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Aerodynamic Design of Very Long-Span Suspension Bridges

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Summary

New deck design and new kinds of structures have been developed in order to increase the stability and the performances of super long-span bridges allowing very long spans which only some years ago were thought not to be reached. On the other hand, to reach these results, numerical and experimental methods must be available in order to assess the good performance of the bridge subjected to turbulent wind. The paper outlines the different solutions for controlling bridge stability and describes the different methods available for evaluating the response of the structure to turbulent wind.

1. Introduction

In the last decade several long-span suspension bridges have been designed: some of them have been already built, such as Great Belt and Akashi, reaching a maximum span length of 2000m. Some other projects are related to bridges with a span length greater than 3000m (Gibraltar 3500m, Messina and Sunda straight crossings, both 3300m): from this point of view the 21th century can be considered the age of the "over 3000m" span bridges. The increase of span length from 2000m to 3000m is not feasible with a simple extrapolation of the existing bridges, since the structural typology of the bridge must undergo important modifications. The fundamental problem that must be solved in the design of a super long suspension bridges is the aeroelastic one, that is the behaviour of the structure in turbulent wind conditions. The aeroelastic stability of the bridge must be ensured above the design wind speed, ranging from 60 to 80 m/s. As an example, for the Great Belt, with a span length of 1680m, the selected design wind speed was 60 m/s, while for the Akashi Bridge, a value of 78 m/s was assumed. As will be discussed in the following, in order to increase the span length over 3000m, critical speed requirements cannot be satisfied using nor the Great Belt's typology (traditionally suspended single box girder) neither the Akashi's one (traditionally suspended truss girder). Several ways can be followed in order to increase the span length without compromising the aeroelastic stability of the structure, anyone of those implying a reconsideration of the three parameters affecting the aeroelastic stability:

i) Torsional vs. flexural frequency ratio r_f.

ii) Ratio r_t between structural stiffness and equivalent stiffness due to aeroelastic effects.

iii) Total amount of damping, including structural and aeroelastic contributions.

All these parameters must be as large as possible in order to increase the bridge stability. An increase of the r_f ratio implies a modification in the design of the structure; in order to increase the r_t ratio, besides structural modifications affecting the structural stiffness, the shape of the deck must be modified, in order to reduce the aerodynamic forces. The total damping of the structure could be increased in several ways: by means of dynamic absorbers, passive aerodynamic damping devices or even active control. In conclusion, in order to assure the feasibility of extra long suspension bridges the following topics must be carefully analysed:

a) Bridge structural design.

b) Aerodynamic deck design.

c) Adoption of active and/or passive control.

All these ways have been investigated by designers and researchers, whose efforts are aimed to demonstrate the feasibility of bridges with span length greater than 3000m. On the other hand, in order to compare different bridge solutions in the design stage, a tool able to evaluate with accuracy the response of the bridge to turbulent wind and to check its aerodynamic stability is necessary. In this work the approaches usually adopted for the three a), b), c) above mentioned tasks are described, with reference to both existing projects and preliminary studies for future realisations. Moreover, the available methods for the analysis of the response of the bridge to turbulent wind will be described, and their advantages and limits will outlined. The paper is organised as follows:

i) Description and critical analysis of the different strategies for increasing the stability and/or the span length of suspended bridges.

ii) Discussion of the available methods for simulating the structure response to turbulent wind (r.t.w.).

iii) Concluding remarks.

2. Structural-Aerodynamic Solutions

As already mentioned in the previous paragraph, one of the most challenging problem to solve in super long-span bridge design is the aeroelastic stability at design wind conditions. The experience gained on the recently built record span bridges Great Belt East Bridge and Akashi Kaikyo Bridge, together with the parametric analysis considered in the preliminary Gibraltar project (Astiz 1993), showed that the static system of the classical suspension bridges and the modern streamlined single deck box girder solution (Humber, Bosporus, Great Belt) reach an intrinsic limit for spans approaching 2000m (Astiz 1996, Miyata 1993). The fundamental reason is the higher trend of reduction of the first torsional frequency ω_t compared to the flexural one ω_f versus span length, as shown in Figure 1, where the frequency ratio $r_f=\omega_t/\omega_f$ changes from $r_f=3.5$ for main span length $L_s=1000m$ to $r_f=2.8$ and $r_f=1.3$ for $L_s=2000m$ and $L_s=3000m$. Being in fact the aerodynamic forces quite significant, due to the lifting characteristics of a shallow streamlined box girder, the structural torsional stiffness becomes too low compared to the equivalent aerodynamic one, lowering in this way the critical wind speed. A first attempt is then to increase the flutter speed by various minor structural modifications of the basic solution, having the common purpose of rising the structure stiffness. An interesting sensitivity analysis (Astiz 1993) shows the effects achievable by this approach (see Figure 2). If moderate stability performances are required, as in the Great Belt case (1624 m span and 60 m/s design flutter limit), an optimum compromise of low drag, low vortex shedding excitation and sufficiently high girder torsional stiffness was obtained using a standard design box girder with minor aerodynamic refinements, and structure efficiency improvements. On the other hand the Akashi project showed that the

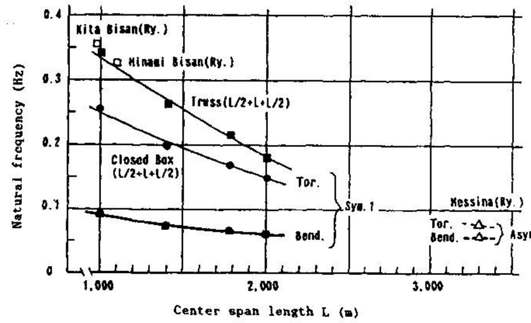

Figure 1 Trend torsional vertical freq. ω_t, ω_f versus span length (Miyata 1993)

Design change with respect to basic solution	Increase in critical velocity (%)
Girder weight (+25%)	+1.1
Distance between cables (+17%)	-8.8
Cable sag (from 1/9 to 1/8)	+2.5
Girder torsional stiffness (+50%)	+11.1
Additional stay cables	+9.1

