Chapter 5

Bridge case studies

5.1 Objectives and scope

This chapter addresses the analysis of VIV effects on the deck of the following bridges: Niterói (Brazil), Volgograd (Russia) and Trans-Tokyo Bay (Japan). These case studies, together with the Alconétar arch (cf. Chapter 2) offer real data that allow the validation of numerical simulations carried out. An equivalence can be established between complex cross sections and basic geometries analysed in Chapter 3, which leads to design criteria in order to diminish the sensibility to VIV.

Strategies traditionally used to minimise vibrations are shown. The placement of damping devices on bridge decks tends to be the most common, although at times it is sufficient to make small modifications in the cross section geometry to modify the vortex shedding regime as the excitation source. Other bridge elements can significantly influence its sensitivity to VIV, as the representative cross section geometry is changed, thus altering the wind structure interaction process.

Lastly, VIV effects can occur not only for the bridge in service, but also during construction, as happened in Alconétar. It is necessary, therefore, to take into consideration the dynamic structural properties and cross section geometries for the different construction phases of the bridge, to avoid any possible configuration that might lead to have VIV effects, as well as some other aerodynamic problems.

5.2 VIV on actual bridges

This section describes some of the most VIV representative cases that occurred on actual bridges in the previous years apart from Alconétar, which provide a basis for an overall analysis of the problem: Niterói, Volgograd and Trans-Tokyo Bay Bridge.
Wind instability phenomena like flutter are very rare to happen on continuous steel girder bridges, due to the fact that these bridge types have a higher stiffness and also higher natural vibration frequencies. However, they are not safe from VIV effects, associated with low regular wind velocities acting during a certain time period, typical from vast flat surfaces such as bays, estuaries or extensive rivers with no big obstacles that could introduce great disturbances in the wind regime. Continuous girder bridge typology is suggested as the best alternative in current projects for the crossing of large rivers and onshore areas to cover the overall bridge length, or as the structural solution for the approach spans of cable-stayed or suspension bridges.

The three cases presented correspond to deck vertical oscillations for VIV effects. However, as noted in Chapter 1, it should not be forgotten that other structural elements also are prone to this type of phenomenon, such as towers of cable-stayed or suspension bridges, as well as arch bridge hangers. Fortunately, vibration effects did not have serious consequences in the presented case studies, but they produced social alarm and media attention that has brought about a more exhaustive surveillance and minimisation of said oscillations through the addition of damping devices to avoid long-term fatigue problems.

Numerical simulations are used to perform problem analysis, in the same way as done for the Alconétar Arch (cf. Chapter 2). The objective is to identify the interaction mechanism in complex sections and compare it with basic section geometries studied in Chapter 3, in order to give some design rules and avoid the resonance phenomena caused by dynamic wind actions. Numerical model validation is accomplished from the actual vibration episodes registered in every case study.

5.2.1 The Niterói Bridge

This bridge crosses the Guanabara Bay (Brazil) from East to West, and connects the cities of Rio de Janeiro and Niterói. It has an overall length of 13,290 m. Whilst most of the bridge is constructed in prestressed concrete, the study concentrates on the central 848-m-long steel structure. It has a 300 m-long main span, two side spans of 200 m, and two other link spans that connects with the concrete deck at the midspan (74 + 200 + 300 + 200 + 74). Fig. 5.1 shows a general view of Niterói Bridge and its basic geometric description. The deck runs at an average elevation of 65 m above water level. It has a very high slenderness of close to 1:45 depth-to-span ratio at midspan. The existence of the Santos Dumont Airport in the vicinity did not allow the adoption of other bridge types such as cable-stayed or arch configurations.

The cross section is made up of two steel boxes joined by an orthotropic steel deck slab. It has a total width of 25.90 m and a central barrier that separates the two traffic directions. The section depth is 12.42 m over the piers and it varies linearly along the first 45 m of the deck to a depth of 7.42 m. This value remains constant along the remaining central length. The side traffic barriers increase the cross section depth to 13.27 m and 8.97 m, respectively.

The bridge was opened to traffic in 1974. The first major episode of VIV took place
Chapter 5. Bridge case studies  Vortex-induced vibrations on bridges

Figure 5.1: General view of Niterói Bridge in Brazil (top, left) and basic geometry description of the structure (Battista & Pfeil [14]).

during a storm in August 1980. Subsequently, other similar oscillation episodes highlighted the need to act against this aerodynamic problem to ensure long-term serviceability of the structure. Fig. 5.2 shows a snapshot from a video recorded by the surveillance cameras of deck vibrations due to vortex shedding in October 1997.

Figure 5.2: Map showing location of the Niterói Bridge within the Guanabara Bay, Brazil (left) and the vibration episode that occurred on October, 1997 (right).
5.2.1.1 Two-dimensional modelling

To perform numerical simulations of the bridge response, it is necessary to define the cross section geometry and to determine the representative vibration mode shapes and their associated modal masses and damping ratios. In [14] the main parameters of the bridge are given. These are summarised in Table 5.1.

<table>
<thead>
<tr>
<th>Cross section geometry</th>
<th>Bare</th>
<th>With traffic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck width ((W))</td>
<td>25.90m</td>
<td>25.90m</td>
</tr>
<tr>
<td>Deck depth on piers ((D_p))</td>
<td>13.92m</td>
<td>16.37m</td>
</tr>
<tr>
<td>Deck depth at midspan ((300\text{m})) ((D))</td>
<td>8.97m</td>
<td>11.42m</td>
</tr>
<tr>
<td>(W/D) ratio</td>
<td>2.89</td>
<td>2.27</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Dynamic properties</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1st bending frequency, vertical ((f_{n,1}))</td>
<td>0.32Hz</td>
<td></td>
</tr>
<tr>
<td>2nd bending frequency, vertical ((f_{n,2}))</td>
<td>0.55Hz</td>
<td></td>
</tr>
<tr>
<td>Total mass ((848\text{ m steel girder})) ((M_T))</td>
<td>13,100t</td>
<td></td>
</tr>
<tr>
<td>Modal total mass ((300\text{ m main span})) for the 1st mode ((M_1))</td>
<td>2,182t</td>
<td></td>
</tr>
<tr>
<td>Effective sectional mass for the 1st mode ((M_{1,2D}))</td>
<td>11,366kg</td>
<td></td>
</tr>
<tr>
<td>Damping ratio ((\xi))</td>
<td>0.01</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.1: Main geometry features and dynamic properties of Nierói Bridge.

\(M_{1,2D}\) is calculated from Eq. (2.6), as shown in §2.3.1.2. Here, Fig. 2.14 is replaced by Figs. 5.3 and 5.4.

\[
\dot{w}_j + 2\xi f_{L0} \ddot{w}_j + f_{L0} w_j = \phi_j^T f_{L0}(t),
\]

Figure 5.3: General view of the bridge deck practically without traffic (left). 150m-half-span bridge deck model (right) and horizontal wind forces acting \((U_\infty, \text{ free stream wind velocity}). The lift force vector is affected by the \(j\)th mode shape \((\phi_j^T f_{L0}(t))\), which converts the problem into an equivalent sectional 1DOF oscillator.

Neither lateral bending nor torsional modes are considered here because they are not expected to be excited by VIV due to high stiffness and frequency. The equation of motion (cf. §1.2.2) represents a 1DOF model defined by the vertical
Chapter 5. Bridge case studies  Vortex-induced vibrations on bridges

Chapter 2

Equations

2.1 Equations

\[ f(t) = f_L, \quad \sin(\omega t) = \sqrt{2}B_U f_L, \quad \sin(2\pi f_st) \]

\[(2.1)\]

where \( f_s \) is the vortex shedding frequency, which depends on the Strouhal number, the equation of motion can be extended to the total length of the bridge \( L \):

\[
\sum_{j=1}^{N} T_j M_j d x \ddot{w}_j + \sum_{j=1}^{N} \left( 2 \beta_j \right) T_j M_j d x \dot{w}_j + \sum_{j=1}^{N} \beta_j T_j M_j d x w_j = f_L, \quad \sum_{j=1}^{N} T_j f_L d x \]

\[(2.2)\]

the previous expression can be divided by the integral

\[
\sum_{j=1}^{N} T_j M_j d x \ddot{w}_j + \left( 2 \beta_j \right) \sum_{j=1}^{N} T_j M_j d x \dot{w}_j + \beta_j \sum_{j=1}^{N} T_j M_j d x w_j = f_L, \quad \sum_{j=1}^{N} T_j f_L d x \]

\[(2.3)\]

where \( R_L T_j M_j d x \) is the 2D generalized mass associated to the \( j \)th mode shape. The simplified expression for the two-dimensional case then is

\[
M_j,2D \ddot{w}_j + \left( 2 \beta_j \right) M_j,2D \dot{w}_j + \beta_j M_j,2D w_j = f_L, \quad \sum_{j=1}^{N} T_j f_L d x \]

\[(2.5)\]

Figure 5.4: General view of the bridge deck practically with traffic (left). 150m-half-span bridge deck model (right) with traffic and horizontal wind forces acting (\( U_\infty \), free stream wind velocity). The lift force vector is affected by the \( j \)th mode shape \((\phi_j^T f_L(t))\), which converts the problem into an equivalent sectional 1DOF oscillator.

Figure 5.5: Two main vertical vibration modes of the structure, taken from a FEM model.

displacement of the cross section \( y \), which is excited by the lift forces generated as a result of wind structure interaction. Here, the 300-m central span steel girder of the bridge was considered, and the first vertical bending mode at a frequency of 0.32 Hz used to compute the equivalent 2D model as described above. Fig. 5.5 shows the two main vertical vibration modes of the bridge.
### Chapter 5. Bridge case studies Vortex-induced vibrations on bridges

<table>
<thead>
<tr>
<th>Model type</th>
<th>Static parameters</th>
<th>Dynamic parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$S_c$</td>
<td>$S_t$</td>
</tr>
<tr>
<td>2D bare c.s. (equiv. to midspan c.s. of pseudo-3D bare deck)</td>
<td>14.20</td>
<td>0.156</td>
</tr>
<tr>
<td>2D c.s. with traffic (equiv. to midspan c.s. of pseudo-3D deck with traffic)</td>
<td>14.20</td>
<td>0.069</td>
</tr>
</tbody>
</table>

Table 5.2: Summary of simulation results of Niterói Bridge.

There is also a second bending mode at a frequency of 0.55 Hz, but it needs much higher wind velocities to be excited by vortex shedding. The corresponding mode-generalised mass is 11,367 kg/m. It has been determined from a finite element model, which assumed the variation of the sectional mass along the whole bridge deck length. The section geometry at midspan, shown in Fig. 5.6, is used as it corresponds to most of bridge deck length and hence is the most realistic approximation. Furthermore, in order to study the potential influence that traffic located on the bridge deck has on the wind structure interaction, four additional rectangles are added to represent the vehicles. In the context of a 2D model this obviously assumes a continuous line of such traffic. The inability to use different cross sections in the same two-dimensional model represents a clear limitation compared to pseudo-three-dimensional models, where there is the option to assign a different geometry to each slice. The structural mass was kept constant, thus disregarding mass contributions of vehicles.

Firstly, simulations with a static cross section are performed. Fig. 5.6 shows corresponding lift coefficient time histories. The full-scale time simulated is 200 seconds, in line with the indications given by Battista & Pfeil [14] on wind tunnel tests performed accordingly. In order to perform an effective comparison between the two types of numerical simulations all parameters are nondimensionalised by using $B = 25.90$ m and $D = 8.97$ m. The bare cross section shows a smoother sinusoidal lift coefficient time history than the cross section with traffic, with a
$C_L$ mean value of 0.55. The root mean square lift coefficient amplitude $C_{L_{rms}}$ is found as 0.30. The mean value of lift coefficient for the cross section with traffic is -0.08, having a $C_{L_{rms}}$ of 0.41. Static parameters are displayed in Table 5.2. Time-averaged surface pressure distributions from static simulations are shown in Fig. 5.7. The more bluff section with traffic experiences significantly higher pressures, specifically in terms of the suction occurring at both the top and the bottom surfaces. The frequency content of the lift coefficient time histories of static simulations determined through a spectral analysis is shown in Fig. 5.8 for the bare cross section, and in Fig. 5.9 for the cross section with traffic. The chart corresponding to the bare cross section shows two close peaks with a similar spectral density, the highest having a Strouhal number of 0.178, which corresponds to a critical wind velocity, $U_{\infty,\text{crit}} = 16.1 \text{ m/s}$, and the smaller, 0.155, with $U_{\infty,\text{crit}}$
Coupled dynamic simulations are then carried out by using the 1DOF model with a damping ratio of 0.01. The wind velocity varies in a range within 10 m/s to 30 m/s for the bare cross section, and from 10 m/s to 60 m/s for the cross section with traffic. The critical wind velocity, $U_{\infty,\text{crit}}$, which corresponds to the highest vertical displacements, matches well with vortex shedding frequency peaks in the case of the bare cross section (Fig. 5.8, left), having a maximum of $y_{\text{rms}} = 0.48$ m at a wind velocity of $U_{\infty,\text{crit}} = 17.0$ m/s, and a $y_{\text{peak}} = 0.86$ m at a wind velocity of $U_{\infty,\text{crit}} = 17.5$ m/s. Also, the lock-in range is delimited around the natural frequency $f_n$, as seen in Fig. 5.8, right, where each single point stands for a different dynamic case with a defined wind velocity. This is not the case for the cross section with traffic, where there is a set of points that indicates a certain frequency coupling, but the other adjacent values do not show a linear dependency between vortex shedding frequency and wind velocity (Fig. 5.9, right). Critical wind velocities of dynamic analyses differ greatly from the peaks determined from the static analyses. The curve of $y_{\text{rms}}$ (Fig. 5.9, left) has a peak of 0.45 m at $U_{\infty,\text{crit}} = 19.0$ m/s. The curve of peak amplitudes shows a relative maximum of $y_{\text{peak}} = 0.83$ m at $U_{\infty,\text{crit}} = 20.0$ m/s, although this is not a clear predominant value.

The differences can be attributed to the lock-in process. Dynamic parameters of critical wind velocities and amplitudes can be found in Table 5.2 too. The critical wind velocity determined for the actual structure is about 17 m/s [14], which ties in well with the results obtained here. The amplitudes of oscillation determined
Figure 5.8: Instantaneous vortex pattern of dynamic simulation for original Niterói bare cross section (top). Comparison of normalised PSD of lift coefficient time history, $S'(C_L)$, of two-dimensional static simulation with peak ($y_{peak}$) and root mean square ($y_{rms}$) vertical vibration amplitudes obtained from dynamic simulations (bottom, left). Identification of the lock-in range (cf. Fig. 1.12) from two-dimensional dynamic simulations (bottom, right).
Figure 5.9: Instantaneous vortex pattern of dynamic simulation for original Niterói cross section with traffic (top). Comparison of normalised PSD of lift coefficient time history, $S'(C_L)$, of two-dimensional static simulation with peak ($y_{peak}$) and root mean square ($y_{rms}$) vertical vibration amplitudes obtained from dynamic simulations (bottom, left). Identification of the lock-in range (cf. Fig. 1.12) from two-dimensional dynamic simulations (bottom, right).
through the two-dimensional simulations are relatively high. Fig. 5.10 displays the time histories of vertical vibrations of the two cases at their respective critical wind speeds. On real VIV episodes of Niterói Bridge the registered values of vertical oscillations were around 0.25±0.05 m, according to [14]. The reason for the discrepancy may well be in a damping ratio higher than 1% in the real structure. Whilst for rectangular cylinders relatively distinct shedding mechanisms can be identified, as seen in Chapter 3, for complex bridge cross sections a number of localised physical processes exist, which also interact. Some bridge details such as barriers (central and lateral), edge details or twin box interaction alter the pure vortex shedding mechanisms and give rise to secondary shedding processes. Here, this is clearly associated with intermittent detachment and reattachment of the boundary layer over the sectional surface and the effect of the gap between the boxes at the bottom side. In the case of a dynamic test, where aerodynamics and
structural dynamics interact in a VIV, the vibration filters the vortex shedding process and the main vortex shedding frequency is mainly associated with the structural frequency. Fig. 5.11 shows instantaneous velocity fields of the two cases. Also shown are streaks identifying the vortex particles used for discretisation. Fig. 5.12 shows corresponding long-term time-averaged velocity fields. A singular vortex structure or recirculation bubble can be identified between the two boxes. Due to its greater depth, the cross section with traffic is characterised by a wider wake, pointing to a significantly higher drag. Fig. 5.13 displays long-term time standard deviations, i.e. fluctuations of the velocity fields. Similarly to the previously averaged fields, a wider wake is associated with the cross section with traffic, which introduces higher fluctuations downstream. Further studies of wind-vehicle interaction on the upper surface of the cross section would cover a close-up view of the influence
Figure 5.12: Time-averaged velocity field of Niterói bare cross section (top) and cross section with traffic (bottom) from static two-dimensional simulations, $U_\infty = 15.0 \text{ m/s}$. 
Figure 5.13: Time standard deviation of velocity field of Niterói bare cross section (top) and cross section with traffic (bottom) from static two-dimensional simulations, $U_\infty = 15.0$ m/s.
of traffic on possible dynamic effects, apart from VIV problems.
Results presented here are published on [130] by Morgenthal, Sánchez Corriols & Bendig. These can be compared with two-dimensional numerical simulations independently performed by Hallak et al[50].

5.2.1.2 Fully coupled pseudo-three-dimensional modelling

For a more realistic modelling of the VIV excitation, the main span was then modelled using the new pseudo-3D method. This allows for a variable cross section geometry and several modes to be included. Here, the first two vertical modes were modelled. To simplify the model, symmetry conditions were exploited and only one half of the central span, i.e. 150 m, was represented. The total length is then divided into 15 slices of 10 m length each. It is important to note that full vortex shedding correlation is effectively imposed for the length of a slice. Any longer correlation can only come about through the structural coupling, which indeed is a physical process of the lock-in phenomenon. In other words, a slice that is too long may underestimate correlation because of neglecting along-bridge wind structure coupling and hence, relatively shorter slices are required. Two scenarios are tested again: the bare cross section and the deck partially occupied by vehicles (10-m-long stretches, same as the chosen slice length). The latter covers a more realistic representation of the traffic influence as it reproduces a partial distribution of vehicles along the bridge deck. Fig. 5.14 shows a schematic model of the fully elastic pseudo-three-dimensional model with vehicles tested. A number of dynamic simulations have been conducted using different wind velocities to identify the critical wind velocity that produces the highest vibration amplitude at midspan. The average particle count per slice is between 200,000 and 220,000 for cross sections with traffic. Bare cross section simulations feature about 140,000 to 160,000 particles per slice. The frequencies of the two modes are 0.32 Hz and 0.55 Hz. Modal masses for the 150 m half-span model are $M_{1,3D} = 1.09 \cdot 10^6$ kg and $M_{2,3D} = 1.10 \cdot 10^6$ kg, respectively.

The critical wind velocity is 17.0 m/s for both fully coupled pseudo-three-dimensional models. Fig. 5.15 shows the time history of oscillation amplitudes for both models at this wind speed. The peak values are 0.58 m and 0.41 m, and the root mean square amplitudes are 0.42 m and 0.33 m, respectively. Fig. 5.16 shows a comparison between the four models tested and also includes results of root mean square amplitudes determined from section model wind tunnel tests performed by [14] with a damping ratio of 0.01. Also included is the amplitude of full-scale observations of a typical real VIV episode on the Niterói Bridge, with the quoted confidence interval. Table 5.2 summarises the main parameters of the two pseudo-three-dimensional simulations performed.
The results need to be seen in the context of correlation considerations. It is apparent that the traditional 2D simulations overestimate the amplitudes as they effectively model a fully correlated vortex shedding process. In contrast, the pseudo-3D model predicts oscillation amplitudes closer to the full-scale observations of 0.25±0.05 m and exhibits a lower bandwidth around the critical wind speed, more
in line with experience. Further, the two-dimensional simulation with traffic provides a higher critical wind velocity of 19.0 m/s, being less representative than the others where the effect of traffic on the wind structure interaction is accounted for more accurately by the corresponding fully coupled pseudo-three-dimensional model.

A comparison with the full-scale observations is difficult, as no indication relative to the degree of traffic occupancy is given. However, good agreement in general is found through the pseudo-3D model, which is able to reproduce the amplitude-reducing effect of traffic correlation and the varying depth of the deck. Since the pseudo-3D model basically features aerodynamically uncorrelated slices, the model obviously is able to reproduce lock-in originating from coupling through structural oscillations. In fact, the degree of correlation seems quite high compared to values given by Ruscheweyh [143] for chimneys. The fact that only one half of the structure has been modelled also remarks the influence of correlation due to the shorter length.

