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Experimental analysis of aerodynamic stability of stress-ribbon footbridges

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Abstract. The dynamic properties of one-span or multi-span reinforced concrete footbridges of catenary form (see e.g., Fig. 1) include the very low fundamental natural frequency, usually near the step-frequency of pedestrians, and the low damping of bending vibrations. The paper sumarizes the results of model as well as full-scale measurements with particular reference to the influence of torsional rigidity of the stress-ribbon on the magnitude of aerodynamic response, the results of measurements on footbridges of catenary form being completed by results obtained on footbridges of some other types. Additionally the influence of the local broadening of the bridge deck on the bridge response was tested. Starting from these results the criterion has been derived for the decision, whether the flutter analysis is necessary for the design of the footbridge.

Key words: footbridges; aerodynamic stability; bending-torsional vibrations; wind-excited vibrations; wind-tunnel in civil engineering.

1. Introduction

The first footbridge consisting of the prestressed concrete stress-ribbon stretched in catenary form, spanning 63 m, was erected across the river Svratka in Brno (Czech Republic) in 1979 (Stráský 1986) and since that time many footbridges of the same type have been built. Another type of these structures has the bridge deck suspended from two load-bearing ropes stretched between the towers on either end of the bridge (see Fig. 2a, 2b). In one of the designs by Prof. J. Stráský, the same bridge deck is supported by a pair of load bearing arches (see Fig. 3).

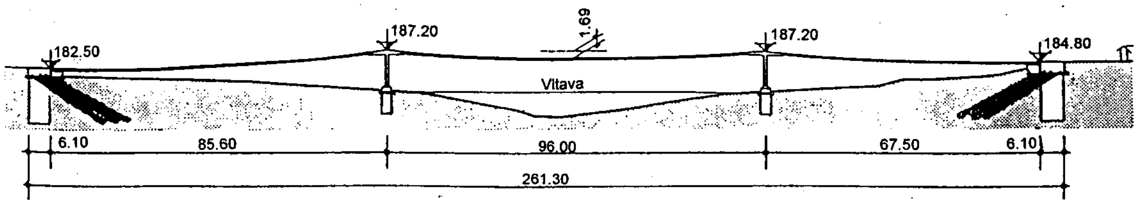

Fig. 1 A three-span stress-ribbon footbridge of catenary form (dimensions in [m])

