ELSEVIER

Available online at www.sciencedirect.com

ScienceDirect

Procedia Engineering 14 (2011) 1493–1500

Procedia Engineering

www.elsevier.com/locate/procedia

The Twelfth East Asia-Pacific Conference on Structural Engineering and Construction

The First Results for a New Layout of the Stay Cables for Great Span Bridges

L. ANANIA, A. BADALÀ , S. COSTA, and G. D'AGATA

Department of Civil and Environmental Engineering, Catania University, Italy

Abstract

The air-elastic vibrations of structures induce fluctuating stresses that lead to fatigue damage accumulation and may determine structural failure without exceeding ultimate strength This paper proposes a new layout of stay cables to be used in the construction or the retrofitting of long span bridges, capable of mitigating the air-elastic problems due to environmental vibrations such as the rain-wind excitations. The structural scheme adopted was derived from the critical conditions in terms of stability obtained by referring to lateral suspension cables stayed bridge with two planes of fan pattern stay cables. The new layout consists in implementing an additional plane of cable stays placed symmetrically just under the deck bridge. The final layout of the cable stays was identified as "duplex". The numerical investigation was carried out in the frequency domain. The results obtained show a sensible increasing of stiffness, as well as a reduction of the natural period of vibrations. In the analysis the deck was considered as thin and very light. The Duplex layout had, also, permitted to mitigate the wind effects, because the presence of the inferior stay cables simulate the viscous dampers.

Keywords: Cable stayed bridge; air-elastic effects; pattern stay.

1. INTRODUCTION

Research on the effects of wind on the bridges began often the collapse of the famous Tacoma bridge. The aim of the paper is to investigate a new scheme of a static cable-stayed bridge that would make this kind of structure, less vulnerable to the insidious effects of the wind. To reduce uncertainties to a minimum, a methodology of comparative study was adopted. We tried to maintain a structural scheme base common to the various bridges examined by varying only the stay cables pattern. It is possible to operate in the certainty that all the differences in the overall behavior of the structure were precisely

doi:10.1016/j.proeng.2011.07.188

attributable to this structural component. The studies in their preliminary nature, were then allowed to verify that the solution devised "worked", but we need further researches to understand exactly how it did. The proposed solution was therefore conceived as an application for the design and construction of new bridges rather than as an instrument to be applied to the existing bridges. Therefore, it was decided to start from scratch by designing an ordinary cable stayed bridge with span of 700 m. This is a medium-sized span that has been specially chosen as a good compromise between the necessity to have a large and deformable structure, to feel the wind effects, and also to calibrate a model of calculation that is not too burdensome from a computational point of view. The ordinary stay cables layout was designed from the start, not only as an independent structure, but also as the basis on which to develop the innovative schemes proposed, from a structural point of view. This also allowed for continued comparative analysis ensuring an overlapping of the full results. This necessity led to the precise choices in defining the overall geometry of the work and, above all, in the arrangement of the stay cables layout.

2. THE NEW LAYOUT OF STAY CABLES

The idea of the new stay cable layout proposed, was prompted by the observation that the wind induced bending-torsional oscillations mainly on the scaffold. Those are mainly due to the bending phenomena of vortex shedding and galloping, while the torsional effects, which may also occur as a secondary response to these two phenomena, are mainly due to flutter; normally attributed to this phenomenon, there are coupled bending-torsional oscillations that can also lead to the collapse of the structure. It was therefore decided to introduce a further impediment to these oscillations, by assisting the effect of the ordinary stay cables, with another stay cables pattern acting from the bottom. Its working principle is illustrated in Figure 1 where we have a comparison with the more ordinary stay cable.