In general, the numerical model is able to predict the phenomena qualitatively in terms of their tendencies and achieves a reasonable good agreement with the quantitative data available. An additional fully coupled pseudo-three-dimensional 8-slice model with an assigned slice length of 18.75 m has also been simulated.
in order to further simplify the analysis and reduce the computational cost. Results obtained were also satisfactory. Fig. 5.17 displays an instantaneous velocity field of the 8-slice model showing all slices. Fig. 5.18 displays a corresponding vortex-streak visualisation.

5.2.2 The Volgograd Bridge

On May 20th, 2010, the new bridge over the Volga River in the city of Volgograd was closed to traffic because it started to oscillate dangerously due to the wind action. Fig. 5.19 presents an upper overall perspective of the bridge. According to meteorological data, average wind velocities between 11.6 m/s and 15.6 m/s were recorded with a deviation of 7 degrees from the normal direction to the longitudinal
Chapter 5. Bridge case studies

Vortex-induced vibrations on bridges

Figure 5.16: Comparison of root mean square vertical amplitudes of all the models tested of Niterói: (○) two-dimensional dynamic simulation of bare cross section, (●) two-dimensional dynamic simulation of cross section with traffic, (□) pseudo-three dimensional dynamic simulation of bare cross section, (■) pseudo-three dimensional dynamic simulation of cross section with traffic, (△) sectional wind tunnel test and (−−−−−−) full scale observations with confidence interval according to Battista & Pfeil [14].

bridge axis. Fig. 5.20 shows a map of bridge location with main wind direction. There were also other facts that led to the hypothesis of VIV occurrence:

- Relatively low average wind velocities and a regular regime due to the terrain conditions, with practically no turbulence. The large river width helped to direct the windflow along a smooth surface formed by the water sheet.

- The vortex shedding frequency was in the range of the bridge fundamental vertical frequency, which led to a frequency coupling.

- Vibration amplitudes were limited, not exceeding 1.0 m, according to the observations made by people who witnessed the event. The image recordings
were used to estimate the vibration amplitudes more accurately around 65-70 cm.

Fig. 5.21 shows a snapshot of the vibration episode with the amplitude estimations for three central bridge spans.

A special commission was created to examine the possible structural damage that would have seriously affected the overall bridge safety. It reopened to pedestrians and traffic five days later. However, heavyweight traffic was not allowed until August 25th, 2010, once the bridge was examined in detail and no external or internal damage was found. After such phenomenon, a monitoring system was installed along the entire bridge to register possible movements, and a weather station was implemented for continuous observance of weather conditions in the area.

Wind tunnel tests on sectional models were performed in the Central Aero-Hydrodynamics Institute (TsAGI), to investigate the problem in detail. Within the last years, a number of cable-stayed and steel deck girder bridges crossing very wide rivers are
The Volgograd Bridge, inaugurated in October 2009, is the only structure that crosses the Volga in the region, in addition to the existing dam on the Volga Hydroelectric Power Station near the city. It has a total length of 7,110 m and seven spans. Steel cross section is composed by a trapezoidal box and a deck slab with an overall width of 17.56 m, including lateral cantilevers. It has two traffic lanes separated by a median, and a narrower line for pedestrians. There are two guardrails on both girder sides and another one that separates vehicles from the pedestrian area. The cross section has an inclination of 2%. The deck has a constant depth and is supported by concrete piers founded on the river bed, as seen in Fig. 5.22. Initially, it was planned the construction of two twin girder decks, although finally it was decided to erect only one deck and leave the other...
for a later phase. The construction of the second deck could have helped to avoid the problems associated with vortex shedding.

\section*{5.2.2.1 Two-dimensional modelling}

The first numerical simulations of the Volgograd Bridge for result validation were performed by the author in 2012 \cite{147}. Updated numerical models that include some improvements are presented below. For implementation of two-dimensional numerical simulations only the 155-m-length bridge main span in considered. The
Figure 5.21: Snapshot of the vibration episode of the Volgograd Bridge, occurred on May, 2010 (picture courtesy of V. Kruglov & S. Mozalev).

Figure 5.22: Basic geometry description of the structure (left and top, right) and detailed view of the bridge deck down from the river (bottom, right) (data provided by V. Kruglov & S. Mozalev).
wind velocity acts perpendicular to the longitudinal bridge axis\(^1\), with a horizontal angle of incidence with respect to the bridge cross section. Table 5.3 sums up the main geometry features and the dynamic properties of the bridge.

<table>
<thead>
<tr>
<th>Cross section geometry</th>
<th>Without barriers</th>
<th>With barriers</th>
<th>Twin config. (up- &amp; downwind)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck width ((W))</td>
<td>17.25m</td>
<td>17.25</td>
<td>35.98m</td>
</tr>
<tr>
<td>Deck depth ((D))</td>
<td>3.61m</td>
<td>4.82</td>
<td>4.82m</td>
</tr>
<tr>
<td>(W/D) ratio</td>
<td>4.78</td>
<td>3.58</td>
<td>7.46</td>
</tr>
</tbody>
</table>

Dynamic properties

\(1\)st bending frequency, vertical \((f_{n,1})\) \quad 0.41Hz  \\
\(2\)nd bending frequency, vertical \((f_{n,2})\) \quad 0.57Hz  \\
\(3\)rd bending frequency, vertical \((f_{n,3})\) \quad 0.68Hz  \\
Total mass \((1,211m, 10\text{-}span steel girder) \((M_T)\) \quad 13,974t  \\
Modal total mass \((155m \text{ main span})\) for the \(1\)st mode \((M_1)\) \quad 315t  \\
Effective sectional mass for the \(1\)st mode \((M_{1,2D})\) \quad 3,191kg  \\
Damping ratio \((\xi)\) \quad 0.0056

Table 5.3: Main geometry features and dynamic properties of Volgograd Bridge.

The damping factor is 0.0056 (0.56%). Its first three vertical vibration modes are considered. The fundamental vertical mode is a sinusoidal function with a peak amplitude at midspan. This allows to define a 1DOF model with the corresponding mass properties, stiffness and frequency for the dynamic simulations, included in Table 5.3.

Three cross section geometries are considered: cross section without barriers, cross section with barriers and twin cross sections with barriers. The last configuration leads to two different cases. On one hand, the static and dynamic parameters of the upwind cross section are evaluated, considering the downwind cross section as a simple static independent obstacle, without including the aerodynamic forces acting on it and identifying the 1DOF model with the geometry of the simple upwind cross section only. In the same way, the other case considers the downwind cross section only for the 1DOF model, being the upwind cross section as a simple static object that influences the interaction mechanism, but does not count for the evaluation of the aerodynamic forces. The aim is to analyse the influence of the barriers and windshields in the original section, as well as the presence of a second additional cross section and its influence in the wind structure interaction process, without contributing to the oscillations of the other section.

First of all, static simulations are performed for the determination of the aerodynamic coefficients. Table 5.4 sums up the static parameters for the four cases.

\(^1\)According to studies of the wind regime, there is a variation range of 7 degrees with respect to the direction perpendicular to the bridge axis.
Chapter 5. Bridge case studies  Vortex-induced vibrations on bridges

<table>
<thead>
<tr>
<th>Model type</th>
<th>$S_c$</th>
<th>$S_t$</th>
<th>$C_D$</th>
<th>$C_L$</th>
<th>$C_M$</th>
<th>$C_{L,rms}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2D c.s. without bars</td>
<td>13.78</td>
<td>0.117</td>
<td>1.12</td>
<td>0.87</td>
<td>0.21</td>
<td>0.09</td>
</tr>
<tr>
<td>2D c.s. with bars</td>
<td>13.78</td>
<td>0.110</td>
<td>1.47</td>
<td>0.66</td>
<td>0.16</td>
<td>0.17</td>
</tr>
<tr>
<td>2D twin (upwind)</td>
<td>13.78</td>
<td>0.057/0.085</td>
<td>1.46</td>
<td>0.24</td>
<td>0.14</td>
<td>0.11</td>
</tr>
<tr>
<td>2D twin (downwind)</td>
<td>13.78</td>
<td>0.059/0.070</td>
<td>-0.14</td>
<td>-0.27</td>
<td>0.01</td>
<td>0.26</td>
</tr>
</tbody>
</table>

### Dynamic parameters\(^2\)

<table>
<thead>
<tr>
<th>Model type</th>
<th>$Re$</th>
<th>$U_{\infty, crit}$ (peak/rms)</th>
<th>$y_{peak}$</th>
<th>$y_{rms}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2D c.s. without bars</td>
<td>$\sim3.3\cdot10^6$</td>
<td>-/-</td>
<td>0.23</td>
<td>0.03</td>
</tr>
<tr>
<td>2D c.s. with barriers</td>
<td>$\sim3.3\cdot10^6$</td>
<td>13.6/13.4</td>
<td>0.52</td>
<td>0.41</td>
</tr>
<tr>
<td>2D twin (upwind)</td>
<td>$\sim6.0\cdot10^6$</td>
<td>-/-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2D twin (downwind)</td>
<td>$\sim6.0\cdot10^6$</td>
<td>-22.0</td>
<td>-</td>
<td>0.34</td>
</tr>
</tbody>
</table>

Table 5.4: Summary of two-dimensional simulation results of Volgograd Bridge.

All the coefficients are nondimensionalised with $B = 17.25$ m and $D = 3.61$ m, i.e. the representative dimensions of the bare cross section (without barriers and windshields). Fig. 5.23 shows the time history of the lift coefficients, $C_L$, for the different cases. The mean lift coefficients are similar for the simple cross section cases, existing a notable difference with respect to the twin sections displayed in the bottom chart, where the partial lift coefficients corresponding to each section are presented, as a result of the integration of the acting aerodynamic forces around the section contour. The root mean square lift coefficient, $C_{L,rms}$, corresponding to the simple section with barriers is 0.17, double that of the section without barriers, 0.09. Such coefficient is directly proportional to the oscillation amplitudes associated to the VIV effects. In the case of the twin section configuration, the upwind section has a $C_{L,rms}$ of 0.11, and the downwind section, 0.26. The overall lift coefficient of two sections is very similar to the downwind section.

Fig. 5.24 shows the average pressure distribution calculated for the time interval considered in the static simulations. In the case of the simple sections, there are suctions in the upper and lower part resulting from the interaction that balance each other mutually. The barriers introduce a positive effect in this case, as they tend to regularise the distribution of pressures around the section contour. In the case of the twin section, the upwind section acts as a main obstacle with higher suctions acting over it, in a similar way to what occurs over the single sections. The downwind section remains to a certain extent protected from the wind actions by the upwind cross section, not having suction pressures on its surface.

\(^2\)The Reynolds number refers to the critical velocity and the other parameters are calculated with $W_{ref} = 17.25$ m and $D_{ref} = 3.61$ m as the reference cross section dimensions.
Figure 5.23: Lift coefficient time-history, $(C_L)$, of two-dimensional static simulations of the Volgograd Bridge: single cross section without barriers (top), single cross section with barriers (middle), twin cross sections (bottom): (—) upwind cross section with barriers (partial $C_L$), (—) downwind cross section with barriers (partial $C_L$) and (—) $C_L$ of overall twin sections.
Fig. 5.24: Time-averaged surface pressure distribution of Volgograd Bridge: cross section without barriers (top), actual cross section with barriers (middle) and configuration with the addition of a twin section downwind (bottom). $C_p$ scaling indicated by arrow label on top.

Fig. 5.25 summarises the dynamic simulations for each one of the cases considered through the peak amplitude curves, $y_{peak}$, and root mean square amplitudes, $y_{rms}$, as a function of wind velocities. The frequency content of the static representative simulations are also included, which are characterised by the existence of multiple peaks that identify various shedding frequencies, given the complex section geometry of study. From the analysis of results it is concluded that only the single section with barriers (top right) offers a clear identification of VIV in a wind velocity range around 13.5 m/s, which moreover is coincident with one of the frequency peak lift force spectrum obtained from the static simulation. For the downwind section (double configuration, bottom left) there are two close peaks in the $y_{rms}$ curve, which indicate certain resonance effects for a wider wind velocity range, although of less consistency than in the case of the single section. In the other two cases, in spite of the existence of determined frequency peaks that seem to predict the presence of VIV, resonant effects in the dynamic simulations have not been detected.

The VIV effects are characterised by the maintenance of a certain regularity of oscillations during a specific period of time. Therefore, the root mean square values of the vibration amplitudes tend to be used as a more accurate indicator than the peak values. These last ones are also of use because they establish an upper threshold of the maximum amplitudes registered by action of the dynamic wind forces. The VIV effects are more evident if the $y_{peak}$ curve also reflects a differentiated peak for the downwind section, as is the case with the section of Volgograd
Chapter 5. Bridge case studies

Vortex-induced vibrations on bridges

Figure 5.25: Comparison of normalised PSD of lift coefficient time history, $S'(C_L)$, of two-dimensional static simulation with peak ($y_{peak}$) and root mean square ($y_{rms}$) vertical vibration amplitudes obtained from dynamic simulations of Volgograd study cases: cross section without barriers (top, left), cross section with barriers (top, right), upwind cross section of twin configuration (bottom, left), downwind cross section of twin configuration (bottom, right).
with barriers. Also striking is the sharp increase of peak values registered for the downwind section (double configuration), as a function of the wind velocity.

The estimation of vibration amplitudes obtained from the dynamic simulations of the simple section with barriers, $y_{rms} = 0.41$ m maintains a good correlation with the average observed values, around 0.35 m ($2y = 0.70$ m, cf. Fig. 5.21). The barriers seem to be determining in the definition of the shedding pattern that gives way to the aerodynamic forces capable of exciting the fundamental vibration mode of the structure in a certain range of wind velocities. Fig. 5.26 shows the instantaneous pattern of the velocity field of the Volgograd cross section, with the presence of a vortex structure in the downstream wake as a result of the interaction process. It is more illustrative to see the video where the evolution of the velocity field and the start of the vibrations associated with the vortex shedding can be seen.

### 5.2.2.2 Twin sections

The consideration of the double section of Volgograd as one of the case studies aims to determine if the vibration episode generated a few months after the inauguration of the bridge could have been avoided if they had constructed the two decks, as was initially expected in the project. Fig. 5.27 covers the geometric configuration of the double section, from which the simulations have been made (left), as well as an instantaneous visualisation of the vortex pattern (right), obtained from one of the dynamic simulations. In this image, one of the shedded vortex of the upwind section is about to reach the downwind section and form the vortex shedding pattern of the downwind wake, which is an ILEV type (cf. Chapter 3, the presence of a von Kármán vortex street is not identified). Fig. 5.28 shows the visualisation of the time average velocity field obtained from static simulations, in which the same wind velocity is used, $U_\infty = 13.5$ m/s, for all cases analysed. In the simple section without barriers (top) the presence of a separation bubble stands out (as consequence of the shedding of the boundary layer), which can introduce certain instabilities over the surface. The barriers help to solve this problem, creating a more uniform velocity regime (middle). In contrast, the cross section
sensitive to the VIV effects. The double section (below) is characterised by a narrower and more defined wake, which indicates a more stable vortex pattern. In the middle gap between the two sections, a recirculation bubble is formed, which contributes to the stability of said pattern. Fig. 5.29 includes the time standard deviations or fluctuations of the velocity field through an statistical analysis from simulation results. Their comparison comes to confirm the previous hypothesis, highlighting also in this case a wider vortex downstream wake characteristic of the simple section with barriers, which indicates the presence of a vortex pattern that leads to the resonant effects.

As a general conclusion, it can be stated that the vibration episode for vortex shedding registered in the Volgograd bridge would not have occurred if the two decks had been constructed together. The twin section configuration geometry does not turn out to be so prone to the resonance effects, given that its vortex shedding pattern is different to that of the simple section with barriers, the actual bridge configuration. However, in the case of VIV would be developed, it is estimated that wind velocities higher than 22.0 m/s would be required, which does not correspond to area’s wind regime.

Nevertheless, the construction of the second bridge deck would require to estimate the peak amplitude response and compare it with full scale oscillations that might appear in the Volgograd Bridge, apart from those related to VIV effects.

In line with the analysis and validation carried out with the numerical models of Niterói, additional pseudo-three-dimensional simulations were brought up. In this case, however, it is not considered strictly necessary since its deck has a constant section depth and therefore it does not require the definition of different cross section geometries. In the same way, oscillations due to the VIV effects were registered with almost not traffic over the bridge deck. It could be also of interest to study the influence of traffic on the dynamic wind response.

### 5.2.3 Trans-Tokyo Bay Bridge

The Trans-Tokyo Bay Crossing Bridge is a part of the Trans-Tokyo Bay Highway Crossing, a combination of a tunnel and multiple bridges completed in 1997 that
Figure 5.28: Time-averaged velocity field of Volgograd cross section without barriers (top), cross section with barriers (middle) and twin configuration (bottom) from static two-dimensional simulations ($U_\infty = 15.0$ m/s).
Figure 5.29: Time standard deviation of velocity field of Volgograd cross section without barriers (top), cross section with barriers (middle) and twin configuration (bottom) from static two-dimensional simulations ($U_{\infty} = 15.0$ m/s).
Chapter 5. Bridge case studies  Vortex-induced vibrations on bridges

crosses the Tokyo Bay in Japan. It includes a 10-span, 1,630-m-length continuous steel girder bridge. Fig. 5.30 displays an overview of the central part of the bridge and a location map in the upper part, with the position of the bridge longitudinal axis. Fig. 5.31 shows a longitudinal scheme of the structure, as well as a detail of its cross section.

![Diagram of the Trans-Tokyo Bay Bridge](image)

Figure 5.30: General view of Trans-Tokyo Bay Bridge, Japan (courtesy of Martinin Perth). At top right corner, bridge location within the Tokyo Bay.

The two 240-m main spans are situated on both sides of the pier P7. Its cross section is of variable depth, this being 10.5 m in the abutments and 6.0 m at the midspan.

Due to the presence of a quasi-steady wind regime in the Tokyo Bay, different wind tunnel tests were done to guarantee the bridge safety against possible aerodynamic phenomena. Extensive information is gathered about the Trans-Tokyo Bay Bridge and its aerodynamic problems on [41].

Initially, sectional models were used in correspondence with the cross section depth at the main span, \( L/2 \), which is the lowest depth, and \( L/6 \), considered as the representative of the full bridge, as the deck has a variable section depth. These models were tested with a steady wind regime (without turbulence) and an angle of incidence of \( \alpha = -3^\circ, 0.0^\circ \) and \( +3.0^\circ \).

A full bridge model was also built up based on the 10-span (1,630 m) bridge, at a scale of 1:170, in order to take into account the three-dimensional effects and compare the results with the ones achieved for sectional wind tunnel tests. An horizontal wind angle of incidence and three different turbulence intensities were considered: uniform flow (without turbulence, \( I_u = 0.0\% \)), turbulent flow with \( I_u = 4.0\% \), and turbulent flow with \( I_u = 8.0\% \).

186
Figure 5.31: Outline of bridge structure for Trans-Tokyo Bay Bridge: longitudinal view and cross section view (left), and representative cross section dimensions (Fujino & Yoshida [41]).

In the sectional tests, the VIV effects are observed from 10 m/s, reaching maximum amplitudes around 15-17 m/s, according to the actual velocities. Galloping effects were detected for velocities higher than 25 m/s, and the torsional VIV effects are associated to velocities higher than 35 m/s, as higher torsional modes result excited.

In full bridge model tests, the 10 lower vibration modes were considered, which are the ones that can be excited by wind velocities lower than the design velocity, $V_d = 67.7$ m/s. VIV effects were detected at various wind velocities, as a result of the excitation of various modes within this wind velocity range. However, oscillations due to galloping effects were not registered, probably neutralised by the variable cross section geometry of the main span. Torsional oscillations are not looked at in the considered modes. Three considerations are taken into account for the control of the VIV effects: user’s comfort as a serviceability condition, fatigue damage and structural strength limit. In Japan, the bridges are closed to traffic when velocities are above 20-25 m/s, in order to avoid the driver’s discomfort. According to the Wind Resistant Guideline for Highway Bridges (Japan Road Association), the maximum acceleration admissible for the serviceability limit state is established at 0.5 m/s$^2$ [41], which corresponds to an amplitude close to 0.10 m for the fundamental natural bending frequency, $f_1 = 0.34$ Hz (cf. Eq. (2.8)). This value represents a relatively low deflection-to-span ratio of 1:2.400. The second mode is also excited by wind velocities lower than 25 m/s. From the third mode on, excitation velocities are higher and the oscillations are registered mostly in the approach spans, according to the observations made on the full bridge model. The
Fig. 5.32: Non-dimensional amplitude of vortex-induced girder vibrations for as-built and future design (in uniform air flow without aerodynamic measures) [(Fujino & Yoshida [41])].

oscillation amplitudes can induce local stresses on the cross section that exceed the steel stress limit, which marks the second oscillation control criterion. Lastly, the fatigue consideration is evaluated according to the rule of accumulated linear damage, although in this case it has been determined that it is not critical. The widening of the bridge deck to host a total of six traffic lanes is planned, according to the evolution of the traffic demand. Initially, a time period of 20 years was estimated, which is also the time period considered for the evaluation of fatigue damage.

