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COMPARATIVE ANALYSIS CONCERNING THE DYNAMIC RESPONSE OF SOME FOOTBRIDGE STRUCTURES WITH STEEL MAIN GIRDERS

BY

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Abstract. In this paper some aspects regarding the dynamic behavior of the footbridge structures under the traffic actions correlated with the people comfort are presented. The comfort criterion during footbridge passing depends of the frequencies and accelerations of the structure which have to be situated between certain limits. In the paper are analyzed four footbridges from the point of view of the dynamic parameters and their influence of the resonance risk and of the traffic comfort.

Keywords: footbridges; traffic comfort; dynamic response; critical frequency and acceleration; practical analysis of footbridges.

1. Introduction

This paper presents several aspects regarding the dynamic behaviour of the footbridge structures subjected to loads generated by pedestrians’ movement on the footbridge, without making references to the dynamic behaviour due to wind action.

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The maximum comfort criterion at footbridge crossing requires total lack of vibrations, but this could lead to building a less slender structure or a slender structure provided with vibration damping systems which increase the costs of the construction and need complex maintenance procedures.

A moderate comfort permits limited structure vibrations and this helps the construction of slender and good-looking structures, potentially provided with damping systems, too.

According to the nature of the deformations produced in the structure members, vibrations can be:

– transverse, when bending and shearing deformations occur;

– longitudinal, when axial compression or tensile and shearing deformations are present;

– torsional, when alternating deformations are torsional.

The paper analyses four structures of pedestrian footbridges built on steel girders from the viewpoint of the dynamic parameters and their influence upon the resonance risk and from the viewpoint of including the footbridges in a certain domain of pedestrian traffic comfort.

2. Footbridge Structure Dynamics and Traffic Comfort

The load exerted by the walking or running pedestrian is made equivalent to a concentrated force, function of time.

The experimental measurements have shown that the loading has a periodical character and is characterised by frequency, respectively by the number of steps per second.

The estimated frequency values are given in Table 1, (Sétra, 2006).

Table 1
Frequencies Induced by Pedestrian Moving

	Specific feature	Frequency [Hz]
Walking	Continuous contact	1.6… 2.4
Running	Discontinuous contact	2.0… 3.5

Conventionally, for the normal walking, frequency can be described as a Gaussian distribution of the average value of 2 Hz and a standard deviation of 0.20 Hz.

The periodical function $F(t)$ can be transposed in a Fourier series, as it has a constant component to which an infinite sum of harmonic forces is added (FIB Bulletin 32, 2005; Sétra, 2006):

$$F(t) = G_0 + G_1 \sin 2\pi f_m t + \sum_{i=1}^{n} G_i \sin\left(2\pi i f_m t - \varphi_i\right), \qquad (1)$$

where: G_0 is the static force (the weight of the pedestrian); G_1 – the amplitude of the first harmonic; G_i – the harmonic i amplitude; f_m – the walk frequency; φ_i – the phase angle of the harmonic i with respect to the first harmonic; n – the number of harmonics.

The simplest method to avoid resonance consists in avoiding the natural frequencies (one or several) of the structure to be included in the range of frequencies corresponding to the pedestrians' walk.

The risk frequency domains for the vertical vibrations are specified in various technical documents, norms and regulations (FIB Bulletin 32, 2005; Sétra, 2006; Moga, 2016).

According to EN 1990-EC 0 – Annex A2, the maximum recommended accelerations are:

– 0.7 m/s^2 – for vertical vibrations;
– 0.2 m/s^2 – for horizontal vibrations;
– 0.4 m/s^2 – for exceptional situations (of crowding).

The verification of the comfort criterion should be found when the basic frequency of the deck is lower than:

– 5 Hz – for vertical vibrations;
– 2.5 Hz – for horizontal vibrations (lateral) and torsional vibrations.

The footbridge dynamic analysis methodology presented in papers (FIB Bulletin 32, 2005; Sétra, 2006) aims at avoiding the resonance phenomenon that can occur in the case of very lightweight footbridges.

