

Article

Computational Fluid Dynamics Approach to Aeroelastic Stability in Cable-Stayed Bridges

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Abstract

Long-span cable-supported bridges, such as cable-stayed and suspension bridges, are highly sensitive to wind-induced effects due to their flexibility, low damping, and relatively light weight. Aerodynamic analysis is therefore essential in their design and safety assessment. This study examines the aeroelastic stability of the Oued Dib cable-stayed bridge in Mila, Algeria, with emphasis on vortex shedding, galloping, torsional divergence, and classical flutter. A finite element modal analysis was carried out on a three-dimensional model to identify natural frequencies and mode shapes. A two-dimensional deck section was then analyzed using Computational Fluid Dynamics (CFD) under a steady wind flow of $U = 20$ m/s and varying angles of attack (AoA) from -10° to $+10^\circ$. The simulations employed a RANS $k-\omega$ SST turbulence model with a wall function of $Y^+ = 30$. The results provided detailed airflow patterns around the deck and enabled the evaluation of static aerodynamic coefficients—drag (C_D), lift (C_L), and moment (C_M)—as functions of AoA. Finally, the bridge's aeroelastic performance was assessed against the four instabilities. The findings indicate that the Oued Dib Bridge remains stable under the design wind conditions, although fatigue due to vortex shedding requires further consideration.

Keywords: cable-stayed bridge; Computational Fluid Dynamics; classic flutter; galloping; modal analysis; static coefficients; torsional divergence; vortex shedding



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1. Introduction

Long-span cable-supported bridges, such as cable-stayed and suspension bridges, are essential components of modern transportation networks. However, their inherent flexibility, low damping, and relatively light weight make them highly susceptible to wind-induced instabilities [1–3]. Among these, vortex shedding, galloping, torsional divergence, and flutter are the most critical, as they can lead to severe serviceability problems or even catastrophic failures [4–6]. For this reason, aerodynamic and aeroelastic analyses play a pivotal role in the design and safety evaluation of such bridges.

The vulnerability of long-span bridges to wind has been tragically demonstrated throughout history. One of the earliest recorded failures was the Dryburgh Abbey Bridge in Scotland (1818), where the cables gave way during a storm. This was not an isolated

case: the Brighton Chain Pier in England (1836) and the Tay Bridge in Scotland (1879) both collapsed under wind loads, with the latter disaster claiming the lives of 80 people. The most dramatic failure, however, was the Tacoma Narrows Bridge in the United States (1940), whose violent oscillations and spectacular collapse were captured on film, shocking engineers and the public alike [7].

The catastrophic collapse of the Tacoma Narrows Bridge in 1940 [8–12] dramatically exposed the destructive potential of aerodynamic instabilities and marked the beginning of bridge wind engineering as a distinct research discipline. In the 1970s, Scanlan advanced the field with his hybrid time–frequency self-excited force theory for flutter analysis [13–17], which provided a rigorous framework for quantitative research. While this theory greatly improved predictive capability, it relied on small-amplitude and linearity assumptions, limiting its accuracy for real-world nonlinear behavior. Subsequent studies revealed a distinct nonlinear flutter mechanism termed soft flutter [18–23], characterized by non-catastrophic limit cycle oscillations (LCOs) that appear once the critical flutter speed is exceeded. In this regime, vibrations stabilize into oscillations whose amplitudes gradually increase with wind speed, rather than diverging catastrophically. Associated Hopf bifurcation phenomena were also documented [24], providing deeper insight into the nonlinear dynamics of aeroelastic instabilities. Building on these findings, several nonlinear self-excited force models were proposed to capture both soft flutter and bifurcation behavior. To address limitations such as spurious numerical modes in certain frequency-domain formulations, researchers later developed time-domain self-excited force models [25–27]. These advanced formulations demonstrated high fidelity in reproducing experimental vibration histories, confirming their effectiveness for simulating complex nonlinear aeroelastic responses in long-span bridges.

Prior to the Tacoma failure, aeroelastic studies had been largely confined to aeronautics. Unlike streamlined aircraft wings, however, bridge decks are bluff bodies, generating separated flows and vortex shedding that cannot be described by classical aerodynamic theories. This realization motivated pioneering works by Bleich [28], Bisplinghoff and Ashley [29], and Scanlan and Tomko [30], who laid the foundations of modern bridge aerodynamics. Complementing these theoretical advances, wind tunnel testing and, more recently, Computational Fluid Dynamics (CFD) have become indispensable tools for investigating flow–structure interactions and for deriving static and dynamic aerodynamic coefficients of bridge decks with greater accuracy.

In Algeria, the demand for modern transportation infrastructure has led to the construction of landmark cable-stayed bridges. The Oued Dib Bridge, inaugurated in 2001, was the country’s first of its kind, spanning 502 m across the Béni-Haroun reservoir with a main span of 280 m. A second example, the Salah Bey Bridge in Constantine (2014), further illustrates the growing national reliance on this bridge type. Among these, the Oued Dib Bridge stands out as an ideal case study: its relatively non-streamlined deck geometry and exposure to strong regional winds make it particularly relevant for aeroelastic investigations.

This research aims to assess the aeroelastic stability of the Oued Dib Bridge under representative wind conditions. The study proceeds in three stages:

1. Modal analysis of a finite element (FE) model to determine natural frequencies and mode shapes.
2. CFD simulations of a two-dimensional deck section, subjected to a steady wind speed $U = 20$ m/s and varying angles of attack (AoA) from -10° to $+10^\circ$, using the RANS $k-\omega$ SST turbulence model with a wall function $Y^+ = 30$.
3. Aeroelastic assessment of the structure against the four principal instability mechanisms—vortex shedding, galloping, torsional divergence, and classical flutter—using the static aerodynamic coefficients (C_D , C_L , C_M) derived from CFD.

The overall framework of this research is illustrated in the flowchart presented in Figure 1.

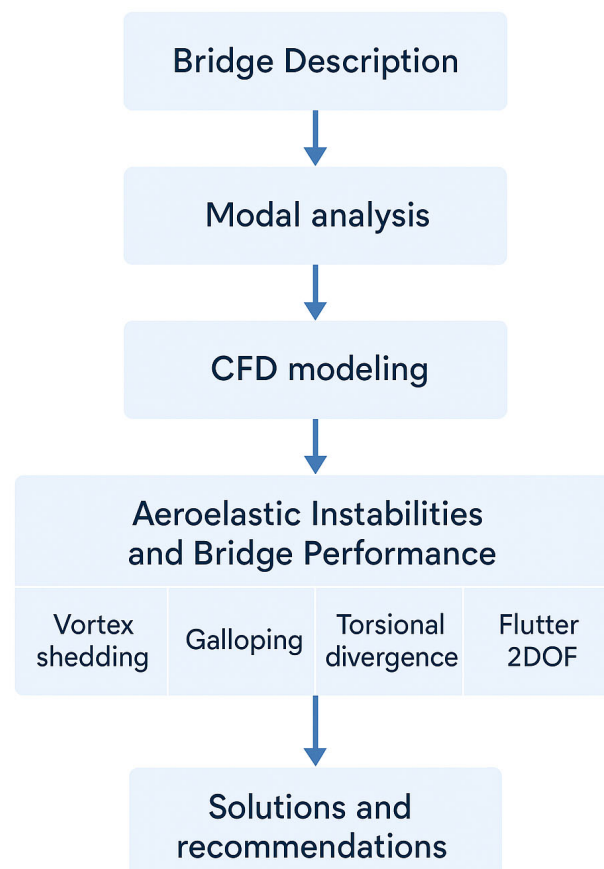


Figure 1. Research methodology flowchart.

The novelty of this work lies in providing the first comprehensive aeroelastic study of Algeria’s inaugural cable-stayed bridge, combining structural modal analysis and CFD to evaluate stability limits of a bluff-body deck geometry. Unlike most studies that focus on streamlined bridge sections, this research addresses the specific challenges posed by non-profiled decks and offers new insights into the applicability of CFD for predicting their aeroelastic performance. The findings not only deepen the understanding of wind resistance in the Oued Dib Bridge but also contribute to the broader field of long-span bridge aerodynamics, where computational approaches are progressively complementing and extending traditional experimental methods.

2. Bridge Description

The Oued Dib cable-stayed bridge, located near Mila in Algeria, was built to connect the cities of Constantine and Jijel. In service since 2001, the bridge crosses Lake Béni-Haroun, the largest dam in the country. It has a total length of 502 m, comprising three spans: a main span of 280 m and two side spans of 111 m each (Figure 2).

The prestressed concrete deck measures 13.3 m in width and 2.04 m in depth (Figure 3). It carries a two-lane roadway with a width of 10.50 m, flanked by two sidewalks of 1.05 m each, and finished with two edge strips of 0.35 m. The bridge is supported by two H-frame reinforced concrete pylons, with heights of 110 m (west pylon) and 140 m (east pylon), of which 60 m rise above the deck level (Figure 4). The free clearance between the deck and the maximum water level is 25 m.



Figure 2. Elevation view of the Oued Dib Bridge.

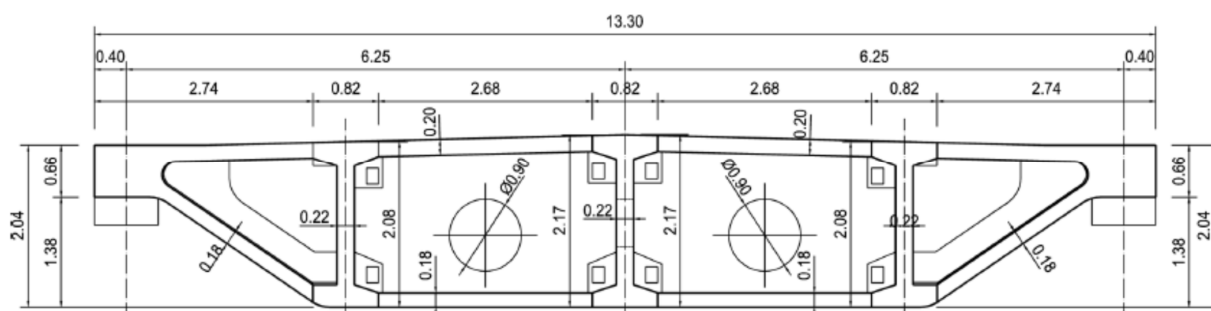


Figure 3. Cross section of the deck (units in meter).

