Universidad Politécnica de Madrid

Escuela Técnica Superior de Ingeniería
Aeronáutica y del Espacio

Fluid-structure interaction in long-span bridges

Doctoral Thesis

Mikel Ogueta Gutiérrez

December 16, 2016

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A Pepe

Anyway the wind blows

Queen, Bohemian Rhapsody

Abstract

Bridge aerodynamics has attracted the interest of wind engineers for a long time. This interest rose after the Tacoma Narrows disaster, when a four-month-old bridge collapsed under a wind condition of only 20 m/s. Several dynamic effects generated by wind in bridges were described and studied, namely, vortex induced vibration, flutter, galloping and buffeting.

In the second half of the twentieth century, improvements in construction techniques lead to longer and lighter bridges. Due to this, natural frequencies of bridges dropped to frequencies around a few Hertz. Dynamic effects were more likely to appear in these bridges.

Many effort has been done to understand better all the effects that wind can create on a bridge. However, some relevant studies have been left aside. One of the main parameters that play a significant role is the barriers. This thesis tries to give a deeper insight on the influence of the barriers in the different aspects.

Even though the aerodynamics of any bridge sections is conditioned by many other aspects than just the barriers, such as the bridge section, mass, natural frequencies, etc., some ideas are extracted to design barriers so that they increase the stability of the bridge.

Several different barriers have been tested over different bridge sections. In the literature, porosity is pointed out as one of the most relevant design parameters. Throughout the thesis barriers with different porosities have been tested on different bridge sections. For the tests, three different wind tunnels have been used, two in the Instituto Universitario de Microgravedad Ignacio da Riva, in the Universidad Politécnica de Madrid and another in the Centro Interuniversitario di Aerodinamica delle Costruzioni e Ingegneria del Vento, in the Universit degli Studi di Firenze.

As the influence of the barriers is not only important in the aeroelastic behavior of the bridge, in some of the configurations other tests have been performed, such as static loads generated on the bridge or on the users of the bridge (in this case, on a train), or flow condition created in the deck by the shear layer shed by the upper part of the barrier.

This thesis is divided in 4 chapters. In the first chapter an introduction to fluid-bridge interaction is presented and the state of the art in the topic is summarized and the objectives of the thesis are described. In chapter 2, the results of a set of tests on a high speed train bridge are displayed and analyzed. In chapter 3 the configuration of a flat plate with barriers is studied. Finally, in chapter 4 the main conclusions of the thesis are drawn, and proposes future work to improve the current knowledge about the influence of the barriers in the stability is proposed.

Resumen

La aerodinámica de puentes ha atraído el interés de la comunidad científica relacionada con la aerodinámica durante mucho tiempo. Este interés aumentó tras el desastre del puente sobre el río Tacoma, cuando un puente inaugurado únicamente 4 meses antes se destruyó bajo un viento sostenido de tan solo 20 m/s. Varios efectos dinámicos generados por el viento en puentes fueron descritos y estudiados tras el evento, por ejemplo, vibración inducida por torbellinos, flameo, galope y bataneo.

En la segunda mitad del siglo veinte, las mejoras en las técnicas de construcción llevaron a puentes más flexibles y ligeros. Debido a esto, las frecuencias naturales de los puentes descendieron hasta valores cercanos a unos pocos Herzios. La probabilidad de aparición de efectos dinámicos en los puentes aumentó por este hecho.

Se ha realizado un gran esfuerzo en entender mejor todos los fenómenos que el viento puede crear sobre un puente. De todos modos, hay ciertos aspectos que no han sido tenidos en cuenta. Uno de los objetos que influye de mayor manera en la aerodinámica de los puentes son las barreras. Esta tesis trata de arrojar más luz sobre la influencia de las barreras en diferentes aspectos de la aerodinámica del puente.

Pese a que las características aerodinámicas de cualquier sección de puente están condicionadas por otros aspectos además de las barreras, como por ejemplo la sección del puente, la masa, las frecuencias naturales, etc., se extraen algunas ideas para diseñar barreras de manera que aumenten la estabilidad del puente.

Se han ensayado diferentes barreras colocadas sobre diferentes secciones. En la literatura, se señala la porosidad como uno de los principales parámetros de diseño. A lo largo de la tesis, se han ensayado diferentes barreras de diferente nivel de porosidad. En los ensayos, se han empleado tres túneles aerodinámicos diferentes, dos en el Instituto Universitario de Microgravedad Ignacio da Riva, en la Universidad Politécnica de Madrid y el otro en el Centro Interuniversitario di Aerodinamica delle Costruzioni e Ingegneria del Vento, de la Universit degli Studi di Firenze.

Debido a que las barreras influyen no solo en la estabilidad aeroelástica del puente, sino también en otros aspectos importantes, en algunas de las configuraciones ensayadas se han realizado otros ensayos, como por ejemplo, ensayos estáticos sobre el puente o sobre los usuarios del mismo (en este caso, sobre un tren), o las condiciones del flujo creadas en el tablero por la capa de cortadura desprendida de la parte superior de las barreras.

Esta tesis está dividida en 4 capítulos. En el primer capítulo, se expone una introducción a la interacción entre fluido y estructura, así como un resumen del estado del arte. También se enumeran

los objetivos perseguidos con la tesis. En el capítulo 2, se presentan y analizan los resultados de un conjunto de ensayos realizados sobre un puente de trenes de alta velocidad. En el capítulo 3, se estudia una configuración de una placa plana con diferentes barreras. Finalmente, en el capítulo 4 se extraen las principales conclusiones de la tesis, y se proponen líneas de investigación con el objetivo de mejorar el conocimiento actual sobre la influencia de las barreras en la estabilidad de los puentes.

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Agradecimientos

Me gustaría agradecer a muchas personas su apoyo y esfuerzo, no siempre vinculado al desarrollo de la tesis. Usando un símil montañoso, todas estas personas han estado en mi cordada, compartiendo las cumbres y también las partes más duras del camino.

En primer lugar, ojalá pudiese agradecer a Pepe todo lo que aprendí de él. Este trabajo no se podría haber llevado a cabo sin la estructura que él creó y desarrolló. Quiero dejar constancia mi agradecimiento a todo el personal del Instituto que, de una forma u otra, ha hecho este proceso sea un poco más sencillo. Me gustaría agradecer especialmente al director de esta tesis, Sebastián y al director del IDR, Ángel, su apoyo, dedicación. Una mención especial para la gente de Montegancedo con la que más y mejor tiempo he pasado, Carlos Pascual, Carlos De Manuel, David, Dulce, Ghazaleh, Ignacio, Javier, Lalo y Manolo. Ha sido un placer compartir carreras, bicis, cumbres y otras experiencias con vosotros.

¡Cómo no pensar en mi familia! Mis padres, Coro y Jesús Mari, han sido mi gran referencia durante más de 30 años. Gracias por todo (que es mucho). A Maite, porque juntos formamos la pareja de hermanos más cabezotas insistentes. Su tesón, aunque a veces me resulte difícil de comprender, me ayuda a esforzarme cuando las ganas me flaquean, que es más a menudo de lo que me gustaría. Ya somos dos Doctores Ogueta. Y, por supuesto, al último “fichaje” de la familia, Luis, porque ante todo, siempre espero conservar su amistad. También el resto de la familia: ti@s, prim@s, etc, han sido un gran apoyo y una buena vía de desconexión. Quiero agradecer a mi abuela Beatriz que, aunque le cueste descolgar el teléfono y llamar, sé que siempre nos tiene en mente.

A mis amigos, ya sean hondarribitarras (por todas las horas que hemos matado juntos), ISFeros (por todas las charlas y discusiones que hemos tenido y que tendremos) o RUGPianos (porque me ayudasteis a ubicarme en Madrid y a abrir mi mente). Me habéis acompañado durante diferentes etapas de mi vida. Puede que en ocasiones no sea la persona más sencilla de tratar, por todas esas ocasiones en las que no me tirasteis lo que teníais a mano a la cabeza, gracias.

Gracias al programa de internacionalización de doctorandos de la UPM, pude disfrutar de 4 meses intensos y provechosos en el Centro Interuniversitario di Aerodinamica delle Costruzioni e Ingegneria del Vento, CRIACIV. Aprendí mucho aquel tiempo y me gustaría agradecer a Luca, Claudio, Gianni, Tomasso, Ninni y Andrea su paciencia conmigo y con mi italiano.

Miguel, hace más de 5 años que no vivimos ni siquiera en el mismo país, pero tu amistad se siente desde la distancia. Gracias por tanto.

En Madrid, he podido disfrutar de una nueva “familia política”. Claudia, Manolo, Marisol, Rubén, Yuste, Raúl, Isa, Ely, me han acogido en todo momento. Gracias.

Si alguien cree merecer ser agradecido y no se siente representado, esto se puede arreglar a continuación. Gracias,

Esto hubiera sido muy diferente sin Sara, mi compañera. Gracias. Sigamos avanzando por Jorge, nuestro futuro común.

1 Introduction

All natural phenomena condition somehow human life on earth. One of the main natural phenomena humans have to face is wind. It has influenced many aspects not only of human life but also of animal and plant evolution. Flying animals or anemochorous plants (plants and trees that depend on the wind for the spreading of their seeds) are clear examples of the close relationship between wind and living matter.

First humans on earth already understood its importance. They worshiped natural events that were not under their control. Mesopotamian and Egyptian cultures already had their wind gods, and the Greek god for the wind, Aeolus, influenced many languages which include still nowadays his name in wind-related words, such as eolian in English or eólico in Spanish.

At first, when human activities were reduced to farming and hunting, the influence was limited. Later, human activities became more complex, and when building a shelter, less windy places were searched. At some point, humans managed to make some benefit from wind. Clear examples for this is the use of sails for navigation and the development of windmills.

Following the settlement of humans in permanent areas, the wind loads on buildings and the flow perception for pedestrians became the most important effects of wind in human life. With the industrial revolution, the improvement of construction techniques and materials lead to bigger and lighter structures. This progress made the new structures more sensitive to wind, specially due to the dynamic loads.

All the previously mentioned wind-sensitive constructions can be considered as bluff bodies. By the beginning of the twentieth century, aerodynamics of streamlined bodies had already been studied by people like Reynolds and Lilienthal, and this lead to the first flight of the Wright brothers, which took place in Kitty Hawk, North Carolina, in December, 1903. But aerodynamics of bluff bodies had not attracted so many the attention from researchers.

A definition of a bluff body is given in [1]. A body can be considered a bluff body if the characteristic length in the flow direction is close or equal to the perpendicular characteristic length. This leads to the most important characteristic of a bluff body which is that, when it is submerged into a flow, the flow is not fully attached to all the surface, and eventually a wake is created beyond the detachment point. In the detachment point, structures of high vorticity (vortexes) are created. These vortexes are low pressure areas that create suction in the surface of the body they have contact with. Irwin [2] underlined the importance that vortexes have in the aerodynamics of a bluff body. In that paper, he made a brief review on some of the effects that wind can produce over bluff bodies. Larose and D'Auteuil cite in [3] another important characteristic of a bluff body, stated by Farquharson [4]. He

defined a bluff body as a body in which “the total drag consists almost exclusively of pressure drag”, in comparison to streamlined bodies, for which the higher contribution of the drag is the friction drag. A similar definition for a bluff body can be found in [5].

The definition includes buildings and structures with sharp edges on their design. In fact, excluding some very particular structures such as airplane wings or helicopter blades, most man-made constructions behave as bluff bodies under the flow of atmospheric wind. Therefore, during the twentieth and twenty-first centuries, bluff body aerodynamics has become a major concern for architects and engineers.

Skyscrapers are the first type of building that comes to mind when talking about influence of wind in bluff bodies. When designing a skyscraper, wind tunnel experiments are generally performed in order to ensure its safety within an acceptable limit. Irwin [2] mentions 3 examples of skyscrapers. The first one, Taipei 101, suffered some vibrations during the wind tunnel tests, so its shape was slightly modified. This reduced the static loads, but was not enough to vanish the dynamic effects that the wind was producing, so a damper was installed. The Petronas twin-towers did not experience any remarkable effect, but the spires in their tops and the bridge connecting the towers did, so dampers were needed again. Finally, world’s tallest building so far, the Burj-Khalifa, did not suffer any instability, due to its cross section, that was designed to change with the height, in order to avoid undesirable wind effects.

Other structures prone to suffer wind effects are solar energy systems, such as heliostats, photovoltaic collectors or parabolic troughs. These facilities, usually installed in open terrains, have not the protection of near buildings, so wind loads on them can be high. Research on these are extensive since the growth of renewable energy plants has been a constant in late years [6, 7, 8, 9]. Because of the huge number of collectors present in each site, the accurate estimation of the wind loads is key. If these loads are well estimated, structural components of the collectors will be optimized. This way, a reduction of the overall cost of the plant can be achieved. It is important to underline that the cost savings can be remarkable due to the number of collectors present.

In addition to skyscrapers and solar energy facilities, long span bridges are also wind-sensitive structures. Usually designed as bluff bodies, they are often exposed to homogeneous laminar flow. For centuries, bridges did not span further than some meters, but since 1950s their length has been constantly increasing: In the fifties, the longest cable-stayed bridge was 81 meters long, whilst by the end of the century, the Tataru bridge had multiplied this length by eleven (890 meters) [10]. Also suspension bridges have reached a span hardly believable at the beginning of the twentieth century, with the Akashi-Kaikyo main span reaching 1991 meters [11]. It is important to remark that before 1931, the longest-span suspension bridge spanned circa 500 m. That year, the George Washington Bridge was opened and it was almost twice this length (more than 1000 m) [12]. Plans for even

longer bridges are still under study, such as the proposed Messina Strait bridge, which could reach a span of 3300 m [13, 14].

The field of study of bluff body aerodynamics is not limited to the wind loads created on the bodies, but ranges from the structure of the wind in the atmosphere to the dynamic response of the buildings. The wideness of this branch of aerodynamics can be seen in several books [15, 16]. They cover the generation and nature of atmospheric boundary layer, the static loads of wind in structures, the dynamic response that can be created, the discomfort that different wind conditions can produce in humans, the wind tunnel procedures to study the effects of wind, the case of particular structures, and more. In addition to congress sections devoted to this field, there is an international conference fully focused to this topic every 4 years, the Bluff Body Aerodynamics and Applications Conference.

The loads that wind produces on bluff bodies can be classified [15] in static and dynamic loads. The difference between them is that in the case of dynamic loads, the shedding of vortexes leads to an oscillatory load on the bridge deck, and therefore, to time-dependent forces [17]. This difference is the reason why their study has to be approached differently.

Costa [18] considers 4 components of wind in its interaction with the structures, namely the steady component depending on the mean wind velocity, the buffeting component due to the turbulent wind fraction, the vortex shedding component and the self-excited component, depending on the structural motion. The first one is the contribution to the static effects, whilst the other three are the contribution to the dynamic effects.

1.1 Static effects of wind on structures

It has already been explained that the presence of a body modifies significantly the flow field, creating detachment areas. The detachment of the flow leads to areas where the flow is accelerated and others where the flow is decelerated. The acceleration/deceleration of the flow creates low pressures/high pressures, respectively. The integration of these pressures along the surface of the body results in an overall wind load.

In the 1960s, Davenport started working on wind loads on structures. In [19], he reviewed the historical evolution of static loads. At first, just the steady wind was considered, but later gust corrections were also introduced, since it was seen that variable winds could lead to higher loads. With the aim of predicting the probability of failure by wind in a certain period, he observed that in wind tunnel tests it was key to reproduce correctly the geometry, the Reynolds number and the kinetic characteristics of the flow.

A deep study in the drag of a circular cylinder has been performed by Roshko [5, 20]. According

to him, the current description of the force generation in bluff bodies is merely empirical. He also pointed the difficulty of predicting the drag of a bluff body.

One of the main characteristics of bluff bodies is that the flow detaches at the edges, independently of the Reynolds number, provided it is beyond a critical value. This assumption is needed when scaled wind tunnel tests are performed, since usually the Reynolds number is several times smaller in the wind tunnel tests than it is in the reality (Scanlan talks about a factor of 1000 or more [17]). This fact has been recently questioned. Schewe and Larsen [21] found differences on the Strouhal number in the bridge and the one measured in the wind tunnel, differences they attributed to the Reynolds number difference (even though they managed 3 more reasons. In [22], Schewe also reported tests in other bluff bodies (Storebælt bridge, circular cylinder, airfoil at a big angle of attack), concluding that the effect exists. Also Larose and D'Auteuil [3] studied its influence on several bridges (Storebælt, Ikara, a general streamlined section tested at IHI wind tunnel, Stonecutters) and on a low rise building. They found the influence not to be negligible, specially when the edges are chamfered or filleted. In the case of Stonecutters, they found that the guide vanes installed to reduce vibration (see section 1.4) worked better or worse depending on the Reynolds number. Also Matsuda studied the influence of the Reynolds number effect in the aerodynamics of a bluff body. Pressure measurement and force coefficient variations were detected in those tests with the variation of the Reynolds number. Despite all, the bad scaling in the Reynolds number leads to a conservative result [23].

1.2 Dynamic effects of wind on structures

Regarding the dynamic loads, the behavior of the flow becomes much more complex. Aeroelasticity is the branch of the aerodynamics that studies the dynamic behavior of objects submerged in an air flow. The problem arises when aeroelastic instabilities appear. The instabilities appear when the flow creates a deformation in the structure, and this deformation leads to a change in the flow configuration, which tends to increase the deformation. Instabilities are potentially dangerous, since they can be unbounded and lead to the destruction of the structure.

Some highlights about the dynamic effects on different elements were studied in [24]. In that paper, Scanlan reviewed the origins of the aeroelasticity (which studies the elastic behavior of elements under wind effects, not only bluff bodies but also streamlined objects such as wings). Regarding the aeroelasticity of bluff bodies, it is widely accepted that the event that focused the interest of researchers and engineers in bluff body aeroelasticity was the Tacoma Narrows disaster, even though previously other bridges had suffered other aeroelastic effects, such as the Menai Strait bridge [25].

It was the third longest suspension bridge when it was opened to public service in July 1940. It

suffered vibrating events early, and earned the nickname of Galloping Gertie. After the first events of vibration, Farquharson started a campaign of tests to understand the reason of the vibration, but never thought it could lead to the collapse of the bridge [26]. His research highlighted the presence of relatively benign vibrations. He completed his study after the destruction of the bridge [4]. The bridge collapsed in November, 7th, under a sustained wind of less than 20 m/s [24, 27]. The destruction, recorded in video by a local photographer, did not cause human casualties, but part of the deck and a few cars ended up in the bottom of the river. This event attracted really early the interest to understand the reason of the failure. At first, the Karman street vortexes were blamed for the destruction of the bridge, but this idea was discarded later, and focus was put on flutter, which will be later explained.

