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Deck Deformability in a Long Span Suspension Bridge

Déformation admissible du tablier d'un grand pont suspendu

Trägerverformungen bei einer grossen Hängebrücke

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SUMMARY

A 3300 meter, single span suspension bridge has been designed for a stable crossing of the Messina Straits, for both highway and railway use. The reliable running of trains requires stringent limits to the deck deformability, in particular in the transition zone between the suspended and the supported deck. For the same diameter of the main cables, and the same general structural arrangement, two alternative solutions to reduce the deck deformability are discussed; additional cable stays to reduce the vertical displacement in the vicinities of the towers, during the train entrance; and inclined suspenders to increase to longitudinal stiffness and the vertical stiffness with regard to concentrated loads.

RÉSUMÉ

Un pont suspendu d'une longueur de 3300 m et d'une unique portée a été projeté sur le détroit de Messine à l'usage de l'autoroute et du chemin de fer. La viabilité du train pose des limites rigides à la déformabilité de la charpente. Pour le même diamètre des câbles principaux et le même arrangement de la charpente, on prend ici en considération deux solutions alternatives pour réduire la déformation admissible du tablier, des étais supplémentaires pour réduire les déplacements verticaux à proximité des tours et des suspentes inclinées pour augmenter la rigidité longitudinale et verticale pour des poids concentrés.

ZUSAMMENFASSUNG

Es wurde eine Hängebrücke mit einer Spannweite von 3.300 m als Ueberquerung der Meerenge von Messina sowohl für den Strassen – als auch für den Eisenbahnverkehr entworfen. Die Befahrbarkeit mit der Eisenbahn erfordert strenge Grenzwerte für die Verformbarkeit des Brückenkörpers, vor allem in der Uebergangszone zwischen dem aufgehängten und dem gestützten Teil. Bei gleichem Durchmesser des Hauptkabels und bei gleicher allgemeiner Struktur werden zwei alternative Lösungen zur Reduzierung der Verformbarkeit des Brückenkörpers diskutiert: zusätzliche Kabelstage in der Nähe der Tragpfeiler zur Reduzierung der vertikalen Versetzung bei der Zugpassage, schräge Tragkabel zur Erhöhung der Längs- und Vertikalsteifigkeit in Bezug auf konzentrierte Belastung.

1. INTRODUCTION

The suspension bridge under consideration is characterized by the following figures (fig.1):

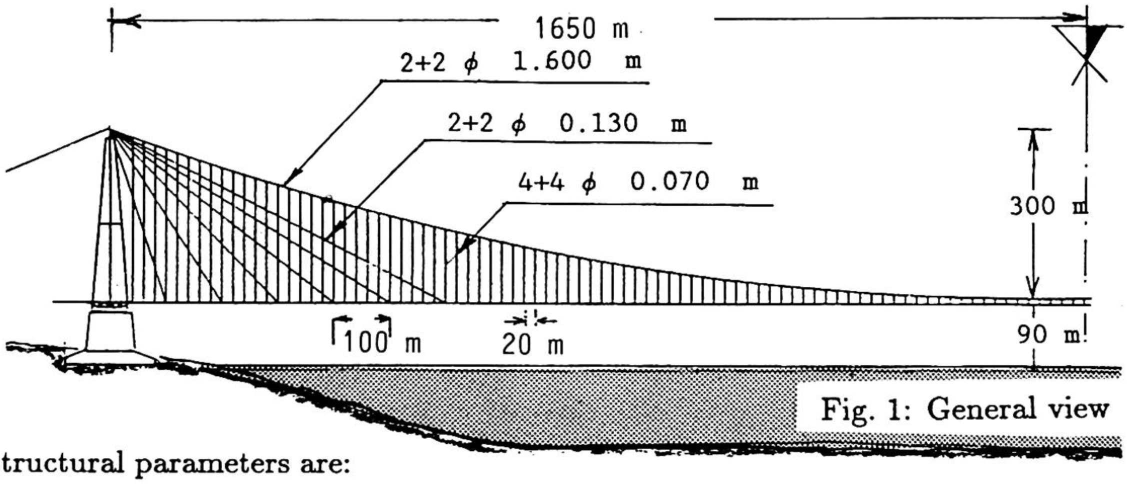

Fig. 1: General view

The main structural parameters are:

Cables:	diameter	1.60 m	(n.4)
	length	990+3370+990 m	
	weight	50.5 t/m	
Deck:	cross section area	1.354 m^2	
	moment of inertia	8.835 m^4	
	dead load	42.70 t/m	
Hangers:	diameter	d = 0.070 m	(n.8 at 20 m)
Cable stays:	diameter	d = 0.130	
Towers:	cross section area	8.64 m^2	
	moment of inertia	187.7 m^4	
	height above mean high water	400 m	
	height over the concrete basis	310 m	
	sag/span ratio = 1/11, height of the towers 400 m (3+3) lateral highway traffic lanes + (1+1) emergency lanes (1+1) central railway tracks + (1+1) service lanes		

Table I

At both ends of the deck damped bumpers have been designed to reduce the longitudinal movements to ±.50m that is the displacement due to the possible deck elongation under the yearl thermal excursions. In absence of bumpers, unsimmetrical live loads cause movements of the decl with a given proportion of longitudinal tension and compression, resulting in a null net elongatio over the entire length. This holds, as a first approximation, also in presence of damped bumper Therefore the above gap amplitude, in any live load combination, is available for the therma elongation.

In any particular thermal condition, the longitudinal movement of the deck due to live load not covered by the remaining gap amplitude, is absorbed by tension and compression of the decl itself. The thermal and live load being uncoupled, the alternative configurations of the suspensio system here considered are to be analysed in both conditions of ends free or one end longitudinall supported.

The two central railway tracks are stiffened by a longitudinal lattice beam, of 10 m height, t guarantee the local runnability requirement. A transition zone at the ends of the bridge is designe to comply with the train runnability specifications.