Figure 2 Effects of structural changes on the critical wind speed (Astiz 1993)

higher flutter limits (78 m/s design critical wind speed) and the 2000m record span length required substantial modifications. The solution was again the attempt to face flutter, maintaining the traditional cable static scheme, through a very rigid and massive truss-box girder, with the final well-known choice of the truss solution (Miyata 1993). On the other hand the lesson learned with the thorough analysis of a large number of different deck solutions in the Akashi design is that it's not possible to rely only on the girder stiffness and mass increase when the span length reaches or exceeds 2000m, and more substantial geometrical-structural changes are needed, or new ideas have to be developed in the aerodynamic design. Incidentally, it helps to mention that, whatever is the effort, the girder contribution to the bridge torsional stiffness becomes negligible when the span is beyond 2000m, due to the prevailing main cable effects. It needs also to be highlighted that a substantial contribution to the aeroelastic stability of Akashi came also from aerodynamic countermeasures like the adoption of a vertical plate-like stabiliser, the optimal position of the inspection ways, and the longitudinal gap in the deck between the two roadway flat plates. If the only structural way is followed, super long suspension bridges have to abandon the traditional static cable system. Crossed hanger system, or a combination of cross stay (vertical) and horizontal stay (Miyazaki 1997), are a first minor modification aimed at rising the bridge torsional stiffness, allowing a not negligible improvement of the critical wind speed (Astiz 1993, 1996). The greatest improvement in torsional stiffness is achieved through the one cable system proposed in (Leonhardt 1968) (Figure 3) and its variants like the three cable system (Astiz 1996), having on the other hand several drawbacks as great load oscillations in the hangers and pronounced pendulum behaviour. Another interesting scheme is the four-cable system requiring on the other hand a quite complex vertical crossed hanger net to realise an effective improvement of performances in terms of torsional stiffness and critical wind speed (Astiz 1996).

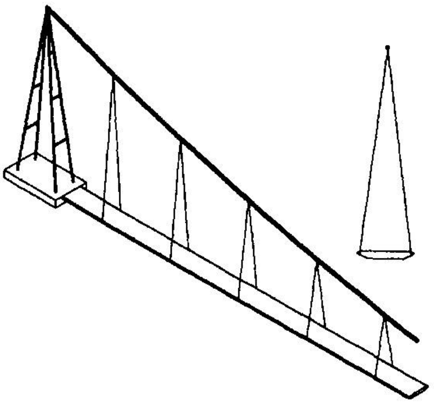

Figure 3 Mono cable system (Astiz 1996)

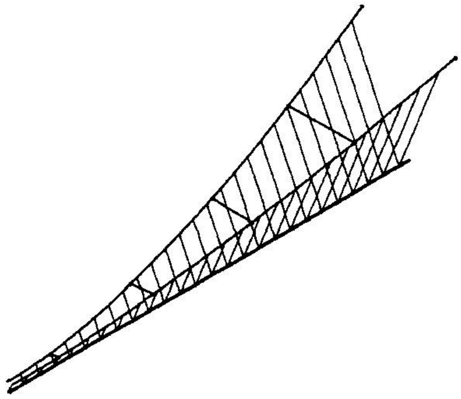

Figure 4 Spatial system with high lateral and torsional stiffness (Gimsing 1997)

The idea of the four cable system was suggested also for the Messina project in (Musmeci 1971) first and in (Borri 1992) later, combined with an hybrid cable-stayed/suspension structure aimed substantially at reducing the total span of the suspension bridge. It has to be noticed however that the lower cables system, adding significant weight to be supported by the overhead main cables, penalises this solution, considering that one of the main advantage of spatial cable systems should be the increase in torsional stiffness avoiding heavy and massive structures. Several authors finally proposed various solutions of spatial cable systems, and the state of the art of this topic is well referenced in (Gimsing 1997). A final spatial solution proposed by Gimsing is shown in Figure 4, combining the advantages of rising both lateral and torsional stiffness of the structure, but requiring very complex tower arrangement and not easy construction procedure. Several authors made parametric comparison of those different solutions, comparing their flutter limits for equal aerodynamic and structural deck characteristics, selecting usually as a reference a standard streamlined box girder and showing for all of them a substantial advantage, with respect to the traditional static scheme (Ito 1996), (Gimsing 1994), (Walther 1994), (Nomura 1994). As previously mentioned, although the central idea to increase the torsional stiffness through a new spatial cable system is conceptually very effective, nevertheless several drawbacks are involved, especially the complexity of the construction stage and the need of developing new not experimented construction technologies, never experienced up to now. It has to be stressed finally that the geometrical complexity of those solutions and the stiffening effects of differently arranged crossed hangers results generally in a quite complex structure dynamic behaviour with structural coupling of vertical and torsional mode shapes. Follows the impossibility to use a simple 2 D.o.F. modal flutter analysis, as in the well experienced standard suspension bridge scheme, making mandatory at least a multimodal flutter analysis to assess the structure stability limits, as well as a careful check of static divergence. A completely different solution for the aeroelastic stability of very long-span suspension bridges could be that of tuning the two frequencies ω_t and ω_f to the same value. In fact, assuming a modal 2 D.o.F. scheme representative of the aeroelastic bridge behaviour, it is easy to show that making the torsional frequency deliberately equal to the vertical one (and controlling this ratio for all the possible coupling modes), the reduced critical velocity goes to infinity, inverting the trend where flutter speed is usually decreasing with r_f (Dyrbye 1997). In such a case the bridge stability should be generally discussed just in terms of static divergence. On the other hand, similarly to the over mentioned non standard-geometry solutions, no example exists of significant structure designed and built following this idea, not only due to the conservative approach still now followed in the final design choices, but also due to the very dangerous drawbacks of possible static divergence accompanying the low torsional stiffness solutions. The feasibility of some proposed solutions of super long suspension bridges is in fact clearly arguable in terms of torsional static divergence or too large lateral-torsional displacements as in the case of the 2-box and 1-box combined girder recently proposed in (Ogawa 1997) showing torsional rotation in the order of 8 Deg at the wind flutter limit of 70 m/s. Adopting a purely structural approach, a final solution to the aeroelastic stability of very long suspension bridges could rely on composite very light materials. As mentioned in (Ostenfeld 1992) and (Meier 1991), the availability of Carbon Fibre Reinforced Polyester (CFRP) with tensile strength of 3300 MPa and density of 1.56 kg/dm^3 (compared with 1700 MPa and 7.8 kg/dm^3 for steel) could be reasonably affordable in the next future at a competitive price for very long main span length, allowing to build very light structures, having a considerably higher ratio payload/unit mass of cable. Although the E modulus of CFRP is slightly lower compared to steel (165000 MPa compared to 205000 MPa), the use of CFRP supposed limited to the main cable, can reduce considerably the total dead load of the structure and rise the equivalent vertical and torsional structure stiffness and frequencies. On the other hand the main cable-mass penalising effects (in terms of low torsional/vertical frequency ratio), due to the high equivalent torsional inertia, can be reduced, and the relative contribution of high stiffness girder can be more significant. Figure 5 shows the parametric dependence of the critical flutter speed on the structure specific weight, assuming unchanged all the other parameters and in particular the tension in the main cables, the structure equivalent stiffness and the aerodynamic derivatives. The increase of the flutter velocity is due in this case to the higher aerodynamic damping associated with the increase of the natural frequencies caused by the lower structure inertia. All the previous analysis were made considering unchanged the deck