In the presence of bending-torsional oscillation (as well as in purely torsional ones) a state of stress is produced in the supporting cables, opposite to the case of an ordinary stay cable layout; in this condition the “Bs” stays tend to be compressed and this results in a compressive stress that reduces the initial “pre-stress” and the stiffness of the stay. However, this condition will be absorbed until the wire maintains its initial pre-stress, but if the compression exceeds this value, there would be the total loss of stiffness and the buckling of the cable. With the introduction of the inferior stay cables we plan to exclude the possibility of a loose stiffness in the cable, by activing, by an appropriate adjustment of the pre stress, a continuing state of tension in all stays. In practice, we want to make sure that even in the presence of strong torsional oscillations the scaffold will never lose one of its side supports, because any failure (excessive compression) of one of the stay cables located at the top will be immediately offset by the pretensions of the correspondent lower.

In fact, the pair stays “Bs” and “Bi” behave exactly equal and opposite to the “As” and “Ai” stays. To increase its stiffness we will need to reduce the lower stay “Bi” while higher Bs. Thus, assuming to place the inferior stay cables as an exact mirror image of the top stay cables layout, each end of the deck will be supported by an elastic constraint that can produce a stiffness that can be equal to or double if compared to that of the individual, stay taking into account that the increases and reductions in axial stiffness compared to the initial one, due to the changes in the pre-stress cables.

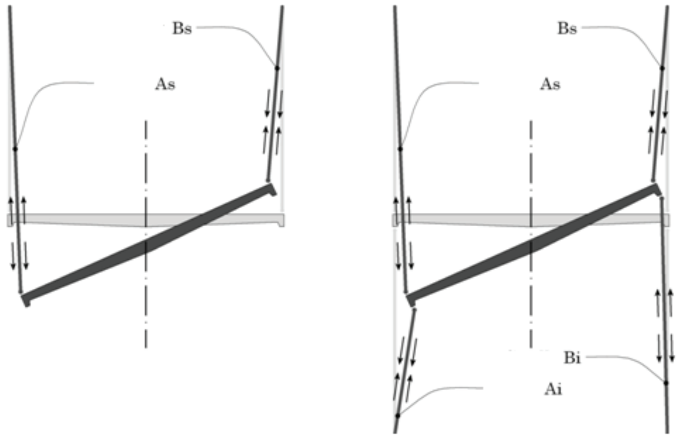

Figure 1: Working principle of the proposed stay cables layout

3. THE STUDIED BRIDGE

By inserting of the stay cables at the bridge bottom we want to give more than the bilateral function of support to the scaffold offered by ordinary stays. In fact they act as bilateral until they can absorb compressive stress, maintaining the initial pre-tension, but they suddenly become one-sided when they lose the latter. At this stage, the pretension of the bottom stays is capable of re-establishing the bilateral nature of the bond. It is clear that this function can be unlocked only in the presence of a double curtain stays on the sides of the deck in two vertical planes, by invoking the scheme and the layout of the fan stays with a fairly small step of 15 mt. Therefore, to allow a uniform distribution of the stays throughout the scaffold, a central span of 690 mt was assumed, while those of the shore side were 232.5 mt, according to what has already been shown, these dimensions should avoid stability problems of the mooring stays. To maintain a slender bridge, the ratio between the height of the towers and the central span of the scaffold about 0.2 was adopted an height of the towers of 160 mt resulting in a slenderness of 0.23, optimal value within the range considered by some authors [4]. To allow maximum freedom in the positioning of the bottom stays, it was decided to place the deck at a height of 160 m from the foundations, so the two towers reach a height of about 320 mt.

3.1. The “SIMPLEX” scheme

The global dimension of the SIMPLEX scheme is shown in the figure 2. The pre-dimensioning of the stays was conducted so that the tension in the cables is such as to maintain the straight configuration of the bridge girder under the action of only permanent loads. However, it was also taking into account the effects of variable loads by setting a stay cables work rate of 0.5, given by the ratio $T_{0\,\max}/F_{\max}$ between the maximum in the cable initial prestressing force and the maximum workload provided by the manufacturer. Then, the sizing of the stay was obtained by solving the equivalent continuous beam scaffold, and replacing the supports fixed to the stays. Therefore, wanting to create a lightweight steel bridge deck, a dead load of 7.5 ton/mt has been fixed. For the scheme adopted, with the stays / supports arranged at a distance of 15 m, we obtained a value of the reaction Ri of 1100 kN . These values were, however, halved to take into account the presence of two planes of the stays supporting the deck.
The final sizing of the stay cables was carried out by taking into account the real configuration of the cable under the its dead load.In the SIMPLEX scheme the tower adopted is reported in figure 2. The

transversal beams are placed at about 7 m from the deck extrados, they must be so to form a rectangle stiffening of 40x42 m at an altitude of about 150 m from the foundations.

