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Railway Traffic on Long Span Suspension Bridges

Circulation ferroviaire sur les ponts suspendus de grande portée

Eisenbahnverkehr auf weitgespannten Hängebrücken

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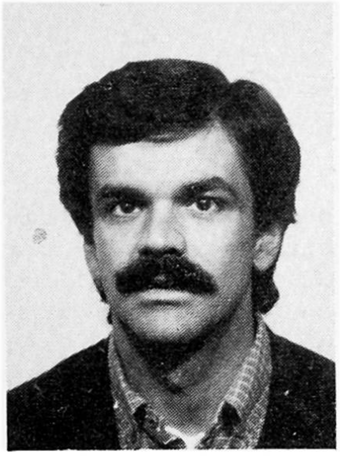

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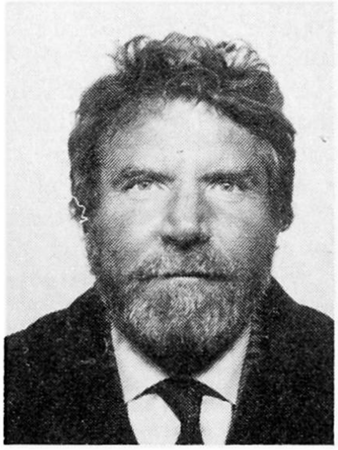

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SUMMARY
General deformation aspects of suspension bridge response are discussed, proceeding subsequently to the examination of static runnability problems deriving from the elevate slopes which can occur for severe loading. The second part of the paper is devoted to dynamic runnability analyses where, beside the structural behaviour, structure-vehicle interaction and vehicle response are simulated, providing indications on transit comfort and safety.

RÉSUMÉ
Les aspects généraux de la déformation des ponts suspendus sont traités ainsi que les problèmes de circulation découlant de la pente élevée consécutive aux charges maximales. Des problèmes de circulation dynamique sont considérés en simulant le comportament de la structure, l'interaction entre la structure et le véhicule et la réponse du véhicule, en donnant des indications sur le confort et sur la sécurité.

ZUSAMMENFASSUNG
Dieser Beitrag behandelt die gesamten Aspekte der Verformung von Haengebruecken, mit Hilfe von Untersuchungen bezueglich der Probleme des statischen Verhaltens bei hohen Neigungen, die von Schwerbelastungen herstammen. Der zweite Teil behandelt Untersuchungen des dynamischen Verkehrs, das Verhalten der Struktur, die Wechselwirkung zwischen der Struktur und dem Fahrzeug. Das Verhalten des Fahrzeuges simulierend, können Angaben bezueglich der Sicherheit und des Komforts des Verkehrs gegeben werden.

1. INTRODUCTION

The debate on the possible performances of large span suspension bridges for railway use is active in the bridge engineering community since several decades [1,2,3,4]. A milestone in the argument has now been set in '88, with the completion of the Kojima-Sakaide route of the Honshu-Shikoku Bridge project, comprising three suspension bridges open to full rail traffic. Not less interesting when progressing towards very large spans the theoretical and technical developments achieved during the feasibility analyses for other large crossings, such as the Great Belt and the Messina Straits.
The present paper, written in the context of the studies carried out for the second, presents nevertheless a full account of related topics for the entire range in which suspension bridge structures can be of interest for very large crossings.
General deformation aspects of their response are discussed first, proceeding subsequently to the exam of static runnability problems deriving from the elevate slopes which can occur for severe loading. Spans from one to three thousand meters are considered; effects of rail loads, road loads and temperature are shown.
A second part of the paper is devoted to dynamic runnability analyses where, beside the structural behaviour, structure-vehicle interaction and vehicle response are simulated. Indications regarding transit comfort and safety are given: railway loads running in different conditions on the bridge deck are considered, with the simultaneous presence of severe environmental actions, namely earthquakes and wind.
Brief attention is also given to relevant numerical procedures adopted in the different sections.