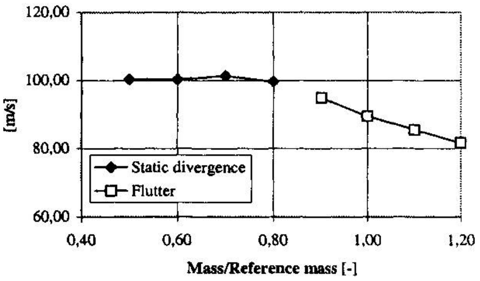

Figure5 Flutter limit as a function of relative modal mass; (Messina bridge)

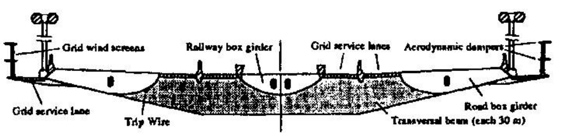

Figure 6 Messina multi-box deck section

aerodynamic characteristics, and examining the only structural solutions to the problem of controlling the aeroelastic instability of very long suspension bridges. On the other hand this is a purely academic exercise, as the aerodynamic improvement of the section is always a tool available for the designer. A complementary approach to the problem is in fact the attempt to reduce the ratio Aerodynamic/Structural forces working on the refinement or on innovative aerodynamic design of the structure. The history of the design experience of the two significant projects, the first already built (Akashi), the second developed up to a definitive stage (Messina), shows in fact that for span length over 2000m, the reduction of the aerodynamic deck lifting characteristics and the use of some aerodynamic damping became mandatory. Having as a target the optimum deck aerodynamic design, four fundamental objectives should be pursued. (a) Low lifting characteristics (low lift and moment derivatives). (b) Low drag. (c) No significant vortex shedding excitation. (d) High aerodynamic damping. In the opinion of the authors, having as a target main spans over 2000m a successful design must search a compromise solution between the four over mentioned objectives. On the other hand, as shown by both Akashi and Messina projects, the adoption of a "slotted" deck solution seems to be unavoidable if low lifting characteristics are searched. While in the Akashi case the solution was a traditional flat plate, supported by a single truss girder and interrupted by a large gap at mid-chord, the Messina project adopted the new idea of multi-box girder, with three highly streamlined longitudinal box girders connected by box-shaped cross girders at 30m distance (Figure 6). Open grids in the areas between the longitudinal and transverse box girders allow pressure equalisation between upper and lower side of the deck, reducing the lifting characteristics of the section. Great advantage of the Messina solution is the possibility of maintaining an overall section streamlined profile with extremely low drag (if compared to Akashi as an example) together with very low vortex shedding excitation. The Messina deck section is on the other hand an example of optimum compromise between the four over mentioned aerodynamic design objectives: as an example, a careful experimental investigation allowed to select the optimum wind barriers porosity giving low drag, effective traffic sheltering, unchanged aerodynamic characteristics between no traffic-full traffic conditions, as well as effective performances of wing shaped aerodynamic dampers integrated in the wind barriers (Diana 1993). The Messina experience showed as a conclusion that, for very long-spans, the multi-box deck solution is the most promising in terms not only of optimum aerodynamic design, but also of light weight of the structure, easy maintenance and modularity construction technology, requiring on the other hand a very careful aerodynamic refinement, in order to control flow separation and allow uniform performances in a wide range of wind angle of incidence (Brown 1996).

Following the design strategy of taking advantage of high aerodynamic deck performances, it becomes of crucial importance the assessment of the right aerodynamic characteristics of the section and the availability of very reliable mathematical models simulating the aerodynamic stationary and non stationary wind effects. As far as concern the first issue, wind tunnel techniques are still an unavoidable step in the design stage: the most critical aspects of such experimentation is the consistency of scale model data with the real full scale behaviour of the structure, as well as the development of numerical-experimental techniques allowing to simulate correctly the turbulence effects on the bridge stability and the non-linear effects associated with high angle of incidence, as will be better explained in the next paragraph.

The possibility to approach the aerodynamic forces not only as a non conservative destabilising field, but also as a possible source of damping for the structure, is on the other hand the very last issue in very long-span bridges aerodynamic design, already experienced in terms of passive solutions, but investigated and considered as applicable also in terms of active control. As far as concern passive aerodynamic damping, several solutions were proposed (Cobo del Arco 1997), (Zasso et al. 1993b) most of them consisting in wing profiles fixed at the section leading or trailing edge zone, adding torsional and vertical direct damping, but increasing also the crossed terms. It's clear that the solutions applicable in a true project must take into account feasibility problems, being again a compromise between the requirements of maximum effectiveness (free stream) and the practical realistic proposals on how to integrate the aerodynamic damper in the overall deck section.

The final tool never applied in real structures, but already considered by several authors in a feasibility stage is the active control (Miyata 1996), (Achkire 1997), (Cobo del Arco 1997). High performance wings in the undisturbed leading and trailing edge are the most effective tools, and a very reliable numerical model simulating the overall deck section aerodynamics together with the global bridge dynamics is then mandatory in order to run the active control in an effective way. In the opinion of the authors, even though very attractive results are devised, this solution is still a research topic, due to the reliability requirements of structures like the civil-transport ones. For safety reason, is difficult to believe that a structure collapsing at a wind speed lower than the design one, if abandoned to its passive resources, could be approved in the next future. In other words the structure must be intrinsically stable and the active control should be used only for increasing the overall performances as an example in terms of comfort, rail and road runnability, fatigue life-expectance of the structure.

3. Models for the Simulation of Bridge Response to Turbulent Wind

In order to define in quantitative terms the effectiveness of different design solutions, a tool allowing the evaluation of all the different aspects of the aeroelastic problem (i.e. bridge buffeting and aeroelastic stability) is needed. At the different stages of the design process, depending on the level of accuracy required, the aeroelastic analysis of a long-span bridge can be performed using methodologies of different complexity. As an example, the tools adopted in the initial stage of selection and optimisation of the structural typology, where repeated calculations are needed, are different from those that must be used for the final check of the optimised solution, where high accuracy is needed. Figure 7 shows the aeroelastic analysis procedure usually adopted: in the left section the typology optimisation process is described, while in the right section the final check procedure is outlined. The paper will not describe in detail the mathematical models used in the optimisation stage, where consolidated linear approaches are adopted, using section model experimentally measured flutter derivatives and modal reduction of the structure degrees of freedom (Scanlan & Tomko 1971, Scanlan 1992). These approaches are affected by the approximations implied by the linearity assumption, where in fact the system shows a nonlinear behaviour, especially under the action of turbulent wind, where high variations in the angle of attack occur (Miyata et al. 1995, Bocciolone et al. 1990).