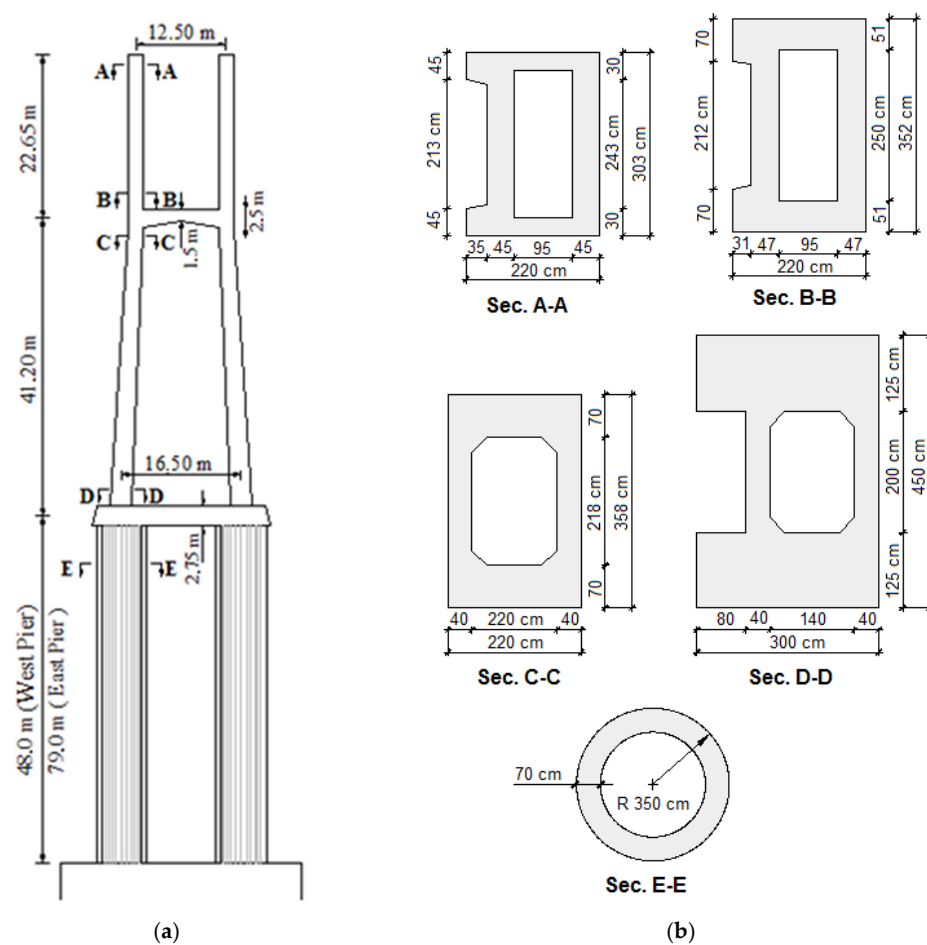


Figure 4. Description of the H-pylon: (a) Elevation view; (b) Cross sections.

The deck is supported by a system of stay cables arranged in a semi-fan configuration, anchored at the tops of the towers. A total of 88 stay cables were installed: 44 supporting

the main span and 22 for each side span. In addition, three pairs of back stays connect each pylon head to the adjacent abutments for stability. The cables are composed of steel bars with a diameter of 7 mm, with cross-sectional areas varying between 22.5 cm² and 55.5 cm².

3. Modal Analysis

The modal analysis is very important for the study of aerodynamic stability of the bridge. It was conducted using a finite element model. A linear model is able to reproduce the dynamic complex behavior of the bridge with a very good precision [31,32]. The material properties adopted for the bridge structure are presented in Table 1.

Table 1. Material properties of the bridge.

Mechanical Characteristics	Deck	Pylons	Cables
Young's modulus (GPa)	39.0	39.0	210.0
Poisson's ratio	0.2	0.2	0.3
Weight density (kN/m ³)	25	25	80

The deck was modeled using beam elements with offset rigid links to represent the cable anchor points (Figure 5). The mass of the deck was calculated as the sum of its self-weight and the superimposed dead load, yielding $m = 21,540$ Kg/m.

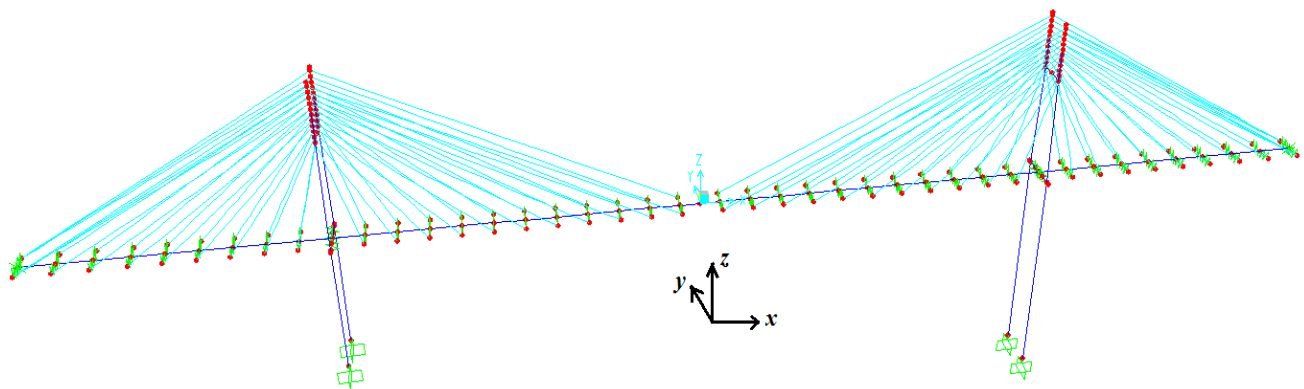


Figure 5. Finite Element Model of the Oued Dib Cable-Stayed Bridge.

The cable-stay elements were modeled as straight chord members, with an equivalent modulus of elasticity (E_{eq}) determined using Ernst's formula [33]:

$$E_{eq} = \frac{E}{1 + \frac{(\gamma L_x)^2 E}{12 (T/A)^3}} \quad (1)$$

where $E_{eq} = 190$ GPa, E is the elastic modulus of the cable material ($E = 210$ GPa), γ is the specific weight of the cable material, L_x is the horizontal length of the chord, T is the cable tension force, and A is its cross-sectional area.

The towers were modeled using frame elements with varying cross-sections, as illustrated in Figure 4. The concrete pipe-section piers were assumed to be fully fixed at their bases in all degrees of freedom. At the deck–tower connections, elastomeric bearings were represented by link elements. Their stiffness values— K_x in the longitudinal direction, K_z in the vertical direction, and K_θ in rotation about the y-axis—were defined in accordance with European Standard EN 1337 [34], yielding $K_x = 18 \text{ MN/m}$, $K_z = 11,084 \text{ MN/m}$, and $K_\theta = 123 \text{ MN}\cdot\text{m/rad}$. The deck rotation about the vertical axis was left free, while its lateral displacement and torsional rotation were restrained.

At the deck–abutment connections, rotation was allowed about the y- and z-axes but restrained about the x-axis. In terms of translation, the west abutment bearings were modeled as fixed, whereas the east abutment bearings were modeled as movable in the longitudinal direction.

3.1. Modal Frequencies

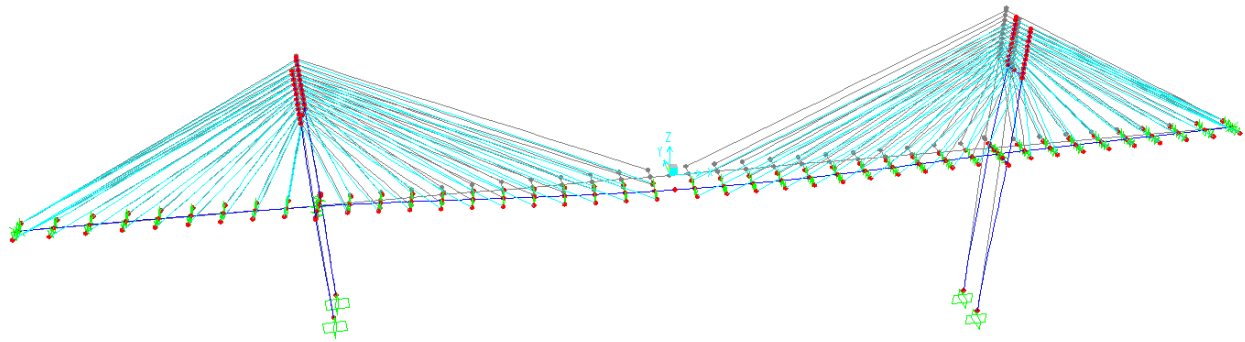
Table 2 presents the first twelve vibration modes, including their components, directions, and corresponding frequencies. Figure 6 illustrates the shapes of the first six modes, along with the twelfth mode, which represents the first torsional mode of the deck. As in most cable-stayed bridges, the Oued Dib bridge exhibits a strong coupling between the vibration modes of the deck and the motion of the tower–cable system [35].

Table 2. Modal characteristics of first 12 modes.

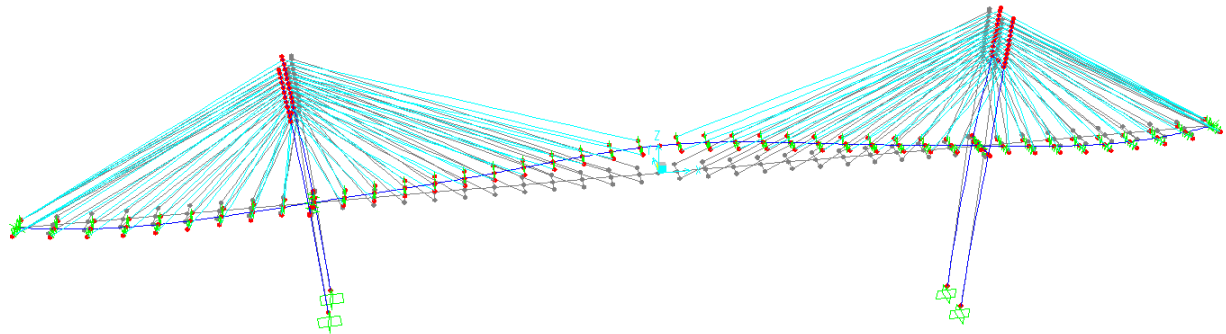
Mode Index	Frequency (Hz)	Principal Component	Other Components
1	0.356	Deck Lat Sym	Pylon East Lat
2	0.415	Deck Vert Sym	Pylons Long
3	0.511	Deck Lat Sym	Pylons Lat
4	0.609	Pylons Long	Deck Vert ASym
5	0.634	Pylon West Lat	Deck Lat Sym
6	0.824	Pylon East Long	Deck Vert ASym
7	0.960	Pylon East Lat	Deck Lat ASym
8	1.064	Pylons Long	Deck Vert Sym
9	1.171	Pylons Long	Deck Vert ASym
10	1.210	Pylons Lat	Deck Lat ASym
11	1.286	Deck Vert Sym	Pylons Long
12	1.309	Deck Tor Sym	---

Sym: Symmetric mode; Asym: Asymmetric mode; Lat: Lateral mode shape; Long: Longitudinal mode shape; Ver: Vertical mode shape; Tor: Torsional mode shape.