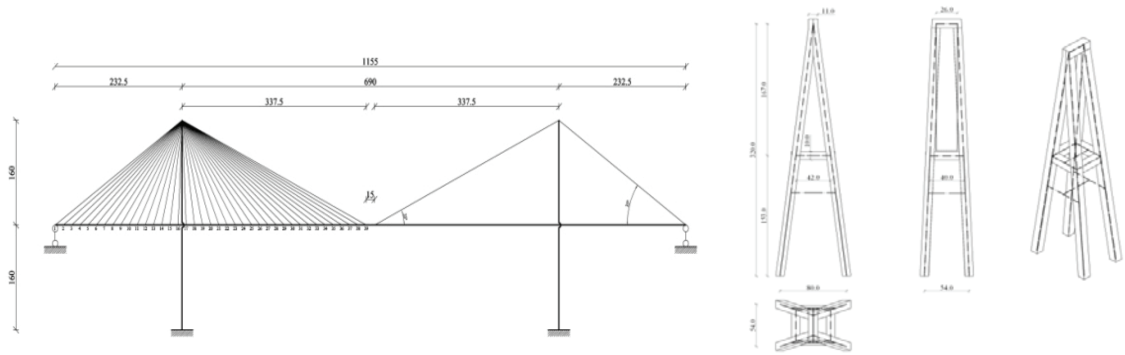

Figure 2: SIMPLEX scheme

3.2. The "DUPLEX" scheme

The global dimension of the DUPLEX scheme is plotted in the figure 4-5. For the stay cables sizing, also in this case a working rate $T_{0\,\max}/F_{\max}=0.50$ of 0,5 was considered.