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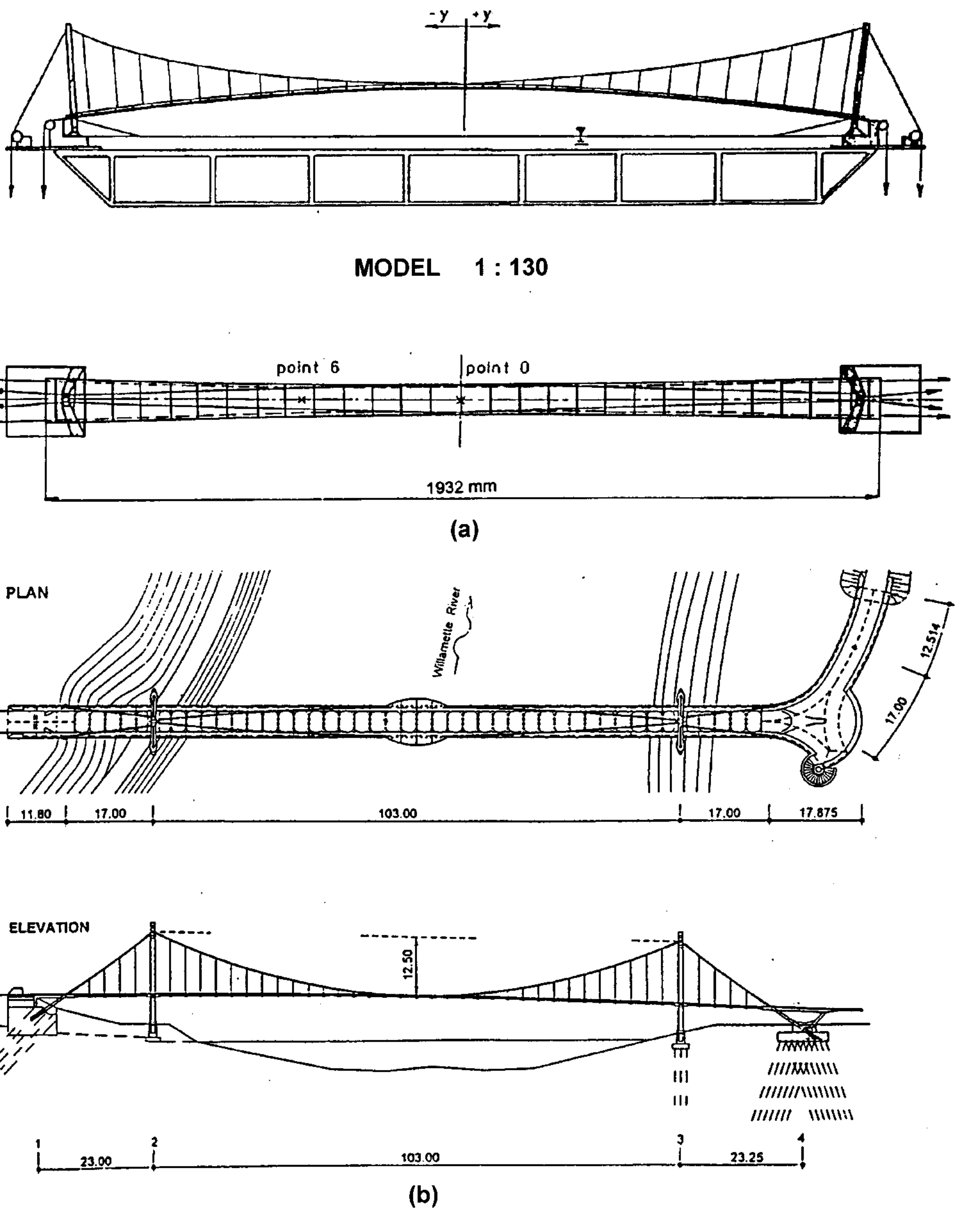

Fig. 2 (a) A footbridge suspended on 2 cables with the bridge-deck prestressed by a pair of tendons, span 252 m. Aeroelastic model, scale 1 : 130, (b) A suspended footbridge-prototype structure, dimensions in [m]

The construction of the above mentioned first footbridge initiated also the theoretical, as well as experimental research of this type of structures, an important part of which was the analysis of their behaviour under dynamic loads, which can be produced not only by various modes of pedestrian movement, but also by wind, and at present - unfortunately - also by vandalism. After disquietening experience with classic suspension bridges, which collapsed