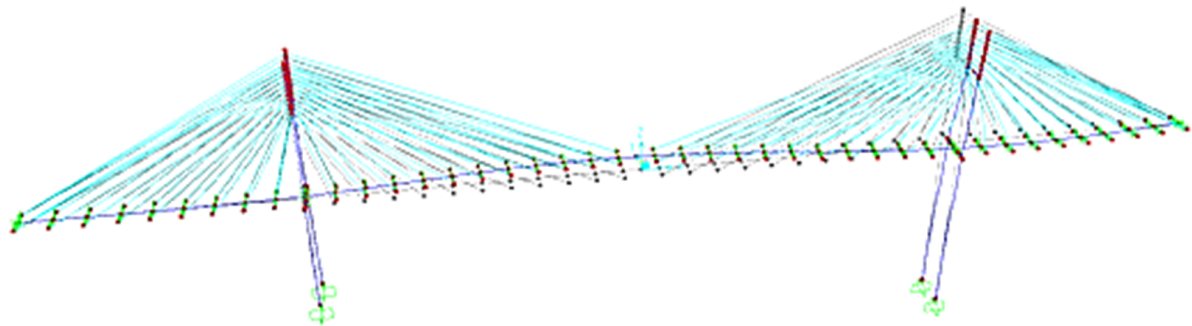
The modal frequencies obtained in this study were compared with the results of ambient vibration tests conducted by Kibboua et al. [36]. A summary of this comparison is presented in Table 3, while Figure 7 provides a graphical illustration. The analytical model shows good agreement with the experimental results in the vertical direction, particularly for the first two modes. However, the finite element model exhibits greater flexibility in the lateral direction and increased rigidity in the longitudinal direction. This discrepancy can most plausibly be attributed to changes in the boundary conditions of the deck supports over the structure’s lifetime, likely caused by pathologies affecting one or more bearings.



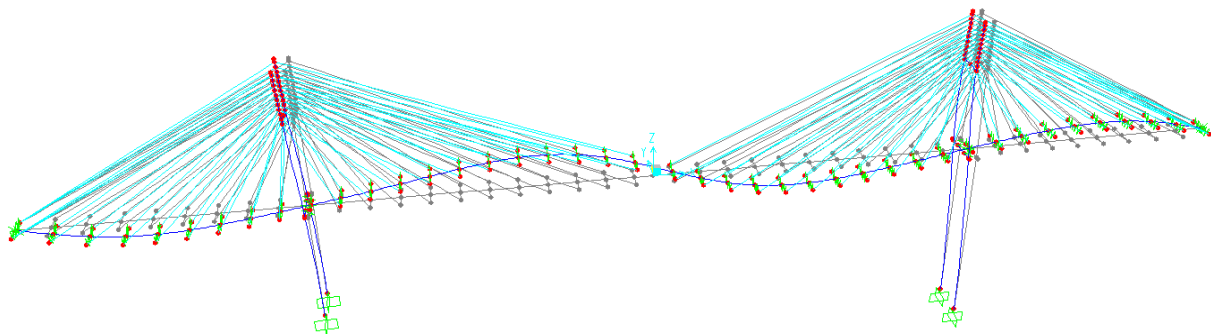
(a) Mode 1: Coupled Deck Lat Sym—Pylon East Lat, $f = 0.356$ Hz.



(b) Mode 2: Coupled Deck Ver Sym—Pylons Long, $f = 0.415$ Hz.

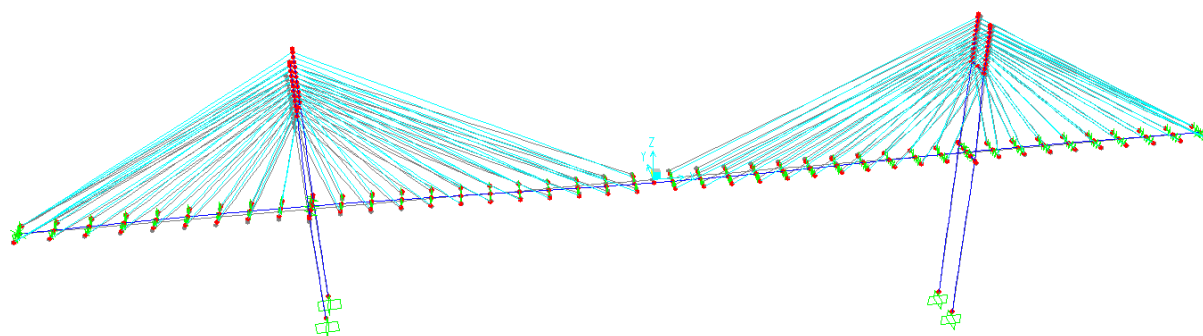


(c) Mode 3: Coupled Deck Lat Sym—Pylons Lat, $f = 0.511$ Hz.

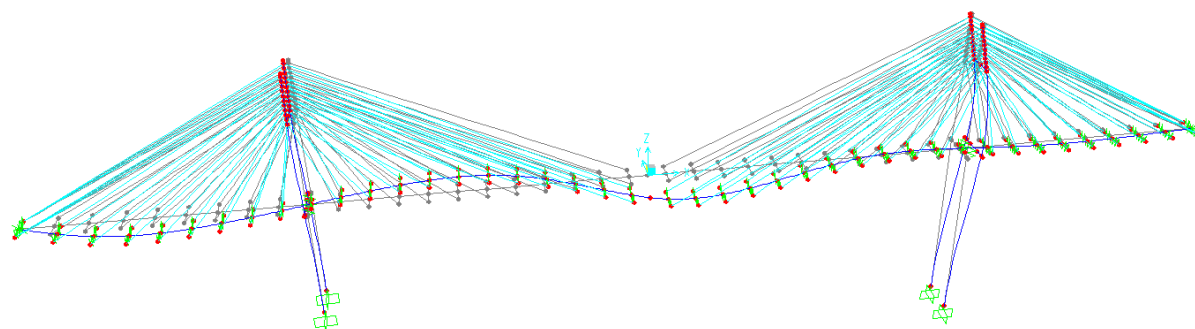


(d) Mode 4: Coupled Pylons Long—Deck Ver ASym, $f = 0.609$ Hz.

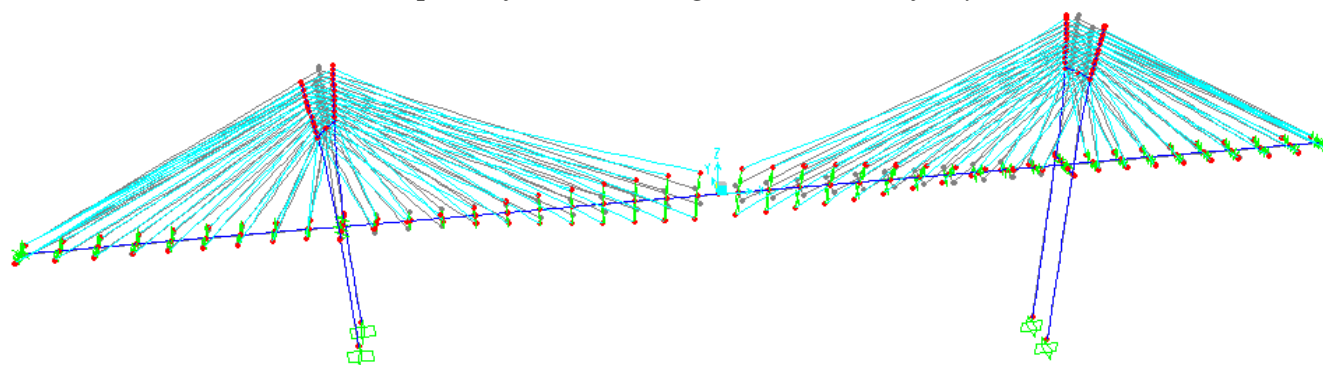
Figure 6. *Cont.*



(e) Mode 5: Coupled Pylon West Lat—Deck Lat Sym, $f = 0.634$ Hz.



(f) Mode 6: Coupled Pylon East Long—Deck Ver ASym, $f = 0.824$ Hz.



(g) Mode 12: Deck Tor Sym, $f = 1.309$ Hz.

Figure 6. The 3D shapes of the first 6 modes plus the 12th mode.

Table 3. A comparison of the first six mode frequencies between the measurement and the FE model in the Oued Dib bridge.

