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Objekttyp: **Article**

Zeitschrift: **IABSE congress report = Rapport du congrès AIPC = IVBH Kongressbericht**

Band (Jahr): **13 (1988)**

PDF erstellt am: **11.09.2024**

Persistenter Link: https://doi.org/10.5169/seals-13102

Behaviour of Long Span Suspension Bridge Construction

Comportement des ponts suspendus de grande portée lors de la construction

Verhalten weitgespannter Hängebrücken während des Baues

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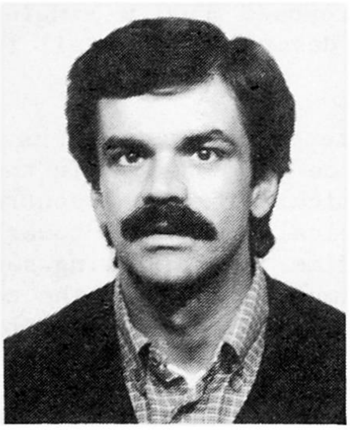

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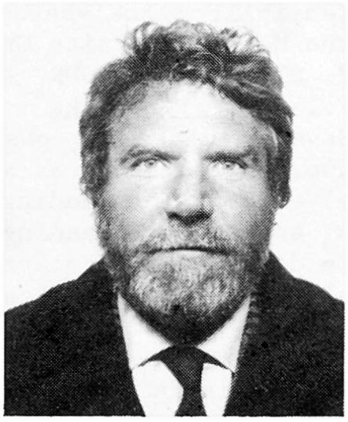

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SUMMARY
Suspension bridge elements can undergo during construction displacements and stresses comparable with those experienced in the service stages, as a consequence both of erection procedures and of environmental effects. The case study of a single span bridge for the crossing of the Messina Straits is presented. Erection simulations are shown for different possible sequences, discussing the structural behaviour, with special attention to aerodynamic stability.

RÉSUMÉ
Pendant la construction, les éléments des ponts suspendus peuvent être sujets à des déplacements et contraintes comparables à ceux rencontrés pendant l'exploitation, soit à cause des procédures de construction, soit des actions locales. On présente ici le cas d'un pont à une travée sur le Détroit de Messine. On montre les simulations de construction dans différentes séquences possibles, en analysant le comportement de la structure, en particulier la stabilité aérodynamique.

ZUSAMMENFASSUNG
Während des Baues können die Elemente der Hängebrücken Verformungen und Belastungen erfahren, ähnlich denjenigen im Gebrauchszustand, dieser Beitrag behandelt die Hängebrücke für die Meerenge von Messina, und die Bauvorgänge für verschiedene mögliche Folgen, mit speziellem Gewicht auf dem strukturellen Verhalten und der aerodynamischen Stabilität.

1. INTRODUCTION

Different structural safety aspects arise during suspension bridges erection stages, both due to construction procedures and to environmental actions. Attention is focused on the deck erection phase, during which the suspension cables and hence the deck undergo considerable displacements, often comparable or greater then those experienced in service, with the associated stresses in the girders. Hangers slacking phenomena can occur, with consequent overstresses in both deck and hangers in the zones involved. Furthermore, it is known that the danger of classic flutter type aerodynamic instability is considerably increased, due to the closeness of the flexural and torsional mode periods, as put into evidence by the damages suffered for the second Firth of Forth Bridge and by the attention given to prevention measures for the Humber Bridge [10,12,13]. A synthesis of the researches carried out on this topic within the feasibility analyses for a proposed 3300 m single span suspension bridge for the Messina Straits Crossing, described in detail in [7], is herein reported.

2. STATIC BEHAVIOUR

2.1 General remarks

Several different possible erection sequences have been analyzed: the first two, to be viewed as a reference and standard in the field, are:

a) erection proceeding symmetrically from the centre span,
b) erection proceeding symmetrically from the towers

To decrease the construction time, the following sequence was also studied:

c) erection symmetrical and simultaneous from the towers and the centre span

A final alternative considered arised by the possible use of a single high performance crane vessel in place of a number of traditional devices. Such cranes have a large lifting capacity but a low motion speed and must hence be used continuously on a same advancement front. This produced the need to analyze a number of unsymmetrical sequences, among which the following is presented:

d) erection proceeding asymmetrically from the centre span, but for two 200 m segments at the towers erected independently from the shore approach. Maximum asymmetry considered was 600 m.