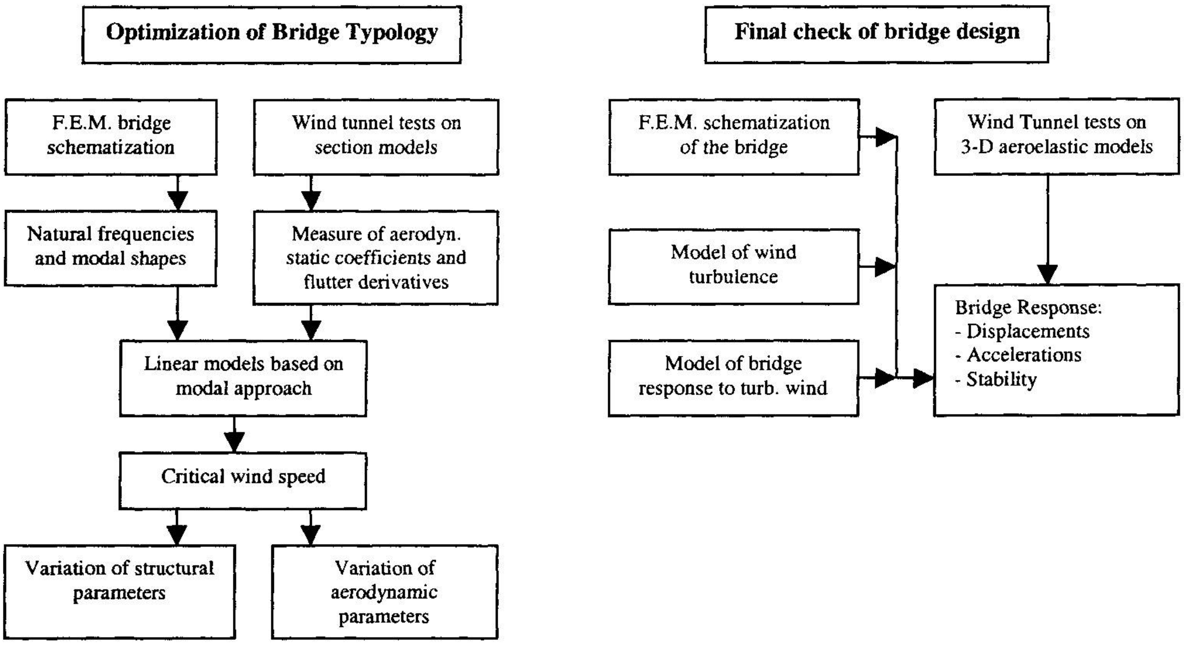

Figure 7 Procedures for the aeroelastic analysis

Nevertheless, the critical wind speed values estimated by linearised methods are generally reliable, as far as traditional typologies of single span bridges are concerned. As already mentioned, when dealing with non traditional typologies like those using crossed hangers (Astiz 1996) or horizontal stay ropes (Gimsing 1997) or bridges with very close main cables (Ogawa et al. 1997), modal approaches based on coupling of a single flexural mode and a single torsional mode should be carefully considered. For these structures multi-modal approaches are in fact recommended as the flexural modes shape is usually different from the torsional one and sometimes it is not possible to make a clear distinction between flexural and torsional modes, showing each mode both components of motion. In the same way, the procedure of superimposing the effects of the mean wind speed and the effects of wind velocity fluctuations, as done in (Ogawa 1997) should be regarded with particular care. In Figure 7 right section, the two main approaches available for predicting the real bridge behaviour, the numerical one and the experimental one, are reported. The numerical approach consists in the artificial generation of a space-time distribution of wind speed and subsequently in the simulation of the bridge r.t.w. using a finite element model of the structure in conjunction with an appropriate non-linear model of the aeroelastic forces. On the other hand the experimental approach consists in wind tunnel tests on complete tridimensional aeroelastic bridge models (Reinhold et al. 1993). Both the analytical and the experimental method, reproducing the real bridge behaviour, allow to estimate the bridge dynamic response, and in particular the mean values (static response), the r.m.s. values and the spectra of vertical and torsional deck motion excited by the turbulence (buffeting response). From these data it is then possible to evaluate the wind induced modifications of the damping ratio and of the natural frequency of each mode of the structure, as a function of the wind speed (Diana et al. 1995, Sumiyoshi et al. 1993). From the trend of flexural and torsional frequencies (Figure 8), and damping ratios (Figure 9), the instability threshold can be obtained. From this figure it can also be defined the stability index of the bridge: in this case, at 62 m/s design wind speed, the stability index is 4%. The static stability and buffeting problem are then considered simultaneously. The two approaches are not in contrast each other, and in fact they can be considered as complementary and sometimes have been used in conjunction.

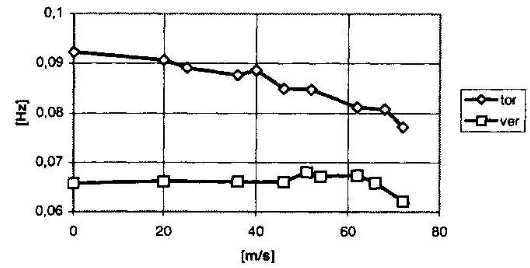

Figure 8 Effect of wind speed on the first flexural and torsional frequencies for 1:250 model of Messina Bridge

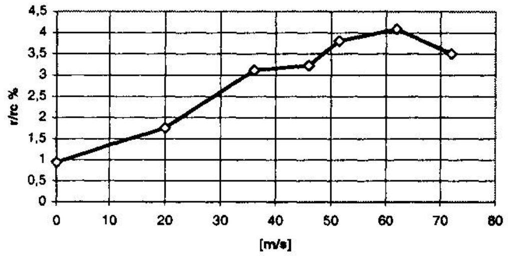

Figure 9 Effect of wind speed on the damping ratio of the first torsional mode, 1:250 model of Messina Bridge

3.1 Wind Tunnel Tests on Full Aeroelastic Models

Being the techniques adopted in these tests well established, the discussion will be limited to the description of the advantages and disadvantages of this method. In particular among the main advantages the following ones can be mentioned:

- It is possible to reproduce the features of the problem in its entirety, including turbulence effects, tridimensional and non-linear effects.
- The aeroelastic model measured response can be used to verify and tune a numerical model of bridge r.t.w., being in this case easy to characterised thoroughly the process input and output i.e. the wind speed field, and the model motion.

The weak point of this experimental method can be summarised as follows:

- It is not easy to reproduce in the wind tunnel the real wind turbulence distribution, and in particular the correct ratio between the integral length of turbulence and the characteristic dimension of the model (e.g. the chord length).
- Scale effects, related to the difference between full scale and test conditions Reynolds numbers. The scales usually adopted for aeroelastic models range from 1:300 to 1:100 (or less in few cases), depending on the available wind tunnel facilities. This effect can be monitored measuring the flutter derivatives of different scale section models (Zasso 1993a).
- It is not always simple to accurately reproduce on the scale model the natural frequencies, modal shapes and damping ratios of the real structure.

Anyway this method has to be considered an indispensable tool for verifying the aeroelastic behaviour of a long-span bridge, although being very expensive and time consuming, and therefore it must be considered the conclusive step of the design process.

