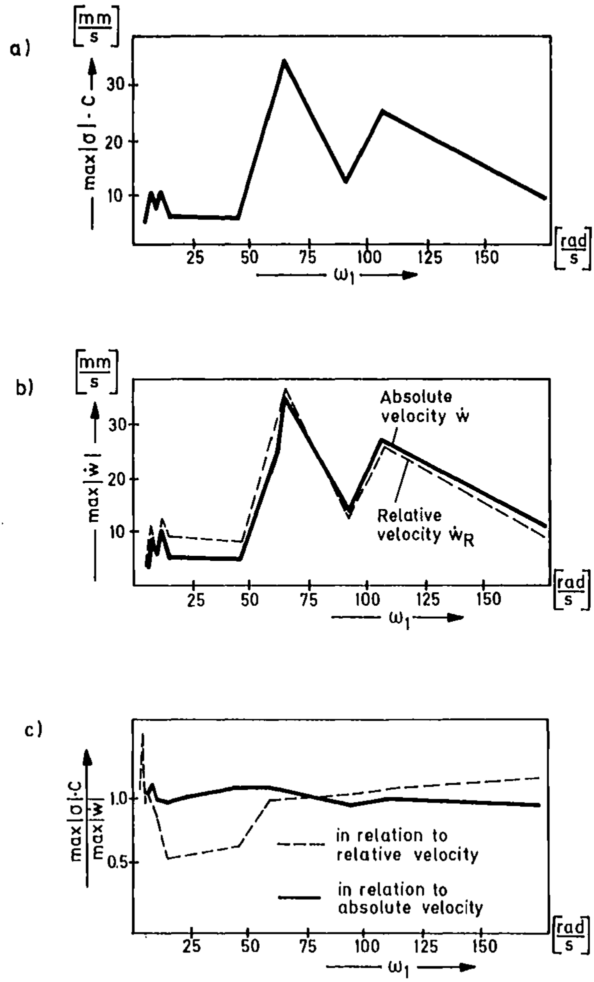

Fig. 7--Representative example--Vibration responses: (a) Maximum (magnitude) bending stress max $|\sigma|$ as a function of the first natural frequency ω_1 of the part (The constant C is composed of the material-, cross-section and loading characteristics

$C = \frac{1}{C_p \cdot C_F \cdot c_M}$); (b) Maximum (magnitude) vibration velocity as functions of the first natural frequency of the part; (c) Relationship between the velocity and the stress

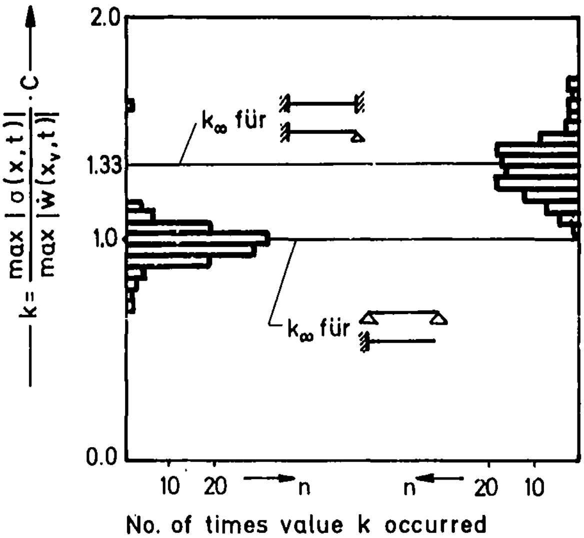

Fig. 8--Frequency distribution of the boundary condition characteristic k due to blasting (single span beam)

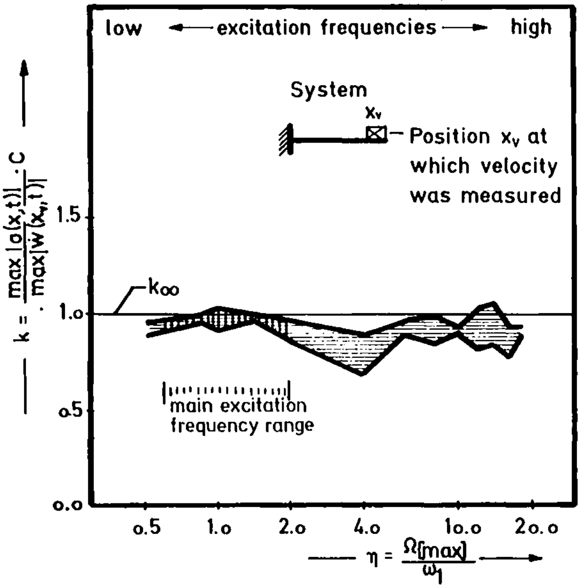

Fig. 9--Relationship between support characteristic k and tuning (dimensions) of structural part (cantilever beam subject to blasting tremors)

