to vertical or inclined web elements. Moreover, box section bridges are known to be more resilient towards vibration effects as compared to classic bridges. In practice, single or multi cell box girders with rectangular or trapezoidal cross-sections may be employed. In the case of Nissibi Bridge, Trapezoidal box girders have been used (Fig. 3).

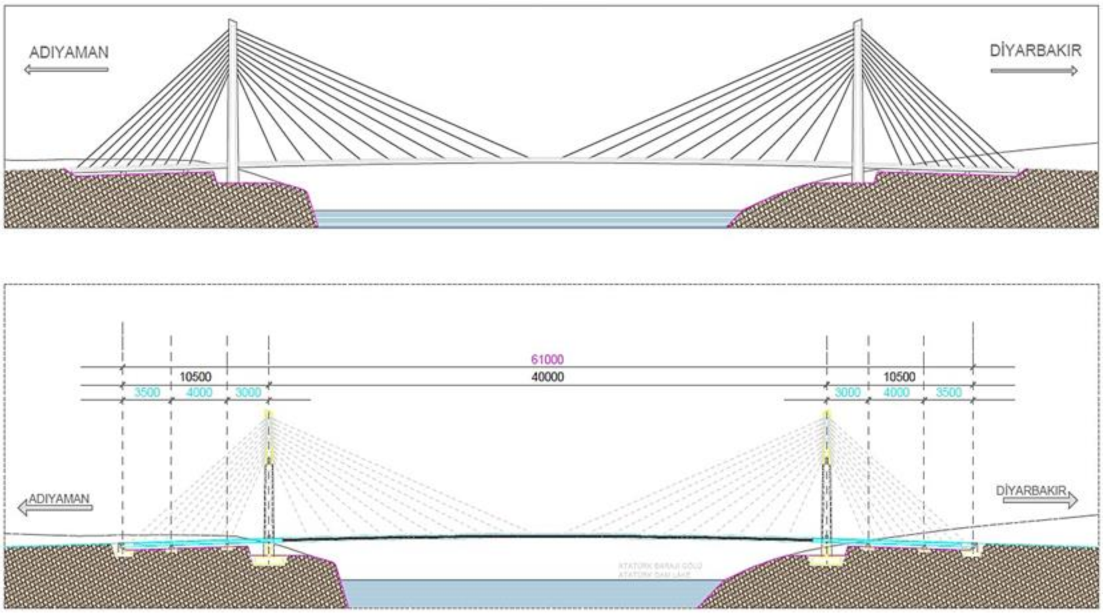

Fig. 1. General view of Nissibi cable-stayed bridge.

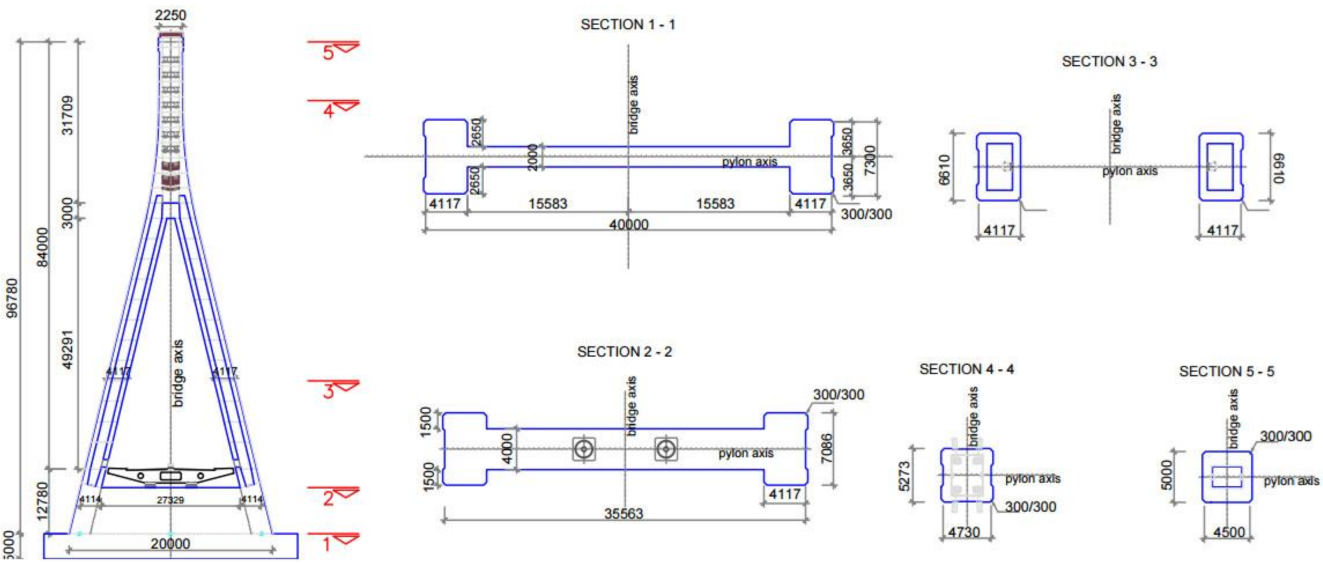

Fig. 2. Cross-section of the pylon.