3.2 Methods for the numerical simulation of bridge buffeting

There are two main approaches to the problem of simulating the bridge response to turbulent wind, the first usually named "frequency domain approach". In this approach the input is represented directly by the statistical properties of the wind (Power Spectral Densities of the wind speed components and coherence functions along the bridge) and the output is represented by the

statistical quantities (e.g. P.S.D.) describing the motion of the structure. This approach is inherently linear: in its more sophisticated formulation the "self excited" component of the aerodynamic forces (that is the component due to the motion of the structure) is represented through the flutter derivatives evaluated in correspondence of a reference static condition assumed "a priori". For this reason this approach cannot be considered adequately accurate for the final check of the aeroelastic behaviour of a long-span bridge. The method analyses separately the static response of the bridge, the stability and the buffeting response.
The second approach, which will be called in the following "time domain approach", consists into two main steps: first a space-time wind distribution is artificially generated from the knowledge of the wind basic statistical properties; as a second step, the numerical integration of the structure motion equations is performed in the time domain, obtaining the bridge motion as output. The different techniques adopted for space-time wind distribution generation will not be discussed here, it is only mentioned that, as an example an ARMA model, or a wave superposition method can be adopted (Bocciolone et al. 1990, Shinozuka 1972). The dependence on the gust size, with respect to the deck size, is taken into account by multiplying the basic wind spectrum for the aerodynamic admittance function, obtaining the corrected target spectrum, used for wind generation. A question that should be investigated is whether the turbulence coherence is also representative of the coherence of the aerodynamic forces along the bridge: some authors found that the transversal coherence of the aerodynamic forces at different locations is greater than the one calculated on the basis of the wind coherence function (Larose 1997). Before analysing the possible time domain simulation techniques, it is useful to introduce the problem of modelling the aerodynamic forces, representing the crucial point for the bridge r.t.w. simulation.

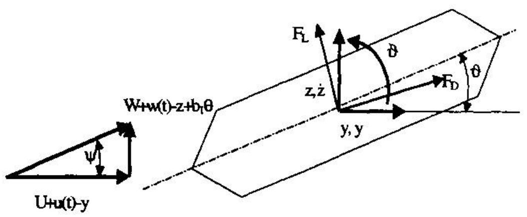

Figure 10 Variables defining wind forces on the deck

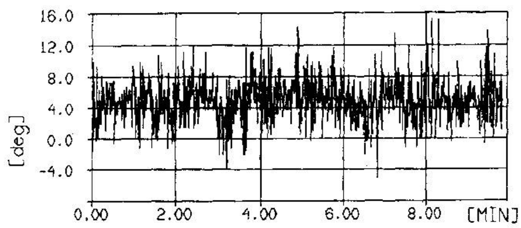

Figure 11 Incidence angle of wind speed measured on Humber bridge, from (Bocciolone et al.)

To this end we will concentrate on a section of the bridge, whose position along the deck is introduced through the spatial co-ordinate ξ: the problem in its essence consists in accurately reproducing the aerodynamic actions on the structure, due to the combined effect of incident flow turbulence and bridge motion, as shown in Figure 10, where U, u(ξ,t) represent respectively the mean value and the fluctuating component of horizontal wind speed, and W, w(ξ,t) are the corresponding vertical components. Moreover y, $\dot{y}$, z, $\dot{z}$, ϑ, $\dot{\vartheta}$ are the parameters describing the deck motion and F_y, F_z and M_θ are the components of the aerodynamic forces. These forces can be considered as the output of a model reproducing the aeroelastic behaviour of the deck, and are non-linear functions of the input quantities U, u(ξ,t), W, w(ξ,t), y, $\dot{y}$, z, $\dot{z}$, ϑ, $\dot{\vartheta}$. In order to experimentally characterise these forces, suitable wind tunnel tests should be carried out on section models, moving the model according to a pre-defined law of motion and introducing controlled components of turbulence in the air flow. Through this kind of tests a model of the wind actions on the deck could be implemented as a "black box" input-output non-linear relationship, for instance by means of NARMAX techniques or neural networks. To have an idea of the importance of non linear effects in those relationships it should be recalled that the values of the angle of incidence ψ in Figure 10, due to the fluctuating components of wind velocity can vary between ±10, as shown in Figure 11, (full scale measurements on the Humber bridge, Bocciolone et al. 1990). Being well known that the flutter derivatives are highly sensitive to the average angle of attack (Zasso 1993a), as a consequence, because of those non linearities, the aerodynamic forces due to turbulence and to deck motion can not be calculated separately and then superposed. Nevertheless, several methods are still based on the separation of the aerodynamic forces due to turbulence from those due to bridge motion, as in (Bucher 1992a,b). Bucher's method adopts the quasi-static corrected theory for modelling the aerodynamic forces related to turbulence. The forces depending on the deck's motion are calculated by means of the convolution integral, with deck's displacements and rotation (and their derivatives) as inputs. To this end, the wind forces response to an impulsive variation of each single input (defined as a superposition of exponential functions), is obtained through an identification procedure performed on the results of wind tunnel measurements similar to those adopted for the extraction of the flutter derivatives. As far as the contribution due to deck's motion is concerned, and assuming the problem as linear, this solution of the problem is rigorous, reproducing the unsteady aerodynamic forces. This method could be further improved, as proposed in (Li 1997) introducing indicial functions also for that portion of forces due to wind turbulence. In fact, the use of indicial functions in time domain corresponds to the use of admittance functions in the frequency domain, but an additional advantage is represented by the possibility of suitably taking into account the effects of bridge motion transients and time variation of incident flow. Nevertheless, the non-linearities cannot be taken into account by this approach. Moreover, the transformation from flutter derivatives to indicial functions is not straightforward and therefore the identification of the parameters of these indicial functions could be problematic. This topic is not highlighted in the mentioned references. Another approach (Miyata 1995) adopts a quasi-steady formulation of the wind forces, considering the instantaneous angle of attack between the incident flow and the deck (ψ in Figure 10) and using the section model static aerodynamic coefficients measured in wind tunnel. The quasi-static formulation is as follows:

$$\begin{aligned}
F_L &= \tfrac{1}{2}\rho b V^2 C_L(\alpha) & F_z &= F_L\cos\psi + F_D\sin\psi \\
F_D &= \tfrac{1}{2}\rho b V^2 C_D(\alpha) & F_y &= -F_L\sin\psi + F_D\cos\psi \\
M &= \tfrac{1}{2}\rho b^2 V^2 C_M(\alpha) & & \\
\alpha &= \vartheta - \psi & V^2 &= \left(W - \dot{z} + b_1\dot{\vartheta}\right)^2 + \left(U - \dot{y}\right)^2
\end{aligned} \tag{1}$$

where F_z, F_y and M are respectively the vertical force, lateral force and moment per deck unit length, C_D,C_L and C_M are the static lift, drag and moment coefficients of the deck, measured as functions of the angle of attack α. Defining the vector $\{q\}$ as follows, equations (1) become:

$$\underline{q}^T = \{y \;\; z \;\; \vartheta \;\; U \;\; W\} \qquad F^T = \{F_y \;\; F_z \;\; M_x\} \qquad \underline{F} = \underline{F}\{q, \dot{q}\} \tag{2}$$