Fig. 5.32 shows the oscillation registers obtained in the bridge between May and June 1995, corresponding to the section at midspan of one of the main spans (240 m). The curves were obtained from full bridge model wind test, adjusted for a damping logarithmic decrement $\delta = 0.028$ (damping ratio of $\xi = 0.0044$, upper curve), characteristic of the steel bridge. The peak amplitude registers are around 0.55 m for wind velocities between 16.0 m/s and 17.0 m/s, while the tests offer a maximum value of 0.58 m for a horizontal wind angle of incidence ($\alpha = 0^\circ$), transverse to the section longitudinal axis and without turbulence. The first vibration frequency, $f_{n,1} = 0.34$ Hz, dominates the wind velocity response for a range between 13.0 m/s and 18.0 m/s, and the second, $f_{n,2} = 0.48$ Hz, from 20.0 m/s on. The prevailing winds, according to the registers made between May 1995 and January 1996, act in directions $\pm 20^\circ$, regarding the transverse direction to the bridge deck axis.

The wind tunnel tests for the complete bridge have been carried out for different turbulence intensities, whose results are referred to [41]. These values are in general some 20% lower for when a turbulent wind regime is used. Sectional test results are not included in [41], but not in Fig. 5.32. These offer peak vertical amplitude of 0.37 m for $\alpha = 0^\circ$ and without turbulence.
Cross section geometry

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Continuous bridge length (offshore section)</td>
<td>1,640m</td>
</tr>
<tr>
<td>Deck width ($W$)</td>
<td>22.90m</td>
</tr>
<tr>
<td>Deck depth on piers ($D_p$)</td>
<td>11.40m</td>
</tr>
<tr>
<td>Deck depth at midspan (240m) ($D$)</td>
<td>6.90m</td>
</tr>
<tr>
<td>$W/D$ ratio</td>
<td>3.32</td>
</tr>
</tbody>
</table>

Dynamic properties

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st bending frequency, vertical ($f_{n,1}$)</td>
<td>0.34Hz</td>
</tr>
<tr>
<td>2nd bending frequency, vertical ($f_{n,2}$)</td>
<td>0.48Hz</td>
</tr>
<tr>
<td>Total mass (1,640m, 12-span steel girder) ($M_T$)</td>
<td>36,400t</td>
</tr>
<tr>
<td>Modal total mass (240m main span) for the 1st mode ($M_1$)</td>
<td>1,020t</td>
</tr>
<tr>
<td>Modal total mass (240m main span) for the 2nd mode ($M_2$)</td>
<td>1,157t</td>
</tr>
<tr>
<td>Modal sectional mass for the 1st mode ($M_{1,2D}$)</td>
<td>6,631kg</td>
</tr>
<tr>
<td>Damping ratio ($\xi = \delta/2\pi$)</td>
<td>0.0044</td>
</tr>
</tbody>
</table>

Table 5.5: Main geometry features and dynamic properties of Trans-Tokyo Bay Bridge.

5.2.3.1 Two-dimensional modelling

On first consideration, two-dimensional numerical simulations have been made taking as representative the cross section geometry at midspan ($x = L/2$) for one of the main bridge spans (240 m), given that this is where the peak vertical amplitudes are produced. Table 5.5 shows the main geometry features, such as the dynamic properties for the numerical model performance. First, static simulations are carried out to determine the mean values of the respective force coefficients ($\bar{C}_D$, $C_L$ and $C_M$) as well as the root mean square lift coefficient, $C_{L,rms}$ (cf. Table 5.6).
Chapter 5. Bridge case studies

Vortex-induced vibrations on bridges

<table>
<thead>
<tr>
<th>Static parameters</th>
<th>Sc</th>
<th>St</th>
<th>( \bar{C}_D )</th>
<th>( \bar{C}_L )</th>
<th>( \bar{C}_M )</th>
<th>( C_{L,rms} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model type</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2D bare c.s.</td>
<td></td>
<td></td>
<td>6.16</td>
<td>0.153</td>
<td>1.02</td>
<td>0.32</td>
</tr>
<tr>
<td>(equivalent to midspan c.s.)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Dynamic parameters</th>
<th>( Re )</th>
<th>( U_{\infty,crit} ) (peak/rms)</th>
<th>( y_{peak} )</th>
<th>( y_{rms} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model type</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2D bare c.s.</td>
<td>( \sim12.0\cdot10^6 )</td>
<td>17.4/16.8</td>
<td>0.64</td>
<td>0.29</td>
</tr>
<tr>
<td>Pseudo-3D deck</td>
<td>( \sim13.8\cdot10^6 )</td>
<td>14.0/13.0</td>
<td>0.37</td>
<td>0.28</td>
</tr>
<tr>
<td>(midspan c.s., 1st mode)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 5.6: Summary of simulation results of Trans-Tokyo Bay Bridge.

The wind angle of incidence is \( \alpha = 0^\circ \) and a steady wind regime is used. A 1DOF vertical oscillation model is considered according to the fundamental frequency, \( f_{n,1} \), with a structural mass of \( M_{1,2D} = 6.631 \text{ Kg/m} \), calculated from a finite element model using the expression of §2.3.1.2. The damping factor is \( \xi = 0.0044 \). Fig. 5.33 shows the visualisation of an instantaneous vortex shedding pattern, obtained from a static numerical simulation. The PSD of the lift coefficient time history identifies a vortex shedding frequency peak for a Strouhal number of \( St = 0.153 \), associated to a critical wind velocity of 15.6 m/s (Fig. 5.33 bottom left). The dynamic simulations carried out subsequently corroborate the existence of VIV resonance phenomena within a wind velocity range close to the previous value. The peak amplitude is \( y_{peak} = 0.64 \text{ m} \) for \( U_{\infty} = 17.4 \text{ m/s} \). This value is slightly higher than 0.55 m, registered for the real bridge and 0.58 m obtained from full bridge model wind tests. The lock-in is observable on the bottom right chart that relates the vortex shedding frequencies, \( f_s \), and free stream wind velocities, \( U_{\infty} \), of various dynamic simulations.

The peak root mean square amplitude is \( y_{rms} = 0.29 \text{ m} \), associated to \( U_{\infty} = 16.8 \text{ m/s} \). The cross section geometry gives way to a wind structure interaction that produces an irregular downstream wake, which justifies the notable difference between the peak amplitudes and the root mean square amplitude. This is typical thing in most of the cases studied, where deck cross section elements, such as the lateral cantilevers, the safety barriers and and windshields contribute to generate secondary vortex structures that interfere in the main vortex shedding patterns, as occurred also in the Niterói and the Volgograd Bridge, as previously analysed. Fig. 5.34 shows an instantaneous visualisation of the wind velocity field, together with the vortex shedding pattern (above), where lower and higher wind velocity regions are identified (bluish and reddish tones, respectively). Also included is the time-averaged mean wind velocity field (middle), as well as the time-averaged
Figure 5.33: Instantaneous vortex pattern of dynamic simulation for Trans-Tokyo Bay cross section (top). Comparison of normalised PSD of lift coefficient time history, $S'(C_L)$, of two-dimensional static simulation with peak ($y_{peak}$) and root mean square ($y_{rms}$) vertical vibration amplitudes obtained from dynamic simulations (bottom, left). Identification of the lock-in range (cf. Fig. 1.12) from two-dimensional dynamic simulations (bottom, right).
Figure 5.34: Static simulation of the Trans-Tokyo Bay cross section ($U_\infty = 15.0 \text{ m/s}$): instantaneous velocity field and vortex particle streaks (top), time-averaged velocity field (middle) and time standard deviation of velocity field (bottom).
velocity fluctuations in the domain, with higher values below the cross section, which indicates a certain instability at the beginning of the downstream wake.

### 5.2.3.2 Fully coupled pseudo-three-dimensional modelling

The two-dimensional numerical simulations achieve a good approximation of the critical wind velocity range and of the peak vertical amplitudes in the estimation of the VIV effects for the Trans-Tokyo Bay Bridge. The variable depth cross section does not seem to be a key factor in this case, and the choice of the midspan cross section as representative of the bridge deck for the simulation performance seems to be adequate.

However, in the two-dimensional numerical models it is only possible to consider more than one vibration mode. Therefore, a 120-m-length pseudo-three-dimensional model is performed, which corresponds to the half of the main span. The overall length is modelled with 12 slices of 10 m each, which include 12 different cross section geometries that take into consideration the section depth variation. An elastic structural model is defined with the corresponding dynamic properties for the two fundamental bending modes, as done for the case of the Niterói Bridge (cf. §5.2.1.2). The frequencies are $f_{n,1} = 0.34$ Hz and $f_{n,2} = 0.48$ Hz, with mode shapes defined on [41] (cf. Fig. 8). The structural masses are $M_{1,3D} = 1,020$ t and $M_{1,2D} = 1,157$ t, respectively. The damping factor is $\xi = 0.0044$ for both cases. The results obtained relative to slice 12 (section at midspan) are covered in Table 5.6. Fig. 5.35 include the results of the pseudo-three-dimensional and the two-dimensional dynamic simulations for their comparison.

First, the two critical wind velocity ranges of the curves corresponding to the pseudo-three-dimensional simulations are observed, which identify the presence of resonance effects associated with each vibration mode considered. The peak amplitude for the first mode is $y_{peak,1} = 0.33$ m, for a wind velocity of $U_{\infty, crit} = 14.0$ m/s. In the case of the second mode, $y_{peak,2} = 0.35$ m for a wind velocity of $U_{\infty, crit} = 20.0$ m/s. The root mean square amplitudes, $y_{rms}$, are slightly lower than the peak amplitudes. Secondly, there is a significant difference in the estimation made by the two-dimensional simulations, where the peak amplitude was $y_{peak} = 0.63$ m, more in agreement with the real registers and the full bridge model results. The critical wind velocity range also decreases its value, being around 14.0 m/s, instead of 16.5 m/s achieved for the two-dimensional simulations, more in line with the real observations. Lastly, the critical wind velocity range is larger for the pseudo-three-dimensional simulations, produced possibly by the effect of structural coupling between the different slices in the wind structure interaction process.

The pseudo-three-dimensional simulations provide lower peak amplitudes than those estimated through the two-dimensional simulations, in addition to offering a slightly different critical wind velocity range. The variable geometry of the deck could be the most probable reason that explains this. It is necessary to implement

---

4Both modes are excited for a different wind velocity range. However, there are superimposed oscillation effects, as the two frequencies are very close.
5.2.4 Equivalent Strouhal number and lift coefficient

The aeroelastic VIV effects in three real bridge case studies have been analysed: Niterói, Volgograd and Trans-Tokyo Bay. Furthermore, in Chapter 2 the case of the Alconétar arch was studied in detail. In Chapter 3 the Strouhal number, $St$, additional pseudo-three-dimensional models that allow one to investigate this aspect in more detail and achieve a good result validation. Nonetheless, the root mean square amplitudes offer very similar estimations both for two-dimensional simulations and three-dimensional simulations, although also the critical wind velocity range is different.
and the root mean square lift coefficient, $C_{L,\text{rms}}$, were determined from static simulations of basic cross section geometries.

The objective is to relate complex cross section geometries of the case studies to an equivalent basic geometry (rectangular, H-shape section or rectangular sections in tandem arrangement), with the purpose of establishing some general design criteria that ensure a low probability of occurrence of the VIV. Some normative for the evaluation of the sensitivity of certain bridges against dynamic wind actions, such as the British design rules for aerodynamic design of bridges, BD 49/01 (Part 3) [54], consider different cross section types and define the reference dimensions for their verification. In this case, following the methodology described in Chapter 3, the evaluation of the sensitivity of the different cross sections to VIV effects are based on the analysis of the fundamental static parameters: the Strouhal number for the estimation of the critical wind velocity range, and the root mean square lift coefficient as a proportional value to the oscillation amplitude in dynamic models.

The determination of the equivalent reference dimensions are based on the study of vortex generation and progression in the wake downstream as a consequence of the wind structure interaction process. Fig. 5.36 shows the Volgograd Bridge representative cross section (cf. §5.2.2), where $D$ is the section depth, $B_1$ is the upper deck width, and $B_2$, the lower deck width. The equivalent depth, $D^*$, tends to be the overall cross section depth, including any type of windshields and safety barriers which would pose an obstacle if the solidity ratio is not very low\(^5\), as said dimension is directly related to the points of separation of the boundary layer and the size of the vortex generated. The cross section width, or rather the aspect

\(^5\)In agreement with the analysis carried out in Chapter 4, a value of $\psi > 0.20$ can be established as a threshold solidity ratio for the consideration of the safety barriers. However, more detailed studies are necessary to establish a more reliable criterion.

---

Figure 5.36: Instantaneous vortex flow pattern of the Volgograd representative cross section with reference dimensions.
ratio, $W/D$, defines the shedding pattern associated to the wake downstream, as seen in Chapter 3, which determined to a large extent the acting dynamic wind actions and their regularity. When there is a complex section of different upper and lower widths, it turns out to be sufficiently approximate to take the average value as an equivalent width, $W^*$, according to the following expression:

$$W^* = \frac{B_1 + B_2}{2}$$  \hspace{1cm} (5.1)

Table 5.7 shows all the case studies analysed so far, where different additional cross section geometry configurations are also included.
### Table 5.7: Equivalent geometry dimensions, $W^*$ and $D^*$, and Strouhal number, $St^*$, of the case studies.

<table>
<thead>
<tr>
<th>Key</th>
<th>Schematic cross section</th>
<th>$W^*$ [m]</th>
<th>$D^*$ [m]</th>
<th>$W^<em>/D^</em>$ [-]</th>
<th>$St^*$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>Alconétar original</td>
<td>7.74</td>
<td>2.41</td>
<td>3.21</td>
<td>0.122</td>
</tr>
<tr>
<td>A2</td>
<td>Alconétar with braces</td>
<td>7.74</td>
<td>2.41</td>
<td>3.21</td>
<td>0.121</td>
</tr>
<tr>
<td>A3</td>
<td>Alconétar with deflectors</td>
<td>8.67</td>
<td>3.36</td>
<td>2.58</td>
<td>0.149</td>
</tr>
<tr>
<td>N1</td>
<td>Niterói bare c.s.</td>
<td>22.95</td>
<td>8.97</td>
<td>2.56</td>
<td>0.178</td>
</tr>
<tr>
<td>N2</td>
<td>Niterói with traffic</td>
<td>22.95</td>
<td>11.42</td>
<td>2.01</td>
<td>0.069</td>
</tr>
<tr>
<td>V1</td>
<td>Volgograd without bars</td>
<td>12.36</td>
<td>3.61</td>
<td>3.42</td>
<td>0.117</td>
</tr>
<tr>
<td>V2</td>
<td>Volgograd with bars</td>
<td>12.36</td>
<td>4.82</td>
<td>2.56</td>
<td>0.110</td>
</tr>
<tr>
<td>V3</td>
<td>Volgograd twin c.s. (upwind)</td>
<td>30.57</td>
<td>4.82</td>
<td>6.34</td>
<td>0.057</td>
</tr>
<tr>
<td>V4</td>
<td>Volgograd twin c.s. (downwind)</td>
<td>30.57</td>
<td>4.82</td>
<td>6.34</td>
<td>0.059</td>
</tr>
<tr>
<td>T1</td>
<td>Trans-Tokyo Bay</td>
<td>19.95</td>
<td>6.90</td>
<td>2.90</td>
<td>0.153</td>
</tr>
</tbody>
</table>
The equivalent width \( W^* \), and the equivalent depth \( D^* \), is defined, as well as the \( W^*/D^* \) ratio. The equivalent Strouhal number, \( St^* \), is obtained from the original value defined for each case, \( St \), according to the expression 1.33, but nondimensionalised with \( D^* \) instead of \( D \). For the analysis of various cross section geometry variants, some reference dimensions tend to be considered, \( W_{ref} \) and \( D_{ref} \), to calculate the nondimensional parameters, i.e. the lift force coefficient and the Strouhal number. These dimensions correspond to the standard section, that is, those lacking safety barriers, windshields, deflectors and other additional elements.

For example, in the case of Alconéter, the Strouhal number of the variants A1, A2 and A3 are given according to \( D_{ref} = 2.41 \text{ m} \), which is the depth of the original section (A1). The equivalent depth, \( D^* \), is the same for A1 and A3, and therefore \( St^* = St \). However, the equivalent depth for the section with deflectors (A2) is \( D^* = 3.36 \text{ m} \), and therefore its equivalent Strouhal number is \( St^* = 0.208 \), different from \( St = 0.149 \). The same criterion is followed in order to define \( W^* \), \( D^* \) and \( St^* \) from the other case studies and their different variants.

Fig. 5.37 reproduces the diagram of Fig. 3.6, with the addition of the new estimated \( St^* \) according to the equivalent aspect ratio, \( W^*/D^* \), related to the case studies covered in Table 5.7. Said values are well correlated with the experimental data provided by different researchers and the results achieved from numerical simulations. Thus, the original Alconéter cross section (A1) and the section with braces (A2) are included within the values related to the rectangular sections in tandem of \( B/D = 0.5 \). The section with deflectors (A3), conversely, gets closer to the curve of the streamlined leading edge rectangular section, which turns out to be coherent. The Niterói bare cross section (N1) presents a \( St^* \) which does not seem to correspond with the characteristic values of any of the basic sections considered. However, the previous analyses of the vortex pattern from visualisations of the velocity field (cf. §5.2.1.1) allow one to identify the presence of a recirculation bubble in the middle gap between the two boxes, similar to what occurs for H-shape cross sections. The same is applied for the Niterói section with traffic (N2). In this case, \( St^* \) is similar to those given for the basic rectangular section and the H-shape section with the same aspect ratio \( W/D \). However, the presence of one recirculation bubble is also observed in the middle gap between the two squares representing the vehicles on the bridge deck, which is closer to the H-shape section interaction mechanism. The simple section without barriers of the Volgograd Bridge (V1) resembles an equivalent rectangular section, meanwhile the simple section with barriers (V2) fits more with a H-shape section geometry. Regarding the case of twin sections (V3 and V4), the appearance of recirculation bubbles between the safety barriers and in the lower middle gap put it closer to an equivalent H-shape geometry.

The Trans-Tokyo Bay Bridge representative cross section is identified as a rectangular section, and its \( St^* \) value is similar to the experimental results presented in [41]. However, it is slightly different from the one corresponding to a basic rectangular section, which leads to consider an effective width \( W^* \) equal to the average value of the overall cross section width (box girder plus lateral cantilevers) and the box girder width only (see Table 5.7).

---

\(^6\)Note the use of Eq. (5.1) in the cases of Niterói (N1, N2), Volgograd (V1, V2, V3, V4) and Trans-Tokyo Bay (T1).
Figure 5.37: Equivalent Strouhal number, $St^*$, of the case studies (original diagram from Fig. 3.6).
Table 5.8 covers the root mean square of the equivalent lift coefficient, $C_{L,\text{rms}}$, from Eq. (1.37), using the total width for each case study variant, $W$, except for the variants of Alconétar, where the effective width is $W_{\text{eff}} = 2B$, being $B$ the box width.