2. STATIC RESPONSE

The interest in the analysis of suspension bridges deformation aspects goes beyond a pure information on their displacement behaviour: it is known in fact that, in certain conditions, elevate displacements can imply slopes of the rail axis not compatible with operation. Besides, other significant deformation parameters, such as deck ends rotation, are of paramount influence on the train-bridge system dynamic response. A general discussion of the related topics cannot be restricted in the space of the present paper and can be found in [1]. Here a number of synthetic results are reported: fig.s 1 and 2 show the maximum and minimum vertical displacements of a suspension bridge deck versus its centre span length, in the hypotheses of anchoring side spans (no suspended deck in the side spans) and of 1/11 sag/span ratio. The loads considered, which are not intended to be design code conditions for safety or service, but indications for an understanding of the orders of magnitude involved in the behaviour of the structure, are:

i) uniform 1 t/m load on half centre span
ii) uniform load on half centre span, with a total of 1500 t (coincident with i) for a 3000 m span
iii) one heavy train, with 300 m length and 1200 t total weight
iv) temperature variations of the cable and deck (dependent on cable diameter, with a reference value of 20 °C)

The enormously varying influence of different loads for different spans is evident: over 1500-2000 m the temperature prevails among those considered, as the increase of cable geometric stiffness consequent to the increased axial force in the same decreases the others. As the slopes due to temperature are obviously constant for equal temperature variations, it can hence be stated that progressing towards very large spans railway slopes must become close to an asymptotic value, but for a minor decrease due to the smaller average temperatures of larger diameter cables.
The above conclusion is demonstrated in the graphs of fig.s 3 and 4, showing the maximum railway slopes at the reference and maximum temperatures for different train lengths, sag/span (f/l) ratios and centre span lengths.

+ 300 m length, 1200 t weight train; Δ live load, distribution i);
◊ live load, distribution ii); □ temperature