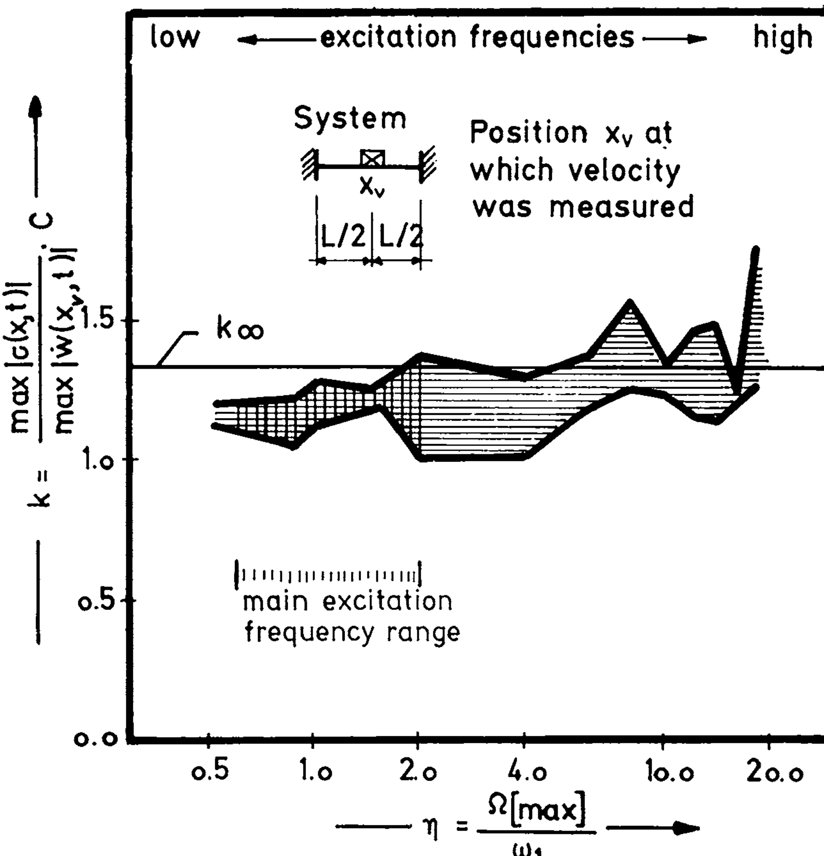

Fig. 10--Relationship between support characteristic k and tuning (dimensions) of structural part (beam rigidly fixed at both ends subjected to blasting tremors)