2.2. Finite element modelling of Nissibi Bridge

Geometric modelling was done in CFD++ by using/importing 3D CAD DWG drawings (Fig. 4) of Nissibi Bridge. In the modelling, especially the main elements of the bridge have been taken into consideration. Due to Nissibi Bridge being cable stayed bridge with inclined cables, the effect of wind, on the structure, to which the cables are subjected, will not be critical and has been neglected.

To get CFD analysis results very close to the real, the surrounding structures with which the wind interacts before reaching the bridge model, have also been included in the analysis work. The modelling of surroundings of the bridge has been done on the basis of height difference from the topographic map/plan of Diyarbakır – Adıyaman cities.

2 grids are created for CFD analysis. An element grid consisting of 5 million hybrid (hexagonal and tetragonal) elements is built. Domain size of 2 km x 2 km is taken to provide atmospheric boundary conditions. The height is selected as 900 m, approximately 5 times the maximum height within the domain. Nissibi Bridge and surroundings are placed within the square domain with the required wind flow provided as bi-directional through surfaces/faces related to this square domain. In the flow domain, wall boundary conditions for bridge and the surrounding are modelled in different element sizes. Inside the domain, there are total 4 wall boundary conditions (as can be seen in Fig. 5); Bridge Pylons/Piers (in yellow), Pavement over bridge (in green), terrain over which the bridge rests (in red) and lake region around the bridge (in blue/cyan). The elements of the Nissibi Bridge have been modelled smaller, as compared to the surrounding elements, as can be seen in Table 1.

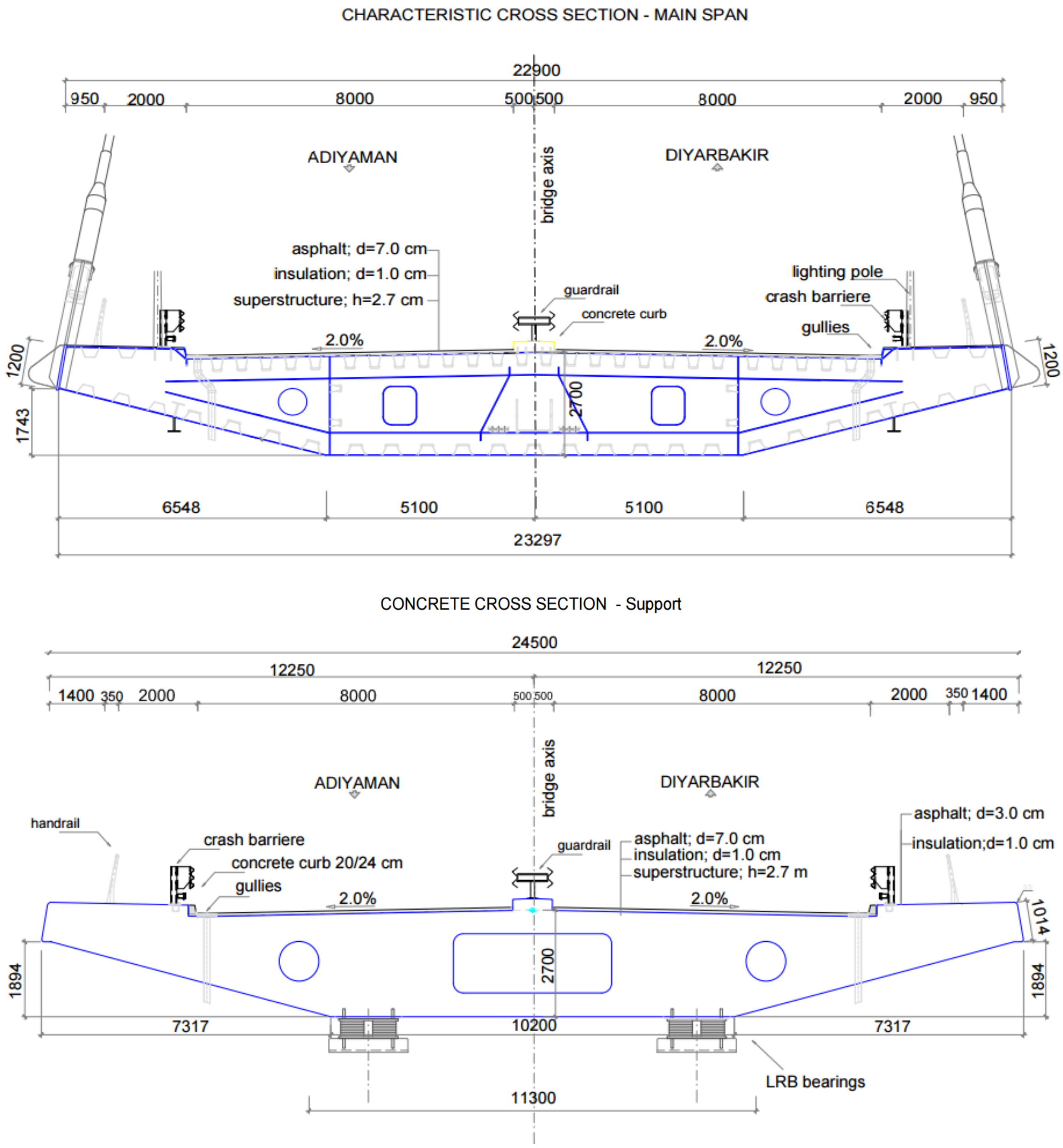

Fig. 3. Cross-section of the main span and support structural sections.