<table>
<thead>
<tr>
<th>Key</th>
<th>$W$ [m]</th>
<th>$D$ [m]</th>
<th>$W/D$ [-]</th>
<th>$C_{L,\text{rms}}^*$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>7.74</td>
<td>2.41</td>
<td>3.21</td>
<td>1.80</td>
</tr>
<tr>
<td>A2</td>
<td>7.74</td>
<td>2.41</td>
<td>3.21</td>
<td>0.46</td>
</tr>
<tr>
<td>A3</td>
<td>8.67</td>
<td>3.36</td>
<td>2.58</td>
<td>1.08</td>
</tr>
<tr>
<td>N1</td>
<td>25.90</td>
<td>8.97</td>
<td>2.89</td>
<td>0.30</td>
</tr>
<tr>
<td>N2</td>
<td>25.90</td>
<td>11.42</td>
<td>2.27</td>
<td>0.41</td>
</tr>
<tr>
<td>V1</td>
<td>17.25</td>
<td>3.61</td>
<td>4.78</td>
<td>0.09</td>
</tr>
<tr>
<td>V2</td>
<td>17.25</td>
<td>4.82</td>
<td>3.58</td>
<td>0.17</td>
</tr>
<tr>
<td>V3</td>
<td>35.98</td>
<td>4.82</td>
<td>7.46</td>
<td>0.22</td>
</tr>
<tr>
<td>V4</td>
<td>35.98</td>
<td>4.82</td>
<td>7.46</td>
<td>0.26</td>
</tr>
<tr>
<td>T1</td>
<td>22.90</td>
<td>6.90</td>
<td>3.32</td>
<td>0.23</td>
</tr>
</tbody>
</table>

Table 5.8: Equivalent root mean square lift coefficient, $C_{L,\text{rms}}$, of the case studies.

Fig. 5.38 reproduces the diagram of Fig. 3.12 with the addition of the $C_{L,\text{rms}}^*$ values depending on the real aspect ratio, $W/D$. It seems more evident to find an equivalence with H-shape sections for the cases of Niterói, (N1 and N2) and Volgograd (V2, V3 and V4). The Trans-Tokyo Bay (T1) geometry offers a lower value than expected, considering an equivalent rectangular section, as well as the case of the single section of the Volgograd Bridge without barriers (V1). The Alconétar original cross section (A1) corresponds to the values of two rectangular sections in tandem arrangement with $B/D = 0.5$, while its variant with bracings (A2) shows lower values due to the influence of these in the interaction mechanism. The variant with deflectors (A3) also shows lower values as indicated in §2.3.1.1, given that in this case the effective width is used for its determination ($W_{\text{eff}} = 3.40$ m), which takes into account the additional attached curved deflectors at box edges instead of $W_{\text{eff}} = 2.40$ m, used as reference section width in Chapter 2.

The methodology presented is of practical use for the evaluation of the sensitivity to VIV of generic cross section geometries equivalent to some basic cross sections. The results obtained from numerical simulations have been contrasted with experimental values offered by various researchers. Therefore, it could be taken as a good reference, even for their incorporation in the current normative related to bridge aerodynamics, as a quick and simple evaluation method.

However, it should be pointed out that the evaluation of representative lift force coefficients in the case of complex cross section geometries is not that easy. In Chapter 6 a new semi-empirical model for the estimation of VIV is presented,

---

7For A1, $W_{\text{eff}} = 2.40$ m, for A2, $W_{\text{eff}} = 3.40$ m (taking into account the width of the bracings, $b = 0.50$ m), and for A3, $W_{\text{eff}} = 3.34$ m (deflectors are taken into account).
Figure 5.38: Equivalent Strouhal number, $St^*$, of the study cases (original diagram from Fig. 3.12).
which pays special attention on the excitation forces intervening in the development of VIV phenomena.

5.3 Countermeasures

5.3.1 Introduction

The VIV effects, like the other possible aeroelastic problems, can be prevented with a good structural design that takes into consideration the representative geometry of the main structural bridge elements, such as the deck, towers, piers and cables. In the case of these latter, specific damping devices tend to be used for vibration control, due mainly to VIV and galloping effects, common in cable-stayed or suspended bridges. Cable bracings are used to group them in order to provide a higher stiffness which minimise the oscillations and avoid other additional effects, such as wake galloping (cf. Chapter 1).

There are basically two ways of preventing the VIV effects: (1) modification of the structure dynamic properties, which in most cases is carried out through the addition of damping elements, and (2) a proper cross section geometry design, which also covers the use of additional elements such as guide vanes, deflectors or flaps, to break or modify the coherence of the vortex shedding in the interaction process. Both methods are shown below through some practical examples applied on bridges, some of these analysed previously. A general vision of said methods is offered, in addition to some literature references of interest. However, no additional analysis is included by the author.

5.3.2 Mass dampers

The addition of tuned mass dampers, also known as TMD, are an effective measure for minimise VIV effects. Such damping devices consist of a mass acting as a dead weight connected to a set of dampers, which can be springs or other more sophisticated mechanisms. They are installed in different points of the structure, normally in the areas where the highest oscillation amplitudes are reached. Their mission is to counteract said oscillations through movements in opposite direction, which are induced in the mass dampers by the oscillations themselves of the structure. These are the called passive systems, given that no control is applied on their functioning. With the active systems, on the contrary, it is possible to control the vibration of the dampers so that their action turns out to be more effective. Thus, for example, the vibration frequency can be automatically adapted in case the structure becomes excited by different vibration modes, and it is even possible to regulate the damping ratio through the use of magneto-rheological dampers, like those used in the Volgograd Bridge [174].
The mass used for the design of the dampers tends to be a percentage of the structural mass, generally lower than 5%. Its manufacture and installation is costly and they are used in structures under service where it is not possible to eliminate the vibrations easily.

In recent years, there have been several cases of real bridges where they have used mass dampers to minimise the vibrations due to the dynamic effects of the wind. Larsen (1992) [95] shows the design aspects of the devices used on the Great Belt East Bridge approach spans. Beyond the VIV effects, Ostenfeld (1992) [134] suggests a new philosophy of design through the introduction of active control systems in order to control the self-excited forces in certain sections of bridges. Shum et al. (2008) [153] defend the use of a new technology in the design of the dampers through the numerous pressurised tuned liquid column dampers. In the line of use of active dampers, Kim et al. (2012) [75] present wind tunnel test results on the use of these devices as the most effective solution for super-long-span bridges. Some years before, Hansen et al. (2000) [51] carried out similar experimental tests.

5.3.2.1 Niterói

The problems of vibrations due to the VIV effects occurred in the Niterói Bridge shortly after being opened to traffic, in 1974. In 1980, the first significant vibration episode took place, which highlighted the real importance of the problem, although it was not until September 2004 when a number of multiple synchronised dynamic attenuators (MDSA) were placed inside the box girders of the bridge [15], which allow the effective passive control of the wind-induced vibrations. It was closed to traffic when excessive vibrations in the bridge were detected until the exciting wind action abated.

The interruption of the main route of communication between the cities of Rio de Janeiro and Niterói caused big traffic problems and the consequential economic losses. Hence, some actions were decided to be taken.

Fig. 5.39 shows an image of the dampers used on the bridge. Engineer R. Battista was in charge of designing the system of 8 TMDs whose mass is 1% of the structural mass and a damping ratio close to 8%. The previous feasibility studies are summarised on [14]. The devices finally installed are described on [15], which includes a description of the calibrated mathematical-numerical model for the aeroelastic problem to upgrade the serviceability of this bridge and the users’ comfort. Subsequent studies on the problem of VIV in the Niterói Bridge performed by other researchers [82] [158] suggest the installation of small wind turbines attached to both sides of the deck with the aim of altering the formation of vortex responsible for the resonant effects, at the same time as they would obtain a sufficient quantity of energy for the installation of auscultation and monitoring systems. Fig. 5.40 shows some diagrams of said proposal.
Second-order differential equations are used to determine the DOF related to one bending mode. For motion control of the modal mass system (TMD), and where apart from the variables and parameters already defined in Fig. 6, the vertical displacement of the passive absorber is described by:

\[ y(t) = x(t) + c(t) \]

where \( c(t) \) is the modal control force. Equation (11a) and (11b) or (4) related to the controlled and uncontrolled systems are then solved for \( y(t) \) relative to the structure, as indicated in Fig. 6.

Figure 5.39: Top: passive control of VIV on the Niterói Bridge: (a) Typical section with tuned-mass-dampers (TMDs), (b) substructure with TMDs (Battista & Pfeil, [14]). Bottom: TMDs installed inside the box girders (picture courtesy of F. Gonçalves).

Figure 5.40: Placement proposal of the small wind turbines on the Niterói Bridge girder, from [158].
5.3.2.2 Volgograd

Conducted wind tunnel sectional model tests confirmed that the oscillations suffered by the Volgograd Bridge were due to a problem of aerodynamic resonance, and it was decided to place three sets of dynamic dampers in its three main spans. Technical works began Summer 2011 and were completed in October. The total cost was about 112 million roubles (3.8 million US dollars), accounting for 4.5% of total budget of the bridge (2,500 million roubles, 84 million US dollars). The traffic was not interrupted during the assembly process. Fig. 5.41 shows a picture of the dampers installed within the girder of the main span. No major vibrations since the dampers were added have been detected. A similar solution was adopted in Niterói Bridge in 2004 to end the nagging vibrations that occurred from time to time. The Volgograd Bridge has thus become the first continuous deck bridge in Russia to adopt this solution. The damping system becomes an expensive alternative to reduce these effects and avoid the lock-in phenomenon also for other natural frequencies that could be excited.

Figure 5.41: Dampers installed inside the girder of the Volgograd Bridge (picture courtesy of volg-vistex.livejournal.com).

An adequate aerodynamic study of the section prior to its erection could have raised the choice of a more efficient geometry against the VIV effects. Figure 5.42 shows a visualisation of the velocity field from a numerical simulation done with an alternative cross section in which a small Savonius-type turbine is added to the lower corner. This solution is only a conceptual idea that would allow for energy production for feeding some sensors located in the bridge for inspection and auscultation purposes.
5.3.2.3 Trans-Tokyo Bay

Like previous cases, the oscillation issues of the Trans-Tokyo Bay Bridge were eliminated through the use of 8 TMD, four in the centre of each central span of 240 m of main span [41]. Fig. 5.43 shows a diagram of the dampers used. For their design, a multimode analysis of the bridge was used, where 10 main first vibration modes were considered, whose excitation was produced for lower velocities than the velocity of the project. The first vibration episode was detected in the bridge in 1995, before the finalisation of the complete work, in 1996. Wind tunnel tests were also carried out in order to predict their importance and to be able to dimensionalise the necessary damping systems. In said tests, galloping problems were also detected for higher wind velocities, but they were exceeding the velocity of the project and therefore the possibility of occurrence was ruled out. As an alternative solution to the TMDs, the use was raised of aerodynamic devices that modified the cross section geometry and, therefore, the vortex shedding causing the vibrations. This solution needed the combination of fairings, double flaps and a skirt along the central spans, as explained on [41].

The dampers were designed to control the vibrations associated with the first and second fundamental mode (vertical), which affected the main spans. To suppress the vibrations associated with the upper modes, which mainly affect the approach spans, horizontal plates were used as an effective countermeasure. The Japanese regulations are very restrictive regarding the vibrations caused by the wind, and forces the closure to traffic of the bridge highways when the 10-minute mean wind velocity exceeds 20-25 m/s, which is a safety measure prior to the countermeasures carried out on the Trans-Tokyo Bay Bridge.
5. The maximum stroke of the TMD for the first mode is 600 mm, while 800 mm for the second mode.

Fig. 16 shows the relationship between the TMD damping factor and the damping of the girder-TMD vibration system, as obtained by numerically solving the analytical bridge model including the TMDs. The frequency response curves are plotted for various values of the ratio of the natural frequency of the TMDs to the natural frequency of the girder. As is apparent, the damping factor of the complete girder-TMD system can be maximized by selecting an appropriate TMD damping factor, and this maximum is reached when the natural frequency of the girder nearly equals that of the TMDs, i.e., when the frequency ratio equals 1.0.

Detailed information on TMD is provided in Table 1.

Performance of Tuned Mass Dampers under Wind Action

The TMDs for controlling first- and second-mode vibration were brought into operation beginning in, respectively, January and late July 1996. Fig. 17 compares field observations made before and after installation of the TMDs for the first mode. In this figure, records for similar wind conditions velocity and direction before and after installation of the TMDs were taken over 10 min from 13:50 to 14:00 on November 1, 1995 and from 19:02 to 19:12 on March 11, 1996, respectively.

5.3.3 Geometry modification

An alternative method to the use of tuned mass dampers (TMD) is the modification of the cross section geometry, which also includes the addition of the guide vanes, flaps, deflectors and other aerodynamic devices whose main purpose is to modify the vortex shedding as a result of the interaction process. This second strategy focuses on the modification on the wind excitation forces that produce the problem, instead of changing the structural dynamic properties. Some typical examples of the use of aerodynamic devices can be found in Larsen & Poulin [91], Naudascher & Rockwell [133], Sun et al. [191], and Yang & Ge [191]. Two other publications regarding the use of guide vanes are [2] and [64]. The use of numerical
simulations in the study of the minimisation of VIV effects have also been studied by Larsen [94], and Shirai & Ueda [152]. An original solution to end up with the VIV problem is to break the correlation of the vortex shedding along the bridge span [34]. The same concept has been traditionally used in flexible high-rise chimneys and in bridge cables, through the placement of spanwise helical fences, also called strakes or spoilers. Passive or active control devices are proposed [55], or when flutter and VIV are considered jointly [184].

Other aeroelastic phenomena such as flutter should be taken into account. Sometimes, a typical bridge deck cross section geometry to fight against flutter can turn out to be not so advisable to minimise the VIV problems. In order to combat the effects of flutter, currently it is preferred to design multi-box section decks, with the aim of increasing the aerodynamic resistance in the interaction process, which also elevates the critical wind velocity. Some examples are included on [73] and [182], among others. The proposals for the future super-long-span bridges are also in line with this strategy [100] [182].

The choice of one or another solution depends on each specific case. The geometry optimisation should be discussed in the bridge design phase to achieve an aerodynamic shape that manages to move the vortex shedding frequency out of the range of the bridge fundamental natural frequencies. However, the cross section is usually chosen by following strength limit and economic criteria, although it is always possible to perform an streamlined section. The placement of guide spans or flaps instead of damping devices tends to be more economic because of its faster assemblage, as it does not require a detailed design of an specific TMD system. Furthermore, if the VIV effects are only relevant during the construction phase, this solution will not be adequate, as that will only turn out to be useful during a short period of time. However, the aerodynamic devices are not effective for all the geometry configurations. When having the ILEV as the vortex shedding pattern type (cf. Chapter 3), this is not affected by the presence of splitter plates [28], and the same can occur with other additional elements. Therefore, it is very important to identify the vortex shedding pattern that characterises the interaction mechanism to carry out a prior detailed analysis of the real effectiveness that can be reached on placing the specific devices. It is usually done through numerical simulations or wind tunnel tests.

Two very similar examples are shown briefly regarding the use of deflector to minimise the VIV effects: the Alconétar Bridge (Cáceres, Spain, 2006) and the Lupu Bridge (Shanghai, China, 2003).

### 5.3.3.1 Alconétar

The use of deflectors on the Alconétar Bridge was shown previously in Chapter 2, broadening here the information with further details. The identification of the aeroelastic phenomenon of vortex shedding as cause of the problem initially raised the dilemma of placing dampers in specific points of the arch structure. However, and adequate design and the damper system setting would excessively prolong the intervention in the structure, which required on the other hand faster action to solve the problem.
The design of the deflectors, managed by Prof. Astiz [8], was based on similar solutions used in other bridges [64] where VIV problem happened as well. Fig. 5.44 shows the details of the deflectors installed on the finished bridge. The steel curved plates used as deflectors were welded to the mounted arch and also to the two semi-arches before erection to avoid vibrations that happened in the first one. The installation of the deflectors was completed quickly on the mounted arch.

To verify the deflector efficiency for VIV minimisation, a number of wind tunnel of the standard Alconétar arch cross section and and several proposed configurations with deflectors [7] were carried out, with the aim of searching for an optimal solution. Different lengths and spacing between the single elements, as well as the necessary distance from the box edges or different angles of inclination were proposed in the study. According to the the wind test results, the adopted solution managed to reduce the VIV peak amplitudes to a half.

### 5.3.3.2 Lupu

The Lupu Bridge is a steel arch bridge that crosses the Huangpu River, in the city of Shanghai, China. It is a tied-arch bridge with a main span of 550 m, and two approach spans of 100 m each [102]. It was surpassed as the longest bridge arch in the world by the Chaotianmen truss arch bridge (Chongqing, China, 552 m of main span), although it continues to maintain the record of box arch bridge typology.

The main arch is composed of two arches of hollow steel box sections and concrete...
filling. Both arches have a total height of 100 m, they are inclined regarding the vertical (1:5) and are joined by parallel braces on their whole length. The cross section of each arch consists of a steel rectangular box which is chamfered in its lower part. It also has a variable cross section depth of 9 m in the abutments and 6 m at midspan. The arches are filled in with concrete after its assembly, to lend the necessary strength to the main arch.

Two temporary steel towers were erected on the main piers during bridge construction, and temporary stays were used for the erection of the arch ribs above deck level in the main bridge span. Once the arch was finished, the assembly of the deck took place, as seen in Fig. 5.45. The concrete filling of the arch was done once its assembly was completed, to facilitate the assembly work and to give the necessary stiffness. It was required to carry out a complete aerodynamic study, as it was a flexible arch because of its long span and the characteristic wind regime of the zone. The main results are summarised on [42]. Said studies consisted of statistical analysis of wind velocities at the bridge site, topographical model testing, sectional model testing for static force measurement, sectional model testing of vibration, numerical analysis of static wind-induced stability, aeroelastic full bridge model testing, combination of equivalent wind loading and probabilistic analysis of vortex shedding oscillation. Three different configurations were considered: maximum cantilever stage, completed arch ribs, and completed bridge. The results of the study concluded that the probability of occurrence of instability phenomena like flutter and galloping was very small. However, the probability of the occurrence of VIV made it to be focused on the control of their effects. Fig. 5.46 shows some of the solutions proposed for VIV effect minimisation, which are based on the addition of splitter plates or guide vanes with the aim of modifying the vortex shedding regime [43]. From all these, the most effective turns out to be the CS-8, given that the upper full cover plate modifies the vortex shedding pattern that takes place in the interaction process (cf. Chapter 3). Fig. 5.47 shows two instantaneous flow visualisations obtained from numerical simulations.
Without the application of CFD techniques, there should be laboriously experimental workload for finding effective ways to aerodynamically stabilize this bridge even with short spans.

Another successful application of CFD techniques is involved in the vortex-shedding vibration simulation taking as an example of Lupu Bridge in Shanghai, China. Lupu Bridge is a half-through arch bridge with the world record-breaking span length of 550 m. Two inclined steel arch ribs are 100 m high from the bottom to the crown, and each has the modified rectangular box section with the 5 m width and the depth of 6 m at the crown and 9 m at the rib bases, shown in Fig. 10, which behaves as a very bluff body.

The random vortex method code RVM-FLUID was performed on the 2D model of the rib cross-section with the average depth of 7.5 m. It was found that the severe vortex-shedding oscillation happens with the amplitude of 0.028 \( H \) at the reduced frequency or Strouhal number \( St \approx 0.156 \). In order to improve vortex-shedding vibration of the bluff cross-section of the ribs, several aerodynamic preventive means, shown in Fig. 11, were numerically tested in this cross-section, and the calculation results including Strouhal number and relative amplitude are listed in Table 6. There are only four means including CS-2, CS-6, CS-7 and CS-8, which can reduce the amplitude of vortex oscillation more or less. Among these four means, the best solution is the preventive means of the full cover plate (CS-8), which can minimize the amplitude to only about 40% of that in the original configuration.

With the further aid of computer simulation, it becomes possible to visually describe how the flow passes by rib cross-sections, in particular, the original cross-section (CS-1) and the cross-section with the full cover plate (CS-8). In the case of the original structure, two large-scale vortices, originally formed at the top side and bottom side of the left rib from the Lupu arch representative cross section: (a) original section, and (b) modified section with an upper full cover plate. In the first case, the vortex shed in the upwind section interfere with the downwind section, as studied in Chapter 3 for rectangular sections in tandem arrangement. The placement of the upper full
cover plate puts an end to the vortex shedding regime, giving rise to a recirculation bubble in the lower part characteristic of H-shape cross sections. A similar configuration can also be observed in the case of Niterói (cf. Fig. 1.20).

The use of two-dimensional numerical simulations should be completed with the analysis of several arch cross section, given that it has a variable depth. Therefore, the tandem configuration geometry is different at every point of the arch. However, the extensive aerodynamic study carried out on the Lupu Bridge, which includes full bridge model wind tunnel tests offers sufficient information on its structural behaviour, the numerical simulations being another additional complement in this case for result comparison.