The first stage consists in establishing the *footbridge class*, by the beneficiary, function of the presumed traffic level and respectively defining the comfort level to be reached to be satisfactory.

After evaluating the natural frequencies (own frequencies) of the structure, one or more cases of dynamic loading are chosen, dependent upon the frequency ranges; with these frequency values one can calculate the values of the accelerations of the structure. Function of the values found for the accelerations, the comfort level can be established, (Sétra, 2006).

Four classes of footbridge traffic are defined, function of the size of the estimated traffic, Table 2, (FIB Bulletin 32, 2005; Sétra, 2006).

Table 2
Footbridge Classes, Function of Traffic Characteristics

Class	Traffic characteristics
I	Urban area footbridges of high pedestrian loading
II	Urban area footbridges, occasionally loaded all along the surface
III	Urban area footbridges of normal use, occasionally crossed by large groups of pedestrians
IV	Occasionally used footbridges

Domains of acceleration values associated to the comfort level

The pedestrians comfort level is correlated with the level of structure accelerations found by calculation using various cases of dynamic loading.

Four conventional domains for the vertical and horizontal accelerations are defined, Figure 1; in an increasing order, they correspond to a maximum, average and minimum comfort level, while domain 4 is related to inadmissible acceleration values (FIB Bulletin 32, 2005; Sétra, 2006).

The horizontal plane acceleration is limited to value 0.10 m/s^2 to avoid the „lock-in" phenomenon (gradual synchronisation phenomenon), concerning the fact that a set of pedestrians made up of units of differing frequencies tends to gradually get to a common frequency, that of the structure, entering in phase with the footbridge motion.

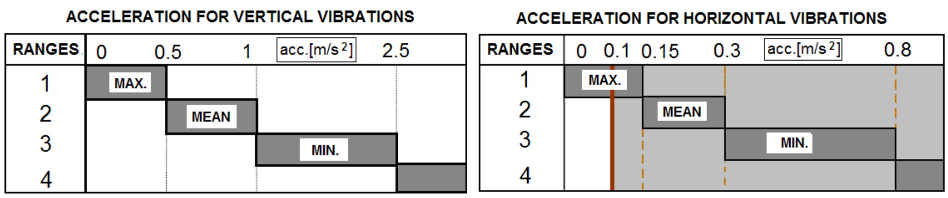


Fig. 1 – Acceleration for vertical and horizontal vibrations.

Determination of frequencies that require the performance of the dynamic calculus

In the case of footbridges in traffic classes I, II and III, it is necessary to calculate the structure own vibration frequencies. The frequencies are calculated for three directions: vertical, horizontal transverse and horizontal longitudinal. They are calculated for two system mass hypotheses:

- unloaded footbridge;
- footbridge loaded with 700 N/m^2 on the traffic surface.

Dependent upon the frequency domain, one can appreciate the resonance risk brought by the pedestrian traffic and can further establish the dynamic loading cases and verify the comfort criterion.

The vertical and horizontal vibrations can be included in four ranges related to the resonance phenomenon risk occurrence Fig. 2 (Sétra, 2006), where:

– domain **1**: maximum resonance risk;
– domain **2**: average resonance risk;
– domain **3**: low resonance risk;
– domain **4**: negligible resonance risk.

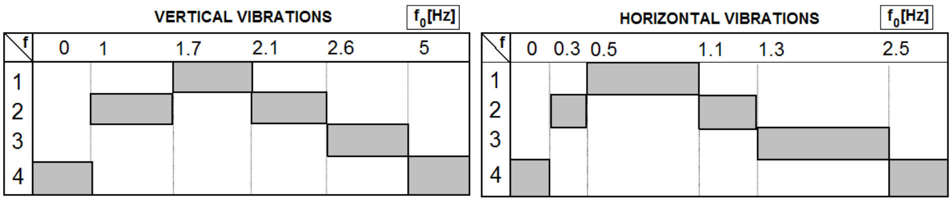


Fig. 2 – Frequency ranges of the vertical and of the transverse horizontal vibrations.