2.2 Modeling and analysis procedure

The numerical model adopted is based on a non linear elastic finite element discretization, with the use of non linear beam, straight cable and curved cable elements. Effects of large displacements and of the addition of structural parts in sequence are taken into account via a modified iterative direct stiffness Newton-Raphson approach in which, once the solution of a set of non linear equations corresponding to a given stage is reached, a separately defined substructure is added through a node connection definition and the solution procedure restarted. The new substructure can be initially prestressed and can be connected to the existing part in any geometric position.

2.3 Displacements and stress response

Exemplificative cables and deck shapes are reported in fig. 1, with respect to the reference state at construction completion. A synthesis of the displacements response is given in fig. 3, where the centre span vertical displacement evolution during construction is shown for the four sequences.

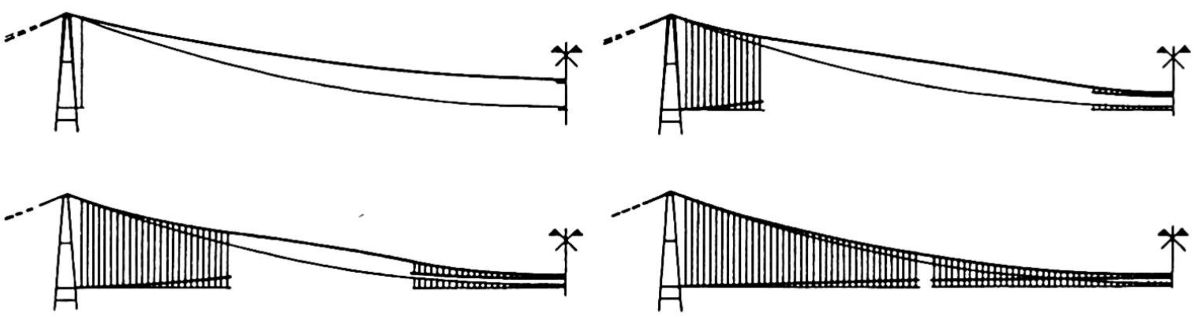


Fig. 1 - Displacements of cables and deck in different erection stages

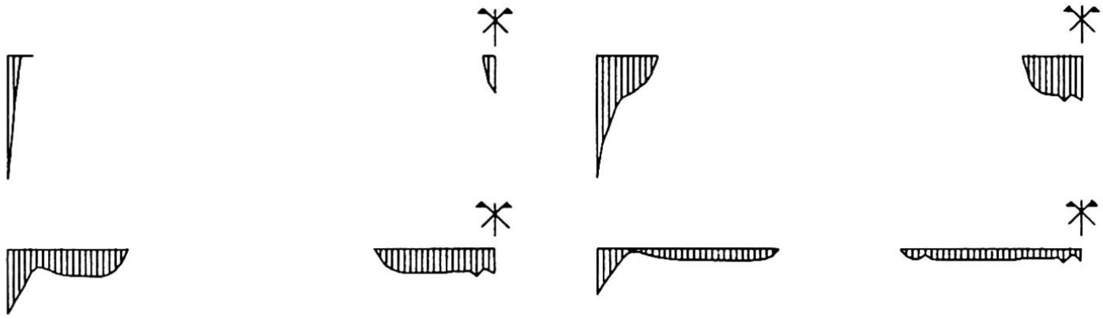
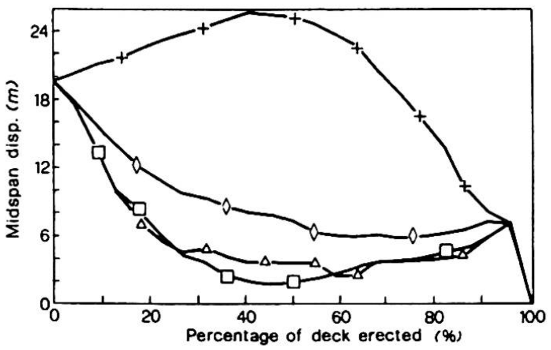


Fig. 2 - Deck bending moment distributions in different erection stages



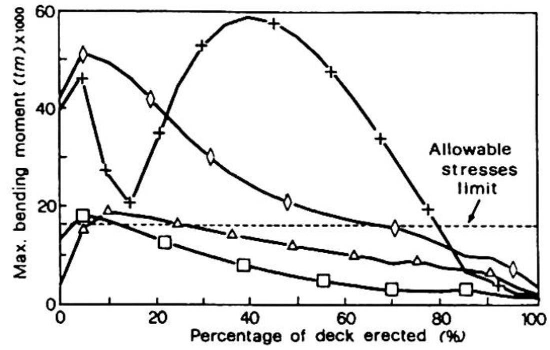


Fig. 3 - Midspan displ. evolution, Fig. 4 - Max. deck bending moment
□ *sequence a), + sequence b), ◊ sequence c), Δ sequence d)*