Fig. 4. The CAD view of the geometrical modelling.

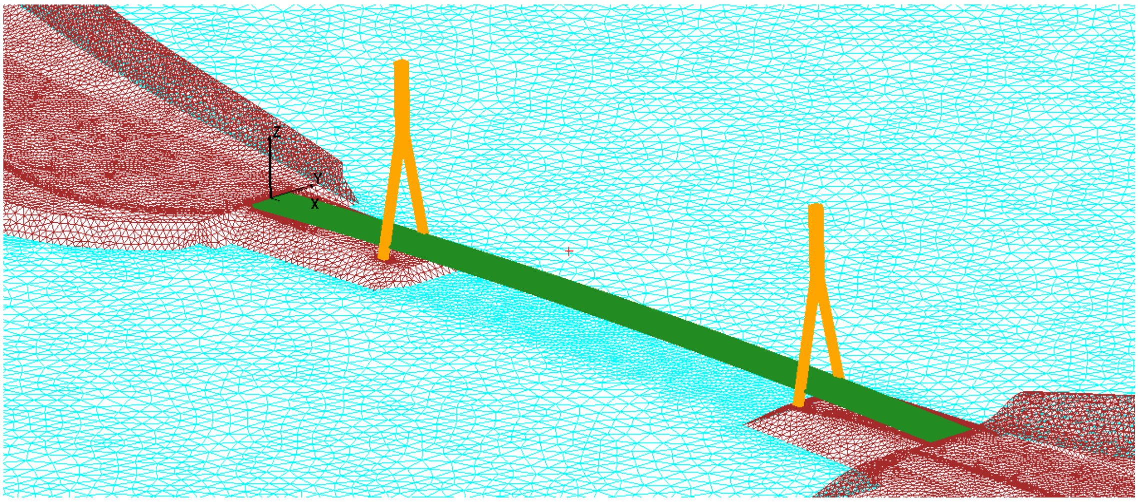

Fig. 5. The wall boundary conditions for the bridge and the surrounding.

Table 1. Maximum element size based on boundary conditions.

Boundary Surface/Face	Maximum Element Size (m)
Pylon/Tower	0.01
Deck	0.01
Terrain Area/Region	0.10
Lake Region	0.10

2.3. Flow characteristics

In the analysis conducted for wind coming parallel to the bridge, velocity of value 30 m/s, from navy-blue face/surface towards yellow face/surface (+x direction), is given. On the other hand, for the wind flow perpendicular to the bridge way, velocity of value 30 m/s, from pink face/surface towards green face/surface (+y direction), is considered (Fig. 6). 'Cubic k- ε' turbulence model has been used for turbulence modelling. Initial and end boundary condition values for k and ε values are automatically calculated, by software itself, while also considering geometric characteristic lengths.

3. Results and Discussion

Acting on x and y-direction (Figs. 7-8) of impact with effect of 30 m/s wind, maximum pressure distribution occurred on vertical surface of the tower and it increases in direct proportion to the height of the tower. This pressure, ruling on horizontal section, has caused large shear forces on the tower. Then, due to the stress on the pylon, where the pylon and bridge deck is fixed, a serious amount of pressure distribution is noticed.

However, the wind load on y direction did not create large amount of pressure on bridge deck (Fig. 9). There will be a sediment dislodging force on the bridge deck however the long distance of bridge and inclined hangered cables blocked it.

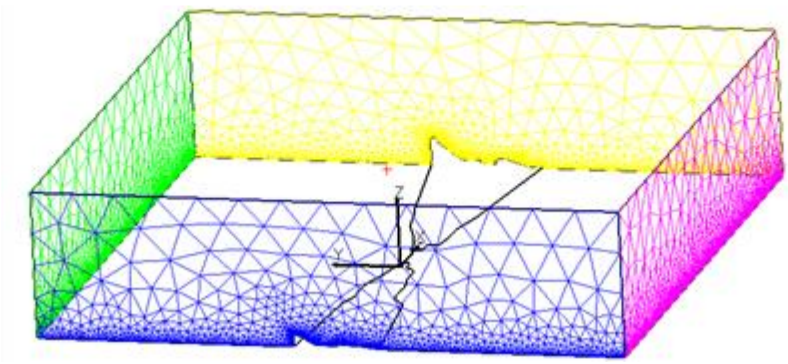

Fig. 6. Flow characteristics for the analysis.

The effect of wind acting on x direction of impact with 30 m/s has caused turbulence and vortex (Fig. 10) on conjugation area of the tower. In the figure, it is observed that the upside down Y shape of the tower breaks down the balance of wind flow. In the wind flow, when the vortexes arrived second pylon, linear flow was broken down. Thus, the second (right) pylon is exposed to more vortex oscillations compared to the first one (Fig. 11).