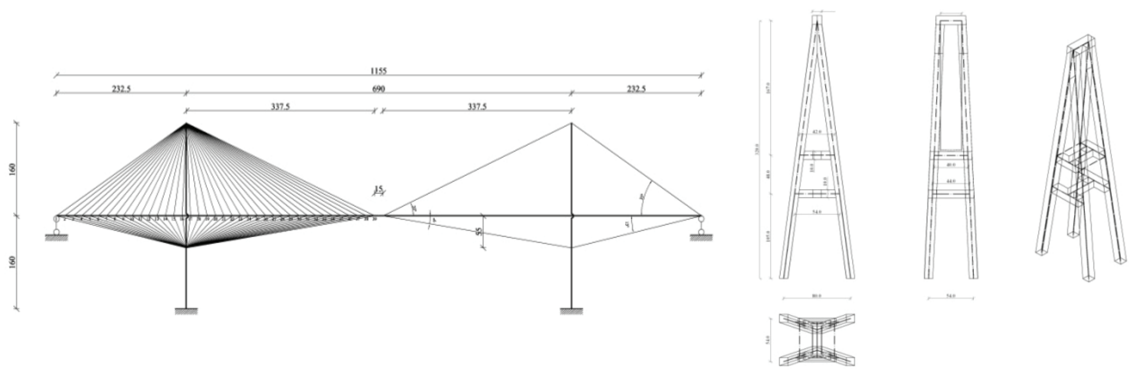

Figure 3: DUPLEX scheme

From an initial comparison between the upper and lower stay cables, we notice that, the number cable (*n)* to be employed in many cases is significantly higher, despite the fact that the inferior stay cables must balance a vertical action that is a little less than half of the one of the upper stays. This is due to less axial efficiency of the inferior stays which have a less slope and therefore they are more affected by the dead load. In addition, the insertion of the bottom stays requires a recalibration of the tension of the top ones and sometimes also their re-dimensioning.
Four different structural patterns were analyzed:
DUPLEX Ti 0.8. This scheme was derived directly from the SIMPLEX routine by adding a stays curtain in addition to those above. Since the ideal formulation had to be characterized by a double stays pattern perfectly specular to the top, we immediately decided to place them in a situation nearer to a hypothetical condition realization and to reduce the stay cable tension to 80% of the one originally calculated, in order to avoid buckling phenomena.

It was therefore decided to locate the point of convergence of the inferior stays at a distance of approximately one third of the upper stays.

DUPLEX Car H Ti 1 Ts Agg: Obtained from the previous model by replacing the pendulums at the ends of the deck of the sliding bearings that allow only the vertical displacement, the tension in the stays located below was applied as initially assessed while those in the upper stays have been updated accordingly.

DUPLEX TH Ti 1 Ts Agg It differs only in the presence of horizontal pre-stressed rods able to absorb the normal stresses girder.

DUPLEX TH DMP. This scheme was derived from the previous one by interposing some viscous dampers in the nodes of convergence of the stay cables and attachment points in the girder, in order to reduce the magnitude of the structural response.

All those schemes have been subjected to the usual hierarchy of cases of non-linear static analysis for the implementation of its dead loads and various combinations of the pre stressing load.

By plotting the normal stress trend in the deck at the end of the loading process under the same conditions (figure 4), both in terms of loads and stiffness, we see that *DUPLEX TH scheme* with the inclusion of horizontal rods is the ideal solution since it produces a reduction of the normal strength also if compared to the ordinary stay cables pattern. The stretch zones increase at the ends of the spansThe introduction of horizontal rods should be designed as to absorb the entire horizontal action transmitted by the stays on the girder as a result they will be arranged between the nodes of convergence of the stays, placed symmetrically in the transverse plane of the scaffold. In the DUPLEX TH scheme, therefore, all three rods of cable ends form, at the central span of the bridge, a self-supported structural element similar to a typical stress-structure regardless of the presence of the deck (figure 5).

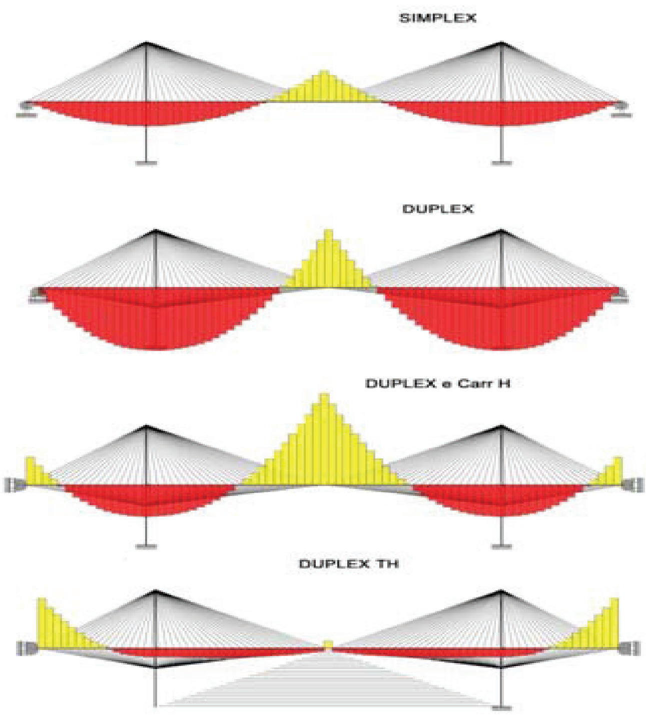

Figure 4: Normal stresses in 2D bridge model

4. Numerical investigation ANALYSIS OF THE RESULTS

The methodology used provides a preliminary analysis which assesses the deformability of the system under the action of static loads and a nonlinear modal analysis capable of evaluating the dynamic behavior. All tests have been carried out by providing a precise succession of non-linear static analysis cases through which all those actions that affect the stiffness have been applied to the structure. The behavior under wind actions was subsequently investigated both through a classical flutter analysis and by evaluating the extent of vortex shedding.