The Tacoma Narrows event is the clearest example of wind-structure dynamic interaction, but many other structures have also suffered these effects. Dynamic effects are more likely to happen in structures with low natural frequencies and damping. Therefore, slender structures are the most vibration-prone structures. Bridges, chimneys and skyscrapers are good examples of slender structures that can suffer dynamic effects. Many skyscrapers have a big damper in the higher levels to mitigate this vibration, such as the Park Hyatt Tower in Chicago [2]. Terrés-Nicoli listed some of the bridges that have suffered dynamic phenomena, such as the Severn bridge or the Kessock bridge [25]. The Menai Bridge was destroyed by wind as soon as 1839 [28]. He also performed a thorough research on another bridge section that vibrated, the Storebælt bridge section. Regarding chimneys, as they are a more common structure, it is difficult to find a list of vibration events, but research papers still try to understand and reduce the phenomenon [29, 30, 31]. Other structures prone to wind induced vibrations range from electric transmission lines to traffic signals [32, 33].

The dynamic effects that wind can cause on structures have the same result (bounded or unbounded vibration), but different origin. This section is devoted to explain briefly each type of vibration and to summarize the state-of-the-art. There are four different types of dynamic responses on bluff bodies, namely vortex induced vibration (which will be referred to as VIV from now on), flutter, buffeting and galloping. These instabilities will be dealt with in the following sections.

1.2.1 Vortex Induced Vibration

As mentioned above, the flow around a bluff body is dominated by detached flow and vortex shedding. The shear layer that has been created at some point of the flow rolls up, and at some point it interacts with the shear layer shed from the other side and “cuts” it. In that moment, a vortex is shed into the wake. The structure of the vortexes in the wake was studied more than a century ago by Theodore von Kármán [34]. A complete description of how vortexes are formed can be seen in [35, 36]. Figure 1.1

presents 6 different moments of the vortex formation. It is important to remark that the interaction of both shear layers is important for the creation of vortices.

Fig. 1.1: Different moments of the formation and shedding of vortexes behind a bluff body. Figure extracted from [35].

For better understanding the VIV phenomenon, a description of flow behavior around a symmetric bluff body will be carried out. At very low Reynolds numbers, the streamlines are attached to the object, but when a certain value is reached, the detachment of the flow happens. At this point is when shear layers are created after the detachment point. These shear layers can reach another point of the object so the detachment area is limited and a bubble is produced instead of a wake. If this is not the case, the shear layer does not reach the object, and a vortex is shed in the wake. The condition for the existence of reattachment can be found in [37].

The process is symmetric in the upper and in the lower sides of the object. Therefore, there are streamlines being shed into the wake from both sides. The interaction between both streamlines creates a pattern in the flow. One of the streamlines rolls creating a vortex. The vorticity supply is cut by the shear layer created in the other side of the body, and finally a vortex is shed into the flow. This shear layer starts now to roll, and the cycle starts again. This way, the Kármán vortex street is created. The formation of the vortexes is well explained in [35, 36].

The frequency at which this vortexes are shed depend on the cross-flow characteristic length of the object and on the velocity of the upcoming flow. They are related to each other via a non-dimensional parameter called Strouhal number:

$$St = \frac{U}{f \times D} \quad (1)$$

where St is the Strouhal number, U the free stream velocity of the flow, f the shedding frequency of the vortices and D the cross dimension of the object. The variation of the Strouhal number with the Reynolds number of the flow for several sections is presented in figure 1.2. As seen, this non-dimensional number is a function of both the studied section and the Reynolds number. Strouhal number is almost a constant in a wide range of Reynolds numbers, for a given cross section. Therefore, equation 1 gives a direct relationship between wind speed and vortex shedding frequency (called “Strouhal law” in what follows) .

Fig. 1.2: Strouhal number of several sections against the Reynolds number. Figure extracted from [15].

As the flow speed increases, the shedding frequency of the vortices equals the first natural frequencies of the structure and VIV occurs. When this velocity is sustained, the loading of the structure follows a cyclic loading with the same frequency of the natural frequency. Therefore, the movement is triggered. Once the movement has been triggered, the vibration of the structure controls the shedding of the vortices, and the lock-in phenomenon appears. If the velocity is increased a bit, the shedding frequency of the vortices does not follow equation 1, but remains almost constant, controlled by the movement of the structure itself. This effect can be seen in figure 1.3. In this figure, vortex shedding frequency (n_s) to structure's natural frequency (n_e) ratio is plotted against the reduced wind speed.

The Strouhal law is not followed in a range of wind velocities, through which the frequency ratio remains constant and equal to 1. When the velocity is increased further, the amplitude of the movement creates self-limiting forces, that ultimately restrict the movement. When the speed is increased even more, the lock-in phenomenon disappears and the shedding frequency matches again the Strouhal law.

Fig. 1.4: Vortex distribution over a rectangular cylinder for different speeds. Figure extracted from [40].

engineers working on the wind loading but also on marine engineering, where long pipes to attach offshore structures or to extract oil from the sea bottom can suffer it [43, 44, 45, 46, 47, 48]. These structures are very slender and can not be easily accessed if they have some structural problem, so it is a major concern for the designers that they are not vibration-prone.

VIV is influenced by many parameters. The influence of the turbulence in the upcoming flow has been studied by many authors [3, 41, 49, 50]. It is widely accepted that turbulence is a desired effect, since the fluctuations in the velocity make it more difficult for the deck to shed coherent vortex structures. In addition, turbulence also alters the angle of attack of the wind, which affects the direction of the vortices shed, and hinders the triggering of the movement [51]. Anyway, some authors still state that its influence is not clear [33]. Kawatani and coworkers studied the influence not only of the turbulence intensity but also of the turbulent length scale. They found that its influence is negligible in vortex induced vibration [52, 53, 54].

On the other hand, turbulence cannot be controlled, and it can trigger other vibration, such as buffeting (see section 1.2.3). This effect has been reported by Honda in some research papers [51, 55].

Flow coherence is also needed to trigger the motion. If the shedding is different at different sections of the bridge, the loads lack coherence, and the motion does not start. One of the most used approaches to avoid or reduce VIV in structures is to break that coherence. This approach will be mentioned further in section 1.4.

1.2.2 Flutter

Flutter is an unbounded phenomenon in which two degrees of freedom are excited (usually bending and torsion) and their frequencies merge, and can lead to the destruction of the structure [56].

The destruction of the Tacoma Narrows bridge is the most remarkable flutter event in history. After it, bridge design and construction has been deeply conditioned by wind effects on the structure. Before the 20th century, bridges were not so large since construction techniques and materials were not so well developed. At the beginning of that century, all these techniques improved, thus longer and lighter bridges were projected. This led to a reduction in natural frequencies, and to a higher sensibility of bridges to wind action. In the years following the Tacoma Narrows event, projects regarding new bridges (such as the Forth and the Severn bridges [57, 58]) were deeply studied in order to avoid aeroelastic instabilities. In that sense, the destruction of this bridge worked as a warning to avoid future disasters.

The first approach for flutter investigation was to compare it to flutter of airfoils, already studied by Theodorsen [59]. But the behavior of a streamlined object and a bluff body is not similar. Even though nowadays bridges are being designed with more streamlined sections, in the past, plate bridges and other type of bridges with bluff sections were used, since they were easy and cheap to manufacture and mount, so their aerodynamic behavior was far from being close to an streamlined object. Nowadays, the trend is to build more streamlined decks, since they increase the stability compared to bluff sections [38, 60]. However, barriers still tend to make them bluff bodies.

Years later, Scanlan and Tomko used Theodorsen's function to validate their semi-empirical model to study flutter. These days, it is widely accepted that most bridge sections are not streamlined and that the use of the Theodorsen function must be limited to airfoil flutter, separating airfoil aeroelasticity and bridge aeroelasticity [16, 24, 61, 62]. Scanlan reasoned that the forces are time dependent in bluff bodies due to the signature turbulence (turbulence that the presence of the object creates in the flow) whilst this does not happen in airfoils or streamlined objects where no detachment occurs [63].

The most used method to face the aeroelastic instabilities of bridges is the one proposed by Scanlan and Tomko [64]. The flutter derivatives had already been studied for airfoils, but they introduced this concept into aeroelasticity of bluff bodies. For airfoils, the derivatives can be approximated by Theodorsen's function, but this approach has already been discarded for bluff bodies. Instead, they proposed to conduct wind tunnel tests to obtain them. The tests were section model tests with two degrees of freedom, namely bending and torsion. Therefore, the system can be modeled as shown in figure 1.5. The equations ruling this system are:

Fig. 1.5: Sketch of a two-degree-of-freedom-system.

$$m\ddot{h} + \xi_h\dot{h} + f_h h = L \quad (2)$$

$$I\ddot{\alpha} + \xi_\alpha\dot{\alpha} + f_\alpha \alpha = M \quad (3)$$

where m is the mass per unit length, I is the moment of inertia, h and α are the vertical and torsional displacements, ξ_h and ξ_α are the dampings in the bending and torsional modes, f_h and f_α are the bending and torsional natural frequencies, L and M are the self-excited wind forces, and $\dot{}$ represents differentiation respect time. These force and moment can be modeled via the flutter derivatives, i.e.:

$$L = \frac{1}{2}\rho U^2 B \left(KH_1^* \frac{\dot{h}}{U} + KH_2^* \frac{B\dot{\alpha}}{U} + K^2 H_3^* \alpha + K^2 H_4^* \frac{h}{B} \right) \quad (4)$$

$$M = \frac{1}{2}\rho U^2 B^2 \left(KA_1^* \frac{\dot{h}}{U} + KA_2^* \frac{B\dot{\alpha}}{U} + K^2 A_3^* \alpha + K^2 A_4^* \frac{h}{B} \right) \quad (5)$$

where ρ is the air density, U is the wind velocity, B is the characteristic width of the bridge, $K = \frac{B\omega}{U}$ is the reduced frequency and H_i^* and A_i^* the flutter derivatives. Usually, H_1^* , H_2^* , A_1^* and A_2^* are called direct derivatives (they relate one degree of freedom with its corresponding load, i.e. vertical displacement with vertical force and torsional displacement with overturning moment) and H_3^* , H_4^* , A_3^* and A_4^* are called cross derivatives (they relate one degree of freedom with the load in the other one, i.e. vertical displacement with overturning moment and torsional displacement with vertical force). In their paper, Scanlan and Tomko presented only 3 of them instead of 4. They proposed a simple method to extract this derivatives from wind tunnel tests. In those tests, a small initial condition is given to the model and the response is recorded for different wind speeds.

This method is called the free vibration method. At first, the initial condition was imposed only in one of the degrees of freedom, and the identification was split between direct flutter derivatives and cross flutter derivatives [64]. To identify them, they used an analytical method. Later, the Ibrahim Time Domain (ITD from now on) technique was proposed by Ibrahim and Mikulcik to extract the parameters of a two degree of freedom system [65]. After that, many other identification methods have been proposed, including modifications of the original ITD [66, 67]. The unifying least-square method and its modified version [68, 69], the iterative least-squares method [70], the extended Kalman filter [71] and others [72] have been proposed. Researchers have not yet found an agreement on which is more accurate or easier to implement.

The other method to determine the flutter derivatives is the forced vibration method. In it, the section model is placed in a rig that can make it vibrate in the two degrees of freedom. Pressure measurements are taken in the surface and then integrated [73]. This method counteracts some of the problems of the free vibration method, such as the difficulties to make a good identification at high wind speeds, because of the high aerodynamic damping in, which makes the bending motion too short in duration. On the other hand, the needed equipment is sophisticated and there is high uncertainty in low speed measurements [69, 74]. Some simplifications of the method have also been proposed [75]. This system has been extensively used by Matsumoto and his co-workers [73, 76, 77, 78].

With the extraction of the flutter derivatives, the critical wind velocity at which flutter occurs can be calculated via a classic eigenvalue problem. This calculation is out of the scope of this thesis, and can be seen elsewhere [79].

The other way to study the critical velocity is ambient vibration tests. In ambient vibration tests, the section model hangs up in a elastic support submerged in a flow. The model has two degrees of freedom (bending and torsion). The displacement of the model is recorded at different velocities. These tests are not useful for determining the flutter derivatives, but give a clear and direct value of the critical velocity.

Flutter has focused the attention of the researchers, since it is one of the most dangerous instabilities. Many efforts have been done to understand the physical phenomenon underlying behind it, but it is not still totally clear today [80].

Many parameters can influence the velocity at which flutter occurs. Scanlan admits the influence of turbulence and initial amplitude in the determination of the flutter derivatives (and therefore, in the determination of the critical wind speed), but the exact form of this influence is not clear [81]. Bartoli also underlines the importance of initial amplitude in the determination of the flutter derivatives [82]. Pulipaka states that the influence of turbulence is still not fully understood [33].

1.2.3 Buffeting

Buffeting is the fluctuating wind load due to the air turbulence. This wind load can lead to the vibration of the bridge [15, 38, 83]. The turbulence can excite certain frequencies of the structure. Some authors have described its effects as an increase in the root mean square response of the bridge with the square of the reduced wind velocity [38, 81].

Meseguer et al. separate two different cases of buffeting, depending on what is the origin of the turbulent flow, the flow itself or another object placed upstream the one suffering vibration [15]. This last case is usually referred to as wake buffeting. Whilst the rest of phenomena are narrow-banded, in buffeting there is not a single characteristic frequency of vibration, so the frequency spectrum is broad-banded [84].

The most used method to study buffeting is through the aerodynamic admittance functions, which are the functions that relate the fluctuating wind velocity and the fluctuating wind load acting on a certain structure. These functions can be considered as functions that account for deviations from quasi-steady theory [83, 85].

Some authors have performed deep studies over buffeting, investigating the influence of parameters such as turbulence intensity, turbulent length scale, and other [86].

1.2.4 Galloping

Some confusion is always present when talking about galloping. Being more precise, the confusion comes when talking about galloping and flutter. Padousis et al. use either of them indifferently, but point out that the term flutter is slowly supplanting the term galloping when talking about torsional vibrations [87]. Usually, the term galloping is restricted to vibrations in bending (only one degree of freedom), whilst flutter is reserved to those vibrations in which torsional movements plays a significant role. In [33], galloping is treated as a single degree of freedom flutter. Blevins also states that the main difference between them is the usage of terms. Flutter came from the aeronautical field to describe the coupled vibration of wings while galloping came from civil engineers describing one degree of freedom instability of bluff structures in winds and currents [88].

Galloping is usually studied with a quasi-steady theory stated by Den-Hartog [89]. The use of this method works well for bending motion, but lacks of precision when it comes to torsional or coupled motion [87]. As most bridges nowadays have very close natural frequencies in bending and torsion, they suffer usually coupled instabilities, and galloping is not usually considered. Some authors have lately tried to expand the quasi-steady theory to make it valid for two degree of freedom systems [90].

Even though the term is slowly disappearing from civil engineering, other engineering fields are still worried about it. In elements such as ice-covered transmission lines, galloping amplitude can reach values of hundreds of times the transverse dimension [91]. In [91, 92], Parkinson gives a deeper sight about the galloping phenomenon.

1.3 Flow condition on a bridge deck

It must always be kept in mind that bridges have the aim to ease the transportation between two places. Therefore, bridge users must not suffer bad wind conditions when they are crossing the bridge. Users can be either people, vehicles or both, depending on the type of the bridge. Each one of the users require different conditions to tag a wind condition as acceptable. Considerable research has been devoted to understand the relation of wind and comfort in pedestrians, not only in bridges but in other areas. These studies have to embrace many difficulties, since it is difficult to state a uniform comfort criteria. Efforts have been done to reach an agreement or at least to be able to compare each criterion [93, 94, 95].

Also vehicles are sensitive to harsh wind conditions on the bridge. Ávila performed several tests to study the risk of overturning of trains depending on the barrier installed [96].

Barriers are one of the key factors influencing the flow condition of the bridge. The protection they provide has been widely studied [15, 97]. Even though the number of parameters that can vary are high, some global conclusions are provided by Meseguer et al [15].

Barriers are also important to protect other industrial facilities such as solar power plants. Torres-García et al. [9] studied the influence of barrier porosity and distance from the barrier of a single row of heliostats.

Therefore, it can be concluded that barriers highly determine both the flow condition on the bridge and the static and dynamic loads that the bridge deck suffers.

1.4 Vibration reduction approaches

The most straightforward reason to reduce or eliminate (if possible) bridge vibration is structural safety. Vibrations can compromise not only the short term safety, but also the long term one due to the additional fatigue loads that they introduce in the structure. When possible, the oscillation motion must be reduced to zero, since even smallest vibrations can lead to visual distractions of the users and to accidents [33, 98]. In [98], a figure shows how a car seems to appear and disappear due to the vibration of the bridge. This image can be seen in figure 1.6.

Fig. 1.6: Effects of bridge vibration in the perspective of a bridge user. The car in the circle seems to appear and disappear because of the vibration of the bridge. This is dangerous since it could lead to visual distractions of the users. From [98].

There are two different ways to solve the vibration of a bridge. The first one is to install in the vibrating bridge a damper, either passive (without a feedback control loop) or active (with a feedback control loop). This issue has been deeply studied. One of the most basic systems used to damp the vibration of a bridge is the tuned mass damper (TMD), where a mass is vibrated/rotated to create forces in counterphase with the forces created by the wind [99]. This type of dampers are usually

challenging to adjust and are very sensitive to the governing parameters [100, 101]. In addition, when they are installed once the bridge is vibrating, they can add dead load to the bridge and give problems to be installed. Despite that, they have been installed in some vibrating bridges, such as the Kessock bridge [102]. In this case, these dampers proved useless since the bridge suffered later events of vibrations. Three vibration events are described and studied in [49]. In that paper, the authors suggest that the problems with the dampers may be related to the bad maintenance labors, underlining the importance of good supervision and control of the parameters of the damper.

Aerodynamic add-ons can also work as dampers if they create the correct forces to balance the vibration. Some aerodynamic dampers that can be found on the literature consist of movable flaps controlled by an algorithm [103], flaps connected to small tuned mass dampers [104, 105], addition of control wings with an active system [53], ailerons whose phase is controlled with a pendulum [106] or a system with two plates that appear or disappear when the bridge vibrates [107]. These devices are theoretically effective in the prevention and reduction of the vibration, but their installation is also difficult, as many of them require that some of their parts should be inside the bridge. Also, active control requires the installation of a computer to control the algorithm ruling the add-ons.