With this theory, deck motion, wind turbulence effects and static effects are all included at the same time and the dependence of the aerodynamic forces on the actual angle of attack, is considered. On the other hand, this approach does not take into account the dependence of the aerodynamic coefficients on the reduced velocity V^*=U/(fB) and therefore its applications should be in principle limited to those cases where V^* is sufficiently high (corresponding in other words to the situations where the time taken by the flow to cross the section is much shorter than the oscillation period of the structure or than the period associated with the incoming turbulence fluctuations, approaching the steady-state conditions). The aerodynamic forces acting along the deck can be calculated according to (2) and then reduced to the degrees of freedom of the structure, generally the nodal co-ordinates of a finite element schematisation, while in some cases modal reduction is also used. The non-linear motion equations have therefore the general form:

$$[M_s]\,\ddot{\underline{X}} + [R_s]\,\dot{\underline{X}} + [K_s]\,\underline{X} = \underline{F}_a(\underline{X},\dot{\underline{X}},t) \tag{3}$$

being $[M_s]$, $[R_s]$ and $[K_s]$ the structural matrices of the bridge, and $\underline{F}_a$ the vector of the generalised forces due to wind action, function of the bridge motion and of the space-time history of turbulent wind. In (Miyata et al. 1995), comparisons with the results of the 1:100 aeroelastic model of Akashi bridge are also reported. Expressions (1) are effective, as already said, for reduced velocities $V^*=V/\omega B$ sufficiently greater than 10. If V^* is smaller, these expressions fail and the dependence of these expressions from the reduced velocity must be introduced or another theory must be developed. In order to reach this goal, the "quasi-static corrected theory" was developed (Diana 1993b, Diana 1994). For a better understanding, the equations (2) are linearised around a reference angle of attack α_0 defined both by the value of the horizontal U_0 and vertical W_0 component of the wind, and by the motion of the section itself defined by the following values y_o, $\dot{y}_o$, z_o, $\dot{z}_o$, ϑ_o, $\dot{\vartheta}_o$. In other words, with reference to equation (2), $\underline{F}\{q, \dot{q}\}$ is linearised around $\{q_o, \dot{q}_o\}$ values giving:

$$\underline{F} = \underline{F}_0 + \left[\frac{\partial \underline{F}}{\partial \underline{q}}\right]_0 \underline{\Delta q} + \left[\frac{\partial \underline{F}}{\partial \underline{\dot{q}}}\right]_0 \underline{\Delta \dot{q}} = \underline{F}_0 + \left[K_{F0}\right] \underline{\Delta q} + \left[R_{F0}\right] \underline{\Delta \dot{q}} \qquad (4)$$

$$\left[K_{F0}\right] = \frac{1}{2}\rho b V_0 \begin{bmatrix} 0 & 0 & g_3(K_{Do}\cos\alpha_0 - K_{Lo}\sin\alpha_0)V_0 & 2(C_{D0}\cos{}_0 - C_{L0}\sin\alpha_0) & \left((K_{Lo} - C_{D0})\sin\alpha_0 - (K_{Do} + C_{L0})\cos\alpha_0\right) \\ 0 & 0 & h_3(K_{Do}\sin\alpha_0 + K_L\cos\alpha_0)V_0 & 2(C_{D0}\sin{}_0 + C_{L0}\cos\alpha_0) & -(K_{Lo} - C_{D0})\cos\alpha_0 - (K_{Do} + C_{L0})\sin\alpha_0 \\ 0 & 0 & a_3 K_{Mo} b V_0 & 2bC_{M0} & -bK_M \end{bmatrix} \qquad (5)$$

$$\left[R_{F0}\right] = \frac{1}{2}\rho b V_0 \begin{bmatrix} -2g_4(C_{D0}\cos\psi_0 - C_{L0}\sin\psi_0) & -g_1\left((K_{Lo} - C_{D0})\sin\psi_0 - (K_{Do} + C_{L0})\cos\psi_0\right) & g_2 b_{1y}\left((K_{Lo} - C_{D0})\sin\psi_0 - (K_{Do} + C_{L0})\cos\psi_0\right) & 0 & 0 \\ -2h_4(C_{D0}\sin\psi_0 + C_{L0}\cos\psi_0) & -h_1\left(-(K_{Lo} - C_{D0})\cos\psi_0 - (K_{Do} + C_{L0})\sin\psi_0\right) & h_2 b_{1z}\left(-(K_{Lo} - C_{D0})\cos\psi_0 - (K_{Do} + C_{L0})\sin\psi_0\right) & 0 & 0 \\ -2a_4 C_{M0} b & a_1 K_{Mo} b & -a_2 b_1 K_{Mo} b & 0 & 0 \end{bmatrix} \qquad (6)$$

being:

$$\begin{array}{llll} \alpha_o : \text{reference angle} & K_{Do} = \left(\dfrac{\partial C_D}{\partial \alpha}\right)_{\alpha=\alpha_0} & K_{Lo} = \left(\dfrac{\partial C_L}{\partial \alpha}\right)_{\alpha=\alpha_0} & K_{Mo} = \left(\dfrac{\partial C_M}{\partial \alpha}\right)_{\alpha=\alpha_0} \\ V_o^2 = \sqrt{U_o^2 + W_o^2} & C_{Do} = C_D(\alpha)_{\alpha=\alpha_0} & C_{Lo} = C_L(\alpha)_{\alpha=\alpha_0} & C_{Mo} = C_M(\alpha)_{\alpha=\alpha_0} \end{array} \qquad (7)$$

The coefficients $h_i = h_i(V^*, \alpha_o)$, $g_i = g_i(V^*, \alpha_o)$ and $a_i = a_i(V^*, \alpha_o)$ ($i = 1, 4$), corresponding to the "flutter derivatives" measured on section models (Scanlan 1971, Singh 1995, Zasso 1996), have been introduced in the aerodynamic forces equivalent stiffness and damping matrices in order to represent their dependence on the reduced velocity V^*. If the appropriate b_{1z}, b_{1y} and $b_{1\theta}$ values are introduced (the ones holding in quasi-static conditions), the consistency with a quasi-static approach is shown by the convergence to unity of h_i, a_i and g_i increasing V^*. No corrective coefficients were introduced for turbulence dependent terms. These expressions become similar to the Scanlan flutter derivatives formulation (Scanlan 1971) if no turbulence and no horizontal deck motion are considered and if a linearisation is done around zero motion of the deck. In this case matrices (6) and (7) become:

$$\left[\hat{K}_{Fo}\right] = \frac{1}{2}\rho b V_0^2 \begin{bmatrix} 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & h_3 K_L & 0 & 0 \\ 0 & 0 & a_3 K_M b & 0 & 0 \end{bmatrix} \qquad \left[\hat{R}_{Fo}\right] = \frac{1}{2}\rho b V_0 \begin{bmatrix} 0 & 0 & 0 & 0 & 0 \\ 0 & h_1(K_{Lo} - C_{D0}) & -h_2 b_{1z}(K_{Lo} - C_{D0}) & 0 & 0 \\ 0 & a_1 K_{Mo} b & -a_2 b_1 K_{Mo} b & 0 & 0 \end{bmatrix} \qquad (8)$$