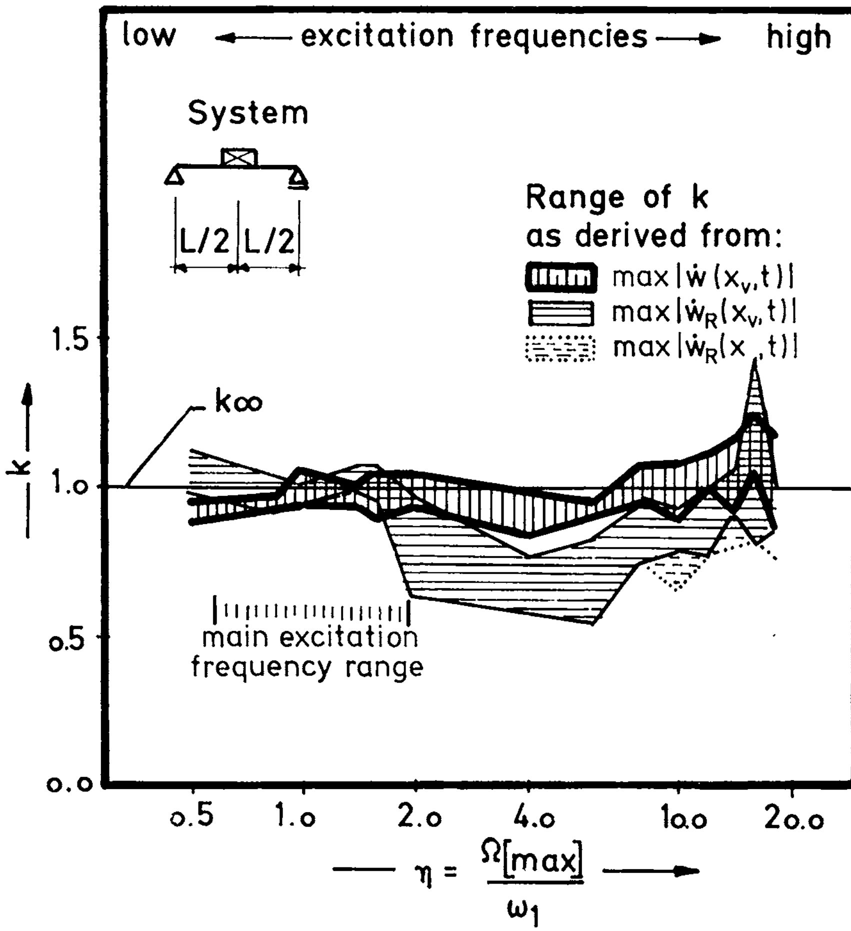

Fig. 11--Relationship between support characteristic k and tuning (dimensions) of structural part and following parameters:

- measured quantity (relative and absolute velocity, $\dot{w}_R$ and $\dot{w}$)
- position at which measurement was made (centre of beam x_V or position at which max $\dot{w}(x,t)$ occurs) (beam mounted on pivots at both ends, subjected to blasting tremors)

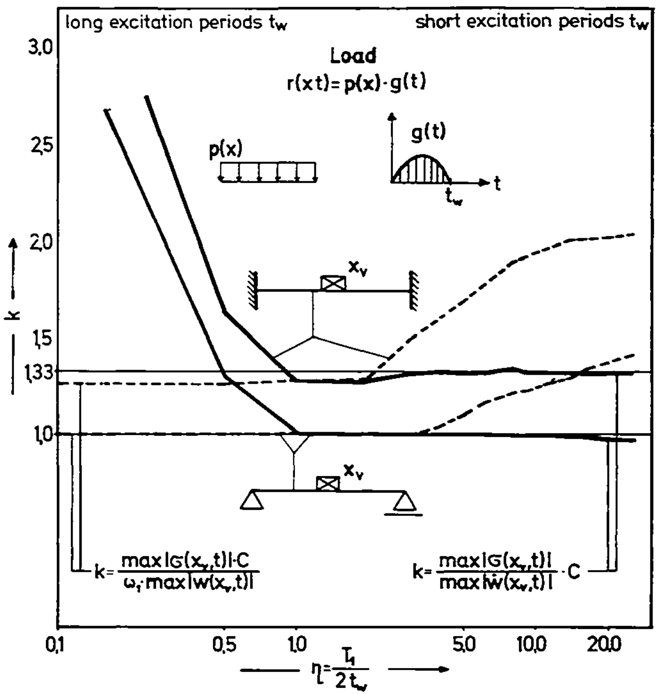

Fig. 12--Relationship between support characteristic k and tuning (dimensions) of structural part (single-span beam subject to shock-like loading)

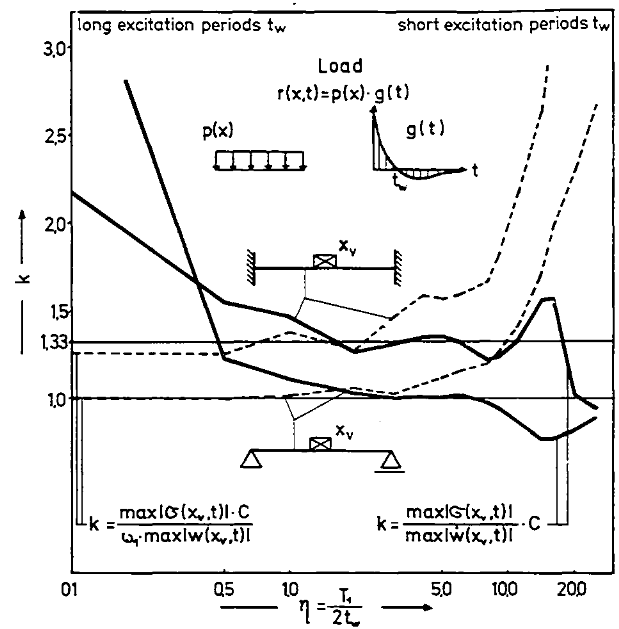

Fig. 13--Relationship between support characteristic k and tuning (dimensions) of structural part (single-span beam subjected to explosion pressure wave)