Considering the difficulties that have to be faced in case dampers are the solution adopted, the focus changes to the design of the bridge. This is the second way to avoid vibrations. In this approach, what it is searched is to design a bridge that is not vibration prone. This solution is the desired one according to Scanlan [17, 61]. The design process should be made flexible, since a design change at the first steps does not create great disturbances and can avoid later problems. Larsen and Wall estimated in 1.5 million euros the cost of adding guide vanes to a 1000 meter long bridge, and underlined the importance of design in order to reduce unnecessary costs [108]. Chen and Kareem emphasize that the origin of the vibrations is aerodynamic, as such it should be treated and the solution should come from avoiding the problem rather than “hiding” it [101].

Kessock bridge is not the only bridge whose vibrations were unacceptable. In January 2006, vibration events were detected in the arch of the Alconétar bridge in Spain [109]. The bridge was still under construction, and only the arch was mounted. The vibrations went up to an amplitude of 0.8 meters. A wind tunnel test campaign was undertaken in the “Instituto Universitario de Microgravedad Ignacio da Riva”. The results observed in the wind tunnel matched well with the vibration of the prototype. To avoid this vibration, the installation of a series of guide vanes was suggested. The vibration was eliminated and works could go on. When the bridge deck was finally mounted, the natural frequencies of the structure had changed and the guide vanes were not useful anymore, but they were not removed because they do no harm and this process would be expensive.

The Storebælt bridge, a remarkable structure connecting two islands of Denmark, is a suspension bridge that also went through several episodes of vibration. The vibrations were considered harmless

from the structural point of view, but their mitigation was desired for physiological reasons for the users of the bridge. The solution was, as in the case of the Alconétar bridge, to install guide vanes to reduce the size of the vortexes shed. The observations in the prototype compared well with the wind tunnel tests performed [84, 98]. The monitoring system remained in the bridge after the installation of the vanes, but some parts were stolen, and hence data are no longer available.

An interesting review on methods to avoid vibration due to VIV was performed by Kumar et al [36]. The paper is focused in marine pipes VIV having a circular section, but it can easily be applied to bluff bodies submerged in wind. In that paper, two factors are underlined as the most determinant to trigger VIV. The first one is that the vortexes created must be energetic and the second one is that the interaction between the shear layer shed from the upper and the lower surfaces is also a requirement. Some methods are proposed to avoid VIV. The first two of them are structural changes (stiffening the pipe, so that the natural frequencies are higher and the lock-in phenomenon happens at a higher velocity, and increasing the damping of the structure, for which it suggests a Stockbridge type damper [110]). The rest of solutions proposed are focused on the aerodynamic characteristics of the pipe, and can be seen in figure 1.7.

Fig. 1.7: Modifications to reduce VIV response in marine structures. From [36].

The first solution is to make the section of the pipe streamlined or even airfoil-shaped (example h of figure 1.7). This solution does not break the interaction of shear layers, but prevents their formation, so the vortexes shed are not so energetic. It is feasible for marine pipes, even though the airfoil must be able to rotate since the flow direction can vary, but difficult to do in other

structures such as bridges. In examples b-g, solutions are proposed regarding the modifications of the aerodynamic characteristics, which are mainly focused on avoiding the creation of energetic vortices. Helical strakes (1.7e), wider covers with porosity (1.7f) or slots (1.7g), installation of plates around the section (1.7d) or installation of fairings with gaps between them (1.7c) are some of the other solutions proposed. Of course, there are drawbacks to all of them, mainly the increase in drag they lead to. Some of these solutions can easily be seen in chimneys, such as the helical strakes in figure 1.9. Also car antennas have these strakes to reduce its vibration, which can be annoying for the driver and passengers. The usual way to avoid the interaction of the shear layers is to place a splitter plate downstream the body, so that the shear layer reattaches and vortices are not created. That is illustrated in figure 1.7a. The effectiveness of the splitter plate can be clearly seen in figure 1.8. In that figure, it is clear that the shear layers do not interact when the splitter plate is added, and the vortices created are smaller and less energetic. This solution cannot be applied in flows where the direction changes. It has been studied, and its efficacy checked, for cylinders and other prisms [20, 73, 111]. Roshko [5] pointed out that it also helps to reduce the drag in bluff bodies, increasing the base pressure. Even solutions with hinged splitter plates (allowing some interaction between the shear layers through the motion of the plate) can be found in reference [112], where it is pointed out that the motion of the plate could be used for energy harvesting purposes. Kumar et al. [36] also proposed to add bumps to the surface. Another proposal is to implement a boundary layer control over the surface of the body so to prevent the detachment. This can be achieved by the installation of guide vanes such as the ones mentioned in the Alcontar bridge or the Storeblt bridge. This way, the boundary layer is energized and the detachment is less likely to occur.

Besides the importance of the energy of the vortices stated by Kumar et al. [36], other requirement for triggering the movement can be seen in [113]. Scanlan remarks that the breakage of the lateral coherence of the flow can reduce or even suppress VIV, and this has also a positive effect on flutter, transforming it in a gradual event instead of a sudden phenomenon.

1.5 Approaches to study fluid-structure interaction

When designing bridges and other structures subjected to wind loads, four main approaches can be taken. First one is analytical study. Equations governing fluid dynamics are complex and very rarely have analytical solutions. Taking also into account that the flow around bluff bodies is more complex than around streamlined bodies, this approach is not useful when facing bridge aerodynamics.

A shortcut for calculating wind loads on different structures is looking to the codes. Many codes, such as Eurocode, ISO, ASCE/SEI, are trying to standardize the wind loads on structures, but the process is complex due to the very different geometries that can arise in civil engineering [114, 115, 116].

(a)

(b)

Fig. 1.8: Flow visualization around an H shaped section. Image (a) presents the visualization without a splitter plate and image (b) the visualization with the splitter plate. From [111].

Because of this, these codes are focused mainly in simple geometries, and it is difficult to extract conclusions for the more complex ones. For example, the Eurocode considers an expression for the VIV velocity, but it depends on the Strouhal number, and the code provides the Strouhal number only for a certain number of sections. Bridges do not usually match any of those geometries, therefore wind tunnel studies are needed. In addition, some doubts arise regarding some of the rules developed

Fig. 1.9: Helical strakes placed in the higher part of a chimney in London.

in the codes [117].

The third approach is via Computational Fluid Dynamics (CFD). Lately, with the increase of the calculation power of the computers, numerical methods have started to gain importance in the study of aerodynamics. They seem appealing because of their relatively low cost in comparison with the experimental approach. Numeric methods are very accurate and useful in streamlined objects and have been extensively used in the field [118]. Unfortunately, even though progress has been made, these methods do not predict correctly the creation and shedding of vortexes. It has been already said that vortex formation is very important in bluff bodies, and therefore, the numerical approach has not been used throughout the thesis.

Finally, the experimental approach can also be used, via wind tunnel tests. In this thesis, this method has been mainly used as a method of study. When wind tunnel tests are performed, scaling becomes a major issue. It is very important to ensure that the wind effects in the wind tunnel model are the same to those in the real object. For that, depending on the type of wind effect that wants to be studied (either static or dynamic), different parameters will have to be adequately scaled. If the behavior to be characterized is static, geometry and Reynolds number are the most important ones. If dynamic phenomena is going to be studied, parameters such as mass, mass moment of inertia and natural frequencies acquire an important role. The importance of scaling is underlined in section 1.5.1.

Two main different approaches can be taken in wind tunnel tests to study the dynamic behavior

of a structure. The most complex one is to make a full aeroelastic test, where a bridge model is manufactured with all its elements (deck, pillars, towers, cables) and even the surrounding topography so as to be as faithful as possible to the wind conditions in the area. The main problem with these tests is that not only the geometry has to be reproduced, but also the frequencies, dynamic properties such as damping and modes of vibration. This requires a complex and expensive manufacturing process. In addition, large wind tunnels are needed and the reproduction of the atmospheric boundary layer at the desired scale is also a requirement, being able to reproduce less details [119].

Another approach is more widespread among researchers. The so-called section-tests consist in the reproduction of a characteristic section of the bridge and building a two-dimensional model with that section. Under the assumption that only lowest modes of vibration are likely to occur, they are the only vibration modes reproduced. These modes are usually bending and pitching modes. Therefore, dynamic properties of those two modes have to be taken into account. In this type of tests, the models are built in smaller scales, hence more aerodynamically relevant details can be added. In these models, it is easier to test small variations of the section, since only the first modes of vibration are reproduced. Some researchers have underlined weaknesses of these tests, such as being too conservative and lacking of 3D wind conditions that are present in the prototype [119]. The lack of three-dimensional effects is not an issue in very long span bridges, since they are negligible in the prototype if the section of the bridge does not change significantly. Other researchers have found good agreement between both methods [57, 58]. Scanlan [113] made a comprehensive comparison between both types. Whilst the section model tests have the advantage of reproducing more details of the bridge and are good at first steps of design (it is easier to make modifications in these tests), they are not useful for studying the effect of yaw. The drawback in full-scale tests is that, as the scale is bigger, the Reynolds number can be too close to the critical value and it has to be carefully traced. More recently, Chen et al. [120] found a good agreement between section model tests and full aeroelastic tests performed on H-section hangers.

1.5.1 Importance of scaling in wind tunnel studies for bridge sections

To pursue the objectives of this thesis, which will be dealt with in subsection 1.6, experimental wind tunnel tests have been performed. Testing structures subjected to wind loads is a common practice in first steps of design. Designing a correct wind tunnel test campaign to achieve results close to reality is not straightforward [15, 16, 121]. Other types of tests do not represent such a challenge, but dynamic testing, which is needed in bridge testing, is a bit more complex than static testing.

When performing dynamic tests, scaling becomes a major issue, since not only geometry but also dynamic properties of the section have to be scaled. The problem of scaling is a classical dimensional

analysis problem to study the non-dimensional numbers influencing a problem. In this case, the parameters influencing the problem are:

- b , characteristic length of the bridge.
- ρ, ν , density and kinematic viscosity of the upcoming flow.
- U_∞, α , velocity and angle of attack of the flow.
- m, I , mass and mass moment of inertia of the bridge per unit length.
- ζ_h, ζ_θ , damping in the bending and torsional movements.
- f_h, f_θ , frequencies in the bending and torsional movements.

Therefore, the problem has 11 parameters. In order to know the number of non-dimensional parameters that influence the problem, Buckingham's π theorem can be applied. The number of fundamental magnitudes in the group of variables is 3 (length, time, mass), so 8 non-dimensional numbers can be constructed. These numbers are the ones that should be kept in the wind tunnel tests in order to represent reliably the real problem. The usual non-dimensional numbers employed are:

- **Reynolds number:** $Re = bU_\infty/\nu$
- **Reduced velocity:** $U_r = U_\infty/(f_h b)$
- **Angle of attack:** α
- **Reduced mass:** $m/(\rho b^2)$
- **Reduced mass moment of inertia:** $I/(\rho b^4)$
- **Bending damping:** ζ_h
- **Torsional damping:** ζ_θ
- **Frequency ratio:** f_h/f_θ

Some observations should be done on the obtained parameters. Reynolds number is one of the most important parameters regarding aerodynamics of airfoils and other streamlined objects. As it has already stated before in section 1.1, in bluff bodies, changes in the Reynolds number should not be considered, provided a critical value has been overcome.

When performing wind tunnel tests, reduced velocity has to be kept. This parameter is important because the wind tunnel can only give velocities in a certain range. Geometric scale and natural frequency of the model in the wind tunnel have to be carefully selected, so as to keep similar values of reduced velocities in both the model and the prototype. Bad selection of any of them could lead

to a situation in which the wind tunnel tests would study wind velocities out of the range of the expected values in the prototype.

Angle of attack of the flow is the easiest parameter to keep, it should just be observed that the upcoming flow has the same angle of attack in the wind tunnel than in the prototype.

Both bending and torsional dampings are dealt with the same way. When setting up the wind tunnel test, dampings in both degrees of freedom are studied. The objective is usually to have a damping value smaller than the damping in the prototype. This way, the tests will lead to conservative results. Some efforts are being done to obtain magnetic dampers that can help to precisely reproduce the damping of the bridge (see [122]), but for now, the smaller value of damping approach is quite common and does not require complex equipment.

Another very important parameter to keep is the frequency ratio. It has a large impact on both the dynamics of the bridge and the flutter conditions. There is not a unique set-up to perform the tests (two different set-ups have been used through this thesis), so several ways to keep the frequency ratio can be found in the literature. One of the most used set ups is one using linear springs to give the bending frequency and the spacing between them to provide the torsional frequency (see section 2.2.2). Others use a torsional spring to give the torsional stiffness. This is the case explained in section 3.2. In that configuration, linear springs have been substituted by a beam that provides the vertical stiffness.

Finally, mass and mass moment of inertia appear also as parameters. Since the air density is the same both in the wind tunnel test and in the prototype, the reduced mass and reduced mass moment of inertia equalities can be written as

$$\left(\frac{m}{b^2}\right)_r = \left(\frac{m}{b^2}\right)_m \quad (6)$$

$$\left(\frac{I}{b^4}\right)_r = \left(\frac{I}{b^4}\right)_m \quad (7)$$

where the subscript r stands for the real bridge and the subscript m stands for the model in the wind tunnel. According to that, the mass has to be scaled with the square of the geometric scale and the mass moment of inertia with the fourth power of the geometric scale:

$$\left(\frac{m_m}{m_r}\right) = \left(\frac{b_m}{b_r}\right)^2 \quad (8)$$

$$\left(\frac{I_m}{I_r}\right) = \left(\frac{b_m}{b_r}\right)^4 \quad (9)$$

This means that a bridge model to perform dynamic tests has to have a certain value of mass and mass moment of inertia. Therefore, the material selected to build the model is very important. As there are a number of uncertainties in the manufacturing of the model, it is very difficult to achieve the desired values just from design. To overcome this problem, the usual procedure is as follows: A model lighter than the objective is designed. Mass and mass moment of inertia of this model are determined (weighing the model the mass is obtained, and vibrating it in rotation the mass moment of inertia is obtained). Ideally, both mass and mass moment of inertia will be smaller than the objective. As the model is lighter than the objective, some extra mass should be added. By adding it at a certain distance, an extra mass moment of inertia will be added to the model, and both objectives can be achieved.

Ensuring the good scaling of these parameters, the behavior of the model in the wind tunnel can be considered similar to the full scale bridge. For a deeper view on the scaling process, Blevins describes it in the first chapter of [88].

1.6 Objectives

The present thesis is aimed at to study the influence of barriers in the stability of bridges and vehicle/pedestrian safety. Even though they are not a key structural object of the bridge, barriers have a very important role in bridges. First of all, and it is always good to remember, obvious as it may seem, they avoid problems of vehicles/pedestrians falling off from the bridge. Secondly, they protect the bridge users from bird or other fauna intrusion that could lead to accidents. Some papers have already studied these effects [123, 124]. Finally, they also protect the users from harsh wind condition. Big vehicles are prone to overturning in case of strong cross-wind [125, 126], and bridges, because of their mission, usually are not surrounded by other structure that could reduce the wind condition, so barriers are needed.

The main problem with the barriers is that protection and dynamic stability of the bridge are usually not aligned: the higher the protection they provide, the higher the sensibility towards vibration. One of the reasons to explain it is that solid barriers create a bigger area exposed to the wind, and in that case, bigger vortexes are shed and, as vortexes are low pressure areas, they create stronger forces. In addition, the presence of solid barriers does not break the longitudinal coherence of the flow, which has been stated above as a very good method to delay and reduce vibration.

Procino [127, 128] and coworkers studied the influence of barrier porosity in the flow condition on the bridge deck, but did not study the influence on the stability. Larsen [108], in a paper studying the shape of bridge decks, concluded that Jersey type barriers, or porous barriers covered by snow can lead to the instability of an otherwise stable bridge. This brings a bit of light in the relationship

between user protection and bridge stability, in the sense that higher protection levels usually imply a more sensitive bridge deck section to dynamic instabilities.

Kubo [129] also studied the influence of Jersey barriers on a π shaped bridge section, reaching a similar conclusion to Larsen, i.e. that solid Jersey-type barriers can make a stable section unstable. Ricciardelli [130] pointed out other effects they provoke, such as an increase in the drag and a decrease of the Strouhal number. The tests on the Sunshine Skyway bridge threw lower values of flutter critical velocity with the barriers installed than without them. Ricciardelli made pressure measurements on the section, and found that barriers also increased the RMS value of the pressure in the leeward side. The origin of that increase is the impact of the shear layer shed by the upper part of the barrier. He also states that aerodynamic appendixes should be installed to increase the stability.

Dyrbye and Hansen [38] underlined the great alterations that barriers and walls produce in the aerodynamic characteristics of the bridge, reducing the critical velocity. In cold places, ice accretions in barriers could lead to even lower critical velocities. They also mention that the own traffic of the bridge can condition its aerodynamic performance. But not only big objects such as barriers or vehicles alter the aerodynamics of a bridge. Even a small detail can change the critical velocity. Jones et al. studied the influence of these details [131]. Also Zasso et al. [132] checked that adding a porous mesh and changing the inclination of the handrails, a bridge section increased its critical velocity a 25%. Using numerical methods, Bruno and Mancini [133] also reported the importance of the details. This fact points out the importance of a faithful reproduction of the geometry of the bridge when performing tests in the wind tunnel.

Honda et al. [51] studied the stability of a concrete bridge section with noise barriers. Being a heavy and streamlined bridge, the model was stable before the installation of the noise barriers. The presence of the noise barriers, which are totally solid, triggered the VIV motion. Two solutions were proposed: a) to separate the barriers and to bend the upper and lower parts of the barriers, and b) install a plate working as vortex generator to create vortexes to interact with the most powerful ones. Second solution showed no instability at any angle of attack of the wind. They also studied the stability in different turbulence conditions.

Barriers, thus, have been studied regarding the problems they can create on bridge stability and, more generally, their influence in other important aspects of bridge aerodynamics. However, there is a lack of systematic studies on the influence of the porosity of the barriers. This thesis tries to cover partially that shortage.

The effect of the barriers can both increase or decrease stability, depending on their design. In the first case, they can help to stabilize a bridge section. In the second case, an application has been

lately developed regarding energy harvesting. The energy of a vibrating motion of a certain object can be harvested distributing thus the energy generation and using in addition a renewable source of energy. Therefore, a lot of research has been done lately in this regard. This thesis is aimed to keep both focuses.

On a bigger scale, the aim of the thesis is to give a deeper insight on the interaction between the air flow and the bridge. To achieve that, an experimental approach has been adopted. Hence, wind tunnel tests are the main source of information and results.