Cases of dynamic loading

Function of the footbridge class and domain of natural frequencies, the dynamic calculus for three cases of loading is required:

- case 1: moderate (disperse) and dense crowding;
- case 2: very dense crowding;
- case 3: very dense crowding to which the effect of the secondary harmonic is added.

The loads taken for each case is given in papers (FIB Bulletin 32, 2005; Sétra, 2006; Moga, 2016).

For the simply supported beam of constant characteristics, the analytical calculus for the vibration modes is made with the relationships given in Table 3.

Table 3
Determination of Dynamic Parameters

Mode	Natural pulsation	Natural frequency	Vibration mode shape
Simple bending with *n* half-waves	$\omega_n = \frac{n^2\pi^2}{L^2}\sqrt{\frac{EI}{\rho S}}$	$f_n = \frac{n^2\pi}{2L^2}\sqrt{\frac{EI}{\rho S}}$	$v_n(x) = \sin\left(\frac{n\pi x}{L}\right)$
Tensile – compression with *n* half-waves	$\omega_n = \frac{n\pi}{L}\sqrt{\frac{ES_N}{\rho S}}$	$f_n = \frac{n}{2L}\sqrt{\frac{ES_N}{\rho S}}$	$u_n(x) = \sin\left(\frac{n\pi x}{L}\right)$
Torsion with *n* half-waves	$\omega_n = \frac{n\pi}{L}\sqrt{\frac{GI_\omega}{\rho I_r}}$	$f_n = \frac{n}{2L}\sqrt{\frac{GI_\omega}{\rho I_r}}$	$\theta_n(x) = \sin\left(\frac{n\pi x}{L}\right)$
Maximum acceleration	$\mathrm{Acc}_{\max} = \frac{1}{2\xi_n}\cdot\frac{4F}{\pi\rho S}$ (complying with (Guțiu *et al.*, 2016; Moga, 2016)		
Measurement units: L [m]; $E = 210 \times 10^9$ [N/mm^2]; I [m^4]; ρS [kg/m]; m [kg/m]. Parameters: ρS – construction linear density; ρI_f – torsion inertia momentum; ES_N – axial stress stiffness; EI – bending stiffness; GI_ω – restraint torsion stiffness.			

In practice, as footbridges are narrow compared to length and stiff to torsion in closed section, the frequencies from torsion and axial stresses are high and the analysis is performed only for bending vibrations (in vertical and horizontal planes).

Analysis of dynamic parameters (Guțiu *et al.*, 2016; Moga, 2016; Moga *et al.*, 2014)

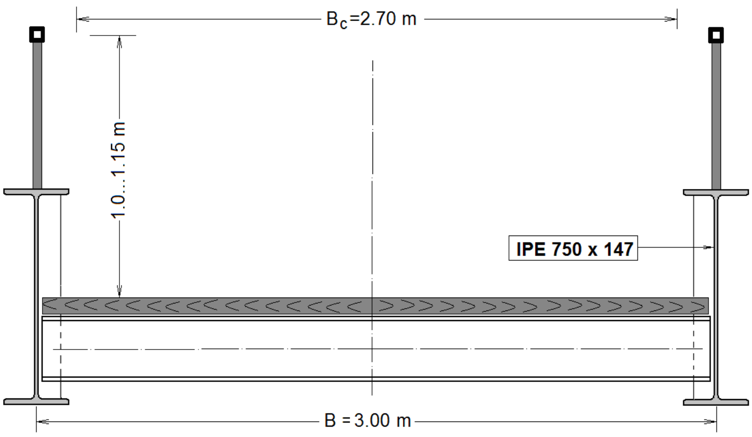
Is being analysed four types of footbridges on steel beams or steel-concrete girders, from the point of view of the vertical vibration frequencies, resulted from their own weight (own vibration frequency) or from the partial loads, considering a rank in the traffic class III.

It must be mentioned that structures of the types 1 to 4 have the same wide span – 24.0 m and approximately the same width between the guardrails (parapets) – 3.00 m.

Type1: Rolled steel girder footbridge

The parameters of the dynamic calculus for a steel girder footbridge with the span of 24.00 m and the distance between the main girders of 3.00 m, and with the structure given in Figure 3 are calculated.