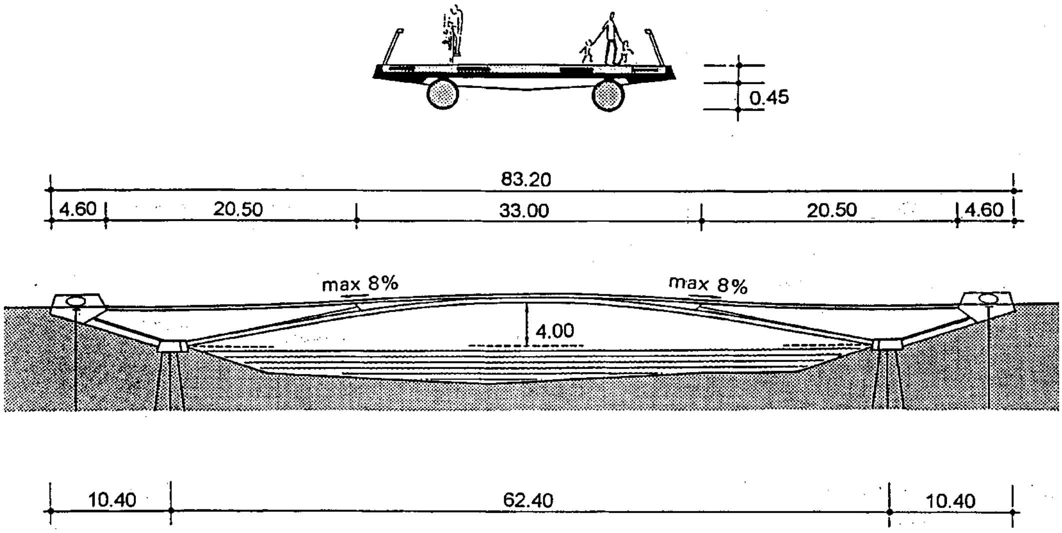

Fig. 3 A footbridge supported by a pair of arches-design, dimensions in [m]

due to wind-excited vibrations in the number of cases since the very beginning of their use in the first third of the 19th century (see e.g., Stamm 1952), it was natural to investigate this new type of footbridges also with reference to its safety against aeroelastic phenomena.

2. Dynamic characteristics of footbridges

Observations of the passengers as well as the in situ measurements have revealed that the lightweight footbridges, particularly in the form of a freely suspended catenary, can be very sensitive to all dynamic loads because their natural frequency, similarly as vibration damping, is very low. The Fig. 4 shows the relation of the footbridge lowest natural frequency to its span, as ascertained by in-situ measurements in the Czech Republic (Stráský 1986). Typical damping values of such footbridges are given in Fig. 5 in terms of the logarithmic decrement plotted against the double-amplitude of displacement of the footbridge shown in Fig. 1. This

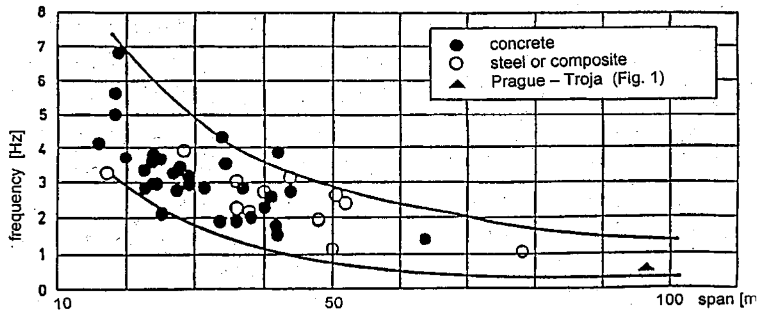

Fig. 4 Measured natural frequencies f of some footbridges vs. span length l. Assessed limiting values : upper curve $f = \frac{112}{l^{0.925}}$; lower curve $f = \frac{217}{l^{1.431}}$

footbridge was subjected to a dynamic test on its completion in 1984 and again 13 years later, in 1997.

3. Wind effects on footbridges

Wind effects on footbridges are assessed with reference to their safety and serviceability. The magnitude of dynamic response depends on the dynamic flexibility and on the character of the air-flow around the cross-section, influencing the vortex shedding behind the bridge deck. The danger of the loss of aerodynamic stability depends on the well-known physical parameters, which are recapitulated in chapter 4.

3.1. Experimental investigation

In the course of a few recent years, 4 types of prestressed footbridges were investigated in the wind tunnels with modelled atmospheric boundary layer in the Aeronautical Research and Test Institute in Prague (VZLU): Stress-ribbon in catenary form (Fig. 1), stress-ribbon supported at its midspan by two parallel arches (Fig. 3), stress-ribbon suspended on a pair of load-bearing ropes (Fig. 2b) and stress-ribbon suspended on a pair of ropes and prestressed by the pair of tendons (Fig. 2a).