However, bridge deck is not exposed to serious amount of vortex influence due to the wind on x direction. Flow transistorizes that the wind created by crashing to the pylons did occur high pressure effect on the bridge deck.

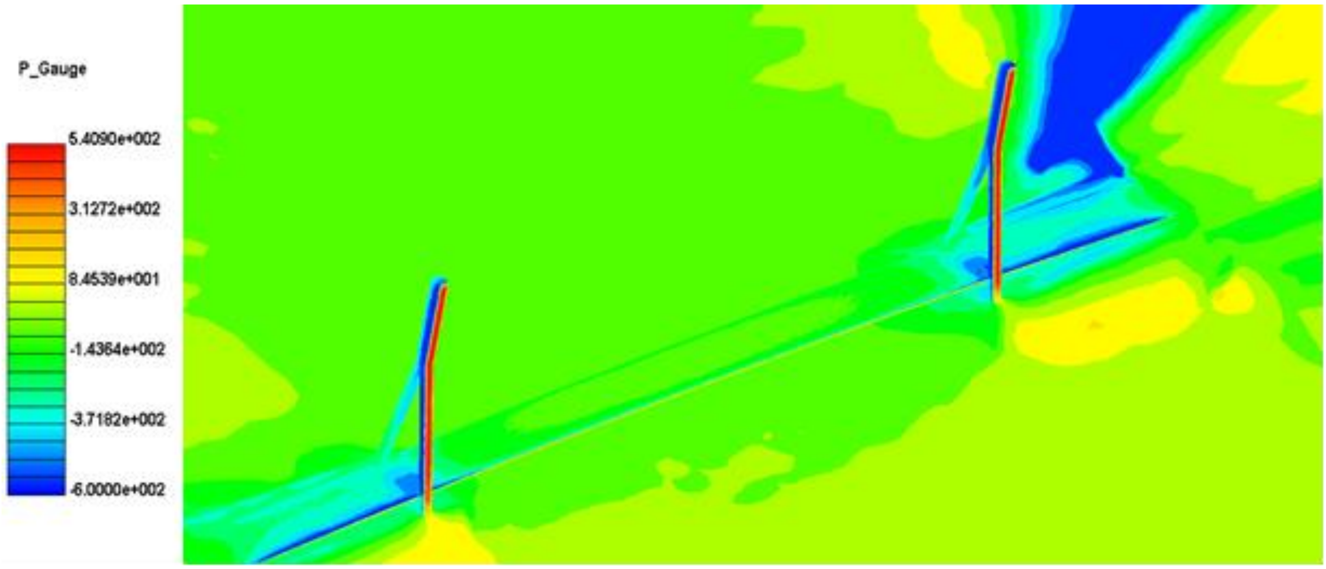

Fig. 7. Measured pressure results perpendicular to bridge.

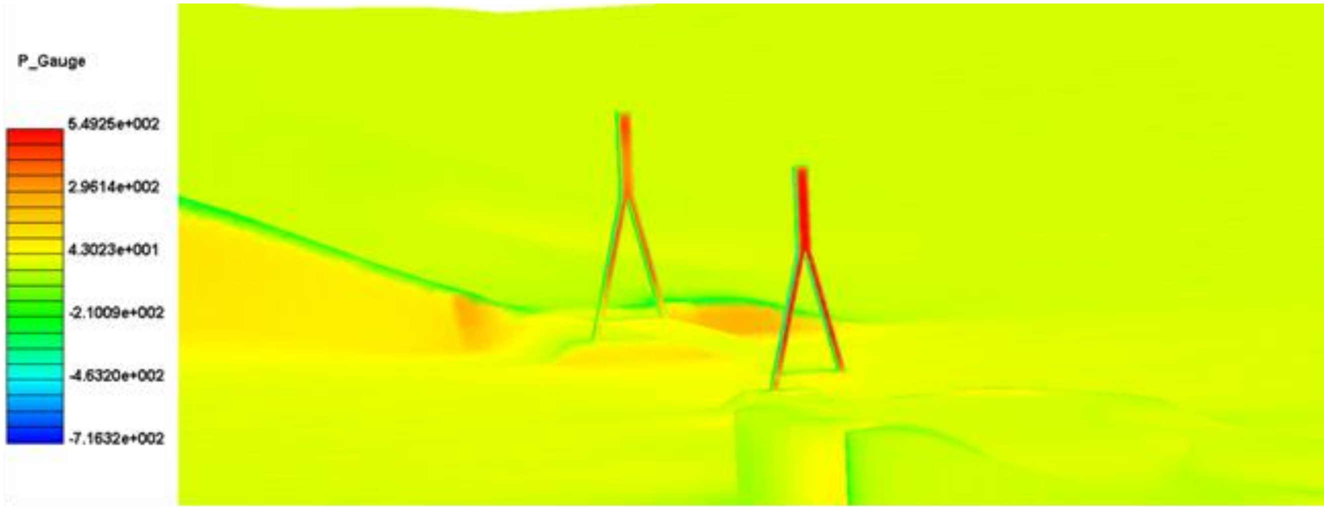

Fig. 8. Measured pressure results parallel to bridge.