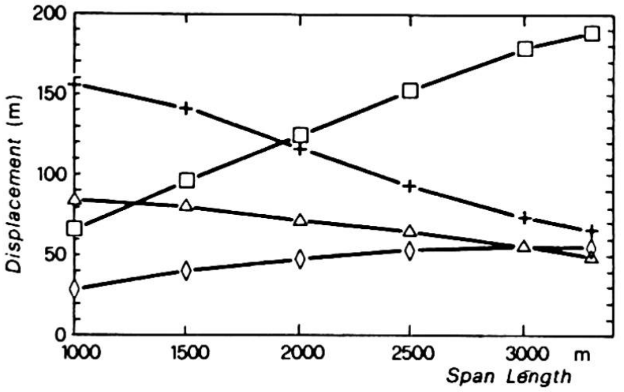

Fig. 1 - Maximum vertical displacements versus centre span length

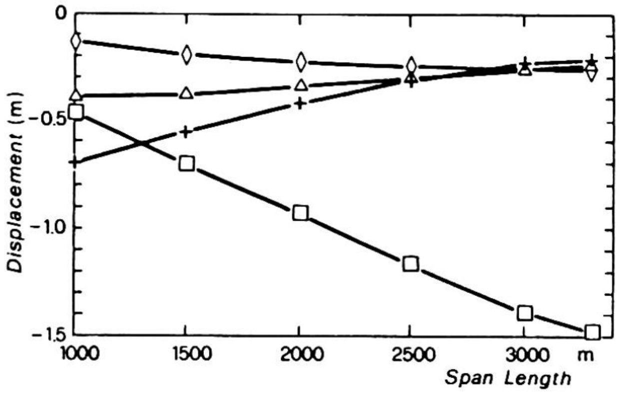

Fig. 2 - Minimum vertical displacements versus centre span length

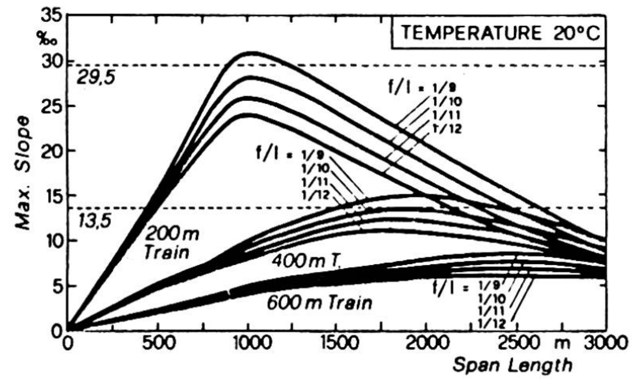

Fig. 3 - Maximum slopes versus centre span length, reference temperature

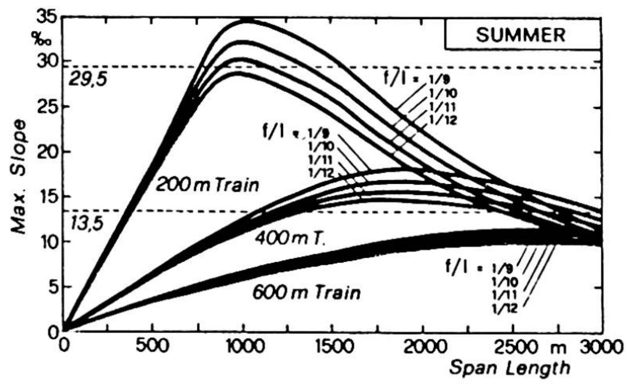

Fig. 4 - Maximum slopes versus centre span length, maximum temperature

The detailed load combinations adopted, which are those assumed for the analysis of service conditions in the Messina Straits crossing project feasibility, are discussed in detail in [1]. Two aspects must be stressed when examining the above results: first, the convergence of all the curves towards a common slope range, which confirms the conclusions drawn before even if the asymptotic values have certainly not been reached; second, and more important, the natural adequateness of the pure suspension scheme to railway needs as far as slopes are concerned for spans over 1500-2000 m, i.e. when temperatures effects become important. The admissible slopes go in fact from 1.3 to 1.5% and an opportune initial deck configuration can provide the necessary correction. It must also be noted that the corrections cannot be too large, as they could cause excessive slopes for minimum temperature conditions.

3. DYNAMIC RESPONSE AND VEHICLE-STRUCTURE INTERACTION

The dynamic response analyses have been restricted to the runnability of the 3300 m one span suspension bridge proposed within the Messina Straits crossing feasibility studied [5], see also [9,10]. The runnability problems derive from the bridge motion caused by wind, traffic and seismic actions: important components of rail generalized deformations are [11]:

a) maximum slopes in the vertical plane;
b) maximum transverse roll angle, due to deck twisting;
c) rail deviation (cusps) in the horizontal plane in correspondence to the bridge-viaduct joint, when the bridge is laterally deflected;

d) minimum curvature radius of the rails in the horizontal plane.
Dynamic phenomena which warrant attention are:
e) possible resonance between the train natural frequencies and the transit frequency associated to the modulus of the deck girder supporting the rails, with subsequent dynamic amplification;
f) high frequency vibrations and noise induced by train transit.

A mathematical model of the train-bridge system, capable of simulating the behaviour of a railcar, was developed [7,8] and tuned through comparisons with the results obtained during a test campaign carried out on an existing steel truss railway bridge in Italy. The research was then extended to the suspension bridge response, obtaining the information summarized in the following.

3.1 The global bridge-train mathematical model

The procedure proposed is based on the separate but simultaneous direct integration in time of the bridge and train equations of motion, accounting for the compatibility conditions of the two systems in terms of displacements at the contact points and for the mutual contact forces. Within a displacement finite element approach to the bridge structure modeling [6,9,10] the associated equations of motion can be set as:

$$\underline{M}_p \, \underline{\ddot{X}}_p + \underline{R}_p \, \underline{\dot{X}}_p + \underline{K}_p \, \underline{X}_p = \underline{F}_{pc} + \underline{F}_{pe} \tag{1}$$

where $\underline{M}_p$, $\underline{R}_p$ and $\underline{K}_p$ are respectively mass, damping and stiffness matrices, while $\underline{F}_{pc}$ are the external generalized contact forces due to interaction and $\underline{F}_{pe}$ are other environmental actions, such as wind and seismic events.

Each railcar is treated as composed by a set of rigid bodies connected through springs and dashpots to form the 23 d.o.f. system shown in Fig. 5. The corresponding non linear equations of motion are derived, as shown in [7], in the form:

$$\underline{m}_j \, \underline{\ddot{Z}}_j + \underline{r}_j \, \underline{\dot{Z}}_j + \underline{k}_j \, \underline{Z}_j = \underline{F}_{vj} + \underline{F}_{yj} + \underline{F}_{zj} \tag{2}$$

At the left-hand side of eq.s (2) are located the linear inertia, damping and stiffness terms; $\underline{F}_{vj}$ represents the external forces directly applied to the railcar (i.e. wind). $\underline{F}_{yj}$ contains the generalized forces due to track displacements; $\underline{F}_{zj}$ contains non linear terms and the generalized wheel-rail contact forces, dependent on train $\underline{Z}_j$ and bridge $\underline{X}_p$ displacements. Equations (1) and (2) are coupled, as the contact forces $\underline{F}_{pc}$, $\underline{F}_{yj}$ and $\underline{F}_{zj}$ are function both of $\underline{X}_p$ and $\underline{Z}_j$ variables. The direct time integration of the equations of motion is performed via a modified Newmark algorithm [8], with an iterative implementation whose convergence is controlled by the wheel-rail contact forces balance.