Direction	Measurement (Hz)	FE Model (Hz)	Error (%)
Lat1	0.390	0.356	8.7
Lat2	0.550	0.511	7.1
Ver1	0.400	0.415	3.7
Ver2	0.630	0.609	3.3
Long1	0.380	0.415	9.2
Long2	0.560	0.609	8.7

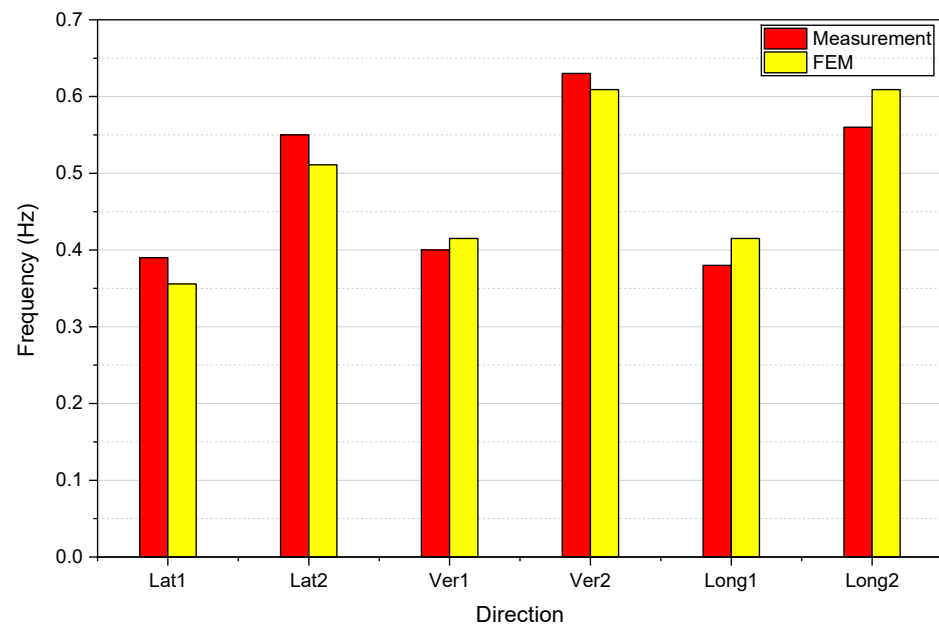


Figure 7. Measurement versus FE model of the first six modal frequencies of the Oued Dib bridge.

The first three modal frequencies in the vertical, lateral, and torsional directions of the Oued Dib Bridge are compared with those of four other cable-stayed bridges (Table 4 and Figure 8): (1) Quincy Bayview Bridge [37], (2) Guanhe Bridge [38], (3) Vasco da Gama Bridge [39], and (4) the Second Severn Crossing [40]. These bridges were selected primarily for their similarity in deck material (concrete or composite concrete–steel) and their span length, which is comparable to that of the studied bridge.

Table 4. A comparison of measured natural frequencies: Oued Dib bridge vs. other bridges.

N°	Name	Country	Deck Material	Main Span L (m)	Side Spans L' (m)	Deck Wide B (m)	1st Vert Mode (Hz)	1st Lat Mode (Hz)	1st Tor Mode (Hz)
1	Quincy Bayview	USA	composite	274	2 × 134	14.2	0.37	0.56	0.56
2	Oued Dib	Algeria	concrete	280	2 × 111	13.3	0.40	0.39	0.64
3	Guanhe	China	composite	340	2 × 150	34.0	0.38	0.39	0.63
4	Vasco da Gama	Portugal	concrete	420	2 × 204.6	31.0	0.34	0.30	0.47
5	2nd Severn Crossing	UK	composite	456	2 × 245.3	34.6	0.35	0.34	0.56

Although the bridges are not identical, the comparison reveals a consistent modal behavior across the cases, with the notable exception of the Quincy Bayview Bridge, which exhibits greater lateral flexibility. Another key observation is that, for all five bridges, the torsional modes remain reasonably separated from the flexural modes. This aligns with general design recommendations for cable-stayed bridges, aimed at reducing the risk of coupled flutter involving two degrees of freedom. Such instabilities typically arise when the two modes are too closely spaced. For the bridges considered, the ratio between the torsional and flexural modes ranges from 1.4 to 1.7, with the Oued Dib Bridge exhibiting a ratio of 1.6. These values exceed the minimum recommended threshold of 1.4, as specified in the technical guidelines [41].

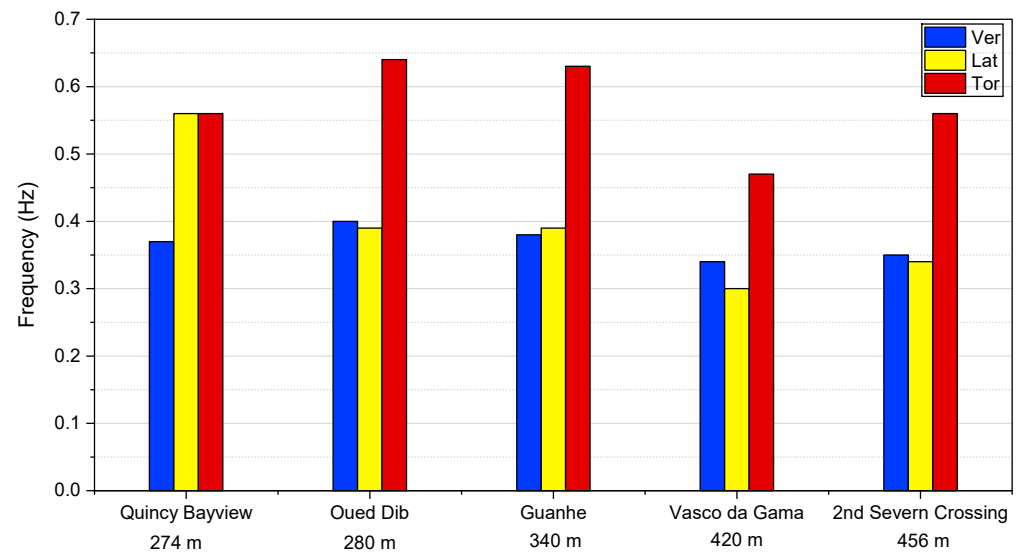


Figure 8. A comparison of measured natural frequencies: Oued Dib bridge vs. other bridges.

3.2. Vibration Mode Shapes

The normalized shapes of the first five vertical and torsional modes are presented in Figures 9 and 10, respectively, based on 43 nodes distributed along the deck. Mode 12 (1.309 Hz), the most dominant in torsion, is symmetrical, unlike the anti-symmetrical torsional mode of the Tacoma Narrows Bridge (0.2 Hz), which is comparable to mode 20 of the Oued Dib Bridge.

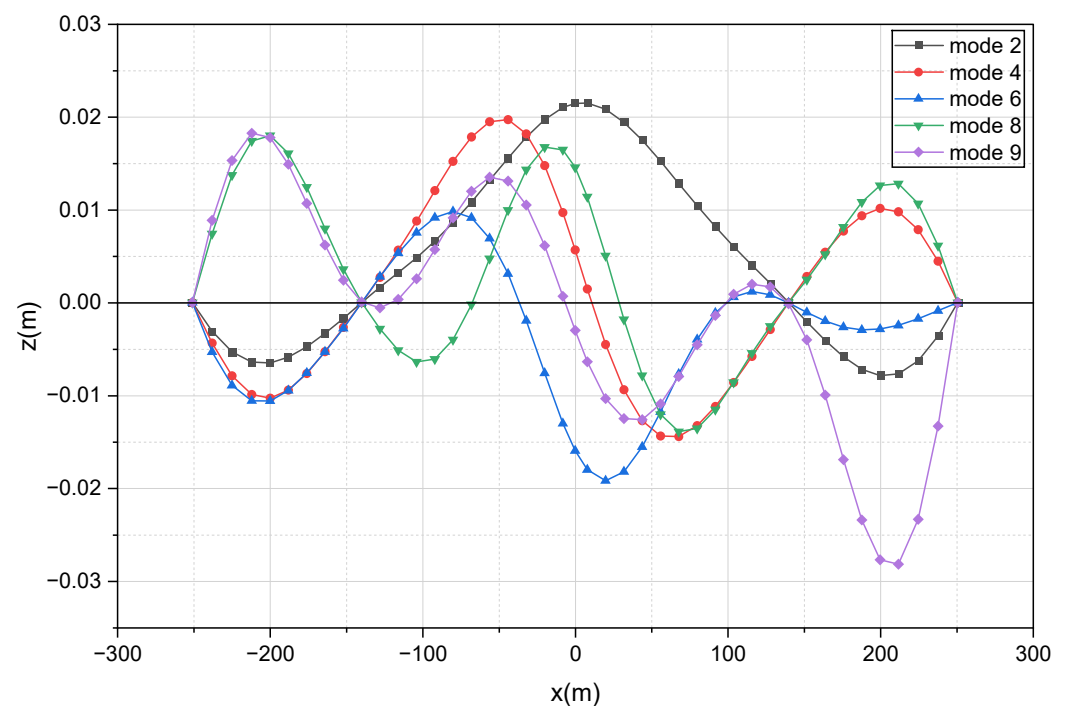


Figure 9. Normalized shapes of the first 5 vertical modes of the Oued Dib Bridge.

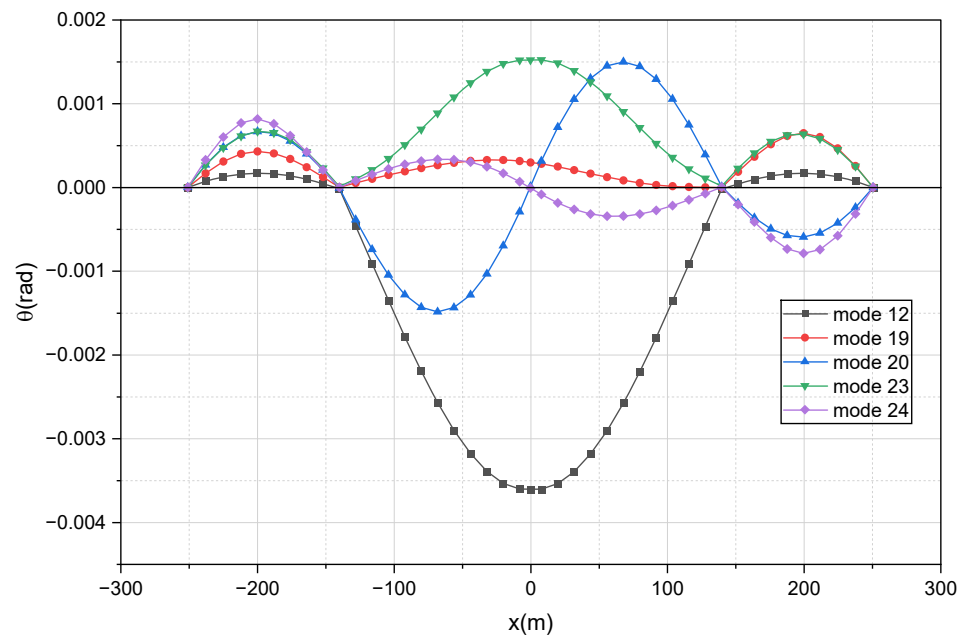


Figure 10. Normalized shapes of the first 5 torsional modes of the Oued Dib Bridge.