In the laboratory of the Institute of Theoretical and Applied Mechanics (ITAM) and in the new boundary layer wind tunnel in VZLU (test section width/high/length 1.78/1.45/2.0 m, max. velocity 35 m/s) the aerodynamic response of an aeroelastic model of a footbridge of the simpliest type (see Fig. 6) was investigated for four configurations. Its ribbon was in all cases equal, 1121 mm long and 75 mm wide. In the first case the ribbon was continuous, of a certain, though small, rigidity in bending and torsion, as it corresponds to the prototype footbridge, the deck of which consists of reinforced concrete segments, additionally tied together by prestressing. In the three other models the bridge-deck continuity was abolished so that the prestressing of the deck was slackened. These last three models differed from each other in torsional rigidity, i.e., in the ratio of natural frequencies of torsional and vertical

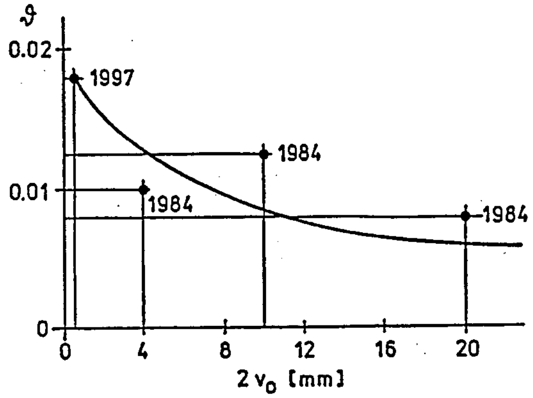

Fig. 5 Damping (logarithmic decrement) of the footbridge according to Fig. 1 vs. double-amplitude of displacement in the middle of the central span

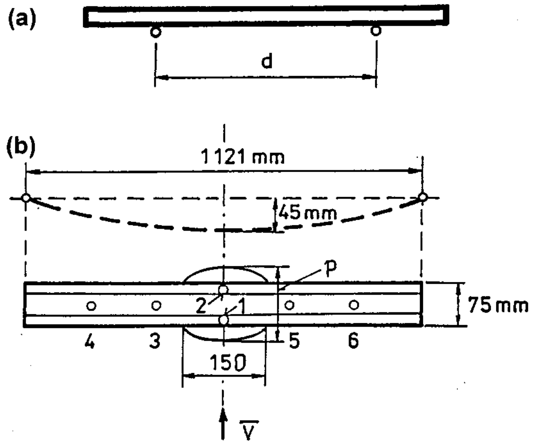

Fig. 6 (a) The cross-section of the model: a couple of cabels with variable distance *d* (see Fig. 7), bearing the ribbon assembled from rigid elements: width (of the bridge model)/thickness/length (in bridge-direction) 75/3/10 mm, (b) The position of accelerometers on the model, as used for the measurements according to the Fig. 7a, b. Broadening p=105; 145; 175 mm

vibrations. Thus the three models consisted of two free-hanging load-bearing cables, on which the rigid segments of 10 mm wide were placed, separated by a gap of cca 0.5 mm (Fig. 6). This arrangement has resulted in a relatively low damping both in bending (logarithmic decrement $\vartheta_B = 0.154$) and torsion ($\vartheta_T = 0.30$). The models differed by the distance of their load-bearing ropes (d = 10 mm, 46 mm and 68 mm), i.e., by their torsional rigididy and natural frequency in torsion. Figs. 7a,b show the relations between the model air flow velocity and the root mean square (RMS) of vertical deflections for the 3 modifications. Fig. 7b - right gives the same relation for the first modification, i.e., for the model with identical dynamic parameters, but with continuous bridge-deck. Fig. 7c shows again the same relations obtained for the model with two arches (Fig. 3), Fig. 7d for the model of suspended bridge (see Fig. 2b). These figures reveal:

(i) The vertical response of the structure decreases with the increasing ratio of natural torsional to bending frequencies.

(ii) Although the loss of stability (the bending-torsional flutter) did not occur, a slight, but distinct torsion appears at midspan, obviously due to vortex separation from the leeward deck-side.