2 Tests on a railway bridge

With the economic development, the transport systems of countries have improved. Highways and railways have extended their networks even in countries with difficult orography (almost 3000 km of high speed railways in Spain were available in 2015). Because of their demanding constraints (small slopes are admitted, and, in case of curves, large radii are required, so that vehicles can keep high mean velocities), tunnels and bridges are a common solution in areas with complex terrains, since they allow for a quite a straight outline to be maintained.

Depending on the area of construction, each kilometer of high speed railway (HSR from now on) has a different cost. In China, a report by the World Bank [134] estimates the cost of each HSR kilometer between 17 and 21 million dollars. Meanwhile, in Spain, that cost reached up to 39 million dollars per kilometer in the Madrid-Valladolid railway [134]. In that report, it is also stated that the cost of bridges and viaducts in HSRs represent one quarter of the total cost of the line, reaching in some cases even the 40 % of the cost. Therefore, all savings are welcome. According to the report, low-cost viaducts cost in China between 57 and 73 million dollars per kilometer (in double track lines). The relatively low cost of this infrastructure is obtained by the standardization of the design, manufacturing and laying bridge beams processes.

In bridges, several different sections are available, each one having its advantages and drawbacks. One of the most used is box girder bridge, which has a very high torsional rigidity and gives an inner space that can be used to accommodate services such as gas or water pipes, etc. This type of section is very popular for railway and highway bridges. Some overview of these bridges can be seen in [135, 136]. They are easily constructed and have many advantages. One of them is the ease of standardizing the section manufacturing and deployment.

The optimization of the behavior of the bridge under wind is important in this process of standardization. In order to give steps towards the standardization of barriers, several tests have been performed for a classical box girder bridge.

2.1 Section characteristics

The section under study is a very common section used both in highway and railway bridges. It is a box bridge with two eaves. A sketch of the section is shown in figure 2.1, together with the definition of the positive angle of attack, α . The geometrical and mechanical properties of the bridge deck are shown in table 1.

The selection of the cross-section is not random. The studied section is a very common section for

post-tensioned concrete bridges [136]. In some occasions, the a multi-cell box girder is employed, but in this case, single-cell box girder has been selected.

Fig. 2.1: Sketch of the bridge section studied, with the reference frame employed, and the definition of angle of attack, α , and bridge deck height, D .

Tab. 1: Main mechanic and dynamic properties of the bridge deck.

Property (units)	Value
Deck depth (m)	4.62
Mass per unit length (kg/m)	37005
Mass moment of inertia per unit length ($\text{kg} \cdot \text{m}^2/\text{m}$)	4.7×10^5
Bending eigenfrequency (Hz)	1.88
Torsion eigenfrequency (Hz)	11.3

For manufacturing the models, a 1:33 scale has been selected. This scale allows to reproduce many relevant aerodynamic details without generating blockage in the test section, which has to be usually considered when the front area of the model is bigger than 10 % of the test section.

In addition to the bridge deck, 4 different bird protection barriers have been built. Their height is 5 meters in prototype scale (corresponding to 0.151 meters in model scale). Two of them are straight and two of them are curved. One of the straight barriers and one of the curved barriers have the lower 3 meters (corresponding to 0.091 meters in model scale) covered with a aerodynamically solid screen. The details of these barriers are presented in figure 2.2, on the left column, the front and side views of the barriers with their dimensions, and on the right, a picture of each barrier installed in the bridge model.

Fig. 2.2: Sketches and pictures of the different barriers: (a), (c), (e), (g) present a sketch of the barrier with the main dimensions and (b), (d), (f) and (h) the picture of the barrier.

Symbol key can be found in table 2.

All the tests performed are supposed to be two-dimensional. Therefore, the models have been tested with endplates which reduce the three dimensional effects. The endplates partially reduce the

three-dimensional effects, but do not totally eliminate them. Roshko [5] studied three-dimensional effects, and discovered that beyond a Reynolds number of $Re = 180$, three-dimensional flow appears independently of the presence or absence of endplates. In fact, Schewe [22] also concluded that wakes are highly three dimensional even in two-dimensional bodies. The presence of endplates avoids the flow injection from windward to leeward sides of the bridge laterally, which could increase the base pressure, thus reducing the aerodynamic drag. The reduction in the induced drag due to the presence of the endplates is bigger than the friction drag that the endplates generate, and therefore, it is a common practice to use them to study two-dimensional objects [137]. Endplates also avoid airflow between the lower surface and upper surface.

2.2 Experimental setup

In order to perform the study of the aerodynamic and aeroelastic study of the bridge, a set of four different tests have been prepared and performed. These tests comprise the study of the static loads, dynamic behavior of the bridge and flow characteristics in the deck. Overall, 4 different phenomena have been studied:

- Static wind loads on the bridge section
- Aeroelastic behavior of the bridge
- Study of the effects of the barriers on the aerodynamic loads on trains
- Determination of the wind conditions (velocity and turbulence intensity) on the position of the catenaries

Two models of the chosen section have been manufactured. The first model is used for the static tests, the tests to evaluate the effect of the barriers on the reduction of the loads on trains and for the determination of wind conditions. The other model, manufactured also at the same scale, is longer and is used for the aeroelastic tests.

The set of experiments performed with different configurations on this section is shown in the ‘Bridge section’ block of table 8.

2.2.1 Static tests on bridges

For the characterization of the aerodynamic static loads on the bridge, A9 wind tunnel is used (see annex B). In this wind tunnel, the model is placed on a six-component strain gauge which is placed just beneath the wind tunnel floor. This device is a Delta 330-30 model from ATI Industrial

Automation, and can measure three forces and three moments along three different axes. Both the model and the balance are mounted onto a turning platform from NEWPORT, which is used to automatically set the angle of attack of the model. The instrumentation also includes a Pitot tube (Airflow 3.3.311), a pressure transducer (model LPM9381, manufactured by Druck) and a data acquisition system.

In this case, the model is placed close to the wind tunnel, and on the top of it (but not touching it) there is a plate working as an endplate. The bridge section has been fixed to the floor carefully, ensuring that the center of the six-component strain gauge is in the position of the center of mass of the section.

The tests consist in the measurement of the aerodynamic forces and moments, varying the angle of attack between -6° and 6° every 2° . The loads have been acquired during 30 seconds, and the sampling frequency is $f_s = 500$ Hz. This procedure has been repeated for the fifteen configurations described in table 2.

Tab. 2: Configurations tested for the static tests. For the definition of each barrier, see figure 2.2.

Code	Description	Figure 2.2 key
SP	Bare deck, with no barriers	-
SP-BR	Deck with BR barriers (porous with straight tubes)	(a), (b)
SP-BC	Deck with BC barriers (porous with curved tubes)	(c), (d)
SP-BRP	Deck with BRP barriers (solid lower part with straight tubes)	(e), (f)
SP-BCP	Deck with BCP barriers (solid lower part with curved tubes)	(g), (h)
SP-TB	SP configuration with a train model in the windward rail	-
SP-BR-TB	SP-BR configuration with a train model in the windward rail	-
SP-BC-TB	SP-BC configuration with a train model in the windward rail	-
SP-BRP-TB	SP-BRP configuration with a train model in the windward rail	-
SP-BCP-TB	SP-BCP configuration with a train model in the windward rail	-
SP-TS	SP configuration with a train model in the leeward rail	-
SP-BR-TS	SP-BR configuration with a train model in the leeward rail	-
SP-BC-TS	SP-BC configuration with a train model in the leeward rail	-
SP-BRP-TS	SP-BRP configuration with a train model in the leeward rail	-
SP-BCP-TS	SP-BCP configuration with a train model in the leeward rail	-

2.2.2 Dynamic tests on bridges

The dynamic tests have been performed in the ACLA16 wind tunnel (see annex A). The setup for these tests is the usual one for two-dimensional dynamic tests: the bridge section is hung from a set of coiled springs, so that the model has two degrees of freedom (vertical and rotational). This configuration allows to simulate the vibration characteristics of the prototype on bending and torsion. The whole system is arranged such that the springs remain with a longer length than their natural length, so that at all times they have a pre-load and are always working within their linear range. The springs are anchored to the wind tunnel floor and ceiling by means of turnbuckles. They allow to adjust and to balance the model. The separation of the springs can be easily changed (the eye bolts to which the springs are joined can be continuously moved through guides installed in the ceiling and the floor of the wind tunnel), thus changing the ratio between the bending and torsional natural frequencies. The ratio of frequencies is set to be the same in the model as it is in the prototype. In this case, as the ratio of frequencies is bigger than 6, torsional movement is considered unlikely to happen, and it is not studied. Matching of bending/torsional frequencies is not usually studied unless their ratio is around 1.2 [138]. Dyrbye stops his analysis of coupled flutter when the frequency ratio is bigger than 3 [38].

Besides, plywood endplates have been added to minimize undesired three-dimensional effects on the wind tunnel. The endplates work as aerodynamic mirrors, so the section behaves as an infinite section, and the flow in the test section can be supposed two-dimensional. The sketch of the setup for these tests can be seen in figure 2.3

Fig. 2.3: Setup for the dynamic tests.

The instrumentation for the dynamic tests comprises two laser distance sensors (M1L/200 models

by Mikroelektronik GmbH) and a hot wire 55p16 hot wire probe. The two lasers have been placed in the same side of the model. One of them is upstream from the center of the deck, whilst the other is placed downstream the center of the deck. This way, the vertical and the rotational displacements can be obtained as:

$$h = \frac{y_1 + y_2}{2} \quad (10)$$

$$\alpha = \frac{y_1 - y_2}{d} \quad (11)$$

where y_1 and y_2 are the measurements of the displacement given by the lasers (from the upstream laser and the downstream laser, respectively), d is the distance between them, and h and α are the bending displacement and rotational displacement of the bridge deck.

The hot wire probe is placed upstream the bridge model, and measures the wind velocity of the upcoming flow without perturbation. This system is widely used in technology applications and research in engineering [139, 140]. The model used in these experiments is a one component probe and it has been manufactured by Dantec Dynamics Ltd. All the measurements (for both the lasers and the hot-wire probe) have been sampled at an acquisition frequency of $f_x = 1000$ Hz, during a sampling interval $T_s = 30$ s.

Once the setup is ready in the test chamber, the dynamic properties of the model are determined by giving it a small initial displacement and letting it oscillate freely. The time history of the oscillations has been recorded and it has been analyzed by the Ibrahim Time Domain identification method [65]. After this process, the calculated natural frequency is $f_n = 3.23$ Hz, and the structural damping coefficient relative to the critical value is $\xi = 0.01$ %.

2.2.3 Static tests on trains

In addition to the loads on the whole bridge, either with or without train model (see section 2.2.1), it is also interesting to study the different wind loads on a train model. Because of these loads, trains may be in the risk of derailment [141, 142]. For that, the needed setup is similar to the one described in section 2.2.1. In this case, the bridge is fixed to the lower endplate, which can rotate, since it is mounted on the angular positioner. The strain gauge is now placed in one of the rails (either windward or leeward) and the train is mounted on it. The train model is placed on the strain gauge ensuring that it is not in contact with other elements of the model, which could alter the measurements. These tests have also been performed in the wind tunnel A9 (see annex B).

A cross view of these tests is presented in figure 2.4 . In this case, the axes of the balance (which is

colored in brown in the picture) have changed: x axis is parallel to the deck floor, z axis is vertical and y axis is in the direction of the bridge.

Fig. 2.4: Axis system for the static tests on trains. In brown, the position of the balance.

The strain gauge employed in this setup is a Mini40 model, manufactured by ATI. The measuring range for the forces is ± 20 N in the x and y axis, and ± 60 N in the z axis. The measuring range for the moments is ± 1 N in all the axes. Due to its smaller size, this gauge is more suitable for these measurements than the Delta employed for the static tests on the whole model.

In order to measure the wind velocity and the dynamic pressure, a Pitot tube has been installed upstream, the same way as it was done for the tests on the bridge deck. The Pitot tube is an Airflow 3.3.311 model, and the pressure transducer, a capsule manufactured by Druck, model LPM9381.

Two details of the set-up are shown in figure 2.5. In figure 2.5a, the bare deck can be seen, where the two holes are the positions where the strain gauge is placed for measuring the loads in both the windward and leeward railways. A detail of the strain gauge placed in the bridge with the train mounted is shown in figure 2.5b.

The procedure for doing the experiments is simple. After placing the train model on the gauge, which is fixed to the deck, the whole model is fixed to the endplate that is rotated by the angular positioner. The wind tunnel is switched on, and the signal is recorded for 60 seconds with a sampling frequency of 1000 Hz. In order to avoid unrealistic effects, the whole left in the

(a)

(b)

Fig. 2.5: Two pictures of the setup for the static loads on trains, (a) bare deck with the position where the strain gauge is placed for the determination of loads on the train model, (b) detail of the train mounted on the strain gauge.

2.2.4 Catenary flow condition tests

The last set of tests consist on the determination of the flow condition in the position of the windward and the leeward catenaries. These tests have been conducted in the ACLA16 wind tunnel (see annex A), but the model is the one used in the static tests. This model is placed with its longitudinal axis perpendicular to the wind tunnel floor, so it works as an endplate. In the upper limit of the bridge section model another plate has been placed to improve the two-dimensional flow. A special support has been designed to fix a two-component hot wire anemometer (manufactured by DANTEC, model 55p61) at the windward and leeward position of the catenaries. A sketch of the section with the position where catenaries are supposed to be is shown in figure 2.6. The anemometer measures the two components of velocity in the plane seen in that figure. u is the velocity component in the x direction whilst v is the velocity component in the y direction. A Pitot tube has been installed upstream the model to measure the velocity of the unperturbed flow. Again, a Druck capsule, model LPM9381, has been installed to measure the dynamic pressure from the Pitot tube. The velocity obtained from the Pitot tube has been used to calculate the non-dimensional velocity in the catenary position.

Fig. 2.6: Sketch of the axis system and the measuring positions.

The tests have been performed for a range of angles of attack from -6° to 6° , with a step of 3° . Only the configurations with no train have been measured (first five configurations listed in figure 2), i.e. the bare deck and the deck with the four different barriers. In this case, the sampling frequency is $f_s = 1000$ Hz and sampling time is $T_s = 30$ s.

2.3 Results

In this section, the results of the tests performed on the railway bridge are displayed. The setups for these tests have been described previously. The contents of the section will follow the order used for the setup explanation: first of all, results of the static tests on the bridge or bridge/train system will be presented, then, the dynamic stability of the bridge, afterwards, the wind loads on the train model, and finally, the flow condition in the catenary positions.

2.3.1 Static loads on bridges

For the static tests, the aerodynamic coefficients per unit length are defined as usual:

$$c_l = \frac{l}{q_\infty D} \quad (12)$$

$$c_d = \frac{d}{q_\infty D} \quad (13)$$

$$c_{m_z} = \frac{m_z}{q_\infty D^2} \quad (14)$$

where l , d and m_z are the aerodynamic lift, drag and torsion moment respectively, per unit length, q_∞ is the reference dynamic pressure, and D is a characteristic length, which in this case is the height of the section (see table 1). In this case, the torsion moment is referred to the center of mass of the section.

The aerodynamic coefficients per unit length for the bridge section with the four different tested barriers are shown in figures 2.7 to 2.9.

Fig. 2.7: Aerodynamic drag coefficient for the bridge section. (a) without train, (b) with the train in the windward position and (c) with the train in the leeward position. Legend explained in table 2.

When there is no train in the bridge, drag coefficient does not vary significantly with the angle of attack for the bare deck (figure 2.7a). Porous barriers, even having high porosity, increase considerably the drag coefficient, and the difference between curved or straight barriers is not noticeable but for high angles of attack. Barriers with solid parts increase even more the drag. This increase is higher in high angles of attack. The explanation for that is that at negative angles of attack wind impinges the upper surface of the deck, and not the bluffer lower surface, hence, less energy from the flow is transferred as drag to the bridge.

Placing the train in the windward railway changes the behavior of the drag coefficient (figure 2.7b). In this case, solid barriers have a very similar behavior to that without train. In this case, barriers provide a protection to the train, so it is submerged in the wake and does not imply higher loads on the whole. Otherwise, in the case of the bare deck or the porous barriers, drag coefficient is higher, not only compared to the solid barriers but also to the configuration without train. In fact, the presence of porous barriers seems useless, since in the upper surface the most important contribution to the drag is the train itself. Porous barriers generate a wake in their elements that reduce slightly the wind velocity, and therefore, the wind load. This could be the reason for the difference in the drag coefficient between the bare deck and the porous barriers.

Finally, figure 2.7c presents the aerodynamic drag of the system when the train is placed in the leeward railway. As happened with the train in the other railway, the configuration with larger drag coefficient is the one without barriers. In positive angle of attack, the deck itself works as a shield for the train, so the drag coefficient reduces for higher angles of attack. This behavior can also be noted for porous barriers. Solid barriers, however, present a minimum for the drag coefficient around an angle of attack of 0° . In this case, porous barriers give a smaller drag, since the wake of the barrier is longer and the reduction of the wind velocity is bigger when the air hits the train model.

(a)

(b)

(c)

Fig. 2.8: Aerodynamic lift coefficient for the bridge section. (a) without train, (b) with the train in the windward position and (c) with the train in the leeward position. Legend explained in table 2.

Lift coefficients have a very different behavior. Figure 2.8 presents the lift coefficient for the configu-

ration without train, and with the train in either windward or leeward railway. Figure 2.8a presents the lift coefficient without train. In this case, the configuration without barriers and with porous barriers show the same trend: lift coefficient increases as angle of attack increases and reaches a somehow stable value at positive angles of attack. Without barriers, the value is higher than the case with porous barriers. The reason for that is that tubes forming the barrier introduce turbulence in the flow, that, as it is known, tends to reduce average wind loads. Solid barriers, on the other hand, show a different pattern. The lift coefficient presents a maximum at small negative angles of attack. In case of positive angles of attack, the lift decreases, since air impinges more directly in the lower surface, which is bluffer and contributes less to the lift. In addition, at negative angles of attack, solid barriers create a recirculation bubble in the upper surface that intensifies the suction in that part. When the angle of attack is increased, the bubble is not closed, so a wake is created, and lift decreases.

The results for the train in the windward railway can be seen in figure 2.8b. For no barrier or high porosity barriers, lift coefficient constantly decreases for increasing angles of attack. When air impinges with negative angles of attack, the recirculation bubble present in the wake of the train creates a suction effect that contributes to increase the lift. When the angle of attack is increased, the bubble can reach a situation such that it is not closed, and therefore a wake is formed. In this case, lift coefficient decreases. Solid barriers have a different trend. Once again, they present a maximum around an angle of attack of 0° . The reason for that is that at this angle the shear layer shed by the train closes at the trailing edge barrier and the bubble created in the deck generates a strong suction. For smaller angles of attack, the bubble closes earlier, and the suction generated is smaller. For higher angles of attack, the bubble does not close, and a wake is created, reducing the lift.