Figure 12 shows, for the final Messina Bridge deck solution, two of the a_i^* and h_i^* corrective coefficients of the "quasi-static theory" normalised assuming b_{1z}, b_{1y} and $b_{1\theta}$ equal to the chord b and constant C_{D0}, K_{L0} and K_{M0} values for the different angles of attack (for a full reference see Zasso 1993a). The diagrams confirm that the "quasi-static theory" is effective with high V^* reduced velocities as the zero angle of attack coefficients go to unity for $V^* > 10$.

These linear expressions of aerodynamic forces will be used in the following discussion of the corrected quasi-static theory method: they are similar to those of Scanlan and the advantages are related only to a better physical understanding. The method consists in dividing the wind spectrum, as shown in Figure 13, into a low frequency range, labelled "0", and in many high frequency sub-ranges, labelled "1", "2", etc. The upper values of the "0" frequency range is defined by the lower value of the reduced velocity for which the quasi-static assumption is still valid, for example $V^*=10$. For each of these contributions, a separate space-time history can be generated: in the following U_0, W_0 represent the time histories of the horizontal and vertical components of the low frequency contribution ("0" range including the mean value), while ΔU_1, ΔW_1, ΔU_2, ΔW_2, ... represent the different high frequency contributions. It should be remarked that the hypothesis of the quasi-steady theory apply only to the "0" contributions.

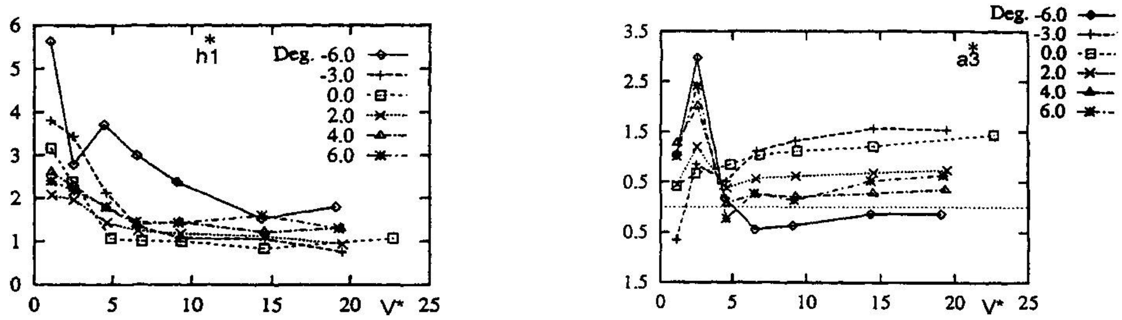

Figure 12 Final Messina Bridge deck design: "corrected quasi-static theory" a_3^* *and* h_1^* *coefficients versus reduced* V^* *wind velocity as a function of the mean* α_o *angle of attack [Deg]. The values here reported refer to the assumption of* b_{1z}, b_{1y} *and* $b_{1\theta}$ *equal to b.*

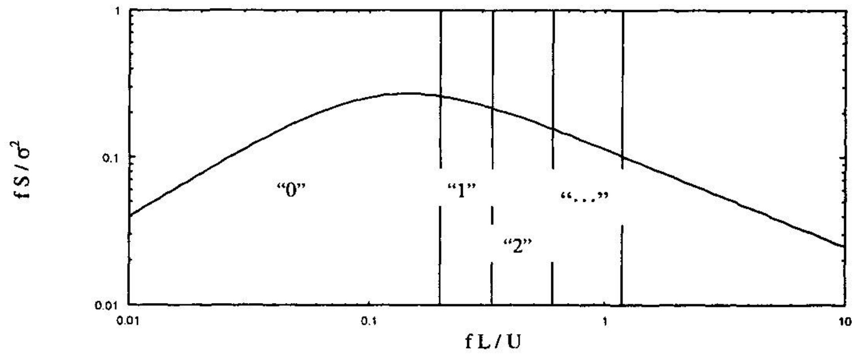

Figure 13 Normalised wind spectrum divided in frequency sub-ranges

In the same way, the total response of the bridge $\underline{X}(t)$ is considered as the superposition of different contributions $\underline{X}_0$, $\underline{X}_1$, ... , corresponding to the frequency ranges previously introduced for the wind turbulence:

$$\underline{X}(t) = \underline{X}_0(t) + \underline{X}_1(t) + \underline{X}_2(t) + \dots \tag{9}$$

As a first step, disregarding the effects of high frequency wind and motion terms, the low frequency response $\underline{X}_0(t)$ of the structure can be calculated according to the quasi-static theory, by means of the following set of motion equations:

$$[M_s]\ddot{\underline{X}}_0 + [R_s]\dot{\underline{X}}_0 + [K_s]\underline{X}_0 = \underline{F}_a(\underline{X}_0, \dot{\underline{X}}_0, t) \quad ^1 \tag{10}$$

In this way the non-linear dependence of the aerodynamic forces on the angle of attack is kept into account and, on the other side, the use of the quasi-steady theory is justified by the fact that in the considered range of frequencies the aerodynamic coefficients are reduced velocity-independent. For what concerns the high frequency contributions to the bridge motion, the motion equations are linearised around the low frequency component $\underline{X}_0(t)$ previously defined. Expressions (5) and (6) represent the linearised aerodynamic forces applied in a generic deck section, and being $\underline{X}_0$ a dynamic solution, the expressions become linear but with time dependent coefficients. This represents a reasonable approximation since the main variations of the angle of attack are related to the range of low frequencies. In other words, components ΔU_1, ΔW_1, are considered small with respect to U_0, W_0 and correspondingly, the $\underline{X}_1$, $\underline{X}_2$, ... components of motion are assumed small with respect to $\underline{X}_0$. In order to take advantage of the knowledge of $h_i=h_i(V^*,\alpha_o)$, $g_i=g_i(V^*,\alpha_o)$ and $a_i=a_i(V^*,\alpha_o)$, functions of the wind reduced velocity and of α_o, a

[1] The integration of this non linear equation is done numerically, filtering the frequencies that are over the "0" frequency range.