5.4 Influence of barriers and windshields

Representative cross sections of structural elements of the bridge, i.e. decks or towers, can have additional details that could be decisive in the development of VIV. The influence of bracings of rectangular cross sections in tandem arrangement have already been studied in Chapters 2 and 4. In this case, the solidity ratio determines the gap between the two main cross sections, which conditions the vortex shedding mechanism. Traffic barriers, which contribute to vehicle and pedestrian safety, can significantly influence the evolution of the flow regime depending on some parameters, such as its size and its solidity ratio. Windshields, which are placed to protect vehicles against wind effects, such as fluctuations, increase the static wind loads and may also cause dynamic excitation of the structure if the geometry is unfavourable (cf. §5.2.4). Fig. 5.48 shows an example of the windshields installed on the Millau Bridge. It is not possible to argue initially if

![Figure 5.48: Windshields installed on cable-stayed bridges: Millau Bridge (left, picture courtesy of Pickyview) and Hangzhou Bay Bridge (right, picture courtesy of Palram).](image)

the presence of safety barriers and windshields represent a benefit or a drawback regarding the VIV effects, given that each case is analysed in a particular way. The article [149] includes some practical examples referred to this issue. A proper design of these elements is essential to control the aerodynamic effects.
Lastly, vehicles crossing the bridge can significantly influence the aeroelastic vibrations regime. Some hypotheses addressing the presence of vehicles on the structure can be made regarding the number of vehicles, distribution along the deck surface, or type (cars, trucks). A practical example was done in §5.2.1 for the Niterói Bridge case study.

5.5 Summary

The numerical simulations carried out in the case studies of Niterói, Volgograd and Trans-Tokyo Bay have allowed result validation obtained through their comparison with full scale observation of a number of vibration episodes. In the case of Niterói Bridge, the main aim was to analyse the possible influence of the traffic on the VIV effects, given that the presence of vehicles on the bridge can significantly influence the vortex formation and shedding. For this, two-dimensional and pseudo-three-dimensional numerical simulations have been carried out. The latter allows the study of the wind structure interaction from the modelling of the main bridge span using several cross section geometries. The presence of the barriers on the Volgograd Bridge seem to be crucial for VIV effects caused by vortex shedding under average wind velocities of around 12 m/s - 15 m/s in May 2010, some months after bridge completion. The existence of a second twin bridge could have avoided the problem, given that the vortex shedding would not have developed in the same way with a distinct geometry configuration of two decks. The oscillation produced by resonant effects were expected to happen before the completion of the Trans-Tokyo Bay Bridge, in which a set of TMDs were placed in the interior of the main span girders. The numerical simulations performed maintain a good correlation with the aerodynamic studies carried out by the Japanese engineers in charge, which confirms its use as effective devices to counteract the vibration effects due to wind actions. A simple methodology is shown for sensitivity of bridge decks to VIV effects, based on the analysis of two static parameters: the root mean square lift coefficient, $C_{L,rms}$, proportional to the vertical oscillation amplitudes, and the Strouhal number, $St$, which defines a theoretical resonance wind velocity, $U_{\infty,crit}$, from which a critical wind velocity range is established. Said parameters can be compared with those corresponding to the so-called basic reference cross sections (rectangular, H-shaped and in tandem), determined as a function of the aspect ratio, $W/D$, from experimental tests and static numerical simulations. The comparison is performed by giving to the complex section of study an equivalent width and depth, $W^*$ and $D^*$, depending on how the interaction mechanism is produced and which is the reference basic section geometry. Finally, the main countermeasures used in the control of VIV effects are described, including the solutions performed in the case studies. An adequate aerodynamic geometry design of the different structural bridge elements can avoid VIV problems, although the other aeroelastic phenomena should also be taken into account.
Chapter 6

Semi-empirical model

6.1 Objectives and scope

In the present chapter a new semi-empirical method is proposed for the estimation of the critical wind velocity range and oscillation amplitudes associated with a structural model represented by a 1DOF system and a cross section geometry. The so-called semi-empirical model uses the general equation of motion, Eq. \((1.40)\) and evaluates the system response, \(y\), for different values of free-stream velocity, \(U_\infty\). The aerodynamic force time history, which depicts the excitation actions, is acquired from numerical simulations or experimental tests carried out with a scale model that reproduces the bridge geometry or the element thereof. With the aim of taking into account the influence of the self-excited forces, some additional actions are included in the equation of motion, which depend on the flutter derivatives, \(H^*_1\) and \(H^*_4\).

The semi-empirical model is applied to the case studies presented in Chapters 2 and 5, with a corresponding analysis of results. Finally, a summary with the main conclusions is undertaken.

6.2 Model formulation

6.2.1 Theoretical background

The characterisation of the VIV phenomenon to evaluate the structure sensitivity has been materialised through relatively simple expressions included in a number of normatives and standards. These simplified expressions are derived from theoretical models developed from experimental tests and real observations. Thus, the model covered in the Eurocode (EN1991-1-4) [35] explicitly includes the expression proposed by Ruscheweyh [141] [142]:

\[
\frac{y_{F,\text{max}}}{b} = \frac{1}{8b^2} \cdot \frac{1}{Sc} \cdot K \cdot K_w \cdot c_{lat}
\]  

(6.1)
where $y_{F,\text{max}}$ is the maximum cross-wind deflection, $b$, the cross section width (a representative cross section dimension), $St$, the Strouhal number, $Sc$, the Scruton number, $K$, the mode shape factor, $K_w$, the effective correlation length factor, and $c_{lat}$, the lateral force coefficient.

The Ruscheweyh model was conceived to estimate the VIV detected in chimneys. Furthermore, the characteristics of the wind regime acting on vertical structures are different of those acting on horizontal line-like structures, mainly when certain simplifications are made.

The Eurocode includes a second approach for the estimation of the VIV amplitude based on the original model of Vickery & Basu [170] and developed by Dyrbye & Hansen [33] in order to take into account the turbulence effects:

$$\frac{\sigma_y}{b} = \frac{1}{St^2} \cdot \frac{C_c}{\sqrt{\frac{Sc}{4\pi} - K_a \cdot \left[1 - \left(\frac{\sigma_y}{b_{aL}}\right)^2\right]}} \cdot \sqrt{\frac{\rho b^3}{m_e}} \cdot \sqrt{\frac{b}{h}}$$  \hspace{1cm} (6.2)

where $\sigma_y$ is the standard deviation of deflection in reference point, $St$, Strouhal number, $C_c$, aerodynamic constant dependent on the cross section shape, $Sc$, Scruton number, $K_a$, an aerodynamic damping parameter, $a_L$, the normalised limiting amplitude giving the structural deflection with very low damping, $\rho$, the air density, $m_e$, the effective mass per unit length, $h, b$, the height and width of the representative cross section, respectively.

In this case, $\sigma_y$ should be estimated through an iterative process.

The Canadian standards [20] uses an adaptation of the analytical model of Vickery & Basu to estimate the VIV structural response:

$$y_i(x) = a_i \mu_i(x)$$  \hspace{1cm} (6.3)

where

$y_i(x) =$ peak member displacement due to vortex shedding excitation at location $x$ for vibration mode, $i$.

$a_i =$ modal coefficient of magnitude of the oscillatory displacement for vibration mode, $i$, for a member with a constant diameter or frontal width, $m$, which is equal to

$$\frac{3.5C_L\rho D^4\pi^{0.25}C}{\sqrt{B\zeta_i(4\pi S)^2GM_i}} \quad \text{if } y_i(x) \leq 0.025D$$

$$\frac{2(\rho)C_L D^3 \int_0^H |\mu_i(x)|dx}{\sqrt{B\zeta_i(4\pi S)^2GM_i}} \quad \text{if } y_i(x) > 0.025D$$

which is also resolved through an iterative process.

Vickery [169] subsequently published an analysis on the problems and difficulties that the use of his methodology entails for the full-scale real cases.

The British Standard (BD 49/01) [54] offers a simpler expression for the estimation of the oscillation amplitude caused by VIV on bridge decks:

$$y_{\text{max}} = \frac{cb^{0.5}d^{2.5}\rho}{4m\delta_s}$$  \hspace{1cm} (6.4)
where $c$ is a solidity ratio, $b$, the overall width of the bridge deck $d$, the effective cross section depth $\rho$, the density of air, $m$, the mass per unit length of the bridge, and $\delta_s$, the logarithmic decrement due to structural damping.

The coefficients of the previous expression were adjusted from results obtained in multiple experimental tests performed, in addition to data collected from full-scale observations of some bridges.

Each expression of those presented previously take into account the main parameters that have an influence on the vibration amplitudes, such as the section geometry, the structural mass and the damping$^1$.

Said expressions are valid for simple or typical geometries considered in the normative. However, good estimations are not achieved when cross sections of more complex geometry are allowed for, a fact that has motivated the development of the new proposed semi-empirical method.

In the preliminary literature review, other analytical models suggested by different authors for the estimation of the VIV response have been investigated. Scanlan [155] and Strømmen [159] propose two different methodologies, the first combined with the use of a number of parameters determined from experimental tests, and the second based on a statistical treatment.

Larsen [88] describes a non-linear model for flexible structures subjected to vortex shedding excitation under lock-in conditions, simultaneously to the development of new numerical methods for the interaction modelling, based on the vortex particle method (VPM) (cf. §1.3.2).

The characterisation of the excitation forces that result in the VIV process turns out to be of special interest. Barrero [12] presents a non-linear model for the estimation of VIV, which takes into account a time of delay between the action of the force and its influence on the response. It is taken for granted that, once the oscillations have started, some self-excited forces also appear due to the movement of the structure. It also asserts that the self-limited character of the oscillation leads one to think of a negative aerodynamic damping term for small amplitudes and a positive one for higher amplitudes. An analytical expression is proposed and performs the check for a 1DOF model with a circular representative section.

In line with the models of Ruscheweyh and Vickery & Basu, Flaga et al. [38] give a new one for the determination of the VIV effects in chimneys and high-rise towers, based on the Weighted Amplitude Waves Superposition (WAWS) method, which use the PSD functions of the aerodynamic forces associated to the vortex excitation.

Furthermore, Diana et al. [30] developed a numerical model to investigate VIV in the Strait of Messina Bridge Project, which highlights the typical non-linear pattern of the vortex shedding. More recently, Wu & Kareem [178] [179] suggest the use of a methodology based on the Volterra series for the VIV analysis, which allows one to establish closer positions with the current study of other aeroelastic effects.

---

$^1$For further details, see the corresponding normative.


6.2.2 Estimation of the vibration amplitude

In the previous section, a brief review of some of the analytical models used for the VIV estimation was made, as well as the expressions included in the current normatives. Taking into consideration the most notable aspects that characterise the phenomenon, below a new proposal is included, applicable to the study of vertical vibrations caused by the resonant VIV phenomena derived from the wind-structure interaction.

The vertical movement corresponding to a 1DOF model is defined by Eq. (1.36), reproduced again below:

\[
M_y \ddot{y} + (2\xi_y \omega_y) \dot{y} + \omega_y^2 y = F_y(t, y, \dot{y}, \ddot{y}) \tag{6.5}
\]

where \(M_y\) is the structural mass and \(\omega_y\) is the natural frequency associated to the fundamental vertical mode of the structural model. In this case, the total forces acting on the system are represented through \(F_y(t, y, \dot{y}, \ddot{y})\), which indicates dependence on time, and also the vertical displacements of the model itself, \(y\), and its first, \(\dot{y}\), and second, \(\ddot{y}\), derivatives. The dependence of the acting forces in the interaction response of the structure converts the equation of motion into an non-linear expression, raised as a starting hypothesis on which the new analytical model is based. The total forces can be separated into two types:

\[
F_y(t, y, \dot{y}, \ddot{y}) = F_a(t) + F_{se}(t, y, \dot{y}, \ddot{y}) \tag{6.6}
\]

where \(F_a\) represents the aerodynamic forces, and \(F_{se}\), the self-excited forces.

The aerodynamic force is equivalent to the lift force defined for the static section:

\[
F_a(t) = \frac{1}{2} \rho U_{\infty}^2 B C_L(t) \tag{6.7}
\]

where \(\rho\) is the air density, \(U_{\infty}\), free-stream wind velocity, \(B\), the section width, and \(C_L\), the lift force coefficient.

This expression can be deduced also from the definition of the lift force, \(C_L\), Eq. (1.9). As seen in §1.3, VIV are a resonance phenomenon caused by the aerodynamic forces \((F_a)\), as a result of the wind-structure interaction. It is a self-limited phenomenon and with low vibration amplitudes. Therefore, it has been traditionally considered that the forces generated as a consequence of the structural oscillations, or self-excited forces, have little or no influence in the dynamic response \([16]\) \([155]\). In its general classification of flow-induced vibrations depending on the excitation source, Naudascher & Rockwell \([133]\) include it within the group of the instability-induced excitations (IIE), given that they are vibrations caused by instabilities originated as consequence of the interaction with the structure. However, the studies carried out in Chapter 3 allowed one to have a wider knowledge of the interaction mechanism, developed in various generic sections, and reinforce the hypothesis that, in certain cases, the self-excited forces can have a significant influence on the development of the VIV phenomenon, and the magnitude of the response.
According to Scanlan’s flutter theory [155], briefly shown in §1.2.4.1, the self-excited forces for a 2DOF system are given as a function of Scanlan’s flutter derivatives (cf. Eq. (1.27) and (1.28)). In the case of a 1DOF, the self-excited forces are reduced to the following expression:

\[ F_{se}(t) = \frac{1}{2} \rho U_\infty^2 B \left[ KH_1^* \frac{\ddot{y}}{U_\infty} + K^2 H_4^* \frac{\dot{y}}{B} \right] \]  

(6.8)

where \( H_1^* \) and \( A_1^* \) are the Scanlan flutter derivatives [155], and \( K \) is the reduced frequency, already defined in Eq. (1.29) as:

\[ K = \frac{B \omega}{U_\infty} \]  

(6.9)

The expression for the self-excited forces can be written as:

\[ F_{se}(t) = \frac{1}{2} \rho B^2 \left[ \omega H_1^* \dot{y} + \omega^2 H_4^* y \right] \]  

(6.10)

The assumption of the Scanlan’s flutter theory to consider the effects of the self-excited forces is carried out based on three reasons: (1) it is a well-known and widely-used methodology in Bridge Aerodynamics for the estimation of flutter characterisation, (2) there are experimental data of flutter derivatives for a number of case studies that can be found in the literature, (3) it is a way to relate VIV and flutter through the use of some of the same parameters, under an overall perspective of study of the aeroelastic phenomena (cf. §1.2.5). On the other hand, the use of the flutter derivatives also involves the assumption of the hypotheses on which Scanlan’s theory is based, such as the consideration of linearity of the self-excited forces for low deflections (cf. §1.2.4.1), and that they must be determined under conditions of laminar airflow regime. Another aspect to highlight is that according to the theory expressed in §1.2.4.1, in the use of the flutter derivatives, Scanlan & Tomko [150] assert that for low oscillations, the self-excited forces can be considered linear in their vertical displacements (\( h \)) and rotations (\( \alpha \)), as well as in their first and second derivatives. In summary, the final equation of motion taking the self-excited forces into account, is:

\[ M_y \left[ \ddot{y} + (2\xi_y \omega_y) \dot{y} + \omega_y^2 y \right] = \frac{1}{2} \rho U_\infty^2 BC_L \underbrace{F_a}_{\text{F_a}} + \frac{1}{2} \rho B^2 \underbrace{\left[ \omega H_1^* \dot{y} + \omega^2 H_4^* y \right]}_{\text{F_se}} \]  

(6.11)

which is a non-linear second order differential equation, whose analytical resolution requires the knowledge of the dynamic properties of the model, such as the structural modal mass, \( M_y \), the modal damping factor, \( \xi_y \), and the modal frequency, \( \omega_y \). In this case, the excitation frequency, \( \omega \), coincides with the natural frequency, given that there is only one vertical vibration mode. Moreover, it is necessary to define the free-stream velocity of the acting wind, \( U_\infty \), as well as the lift coefficient, \( C_L \), and the flutter derivatives, \( H_1^* \) and \( H_4^* \), which depend on the wind-structure interaction. These latter parameters are obtained from wind tunnel tests or static numerical simulations, in the case of \( C_L \), and tests or numerical
simulations of forced oscillation, in the case of the flutter derivatives, and for that reason the model is named semi-empirical.

A number of numerical methods for solving Eq. (6.11) can use and obtain the time history of vertical displacements for each $U_\infty$, e.g. central differences or Newmark’s [24], among others. However, given that the main objective of the semi-empirical model is to achieve the response amplitude of the 1DOF system associated with a specific wind velocity range to identify the possible presence of VIV, the previous differential equation will be solved in the frequency domain. It is also advantageous, given that the Scanlan’s flutter derivatives are a function of frequency and not of time. The solution to Eq. (6.11) consists of two parts:

$$y(t) = y_t(t) + y_s(t)$$

(6.12)

where $y_t(t)$ is the transient response and $y_s(t)$ is the stationary response [24]. The damping effect makes the transient response become null after a specific time, leaving only the excitation force of the stationary response: $2$:

$$y(t) = y_t(t) + y_s(t)$$

(6.13)

The homogeneous part of Eq. (6.11) will be equal to the characteristic free damped oscillation systems, and the particular solution of the non-homogeneous one to establish the periodic term, which is of the form:

$$y(t) = Ye^{i\omega t}$$

(6.14)

This expression has the structure of the Fourier transform, which is defined as:

$$y(t) = \frac{1}{2\pi} \int_{-\infty}^{\infty} Y(\omega)e^{i\omega t} d\omega$$

(6.15)

where $y(t)$ is the response function in the time domain, and $Y(\omega)$ is the response function in the frequency domain, or Fourier transform of $y(t)$, whose expression is:

$$Y(\omega) = \int_{-\infty}^{\infty} y(t)e^{-i\omega t} dt$$

(6.16)

and is extended to the whole frequency range. So that the Fourier transform exists, the function $y(t)$ should be locally integrable in $\mathbb{R}$, and have a limited value. Its first derivative takes the form of:

$$\dot{y}(t) = i\omega Ye^{i\omega t}$$

(6.17)

and its second derivative:

$$\ddot{y}(t) = i^2\omega^2 Ye^{i\omega t} = -\omega^2 Ye^{i\omega t}$$

(6.18)

$^2$To consider this aspect in the analytical response by taking the lift force time history, the initial time steps in which the response is clearly irregular, i.e. the transient phase, is omitted.
Substituting the previous expressions in Eq. (6.11), leads to:

\[
M_y (\omega_y^2 - \omega^2 + 2i\omega_y\omega \xi_y) Y(\omega) = \left(\frac{1}{2}\rho B^2 H^*_4 i\omega^2 + \frac{1}{2}\rho B^2 H^*_1 \omega^2\right) Y(\omega) + F_Y(\omega)
\]

(6.19)

where \(F_Y(\omega)\) is the aerodynamic force, \(F_a\), expressed also in the frequency domain through its corresponding Fourier transform.

Reorganising the terms of Eq. (6.19)

\[
\begin{bmatrix}
Z_{SS} & Z_{SA} \\
Z_{DS} & Z_{DA}
\end{bmatrix}
\begin{bmatrix}
Y(\omega)
\end{bmatrix}
= \begin{bmatrix}
F_Y(\omega)
\end{bmatrix}
\]

(6.20)

where the \(Z_i\) terms represent the different impedance functions that come into play in the system. In this way, \(Z_S\) is the impedance function associated with the overall stiffness, and \(Z_D\) is the impedance function associated with the overall damping. Each one of the previous terms can be subdivided into two, in a way that \(Z_{SS}\) is the impedance function due to the structural mass of the 1DOF model\(^3\), \(Z_{SA}\), is the impedance function due to the aerodynamic stiffness (related to the flutter derivative \(H^*_4\)), \(Z_{DS}\) is the impedance function of the structural damping, and \(Z_{DA}\), is the impedance function due to the aerodynamic damping (related to the flutter derivative, \(H^*_1\)).

Eq. (6.20) allows one to distinguish the different parameters that intervene in the interaction process. The ‘structural’ terms are related with the dynamic properties of the 1DOF model, which represents the structural behaviour, and the ‘aerodynamic’ terms introduce an additional damping due to the oscillation effects, which stands for the self-excited forces mentioned previously.

Eq. (6.19) can be simplified as:

\[
[Z_S + iZ_D] Y = F_Y
\]

(6.21)

leaving \(Y\):

\[
Y = [Z_S + iZ_D]^{-1} F_Y = \frac{(Z_S - iZ_D)}{(Z_S^2 + Z_D^2)} F_Y
\]

(6.22)

where

\[
H(\omega) = \frac{(Z_S - iZ_D)}{(Z_S^2 + Z_D^2)}
\]

(6.23)

is the impedance function of the system, defined also in the frequency domain.