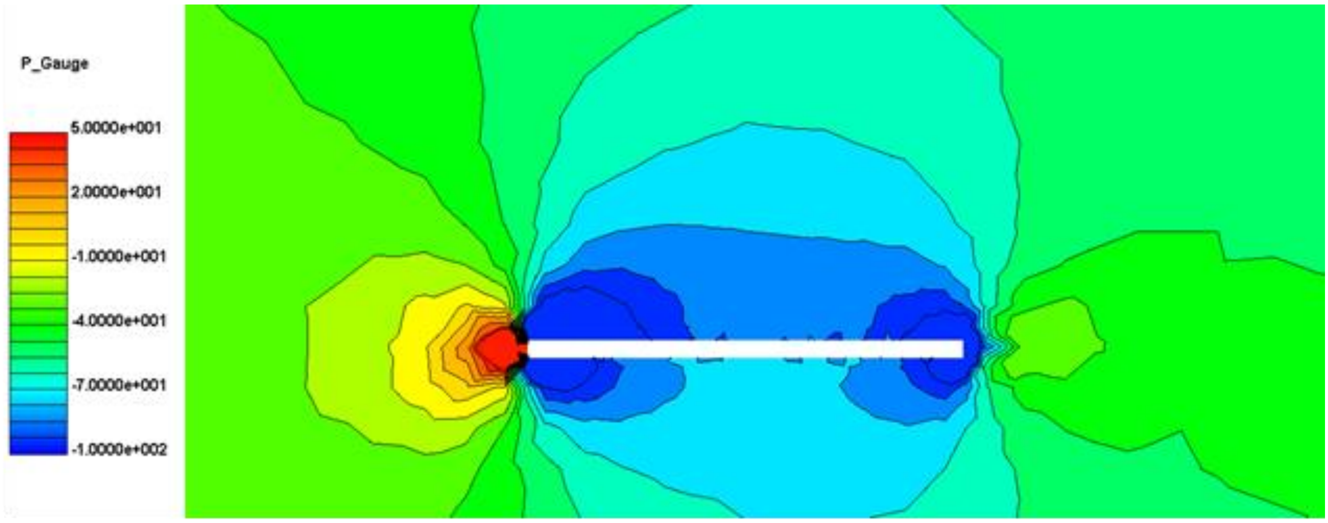

Fig. 9. Measured pressure results on the bridge deck in y-direction.

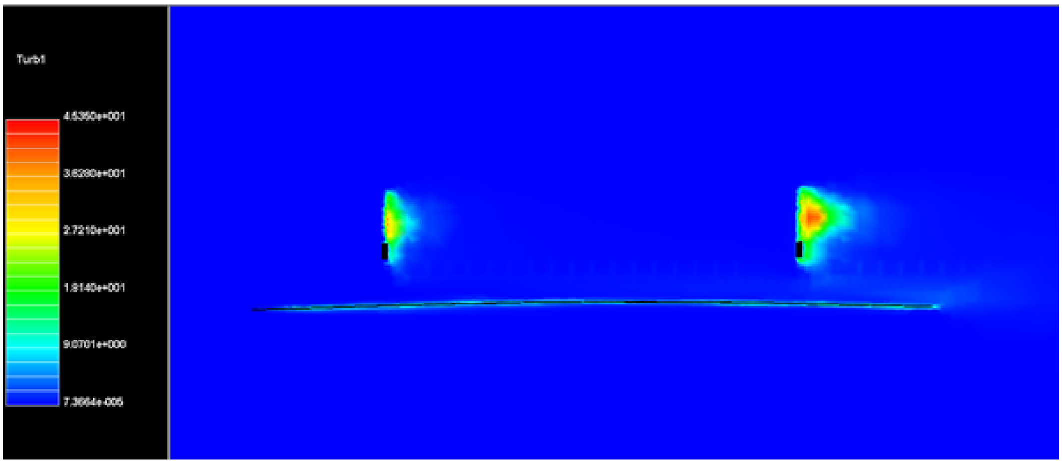

Fig. 10. Kinetic energy with turbulence in the parallel direction.

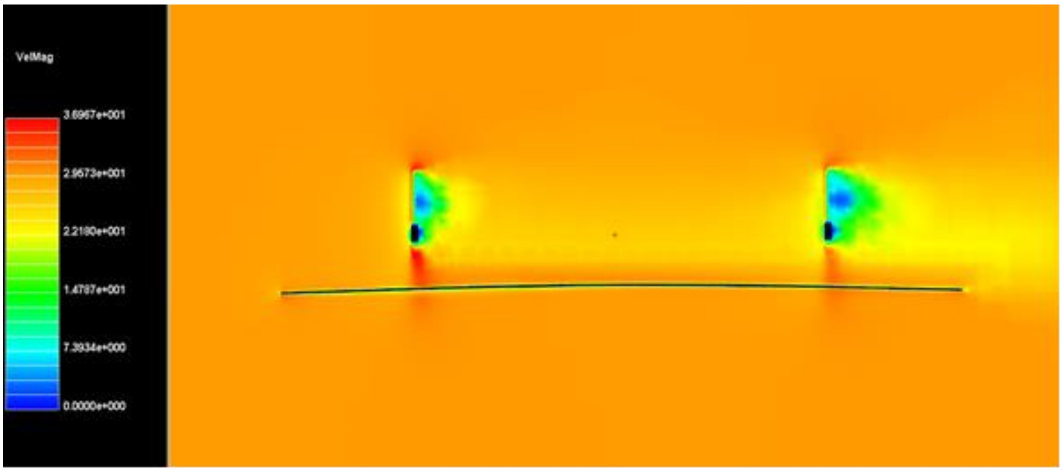

Fig. 11. Wind flow velocity on the pylons in the *x*-direction.