Finally, for the train placed in the leeward railway, figure 2.8c, solid barriers present, once again, a maximum at an angle of attack of 0° . In this case, the value of the coefficient seems bigger, but trend is similar to the behavior of the previous case, i.e., the presence of the train in either railways does not significantly vary the lift coefficient. This is not the case for the porous barriers or the bridge without them. In this case, the lift coefficient increases as the angle of attack increases. For small angles of attack, the air impinges directly the deck and the train. In front of the train a stagnation area is created, presenting therefore high pressures. These pressures balance the contribution of the lower surface to the lift coefficient, and at -6° , the lift is almost zero. As the angle of attack increases, this high pressure is reduced, and the contribution of the lower surface increases, increasing therefore the overall lift coefficient. For positive angles of attack, the shear layer shed in the leading edge of the bridge impinges the train and closes a bubble, that increases the lift. The effect of the porous barriers follows the same trend, but reducing the overall value. This is due to the turbulence introduced in the flow by the tubes forming the barriers and the smaller size of the recirculation bubble compared

to the case with solid barriers.

(a)

(b)

(c)

Fig. 2.9: Aerodynamic moment coefficient for the bridge section. (a) without train, (b) with the train in the windward position and (c) with the train in the leeward position. Legend explained in table 2.

Regarding the moment coefficient, the measurements shown in figure 2.9. The deck without train, figure 2.9a, presents a negative value of the coefficient, regardless the barrier. The moment coefficient goes to more negative values as the angle of attack increases for the porous barriers or the single deck. As α increases, it reaches a constant value, which remains unchanged for all the positive angles of attack. The reason for the decrease in the moment coefficient is that lift, whose resultant force is not located in the center of mass but forward, increases its value with the angle of attack (figure 2.8a). Configurations with solid walls do not alter their behavior with the angle of attack. This means that the rotation point, which is intentionally the center of mass, is also the aerodynamic center of the section, where no variation of the moment coefficient with the angle of attack is found.

The presence of the train does not influence the moment coefficient for the solid barriers. Figures 2.9b and 2.9c show clearly that the moment is independent of both the angle of attack and the presence or absence of the train. The value of the moment coefficient remains constant, ranging in values from -2 to -2.5. It is remarkable that in this case, the curved barrier seems to have more negative values than the straight barriers.

Configurations with porous barriers, on the other hand, have a decreasing value of the moment coefficient with increasing angles of attack, when the train is present. The slope is steeper when the train is in the leeward position, figure 2.9c, since at the most negative angle of attack, -6° , the value is close to 0, and at 6° , the values for the train in any of the railways is the same, around -1.5.

2.3.2 Dynamic effects on bridges

The raw results of the ambient vibration tests are the time histories registered for each wind speed steps. For instance, in figure 2.10, in the bottom graph, the response in bending of vertical position h for the VIV wind velocity is presented, and in the top graph, the response in bending for a lower wind velocity. The difference in amplitude and frequencies of the signal is clear.

Fig. 2.10: Evolution of the vertical position, h , with time, t . Dynamic response of the bridge in bending at two different wind speeds: (b) the response at a velocity lower than the VIV critical speed, (a) the response at the VIV velocity.

To analyze globally the measurements, the normalized root mean square (RMS) value of each time history is calculated as:

$$h^* = \frac{h_{RMS}}{D} = \frac{1}{D} \sqrt{\frac{1}{N} \sum_{i=1}^N h_i^2} \quad (15)$$

where N is the number of samples acquired for each velocity, D the model dimension normal to the incident wind, and h_i is the displacement at the time instant i . h^* is a non-dimensional measure of the amplitude of vibration.

The results are plotted in figures 2.11 and 2.12. In both cases, the normalized RMS defined in equation 15 is plotted as a function of the prototype incident wind velocity. This velocity is defined as

$$U_p = \frac{D_p}{D_m} \frac{f_p}{f_m} U_m \quad (16)$$

where D_p/D_m is the geometric scale (ratio between prototype deck depth and model deck depth), f_p/f_m is the time scale (ratio between prototype bending frequency and model bending frequency) and U_m is the wind velocity measured in the model scale (incoming velocity in the wind tunnel). In both figures, configuration with no barriers (SP) has been plotted to make comparison easier.

Fig. 2.11: Normalized RMS of the vertical displacement, h^* , of the bridge model with straight barriers as a function of the prototype scale incident wind velocity. Legend explained in table 2.

Fig. 2.12: Normalized RMS of the vertical displacement, h^* , of the bridge model with curved barriers as a function of the prototype scale incident wind velocity. Legend explained in table 2.

The bare deck presents VIV phenomenon at a prototype velocity of 40 m/s. In this case, the movement is not so strong. No other instabilities have been seen for this configuration. When the

straight barriers are added (figure 2.11) great differences are appreciated. First of all, with the porous barriers, the VIV velocity is increased (the peak appears at $U_p = 52$ m/s), and also is the amplitude of the motion (more than doubled). The configuration with porous barriers does not present any other type of instability. This result is in agreement with results presented by Wardlaw in [119], where he found that VIV response is altered if the handrails are covered.

Solid barriers change not only the value of the critical velocity at which VIV happens (reducing it to $U_p = 30$ m/s), and its amplitude (being it sixfold the amplitude of vibration of the bare deck), but also the behavior at higher velocities. In this case, from a velocity of $U_p = 70$ m/s on, the response constantly increases, following a curve close to a parabola. This is the typical response to a buffeting effect [143, 144]. The reduction of the VIV velocity will be explained later.

Buffeting is usually linked to upcoming flow turbulence (see section 1.2.3), but another component of turbulence can also trigger the movement [145]. The signature turbulence is the turbulence that the object itself introduces into the flow [146]. This turbulence can be the reason for which buffeting appears in this case and not in the others. It is important to remember that the wind tunnel in which the tests have been performed has a turbulence intensity of $I_u = 2\%$, so turbulence from the upcoming flow can be discarded as the motion trigger. Galloping could also be responsible for the vibration of the bridge. Meseguer et al. [15] cite the importance of the reattachment of the recirculation bubble/wake in galloping. In the case of solid barriers, a big wake/bubble is created. However, this section does not fulfill the Glauert-Den Hartog criterion (see [89, 147] for more information), which is a quasi-static approximation to galloping.

For the curved barriers, figure 2.12, results are very similar to the straight barriers. VIV amplitude peaks are more intense for the curved barriers (threefold for the porous barriers, sevenfold for the solid barriers), but the velocities at which they reach the maximum are very close to the velocities for the straight barriers. High-velocity instabilities are also negligible for the porous barrier, but the solid curved barrier again suffers them. At first, the increase can be related to buffeting due to the signature turbulence, but at very high velocities the sudden growth suggests the presence of bending flutter. Just one measurement was done after the growth, so it is difficult to say whether flutter is the cause or there is another reason. Anyway, this growth happens at a really unlikely velocity to appear at any point in the surface of the earth, so it is not of interest.

To further study the effect of porosity on bridge dynamic stability, some other configurations of barriers have been considered. These new barriers are more solid than the porous barriers, but more porous than the solid ones. For that, several patterns of tapes have been placed on the porous barriers. The tape is opaque for the air, but does not significantly change the other properties of the section (mass, mass moment of inertia, frequency, etc.). Therefore, values presented in table 1 are still valid.

The set of six new barriers tested in the wind tunnel for dynamic stability is presented in figure 2.13. The code name for them can be seen in the lower part of each sketch. These barriers are continuous along the length of the bridge. The first four configurations, B-1 to B-1-2-3-4, are generated by adding strips to the porous straight barrier (barrier model BR) one by one, from the lower point of the barrier to a height of 7.8 cm, which is 1.2 cm smaller than the solid barrier. This height corresponds to 2.57 m in the prototype scale. Another two barriers have been tested by adding just the stripes 1 and 3, and 2 and 4 respectively, but always keeping the longitudinal continuity.

Fig. 2.13: Alternative bird protection barrier configurations (continuous along the barrier length).

Dimensions are in cm, being inside the brackets the dimensions in the model scale, and outside the brackets the dimensions in the prototype scale.

Yet another set of barriers have been tested, but in this case barriers are discontinuous. A sketch of these configurations is shown in figure 2.14. The coding is in the lower part of each sketch. T-2 is the configuration where two gaps between vertical tubes are covered and another two are left uncovered, and T-3 is the configuration where three gaps between vertical tubes are covered and another three are left uncovered.

Fig. 2.14: Alternative bird protection barrier configurations (discontinuous along the barrier length). Dimensions are in cm, being inside the brackets the dimensions in the model scale, and outside the brackets the dimensions in the prototype scale.

The results for the new barriers are shown in figures 2.15 to 2.18. In these figures, the reference configuration has not been plotted. The results for the first set of new configurations, B-1 to B-1-2-3-4, adding tape strips longitudinally are shown in figure 2.15. In this case, there is a clear trend in the graph: when more strips are added, the critical velocity is constantly reduced. The reason for that can be seen in the fact that the critical velocity is inversely proportional to the cross dimension of the object [148]. Therefore, the thicker the object, the smaller the critical velocity. This is what happens when strips are added.

At higher velocities, the response in all configurations tends to gradually increase in amplitude, but it is not so significant. Only in one case, B-1-2-3 presents a sudden increase that could correspond to flutter, but, once again, the velocity is too high and only one point of the post-critical behavior has been recorded, so it will not be analyzed as such.

Fig. 2.15: Normalized RMS of the vertical displacement, h^* , of the bridge model with alternative bird protection barrier configurations, as a function of the prototype scale undisturbed wind velocity. Legend explained in figure 2.13.

It is interesting to point out that for the increasing height of the solid barrier, the VIV critical velocity is smaller. This fact was empirically stated by Shiraishi and Matsumoto [148, 149] as:

$$U_{crb} = \frac{1}{0.6n} \frac{B}{D} \quad (17)$$

$$U_{crt} = \frac{1}{0.6(0.5 - n)} \frac{B}{D} \quad (18)$$

where n is the number of the harmonic to consider ($n = 1$ for the natural frequency, $n = 2$ for the harmonic, and so on), B the bridge width and D the bridge height. Equation 17 is for the critical reduced velocity in bending motion, U_{crb} and equation 18 for torsional motion U_{crt} . This model is valid for bluff bodies.

Fig. 2.16: Critical reduced velocity, V_r , as a function of the slenderness ratio, B/D for different bluff bodies. White dots represent vibration due to the lock-in phenomena, and black dots represent vibration due to the Kármán vortices. From [149].

Fig. 2.17: Normalized RMS of the vertical displacement, h^* , of the bridge model with alternative bird protection barrier configurations, as a function of the prototype scale undisturbed wind velocity. Legend explained in figure 2.13.

The results for configurations B-1-3 and B-2-4 are shown in figure 2.17. The difference between them is clear. The first one has a VIV critical velocity of $U_p = 50$ m/s, and the second one increases it up to $U_p = 80$ m/s. Beyond that, none of them presents any other instability (flutter or buffeting).

The reason for this difference is that for configuration B-1-3, the behavior is very similar to configuration B-1 in figure 2.15. Therefore, the second strip does not affect significantly the flow condition. The extra stripes were added to break the Kármán vortex street, but this stripe is not enough to interact with the vortex street shed from the deck. Meanwhile, configuration B-2-4 does not match any behavior seen before. In this case, both the second and the fourth strips introduce the vortex street, that, interacting with the vortex street shed by the deck, delay the trigger of the motion.

One of the known solutions for reducing dynamic effects on bridges is to break the longitudinal coherence so that each section of the bridge is suffering a different load, and coupling along the bridge axis is avoided. To do this, the barriers presented in figure 2.14 are installed. The results for these barriers can be seen in figure 2.18.

Fig. 2.18: Normalized RMS of the vertical displacement, h^* , of the bridge model with alternative bird protection barrier configurations, as a function of the prototype scale undisturbed wind velocity. Legend explained in figure 2.14.

In this case, motion still exists, but the critical velocity has been greatly increased. For configuration T-2, VIV appears at $U_p \approx 75$ m/s and for configuration T-3, at $U_p \approx 85$ m/s. In this last case, the vibration amplitude is reduced in comparison to configuration T-2. In addition, no high-speed instabilities have been found in the studied wind velocity range. Therefore, it is easy to conclude that these configurations are better for the stability. The reason for that is that the breakage of the longitudinal coherence leads to three dimensional effects. These effects induce non-coherent loads in the bridge section, that lack the needed continuity to trigger any movement. The three-dimensional effects are more developed in the T-3 configuration. However, they are not strong enough

to completely eliminate the VIV, because the lower surface of the deck still sheds its own vortices which are not altered by the change in the barriers.

2.3.3 Static loads of trains

Force and moment coefficient definitions (per unit length) vary slightly compared to those used in equations 12 to 14:

$$c_d = \frac{d}{q_\infty h} \quad (19)$$

$$c_{m_y} = \frac{m_y}{q_\infty h^2} \quad (20)$$

where c_d and c_{m_y} are the aerodynamic coefficients per unit length, d and m_y the aerodynamic drag and the overturning moment per unit length, q the dynamic pressure and h the height of the train model. For the loads on the train, lift force is not interesting, since the main problems regarding derailment arise from the overturning moment. It is important to remark that the moments have been measured with respect to the center of the balance, which is in touch with the train model (see figure 2.4 for the axis system displayed in the same figure).

The mean values of the drag and overturning moment coefficient are displayed in figures 2.19 and 2.20, respectively, in the axis system defined in figure 2.4. The drag coefficient presents an expected result. Porous barriers reduce slightly the drag compared to the case without barriers when the train is in the windward position. When solid barriers are placed, the wind load is reduced and its sign changes. This can be explained considering the fact that the train is in fully immersed in the wake of the barrier. The wake behind the barrier is a low pressure area, that tends to suck the train, and negative drag is created. Therefore, a negative value of the resistance appears for the solid barriers in figure 2.19a. No big differences can be appreciated in the drag coefficients when the angle of attack changes. Therefore, it can be concluded that, regardless the angle of attack, the train model is always in the wake of the barriers.

In this case, the influence of the angle of attack of the wind flow does not have a significant influence. Porous barriers provide less protection to the trains, so the aerodynamic performance of the train varies a bit with the angle of attack. In this case, as the angle of attack grows, the train gets into the wake of the deck, and therefore the drag is reduced.

Wind loads on the train model with the train placed in the leeward railway, figure 2.19b, have a different behavior. Once again, the porous barriers reduce the drag compared to the situation without barriers, but in this case the reduction is bigger than with the train placed in the windward

railway, specially for negative angles of attack. When the angle of attack is increased, the whole train is immersed in the wake of the deck, so the drag coefficient drops to half of the value respect the initial angle of attack. The barriers with solid parts induce a similar behavior, independently of the position of the train. In the case of the train in the leeward position, the minimum of the drag coefficient is more evident. This minimum happens at an angle of attack of 0° .

The train placed in the windward position suffers higher loads than in the leeward position, specially for configurations with porous barriers or no barrier. In the case of the train in the windward position, the wake of the barriers is not fully developed, therefore, the protection they provide to the train is smaller. In the leeward railway, the wake has had more time to develop, so it is more turbulent, and it reduces overall the loads.

Fig. 2.19: Drag coefficients, c_d , for the train model as a function of the angle of attack, α , with the different barrier configurations. (a) presents the drag coefficient when the train is placed in the windward railway and (b) the drag coefficient when the train is placed in the leeward railway. Legend explained in table 2.

The case of the overturning moment is very similar to the one of the drag coefficient (figure 2.20). This is an very reasonable result, since the main contribution to the moment coefficient in this experiment comes from the drag. Once again, a clear difference can be appreciated between the train at the windward or in the leeward position. In the second case, the train is more protected, even for the configurations with high porosity, due to the protection of the deck itself. This effect is more clear at high angles of attack.

In the windward position, the train is exposed more openly to the upcoming wind, and the effect of the protection of the deck is negligible. The main effect for that position is the presence of solid/porous barriers. Solid barriers reduce the absolute value of the overturning coefficient and even change its sign, following the same reason explained above for the drag coefficient.

Fig. 2.20: Overturning moment coefficients, c_m for the train model as a function of the angle of attack, α , with the different barrier configurations. (a) presents the moment coefficient when the train is placed in the windward railway and (b) the moment coefficient when the train is placed in the leeward railway. Legend explained in table 2.

2.3.4 Catenary flow condition

From the measurements of the wind speed in the catenary positions, see section 2.2.4, the non-dimensional velocity components are defined as:

$$u^* = u/U_\infty \quad (21)$$

$$v^* = v/U_\infty \quad (22)$$

where u is the horizontal velocity component measured at the catenary point, v is the vertical velocity component measured at the catenary point, and U_∞ is the undisturbed velocity, measured with the Pitot tube placed upstream the model.

Another interesting parameter to study is the turbulence intensities in both directions (in the direction of the flow and in the vertical direction). They can be computed as:

$$I_u = \sigma_u/U \quad (23)$$

$$I_v = \sigma_v/U \quad (24)$$

where σ_u and σ_v are the standard deviations of, respectively, u and v , and $U = \sqrt{u^2 + v^2}$ is the modulus of the measured velocity. The variation of these magnitudes with the angle of attack is shown in figures 2.21 to 2.24.

The flow condition in the position of the windward catenary for the deck without barriers and the four initial barriers is shown in figure 2.21. In the top graph, the non-dimensional velocity can be seen and in the bottom graph, the turbulence intensity. Shielding of the porous barriers, either straight or curved, is almost negligible. Comparing with the bare deck, only at high angles of attack (higher than 3°), the porous barriers make any difference in the reduction of the mean velocity. For solid barriers, the shielding they provide increases with the angle of attack, and the reduction of mean velocity is around 80% for an angle of attack of 6° . Regarding the turbulence intensity, the behavior with porous barriers again do not differ much from the behavior of the bare deck. Meanwhile, solid barriers increase the longitudinal turbulence intensity constantly for higher angles of attack, until it reaches a stable value of more than 50%. The increase in the turbulence intensity is due to the fact that the shear layer that detaches from the edge of the solid part of the barrier impinges not directly at first (small angles of attack), but its deflection reaches the position of the windward catenary for large angles of attack. No big differences can be appreciated between straight and curved barriers.

The shear layer is a thin zone of the flow in which a great gradient of velocity is present containing a vortex sheet that usually separates the free flow from a recirculation bubble or a wake. Because

of its own nature, it is an area with high turbulence intensity. It is straightforward to conclude that when the angle of attack is between -3° and 0° , the shear layer is directly impinging the windward catenary. This can be affirmed because the velocity value is changing from the value of the free flow to the value of the recirculation bubble or wake, therefore it is decreasing, and the increasing value of the turbulence intensity, which is increasing from the free flow value (low turbulence) to the high turbulence value in the wake or the recirculation bubble.