The footbridge belongs to traffic class III. The structure was used in a designed structure (Project of XC PROJECT - Cluj-Napoca).



Characteristics of IPE 750 × 147

G = 147 kg/m
I_y = 166,100 cm^4
$W_{el.y}$ = 4,411 cm^3

Fig. 3 – Superstructure of rolled steel girder footbridge

In Fig. 4 a footbridge built in Bihor County is presented, which have the structure of type presented in the Fig. 3 (Project of XC PROJECT – Cluj-Napoca).

Fig. 4 – Footbridge built in Bihor County.

Solution:

After the evaluation of the actions, a total permanent load (own weight) of about 450 kg/m has been found.

Deck characteristics

– the natural linear density: m = 450 kg/m;

– the inertia moment: $I = 2 \times 166{,}100\ \text{cm}^4 = 0.00332\ \text{m}^4$;

– the deck linear density taking into account the pedestrians' density which for the traffic class III is d = 0.5 P/m^2;

– the number of pedestrians on the footbridge: $n = S \times d = (24 \times 2.70) \times \times 0.5 = 32.4$ P;

– the total pedestrians' weight: 70 × 32.4 = 2,268 kg;

– the linear density of the pedestrians $m_P = m/L$ = 2,268/24 = 94.5 kg/m;

– the linear density:

- unloaded footbridge: ρS = 450 kg/m;
- footbridge loaded with density d: ρS = 450 + 94.5 = 544.5 kg/m.

The frequencies for Vibration mode 1are found:

– higher frequency: $$f_1 = \frac{1^2\pi}{2\times 24^2}\sqrt{\frac{210\times 10^9 \times 0.003322}{450}} = 3.39\ \text{Hz} \quad (2)$$

– lower frequency: $$f_1 = \frac{1^2\pi}{2\times 24^2}\sqrt{\frac{210\times 10^9 \times 0.003322}{544.5}} = 3.08\ \text{Hz} \quad (3)$$

The frequencies for Vibration mode 2 will be:

– higher frequency: $$f_2 = \frac{2^2\pi}{2\times 24^2}\sqrt{\frac{210\times 10^9 \times 0.003322}{450}} = 13.56\ \text{Hz} \quad (4)$$

– lower frequency: $f_2 = \dfrac{2^2 \pi}{2 \times 24^2} \sqrt{\dfrac{210 \times 10^9 \times 0.003322}{544.5}} = 12.32 \text{ Hz}$ (5)

It is noticed that the frequencies in Vibration mode 1 are included in the Risk domain 3: low resonance risk, while the frequencies in the Vibration mode 2 range in risk domain 4: negligible resonance risk.

For these domains it is not necessary to perform the dynamic calculus and respectively of the system acceleration (ψ = 0) (Sétra, 2006).

Type 2: Flexible rolled steel girder footbridges

The case with Type 2 is analysed, where the main girders are made from IPE 500 laminated profiles, whose stiffness is much more reduced than that of IPE 750 profiles.

Solution:

After the evaluation of the actions, it yielded a total permanent load (own weight) of about 300 kg/m.

Deck characteristics

– the inertia moment: $I = 2 \times 48{,}200\ \text{cm}^4 = 0.000964\ \text{m}^4$;

– the natural linear density: m = 300 kgt/m;

– the modulus of elasticity: $E = 210 \times 10^9\ \text{N/m}^2$;

– the deck linear density is calculated taking into account the pedestrians' density which for the traffic class III is $d = 0.5\ \text{P/m}^2$;

– the number of pedestrians on the footbridge: $n = S \times d = (24 \times 2.70) \times \times 0.5 = 32.4$ P;

– the total pedestrians' weight: 70 × 32.4 = 2,268 kg;

– the linear density of the pedestrians $m_P = m/L$ = 2,268/24 = 94.5 kg/m;

– the linear density:

- unloaded footbridge: ρS = 300 kg/m;
- footbridge loaded with density d: ρS = 300 + 94.5 = 394.5 kg/m.