For the total velocity area result shown in the Fig. 12, the wind which has 30 m/s on the *x*-direction arrived the first pylon by gaining velocity. The wind flow, which goes forward by hitting the first pylon, went forward by forming vortex streets. Therefore, the wind flow has taken the second pylon by losing its velocity. In the figure, the *x*-component of velocity field shows that the vortexes, which occurred by the wind hitting to the first pylon, scattered and went forward to the second pylon. The *x*-component of the negative direction resultant of vortex streets' starts are on the merging on peak points. At these points, periodic vortex formations might be seen. The increase of vortex depth can be understood from the scattering of the wind flow velocity.

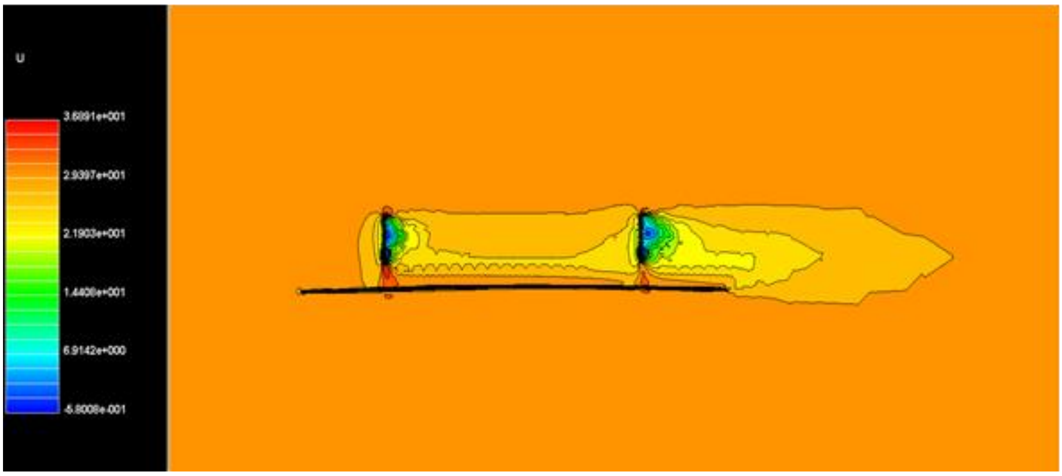

Fig. 12. Total velocity area results in the *x*-direction.

The velocity result perpendicular to the bridge way was shown in the Fig. 13. The figure indicates the wind flow velocity was not so high on the bridge deck section. This could be attributed to a larger torsional and buckling rigidity of the steel box deck on that direction.

Measurement pressure results, which affect perpendicularly to bridge tower surfaces have been nondimensionalized with dynamic pressure and served below the graphics (Figs. 14-15) with details. Pressure distribution graphics with 30m/s velocity has been interpreted by multiplying with normalized coefficients for different velocities for same graphics.

4. Conclusions

Aerodynamic behavior is the most important factor in the design of cable stayed bridges as these bridges can suffer from a high-level of vibration due to wind.

In general, maximum pressure distribution occurred on vertical surface of the tower and it goes up in direct proportion to the height of the tower. This pressure, ruling on horizontal section, has led to large shear forces on the tower. In addition, the wind flow, which goes forward by hitting the first pylon, went forward by forming vortex streets. Therefore, the wind flow has taken the second pylon by losing its velocity. However, at the effect of wind acting on x direction, when the vortexes which are occurred from the first pylon arrived second pylon, linear flow was broken down. Thus, the second pylon was exposed to more vortex oscillations compared to the first one. Finally, it was observed that the critical wind direction was perpendicular to the longitudinal axis of the bridge.

In a future research study, the wind-cable interaction or different wind flow velocities on the aerodynamic response of cable-stayed bridges would be studied.