From Eq. (6.22) it can go to the expression in spectral terms:

\[
S_y(\omega) = S_{F_a}(\omega) \cdot H^2(\omega)
\]

(6.24)

\(^3\)Note that the structural stiffness of the system is \(K = M_y\omega_y^2\).
where \( S_y(\omega) \) is the PSD of vertical deflections, and \( S_{F_a}(\omega) \) is the PSD of the aerodynamic forces, assuming that they are of a random nature in the most general case.

Substituting Eq. (6.23) in Eq. (6.24), it leads to:

\[
S_y(\omega) = \frac{(Z_S - iZ_D)}{(Z_S + Z_D^2)} \cdot S_{F_a}(\omega) \cdot \frac{(Z_S + iZ_D)}{(Z_S + Z_D^2)} = \frac{S_{F_a}(\omega)}{(Z_S + Z_D^2)}
\]

being, therefore

\[
H^2(\omega) = \frac{1}{Z_S^2 + Z_D^2}
\]

The final expression is:

\[
S_y(\omega) = \frac{S_{F_a}(\omega)}{(Z_S^2 + Z_D^2)}
\]

which in its extended form, if the terms are substituted from Eq. (6.20) in Eq. (6.27), is:

\[
S_y(\omega) = \frac{S_{F_a}(\omega)}{[\left( M_y(\omega_y^2 - \omega^2)^2 + M_y(\omega_y^2 - \omega^2)^2 + M_y \omega_y^2 \omega_y \cdot \xi_y - \frac{1}{2} \rho B^2 H^2(\omega_y^2)^2 ]}
\]

As indicated previously, the aerodynamic forces, \( F_a \), are of a random nature with an average null value, \( E[F_a] = 0 \), whose PSD is \( S_{F_a} \). A fundamental property of the PSD is that the enclosed area under its curve corresponds to the variance, \( \sigma^2 \), from the corresponding variable, \( F_a \) or \( y \). Thus, one can have:

\[
\sigma_y^2 = \frac{1}{2\pi} \int_{-\infty}^{\infty} S_y(\omega) \, d\omega
\]

The spectrum of a real valued process is an even function of frequency, \( S_y(-\omega) = S_y(\omega) \), and hence:

\[
\sigma_y^2 = \frac{1}{2\pi} \left( \int_{-\infty}^{0} S_y(\omega) \, d\omega + \int_{0}^{\infty} S_y(\omega) \, d\omega \right) = \frac{1}{2\pi} \left( 2 \int_{0}^{\infty} S_y(\omega) \, d\omega \right)
\]

The typical deviation is:

\[
\sigma_y = \sqrt{\frac{1}{2\pi} \left( 2 \int_{0}^{\infty} S_y(\omega) \, d\omega \right)}
\]

which is also the root mean square of the response (cf. Eq. (1.41)), or \( y_{rms} \). This is the representative value of the response amplitude of the 1DOF model for a certain free-stream wind velocity, \( U_\infty \).

If, instead of the angular frequency, \( \omega \), the natural vibration frequency is used, \( d\omega = 2\pi df \), one can have:

\[
\sigma_y = \sqrt{2 \int_{0}^{\infty} S_y(f) \, df}
\]
The use of similar analytical schemes has also been raised by other authors, such as Astiz [9], for the study of the aeroelastic instability of bridges in turbulent regime. For specific geometry configurations, such as rectangular sections with $B/D \leq 0.5$ (cf. Fig. 3.23, $C_L$ time history), the time history of the aerodynamic forces resulting from the interaction is very similar to a periodic harmonic function that can be characterised by the following expression:

$$F_a = \frac{1}{2} \rho U_{\infty}^2 B C_L \cdot \sin(\omega_s t) = F_{a,0} \cdot \sin(\omega_s t) \tag{6.33}$$

where $\omega_s$ is the vortex shedding frequency, and $F_{a,0}$ is the amplitude or maximum amplitude value of the aerodynamic forces.

The correlation function is [25]:

$$R_{F_a}(\tau) = E(F_a(t), F_a(t + \tau)) = F_{a,0}^2 \frac{\cos(\omega_s \tau)}{2} \tag{6.34}$$

being the PSD:

$$S_{F_a}(\omega) = \frac{1}{2\pi} F_{a,0}^2 \frac{\delta(\omega_s)}{2} \tag{6.35}$$

where $\delta(\omega_s)$ is the Dirac delta function, which acquires a unit value when $\omega = \omega_s$, and null value for $\omega \neq \omega_s$. $S_{F_a}(\omega)$ can be substituted in Eq. (6.28) and evaluated for a harmonic aerodynamic force. Eq. (6.35) also can be expressed as a function of the vortex shedding frequency, $f_s$:

$$S_{F_a}(f) = F_{a,0}^2 \frac{\delta(f_s)}{2} \tag{6.36}$$

The root-mean square of the aerodynamic force is:

$$\sigma_{F_a}^2 = \int_{-\infty}^{\infty} S_{F_a}(f) df = \int_{-\infty}^{\infty} F_{a,0}^2 \frac{\delta(f_s)}{2} df = \frac{1}{2} F_{a,0}^2 \tag{6.37}$$

and the typical deviation is:

$$\sigma_{F_a} = \sqrt{\frac{1}{2} F_{a,0}^2} = \frac{\sqrt{2}}{2} F_{a,0} \tag{6.38}$$

Going back to the general case, the application of the semi-empirical model for the study of the VIV phenomenon is based on the estimation of the representative vibration amplitude, $\sigma_y = y_{rms}$, for each velocity value of uniform wind, $U_{\infty}$, within a specific range in which it is expected to have resonant effects. Eq. (6.28) is evaluated in the interval $(0, \, \omega \rightarrow \infty)^4$, through the use of a discretisation method [24] [25], taking a frequency step of $\Delta \omega$, that allows a proper discretisation. The representative amplitude in discrete form is then:

$$\sigma_{y} = \left[ \frac{1}{2\pi} \left( \sum_{i=1}^{N} (S_y(\omega_i) \Delta \omega) \right) \right]^{1/2} \tag{6.39}$$

\(^4\text{It is enough to consider a sufficiently high } \omega \text{ value, e.g. } \omega = 1,000 \text{ rad/s} \)
where

\[ \Delta \omega = \frac{1}{N} \frac{2\pi}{\Delta T} = cte \]  \hspace{1cm} (6.40)

being

\[ \Delta T = t_e - t_0 \]  \hspace{1cm} (6.41)

the total time interval considered, and \( N \), the number of time steps, i.e. divisions of the time interval.

Eq. (6.39) can be written as:

\[ \sigma_y = \sqrt{\frac{2}{N \Delta T} \left( \sum_{i=1}^{N} (S_y(\omega))_i \right)} \]  \hspace{1cm} (6.42)

Fig. 6.1 shows the flow chart of the calculation algorithm developed for the evaluation of various case studies included in §6.3. To determine a wind velocity range in which to carry out the evaluation, it is advisable to first perform a Fourier analysis of the aerodynamic force time history determined experimentally (wind tunnel tests) or through numerical simulations. The theoretical wind velocity or velocities that places the vortex shedding frequency close to the natural vibration frequency is then defined.

Through the evaluation of the equation of motion in the frequency domain through an iterative process, a single value is obtained, \( y_{\text{rms}} (= \sigma) \) for each value of free-stream wind velocity, \( U_\infty \). A velocity-displacement curve is depicted to identify the presence of VIV. Moreover, a set of those curves associated with different values of structural masses, \( M_y \), and damping ratios, \( \xi_y \), can be drafted, which allows the study of the variation of the representative amplitudes depending on the structure dynamic properties\(^5\).

Next, the results of the case studies included in this research work are presented, highlighting their most significant aspects too.

### 6.3 Applications

#### 6.3.1 The Alconétar Bridge

Fig. 6.2 shows the values of the flutter derivatives of the Alconétar cross section, whose study was performed in Chapter 2. These values have been obtained from numerical simulations. In this case, only the functions \( H_1 \) and \( H_4 \) are needed, given that it is intended to apply the semi-empirical model for the estimation of the vertical vibrations.

Fig. 6.3 shows the amplitudes obtained for each free-stream wind velocity, \( U_\infty \), applying the semi-empirical model. The corresponding curve of amplitudes of the Alconétar study case achieved through dynamic simulations is also included (cf. Fig. 2.15). The semi-empirical model records a resonance peak for a critical wind

---

\(^5\)The parameters of structural mass and damping ratio can be considered jointly through the Scruton number (cf. Eq. (1.38)).
Chapter 6. Semi-empirical model  Vortex-induced vibrations on bridges

Read output file (wind forces from numerical simulations)

Fourier analysis
Get Strouhal: \( \text{St} \)

Semi-empirical model

Input: \( M_e, \omega_n, \xi \)
(structural dynamic prop.)

Input wind veloc.: \( U_\infty \)
(range or specific value)

Choose time interval for wind forces (avoid transient regime)

Read data files:
\( v_r, (d_r), H_1^*, H_4^* \)
(forced simulations)

Does \( y_0 \) exist?

Semi-empirical model
Calculate:
\( SFa(\omega), Z_S, Z_D \)

PSD of vertical disp.
\( S_y(f) = \frac{SFa(\omega)}{Z_S^2 + Z_D^2} \)

Amplitude of vertical disp.
\( y_{rms} = \sigma_y = \sqrt{\int S_y(\omega)} \)

End of iteration
\( \epsilon = \frac{|y_{rms} - \sigma_y|}{\sigma_y} \leq 0.01 \)?

Write output files (.o5, .o6)
Model parameters
\( SFa(\omega), Z_S, Z_D, \ldots \)

End wind speed range?

Write output file (.o7)
Pairs \( (U_\infty, \sigma_y) \)

Use \( \sigma_y \) from previous iteration as \( y_0 \)

Figure 6.1: Semi-empirical model implementation: flow chart.
Chapter 6. Semi-empirical model  Vortex-induced vibrations on bridges

Figure 6.2: Computed Scanlan’s flutter derivatives of the Alconétar original cross section model (cf. Chapter 2) from numerical simulations.

Figure 6.3: Comparison of vertical oscillation amplitudes of the Alconétar original cross section model of numerical dynamic simulations (cf. Chapter 2) and semi-empirical model estimations (aerodynamic forces from static simulations).
velocity of 13.2 m/s, the representative oscillation amplitude being 0.52 m, much higher than that obtained through the numerical simulations, 0.24 m.

In addition to Alconétar, the semi-empirical model was applied to other case studies for result validation. While acceptable estimations of the critical wind velocity range were obtained for critical wind velocity ranges, amplitude estimations registered some deviations with respect to the expected values, taking as a reference the ones from full scale observations and dynamic numerical simulations previously performed. In order to improve the model, it was considered to take the aerodynamic force time history from forced simulations rather than from the static ones, in order to gather the influence of oscillations on the excitation forces acting over the system. The determination of Scanlan’s flutter derivatives was carried out also through a set of forced numerical simulations for different wind velocities, in which the sectional 1DOF model is subjected to a constant oscillation period, $T_F$, and a forced amplitude, $d_F$. Generally, forced amplitude values close to 0.1$D$ are used, being $D$ the section depth. According to the Scanlan’s flutter theory, flutter derivatives should be calculated under a laminar airflow regime, i.e. low Reynolds numbers. Theoretically, those flutter derivative functions are independent of the forced oscillation amplitude used for their determination, $d_F$. However, Reynolds numbers of the wind-structure interaction process are $Re > 10^6$, characteristic of a turbulent airflow regime, and the flutter derivatives are calculated following this premise. Fig 6.4 shows the corresponding values for the Alconétar case study depending on the reduced velocity, $v_{red}$, for different values of forced heave amplitude, $d_F$, i.e. 0.1$D$, 0.2$D$, 0.3$D$, 0.4$D$, 0.5$D$ and 1.0$D$. In the case of $H_s^*$, big

![Figure 6.4: Computed Scanlan’s flutter derivatives of the Alconétar original cross section model from numerical forced simulations using different heave amplitudes, $d_F$.](image)

differences are not seen, the curves following a similar tendency. For $H_s^*$, however, the values show a wider variation. The values of the aerodynamic forces affected by the model oscillations can also show significant variations with respect to those determined from static numerical simulations, where the section model is fixed.
Chapter 6. Semi-empirical model

Vortex-induced vibrations on bridges

To take into account the influence of the oscillations on the aerodynamic force time history, a series of forced numerical simulations are performed using different forced amplitudes $d_F$, that address a sufficient range that covers the bandwidth within the VIV effects are expected to happen. Fig. 6.5 shows a comparison between the PSD of the aerodynamic forces obtained through static numerical simulation, $S_{F_a}(\omega)$, and through forced ones, $S_{F_a}(\omega, d_F)$, being $d_F$ the forced amplitude, which corresponds with the estimated amplitude associated to the critical wind velocity for VIV, $U_\infty = 13.2$ m/s. Such amplitude is calculated through an iterative process, as indicated on Fig. 6.1. One can observe that the PSD peak value is reached for the natural vibration frequency of the Alconétar study case, $f_n = 0.70$ Hz. In this case, both spectra are quite similar, which happens when having a simple section geometry, as it is the case of the two rectangular boxes in tandem of the Alconétar arch. For more complex cross section geometries, more appreciable differences can be observed between the two aerodynamic force spectra, $S_{F_a}$, which also has an impact on the deflection amplitudes.

The implementation of the semi-empirical model is more complex if the forced simulations are chosen for the determination of the aerodynamic forces, since it requires a higher number of iterations to achieve the convergence, given the non-linearity of the problem.

Fig. 6.6 shows several charts of the main parameters associated with the model (cf. Eq. (6.27)) for the critical wind velocity, $U_{\infty, crit}$. The vertical line indicates the value of the natural frequency of Alconétar, $f_n = 0.70$ Hz. The PSD of displacements, $S_y$, can be calculated from the PSD of aerodynamic forces, $S_{F_a}$, which is smoother, since the impedance function, $1/(Z_D^2 + Z_D^2)$, acts as a filter.

Fig. 6.7 shows the results of the improved semi-empirical model (left), where the influence of the oscillations on the input parameters is taken into consideration.

![Figure 6.5: Alconétar cross section model: lift force PSD of static simulations ($S_{F_a}(\omega)$), and forced simulations ($S_{F_a}(\omega, d_F)$) for $U_\infty = 13.6$ m/s.]
Figure 6.6: Alconétar cross section model: lift force PSD ($S_{F_a}$, top), admittance function (middle) and vertical displacement PSD ($S_y$, bottom). The vertical line defines the actual natural frequency ($f_n = 0.70$ Hz).
The estimation of critical wind velocity, $U_{\infty,\text{crit}} = 13.2$ m/s, is slightly lower than

![Figure 6.7: Comparison of vertical oscillation amplitudes of the Alconétar original cross section model of numerical dynamic simulations and semi-empirical model estimations(left). Semi-empirical model estimations of root mean square vertical oscillation amplitudes for various damping ratios (right).](image)

that obtained from the dynamic simulations presented in Chapter 2, being the oscillation amplitudes very similar. Table 6.1 summarizes the results of the different case studies. The curve of amplitude-wind velocity ($y_{\text{rms}} - U_{\infty}$) of previous version of the semi-empirical model (cf. Fig. 6.3) is also included in Fig. 6.7, and both curves can be compared. On the right, it is included a chart that shows amplitude estimations for various damping ratios, $\xi$, being the curve of the Alconétar case study the one that corresponds to a $\xi = 0.30\%$. This parameter significantly conditions the oscillation amplitudes associated with the VIV effects. Therefore, it turns out to be of great importance the sensitivity analysis of the aeroelastic effects according to different damping ratios. In this case, it can be observed that for values higher than 10%, the VIV effects practically disappear.
6.3.2 Other bridge study cases

Following the methodology of the previous example, presented below are the results obtained through the application of the semi-empirical model for the case studies of the Niterói, Volgograd and Trans-Tokyo Bay bridges, whose numerical analyses were carried out in Chapter 5 are presented below. The improved version of the model was used, in view of the results achieved for Alconétar.

Fig. 6.8 shows the values of the Scanlan’s flutter derivatives, $H_1^*$ and $H_4^*$, for the three cases according to $v_{\text{red}}$. Like Alconétar, the $H_1^*$ values are more uniform, although there are also significant differences for specific curves, above all for higher reduced wind velocities. However, the greatest differences continue to be maintained for the $H_4^*$ values.

Figs. 6.9, 6.11 and 6.13 show the estimations performed for the cases of Niterói ($U_{\infty,\text{crit}} = 16.0 \text{ m/s}$), Volgograd ($U_{\infty,\text{crit}} = 13.6 \text{ m/s}$), and Trans-Tokyo Bay ($U_{\infty,\text{crit}} = 15.2 \text{ m/s}$), respectively. In all these, the vertical line represents the fundamental vertical natural frequency. It is observed that the excitation frequencies take values slightly below the theoretical natural frequencies. This is due to the fact that in the underdamped systems \cite{25}, the real natural frequencies are given by the following expression:

$$f_{n,d} = f_n \sqrt{1 - \xi^2}$$  \hspace{1cm} (6.43)

where $f_{n,d}$ is the damped natural frequency and $f_n$ is the non-damped natural frequency, being $\xi$ the damping ratio. Figs. 6.10, 6.12 and 6.14 show the results of the application of the semi-empirical model for the three case studies: Niterói, Volgograd and Trans-Tokyo Bay, respectively. In the charts on the left, results from dynamic numerical simulation performed in Chapter 5 have also been included. In dashed line, results of the previous semi-empirical model\cite{6} are also included. In the chart on the right, amplitude-wind velocity curves for different damping ratios are displayed. The estimation of the critical wind velocity range of the semi-empirical model for the case of Niterói is very similar to that obtained from dynamic simulations, although the amplitude value is almost twice higher. A deeper analysis of results given by the semi-empirical model for wind velocities higher than 18.0 m/s remains pending, as amplitudes increase with the wind velocity, while the dynamic simulations reflect a stabilisation for a defined wind velocity range, between from 20 m/s to 30 m/s. Also notable is the lower amplitude peak that appears around $U_\infty = 8.0 \text{ m/s}$, which can be related to secondary harmonic resonance frequencies \cite{133} (p.309). This peak is even higher for lower damping values, as shown in the charts on the right.

In the case of the Volgograd Bridge (cf. Fig. 6.12), the estimation of the critical wind velocities coincide with that achieved from the dynamic simulations and the full scale observations, although this is not so for the vibration amplitudes, which seem to be overestimated. Like in the previous case, the progressive growth for velocities immediately higher than the critical wind velocity range does not seem to fit very well with the results obtained from the dynamic simulations. Regarding the damping dependence, the VIV peak is reduced significantly from $\xi = 2.0 \%$

---

\textsuperscript{6}Aerodynamic forces determined through static simulations, $S(\omega)$, and flutter derivatives calculated only for a forced heave of $d_F = 0.1D$. 

231
Figure 6.8: \( H^*_1 \) and \( H^*_4 \) Scanlan’s flutter derivatives of Niterói (N1), Volgograd (V2) and Trans-Tokyo Bay cross sections for different forced amplitudes: \( d_F = 0.1D, 0.2D, 0.3D, 0.4D \) and \( 0.5D \).
Figure 6.9: Niterói bare cross section model: lift force PSD ($S_{F_a}$, top), admittance function (middle) and vertical displacement PSD ($S_y$, bottom). The vertical line defines the actual natural frequency ($f_n = 0.32$ Hz).
In the last case analysed, Trans-Tokyo Bay, critical wind velocity range given by the semi-empirical model is narrower than that corresponding to the dynamic simulations. The VIV peak amplitude at $U_{\infty,\text{crit}} = 15.2$ m/s, in contrast to $U_{\infty,\text{crit}} = 16.8$ m/s from 2D simulations. A minor peak for wind velocities around 8 m/s (two times lower than the critical wind velocity) is also present, probably due to a secondary vortex shedding as a consequence of the interaction process\cite{133}. In this case, as seen in the chart on the right, the peak amplitudes are not so sensitive to the damping variation, as in the case of Volgograd. The VIV effects are removed for $\xi$ values close to 10%, ratios out of the natural damping range for bridges.
Figure 6.11: Volgograd actual cross section (with bars) model: lift force PSD ($S_{F_a}$, top), admittance function (middle) and vertical displacement PSD ($S_y$, bottom). The vertical line defines the actual natural frequency ($f_n = 0.41$ Hz).
Figure 6.12: Comparison of vertical oscillation amplitudes of the Volgograd actual cross section (with bars) of numerical dynamic simulations and semi-empirical model estimations (left). Semi-empirical model estimations of root mean square vertical oscillation amplitudes for various damping ratios (right).