3.3. Estimation of the Modal Damping

The modal damping ratios measured on the Oued Dib Bridge are presented in Figure 11, alongside results from ambient vibration tests conducted on other bridges. Table 5 summarizes the damping ratio values for each case. These values are generally low, ranging from 0.12% to 1.4%, which makes this type of cable-stayed bridge particularly vulnerable to wind effects.

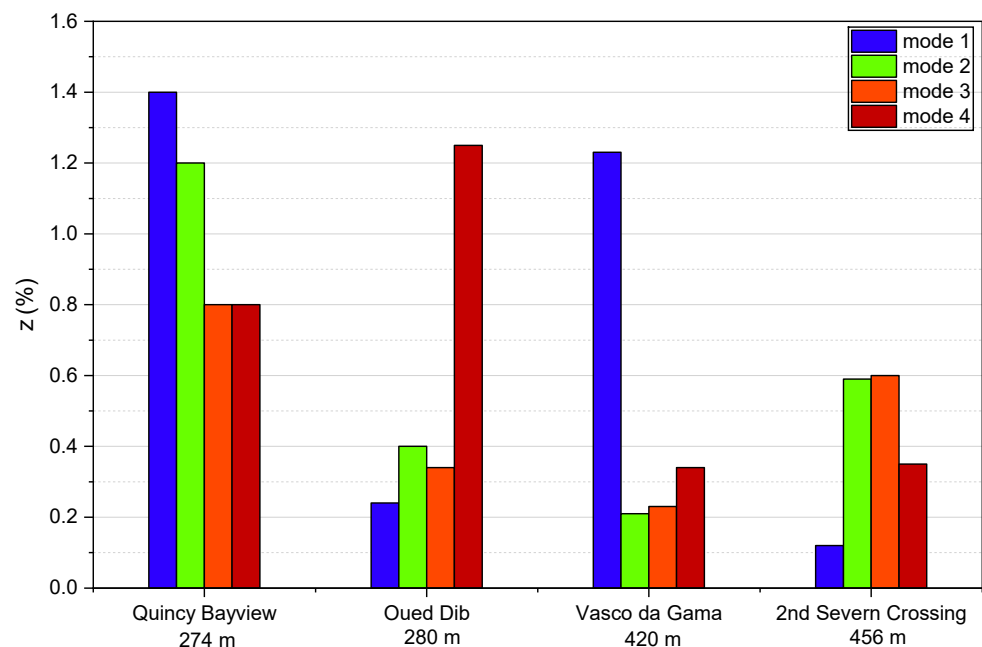


Figure 11. A comparison of the damping ratio measured for the first four modes: Oued Dib bridge vs. other bridges.

Figure 12 illustrates the relationship between the measured damping ratios and the natural frequencies for the different cable-stayed bridges. This comparison highlights the complex nature of the damping–frequency relationship in such structures. Unlike buildings,

where damping behavior can often be represented by simple or extended Rayleigh models, no straightforward law appears to adequately describe damping in cable-stayed bridges.

Table 5. Modal damping ratios measured across four distinct cable-stayed bridges.

Quincy Bayview L = 274 m		Oued Dib L = 280 m		Vasco da Gama L = 420 m		2nd Severn Crossing L = 456 m	
f [Hz]	ζ [%]	f [Hz]	ζ [%]	f [Hz]	ζ [%]	f [Hz]	ζ [%]
0.37	1.4	0.39	0.24	0.30	1.23	0.34	0.12
0.56	1.2	0.4	0.4	0.34	0.21	0.6	0.59
0.63	0.8	0.56	0.34	0.46	0.23	0.82	0.60
0.7	0.8	0.63	1.25	0.59	0.34	0.98	0.35
0.74	1.3	-	-	0.65	0.37	1.34	0.36
0.8	1.0	-	-	0.71	0.78	-	-
0.89	0.9	-	-	0.81	0.48	-	-
1.18	1.0	-	-	0.98	0.74	-	-
1.37	0.9	-	-	-	-	-	-

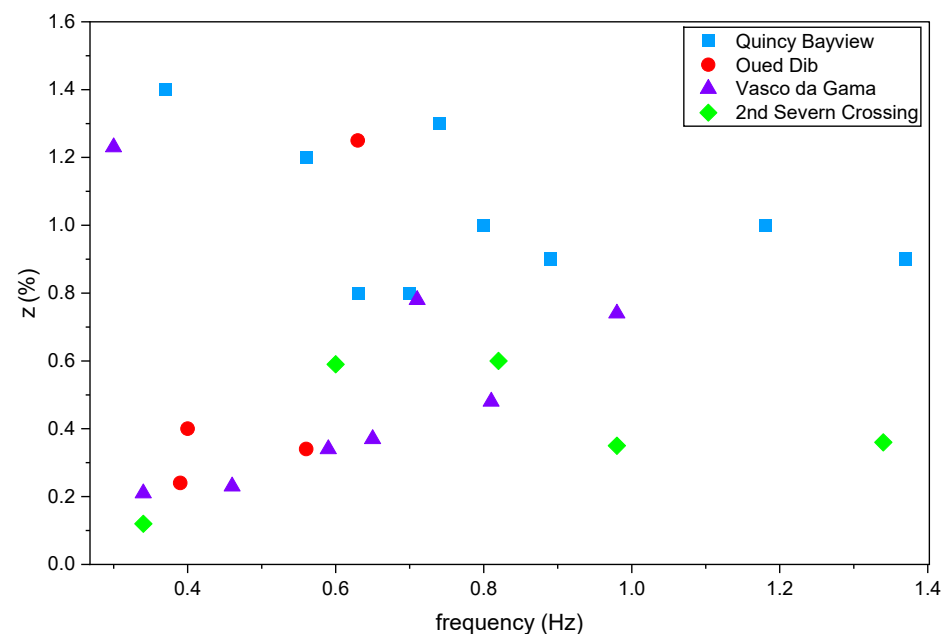


Figure 12. Measured damping ratios versus natural frequencies across various cable-stayed bridges.

The modal damping ratio measured on the Oued Dib bridge by Kibboua et al. [36] is $\zeta = 0.40\%$ for the first vertical bending mode. To avoid overestimating damping in the aeroelastic analysis of the deck, this same value was adopted for all other modes.

4. CFD Modeling

4.1. CFD Set-Up

According to Algerian snow and wind regulations (RNV 2013) [42] and Eurocode 1 (ENV 1991-1-4) [43], the maximum wind speed at the bridge site is estimated as $U_{max} = 33.3$ m/s (120 km/h), corresponding to a 50-year return period. The turbulence intensity is $I_v = 12.8\%$ [44]. For the CFD simulations, a mean wind velocity of $U = 20$ m/s was applied, with angles of attack (AoA) ranging from -10° to $+10^\circ$ in increments of 2° . Given the air density $\rho = 1.225$ kg/m³, the dynamic viscosity $\mu = 1.837 \times 10^{-5}$ kg/(m·s), and the deck width $B = 13.3$ m, the Reynolds number (Re) is obtained from the relation

$$Re = \frac{\rho U B}{\mu} \quad (2)$$

The resulting Reynolds number is $Re = 1.77 \times 10^7$, indicating a fully turbulent flow regime. The fluid domain around the deck was defined according to the recommendations of [45], as shown in Figure 13, with the corresponding boundary conditions summarized in Table 6. The deck surface was assigned a concrete roughness of 0.3 mm. The computational mesh was structured, consisting of 166,604 cells and 167,567 nodes (Figures 14 and 15).

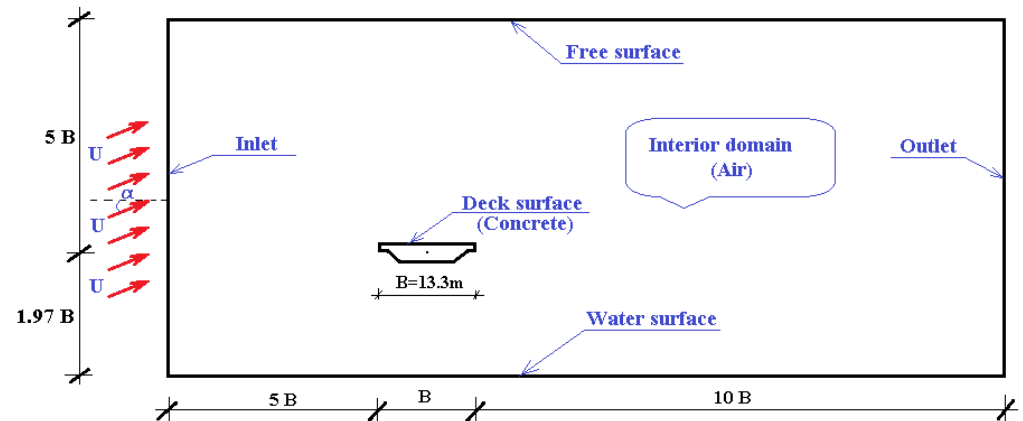


Figure 13. Domain regions and dimensions.

Table 6. Boundary conditions.

Boundary	Velocity (U)	Pressure (P)
Inlet	$U = 20 \text{ m/s}$; $\alpha = \text{variable}$	Zero gradient
Outlet	Zero gradient	$P = 0$
Deck surface	$U_x = 0$; $U_y = 0$; roughness = 0.3 mm	Zero gradient
Water surface	$U_x = 0$; $U_y = 0$; roughness = 0	Zero gradient
Free surface	Zero gradient	Zero gradient

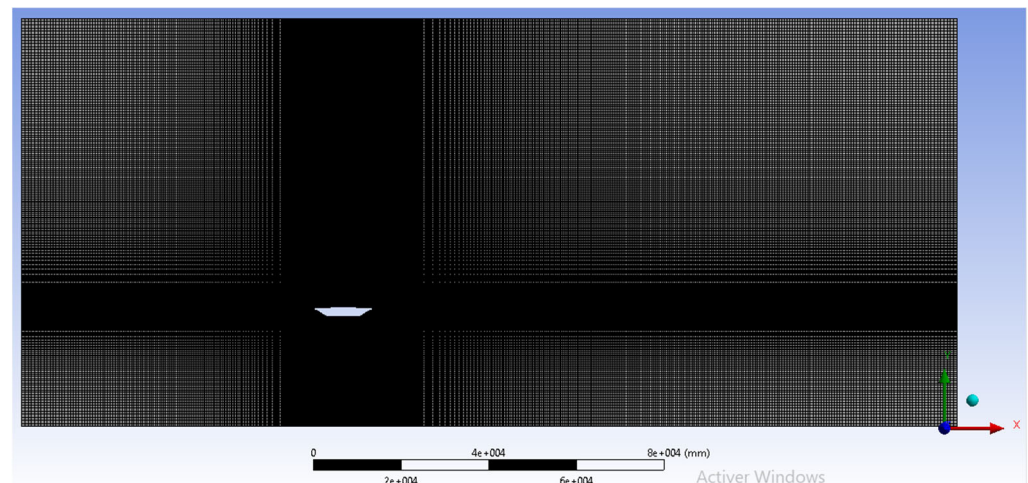


Figure 14. Mesh for the entire flow domain.