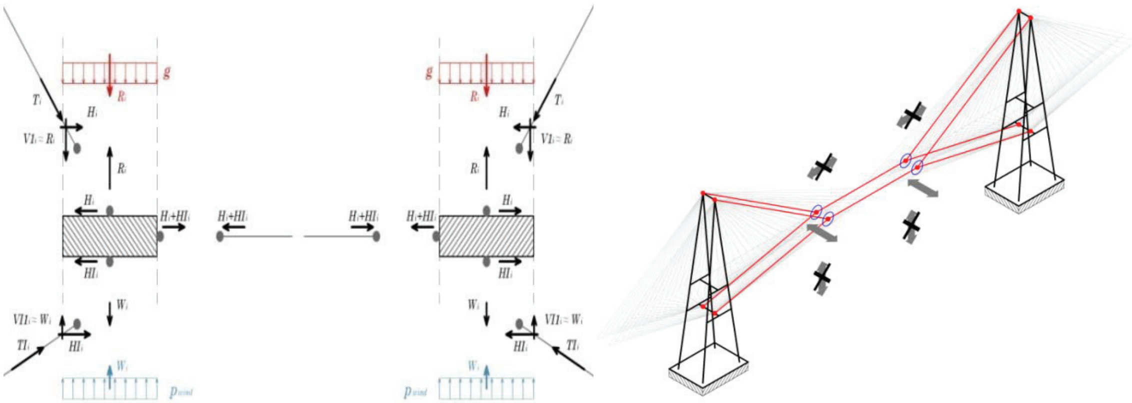

Figure 5: Equilibrium conditions (a) and self supported scheme (b)in presence of horizontal load

Among all those modes derived from the analysis the most important ones for the aeroelastic behavior of the bridge, are only purely flexural and torsional ones, this is true in particular for the classical flutter behavior (Theodorsen: 1935; Scanlan, et co1990).

Table 1: Periods of the selected main mode shapes in different scheme

Mode	***Period*** [s]									
	SIM.		*DUP. Ti 0.8*		*DUP. CH*		*DUP. TH*		*DUP TH DMP*	
	n	***T***	***n***	***T***	***n***	***T***	***n***	***T***	***n***	***T***
Lat xy I	1	17.42	1	13.28	1	8.63	1	8.30	1	8.33
Lat xy II	3	5.81	4	5.23	4	3.66	4	3.63	4	3.63
Bend zx	4	2.79	9	2.54	8	2.43	11	2.49	11	2.48
Tors yz	11	1.71	25	1.54	21	1.43	22	1.48	20	1.46

Table 1 reports the period obtained from the schemes studied regarding the mode shape just defined. It shows unequivocally that the various schemes adopted result in a significant stiffening of the whole structure. In particular, a decrease of periods that are, of course, representative of the stiffness of the system between 9% and 52% has been achieved. It is therefore important to assess how this increase in stiffness influences the wind structural response. The frequency analysis of the deck for bending and torsion actions lead to the determination of areas that define the frequency response of the deck for the various schemes tested. In the figure above there are the graphs obtained dissecting these areas with a vertical plane passing through the x-axis of the bridge where the maximum response occurs. This usually happens at the mid span section of the deck (figure 6-7) The peaks of the various responses are located at the frequency values which are almost identical to those of both first flexural and torsional modes as evaluated by the previous modal analysis. Moreover, these frequency values, in particular those relating to the SIMPLEX scheme, have been assigned as harmonic forces capable of simulating the phenomenon of vortex shedding. Figures 8 and 9 show that the structural scheme in which the maximum increase in both flexural and torsional stiffness, is the one called DUPLEX CH *CH Ti 1 Agg*. The models in which there are also the horizontal rods, including the one with the dampers, the responses are almost identical. The results of the frequency analysis were used to draw the critical speeds in the various sections of the deck.