As to generalized stresses, fig. 2 shows four sample bending moments distributions, while fig. 4 reports the evolution of the maximum bending moment in deck, in the hypothesis of complete connection during construction. Cusps in graphs correspond to changes in the zones in which the moment occurs. As it can be seen, all the sequences present, in different measure, parts in which the limit allowable stress minimum moment is transpassed, indicating the need for a more flexible partial connection even on such a large span.

3. AERODYNAMIC STABILITY

3.1 General remarks

This field has received a wide attention in the last twenty years: fig. 5 summarizes the literature results available for different bridges of span from 700 to 1410 m, showing the relative evolution of the critical instability windspeed versus percentage of deck erected [10,12,13,14].

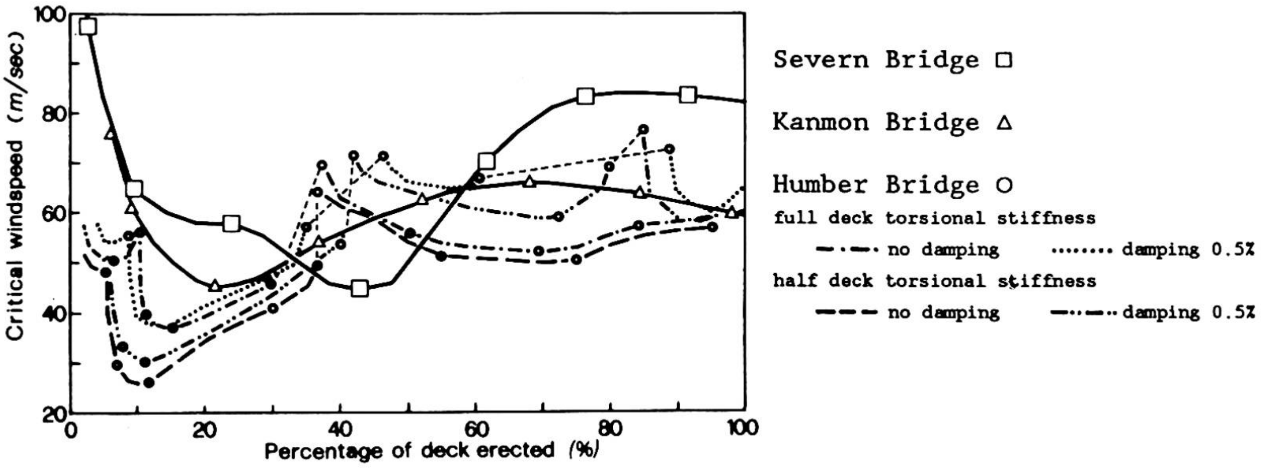


Fig. 5 - Critical windspeed versus percentage of deck erected

The minimum critical speed is typically associated to the early erection stages, in which the deck geometric stiffness contribution and translational-rotational inertial properties are negligible with respect to those of the suspension cables and hence the natural frequencies of modes subjected to possible coupling in the dynamic instability are closer then at structure completion. The subsequent evolution of structural dynamic properties causes a frequency spacing and a mode shapes modification; the former aspect implies higher critical speeds, the second changes in the mode coupling. Consequently different modes are involved in the instability, which becomes even impossible in certain stages [1,2,3,4,10]. To be noted that for the Humber Bridge, whose critical speed reached a minimum around 25 m/sec, measures were taken to prevent instability during erection [10,12]. In the following the problem is analyzed for the proposed bridge, with reference to construction sequence c).

3.2 Arrangement of the proposed deck

The proposed deck, fig. 6, was conceived to be aerodynamically neutral and "transparent". This target was obtained through a careful study of the air flow around and through the deck: highway and railway box girders A are spaced with grids B, so decreasing the pressure difference between the upper and the lower part and hence the aerodynamic lift. The longitudinal external girders C are for feasibility purposes wing-shaped, with inclination determined by wind tunnel tests, so as to act as stabilizers. To make the bridge aerodynamic behaviour independent of the road traffic presence, a lateral aerodynamically transparent curved grid protection D was adopted.