6.3.3 Analysis of results

The results obtained for the case studies through the semi-empirical model can be considered satisfactory, although for some of them the estimated values of the resonance amplitudes are a bit high, as in Niterói and Volgograd. However, the critical wind velocity range is very similar to that given from the dynamic simulations.

The influence of oscillations in the determination of aerodynamic forces, and also in the estimation of Scanlan’s flutter derivatives turn out to be very important for achieving better estimations. When the determination of the aerodynamic force time history used in the semi-empirical model is done through the performance of static numerical simulations, VIV effects are not gathered properly, or they turn out to be very weak, as observed in the different amplitude - wind velocity curves.
Figure 6.13: Trans-Tokyo Bay cross section model: lift force PSD ($S_{Fa}$, top), admittance function (middle) and vertical displacement PSD ($S_y$, bottom). The vertical line defines the actual natural frequency ($f_n = 0.34$ Hz).
On the charts showing different amplitude curves as a function of damping ratios, it is observed that for lower $\xi$, the VIV peak amplitudes do not experience big growths. There is an upper threshold as this is a self-limited phenomenon. Fig. 6.15 shows, in a nondimensional form, the estimation curves from the semi-empirical model of all the case studies, in order to carry out a quantitative comparison. All the cases reach their resonant state for reduced wind velocities around 2.0. However, for Alconétar, with its specific geometry of two rectangular boxes in tandem arrangement, the resonance value is a bit higher, around 2.5. Regarding amplitudes, Volgograd shows a maximum nondimensional value of $y/D = 0.17$. None of these exceed the threshold of $0.2D$ (a representative limit value is considered to be $0.5D$). However, these values are also relative, given that they depend on the mass and the damping factor associated with each model. Fig. 6.16 includes the results of the analysis as a function of the Scruton number, a nondimensional variable which includes these two dynamic properties, as seen in §1.3.3. A double logarithmic scale has been used on the chart. The VIV peak
values obtained for each damping \(^7\) are shown, in addition to the fit curves that corresponds to each case. Lastly, the full scale observations, which serve as comparison of the estimations carried out with the model. Table 6.1 collects critical wind velocities (peak and root mean square), and the associated VIV amplitudes for the two-dimensional dynamic simulations, the semi-empirical model and the full scale observations.

\(^7\)For the study of sensitivity to \(\xi\), the structural mass has been kept as constant for each case study. Therefore, the Scruton number varies only due to the damping factor.
Figure 6.16: Nondimensional root mean square vertical oscillation amplitudes \((y_{\text{rms}}/D)\) from semi-empirical model estimations as a function of Scruton number \((Sc)\) and full scale observations of the study cases.

<table>
<thead>
<tr>
<th>Key</th>
<th>2D simulations</th>
<th>Semi-emp. mod.</th>
<th>Observ.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(U_{\infty,\text{crit}}) (peak/rms)</td>
<td>(y_{\text{peak}})</td>
<td>(y_{\text{rms}})</td>
</tr>
<tr>
<td>A1</td>
<td>14.6/15.0</td>
<td>0.51</td>
<td>0.24</td>
</tr>
<tr>
<td>N1</td>
<td>17.5/17.0</td>
<td>0.98</td>
<td>0.45</td>
</tr>
<tr>
<td>V2</td>
<td>13.6/13.4</td>
<td>0.52</td>
<td>0.41</td>
</tr>
<tr>
<td>T1</td>
<td>17.4/16.8</td>
<td>0.62</td>
<td>0.29</td>
</tr>
</tbody>
</table>

Table 6.1: Summary of results of the case studies of two-dimensional simulations (cf. Chapters 2 and 5), semi-empirical model and full scale observations. Values denoted with (*) refer to \(y_{\text{peak}}\).

6.4 Summary

The development of the new proposed semi-empirical model starts with the compilation of the main analytical expressions and previous models established for VIV estimations, which are basically summarised in the determination of two parameters: critical wind velocity and representative amplitude.

Based on the specific hypotheses and simplifying considerations, a new semi-empirical model is proposed from the general equation of motion formulated for a 1DOF model, where two types of excitation forces are established: aerodynamic
forces and self-excited forces. The first is characterised from the lift force time history, while the second is defined through the use of the flutter derivatives from the Scanlan theory. The equation is solved in the frequency domain, considering the aerodynamic forces as random excitations through a statistical treatment. The model has been applied to the aforementioned case studies, being in good agreement with critical wind velocity range and representative oscillation amplitude estimations. However, whilst wind velocity ranges are quite similar, in some cases estimated amplitudes seem to be a little high in comparison with those acquired from dynamic simulations and the full scale observations of the case studies. Furthermore, lower resonant peaks have been detected for lower wind velocities that could be related to secondary vortex shedding frequencies associated with the interaction process. This requires further analysis which can be carried out in subsequent research works.
Chapter 7

Conclusions and future scope

7.1 Summary. Original contributions of the research work

The aim of this work has been to further the knowledge of the vortex-induced vibrations effects in bridges, in order to offer some practical recommendations intended to avoid their appearance or to minimise their effects. The work has focused solely on the study of the vertical oscillations, i.e. in transverse direction to the oncoming air flow. Structural models have been implemented with their representative cross sections and their corresponding dynamic properties. Below, main conclusions are drafted, highlighting the original contributions achieved through the previous pages.

The vibration episode of the Alconétar Arch Bridge marked the beginning of the investigations carried out on the analysis of the resonant effects of two box sections in tandem arrangement. The studies related to the vortex shedding mechanism for this configuration are rare, and therefore there is a clear gap that needs to be covered. A first series of simulations were implemented, as well as its validation through comparison with full scale observations of VIV vibration episodes. The results obtained were in good correlation in both the estimation of the critical wind velocities and vibration amplitudes. On the other hand, it has not been possible to determine with clarity the role of the braces between the two boxes in the development of the resonant phenomenon, given its three-dimensional nature, which can condition the interaction process as a whole.

After a relevant literature compilation and an exhaustive review of recent publications, it was decided to look further into the identification of the fundamental vortex shedding interaction mechanisms, with the inestimable help of the visualisations performed through numerical simulations. Thus, the existence of different vortex patterns identified in the wind-structure interaction stands out, separating the almost-exclusive, traditional identification made of the VIV phenomenon with the Von Kármán (KV) vortex wake pattern. The other two possible vortex shedding patterns, associated mainly with complex sectional geometries, are the impinging leading-edge vortex (ILEV) and the trailing-edge vortex shedding.
(TEVS), which depend mainly on the aspect ratio of the representative cross section of the structural element.

The parametric study on simple cross sections such as rectangular, H-shaped, and in tandem arrangement through static two-dimensional numerical simulations, has permitted the evaluation of some of the representative parameters for the study of VIV sensitivity: the Strouhal number, $St$, used for the estimation of the critical wind velocities, and the root mean square of the lift coefficient, $C_{L_rms}$, which can be used to estimate the oscillation amplitude. From said values, some recommendations relative to the cross section design have been established to avoid VIV effects. In general, the ILEV and TEVS patterns are characteristic of aspect ratios $W/D$ higher than 2, when the vortex generated and shed from the windward section edge on the interaction process are reattached over the section surface. This gives rise to a wake configuration translated into a more attenuated excitation forces than those related to the Von Kármán vortex street pattern, associated to $W/D$ ratios lower than 2.

The existence of recirculation bubbles as a result of the interaction process in H-shaped sections and their influence in the vortex shedding pattern help to explain the effect of the safety barriers and windshields installed on the bridge decks. Also in the inner gap between twin decks or multi-box sections, the appearance of such recirculation bubbles maintain a stable vortex shedding pattern even for higher $W/D$ aspect ratios.

The parametric study of rectangular sections in tandem arrangement is the most interesting and extensive topic. Strouhal ($St$) and root mean square lift coefficients ($C_{L_{rms}}$) were estimated for different aspect and spread ratios of tandem sections, $B/D$ and $s/D$ respectively. The results show maximum $C_{L_{rms}}$ values, which stand for aerodynamic forces, for tandem geometries that allow an in-phase coalescence between the vortex shed from the upwind section, which is dragged by the wake downstream, and the vortex formed in the downwind section. On the other hand, minimal values are shown for those tandem configurations that lead the vortex coalescence in phase opposition. The upwind and downwind vortex interaction starts to develop for a minimal separation between the two sections in tandem arrangement, which depends on $B/D$ and $s/D$.

The aerodynamic forces become lower for higher $s/D$ ratios, following the shape of the curve that describes the amplitude variation of underdamped systems with time. When the spread ratio is very large, $St$ and $C_{L_{rms}}$ are similar to those of the simple rectangular section with the same $B/D$ ratio. Analysing the case of the Alconétar Bridge, the tandem configuration of the arch cross section is found very close ($s/D = 2.2$) to the spread ratio for which the highest values of aerodynamic forces are reached ($s/D = 2.0$). This undoubtedly contributed to the development of relatively high VIV amplitudes, close to 0.40 m according to the full-scale observations.

For the study of the influence of the braces, aerodynamic tests were carried out in a small facility at the IDR/UPM, Technical University of Madrid. Firstly, H-shaped section models were tested to the scale of 1:240, with various $W/D$ ratios. Initially, the hypothesis was ruled out of the braces in ‘X’ completely blocking the spacing between the both sections in tandem, like a cover plate, which does not allow to the airflow to freely circulate between the upper and lower part of the
cross section. A second series of tests was carried out using models with an aspect ratio of \( W/D = 4.0 \), and solidity ratios from \( \phi = 0.0 \) (no obstacles or braces in the middle gap) to \( \phi = 1.0 \) (H-shaped section), achieved from the use of the plates with grooves of different openings for experimental test performance. The results obtained indicate that the presence of obstacles in the middle gap interferes significantly in the interaction process, progressively reducing the \( C_{L,rms} \) in solidity ratios from 0% to 20%. Then, static root mean square lift force coefficients experiment a progressive smooth increase to end up in the H-shape characteristic value when \( \phi = 1.0 \) is reached, lower than those of equivalent tandem sections (\( \phi = 0.0 \)) with same spread ratio. This indicates a favourable effect of the obstacles in the middle gap for the mitigation or reduction of the vibration amplitudes caused by vortex shedding, determined through the analysis of a representative parameter of the aerodynamic forces. The aerodynamic tests have been completed with numerical simulations for the comparison of results and the implementation of other additional tests for the study of a greater larger number of geometry configurations of rectangular sections in tandem.

The vibration episodes registered in the main spans of the steel girder bridges of Niterói, Volgograd and Trans-Tokyo Bay, make up three good examples for the validation and comparison of the hypotheses established from the previous studies on the interaction mechanism. In the case of Niterói, a comparative study has been performed using the bare cross section and the cross section with traffic in a set of different numerical models. However, in order to correctly reproduce the influence of the traffic on the bridge deck, it is required the use of multi-slice dynamic models, which combine in the same model the presence of typical bare cross section and the cross section with traffic. The estimated vertical oscillation time history fits better to the full scale observations, as well as with the critical wind velocity range. The side barriers in the Volgograd Bridge seem to be crucial for the development of VIV. The presence of a second bridge girder downstream could have avoided the problem, given that the vortex shedding could not have developed in the same way having to parallel bridge decks. Lastly, the numerical simulations carried out keep in good correlation with the aerodynamic studies in the Trans-Tokyo Bay Bridge, which confirms its use as an efficient tool in the identification of the aerodynamic problems.

A sensitivity analysis of the case studies has been carried out through evaluation of the \( St \) and \( C_{L,rms} \), previously mentioned. First, it becomes necessary to identify each cross section deck with the effective width and depth, \( W^* \) and \( D^* \) respectively, which describe the two main dimensions used of the simplified section. The so-called equivalent rectangular sections can be compared with the H-shape sections for the cases of Niterói and Volgograd, also paying attention to the similarity between their vortex shedding patterns and the presence of recirculation bubbles. The case of Trans-Tokyo Bay can resemble an equivalent rectangular section, as well as the case of Volgograd without barriers. The original section of Alconétar corresponds to the values of the two rectangular boxes in tandem, with \( B/D = 0.5 \).

The placement of dampers to minimise the VIV effects is raised as a solution for the three previous case studies, since they are steel cross sections where the installation of deflectors, guide vanes or any other device to modify the shedding regime
of vortex does not turn out to be profitable. Furthermore, it is very important to improve the cross section aerodynamic contour of the different structural bridge elements, such as decks, towers, piers and cables, in order to avoid the VIV problems without needing to carry out subsequent interventions.

Based on the specific hypotheses and simplifying considerations, a new semi-empirical model is proposed from the general equation of motion formulated for a 1DOF model, where two types of excitation actions are established: aerodynamic forces and self-excited forces. The first are characterised from the lift force time history, while the second are defined through the use of the flutter derivatives from the Scanlan theory. The equation is solved in the frequency domain, considering the aerodynamic forces as random excitations through a statistical treatment. The model has been applied to the aforementioned case studies, being in good agreement with estimations of critical wind velocity range and representative oscillation amplitudes. However, in some cases these seem to be a little high in comparison with the results acquired from dynamic simulations and the full scale observations of the case studies.

7.2 Future scope

This research work has managed to give answers to the initial questions regarding the VIV effects on bridges. Furthermore, during its development many other interesting issues and topics have risen, being impossible to address them extensively. However, they are suggested as lines of future investigation derived from the present work, which are included below.

Firstly, one of the points of interest is to implement numerical simulations using turbulence models to determine their influence on VIV effects in the different case studies. It is well known that the presence of a turbulent wind regime interferes in the generation and shedding of vortex, reducing the oscillation amplitudes and decreasing the probability of maintaining a uniform mean wind velocity during the required time period for the development of the resonant phenomenon.

No example of torsional vibrations due to vortex shedding has been included, since all the efforts have been focused on the comprehensive description of the vertical ones. Although this phenomenon is less frequent, it also requires the attention of the researchers, for example in the analysis of its possible role in the development of torsional instabilities relating to flutter for higher wind velocities.

The implementation of numerical simulations widens the parametric study starting with the rectangular, H-shaped and, most of all, rectangular sections in tandem arrangement. Said parametric studies can be extended to sections of other more complex geometries derived from the three previous ones, and even to establish a methodology for the analysis of other aeroelastic effects, such as flutter. Streamlined multi-box sections are a commonly used solution currently as a typical cross section for long-span bridge decks, given that great precaution must be taken to avoid dynamic effects of wind actions.

The influence of obstacles in the intermediate space of the section can also be extended to other tandem configurations. There are different studies with safety
barriers and windshields, but very few focus on a detailed analysis where braces and other elements of union are also taken into account, as well as their spatial distribution.

The new proposed semi-empirical model offered some good initial results for the analysed case studies, although further test with real bridge data and wind tunnels should be addressed for their validation. Also, the comparison with similar models proposed by other researchers remains pending. For some case studies, lower resonant peaks maybe due to a secondary vortex shedding on the interaction process are displayed on the amplitude - wind velocity curves. The cause of these peaks requires a further analysis.

Finally, it is worth pointing out the unquestionable practical use of the numerical simulations for the realization of this research work. The progressive development of the Computational Fluid Mechanics (CFD) codes in recent years allows the visualisation of the interaction process and the estimation of the dynamic wind forces and model response for the comparison of experimental results and full scale observations. There is a wide range of open issues in CFD regarding the development of these tools and numerical models, which will allow further progress in the knowledge of Bridge Aerodynamics.
Appendix A

CFD simulations with $VXflow$

A.1 Introduction

In the present research work three different types of numerical simulations have been used: static, forced and dynamic. The static simulations are the simplest to perform and come normally in first place. The cross section is fixed during the interaction process, and they are used to evaluate the aerodynamic forces acting on the section, which allow to calculate the force coefficients from the integration of pressures around the section contour. The dynamic simulations, also known as fluid-structure interaction (FSI), are used to evaluate the structural response to the wind actions, for which it is also required to define the dynamic properties of the structural model. They also help to identify the critical wind velocities of the aeroelastic instabilities, such as flutter. Lastly, the implementation of forced simulations assigns to the model a harmonic oscillation with a constant amplitude and frequency previously defined. This allows the evaluation of the aerodynamic forces taking into account the model oscillation, which modifies the interaction mechanism. The forced simulations are used, for example, for the determination of the Scanlan flutter derivatives.

The numerical simulations have been implemented through the $VXflow$ package, which includes a solver and various software tools for results post processing. An extensive information is included below.

A.2 The solver

$VXflow$ is a numerical code based on the vortex-particle method (VPM), developed by Prof. Guido Morgenthal as part of his doctoral dissertation (2002) [123]. It allows the implementation of numerical simulations for the study of the interaction between a fluid flow and an object immersed on it, which is the representative cross section geometry of a structural composed of one (two-dimensional) or several (multi-slice) interaction surfaces. The object is rigid, so it cannot be deformed,
thus being the section contour invariable. Depending on the type of simulations, the cross section is fixed when performing static simulations. For dynamic simulations, a 1DOF or 2DOF model is required, defining as well its respective dynamic properties (natural vibration frequency, structural mass, stiffness, damping ratio). Forced simulations are performed to study the influence of oscillations in the acting wind forces. A constant vertical movement (heave) or rotation (pitch) can be set with a certain oscillation frequency for conducting such simulations. In the case of the multi-slice simulations, it is possible to define various vibration modes from their mode shapes and their respective frequencies, the structural masses and the damping ratios.

The user manual of VXflow [124] offers a complete guide to carry out any simulation through the definition of the input parameters. The basic parameters that are necessary to include in the input file are summarised below:

- Section geometry. The section contour is generated from a set of points defined by its Cartesian coordinates (X,Y), defined in anti-clockwise direction. It is possible to have a section formed by several sub-sections. The code automatically creates straight lines or curves between correlative points, which can be divided into a specific number of panels from which the particle generation and release is produced. Said particles entail a vorticity calculated at every time step, from which the representative values of the velocity and pressure in the defined study domain can be obtained\(^1\).

- Fluid flow properties. The fluid is characterised by its density value, \( \rho \), kinematic viscosity, \( \nu \), and free-stream velocity in the two-dimensional plane, \( U_x \) and \( U_y \). In most of the simulations, the flow has only an horizontal component, \( U_x \). The integration of turbulence models in the code is being performed and will be shortly available.

- Interaction domain. The code uses numerical algorithms based on the vortex particle method, which follows a Lagrangian formulation and does not require a mesh for simulation performance. However, it is necessary to establish a non-deformable temporary mesh in the computational domain, which allows one to determine at every time step and for each cell the values of the different requested parameters. In this way, pressure and velocity field visualisations can be generated subsequently. Fig. A.1 shows the recommended domain dimensions as a function of the cross section dimension, \( B \). The domain width is \( 12B \), being the domain height half of the width, \( 6B \).

In the input file, data to be stored in the output files can be selected, in addition to the aerodynamic force time histories. Then, the contour section pressures, the velocity field and vortex streaks visualisations can be obtained by data processing. The time step, \( \Delta t \), is a fixed parameter which depends on the velocity flow, \( U_x \), and the averaged panel length, \( \Delta s \):

\[
\Delta t = \frac{\Delta s}{U_x} \quad (A.1)
\]

\(^1\)For further information, please refer to [123] and [124].
In X-direction one will usually select a grid running from 1...2 chord lengths upstream from the leading edge to about 10...25 chord lengths downstream from the trailing edge depending on the cross sectional shape and the wake features expected. It is then usually sufficient to provide half the X dimension in the Y direction, such that a 2:1 grid ratio is used.

Example:

\[
\text{GRIDMINX} = -10 \\
\text{GRIDMAXX} = 50 \\
\text{GRIDNX} = 511 \\
\text{GRIDMINY} = -14.97 \\
\text{GRIDMAXY} = 14.97 \\
\text{GRIDNY} = 255
\]

It is important to use the vortex plots created from a simulation to check the assumptions above. For example, when bodies oscillate vertically, the wake can be substantially wider than for a static body, possibly requiring a wider grid in the Y direction.

### 4.3.3 Use of the grid

The computational grid is used by the code in two very different numerical schemes, either in the traditional Cell-to-cell algorithm by Leonard or the P3M scheme by the author [7]. The latter is recommended as it enables highly accurate flow modelling at low computational cost. Hence, setting ‘FASTMODE=2’ is recommended. For the accuracy parameter \( N_r \) of the P3M method ‘NR=3’ usually suffices, \( N_r=4 \) provides almost exact results for the velocity but is not needed in engineering applications. Note, that the solution time almost scales with \( N_r^2 \!).