Nowadays the architects of the footbridges often require to broaden the bridge deck near the middle of the span to form there a viewing platform, on which the passengers could stop for a while and admire the surroundings. Such a broadening need not be favourable from the point of view of aerodynamics of the structure. For this reason the authors examined the influence of such a platform on vertical response of the middle of the span of the footbridge. The model was similar to that described on Fig. 6, the broadening in the middle of the span of the bridge deck was scaled from the original 75mm to 105, 145 and 175 mm. Fig. 10 shows the RMS of vertical response of the point No 5 (see Fig. 6b) at the middle of the bridge span. It can be seen that the broadening does not influence the response in a substantial manner, the increase amounts to about 15%. No symptom of aerodynamic

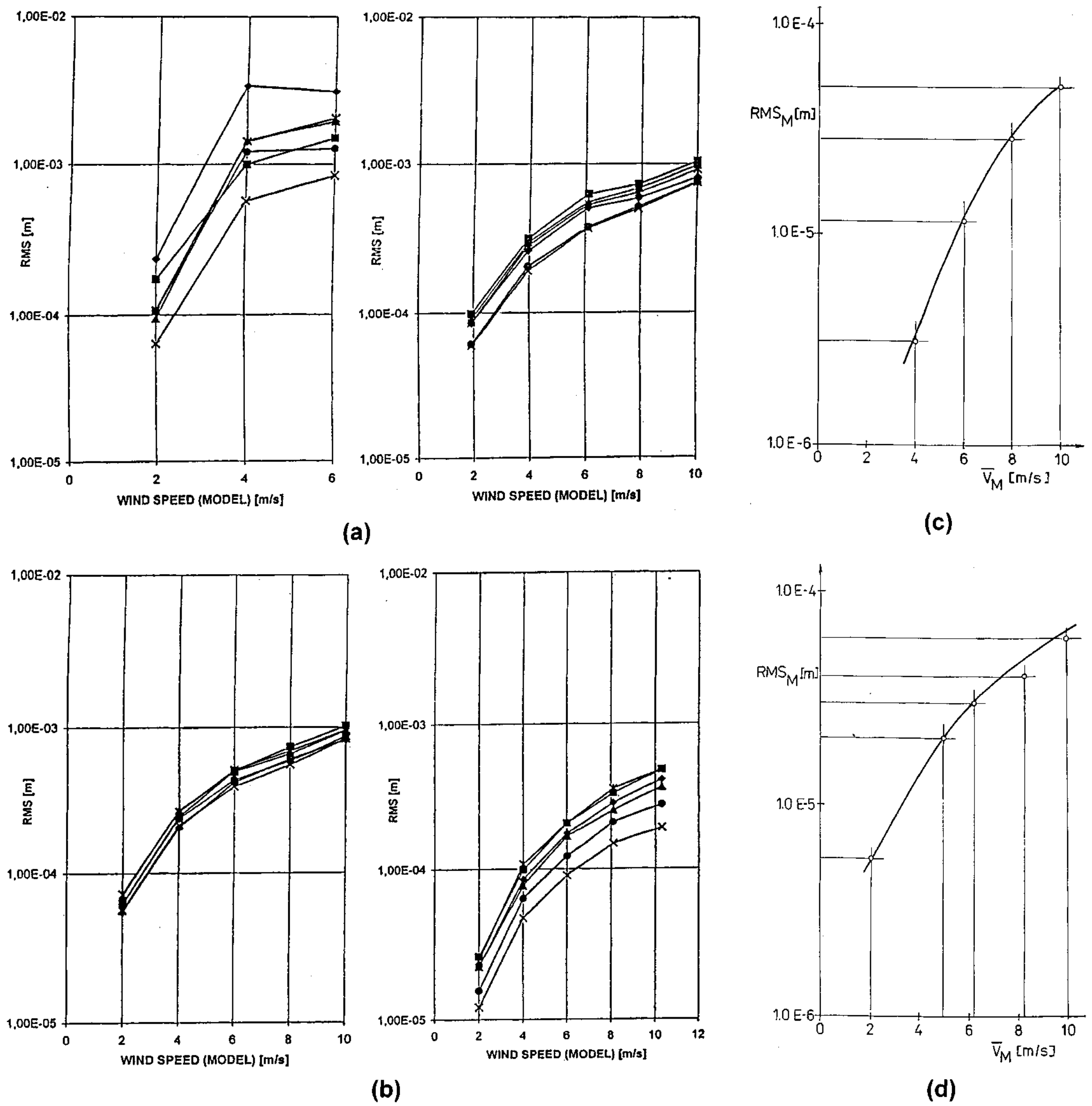

Fig. 7 (a) Relation of the model air-flow velocity and the root mean square value of vertical deflection in the middle of the ribbon (model according to Fig. 6):
left-d=10 mm, f_T=2.97 Hz, f_B=5.50 Hz, ω_T/ω_B=0.54
right-d=46 mm, f_T=13.00Hz, f_B=5.50 Hz, $\omega_T/\omega_B = 2.36$

(b) Relation of the model air-flow velocity and the root mean square value of vertical deflection in the middle of the ribbon (model according to Fig. 6):
left-d=68 mm, f_T=20.26 Hz,
f_B=5.50 Hz, ω_T/ω_B=3.68
right-continuous ribbon f_T=20.26 Hz

(c) Relation of the model air-flow velocity and the root mean square value of vertical deflection in the middle of the ribbon: model of the footbridge according to Fig. 3

(d) Relation of the model air-flow velocity and the root mean square value of vertical deflection in the middle of the ribbon: model of the footbridge according to Fig. 2b