The frequencies for Vibration mode 1are found:

– higher frequency: $f_1 = \dfrac{1^2 \pi}{2 \times 24^2} \sqrt{\dfrac{210 \times 10^9 \times 0.003322}{300}} = 2.24 \text{ Hz}$ (6)

– lower frequency: $f_1 = \dfrac{1^2 \pi}{2 \times 24^2} \sqrt{\dfrac{210 \times 10^9 \times 0.003322}{394.5}} = 1.95 \text{ Hz}$ (7)

The frequencies for Vibration mode 2 will be:

– higher frequency: $$f_2 = \frac{2^2\pi}{2\times 24^2}\sqrt{\frac{210\times 10^9\times 0.003322}{300}} = 8.96\ \text{Hz} \tag{8}$$

– lower frequency: $$f_2 = \frac{2^2\pi}{2\times 24^2}\sqrt{\frac{210\times 10^9\times 0.003322}{394.5}} = 7.80\ \text{Hz} \tag{9}$$

It is noticed that the lower frequency in Vibration mode 1 range in domain 1: maximum resonance risk, while frequencies in Vibration mode 2 are in the negligible resonance risk domain, Fig. 5.

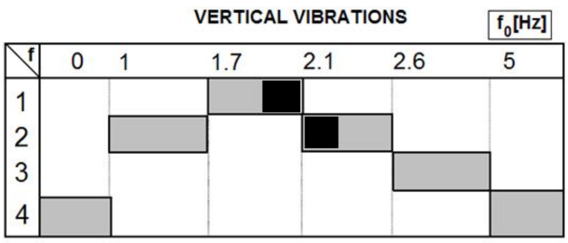


Fig. 5 – Ranges of vertical vibrations.

Evaluation of dynamic loading

We evaluate the dynamic loading in Vibration mode 1, for a damping ratio of ξ_n = 0.4% = 0.4 × 10^{-2} (steel structure) (Sétra, 2006).

$$F_s = d\times(280N)\cos(2\pi f_v t)\times 10.8\sqrt{\xi/n}\times\psi = 16.8\cos(3.9\pi t),\ [\text{N/m}^2]. \tag{10}$$

Coefficient ψ = 1, Fig. 6.

Load per unit length: $F = 2.7m\times F_s = 54.4\cos(3.9\pi t),\ [\text{N/m}]..$

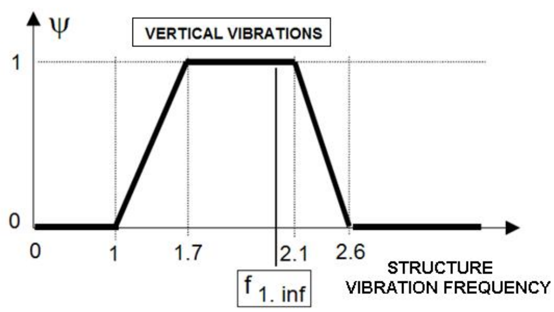


Fig. 6 – Factor in the case of walking for vertical vibrations.

Evaluation of dynamic response

The system acceleration will be:

$$\mathrm{Acc}_{\max} = \frac{1}{2\xi_n} \cdot \frac{4F}{\pi\rho S} = \frac{1}{2\times 0.4\times 10^{-2}} \cdot \frac{4\times 45.4}{\pi\times 394.5} = 18.3 \gg \text{ acceptable level} \quad (11)$$

It is found that the system acceleration frames in Domain 4, Figure 7 corresponding to an unacceptable level (>> 2.5).

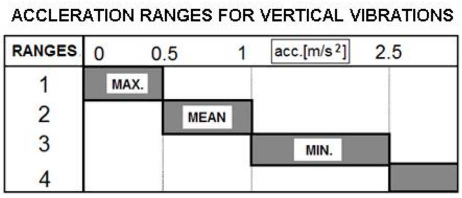


Fig. 7 – Acceleration ranges for vertical vibrations.

The recommended solution in this case to reduce the acceleration under 2.5 m/s^2 for the main girders could be to choose laminated profiles between the two put to analysis and a repetition of the dynamic calculus. The SLU and SLS (vertical sag) should also be verified.