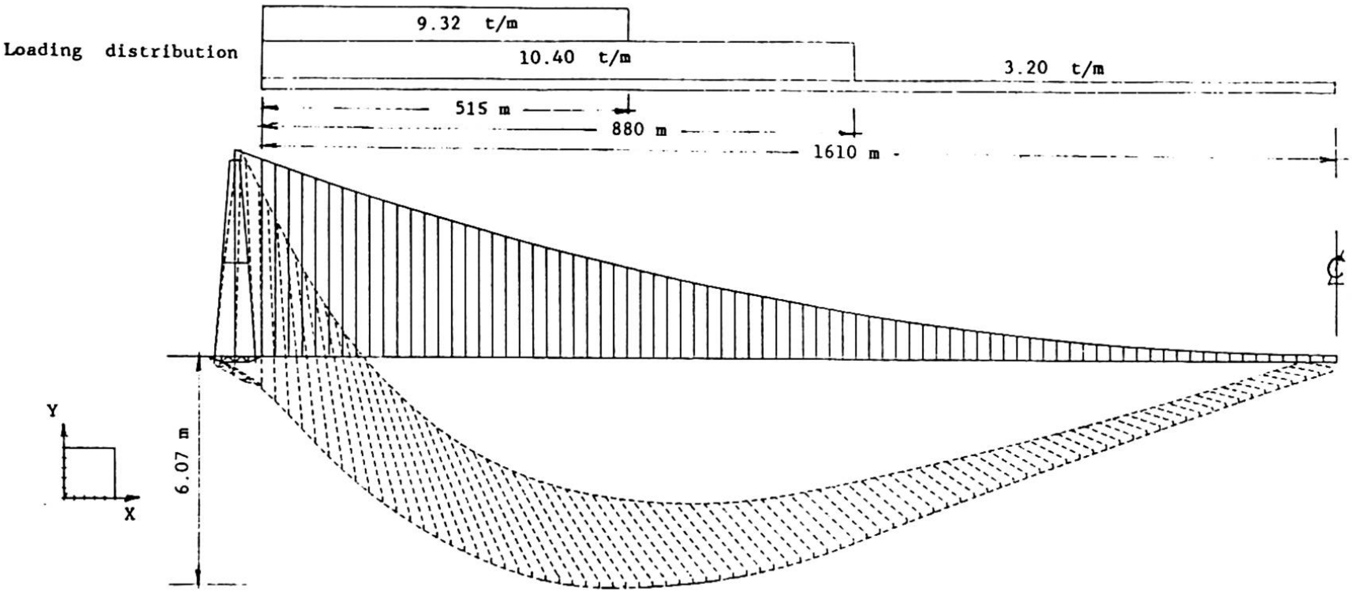

Fig. 2: deck deformed shape without stays

The mathematical analyses have been performed with an ADINA (Automatic Dynamic Incremental Nonlinear Analysis) bidimensional finite element model taking into account geometric nonlinearity with finite displacements.

The towers and the deck have been modeled through beam elements; cables, hangers and fan cable stays have been modeled through truss elements. Hydraulic dashpots at the end of the deck have been simulated through gap elements, which allow longitudinal displacements of ±.50 m.

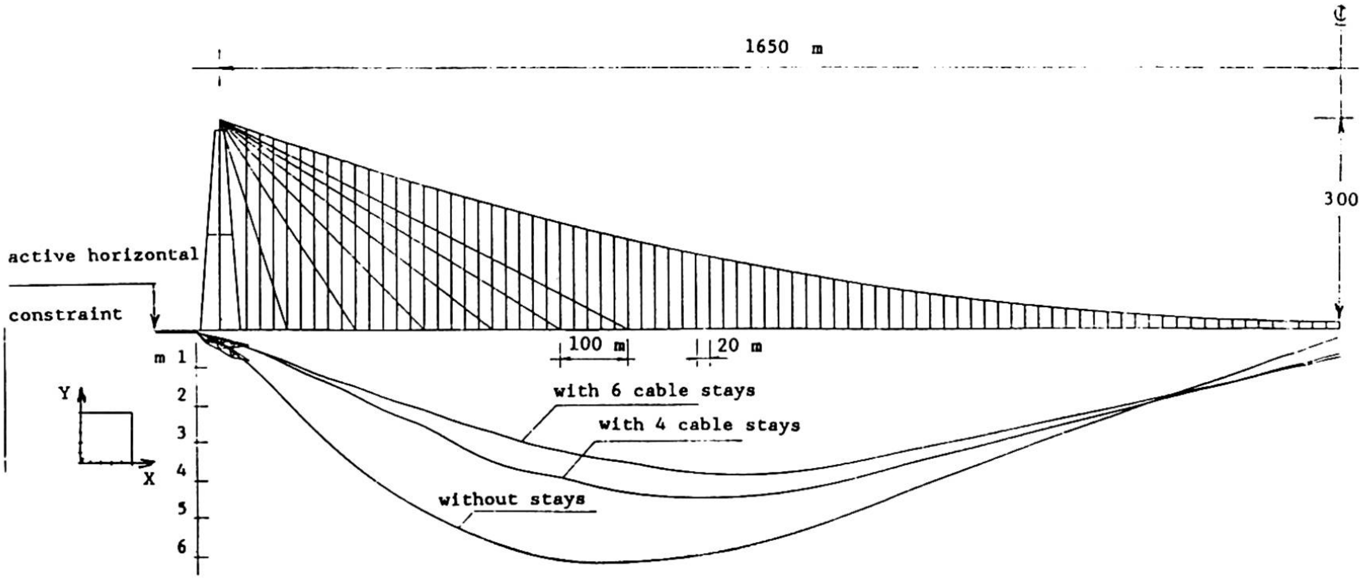

Fig. 3: deck deformed shape for different shapes

2. ADDITIONAL CABLES AT THE TOWERS

Combining the suspension and the cable stayed system has been proposed in several occasions in the design of long span crossings, since its first appearance at the Brooklyn Bridge.