modal approach is introduced for each sub-range, considering only the modes pertaining to that particular range of frequency:

$$\underline{X}_1 = [\Phi_1]\underline{q}_1 \quad ; \quad \underline{X}_2 = [\Phi_2]\underline{q}_2 \quad ; \quad \ldots \tag{11}$$

where $[\Phi_1]$ represents the modal shapes matrix corresponding to the bridge natural frequencies falling in the "1" frequency range, $\underline{q}_1$ is the corresponding vector of modal co-ordinates and so on. A separate set of equations of motion is then written for each sub-range, neglecting the coupling terms between the different sub-ranges. In each of these sets of equations, the appropriate value of corrective coefficients is used, according to the value of the reduced velocity pertaining to the sub-range. It must be again pointed out that, since the equations of motion are linearised around the $\underline{X}_o$ solution, the flutter derivatives are modulated by the variation of the angle of attack corresponding to $\underline{X}_o$ solution, so that the equations are linear, but with time dependent coefficients. The results of this method have been compared with both full-scale measurements (Humber bridge, Diana 1990, Diana 1994) and with wind tunnel tests on a 1:250 scale full bridge aeroelastic model (Messina Bridge, Diana 1995). A detailed illustration of the experimental results compared to the numerical simulations of those test cases can be found also in (Diana 1998). Figure 14 shows a photograph of the Messina Bridge aeroelastic model in the Danish Maritime Institute (D.M.I.) wind tunnel in Copenhagen.

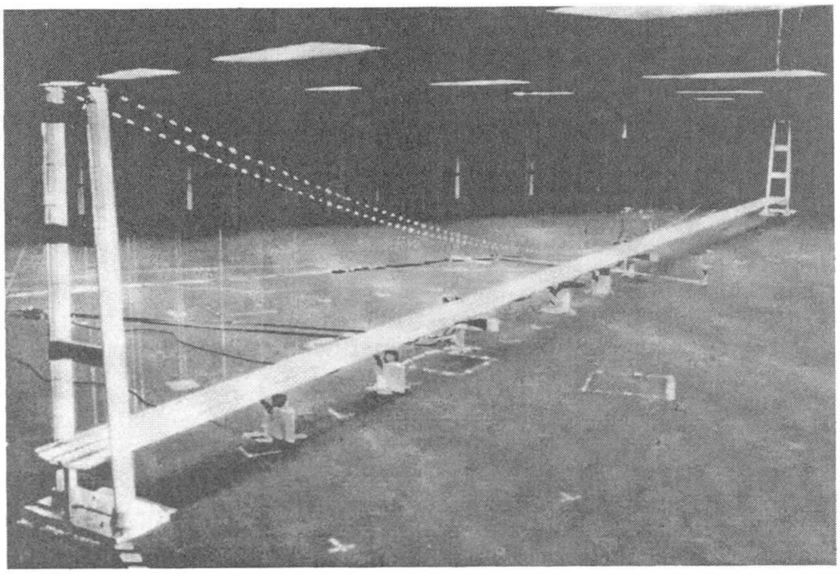

Figure 14 The Messina Bridge full model in the D.M.I. tunnel

The analysis and comparison of the numerical and experimental wind tunnel results was very interesting, showing substantial good agreement. The consistency of the experimental and numerical results in the over mentioned test cases gave confidence for the extension of the numerical simulation approach to the real Messina Bridge behaviour. As an example, Figure 15 shows the trend of the first torsional frequency of the bridge and the related damping factor as a function of the wind speed: these results are obtained both using the 3D full model in the experimental wind tunnel tests and using the numerical simulation model. As it can be observed, a good correspondence between the numerical and the experimental results is obtained. The not negligible difference in the experimental values of damping factor obtained in smooth-flow and turbulent-flow conditions is anyway something that needs further investigations. It can be underlined the importance of these kind of analysis as they allow to obtain some meaningful parameters for the stability definition.

The knowledge of the stability index ($h = r/r_c$), as a function of the mean wind speed, allows both to evaluate with precision the instability threshold (defined as the wind velocity corresponding to zero value of the non-dimensional damping factor) and also to define the stability index value at the design maximum velocity (62 m/s for the Messina bridge). In the Messina Bridge case, the instability index value at the design wind speed is 4% for the first torsional mode (see Figure 15), being the non-dimensional structural damping factor $\cong$1%. This result means that the aeroelastic deck design adds to the bridge an high aerodynamic damping, at the maximum design wind speed. A high value of the instability index means also a low buffeting response of the bridge to the turbulent wind as it was confirmed from the measured response of the full model in wind tunnel tests and from the numerical simulation.

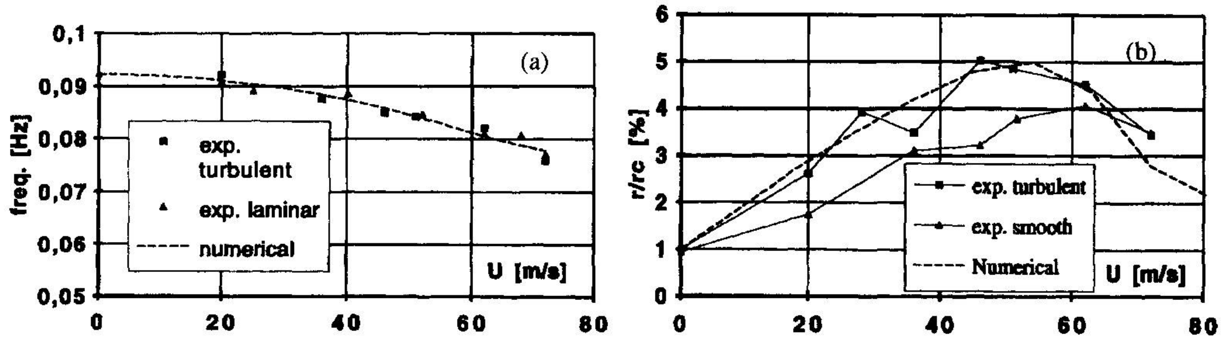

Figure 15 Messina bridge: comparison between numerical and experimental wind tunnel full model results: first torsional mode: (a) frequency and (b) damping factor variation as a function of the wind speed (real scale)

5 Concluding Remarks

The following conclusions can be drawn:

- Some possible strategies for the design of suspension bridges with span length greater than 3000m are available, and some of them have been proved to be feasible.
- The accurate simulation of the bridge response to turbulent wind, and the estimate of the corresponding instability indexes, are fundamental tools for the assessment of the bridge feasibility and for its final check.
- On the other hand, simplified methods, which do not take into account all the aspects of the aeroelastic problem, if applied to non conventional solutions, may result highly inaccurate for the selection of an optimal solution. Particular care is required in the separation of the static problem from the dynamic one, or in the modal approach applied to non conventional typologies, where limiting the analysis of the critical speed to only a couple of modes could be misleading.
- The development of the field of wind engineering, with particular reference to wind-bridge interactions, requires a validated procedure for the calculation of the bridge response to turbulent wind: this could be done by means of a benchmark test on the existing simulation codes.

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