We see that in all the schemes, the highest vulnerability condition to the wind for classic flutter instability, occurs at the mid span (figure 8) in which, in effect, the stiffening action of all the stays (although only ordinary ones) is minimized. In contrast, in all the schemes with tested system DUPLEX, the stiffening effect due to the double stay cables pattern is so high close to the tower that the critical condition of flutter instability cannot occur (figure 9). These two curves show the increase in the critical velocity obtained through the introduction of the new stay cables scheme. We have moved from a value of 82.3 m/s for the SIMPLEX scheme to a value 97.5 m / s for DUPLEX CH Ti. So an increase in wind resistance of 19.0% resulted, and this may however be considered aligned to the increase of stiffness evaluated by the modal analysis, although it is slightly higher than the increase recorded in the first and bending modes of the deck. At the same time, in the SIMPLEX scheme, the section near the tower, however, is more rigid, in fact the condition of instability occurs, but for a very high value of the critical speed equal to 180.7 m / s.

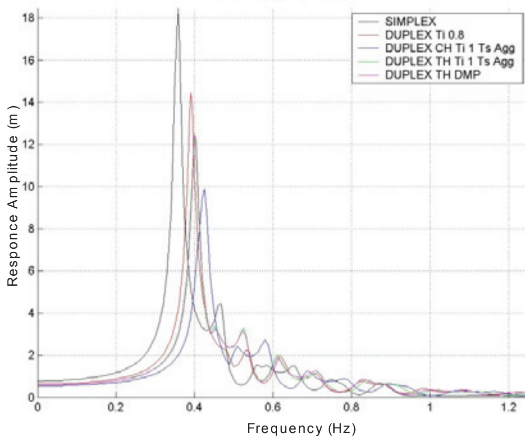

Figure 6: Torsional response at mid span

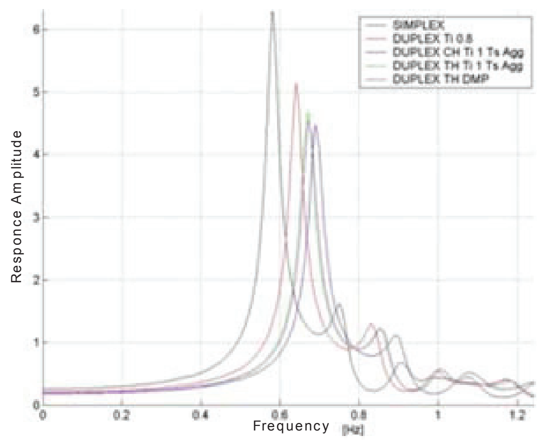

Figure 7: Flexural response at mid span

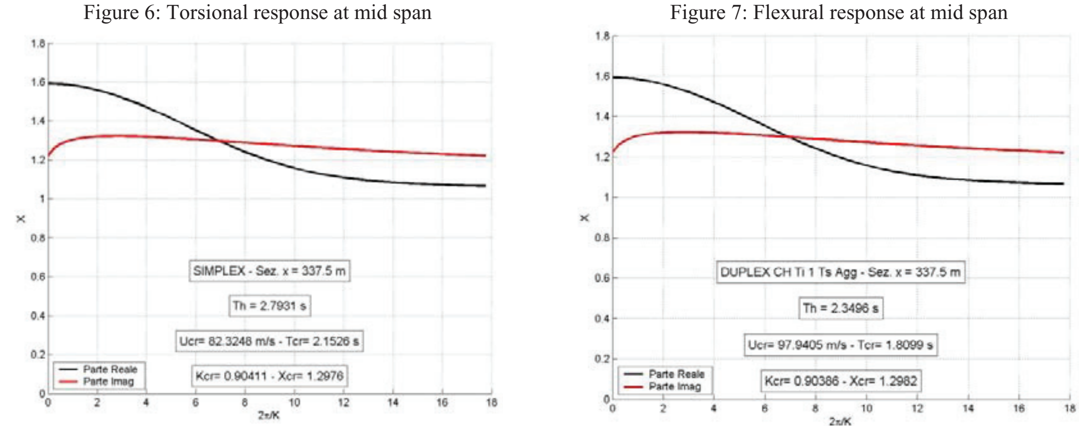

Figure 8: Critical flutter speed at the mid span

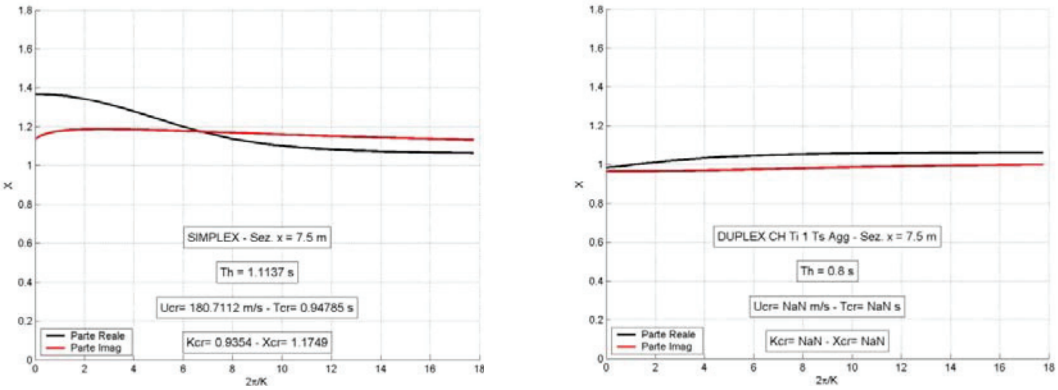

Figure 9: Critical flutter speed at the mid span

5.CONCLUSIONS

In this work we are discussing a new scheme of stay cables characterized by the presence of a stay cables pattern located at the bottom of the girder. In general, all the schemes tested have shown a significant increase in overall stiffness, as evidenced by a reduction of the natural periods of vibration. The effect that was achieved without action on a deck that had already been