To achieve accurate resolution of the boundary layer at high Reynolds numbers, it is recommended to apply a wall function in the range $30 < Y^+ < 200$. In this study, a value of $Y^+ = 30$ was adopted, corresponding to a minimum cell height of 1.3 mm at the air–deck interface. A steady-state approach was employed using the turbulent RANS $k-\omega$ SST model [46,47]. This model ensures reliable prediction of the flow field, as it applies the $k-\omega$ formulation near the air–deck interface and transitions to the $k-\epsilon$ formulation in regions farther from the wall.

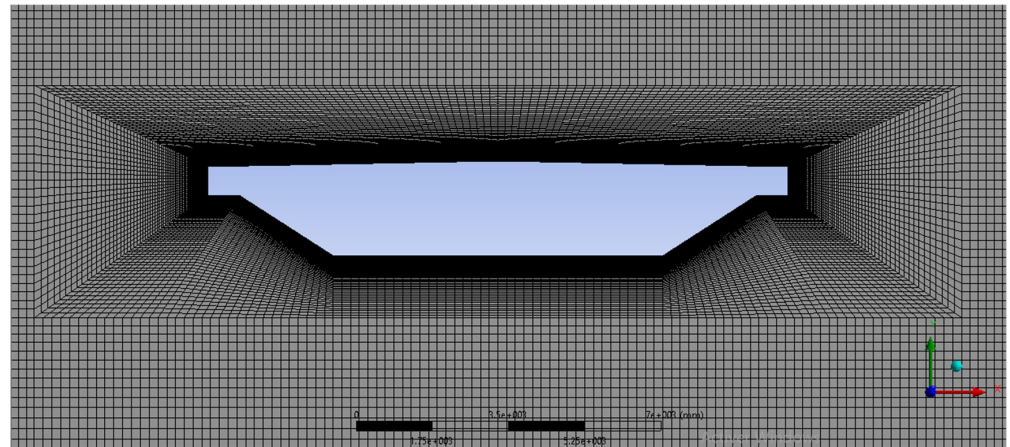


Figure 15. Mesh for the bridge section.

4.2. CFD Results

The pressure contour in Figure 16 reveals the presence of overpressure at the left edge of the deck, which obstructs airflow and induces backflow in the opposite direction. Since the deck is a bluff body rather than a streamlined profile, a significant portion of its surface is subjected to under pressure, leading to boundary layer separation. This behavior is further confirmed by the velocity contour (Figure 17) and the velocity vectors (Figure 18), both of which clearly highlight the separation zones. These flow characteristics give rise to the vortex-shedding phenomenon [48,49], as illustrated in Figure 19.

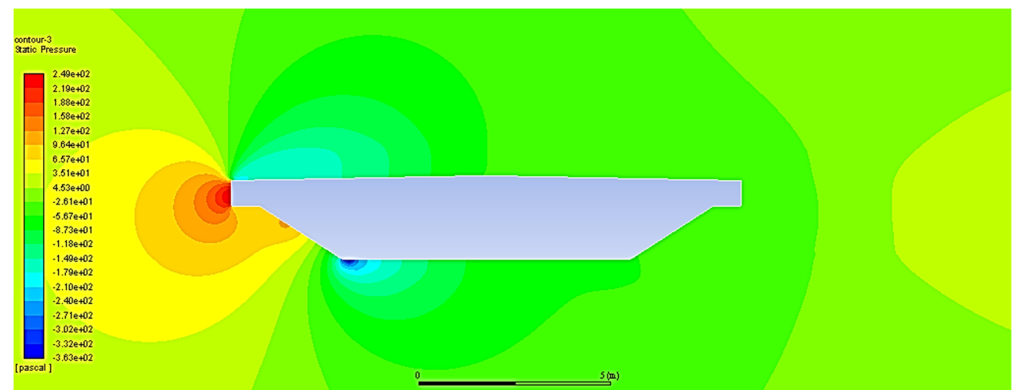


Figure 16. Contour of static pressure ($\alpha = 0$).

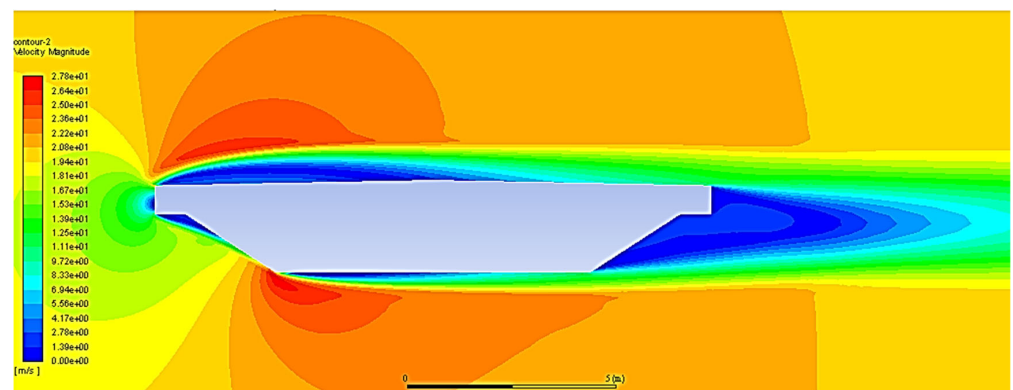


Figure 17. Contour of velocity magnitude ($\alpha = 0$).

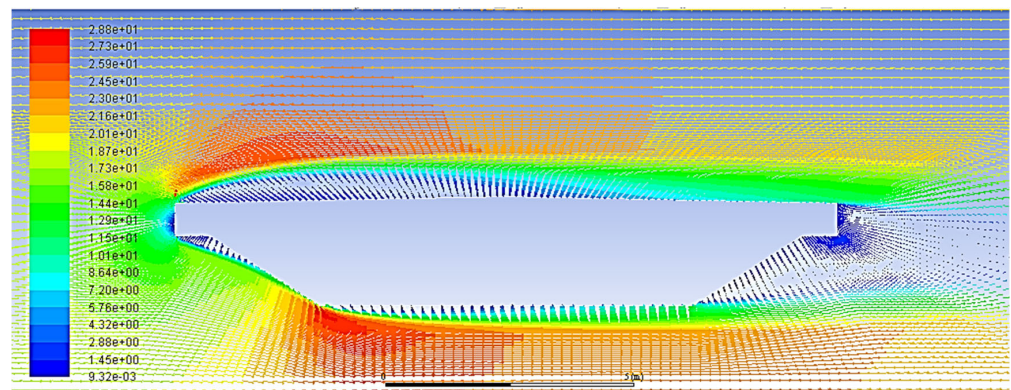


Figure 18. Vectors of velocity, direction and magnitude ($\alpha = 0$).

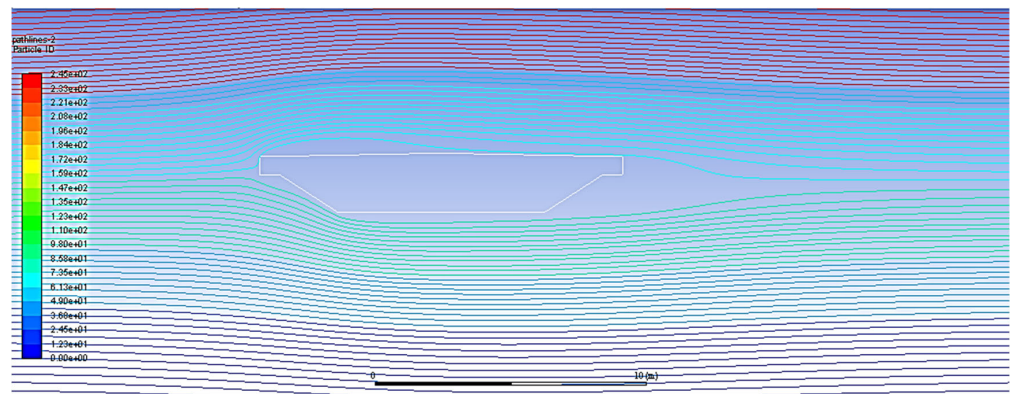


Figure 19. Streamlines around the deck ($\alpha = 0$).

The CFD analysis also enabled the calculation of the static aerodynamic coefficients for each angle of attack (α), ranging from -10° to $+10^\circ$ in 2° increments. These coefficients— C_D for drag, C_L for lift, and C_M for moment—are illustrated in Figure 20, with their numerical values summarized in Table 7. The C_M curve decreases with increasing α , reflecting the fact that the deck is a bluff body; in contrast, profiled decks typically exhibit increasing C_M curves.