Fig. 2.21: Non-dimensional longitudinal velocity, u/U_∞ , (top) and turbulence intensity, I_u , (bottom) as a function of the angle of attack, α for the catenary placed windward.
Legend explained in table 2.

Figure 2.22 presents the same plots but for the vertical component. In this case, the behavior is different to the one described before. Mean velocity value decreases with the angle of attack for the bare deck and the porous barriers. But in this case, it looks like the effect of the porous barriers is higher than in the longitudinal velocity case. For high angles of attack, the behavior for the bare deck and the porous barriers split. In the case of the porous barriers, vertical velocity can reach up to 25% of the modulus of the velocity. Regarding the turbulence intensity, porous barriers also differ from the bare deck case for high angles of attack, but the values of turbulence intensity reached do not exceed 10%.

Solid barriers, instead, slowly increase the value of the mean velocity in the position of the catenary for increasing angles of attack. Finally, at 6° , the vertical component of the velocity vanishes. Turbulence intensity increases for larger angles of attack. In this case, it can reach values higher

than 30%.

It is difficult to extract physical conclusions for the vertical velocity, since flow can be really complex in the wake of a bluff body.

Fig. 2.22: Non-dimensional vertical velocity, v/U_∞ , (top) and turbulence intensity, I_v , (bottom) as a function of the angle of attack, α , for the catenary placed windward. Legend explained in table 2.

The results for longitudinal velocity in the leeward catenary are presented in figure 2.23. Both the mean velocity and turbulence intensity behave very close to the measurements done in the windward catenary. Porous barriers have a very similar behavior to the bare deck configuration. Only at high angles of attack, there is a small difference in mean velocity between them. Regarding the turbulence intensity, porous barriers and the bare deck have a similar behavior, not being affected by the change in the angle of attack. The value of the turbulence intensity in these cases is very small ($I_u < 5\%$).

When the barriers are solid, the mean velocity is reduced for larger angles of attack. In fact, it seems that the decrease of the mean velocity is stopped or greatly reduced at high angles of attack. The reason for that is that at those angles, the solid wall totally covers the leeward catenary, and therefore it is submerged in the wake. The measurements in the smaller angles of attack are still done in the shear layer, but the last two are in the limits or even outside the shear layer. The results in turbulence intensity point in the same direction. It increases for negative angles of attack, but

at positive angles of attack it remains almost constant, at a value of 50%. Same conclusion can be taken: for high positive angles of attack, the leeward catenary is immersed in the wake that it forms with the bridge deck. In the wake, mean wind velocity is low, and turbulence intensity high.

Fig. 2.23: Non-dimensional longitudinal velocity, v/U_∞ , (top) and turbulence intensity, I_u , (bottom) as a function of the angle of attack, α , for the catenary placed leeward. Legend explained in table 2.

In figure 2.24, the vertical velocity and turbulence intensity in the position of the leeward catenary are shown. When porous barriers are installed, a decreasing trend in the vertical mean velocity can be appreciated, which is not visible for the bare deck configuration. Even though there is a variation in mean velocity, turbulence has an almost constant value in all the angles of attack, and it is $I_v < 5\%$.

For solid barriers, the reduction of mean velocity with the angle of attack disappears, and the vertical component of the velocity can be almost neglected (figure 2.24). The velocity in the leeward catenary is thus horizontal. In this case, the behavior is closer to the bare deck situation. Turbulence intensity in the vertical direction is not greatly affected by the angle of attack for the solid barriers, ranging always between $I_v = 20\%$ and $I_v = 35\%$. In this case, it looks like the solid barriers are introducing a fluctuation of the vertical velocity with zero mean value.

Fig. 2.24: Non-dimensional vertical velocity, v/U_∞ , (top) and turbulence intensity, I_v , (bottom) as a function of the angle of attack, α , for the catenary placed leeward. Legend explained in table 2.

3 Flat plate tests

This section is focused on the tests performed on a flat plate model in the Centro Interuniversitario di Aerodinamica delle Costruzioni e Ingegneria del Vento, CRIACIV, located in several cities across Italy with its headquarters in Florence.

In the host research center, many research has been performed regarding aeroelasticity in general and the stability of the bridges in particular [69, 82, 150, 151]. In the last times, another application of the aerodynamic instabilities has risen: energy harvesting. Energy harvesting is intended to collect relatively small amounts of energy. Several proposals for energy harvesting are presented in the introduction of [152]. Research has been made in many areas to obtain energy-collecting distributed systems. These systems seem to be a good step in reducing carbon print and leading the path towards a distributed generation network [153]. Caracoglia presents some of the proposed apparatuses and introduces a new one in [154]. Therefore, it is clear that this investigation line is far from being over, and many researchers work on it [147, 150, 155].

The two approaches to aeroelastic instabilities aim different objectives: the highest stability and critical velocity is desired in bridge design, the lowest critical velocity is desired in energy harvesting, since it means that energy can be collected from even the smallest wind speed. However, the determination of the critical velocity is a common objective (some other objectives differ), and therefore the two approaches can find a common path.

This section is divided in 4 subsections. In subsection 3.1 the section characteristics are presented. In 3.2 the experimental set-up is described, which, in this case, differs significantly from the classical set-up for dynamic tests presented in subsection 2.2.2. This set-up has been developed during the stay of the author at the host institution. In subsection 3.3, main results are summarized and finally, in subsection 3.4 an analysis of the results is performed.

3.1 Section characteristics

In this case, the section is just a flat plate, with an aspect ratio of 25. To this plate, several barriers have been attached, with different porosities. The barriers have been built from commercial mesh. The dynamic and geometric characteristics of the flat plate tested are summarized in table 3. A clarification is needed regarding the model length. The model is 517 mm long, but the part of the model submerged in the flow is 500 mm. The rest of the model is used to fix it to the set-up.

Tab. 3: Main mechanic and dynamic properties of the flat plate.

Property (units)	Value
Model height (mm)	4
Model depth (mm)	100
Model length (mm)	517
Mass of the model per unit length (kg/m)	3.34
Mass moment of inertia per unit length ($\text{kg} \cdot \text{m}^2/\text{m}$)	4.19×10^{-6}

To design the barriers, commercial mesh has been used. The meshes employed were selected by their porosity. Porosity is usually defined as:

$$\phi = \frac{A_{hole}}{A_{front}} \times 100 \quad (25)$$

where ϕ is the porosity (in percentage, $\phi = 0 \%$ meaning solid barriers and $\phi = 100 \%$ meaning no barrier, or totally porous barrier), A_{hole} is the area of the holes in the mesh, and A_{front} is the total area of the barrier.

Two different section shapes have been tested. The H-shaped section has been tested with barriers with porosity $\phi = 0 \%$, $\phi = 25 \%$, $\phi = 50 \%$ and $\phi = 100 \%$ (which is just the rectangular model and can be used as reference). The U-shaped section has been tested with barriers with porosity $\phi = 0 \%$, $\phi = 50 \%$ and $\phi = 100 \%$ (which is used as the reference and to make it easier to compare with the H-shaped section). When the barriers are installed, the section aspect ratio (B/D) is reduced to 12.5, instead the 25 of the bare section.

The reason for the selection of these two sections is that they are known to be unstable at lower section aspect ratios. Therefore, the hypothetical elimination of the vibration can be totally attributed to the barrier porosity.

The set of experiments performed with different configurations on this section is shown in the 'Flat plate H-section' and 'Flat plate U-section' blocks of table 8.

3.2 Experimental set-up

The usual dynamic set-up for the test of dynamic bridge sections is the one already explained in section 2.2.2. However, the set-up used in these tests was different. This set-up has the advantage of a continuous control of the vertical frequency, while with the system of section 2.2.2 the frequency step is conditioned by the available spring combination.

For following the explanation of the set-up, two sketches are shown in figures 3.1 and 3.2. A detail of the system is displayed in figure 3.1, focusing on the most complex zone, and the whole system is shown in figure 3.2.

First of all, two columns are installed inside the wind tunnel. These columns have two lateral plates each (one in a higher position and the other one in a lower position), with two plates that work as a clamp. They are not shown in the figures. In each clamp, an aluminum beam is installed. This beam has a width of 50 mm and a thickness of 3 mm. The length of the beam is 1.2 meters. In figure 3.2, the four beams are presented in turquoise. Adjusting the position where the beams are clamped, the vertical frequency is adjusted.

At the end of these beams, a vertical Vierendeel beam is installed, joining each two aluminum beams. The system described so far provides the vertical stiffness. The Vierendeel beam makes the two aluminum beams behave as a single complex beam. The Vierendeel in figure 3.2 is coloured in turquoise, and each one of them joins the two initial beams.

For providing the rotational stiffness, the Vierendeel beam has a hole in the middle. In that hole, a bearing is placed. The bearing is the yellow part in figure 3.1, which will be the reference figure for the upcoming description. In the center of that bearing, there is a hole. The inner part of the bearing is fixed to a cylinder that has a prism on one end. In the same figure, this piece is depicted in green. The cylindrical part has a groove to which a rotational spring is attached. The spring is represented in the figure as a purple spiral. The other end of the spring is fixed to a T-shaped piece that is joined to the Vierendeel. In the preliminary design, this piece was just a prism and had no T-shape, and that is what can be seen in figure 3.1. The prism has a inner round hole. One of the sides of that prism is detachable from the rest, so it works as a clamp. In the upper view shown in figure 3.1, the position of the four screws that are used to remove the detachable part can be seen. This hole is useful to set different steady angles of attack to the model. In that hole, another piece with a cylindrical end is placed. In the detailed figure, this piece is shown in red. At the other side of this piece, there is another prism. In this case, the prism has a groove and two bolts, that clamp the model (in grey). Between the bolts and the model subject to the flow, a circular endplate reduces the three-dimensional effects. The last piece of the system relevant to this study is a beam placed over the green piece. This beam, in purple in the figure 3.1, allows to place extra masses and changing thus the elastic center. Even though at first they were not planned to be used, accelerometers have been finally used to measure the rotation of the model, and they have been placed in this piece.

To avoid the disturbances that all the system could induce in the flow, a carter has been designed that covers the main part of the system (aluminum bars, Vierendeel beam, rotational spring, the first cylinder-prism piece). These carters (one for each side) reduce the effective area of the wind tunnel, so therefore a increase in the wind velocity is expected in the measurement area compared to the

Fig. 3.1: Detail sketch of the setup for the flat plate tests.

Fig. 3.2: Overall sketch of the setup for the flat plate tests.

free flow. The leading edge of the carter, which can be seen in figure 3.3, has been manufactured with a thin iron layer, and the rest, with wood beams and methacrylate.

Fig. 3.3: H-shaped section, carter to avoid three dimensional effects in the background and endplate (circular plate).

The system is now almost fully characterized. The last components of the set-up are 2 sets of springs, which are placed outside the wind tunnel. In the upper and lower aluminum beams, a wire is tied, and it crosses the ceiling/floor of the wind tunnel. Outside the wind tunnel, a structure is placed. In that structure, a spring is fixed, and the wire from the wind tunnel is fixed to the other end of the spring. This way, an extra stiffness can be provided to the system.

For the study of the effect of the presence of the carters, a test has been performed where a Pitot tube has been placed in the intake of the wind tunnel (just downstream the contraction) and another one in the test chamber, where the studied section will be placed. The procedure is to fix a frequency in the fan and record the measurement of both Pitot tubes simultaneously. The acquisition program directly calculates the average of the wind velocity. The measured results are shown in table 4 and in figure 3.4. In that figure, the triangles represent the measured velocity, the blue line represents the least-squared adjusted values, and the red line represents the case of $U_{upstream} = U_{downstream}$. The value of the slope of the adjusted values of the velocity is 1.101, i.e., the presence of the carters increases the velocity perceived by the model in a 10 %.

Tab. 4: Velocity relationship between the downstream and upstream Pitot tubes' measurements.

Fan frequency	$\frac{U_{downstream}}{U_{upstream}}$
5	1.142
10	1.128
15	1.122
20	1.118
25	1.114
30	1.104
35	1.101
40	1.100
45	1.097
50	1.096

Fig. 3.4: Velocity conversion between the undisturbed flow and the flow in the measurement area.

Even though the discrepancy is small, the correction factor used in the tests has not been the calculated with the adjusted values, but with the slope of the velocity in the immediate higher and

lower points measured in table 4.

To perform the tests, the wind tunnel in the CRIACIV facilities has been used. This wind tunnel is an atmospheric boundary layer simulation wind tunnel. It has a rectangular section, 2.4 m wide and 1.6 m high. The air is driven by a single fan, and can reach velocities of 35 m/s. Having only one fan makes more complex the design of the diffuser, since the shape has to become a circumference from a rectangle. The contraction is in this case made in the two dimensions, since the room where it is placed is bigger. A honeycomb has been installed in the contraction to improve the flow condition and to reduce the vertical and lateral velocity components. The whole room is used as the return duct, and there are two areas covered from the wind that are used as control room and as workshop. It is a bit shorter than the previously described ACLA16, being its length of 10 m. The free stream turbulence is around 1 %.

3.2.1 Bending and pitching stiffness characterization

As the set-up had just been designed, first step was to characterize it. The first characterization needed is the stiffness of the system. As two degrees of freedom are considered, two stiffnesses have to be characterized: bending and pitching.

To characterize the bending stiffness, different loads have been applied to the model in the center, and the displacement have been recorded. The loads have been applied by placing different masses. The pitching stiffness is measured by placing the masses not in the vertical line of the center of gravity, but with a displacement, that creates a moment. Pitching of the model is measured both with a laser and an inclinometer, because with high loads, laser beams do not impinge in the model due to the high rotation.

The results of the bending stiffness characterization can be seen in figure 3.5. It is clear that the system behaves linearly. The calculated stiffness is 1132 N/m. Before and after applying the loads, the displacement has been measured, and no significant difference has been found (no permanent deformation was noticed during the tests campaign).

Fig. 3.5: Bending stiffness of the set-up. Squares mark the measured points, and the discontinuous line represents the linear fitting.

Pitching characterization results are shown in figure 3.6. Two ranges have been studied, one with smaller torque and one with bigger torque. In the figure, low range of loads are presented in blue (triangles are the measured points, and the line is the linear fitting), and the displacements have been measured with inclinometer. High range of loads have been measured both with the inclinometer (black circles are the measured points, black line the linear fitting) and the laser sensors (red squares are the measured points, red line the linear adjustment).

Tab. 5: Values of the pitching stiffness for each case.

Case	Pitching stiffness (Nm/rad)
Low range of loads (with inclinometer)	3.89
High range of loads (with inclinometer)	4.10
Low range of loads (with lasers)	4.18

Fig. 3.6: Pitching stiffness of the set-up. Squares, circles and triangles mark the measured points, and the lines represent the linear fitting of each case (high load measured with laser, high load measured with inclinometer, low load measured with inclinometer).

In this case, the stiffness results are shown in table 5. The low load case has a slightly smaller stiffness, around 5 %, which is acceptable.

3.2.2 Frequency and damping characterization

The first step of the research was to estimate the vertical frequency of the system and the maximum stress, depending on the length and section of the beams, the material, the maximum displacement expected and the mass of the system. The problem is a classic beam problem, as sketched in figure 3.7.

Fig. 3.7: Reduced beam system to study the vertical frequency of the system.

where M is the end mass (the model and the rest of the system), L the length of the beam, E the elastic modulus and I the mass moment of inertia. Applying basic structural knowledge to calculate the natural frequencies of beams (see [156]), the frequency and the maximum stress expressions for this system is:

$$f = \frac{1}{2\pi} \sqrt{\frac{2 \times 12EI}{L^3 \left(M + \frac{4bdL\rho}{3} \right)}} \quad (26)$$

$$\sigma = \frac{6E\delta d}{L^2 \times 10^6} \quad (27)$$

where $b = 0.05$ m is the width of the beam, d the height of the beam, and δ the expected maximum displacement. The 2 inside the square root means the presence of two beams (the upper one and the lower one). The selected beam has a rectangular section, being the width fixed to $b = 0.05$ m. The area moment of inertia is $I = \frac{1}{12}db^3$ for a rectangular section.

The estimated end mass is 4.756 kilograms (therefore, each side has to deal with 2.378 kg). The distribution of masses is given in table 6.

The maximum displacement is 10 cm (because of the measuring range of the lasers). Imposing these known values (end mass, maximum displacement, width of the bar), a numerical study can be conducted to check the influence of the thickness, the length of the beam and the material (either aluminum or steel can be selected). It should be remarked that all these calculations have been made under the assumption of a static situation, and some dynamic effects have been left aside.

Under all these assumptions, an estimation of the obtained frequencies and stresses is displayed in

Tab. 6: Vibrating mass.

Element	Number of elements	Mass per element (gr)
Torsional spring	2	142.5
Fixing Ts	2	34.5
Steel hook	2	211
Bar for extra mass	2	42.5
Bearing	2	112
Aluminum hook	2	117
Linear spring	4	106
Endplate	2	482
Ls for the endplates	2	40.5
Model	1	1670
Virendeel beam	2	361

figure 3.8. In that figure, the x axis presents the length of the beam and the different colors on the legend represent the thickness of the beam in meters.

Fig. 3.8: Expected frequencies (upper figure) and stresses (lower figure) for long beams as a function of the length, for several values of the thickness of the beam (see legend, in meters). Results of equations 26 and 27.

After watching these results, aluminum bars have been selected, with 3 mm thickness. The length of the beams is 1.2 m and the effective length (length of the beam that will be working as a spring) is 1.04 m. This gives an expected value of the frequency of 1.2 Hz.

After the numerical characterization of the model, the characterization of the actual set-up must be performed. For that, an initial condition is given to the model and the response is recorded. Three tests have been done, one in bending, locking the pitching motion, one in pitching, locking the bending motion, and finally one with both degrees of freedom free. For each case, three repetitions have been made, in order to ensure the result. Locking of the bending motion is done by fixing the long beams to the floor and ceiling, avoiding thus the vertical movement. Torsional motion is locked by using a cable tie to join the Vierendeel and the beam used to add extra mass.

The bending and pitching displacements are calculated from the measurements of the lasers and the accelerometers, respectively. Equations 28 and 29 show the calculations to extract the displace-

ments:

$$h = \frac{d_1 + d_2}{2} \quad (28)$$

$$\alpha = \frac{a_1 + a_2}{b_1 + b_2} \quad (29)$$

where d_1 and d_2 are the displacements measured by the laser sensors, a_1 and a_2 are the displacements measured by the accelerometers, and b_1 and b_2 are the distances of the accelerometers to the rotation axis of the model. To obtain the displacements from the accelerometers, which have an output of acceleration, the signal is twice integrated and filtered with a Butterworth filter.