Type 3: Integral composite steel-concrete girder

The structure of the deck with a composite steel-concrete structure with the following design data is analysed: L = 24.0 m; net width 3.00 m; cross bars at: l_a = 2.40 m.

The structure of the deck is made from two main girders with a composite steel-concrete section, crossbars structurally in action with the slab of 12 cm thick monolith reinforced concrete, Fig. 8.

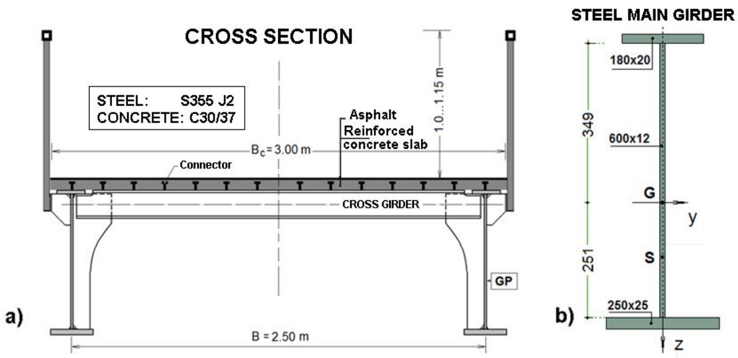


Fig. 8 – Composite steel concrete welded girder footbridge: *a* – cross section of footbridge deck; *b* – cross section of the main girder – steel component.

Solution:

In this case, to simplify calculations, a unique steel concrete equivalence coefficient for long term and short term loads (admitted for building structures), is calculated as follows:

$$n_0 = \frac{E_a}{E_{cm}}, \tag{12}$$

where: E_a is the modulus of elasticity of the steel in the steel girder; E_{cm} – the secant modulus of elasticity secant of the concrete in the slab.

For the slab concrete of grade 30/37, E_{cm} = 33 GPa = 3.3 × 10^5 daN/cm^2.

It yields: $n_0 = \frac{2.1\times10^6}{3.3\times10^5} = 6.36..$

One considers the unique equivalence coefficient [SR EN 1994-1-1:2004 § 5.4.2.2(11)]:

$$n = 2n_0 = 2\times6.36 = 12.72. \tag{13}$$

The steel equivalent width of the concrete slab will be:

$$b^*_{\text{eff}} = \frac{b_{\text{eff}}}{n} = \frac{1{,}500}{12.72} = 188 \text{ mm}. \tag{14}$$

The characteristics of the mixed girder are shown in Fig. 9.

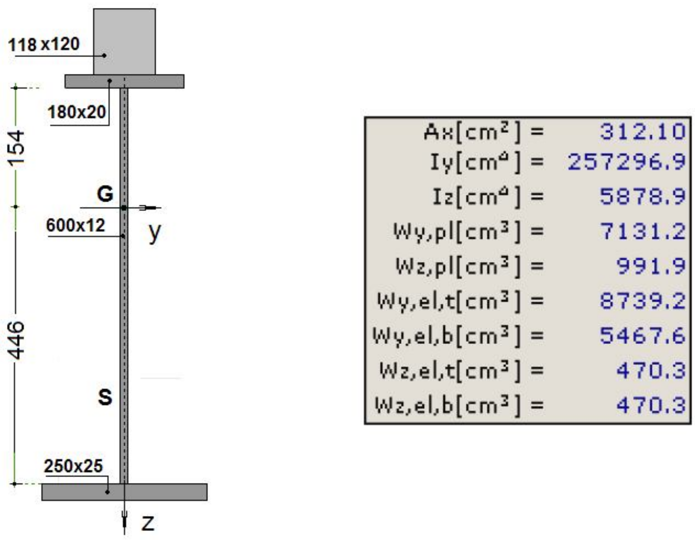


Fig. 9 – Characteristics of the composite girders.