3.3 Procedures adopted for instability windspeed determination

The bridge structured was modeled via a spatial finite element approach [5,6,7]. The corresponding discrete non linear equations of motion are:

$$\underline{M}_p \underline{X}_p + \underline{R}_p \underline{X}_p + \underline{K}_p \underline{X}_p = \underline{F}_{aer}(U(t,s),\underline{X}_p,\underline{X}_p,\underline{X}_p) \qquad (1)$$

with $\underline{M}_p$, $\underline{R}_p$, $\underline{K}_p$ mass, damping and stiffness matrices; $\underline{F}_{aer}$ are the external generalized wind forces, defined according to the steady state theory [1,3,4,9]. The force components per unit length are given as, see fig. 7:

$$\begin{aligned} F_y &= 1/2\ \rho\ V_r^2\ B\ (C_d(\alpha)\ \cos\psi + C_l(\alpha)\ \sin\psi\ ) \\ F_x &= 1/2\ \rho\ V_r^2\ B\ (C_d(\alpha)\ \cos\psi - C_l(\alpha)\ \sin\psi\ ) \\ M &= 1/2\ \rho\ V_r^2\ B^2\ C_m(\alpha) \end{aligned} \qquad (2)$$

$$\text{with} \quad \tan\psi = x/V + \theta\ b_1/V, \qquad \alpha = \theta - \psi \qquad (3)$$

The aerodynamic coefficients are determined through static experimental wind tunnel tests on deck sectional models, while b_1 is determined through oscillatory tests, also in wind tunnel. Once defined a wind space-time history U(s,t), the bridge response could be obtained via direct integration in time of eq.s (1) [6,8,10,11], but this procedure is too time-consuming for a systematic research. If a constant windspeed $U=\overline{U}$ is assumed and eq.s (1) are linearized in the neighborhood of the static equilibrium configuration [8], they become:

$$(\underline{M}_p+\underline{M}_a)\ \underline{X}_p + (\underline{R}_p+\underline{R}_a)\ \underline{X}_p + (\underline{K}_p+\underline{K}_a)\ \underline{X}_p = \underline{0} \qquad (4)$$

where $\underline{M}_a$, $\underline{R}_a$, $\underline{K}_a$ account for the linearized terms of the aerodynamic forces. The instability type and critical speed can then be evaluated analyzing the real part of the eigenvalues of eq.s (4), for variable U. Numerical problems make preferable the use of eigenvalues-vectors calculated for the structure alone, i.e. without the said aerodynamic terms, see [6]. A third alternative considered is the so called forced method [4]: fig. 8 shows a comparison between results obtained by the different approaches: the instability threshold is determined with good agreement, while the total (structural + aerodynamic) damping factor evolution presents considerable differences, see [6] for a discussion of the problem.

3.4 Validation of the theory with a two d.o.f. sectional model
A preliminary investigation was devoted to a validation of the theoretical results via experimental tests on a sectional model. The deck, in the configuration described, is extremely stable: the flutter speed of the physical model was higher than the maximum obtainable in the wind tunnel (60 m/sec). To allow comparisons the critical speed was reduced closing the central grids, obtaining the aerodynamic drag, lift and moment coefficients (C_d, C_l, C_m) versus deck rotation shown in fig. 9, while those of the deck with open grids are reported in fig. 10. In the tunnel tests the experimental critical speed of the model with closed grids was 38 m/sec, while the analytical threshold was calculated as 36 m/sec.

3.5 Synthesis of results for the full bridge
Proceeding to the behaviour of the full bridge, fig. 11a shows the first symmetric and antisymmetric flexural and torsional natural modes of the bridge with 30 % of the deck erected, while fig. 11b shows the same modes when the deck is erected at 80%. In fig. 12 the variations of the first natural flexural and torsional frequencies as a function of the percentage of the erected deck are reported. The minimum ratio between the periods of the analogous flexural and torsional modes are 1.32 for the antisymmetric modes and 1.23 for the symmetric one when the deck is entirely assembled; the same ratios reduce to 1.06 and 1.16 when the deck is erected at 20%. Fig. 13 shows the trend of the critical speed versus the percentage of deck erected . The curve has, of course, a vertical asymptote at 0%; a first minimum takes place at 40%: for this erection stage the flexural and torsional coupled have shapes characterized by an antinode at midspan; a second minimum is found at 65%: in this zone the flexural and torsional modes coupled are characterized by a node at midspan. In the intermediate zones, where the mode shape correspondence is lower, the critical speed is higher, but the variations are generally lower than those discussed for smaller bridges, despite of the frequencies closeness, due to the lesser influence of the deck inertial properties with respect to those of the suspension cables. The critical speed at deck completion was found to be 130 m/sec.