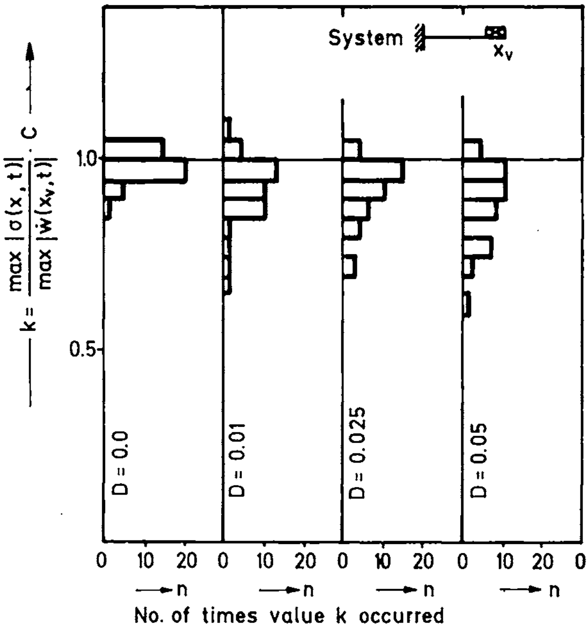

Fig. 14--Frequency distribution of boundary condition characteristic k--Effect of damping

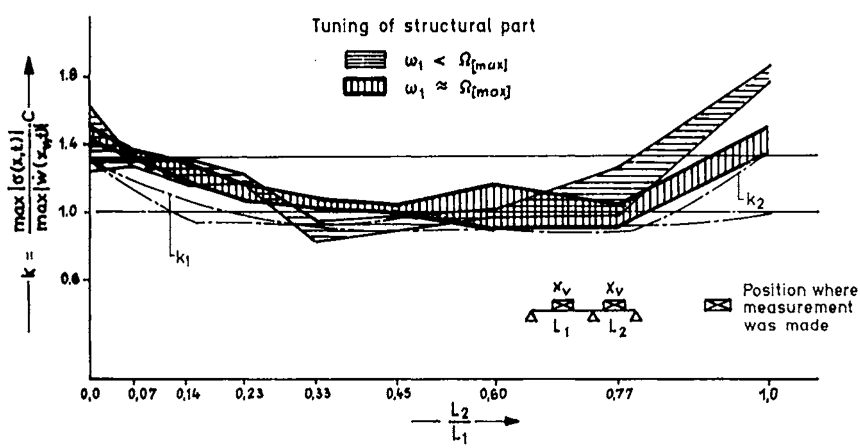

Fig. 15--Range of support characteristics k as a function of span-length ratios and tuning (dimensions) of structural part (dual-span beam subjected to blasting tremors)

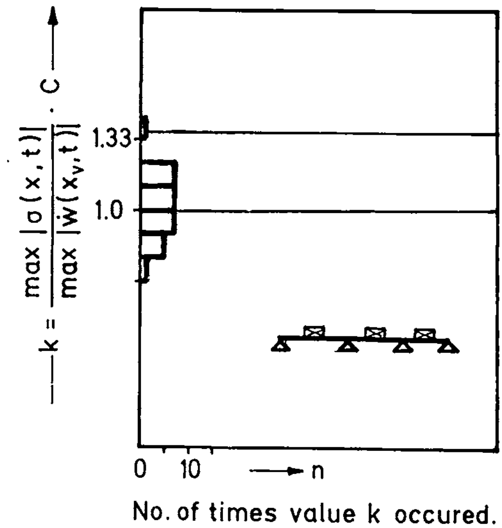

Fig. 16--Frequency distribution of support characteristic k for blasting tremors (triple-span beam)

stady-state vibration ([1], fatique factor of 3) maximum vibration velocity $\max\|\dot{w}\|$ $\left[\frac{mm}{s}\right]$	transient vibration (extension, fatique factor of 1.5) maximum vibration velocity $\max\|\dot{w}(x_v)\|$ $\left[\frac{mm}{s}\right]$	Possible effects and recommended prophylactic measures
< 2.5	< 5	No damage possible
2.5 ... 6	5 ... 12	Damage highly improbable
6 ... 10	12 ... 20	Damage not probable Stress check recommended
> 10	> 20	Stress check necessary Damage possible

Fig. 17--Rough evaluation of the effects of transient vibrations in floors by measurement of the amplitude of the vibration velocity, $\max|\dot{w}|$