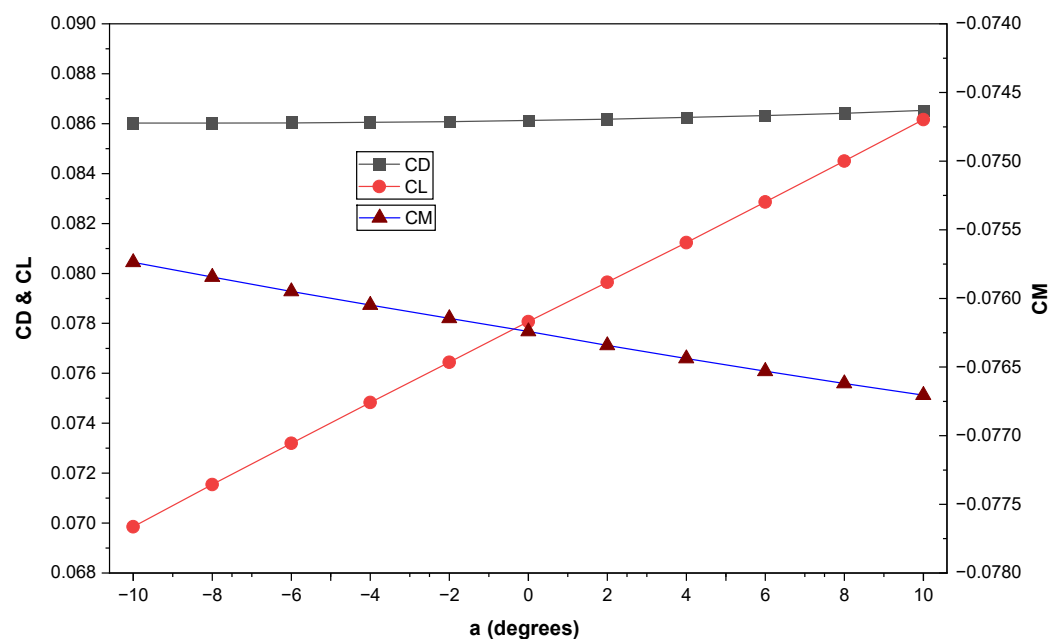


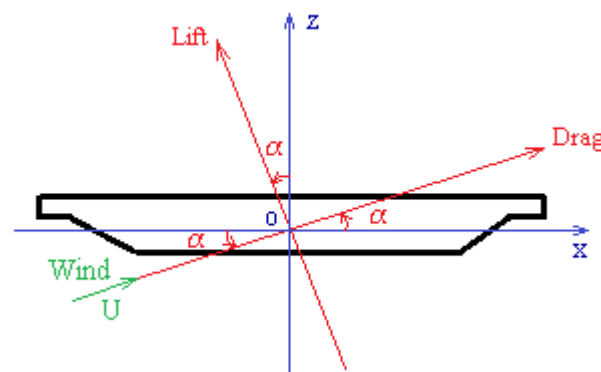
Figure 20. C_D , C_L and C_M versus angle of attack α .

Table 7. C_D , C_L and C_M vs. angle of attack α .

α (°)	C_D	C_L	C_M
−10.0	0.086031	0.069853	−0.075736
−8.0	0.086031	0.071542	−0.075845
−6.0	0.086036	0.073197	−0.075949
−4.0	0.086052	0.074833	−0.076048
−2.0	0.086088	0.076442	−0.076145
0.0	0.086128	0.078070	−0.076241
2.0	0.086181	0.079649	−0.076342
4.0	0.086253	0.081237	−0.076438
6.0	0.086329	0.082864	−0.076530
8.0	0.086419	0.084508	−0.076619
10.0	0.086529	0.086167	−0.076705

Instead of expressing the static coefficients in the wind coordinate system (D, L), they can also be represented in the deck coordinate system (x, z). From Figure 21, it follows that

$$\left. \begin{aligned} C_x &= C_D \cos \alpha - C_L \sin \alpha \\ C_z &= C_D \sin \alpha + C_L \cos \alpha \end{aligned} \right\} \quad (3)$$

**Figure 21.** Static coefficients in both the wind and deck reference frames.

The C_M coefficient remains unchanged in both coordinate systems.

Using these coefficients, the aerodynamic forces exerted on the bridge for a unit length (1 m) can be calculated according to the following formulas:

$$D = \frac{1}{2} \rho U^2 B C_D(\alpha) \quad (4a)$$

$$L = \frac{1}{2} \rho U^2 B C_L(\alpha) \quad (4b)$$

$$M = \frac{1}{2} \rho U^2 B^2 C_M(\alpha) \quad (4c)$$

where D , L , and M denote the drag force, lift force and pitching moment, respectively.

5. Aeroelastic Instabilities and Bridge Performance

5.1. Vortex Shedding

Like any other bluff body, the bridge deck experiences boundary layer separation when subjected to airflow (Figure 19). This separation leads to the formation of vortices

along both the right and left edges of the deck (Figure 22). These vortices are shed at a characteristic frequency [50]:

$$f_{Vor} = \frac{St U}{D} \quad (5)$$

where St is the Strouhal number which depends on the geometric shape of the deck.

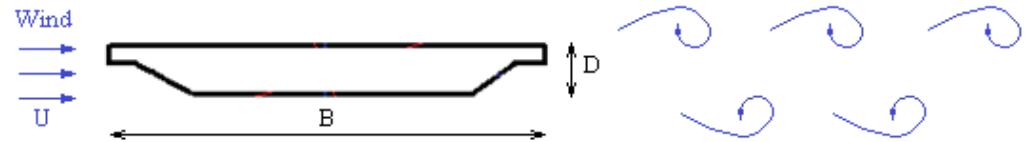


Figure 22. Vortex generation behind the bridge deck.

If the vortex-shedding frequency f_v is close to the vertical vibration frequency f_{vs} of the deck in flexural mode, a resonance phenomenon may occur, amplifying the vertical displacements of the deck and potentially leading to instability. The corresponding critical wind speed is determined using Equation (6) as follows:

$$U_{cr,v} = \frac{f_{vs} D}{St} \quad (6)$$

This instability is further aggravated by the lock-in phenomenon, which has been observed experimentally. Even when the wind speed exceeds the critical value by a small margin, the resonance persists (Figure 23) [51].

EN 1991-1-4 [43] provides Strouhal number values based on the geometry of the obstacle. For the bridge deck, the value is estimated as $St = 0.1$. With a deck thickness of $D = 2.04$ m, and by substituting the frequencies of the first five bending modes into Equation (6), the corresponding critical wind speeds are obtained, as presented in Table 8.

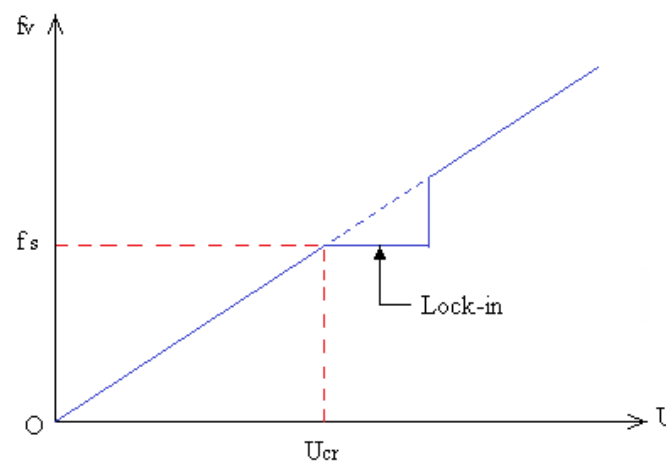


Figure 23. The lock-in phenomenon in vortex shedding.

Table 8. Critical wind speeds for vortex shedding.

Mode Index	f_{vs} (Hz)	$U_{cr,v}$ (m/s)
mode 2	0.415	8.47
mode 4	0.609	12.42
mode 6	0.824	16.81
mode 8	1.064	21.71
mode 9	1.171	23.89

The critical speeds in Table 8 are lower than the maximum wind speed $U_{max} = 33.3 \text{ m/s}$. In this case, the sensitivity of the deck to vortex shedding must be verified. This sensitivity is governed by the Scruton number, expressed as

$$S_c = \frac{2 \delta_s m}{\rho D^2} \quad (7)$$

where δ_s is the logarithmic decrement of structural damping, expressed by the equation

$$\delta_s = \frac{2 \pi \zeta}{\sqrt{1 - \zeta^2}} \quad (8)$$

The calculation of the Scruton number is shown in Table 9.

Table 9. Scruton number calculation.

$\zeta = 0.4\%$	$\delta_s = 0.025$	$m = 21,540$ Kg/mL $S_c = 211.26$	$\rho = 1.225$ Kg/m ³	$D = 2.04 \text{ m}$
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Experience has shown that no risk of excessive vibrations occurs when the Scruton number exceeds 20, whereas the risk becomes significant for values below 10 [52]. In the present case, since $S_c = 211.26$, the bridge deck is not susceptible to instability due to vortex shedding.

5.2. Flutter in a Single Vibration Mode

5.2.1. Galloping

Galloping is a form of vertical vibration of the bridge deck, occurring in the bending mode along the z-axis. The non-profiled (bluff body) sections of the deck are particularly susceptible to this type of instability.

As shown in Figure 24, the analysis focuses exclusively on the galloping motion along the z-axis, since no drag divergence is observed along the x-axis [53]. The corresponding equation of motion in the z-direction can be expressed as follows:

$$m \ddot{z} + 2 m \zeta \omega_z \dot{z} + m \omega_z^2 z = F_z = \frac{1}{2} \rho U^2 D C_z(\alpha) \quad (9)$$

where $\ddot{z}$, $\dot{z}$ and z represent the vertical acceleration, velocity, and displacement of the deck, respectively.

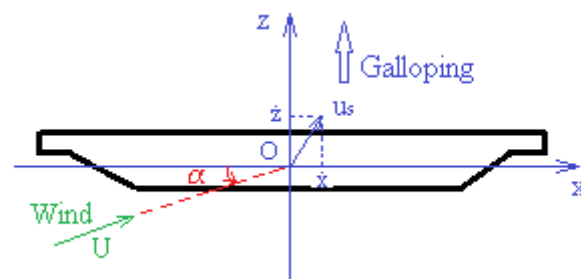


Figure 24. Schematic representation of the galloping.

The linearization of the aerodynamic force F_z as a function of speed $\dot{z}$, for an angle of attack $\alpha = 0$, is written as

$$F_z = -\frac{1}{2} \rho U D C'_z(0) \dot{z} \quad (10)$$

Equation (9) of motion becomes

$$m\ddot{z} + \left[2m\xi\omega_z + \frac{1}{2}\rho U D C_z'(0) \right] \dot{z} + m\omega_z^2 z = 0 \quad (11)$$

The motion becomes unstable if the damping term is zero or negative. The corresponding critical speed is

$$U_{C,G} = \frac{-4m\xi\omega_z}{\rho D C_z'(0)} \quad (12)$$

The condition for $U_{C,G}$ to be positive is $C_z'(0) < 0$. According to Equation (3), this gives

$$C_z'(0) = C_D + C_L' < 0 \quad (13)$$

This corresponds to Den Hartog's criterion for gallop instability [54]. In this study, the values are $C_D(0) = 0.086$ and $C_L'(0) = 0.045$, which yield $C_z'(0) = 0.131 > 0$. Therefore, there is no risk of galloping.