The frequency characterization took many attempts. Even the smaller modification in the mechanical set-up implied a change in the frequency or the damping (which has also been characterized).

The signal obtained was analyzed with the Modified Unifying Least Squares system (MULS), developed by the host institution (CRIACIV, see [69]) based on the Unifying Least Squares system (developed by Gu et al., see [68]). This system allows to study accurately not only the frequency of a system but also the damping.

The results for the frequency test are displayed in figures 3.9 to 3.11. It is clear that no big difference can be appreciated. In the single degree of freedom tests, biggest discrepancies in frequency do not reach 0.25 %. When both degrees of freedom are left free to vibrate, however, a bigger variation can be seen in the first test for the pitching degree of freedom, but the two other tests are in better agreement, and this value will be taken as the good one. With these results, it can be stated that the ratio of frequencies is $\gamma = 1.11$.

Fig. 3.9: Frequency of the three repetitions for the bending degree of freedom.

Fig. 3.10: Frequency of the three repetitions for the pitching degree of freedom.

Fig. 3.11: Frequency of the three repetitions for the two degrees of freedom combined.

Damping was also studied. For bridge testing, damping is sought to be as small as possible, to ensure conservative results. In addition, good characterization of damping is important for the energy harvesting studies, since the starting of the motion is linked to the appearance of negative values of damping, which are the objective of the CRIACIV. Damping is presented in figures 3.12 to 3.14. Damping results are always more difficult to extract, and present therefore a bigger variation between tests. However, very low values are obtained in the single degree of freedom tests (figures 3.12 and 3.13), specially when they are compared to characteristic damping values of real bridges. When two degrees of freedom take part in the movement, pitching damping is dramatically increased. As happened to the frequency tests, first test has an abnormal value of the damping, and it should be discarded. Second and third tests deliver a more realistic value of the damping, and their average will be taken as the correct value. In this case, compared to the single degree of freedom, bending damping is not altered, but pitching damping is approximately doubled.

Fig. 3.12: Damping of the three repetitions for the bending degree of freedom.

Fig. 3.13: Damping of the three repetitions for the pitching degree of freedom.

Fig. 3.14: Damping of the three repetitions for the two degrees of freedom combined.

The total height of the barriers is 8 mm, giving overall an aspect ratio of 12.5. Tested configurations are displayed in figure 3.15. Even though it may seem a small barrier, it is known that, depending on their position, the presence of even tiny details can significantly vary the aeroelastic behavior of a body [131].

Fig. 3.15: Set-up of two of the configurations tested: (a) the solid barrier and (b) the barrier with $\phi = 50\%$.

3.2.3 Test procedure

The procedure of testing is the usual for free vibration tests: a wind velocity is fixed and some time is left for the wind flow and the model to reach a steady state. With a sampling frequency of 2000 Hz, the signal is recorded. The sampling time depends on the signal itself: if the amplitude of the signal seems to change, it is recorded for a longer time, otherwise it is recorded for 60 seconds. After recording the signal, the velocity is increased and the process is repeated. When the maximum amplitude that can be measured with the lasers is reached, the process is performed backwards, reducing the velocity.

Measuring backwards can help to understand the sub-critical vibration range. In this range, motion is not started automatically but if the section is given the appropriate initial condition, motion can be started and can remain. When the sub-critical range is entered from the critical range (which happens at higher velocities and means self-excited vibration), motion is usually maintained, even though the wind velocity is below the critical value.

In some of the measuring points, in the sub-critical zone, an initial condition has been imposed to study the stability. This condition has been imposed by pulling the wires that connect the lower springs with the floor.

The measuring equipment slightly varies depending on the type of test performed. For the velocity relation two Pitot tubes with their corresponding capsules have been used. For the free vibration tests, 3 lasers have been installed and used. Because of the difficulty of measuring high amplitude ranges of rotational motion, in some tests 2 accelerometers have been installed. The reason for that

is that when the laser hits a surface with a not-small angle, the measurement loses its accuracy and in the extreme case, the model laser beam is not reflected on the model because the angle is too high. In the vibration tests, angles up to 50° have been reached, so the lasers lost their ability to measure.

The system instrumentation also comprises another instrument which records the atmospheric pressure and temperature, in order to calculate the density which is needed to calculate the velocity by means of the dynamic pressure measured with the Pitot tube and the capsule.

3.3 Results

In this part, the results of the tests performed on the different configurations are displayed. In the tests both degrees of freedom have been left free to vibrate.

3.3.1 Critical velocity and vibration amplitude determination

The results for the four porosities with increasing values of velocity for the H-shaped sections are shown in figures 3.16 to 3.19. In those figures, p100 represents the model without barrier (or totally porous barrier). In the bending case, the non-dimensional root mean square value of the displacement has been displayed (see equation 15), and in the pitching case, the root mean square value of the angle in radians, α_{RMS} has been displayed. This value is an estimation of the amplitude of the oscillation

$$a_{RMS} = \sqrt{\frac{1}{N} \sum_{i=1}^N a_i^2} \quad (30)$$

where N is the number of samples acquired for each velocity, D the model dimension normal to the incident wind, and a_i is the pitching at the time instant i , in radians. a_{RMS} is a non-dimensional measure of the amplitude of pitching vibration.

When wind speed is increased, bending motion is triggered at a wind speed of around 12 m/s. With the totally solid barrier, the motion starts a bit later than the rest (highest stable velocity is 13.9 m/s). The configuration with the 50% porosity barrier has the smaller critical velocity, 11.6 m/s. Both the totally porous barrier and the barrier of 25% porosity have a critical velocity of 12.75 m/s. It is interesting to observe that even though the differences are small, the configuration which was expected to have the smallest critical velocity, p0, has indeed the highest one. The reason may be the high aspect ratio. This fact should be confirmed with further studies.

Fig. 3.16: Root mean square of the bending motion for increasing values of velocity for H-shaped sections.

Fig. 3.17: Root mean square of the pitching motion for increasing values of velocity for H-shaped sections.

Fig. 3.18: Root mean square of the bending motion for decreasing values of velocity for H-shaped sections.

At some velocities, two values of the displacement have been plotted. The reason for that is that around the sub-critical range, some velocities have been tested twice, one triggering the movement and the other one not triggering it. Once the movement has started, the response grows in a slightly concave trend. It is remarkable that the values of the displacement are higher than those obtained for common bridge sections, because the height of the model, with which the root mean square is made non-dimensional, is really small. These amplitudes are not desirable in bridge sections, but, as said, the host institution is studying energy harvesting from flutter phenomenon, and one of the objectives is to obtain big amplitudes. No difference in the results can be appreciated between the different porosities.

Pitching motion has a similar behavior. The motion is triggered at the same velocities, but the critical range has a smaller slope, and a less regular behavior. In this case, the absence of barriers seems to have a slightly bigger amplitude in the critical range, compared to the others.

After reaching the maximum displacement allowed by the lasers, the velocity is reduced and the process is repeated at decreasing velocities (figures 3.18 and 3.19). As expected, the motion now lasts for smaller velocities, i.e., some hysteresis phenomenon is present. All configurations keep their motion until at least 7 m/s. The first configuration to decay is the one without barriers. On the other hand, configurations with barriers keep the motion until 6 m/s. This is interesting for energy harvesting purposes, since the motion can be kept at smaller wind speeds (and externally triggered at smaller speeds).

In bending, shown in figure 3.18, the downslope is again a bit concave, and follows faithfully the same path as the results of the increasing velocity. It is remarkable the small values of the non-dimensional root mean square that have been recorded, which reach one tenth of the maximum RMS

Fig. 3.19: Root mean square of the pitching motion for decreasing values of velocity for H-shaped sections.

measured, approximately, i.e., the difference in vibration amplitude between the highest velocity and the lowest velocity within the sub-critical range is not negligible. In the low velocities of the sub-critical range, the measurements have been taken for long sampling periods (60 seconds at least), and flow stabilization time has been left between measurements, so the small amplitude is not attributable to a decreasing signal. This fact shows the good design and the low damping achieved.

In pitching, shown in figure 3.19, the configuration without barrier has again a slightly bigger amplitude than the other. This behavior is more remarkable at higher wind speeds. In case of the pitching movement, once again the slope seems smaller and with a bigger dispersion, even though a clear trend can be observed. In this case, the smallest RMS values observed are just a bit less than half of the maximum values. Therefore, at higher wind speeds, the bending movement gains more amplitude than the pitching movement.

The phase plane of the reference configuration, p100 is displayed in figures 3.20 (for increasing velocities) and 3.21 (for decreasing velocities). In both of them, the plots with brighter colors mean tests performed at higher velocity. They reaffirm the previously stated fact that at higher velocities, the increase in bending is higher than the increase in pitching, reaching up to 200 mm in amplitude, which is the limit of the sensors. In both graphs, not all the velocities tested have been displayed, but just one of every three, for sake of clarity. The reference configuration has been selected, but all the other configurations show the same pattern. The wide range of the sub-critical velocities is shown in figure 3.21, where all the velocities show a significant vibration amplitude. At low velocities, pitching has still a vibration amplitude of 0.6 radians, whilst bending has been reduced to 20 mm of amplitude.

Overall, it is clear that no big difference between the different barriers can be found, neither in

Fig. 3.20: Phase plane of the p100 configuration for increasing values of the velocity. Brighter colors mean higher velocity.

Fig. 3.21: Phase plane of the p100 configuration for decreasing values of the velocity. Brighter colors mean higher velocity.

Fig. 3.22: Root mean square of the bending motion for increasing values of velocity for U-shaped sections.

Fig. 3.23: Root mean square of the pitching motion for increasing values of velocity for U-shaped sections.

increasing or decreasing velocities, in any degree of freedom. In addition, as expected, only the sub-critical region makes the difference between the increasing and the decreasing velocities, but no differences have been found in amplitudes of vibration.

The behavior for the U-shaped sections has also been studied, but in this case only configurations Up0 and Up50 have been tested. The aspect ratio remains the same as it was in the H-shaped sections. The results for these configurations are presented in figures 3.22 to 3.25. In those figures, the reference configuration (model without barrier) has also been plotted.

In increasing velocities, the motion starts at values of wind velocity between 12 and 13 m/s for all the configurations. The first configurations to trigger the movement are the ones with the 50% and the 0% porosity barriers (12.2 m/s). The configuration without barriers triggers its motion at

Fig. 3.24: Root mean square of the bending motion for decreasing values of velocity for U-shaped sections.

Fig. 3.25: Root mean square of the pitching motion for decreasing values of velocity for U-shaped sections.

a slightly higher velocity (12.75 m/s). This points out that the shape of the section is important, since in the H-shaped configuration, p0 and p50 tended to higher and lower critical velocities than the reference configuration, respectively, but in the U-shaped configuration both reduced the critical velocity.

After the movement has started, all configurations keep the same increasing slope with the velocity. In pitching, a discrepancy can be found between the configurations: the reference configuration has a larger amplitude than the configurations with barriers.

Decreasing velocities show the expected behavior (figures 3.24 and 3.25). The bending movement keeps vibrating at even half the velocity of the triggering of the movement with increasing velocities: the Up0 configuration reaches values of 6 m/s without stopping. This is not the case for the p100 and

Up50 configurations, which stop their movement at higher velocities (around 7 m/s). In pitching, a discrepancy is seen between configurations (higher amplitude for the reference configuration, p100), even though the trend is similar. In the case of pitching, the Up0 configuration reaches values of one fourth of the maximum amplitude reached. In fact, at low velocities, the trend in pitching for the Up0 configuration is markedly convex, reaching very small amplitudes. At first, it was thought that the movement could stop, but long times between consecutive measurements were left and no indication of the stopping was found.

In the U-shaped section, thus, some difference has been found between the different configurations, having the Up0 configuration the widest sub-critical zone, because of the small velocity at which it still keeps vibrating. Configurations Up50 and Up100 show no significant difference.

Table 7 shows the minimum wind speed at which vibration movement has been recorded and the highest wind speed at which each configuration was stable. Therefore, this can give an idea of the width of the sub-critical range.

Tab. 7: Summary of the results for the determination of the critical velocity.

Configuration	Minimum velocity with vibration [m/s]	Critical velocity [m/s]
Reference, p100	7.5	12.75
H-section p0	6.3	13.9
H-section p25	6.3	12.75
H-section p50	6.3	11.6
U-section p0	6	12.2
U-section p50	7.5	12.2

3.3.2 Initial condition influence

To study the influence of the initial condition, in several points of the sub-critical range, an initial condition has been given to the model. As it is known, for triggering the motion, a suitable small disturbance is needed in the position or speed of the object. At different wind velocities, the required initial amplitude varies [82, 157].

The series of initial conditions set in bending are shown in figures 3.26 to 3.31. In those figures, the same scale has been used, even though, in some cases, it makes it a bit harder to see, but this way, the comparison between the different configurations is easier. H-shaped configurations do not present any unstable initial condition for low porosity barriers. p0 and p25 configurations are stable even for initial conditions of 60 mm, which is 15 times the flat plate height. It can be concluded that

Fig. 3.26: Results of the initial condition tests in bending for the p0 configuration, with H cross-section, in the amplitude-velocity plane. Unfilled marker means that motion triggered after release, filled marker means unsuccessful trigger.

these configurations are stable to bending initial conditions tested. It is specially significant that for the solid barrier, which has been repeatedly tested as very unstable, 29 initial conditions have been tested at 10 different velocities, and none of them has been found unstable. Configuration p25 has been tested with 12 initial conditions at 4 different velocities and no instability has been found in the range of initial conditions tested.

According to figure 3.16, p0 configuration should have started the motion at 11.5 m/s. Nevertheless, the motion did not start imposing bending initial conditions at that velocity or even higher according to 3.26. This fact could be explained by the hypothesis that the model has not been separated enough from the equilibrium position, or that the model is very insensitive to bending initial conditions.

For more porous barriers (figures 3.28 and 3.29), situation changes. Configuration p50 becomes unstable to bending initial conditions at 11.6 m/s. Prior to that, for smaller wind speeds, initial conditions up to 64 mm have been imposed but no instability has been found. At higher velocities, smaller initial conditions (but still large, since they are 10 times bigger than the model height), become unstable. The threshold of the initial condition to trigger the motion is between 36 mm and

Fig. 3.27: Same results as figure 3.26 but for the p25 configuration.

Fig. 3.28: Same results as figure 3.26 but for the p50 configuration.

41 mm.

For the p100 configuration (model without barrier, figure 3.29), the initial condition needed is even smaller (at high velocities). Three unstable initial conditions have been found, at high velocities (higher than 12 m/s), and the threshold for the initial condition is 13 mm of displacement..

U shaped configurations have a similar behavior in bending than the H-shaped sections. The results for the U-shaped sections are displayed in figures 3.30 and 3.31. Indeed, solid barrier, displayed in figure 3.30, is totally stable, as happened to the H-shaped configuration with the solid barrier. Initial conditions of 20 times the deck height have been tested, but none of them have been found unstable (figure 3.30), even though the critical velocity for this configuration is 12.2 m/s according to figure 3.22. However, and as it happened for the H-shaped configurations, 50 % porosity seems to introduce some instability (figure 3.31). In this case, at all tested velocities but one, an unstable configuration has been found and the lowest unstable initial condition is 23.5 mm. It can be concluded that porosity seems to influence not only the response of a section, but also the initial condition needed to trigger the motion. Configurations Up0 (figure 3.30) and Up50 (figure 3.31) have, at the same velocities, different behaviors. For larger initial conditions, Up0 seems stable, whilst Up50 seems unstable.

Fig. 3.29: Same results as figure 3.26 but for the p100 configuration.

Fig. 3.30: Results of the initial condition tests in bending for the Up0 configuration, with U cross-section, in the amplitude-velocity plane. Unfilled marker means that motion triggered after release, filled marker means unsuccessful trigger.

Fig. 3.31: Same results as figure 3.30 but for the Up50 configuration.

Fig. 3.32: Results of the initial condition tests in pitching for the p0 configuration, with H cross-section, in the amplitude-velocity plane. Unfilled marker means that motion triggered after release, filled marker means unsuccessful trigger.

Initial conditions in pitching have also been studied. Once again, the same scale has been used in all the figures to ease the comparison between configurations. The results for those tests are displayed in figures 3.32 to 3.37. In the case of pitching initial conditions, behavior radically changes. All configurations, regardless porosity or shape, show unstable initial conditions. H-shaped solid barriers, whose results are displayed in figure 3.32, can trigger the motion at very low velocities but with very high angle, i.e. around 12° . However, at higher velocities, the threshold between stable and unstable situations is set at 1.5° , more or less. At very large velocities (which are not located in the sub-critical range but in the critical one), as small as possible initial conditions have been tried to set, in order to find a stable situation, but even an initial condition of just 1.1° has started the motion.

Increasing porosity to 25 % (results in figure 3.33) does not seem to alter much the results. Low velocities need again a relatively big initial condition (even though it is not rare value to appear in reality, 5°), but when increasing the velocity, the threshold gradually decreases from around 3.5° at 10.4 m/s to less than 1.5° at 12.7 m/s. Doubling the porosity to 50 % (figure 3.34), does not represent any big difference, as it happened in bending.

Fig. 3.33: Same results as figure 3.32 but for the p25 configuration.

Fig. 3.34: Same results as figure 3.32 but for the p50 configuration.

Fig. 3.35: Same results as figure 3.32 but for the p100 configuration.

For the totally porous barrier (no barrier at all, figure 3.35), situation is also similar, but less conclusions can be extracted, since less tests were performed in this configuration. At low velocities, an initial condition of 21.5° triggered the motion, but this is an unrealistic value to happen in real atmospheric wind. At higher wind speeds, the previously stable situations (around 5°) become unstable, since the motion is triggered for initial conditions higher than 2° approximately.

Changing the shape changes partially the results. Figure 3.36 can be compared to figure 3.32. In this comparison, U shaped section behaves similarly at low speeds (big initial condition needed), but the values at which the motion starts are increased at higher speeds. In the H-shaped barriers, initial conditions of 2° already triggered the motion at 11, 11.5 and 12.7 m/s, but in the U shaped section, this condition leads to stability. In this perspective, it seems safer to chose the second shape, since it can be considered more stable even with bigger initial conditions.

A similar conclusion can be reached when comparing the 50 % porosity barriers (figures 3.34 and 3.37). An initial condition of 2° was unstable for the H-shaped section, but it is unstable for the U-shaped section at the highest velocity tested. Even a stable condition of almost 2.5° has been found at 11 m/s.