SP 60-7

Frequency Matching in Continuous Post-Tensioned Concrete Highway Bridges

By T. I. Campbell, P. F. Csagoly, and A. C. Agarwal

Synopsis: Results from dynamic load tests on a number of highway bridges of various types are presented. Impact factors greater than those recommended by AASHTO have been observed for bridge structures having frequencies of vibration in the range 2.5 to 4.5 Hz. This range corresponds to that of the bounce frequencies of trucks and hence a pseudo resonance condition is approached when a truck passes over the bridge. Testing of three bridges having longitudinally-voided, post-tensioned continuous concrete decks is described and results are compared with theoretical predictions obtained from dynamic analyses of equivalent continuous beams. It is concluded that the beam idealisation yields frequencies which are sufficiently accurate for design purposes. Further the dynamic response is dependent upon the 'frequency match' between the bridge deck and the traversing vehicle. The concept of frequency matching forms the basis of the dynamic load allowance provisions proposed by the new Ontario Highway Bridge Design Code.

Keywords: bridge decks; concrete slabs; continuous beams; damping capacity; dynamic loads; dynamic tests; highway bridges; impact; post-tensioning; structural analysis; vibrations.

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INTRODUCTION

The Ministry of Transportation and Communications (MTC) of the Province of Ontario has recently carried out dynamic response tests on a number of bridge structures having continuous superstructures and which were known to exhibit a marked dynamic response when traversed by vehicular traffic. Results from these tests have been discussed elsewhere (1) and are summarised in Table 1 and Fig. 1. Table 1 shows measured values of the impact factor and frequency of vibration for thirteen bridges of varying types. Figure 1(a) shows a plot of the impact factor against the length of the main span, while in Fig. 1(b) the impact factor is plotted against the measured frequency. It can be seen from Fig. 1 that, with the exception of structures #8 and #13 the impact factors are higher than the AASHTO (2) values, and the frequencies of vibration lie within the range 2.5 to 4.5 Hz. The bounce natural frequencies of present day trucks have been measured by MTC and found to lie within this range. Thus a pseudo resonance condition, with associated high amplitude of vibration, may be approached when a truck passes over the bridge. This can lead to a high impact factor being recorded for the structure.

Results from tests on three of the above structures, numbers 1, 7 and 12, are discussed in this paper. These three structures have longitudinally-voided, post-tensioned continuous concrete decks and are typical of many of the structures located on the highway system in the Province of Ontario.

COMPUTATION OF NATURAL FREQUENCIES

Figure 2 shows a typical cross-section of the type of voided post-tensioned concrete deck used in the Province of Ontario. The voids are terminated a short distance on either side of each intermediate support and also at each end support to give a solid cross section at all the support locations. The deck is usually supported on elastomeric bearings at the end supports and on pot bearings at the intermediate supports. The intermediate supports are commonly isolated columns with each column carrying a pot bearing.

This type of deck is normally designed for static response by assuming that longitudinal moments, shears and torques in the deck can be computed from a beam analysis where the entire deck section is assumed to act as a beam (3). Consequently, the same assumption has been made in formulating the dynamic analysis. The dynamic analysis capability of the ICES STRUDL II general purpose program (4) was used to compute the natural frequencies of the equivalent straight continuous beam. The lumped mass formulation was used and the accuracy of the resulting model was established by comparison with known theoretical solutions. Figure 3 shows that for three equal continuous spans, sufficient accuracy (within 3%) could be obtained by using five lumped masses per span. The effect of the increased moment of inertia of the deck over the supports (due to solid section as opposed to voided section) was investigated and found to be negligible. Consequently, the natural frequencies of structures #1, #7 and #12 were computed using five lumped masses per span and assuming uniform inertia throughout the length of the deck.

FIELD TESTING

The basic layouts of structures #1, #7 and #12 are shown in Fig. 4. Structure #1 has five spans on a straight alignment but the supports are skewed at an angle of approximately 15 degrees. Structure #7 has three spans aligned on a horizontal circular curve of radius 5,730 ft, which for practical purposes can be regarded as straight. Structure #12 has six spans aligned on a horizontal circular curve of radius 1,430 ft. Each intermediate support of structures #1 and #7 comprises twin columns while a single column is present at each intermediate support of structure #12.

In the field tests, the vertical displacement of the deck, in a state of vibratory motion, was monitored at a number of locations along its length by means of deflectometers placed at ground level below the deck. The deflectometer (Figure 5) consists basically of a heavy steel base from which is cantilevered a short length of flexible steel strip. This strip is fitted with electrical resistance strain gauges which measure flexural strains. The connection between the cantilever beam and the desired location on the deck was provided by a fine steel wire under a tension sufficient to induce an initial deflection in the cantilever slightly