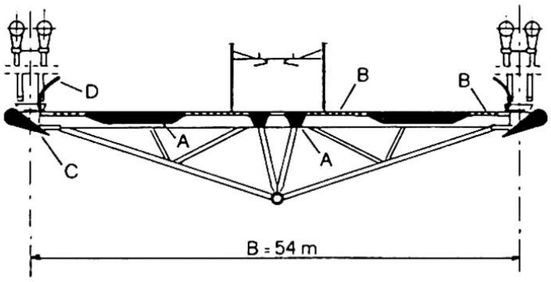


Fig. 6 - Deck cross section

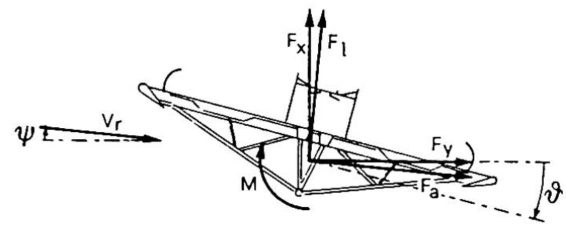


Fig. 7 - Aerodynamic forces

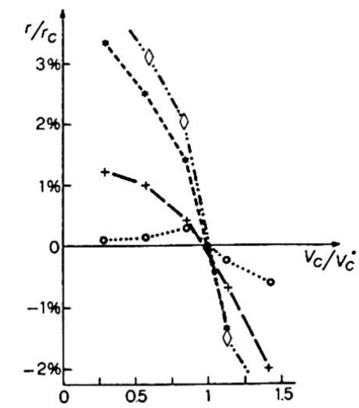


Fig. 8 - Critical speed analysis, different methods

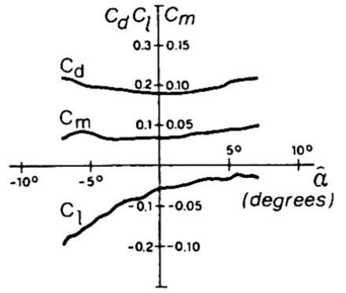


Fig. 9 - Aerodynamic coeff. with open grids

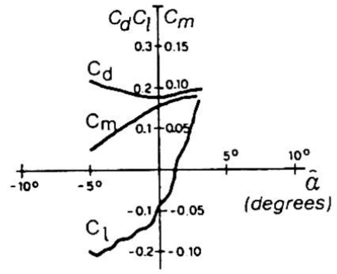


Fig. 10 - Aerodynamic coeff. with closed grid

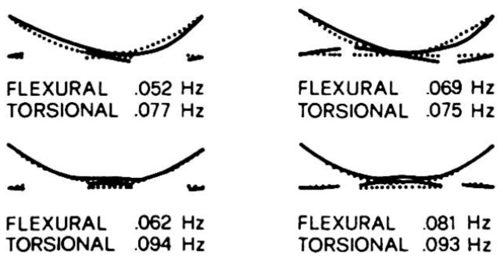


Fig. 11 - Mode shapes for deck erected at a) 30%, b) 80%

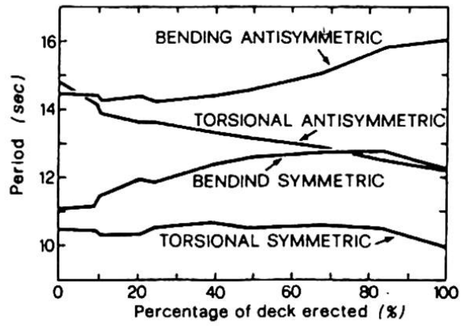


Fig. 12 - Natural frequencies evolution during the erection

4. CONCLUSION

Several aspects connected to suspension bridge structural behaviour in erection have been discussed. Considerable importance has been shown to have dynamic instability phenomena due to wind action, for the prevention of which it is believed to have a paramount influence a sound aerodynamic design of the bridge deck.

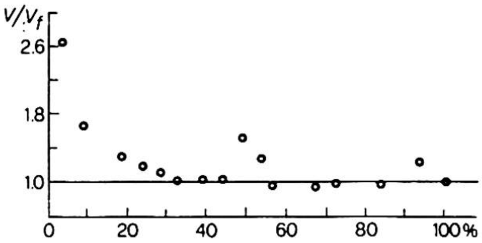


Fig. 13 - Trend of critical speed evolution during the erection

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