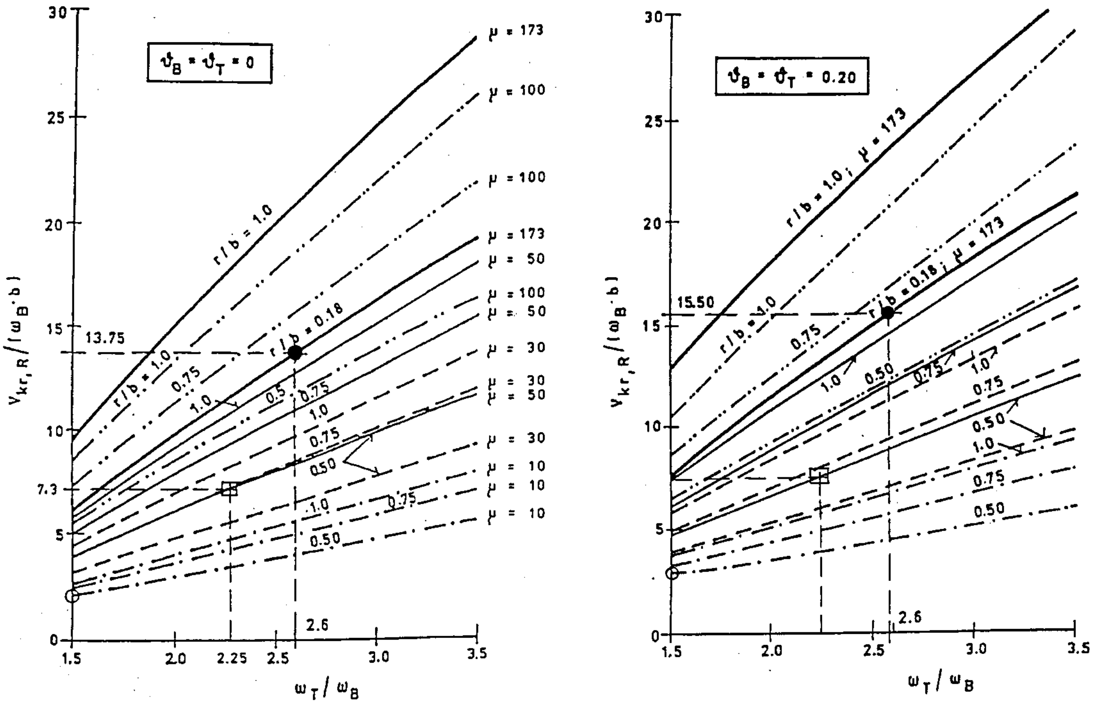

Fig. 8 Critical flutter onset wind-speed, after (Klöppel 1962) for zero (left diagram) and 0.20 (right diagram) damping. ω_T, ω_B-circular frequency in torsion/bending, ϑ_T, ϑ_B-logarithmic decrement of damping in torsion/bending, $\mu = \frac{m}{\pi \rho b^2}$, $r = \frac{1}{b}\sqrt{\frac{\Theta}{m}}$, m-mass of the bridge per unit length, Θ- mass moment of inertia per unit length, b-half of the bridge-width, ρ- mass of the air
□ -footbridge, Fig. 3, ○ -footbridge, Fig. 2a, ● -footbridge, Fig. 2b

instability due to the broadening could be observed. Nevertheless, the spectra of the response have more peaks with respect to the response of the prismatic ribbon, evidently due to the separation of other vortices, generated on the broadened part of the deck.

3.2. In-situ research

Wind effects were monitored on a three-span stress-ribbon catenary footbridge, illustrated in Fig. 1, and on many single-span suspended stress-ribbon footbridges (see Stráský 1986). None of the structures showed even a slightest indication of the loss of aerodynamic stability of the bending-torsional flutter type.

4. Bending-torsional flutter

The observed bridge behaviour can be confirmed by the theory given in (Klöppel 1962), which was applied to the following values of the dynamic parameters typical for footbridges:

mass per unit length $m = 2000 \div 6000$ kg/m

mass moment of intertia per unit length $\Theta = 15000 \div 20000$ kg · m
half of the width of the bridge deck $b = 3 \div 4$ m mass ratio

$$\mu = \frac{m}{\pi \rho b^2} = 10 \div 200, \quad \rho = 1.25 \text{ kg/m}^3 (\text{air}),$$

the ratio of inertia-radius to the half of bridge-deck width

$$r = \frac{1}{b}\sqrt{\frac{\Theta}{m}} = 0.40 \div 1.06,$$

and the ratio of the torsional and bending frequencies

$$\frac{\omega_T}{\omega_B} = 1.5 \div 6.5$$

Four domains determining the necessity of flutter analysis can be defined, as given in Figs. 8 and 9. The domains are defined in terms of nondimensional parameters of torsional to bending frequency ratio and reduced velocity, viz.

$$\frac{\omega_T}{\omega_B}, \quad \frac{\overline{V}_{kr,R}}{b \cdot \omega_B},$$

where $\overline{V}_{kr,R}$ is the flow velocity, reduced with respect to the cross-section shape according to wind-tunnel tests. For the footbridges of the type in question the design value of critical wind-velocity can be taken as given in (Klöppel 1962)

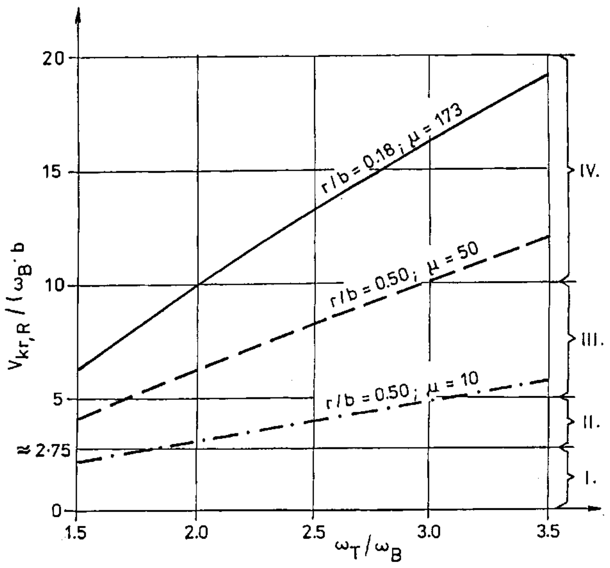

Fig. 9 Diagram for the assessment of aeroelastic instability: I-very serious, aeroelastic experiment is necessary; II-flutter analysis of the model is required; III-theoretical flutter analysis is sufficient; IV-the structure is safe