3.2 Simulation of railway-bridge system behaviour

The train transit on the suspension bridge was simulated, see [9,10] for further details, via a computer code based on the theory described. Beside the railway loads, simultaneous presence of wind or earthquake was accounted for.

3.2.1 Wind effects

Transverse turbulent wind acting on the bridge (average speed U = 32 m/s, turbulence index I = 0.17) was simulated: vertical forces due to the most unfavourable distribution of moving loads for the central span were also considered. A sensitivity investigation for variable forward speed V of the train was carried out, so as to evaluate the safety and comfort coefficients in different conditions. In Fig. 6 the time history of the overturning coefficient of a wheelset for the vehicle entering the bridge at V = 130 Km/h (i.e. in correspondence to an expansion joint) is shown: the C_{ovt} coefficient maximum value is well below the safety threshold (C_{ovt} = 60%). The time history of the C_d derailment coefficient, evaluated on the left and right wheel of the same wheelset is shown in Fig. 7: also in this case the C_d coefficients are considerably lower than the limit value (C_d = 120%).

The accelerations of the carbody for the train entering the bridge versus V are shown in Fig. 8. Fig. 9 shows the shock (time derivative of the acceleration) trend on the carbody, while Fig. 10 shows the overturning and derailment coefficients: for any speed C_{ovt} is lower than 40%, while C_d does not exceed the value of 85%. Resonance effects on the train due to the modularity of the bridge deck structure were found to be negligible. Local effects, even on the wheelsets, proved to be modest. Railway runnability conditions are always acceptable, particularly along the span.

3.2.2 Seismic effects

Earthquake effects on a train, running either on ground or on the bridge at V = 60 km/h, have been simulated [9,10]. Seismic conditions are specified by means of acceleration, velocity and displacement time histories: max. ground acceleration of the event considered was 0.64 g. These values are assumed to describe the rail motion on ground: the rail motion on the deck is computed by imposing opportune time histories to one bridge tower foundation and to the relative anchor block. For the train running on ground, Fig.11 reports the time histories of C_{ovt} and C_d. In Fig. 12 the same quantities are shown for the train running over the bridge. It is apparent how earthquake effects are strongly attenuated over the bridge, which behaves like a low-pass mechanical filter.

4. CONCLUSIONS

The following main conclusions can be drawn:

- very large pure suspension bridge schemes become naturally adequate for rail_ way service, due to their elevate geometric stiffness: this stand both for the dynamic and the static response
- as to the specific case of the 3300 m single span bridge proposed within the feasibility studies for the Messina Straits Crossing project, it has been set into evidence as the transit parameters are always favourable during severe wind action, while earthquake effects, which are filtered by the structure, are far more dangerous on ground than on the bridge deck.

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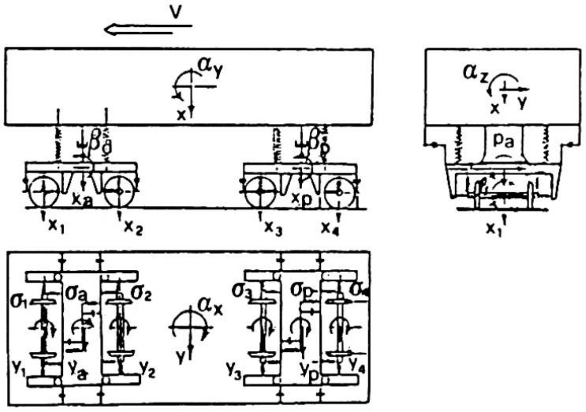