As discussed in ref. (6), the advantage of additional cable stays may be directed towards minimal deck slope - to cope with train runnability requirements - or towards a more general objective, in particular the minimal material requirement. The results here shown refers to the first objective only.

Among the different combinations, the loading condition which governs the runnability is that in fig.2, where three kinds of loads are assumed: 1) a heavy train of 9.32 t/m, and 515 m length; 2) a heavy lorries of 10.4 t/m, and 880 m length; and 3) a car traffic of 3.2 t/m along the entire deck length. Results of the optimization process will be discussed with reference to this loading distribution.

The corresponding diagrams for displacement are shown in fig.2 and 3. Fig.2 shows the bridge deformed shape without stays, when the horizontal constraint is active at the left pier. For the same abscissa, the displacement of the cables is different from that of the deck. The resulting inclination of the hangers provides thus an axial force in the deck.

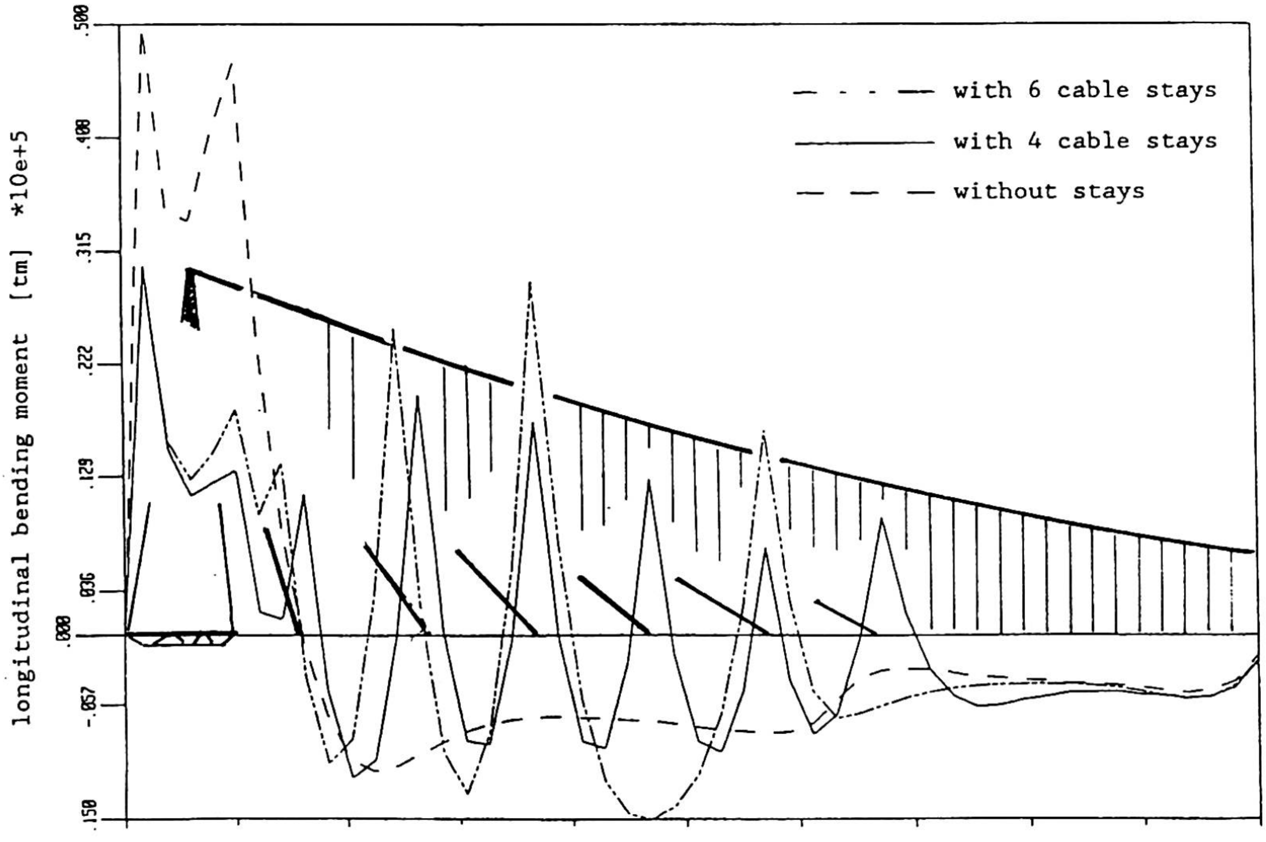

Fig. 4: deck bending moment for different configurations

The effect is enhanced by the cable stays presence which plays a different role according to whether left or right horizontal constraint is active. In general, the middle portion of the deck is in tension, both for the prestressing action provided by the tendons and for the live load transfer. The end portion of the deck, near that constraint which is in force, is in compression. The higher axial forces are in this portion, and therefore are of the compression type, (see table 2). Vertical displacements and thus the deck slope are only marginally affected by the horizontal constraint.

longit. bond	significant values	without stays		with 6 cable stays	
		deck	cables	deck	cables
free	ΔX [m]	-1.12	-1.78	-2.10	-1.50
	ΔY [m]	-6.35	-6.23	-5.54	-5.51
	θ [‰]	19.72	-----	14.31	------
	Nmax [t]	-225.		-7108.	
	ΔN stays			1476.	
fixed at left edge	ΔX [m]	-0.13	-1.67	-0.13	-0.90
	ΔY [m]	-6.07	-5.96	-4.04	-3.93
	θ [‰]	19.03	-----	9.52	------
	Nmax [t]	3465.		11208.	
	ΔN stays			2219.	
fixed at right edge	ΔX [m]	-0.16	-1.67	-0.73	-1.03
	ΔY [m]	-6.08	-5.96	-4.46	-4.28
	θ [‰]	19.05	-----	10.87	------
	Nmax [t]	-3408.		-10211.	
	ΔN stays			1917.	
with gap +/-0.50m	ΔX [m]	-0.50	-1.72	-0.50	-1.03