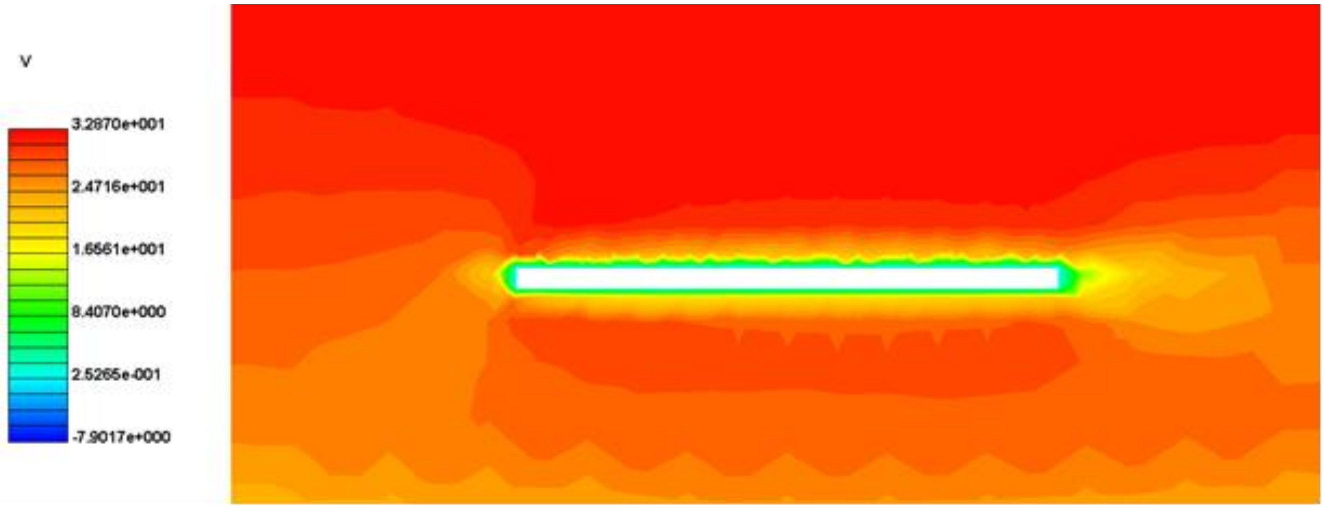

Fig. 13. Wind flow velocity on the bridge deck section in y-direction.

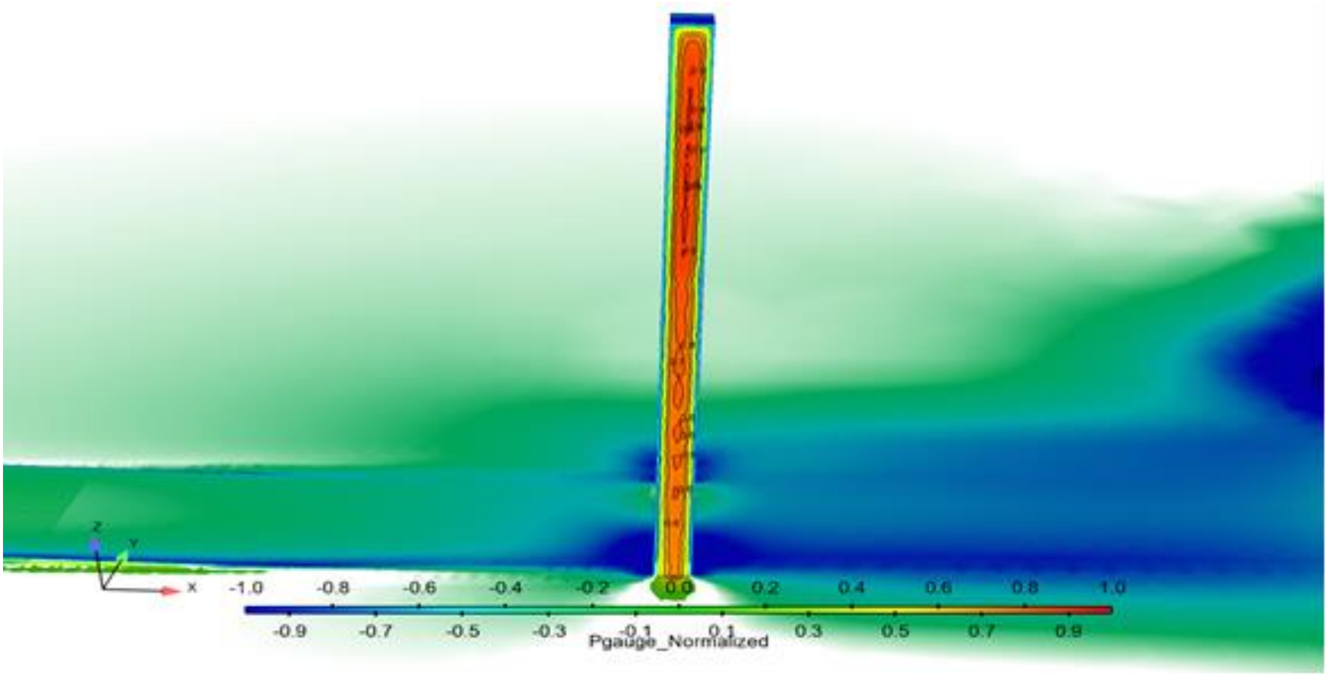

Fig. 14. Nondimensionalized pressure results of right bridge tower (wind from y-direction).