Eurocode 1 [43] also provides a formula for calculating the critical wind speed that may induce galloping:

$$U_{C,G} = \frac{2S_c}{a_G} f_{1,z} D \quad (14)$$

where $f_{1,z}$ is the first natural frequency of the deck in vertical displacement, and a_G is the instability coefficient defined in EN 1991-1-4 [43], with $a_G = 4.7$. The condition to be satisfied is:

$$U_{C,G} > 1.25 U_{\max} \quad (15)$$

According to the calculations in Table 10, we have $U_{C,G} = 76.11 \text{ m/s} > 1.25 U_{\max} = 41.63 \text{ m/s}$. So, there is no risk of galloping.

Table 10. $U_{C,G}$ calculation.

$S_c = 211.26$	$a_G = 4.7$	$f_{1,z} = 0.415 \text{ Hz}$ $U_{C,G} = 76.11 \text{ m/s}$	$D = 2.04 \text{ m}$	$U_{\max} = 33.3 \text{ m/s}$
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5.2.2. Torsional Divergence

The analysis is limited to the linear behavior of the bridge deck with respect to torsional vibration. Under the action of the aerodynamic moment, and neglecting the effect of damping, the deck oscillates around the center of torsion C with an angle θ (Figure 25).

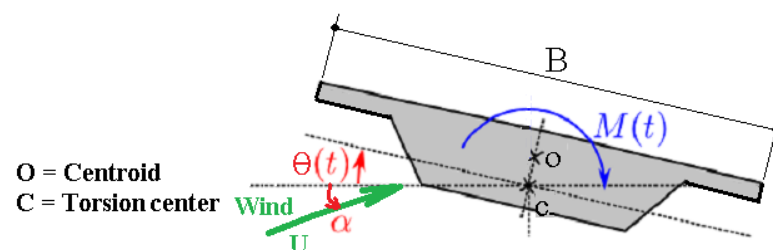


Figure 25. Torsion motion of the bridge deck.

The corresponding equation of motion is expressed as

$$J\ddot{\theta} + K_\theta\theta = M = \frac{1}{2}\rho B^2 U^2 C_M(\alpha) \quad (16)$$

where J is the torsional constant of the deck around the center C . K_θ is the torsional stiffness, which is expressed as a function of the natural frequency ω_θ in the form

$$K_\theta = J \omega_\theta^2 \quad (17)$$

After linearizing the aerodynamic moment M as a function of rotation θ , for an angle of attack $\alpha = 0$, one obtains

$$M = \frac{1}{2} \rho B^2 U^2 C'_M(0) \theta \quad (18)$$

Substituting Equations (17) and (18) into Equation (16), the equation of motion becomes

$$\ddot{\theta} + \left(\omega_\theta^2 - \frac{\rho B^2 U^2 C'_M(0)}{2J} \right) \theta = 0 \quad (19)$$

For aerodynamic instability to occur, the stiffness term in Equation (19) must be equal to or less than zero. This gives a critical speed

$$U_{cr,tor} = \frac{\omega_\theta}{B} \sqrt{\frac{2J}{\rho C'_M(0)}} \quad (20)$$

EN 1991-1-4 [43] sets the condition

$$U_{cr,tor} > 2 U_{max} \quad (21)$$

The term under the square root in Equation (19) must be positive. Accordingly, $C'_M(0)$ must also be positive. For the present case, where the deck is non-profiled, the value is $C'_M(0) = -0.00286 < 0$, indicating that there is no risk of torsional divergence.

However, recent research [55] has shown that, in the case of cable-stayed bridges where structural non-linearity is significant and in the presence of wind turbulence, these two parameters can cause torsional divergence.

5.3. Flutter with Two Degrees of Freedom

This type of flutter caused the collapse of the Tacoma Narrows Bridge in the United States in 1940 [10]. Under the action of the wind, the deck oscillates simultaneously in vertical translation along the z -axis and in torsion about point C (Figure 26).

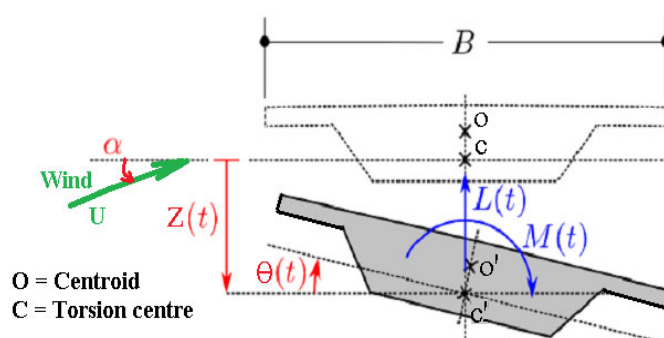


Figure 26. 2DOF floating bridge deck.

Neglecting the effects of damping, the equation of motion can be written as

$$\begin{aligned} m \ddot{z} + m \omega_z^2 z &= F_z = \frac{1}{2} \rho B U^2 C_z(\alpha) \\ J \ddot{\theta} + K_\theta \theta &= M = \frac{1}{2} \rho B^2 U^2 C_M(\alpha) \end{aligned} \quad (22)$$

After linearizing the force F_z and the moment M as a function of the rotation θ , for an angle of attack $\alpha = 0$, one obtains

$$\begin{aligned} m \ddot{z} + m \omega_z^2 z &= F_z = \frac{1}{2} \rho B U^2 \theta C'_z(0) \\ J \ddot{\theta} + J \omega_\theta^2 \theta &= M = \frac{1}{2} \rho B^2 U^2 \theta C'_M(0) \end{aligned} \quad (23)$$

By developing the system of Equation (23) which is coupled in terms of rotation θ , the critical velocity is obtained:

$$U_{cr, fl} = \frac{2\pi}{B} \sqrt{\frac{-2J(f_z^2 - f_\theta^2)}{\rho C'_M(0)}} \quad (24)$$

From Equation (24), since $C'_M(0) < 0$, the term under the square root is positive only if $f_z > f_\theta$. As mode 12 is the first torsional mode ($f_\theta = 1.309$ Hz), the first bending mode with an order greater than 12 must be considered. Mode 13 is a symmetrical flexural mode of the deck, coupled with the longitudinal displacement of the west pylon. Therefore, this mode is selected, with a frequency of $f_z = 1.402$ Hz.

The calculation of the critical speed is presented in Table 11. Note that $U_{cr, fl} = 1143$ m/s is much higher than $U_{max} = 33.3$ m/s. Therefore, there is no risk of flutter at 2DoF.

Table 11. $U_{cr, fl}$ Calculation.

$B = 13.3$ m	$J = 40,694$ Kg.m	$\rho = 1.225$ Kg/m ³	$C'_M(0) = -0.00286$ $U_{cr, fl} = 1143$ m/s	$f_\theta = 1.309$ Hz	$f_z = 1.402$ Hz
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5.4. Study Limitations

While the study provides valuable insights, several limitations must be acknowledged:

- Potential fatigue due to nearly continuous vortex shedding vibrations remains a concern and should be carefully monitored.
- The CFD simulations were conducted in steady-state conditions, which may not capture unsteady flow phenomena such as buffeting or transient gust effects.
- Only the deck section was modeled in CFD, without explicitly considering the influence of pylons, cables, and traffic loading.
- Aeroelastic checks were performed using linear assumptions; nonlinear effects, especially under extreme wind events, were not fully addressed.
- The analysis was limited to a single wind speed ($U = 20$ m/s) for static coefficient extraction, whereas variable wind intensities and turbulence spectra could affect stability margins.

6. Conclusions and Outlook

This study investigated the aeroelastic stability of the Oued Dib Bridge, Algeria's first cable-stayed bridge, through a combined modal analysis, CFD simulations, and N aeroelastic assessment. The key findings can be summarized as follows:

- The modal analysis in the vertical direction gave results in very good agreement with ambient vibration measurements, with a maximum deviation of 3.7% of the natural frequencies.
- The Oued Dib Bridge, when compared with four other bridges, exhibits good consistency in its modal behavior. The separation between its torsional and bending natural frequencies, with a ratio of 1.6, places it outside the risk associated with two-degree-of-freedom flutter. This demonstrates the good structural design of this bridge.
- Cable-stayed bridges exhibit low structural damping, which makes them vulnerable to wind effects. In addition, this type of structure shows strong coupling between vibration modes, which makes it impossible to use Rayleigh's law to calculate modal damping.
- CFD analysis highlighted flow separation, vortex initiation, and backflow regions around the bluff deck section, confirming its susceptibility to vortex-induced phenomena.
- The adopted wall function value of $Y^+ = 30$ proved appropriate for capturing boundary-layer behavior at high Reynolds numbers ($Re = 1.77 \times 10^7$).
- The coefficient curve $C_M(\alpha)$ decreased with AoA, consistent with typical bluff-body decks, unlike streamlined profiles where it increases. The negative sign of the derivative $C_M'(\alpha)$ places the structure outside the risk of torsional divergence and two-degree-of-freedom flutter.
- Aeroelastic checks confirmed no risk of vortex shedding resonance, galloping, torsional divergence, or classical flutter under the design wind speed ($U_{max} = 33.3$ m/s).

Building upon the current research, future studies should aim to

- Extend CFD simulations to unsteady (URANS or LES) approaches to better capture vortex shedding dynamics and buffeting responses.
- Investigate the combined aeroelastic behavior of the deck and stay cables, particularly under rain–wind excitation, which can accelerate cable fatigue.
- Explore aerodynamic countermeasures, such as deck edge modifications or fairings, to mitigate vortex-induced vibrations.
- Perform aeroelastic studies under traffic loading, as moving vehicles can significantly alter the aerodynamic response of the bridge.
- Validate CFD results with wind tunnel experiments to further confirm accuracy and reliability.

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Nomenclature

U	wind speed [m/s]
α	wind angle of attack [°]
ρ	air density [Kg/m ³]
μ	dynamic viscosity of air [Kg/m.s]
B	bridge deck width [m]
D	bridge deck thickness [m]
m	mass of the deck per unit length [Kg/m]
J	torsional constant of the deck per unit length [Kg.m ² /m]
f_z, f_θ	ordinary frequencies in the vertical and rotational directions, respectively [Hz]
ω_z, ω_θ	circular frequencies in the vertical and rotational directions, respectively [rad/s]
ζ	damping ratio

Abbreviations

The following abbreviations are used in this manuscript:

Sym	Symmetric mode
Asym	Asymmetric mode
Lat	Lateral mode shape
Long	Longitudinal mode shape
Ver	Vertical mode shape
Tor	Torsional mode shape

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