The time step is usually given in nondimensional form as:

\[
\Delta t^* = \frac{\Delta t \cdot U_x}{B}
\]  

(A.2)

It is recommendable to assign a similar panel length to the whole section contour. In the corners or contour parts with details, smaller sizes can be chosen. The choice of the number of panels depends on the overall section dimensions. This parameter is normally conditioned by the average number of particles, \( N_p \), used. For numerical simulations with simple geometry sections, it tends to be enough to have between 100,000 and 200,000 particles. For more complex sections, it is necessary to increase up to 400,000 - 5000,000 particles.

### A.3 Postprocessing

For data postprocessing, the VXflow package has two different tools: VXpost, which includes a set of Matlab scripts, and VXviz, a simulation result visualiser. VXpost is used for chart and graphic generation, calculation of force coefficients and other parameters from the output data. The aerodynamic coefficient time history and their mean values, the Strouhal number, diagram of pressures distribution around the section contour, the oscillation time history (only for dynamic simulations), and the Scanlan flutter derivatives (only for forced simulations), are some of examples. It is also possible to generate visualisations of the velocity field of the selected domain, and to analyse their time progression (instantaneous) or the time-averaged and velocity fluctuation values (a specific time range), as well as the generation and evolution of vortex streaks along the downstream wake. In
the whole interaction process, there is an initial transient phase, which is characterised by the irregular distribution of dynamic pressures on the section, given that the process has not been stabilised, that is, it has not reached a stationary state. Generally, the data processing should be done avoiding the transient phase, which can be identified through the progression of the number of particles, \(N_p\), as can be seen on Fig. A.2. The transient phase is reached around \(t_{\text{sim}} = 20\) s, so in

![Figure A.2: Typical time history of particle count, with number of particles \((N_p)\) as a function of the simulation time \((t_{\text{sim}})\).](image)

data processing this time interval should not be included.

\(\text{VXviz}\) is a powerful visualiser, which is used for the creation of high resolution images and videos from the \(\text{VXflow}\) output files. It has a graphical user interface (GUI) with a wide range of menu options, which allows one to configure different aspects of the visualisations related to the vortex particles (size, colour, opacity), geometry (filled/not filled, pressure contour), field options (opacity, reference wind velocity, colour map), among others. It has interactive zoom options and view settings to choose the visualization angle. It also allows the storage of frames for the later generation of videos in several file extensions. This tool is particularly useful for the visualisation of the multi-slice simulations, which cannot be generated by \(\text{VXpost}\). Fig. A.3 shows a snapshot of a visualisation example with \(\text{VXviz}\).

Both tools allow data processing of the output files in real time, without the need of stopping the runnings.
Figure A.3: Graphical user interface of VXviz postprocessing tool for numerical simulations.
Appendix B

Setup of wind tests

B.1 Introduction

This Appendix provides additional information about the facility used for the implementation of the wind tests with models at a scale of 1:240, whose most important results and conclusions were covered in Chapter 4.

The instruments used, the implementation of the calibration tests in order to obtain the system dynamic properties, and the model manufacturing and mounting employed are described in detail.

B.2 Facility and instruments

Fig. B.1 shows a general diagram of the facility. The model is placed in the central part, mounted on two aluminium plates that reproduce an elastic 1DOF system, which is used for the H-shape section wind tests. The model geometry is defined by the section with, $W$, section depth, $D$ and its length, $L$. The section depth is $D = 0.01$ m, and the length, $L = 0.30$ m, for all the models.

It is assumed that there are no three-dimensional effects and tests are carried out in laminar airflow regime. To get this, two square plates of 8 cm x 8 cm are placed at both edges of the model (cf. Naudascher & Rockwell [133] p.128).

The centrifugal fan (Fig. B.2, top left) produces an airflow which is directed towards the settling chamber (Fig. B.2, bottom left), where it passes through various honeycomb meshes to straighten out and reduce its level of turbulence. Then, it is projected outside by the outlet mouth, where the test model is placed (Fig. B.2, right).

In order to regulate the flow velocity, a wooden trapdoor in the side part of the fan (Fig. B.2, top left) and a potentiometer (cf. Fig. B.3, right) were used. The airflow horizontal velocity is measured through a hot-wire anemometer placed in the lower part of the model (Fig. B.2, right). The velocity values are controlled on real time by a data reader connected to the anemometer (Fig. B.3, left).

The model vertical oscillations are measured with a laser vibrometer (Fig. B.2, right).
Figure B.1: Diagram of the wind facility with all the components and the measurement systems.

Figure B.2: Fan rotor for airflow generation (top left), lateral view of the settling chamber containing a honeycomb-shaped mesh (bottom left), and view of the H-shape section model assembled in the facility with the hot-wire anemometer and the laser vibrometer (right).
right), which registers the distance variations of a small steel plate attached to one edge of the model. The laser interface (Fig. B.4, top left) records the laser electric pulses and converts them into distance values from according to a calibration ratio (190,000 mV ≡ 1.00 m), previously fixed from the laser power supply (Fig. B.4, top centre). The analogical laser registers are converted into digital ones through a data logger (Fig. B.4, top right), and sent to a PC for data storing and analysis. The laser registers data at a frequency of 100 Hz, that is, 100 values per second, which makes necessary a previous data filtering for the elimination of noise. Alternatively, an oscillometer can be connected (Fig. B.4, bottom) which allows for visualisation of oscillations in real time, to better identify the wind velocity range where the VIV phenomenon occurs.

B.3 Calibration tests: frequency and damping

As seen in Chapter 4, all the models have a section depth of $D_m = 0.01$ m. They are made at a scale of 1:240, taking as a reference the Alconétar cross section dimensions, with a representative cross section depth of $D = 2.40$ m. The length scale factor is, therefore, $\lambda_l = 1/240$. The model oscillation frequency is determined according to the plate-supported system, and they vary substantially between models. The frequency scale is determined from an average value of $f_n = 4.75$ Hz (taking into consideration all the model tested), and the fundamental
vertical frequency of Alconétar, $f_n = 0.70$ Hz, thus being the scale factor $\lambda_{f_n} = 4.75 / 0.70 = 6.8/1$. Lastly, the velocity scale is determined by the two previous ones, $\lambda_{U_\infty} = \lambda_l \cdot \lambda_{f_n} = 1/35$. The damping factor scale is $\lambda_\xi = 1.0$, as this is a nondimensional parameter. To establish equivalence ratios, please refer to [119] and [13].

The calibration tests have the aim of determining the parameters $K$, $f_n$, and $\xi$, necessary for test performance. Below, the particularities of every set of wind tests are presented, as well as the calibration parameters from the different calibration tests performed.

## B.3.1 H-shape models

H-shape section models have been manufactured by hand with low-density wood at the IDR/UPM. Fig. B.5 shows one of the models placed on the elastic support
Figure B.5: View of the model ready for identification tests with the weight of several nuts quickly removed to initiate free vibration.

during a calibration test performance.

The dynamic properties of the system are the stiffness, $K$, the damping factor, $\xi$, and the mass of the system, $M_{\text{system}}$, which is defined as

$$M_{\text{system}} = \frac{K}{(2\pi f_n)^2}$$  \hspace{1cm} (B.1)

where $f_n$ is the system natural vibration frequency, determined by the mechanical properties of the system supports (aluminium plates or springs).

The mass $M$ of the model contributes to the representative mass of the system, $M_{\text{system}}$, which is calculated from the system stiffness, $K$.

The stiffness is determined from a deflection test, which consists on placing a certain number of nuts of a known weight on the model and removing it quickly. The deflection is measured with the laser, taking its initial position as reference. Table B.1 includes the registered values from the calibration tests for the $W/D = 4.0$ model, where $w$ is the weight placed for every test case, $y_0$, the deflection or vertical displacement due to the weight removal.
value of the elastic system is $K = 91.96 \text{ N/m}$, associated with the deflection caused by the weight of one nut, given that the VIV oscillation amplitudes for the wind tests do not exceed this threshold value.

The determination of the natural frequency and the damping factor is done from the free oscillation registers of the excited model after the weight removal (a nut or a set of nuts, cf. Fig. B.5). Fig. B.7 shows the oscillation time history, typical of a subcritically damped system (left), and its frequency spectrum (right), with a peak in $f_n = 4.76 \text{ Hz}$. The damping factor, like the stiffness, is calculated for various different weights that originate the initial excitation of the model, and its

\footnote{Laser calibration: 190,000 mV $\equiv 1.00 \text{ m}$}
Figure B.7: Characterisation of the natural frequency \((f_n)\) and damping factor \((\xi)\) of a free vibration damped, 1DOF system: vertical vibration time history (left) and PSD (right). Identification test of the H-shape section model of W/D \(= 4.0\).

The corresponding fit curve is displayed (Fig. B.6, right). In this case, the representative value of the time history obtained after the quick weight removal from the model is also considered, \(\xi = 1.31 \times 10^{-3}\), as pointed out previously. The dispersion of the damping values is higher than in the case of the stiffness collected data.

### B.3.2 Models with different solidity ratios

The models for solidity ratio wind tests have been manufactured with resin through 3D printing, from CAD models previously defined. This technique offers a higher level of precision to achieve the accurate solidity ratios for every model used. Fig. B.8 shows the manufactured models before removing the auxiliary material (left) employed during fabrication, and one of the models ready to start with test performance (right). Fig. B.9 shows three different views of one of the models ready to start the calibration tests. The plates with different solidity ratios can be seen on the test table (top left). A bubble level is used to check that the section is placed in a perfectly horizontal position regarding the oncoming airflow.

Table B.2 shows the resulting values of the calibration tests for the model W/D \(= 4.0\) and \(\psi = 0.60\), which are completed with the fit curves displayed in Fig. B.10.
App. B. Setup of wind tests

Vortex-induced vibrations on bridges

Figure B.8: Models for solidity wind tests manufactured by a 3D printer before removal of auxiliary material (left). Assembled solidity model ($\psi = 0.20$) with supporting springs before being placed on the rigid frame (right).

Figure B.9: Upper view of the spring-supported model in the facility for solidity wind test performance. On the table, various models of different solidity ratios to be tested (top left). View of the model ready for identification tests with a small weight quickly removed from the model section to initiate free vibration (bottom, left). Mounting and check with a bubble level (test model of $\psi = 0.10$) before test performance (right).
Table B.2: Summary of characterisation parameters from identification test of H-shape section model of $W/D = 4.0$ and $\psi = 0.60$.

Also on this occasion it has been decided to take as $K$ and $\xi$ parameters those associated with the first calibration point (deflection corresponding to the weight of a nut quickly removed to get the model oscillations), because amplitudes of model during wind tests performance do not exceed the ones got from one nut removal.

Figure B.10: Characterisation of the natural frequency ($f_n$) and damping factor ($\xi$) of a free vibration damped, 1DOF system: vertical vibration time history (left) and PSD (right). Identification test of the solidity model of $W/D = 4.0$ and $\psi = 0.60$.

---

2Laser calibration: 190,000mV $\equiv$ 1.00m
Figure B.11: Fit curve of the stiffness ($K$) and damping factor ($\xi$) from various identification test at different initial amplitudes ($y_0$) of the solidity model with $\psi = 0.60$. 
Appendix C

Shark and car CFD modelling

C.1 Introduction

As a result of the author’s curiosity, this Appendix includes some results and CFD visualisations related to other cross section shapes different to bridge representative cross sections, independent of the main research topic dealt with throughout the previous pages. Firstly, a simple analysis of the interaction process produced by a shark moving forward within the sea is shown. Subsequently, a study of the aerodynamic coefficients of three sport cars of equivalent features are shown and results are compared with the corresponding aerodynamic coefficients given by the manufacturers. Although initially the topics presented seem to be part of disciplines distant from Bridge Aerodynamics, engineers should be open to any idea from other fields of knowledge which could help to enhance design and product optimization, in this case the geometry and configuration of the structural elements of a bridge.

C.2 Shark hydrodynamics

In nature, there are fish and marine animals of very different shapes, sizes and colours. Their morphological characteristics are the result of a long process of adaptation to the hostile environment, which has eliminated those individuals who find themselves at a competitive disadvantage, in accordance with the theory of natural selection. Fig. C.1 shows a collection of images of different fish, which gives an idea of the existing extensive variety. One of the characteristics that represents a clear advantage for these animals is their motion velocity within the water, which allows them a greater disposition to search for food in a larger area or flee from possible enemies. As an practical example, the shortfin mako shark has been chosen, as this is the fastest animal in the aquatic environment, which reaches motion velocities close to 60 km/h \( (U_{\infty} \sim 17 \text{ m/s}) \). An adult specimen measures between 3.5 m and 4.0 m in length, and can weigh as much as 750 kg. Fig. C.2
Figure C.1: Collection of fish samples showing distinct body profiles (Source: http://vintageprintable.swivelchairmedia.com).

shows the basic geometry profile of the shark to be considered in the numerical simulation. The kinematic viscosity of the seawater is $\nu (10^\circ \text{C}) = 1.31 \cdot 10^{-6}$ and the representative cross section dimensions, $D = 0.85$ m. The Reynolds number is:

$$Re = \frac{U_{\infty} D}{\nu} = 1.1 \cdot 10^7$$ (C.1)

which is in the same order of magnitude than those of bridge study cases analysed in this research work ($Re \sim 10^6 - 10^7$).

Fig. C.3 shows the force coefficient time history corresponding to a static simulation, and Table C.1 gives the geometry features of the cross section profile and the mean force coefficients.
Figure C.2: External body contour (highlighted in white) of a shortfin mako shark as the basic geometry profile for numerical simulation performance (picture courtesy of Patrick Doll).

Figure C.3: Force coefficients of static two-dimensional simulation of a shortfin mako shark under the seawater moving forward at a velocity of $U_\infty = 17.0$ m/s (detailed view of 5-second time period).
The high Strouhal number, \( St = 0.23 \), indicates that the profile presents a good hydrodynamic behaviour, which is also reflected in the regularity of the force coefficients. As can be observed, the profile studied bears some resemblance to the aerodynamic profiles of the wings of airplanes, if the tail fin is not taken into account.

Lastly, in Fig. C.4 a series of visualisations obtained from static two-dimensional simulations is included\(^1\). The static profile and the fluid moving in horizontal direction with a free-stream velocity, \( U_\infty \), from left to right, has been considered. The instantaneous velocity field and vortex particle streaks (top) show how the separation of the boundary layer occurs in the rear part, thus avoiding excessive turbulent effects and minimising the friction forces in the interaction. Also seen is the regularity of the vortex shed, which continue their progression in the wake downstream. The time-averaged velocity field (middle) made once the stationary state is reached shows once again the streamlined section contour of the shark profile, because of its soft alteration of the wake. Time standard deviation of the velocity field (bottom) show very low velocity fluctuation values. Mean velocity peaks, as well as the highest fluctuations are reached in the tail fin edge.

The two-dimensional modelling is a first approximation to the study of the interaction mechanism of the shark immersed into the water, given that a good analysis can not leave out the importance of the three-dimensional effects, given the shark as a complex body, not comparable with a linear structure like a long-span bridge. In addition to this, it is necessary to consider the influence of the individual elements, such as the different fins. The tail fin could not have been considered either in the numerical models carried out, given that it is not a ‘rigid’ element of the animal, as it is continuously moving in order to keep its body balanced during the motion. However, additional simulations made without taking into account this fin produce similar results, although this requires more detailed analyses that are not the object of this Appendix.

\(^1\)An image of the mako shark has been superimposed over two of the visualisations (middle and bottom) where the dorsal and pelvic fins are included. However, in the profile considered in the simulations only the tail fin has been taken into account, as seen in the visualisation on top.
Figure C.4: Visualisations from static simulation of the original mako shark profile \( (U_{\infty, \text{crit}} = 17.0 \, \text{m/s}) \): instantaneous velocity field and vortex particle streaks (top), time-averaged velocity field (middle) and time standard deviation of the velocity field (bottom). Mako shark profile image courtesy of D. Rome Peebles.
C.3 Automotive aerodynamics

There is no doubt about the importance of the improvement of the aerodynamic properties in the evolution of designs in the automobile industry. Fig. C.5 shows a image collection that provides a quick review with some of the most representative models and prototypes since the appearance of the first cars. The main characteristic of all these is the use of streamlined profiles to minimise the friction forces due to the wind actions at high motion velocities, although at times the final design is taken also regarding aesthetic reasons.

The wind tunnel tests are used by car manufacturers to optimise the aerodynamic profile of new models, which are usually made at a real scale dimensions. The aerodynamic coefficient $C_d$ is the parameter generally used to measure the aerodynamic resistance. It is defined as:

$$C_d = \frac{F_D}{\frac{1}{2} \rho U_\infty^2 D} \quad (C.2)$$

where $F_D$ is the drag force, $\rho$, the air density, $U_\infty$, the free-stream wind velocity, and $D$, the car’s vertical width.

Currently, there is a certain rivalry between the big automobile manufacturers to design their new car models with the minimum $C_d$, given that it translates into higher velocities with lower fuel consumption, a factor of great importance for
velocities from 100 km/h upwards.

In recent years, like in other fields such as Bridge Aerodynamics and Aerospace Engineering, the automobile industry has resorted to CFD tools to carry out numerical simulations that allow one to compare the results obtained in the experimental tests and make a pre-design of the prototypes more rapidly and economically. As a practical application example, the VXflow software is used for the numerical analysis of three models of similar sport cars from different brands: Mercedes SLK (MSLK), Porsche 911 Turbo (P911T), and BMW Z4 Coupé (BMWZ4).

The aim is to determine their aerodynamic coefficients from numerical simulations taking a representative profile of the three previous models. Unlike long-span bridges, which can be considered linear elements in the analysis of the wind-structure interaction under certain simplifications, the automobiles are typically three-dimensional elements with lots of details that should be considered in further design stages. However, with these simple cross section profiles it is possible to achieve very acceptable results in the overall estimation of the aerodynamic coefficients. This is useful to give an idea of the suitability of the chosen profile, although of course other types of simulations are required for the car’s detailing (rearview mirrors, antennas, windshield wipers, etc.).

Fig. C.6 shows the drag coefficient time history of the tested models, using a uniform wind velocity $U_\infty = 33.33$ m/s ($\sim$120 km/h). Figs. C.7 and C.8 show the time-averaged and the time standard deviation of the velocity field of the two-dimensional static simulations of the tested models, respectively. The visualisations are quite similar, and it is difficult to highlight big differences between the three models analysed.

Table C.2 shows a summary of the main parameters, where $W$ is the overall length of the chosen profiles, and $D$, the vertical width. $\bar{C}_D$ stands for the drag coefficient mean value obtained from the numerical simulations performed, and $C_d$ is the aerodynamic coefficient given by the manufacturers. $C_d \cdot A$ is the drag area, a parameter also given which is normally used to express the aerodynamic resistance of the models, where $A$ is the car’s cross section frontal area, perpendicular to the direction of advance. The results obtained through the simulations are very close to the values given by the manufacturers, being the P911T model the one which presents bigger differences regarding the theoretical value achieved from the wind tunnel tests.

<table>
<thead>
<tr>
<th>Model</th>
<th>$W$ [m]</th>
<th>$D$ [m]</th>
<th>$\bar{C}_D$ [-]</th>
<th>$C_d$ [-]</th>
<th>$\epsilon$ [%]</th>
<th>$A$ [m²]</th>
<th>$C_d \cdot A$ [m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>P911T</td>
<td>4.52</td>
<td>1.18</td>
<td>0.36</td>
<td>0.31</td>
<td>16.1</td>
<td>2.04</td>
<td>0.61</td>
</tr>
<tr>
<td>MSLK</td>
<td>4.08</td>
<td>1.16</td>
<td>0.31</td>
<td>0.32</td>
<td>3.12</td>
<td>1.93</td>
<td>0.62</td>
</tr>
<tr>
<td>BMWZ4</td>
<td>4.23</td>
<td>1.15</td>
<td>0.35</td>
<td>0.34</td>
<td>2.94</td>
<td>1.96</td>
<td>0.67</td>
</tr>
</tbody>
</table>

Table C.2: Summary of two-dimensional static simulation results of tested models.
Bibliography


275


278


Bibliography


282
Bibliography


283
Bibliography


[122] *Instrucción de acero estructural (EAE)*. Ministerio de Fomento (Spain), May 2011.


Bibliography


286