	ΔY [m]	-6.19	-6.07	-4.35	-4.27
	θ [‰]	19.31	-----	10.63	------
	Nmax [t]	1127.		8600.	
	ΔN stays			2028.	

Table 2: the four models significant values

The way the hydrolic dashpots are devised, the end constraint condition may be either fixed or sliding, depending on temperature and the involved horizontal displacement. Thus, in practice, any one of the three conditions shown in table 2 is to be accounted for. The introduction of cable stays results therefore in a sensible increase in the axial force in the deck, mainly of a compression type. For a deeper understanding of the problem, however, the change of the bending moment diagram is to be accounted too, contemporary to the axial force. See, for instances, fig.4.

3. INCLINED HANGERS

Inclined hangers provide a limited increase in the deck stiffness, at the expenses of an additional longitudinal displacement. When this is prevented by the end bumpers, an additional normal stress arises in the deck. Table 3 shows the main terms of the comparison between alternative solutions, under the same loading combination previously considered. Here configuration 1 refers to inclined hangers all along the suspended span, and configuration 2 refers to inclined hangers along a central portion of 240 m length. Notice that, among bridge designers, inclined hangers are reported to offer increased damping with reference to vertical hangers, under longitudinal and vertical dynamic excitation. Apparently this assesment is based on a single measure given by reduced scale physical model. In this regards, the present authors believe that, unless similar results are got in a full scale structure, one cannot rely on then. Besides the additional damping could not be other than a measure of internal friction or of alternate bending stresses in the hangers related to the peculiar design of the connection. Some possible disadvantage in terms of fatigue have been reported in the literature.

Table 3. Performances of inclined hangers solutions

longit. boundary		Inclined hangers Configuration 1	 Configuration 2
free	ΔX (m)	1.34	1.26
	ΔY (m)	6.38	6.54
	N_{max} (t)	5500	880
fixed at left edge	ΔX	0.76	0.71
	ΔY	5.44	5.52
	N_{max}	17600	13200
fixed at right edge	X	0.92	0.76
	Y	6.14	6.18
	N_{max}	16000	13000

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