Fig. 3.36: Results of the initial condition tests in pitching for the Up0 configuration, with H cross-section, in the amplitude-velocity plane. Unfilled marker means that motion triggered after release, filled marker means unsuccessful trigger.

Fig. 3.37: Same results as figure 3.36 but for the Up50 configuration.

It is reasonable to think that pitching is the critical movement in this case, since very big and even unrealistic initial conditions are needed in bending to trigger the motion but very low values of pitching initial condition can start the motion.

To understand the relevance of pitching initial condition, it is important to remark that usually, when performing wind tunnel tests in bridge sections, angles of attack of up to 10° are studied, which means that such values are expected. In this case, much smaller initial conditions in pitching were needed to start the motion.

3.3.3 Frequency evolution

In the critical condition determination tests, it is also interesting to study the evolution of the vibration frequencies as a function of the wind speed. Since after the start of the motion several values of higher velocities are measured, the signals in those points have been analyzed via a Fast Fourier Transform algorithm (FFT) to obtain the main harmonic components of the signals.

The FFT is useful to study the energy distribution in different frequencies. Applying the FFT to a signal, the main frequencies present in the signal are obtained. The spectrum of the signal at a wind velocity right before the flutter condition, for the Up0 configuration, is presented in figure 3.38. The spectrum is noisy, even though some areas seem to have a bigger spectrum amplitude than the rest. At higher velocities, when the motion has been triggered, the spectrum has a more clear peak, as shown in figure 3.39. Smaller peaks appeared in the rest of the spectrum, which could be related to the harmonics of the vibration frequencies. In that figure, the vibration frequencies in each mode have merged, and both graphs have increased their mean value compared to figure 3.38. Therefore, more energy is involved in both movements.

Fig. 3.39: Fast Fourier Transforms of the bending and pitching motions as a function of the frequency. Up0 configuration, wind velocity immediately over the flutter critical velocity.

Fig. 3.38: Fast Fourier Transforms of the bending and pitching motions as a function of the frequency. Up0 configuration, wind velocity immediately below the flutter critical velocity.

It is important to remark that in both graphs, a logarithmic scale has been used in the y axis. The x axis has been cut in 10 Hz, since the rest of the spectrum does not provide extra information.

The evolution of bending and pitching frequencies with the velocity for the six different configurations tested is presented in figures 3.40 to 3.45. The FFT has been applied to the points of the increasing velocity evolutions. Even though it is not the primary information wanted from these graphs, the

Fig. 3.40: Evolution of bending and pitching frequencies as a function of the wind speed in the critical range for the p0 configuration. H-shaped cross-section.

value of the critical velocity could be extracted from them. This value would be the first one at which the two motions merge their frequencies (flutter condition). However, in these tests, there were configurations at which an initial condition has been imposed, and therefore, the FFT algorithm already detects the natural frequencies, even though the movement has not been triggered. If motion had not been triggered artificially, the critical velocity of each configuration could have been extracted from this analysis.

In those figures, the discontinuous lines represent the bending and pitching natural frequencies of the system without wind, in blue and red, respectively. The continuous lines with markers (circles or stars) present the frequency of the system under wind. Blue circles represent the bending motion and red stars represent pitching motion.

A clear trend can be seen in all the figures. At the critical speed, the merging of frequencies occurs (which is the moment at which the motion can be considered triggered), and then a decrease of the frequency of vibration can be observed, no matter the configuration. The wind speeds lower than the critical velocity have also been displayed but should not be taken into account, since no clear vibration events have happened, as the FFT has been applied over a mainly noisy signal.

The matched frequency happens to be close to the unperturbed pitching frequency. Both degrees of freedom are excited at a frequency slightly higher than 2.05 Hz. Then, it decreases when the velocity increases, and at the largest wind velocities the vibration frequency reaches 2.02 Hz. This situation is similar in all configurations. In fact, configurations that have a smaller critical velocity meet at higher frequencies. Therefore, it can be concluded that the decreasing trend in all the configurations

Fig. 3.41: Evolution of bending and pitching frequencies as a function of the wind speed in the critical range for the p25 configuration. H-shaped cross-section.

Fig. 3.42: Evolution of bending and pitching frequencies as a function of the wind speed in the critical range for the p50 configuration. H-shaped cross-section.

Fig. 3.43: Evolution of bending and pitching frequencies as a function of the wind speed in the critical range for the p100 configuration. H-shaped cross-section.

Fig. 3.44: Evolution of bending and pitching frequencies as a function of the wind speed in the critical range for the Up0 configuration. U-shaped cross-section.

Fig. 3.45: Evolution of bending and pitching frequencies as a function of the wind speed in the critical range for the Up50 configuration. U-shaped cross-section.

is the same.

This behavior of matching frequencies at values between both natural frequencies is already known in the literature. Also the decreasing trend with the increasing velocities has been pointed out by authors like Amandolese or Matsumoto [155, 158, 159]. According to this last reference, the plate is suffering a high-speed torsional flutter.

The analysis of the frequencies has been repeated for decreasing velocities. Figure 3.46 displays the results for both the increasing and the decreasing velocity sweeps for the p100 configuration. The trend is clearly maintained. When the velocity is decreasing, the vibration frequency keeps increasing up to values that reach, in this case, 2.07 Hz at 7.5 m/s. The rest of the configurations have a similar behavior, and have not been displayed for the sake of brevity.

3.4 Analysis

Overall, no big differences have been found between the different barriers or shapes (U or H). The reason for this may be that the height of the barriers is not enough to significantly alter the aerodynamic behavior of the plate. Kubo performed tests comparing rectangular and H-shaped sections, but of smaller aspect ratios [40]. To the author's knowledge, no tests on such elongated H-shaped sections have been performed.

The only difference is that in U-shaped sections, the solid barrier, Up0, has a wider sub-critical range (figure 3.24). This is not unexpected, since in H-shaped sections, barrier is equally divided in the

Fig. 3.46: Frequency evolution in the critical and sub-critical ranges for the p100 configuration (increasing and decreasing velocities).

intrados and the extrados, and in the U-shaped section, barrier is just placed in one of them. This way, the recirculation bubble or wake created behind the barrier is bigger in U-shaped sections, and it may be the reason for the motion to keep even in lower wind velocities.

It also seems clear that even though the motion can be triggered with bending initial conditions, it is much more difficult to do it by bending than pitching. Initial conditions in bending require several times the height of the model, whilst in pitching case, even angles smaller than 1° have triggered the motion. Regarding energy harvesting, this fact should be taken into account in order to trigger the motion at very small velocities.

It is curious that the U-shaped Up0 configuration, which has been said it has the widest sub-critical zone, does not respond to any bending initial condition. However, with small pitching initial conditions, the movement can be started.

Regarding the frequency analysis, the influence of shapes and barrier porosities does not seem to have a significant effect. All configurations present a decreasing value of the frequency after the critical velocity for increasing velocities. There is no difference when the velocity is decreased, excluding the fact that, due to the hysteresis effect, the frequency analysis can be extended to smaller velocities due to the change in critical speed. The behavior is the same: smaller velocities mean higher vibration frequencies.

4 Concluding remarks and future work

From the experience gained while working in this thesis, some conclusions can be drawn. First of all, the importance of wind tunnel tests in bridge design has been underlined. This is not a novel conclusion, but due to the non-negligible changes that appear on the measurements under small changes, wind tunnel tests are a useful tool in the first design stages.

Focusing on the results of the experiments, the following has been concluded:

- **Relevance of the porosity.** Porosity of the barriers is one of the most important aspects to take care about in their design. Overall, a more porous barrier reduces the protection of the users and reduces the static wind loads on the bridge. The influence on the stability is not straightforward. In the case studied, the highest porosity lead to the lowest vibration amplitude. However, this configuration did not have the highest critical velocity, which is one of the most interesting values in bridge design. In this case, longitudinally discontinuous barriers reached higher critical velocities, but higher vibration amplitudes. The width of gaps and covered parts seems to be relevant. In the tests performed, the wider gaps/covered parts lead to higher critical velocities and smaller vibration amplitudes. The increase in the critical velocity can also be obtained by introducing vortex generators in the windward barrier. These generators should introduce small vortexes that interact with the more energetic ones, generating interaction between them that weakens the big vortexes. The last results suggest that not just the value of the porosity is relevant, but also the distribution of that porosity.
- **Relevance of the curvature.** No significant differences have been found between the straight and the curved barriers, whatever the porosity is. The experiments have been performed with a curved barrier of large radius and barrier length values. Thus, more experiments are needed studying a wider range of values of these two parameters. For smaller radius of the curved barriers, it is expected that the flow remains attached to the barrier, reducing considerably the drag coefficient. Vortex-shedding can also be reduced with this approach, mainly in the upper part of the deck (the box-girder part should not perceive any difference in the vortex shedding or in the drag coefficient).
- **Protection of the users.** As expected, least porous barriers protect more the users, reducing the loads on them. The reduction is so remarkable that, regardless the position of the train (either windward rail or leeward rail), the wind loads are very close.
- **Flow condition on the deck.** The flow condition in the key position of the catenaries is highly conditioned by the shear layers shed by both the barrier and the bridge deck. Direct impingement of the shear layer on the catenary should be avoided, since it can create catenary

motion by buffeting. This vibration can create extra loads and fatigue on the catenary and the supporting elements.

- **H-section.** From the literature, experiments show that in H-shaped sections aspect ratio is relevant. In this case, a single aspect ratio have been studied, and the results obtained have proved very small differences to the flat plate, because of the large aspect ratio. Barriers seem to alter slightly the behavior, but this slight variation may mean better performance of certain sections if they are going to be used in energy harvesting purposes. Very small critical velocities can be achieved, if energy harvesting is the objective of the research.
- **U-section.** The U-section has a very similar behavior to the H-section, but some differences can be underlined. The critical velocity slightly varies compared to the H-section. In pitching, the amplitude is slightly smaller. All these effects can be due to the fact that the bigger barrier (in this case, 6 mm of the barrier are exposed to the wind, instead of the 4 mm in the H-section) creates bigger vortexes, which can trigger the movement earlier. Configuration Up0 has reached the smallest sub-critical range, having started motion at only 6 m/s. This result is of interest for energy harvesting applications.
- **Initial conditions.** In the amplitude range tested, bending initial conditions very rarely lead to triggered motion. Relatively large initial conditions are needed in bending to trigger the motion, up to 5 times the deck depth. In some cases, initial conditions of 20 times the deck depth did not trigger the motion. Pitching initial conditions, however, can very easily trigger the motion. Even small initial conditions started the motion. The motion was triggered for all the configurations with angles smaller than 5° , some even at angles smaller than 1° . With this, it is clear that the pitching motion is driving the movement. Therefore, when an initial condition in bending is imposed, the effective angle of attack reached is small, and the motion does not start. This situation could be changed by eliminating the coincidence of the elastic center and the center of mass, by placing some extra mass out of the elastic center.

4.1 Future works

To fully understand the bridge aeroelasticity, a significant amount of work is still ahead. Many parameters take part into the design of a bridge. The following are some proposals to keep the path in order to obtain better designs, less sensitive to wind effects.

- **Influence of porosity.** Deeper and more systematic studies are needed regarding the influence of barrier porosity. Design of barriers is influenced by many parameters, and just a couple of them has been studied in this thesis. Some of the parameters that can be studied are size of

the barriers, inclination, presence of eaves, etc.

- **Influence of other parameters.** Further tests should comprise not only porosity (main parameter studied here) but also shape and section of the bridge. Dynamic parameters should also be studied, due to the importance of the frequency ratio or the mass in the aeroelastic behavior of the bridge.
- **Influence of the initial conditions.** In the tests for studying the influence of the initial conditions, they have been introduced manually. This is the reason for the big dispersion in the initial conditions. A system to introduce a controlled initial conditions could help to systematize initial conditions and to ensure the repeatability of the tests. This system should be able to impose the initial conditions in all the degrees of freedom. It should also be capable of introducing initial conditions in combined degrees of freedom, i.e., in bending and pitching at the same time. This combined initial conditions could lead to trigger the motion at smaller wind speeds.
- **Standardization of barrier design.** Standardization and normalization of barrier design from the aerodynamic point of view should be one of the long-term goals. It is a challenging task, because of the many parameters involved, but a database with the performance of standardized barriers would simplify the design stages of bridges. This process could start with the standardization of barriers installed in standard bridge sections, such as the ones studied here.
- **Flow behavior during vibration: Velocity field.** Some techniques have been lately developed and could help to improve the knowledge on the fluid-structure interaction. Particle image velocimetry (PIV) shows information about the characteristics of the flow. The comprehension of the flow condition over bridge decks could be improved applying this technique. Moreover, time-resolved PIV can be even applied to understand the dynamic behavior of the bridge deck, by synchronizing the acquisition of the displacement measurements and the PIV measurements. This way, the influence of the vortexes and other flow structures in the aeroelastic phenomena can be understood. However, common PIV systems have a limited sampling frequency, and a specific and expensive system is needed for performing time-resolved tests.
- **Flow behavior during vibration: pressure field.** Pressure measuring systems could also give some more quantitative information on the behavior of the flow. Issues arise when dealing with the location of the measuring device. Because of the vibrations of the section, it should not be placed in the own section, but outside, in a fixed position. Unfortunately, in some cases, this could lead to excessively long tubes, which are known to filter the pressure signal and to damp the oscillation motion. The solution is to perform tests in a two-dimensional wind tunnel, but this way longitudinally discontinuous barriers can not be studied.

A ACLA 16 wind tunnel

ACLA wind tunnel is sited in the Centro de Investigación y Desarrollo Aeroespacial of the Escuela Técnica Superior de Ingeniería Aeronáutica y del Espacio, located at Parque Tecnológico de la UPM (Montegancedo campus). The space occupied by the wind tunnel is a parallelepiped of 40.1 m length, 7.7 m width and 4.3 m high. In the figure A.1 a scheme of the wind tunnel can be seen.

The test chamber of the ACLA-16 wind tunnel is closed. The section is square, being each side 2.2 m long. The total length of the test chamber is 20 m. It is divided in two parts. In the first part, turbulence generators are usually placed, together with the superficial roughness to create the scaled atmospheric boundary layer if necessary. With these elements, the profile of velocities and the profile of turbulence can be correctly reproduced. Side walls are made of glass, as can be seen in figures A.1 and A.2. The test chamber is located after the preconditioning chamber. For civil structures, this is the most adequate section to perform the test. Depending on the type of test, some other sections through the chamber can be used. In the middle of the measurement section, velocities up to 32 m/s can be reached, being the turbulence intensity of 3%. This condition is reached without any turbulence generator or roughness.

Fig. A.1: Scheme of the ACLA16 wind tunnel. The upper image presents an upper view and the lower presents a side view.

Upstream the test chamber, the contraction is located. Overall, it has 6 m from the beginning of the inner conduct. Because of space limits, contraction is bi-dimensional, i.e. the flow is only contracted vertically. Floor and ceiling converge from the intake to their relative position in the test chamber, whilst vertical walls remain parallel in this part of the duct.

The diffuser starts right after the test chamber. In the first part, horizontal and lateral surfaces are scarcely divergent. This first part is 9 m long, and finishes in a bifurcation that redirects the flow to

the lateral diffusers. In the bifurcation, ceiling and floor are parallel to each other. Lateral walls are circular shaped. In each of the branches of the bifurcated duct, a deflector vane avoids the boundary layer to detach. After the bifurcation, the resulting ducts are again divergent until the section of where the fans are located.

(a)

(b)

Fig. A.2: Pictures of the ACLA16 wind tunnel: (a) presents the return chamber and (b) the test chamber.

The wind tunnel is equipped with 16 fans, distributed in 2 groups of 8 fans each. Each group forms a matrix of 2 columns of 4 fans. Each fan has 6 polyamide blades, and is powered by a three-phase electric engine of 7.5 kW. The overall power of the wind tunnel is then 120 kW. The rotating speed of the fans is controlled by means of two variable frequency drives. Therefore, it is possible to modify the velocity in the test chamber in a continuous way. Fans discharge to the room, which works as a return chamber. Through this chamber, the air goes back to the entrance section.

B A 9 wind tunnel

A9 wind tunnel is an Eiffel type wind tunnel placed in the IDR/UPM facilities of Escuela Técnica Superior de Ingenieros Aeronáuticos, in Madrid.

The test chamber of the A-9 wind tunnel is closed. the section is rectangular, being 1.5 m wide and 1.8 m high. Upstream the test chamber, turbulence generators can be installed. In the midpoint, the maximum velocity that can be reached is around 30 m/s, being the turbulence intensity of 2%. This condition is reached without any turbulence generator.

The contraction is located before the test chamber. Because of space limits, the contraction is two-dimensional, i.e. the flow is only contracted horizontally. Side walls converge from the intake to their relative position in the test chamber, whilst the floor and the ceiling remain parallel in this part of the duct. A sketch of the wind tunnel can be seen in figure B.1.

Fig. B.1: Top and side view of the A9 wind tunnel. TC is the test section, D the diffuser, F the fans, R is the return duct (the whole room works as duct) and C is the bi-dimensional contraction.

The diffuser starts right after the test chamber. Horizontal and lateral surfaces are divergent, and finish in the position the fans, 9 overall, that are organized in a matrix of 3x3. From there, the air is discharged to the room, that serves as the return chamber. The fans providing the power for the wind tunnel are made of 6 polyamide blades, and the engine moving each fan is a three-phase electric engine of 7.5 kW. The overall power of the wind tunnel is then 67.5 kW. the rotating speed of the fans is controlled by means of a variable frequency drive. Therefore, it is possible to modify the velocity in the test chamber in a continuous way.

C Set of tests performed to each configuration

This annex presents all the configurations tested throughout the thesis, and the different tests performed to each.

Tab. 8: Set of experiments performed in the thesis.

Section	Type of barrier	Static tests	Dynamic tests	Catenary tests
Bridge section	SP	x	x	x
	SP-BR	x	x	x
	SP-BC	x	x	x
	SP-BRP	x	x	x
	SP-BCP	x	x	x
	SP-TB	x		
	SP-BR-TB	x		
	SP-BC-TB	x		
	SP-BRP-TB	x		
	SP-BCP-TB	x		
	SP-TS	x		
	SP-BR-TS	x		
	SP-BC-TS	x		
	SP-BRP-TS	x		
	SP-BCP-TS	x		
	B-1		x	
	B-1-2		x	
	B-1-2-3		x	
	B-1-2-3-4		x	
	B-1-3		x	
	B-2-4		x	
	T-2		x	
	T-3		x	
Flat plate H-section	p0		x	
	p25		x	
	p50		x	
	p100		x	
Flat plate	Up0		x	
U-section	Up50		x	

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