Fig. 5 - Railway car model

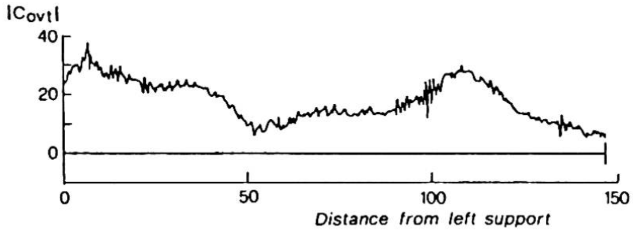

Fig. 6 - C_{ovt} coeff. on a wheelset for the train entering the bridge

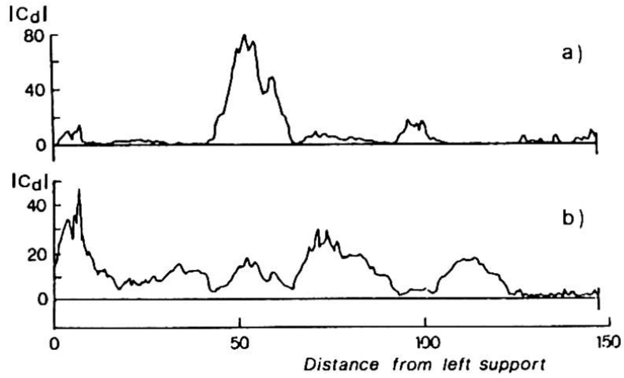

Fig. 7 - C_d, train entering the bridge: a) right wheel b) left wheel

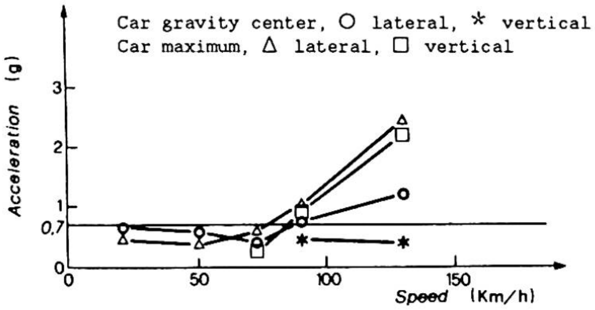

Fig. 8 - Max. lateral carbody accel.

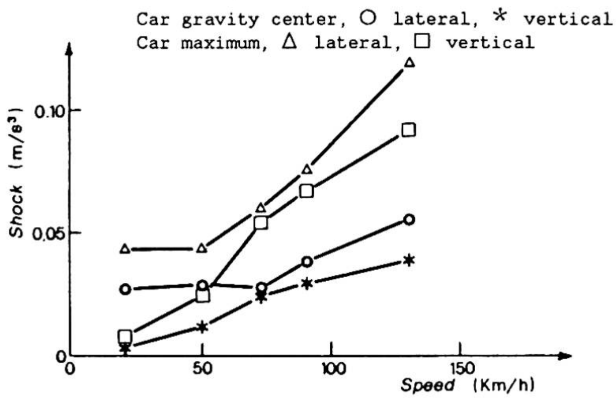

Fig. 9 - Maximum carbody shock

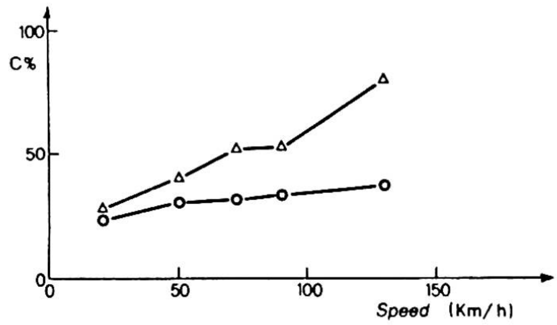

Fig. 10 - C_{ovt} (O) and C_d (Δ) maximum values versus train speed

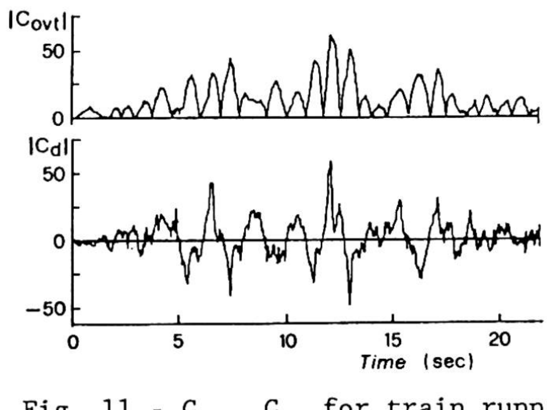

Fig. 11 - C_{ovt}, C_d for train running on ground during a seismic event

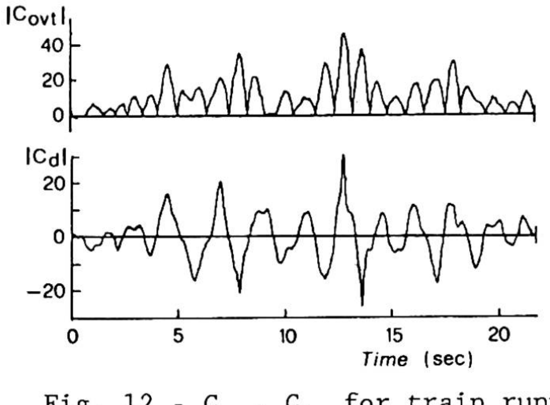

Fig. 12 - C_{ovt}, C_d for train running on the bridge during a seismic event