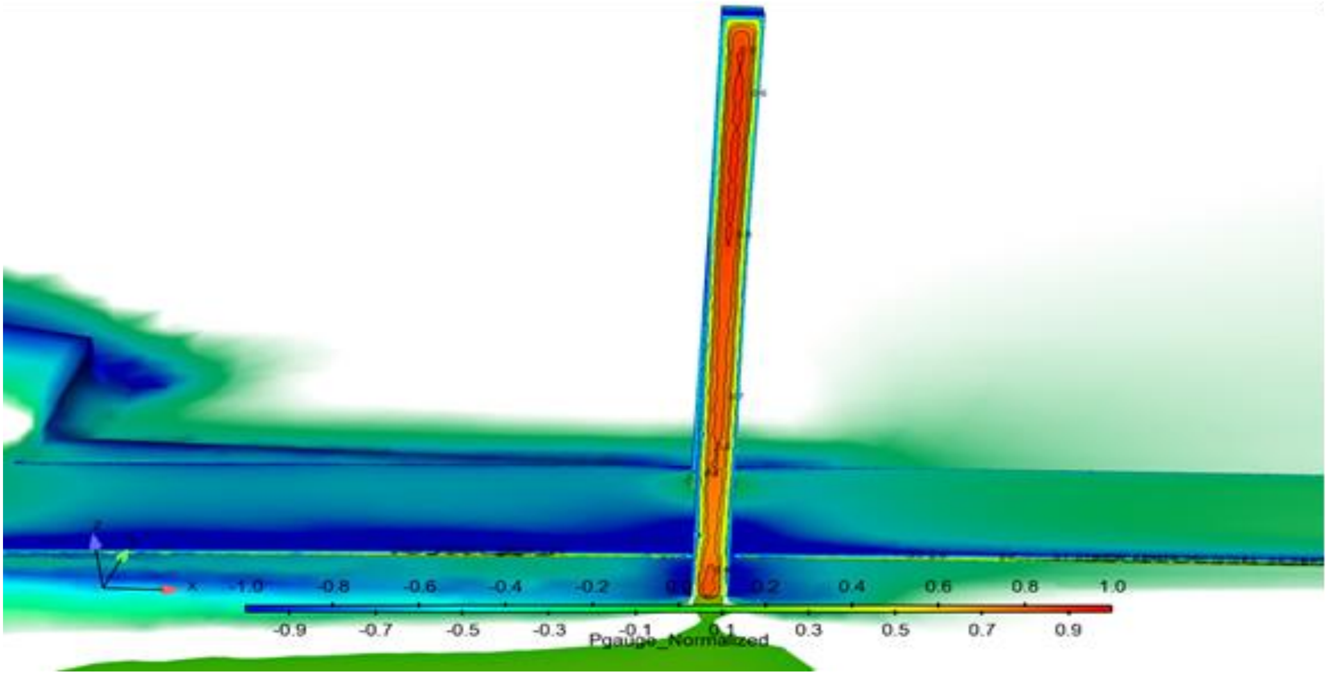

Fig. 15. Nondimensionalized pressure results of left bridge tower (wind from *y*-direction).

REFERENCES

Arioli G, Gazzola F (2013). Old and new explanations of the Tacoma Narrows Bridge collapse. *Atti XXI Congresso AIMETA*, Torino, p. 10.

Bai Y, Sun D, Lin J (2010). Three dimensional numerical simulations of long-span bridge aerodynamics, using block-iterative coupling and DES. *Computers and Fluids*, 39, 1549-1561.

Bruno L, Khris S (2003). The validity of 2D numerical simulations of vortical structures around a bridge deck. *Mathematical and Computer Modelling*, 37, 795-828.

Diana G, Fiammenghi G, Belloli M, Rocchi D (2013). Wind tunnel tests and numerical approach for long span bridges: The Messina Bridge. *Journal of Wind Engineering and Industrial Aerodynamics*, 122, 38-49.

Diana G, Yamasaki Y, Larsen A, Rocchi D, Giappino S, Argentini T, Pagani A, Villani M, Somaschini C, Portentoso M (2013). Construction stages of the long span suspension Izmit Bay Bridge: Wind tunnel test assessment. *Journal of Wind Engineering and Industrial Aerodynamics*, 123, Part B, 300-310.

Ge YJ, Tanaka H (2000). Aerodynamic flutter analysis of cable-supported bridges by multi-mode and full-mode approaches. *Journal of Wind Engineering and Industrial Aerodynamics*, 86(2–3), 123-153.

Matsumoto M, Yagi T, Shigemura Y, Tsushima D (2001). Vortex-induced cable vibration of cable-stayed bridges at high reduced wind velocity. *Journal of Wind Engineering and Industrial Aerodynamics*, 89, 633-647.

Park G, Hewson N (2008) (Eds.). ICE Manual of Bridge Engineering (2nd ed.). Thomas Telford.

Ren W, Peng X, Lin Y (2005). Experimental and analytical studies on dynamic characteristics of a large span cable-stayed bridge. *Engineering Structures*, 27, 535-548.

Xu YL, Zhang N, Xia H (2004). Vibration of coupled train and cable-stayed bridge systems in cross winds. *Engineering Structures*, 26, 1389-1406.

Yongle L, Liao H, Qiang S (2004). Simplifying the simulation of stochastic wind velocity fields for long cable-stayed bridges. *Computers and Structures*, 82, 1591-1598.