chosen very slender and lightweight. The increase in stiffness and overall was between 10% and 15% for bending and torsional modes of the main schemes tested. The study of an aero-elastic bridge has shown that the increase in the values of critical speed is slightly higher than the increase of stiffness to further confirm the effectiveness of the solutions adopted. With regard to the critical speed is necessary to emphasize that, in absolute terms, those obtained are rather high. These values, however, have been reflected in the literature with specific reference to cable-stayed bridges. These in fact, have a significantly higher stiffness compared to suspension bridges, making them less vulnerable to the effects of the wind. Also, we must not forget that all aero-elastic studies were carried out with reference to the theory of Theodorsen on thin airfoils. The deck although slender was treated as if it were a wing and not a bluff body, and given an aero-dynamicity that it doesn't actually possess. So, it is right to underline that even in suspension bridges with a very aerodynamically efficient deck, you can reach high critical speed. This is the case of Messina bridge where in some cases the critical speeds of 200 m/s is reached (Diana, 2003). So in light of the results obtained from the aeroelastic instability analysis, we can say that the solution proposed is effective in providing an increase of scaffold rigidity to increase the resistance to flutter, however, by acting externally outside without changing the structure of the deck.

REFERENCES

[1] T. Theodorsen: *General Theory of Aerodynamic Instability and Flutter*, NACA report No. 496, 1935

[2] R. H. Scanlan, N.P. Jones: *Aeroelastic Analysis of Cable-Stayed Bridges*, J. Struc. Eng., ASCE, 116(2) 1990, 279-297

[3] G. Diana, et al.: *The Aeroelastic Study of the Messina Straits Bridge* Natural Hazards 30: 79-106, 2003, Kluwer A.P.