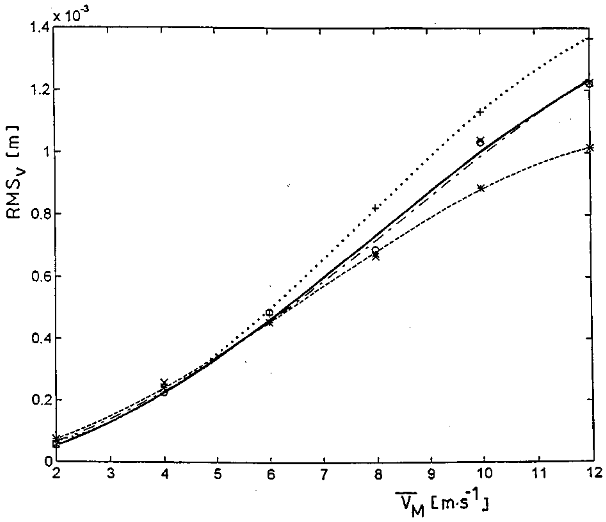

Fig. 10 The influence of the broadening of the middle of the span (vertical response of the footbridge according to Fig. 3 vs. flow-velocity)
without broadening: ○ -point No 2, *-point No 5
with largest broadening: + - point No 2, × - point No 5

$$\overline{V}_{kr,des} \approx 0.80\ \overline{V}_{kr,R}$$

Fig. 8 shows the relations between $V_{kr,R} / (\omega_B \cdot b)$ and ω_T / ω_B for zero damping of vibrations (left) and for the damping (log. decrement) $\vartheta_B = \vartheta_T = 0.20$ (right). As the exact values of damping of the real structure can be determined as late as in full scale, the interpolation between these two values seems to be sufficient for the design. In the same Fig. 8 the values of $V_{kr,R} / (\omega_B \cdot b)$ for three footbridges have been inserted. Well comparable results were obtained by Scanlan's solution (see e.g., Simiu & Scanlan 1996).

5. Conclusions

The problem of aeroelastic safety of stress-ribbon footbridges was investigated experimentally on the model in a boundary layer wind-tunnel, with particular reference to the influence of the torsional rigidity of the bridge-deck and of the viewing platform at the middle of the bridge span. The results obtained, together with previous studies of the same authors (Pirner 1997) and with the results of full-scale measurements, have confirmed that the danger of bending-torsional flutter does not threaten, if the aeroelastic parameters of the structure have values within Zone IV in Fig. 9. Also, the influence of local broadening of the bridge deck does not cause important increase of the dynamic response of the footbridge in wind.

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References

Klöppel, K., Thiele, F. (1962), "Modelversuche im Windkanal zur Bemessung von Brücken gegen Gefahr Wind-erregter Schwingungen", *Der Stahlbau*, Jg. 36, H. 12.

Pirner, M. (1994), "Aeroelastic characteristics of a stressed pedestrian bridge spanning 252 m", *Journal of Wind Engineering and Industrial Aerodynamics*, **53**, 301-314.

Pirner, M., Fischer, O. (1997), "Wind induced vibrations of concrete stress-ribbon footbridges", *Proc. 2nd European and African Conf. on Wind Engineering*, Genova, June, 1537-1544.

Simiu, E., Scanlan, R.H. (1996). *Wind Effects on Structures*, John Wiley and Sons, Inc., N. York.

Stamm, C. (1952). *Brückeneinstürze und ihre Lehren*, Leemann, Zürich.

Stráský, J., Pirner, M. (1986). *Stress-Ribbon Footbridges* (in Czech). Special issue of the estabilishment "Dopravni stavby (Constructions for Transport)", Olomouc.

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