Natural frequency of the structure

The higher natural frequency of the structure for Vibration mode 1 is verified with relationship:

$$f_n = \frac{n^2\pi^2}{2L^2}\sqrt{\frac{EI}{\rho S}} \Rightarrow f_1 = \frac{\pi}{2L^2}\sqrt{\frac{EI}{\rho S}}, \tag{15}$$

where: L = 24 m; $E = 210 \times 10^9$ N/m^2; ρS = 1,380 + 370 = 1,750 kgm/ – linear density for unloaded footbridge.

It yields: f_1 = 2.14 Hz – *average resonance risk.*

The same result is found with the relationship in Eurocode 1-2, (SR EN 1991, 2005):

$$n_0\ [\mathrm{Hz}] = \frac{17.75}{\sqrt{\rho_0\ [\mathrm{mm}]}} \text{ (notations complying with EC1-2).} \tag{16}$$

When the steel concrete equivalence coefficient is used to assess the structure frequency, namely the short term loading aspect, the result is:

$$n_0^{\mathrm{st}} = \frac{E_a}{E_{cm}} = \frac{2.1\times10^6}{3.3\times10^5} = 6.36 \text{ (s.t. = short term).} \tag{17}$$

It yields:

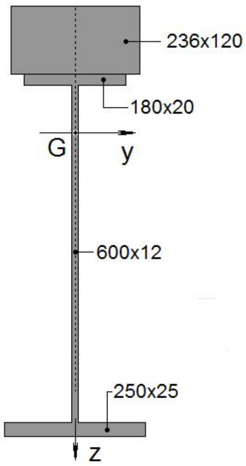
$$\rho_0 = \frac{5qL^4}{384EI_{\mathrm{y.el.}}^{\mathrm{compus}}} = \frac{5\times17.5\times2{,}400^4}{384\times2.1\times10^6\times2\times312{,}522} = 5.76\ \mathrm{cm} \approx 58\ \mathrm{mm}, \tag{18}$$

$$n_0 = \frac{17.75}{\sqrt{58}} = 2.3\ \mathrm{Hz} \text{ – average resonance risk.}$$

where:

$$I_{\mathrm{y.el.}}^{\mathrm{compus}} = 2\times312{,}522\ \mathrm{cm}^4 = 625{,}044\times10^{-8}\ \mathrm{m}^4. \tag{19}$$

The steel concrete equivalence coefficient for short term loads is considered to calculate the sag and frequency in the software specific for bridge mixed steel-concrete structures, namely *ACOBRI software* (Fig. 10).



Ax[cm²] =	453.70
Iy[cm⁴] =	312521.6
Iz[cm⁴] =	17380.1
Wy,pl[cm³] =	7861.5
Wz,pl[cm³] =	2245.1
Wy,el,t[cm³] =	14125.0
Wy,el,b[cm³] =	5747.6
Wz,el,t[cm³] =	1390.4
Wz,el,b[cm³] =	1390.4

Fig. 10 – Characteristics of the composite girders in short-term loading.

Using *ACOBRI* software, the identical value of own frequency is found: n_0 = 2.3 Hz.

The footbridge sag from the net load is:

$$\rho_{0,p}=\frac{5p_tL^4}{384EI_{\text{y.el}}^{\text{compus}}}=\frac{5\times15\times2{,}400^4}{384\times2.1\times10^6\times2\times312{,}522}=4.9\ \text{cm}. \quad (20)$$

This sag represents:

$$\rho_{0,p}=\frac{L}{490}\approx\rho_{\lim}=\frac{L}{500}. \quad (21)$$

If we calculate the deck linear density taking into account the density of the pedestrians, which is d = 0.5 P/m^2 for traffic class III, it yields:

– the number of pedestrians on the footbridge: $n = S \times d$ = (24 × 3) × 0.5 = = 36 P;

– the total weight of the pedestrians: 70 × 36 = 2,520 kg;

– linear density of pedestrians: $m_P = m/L$ = 2,520/24 = 105 kg/m;

– linear density:

- unloaded footbridge: ρS = 1,750 kg/m;
- footbridge loaded with density d: : ρS = 1,750 + 105 = 1,855 kg/m.

The frequencies for Vibration mode 1 are calculated:

– higher frequency: f_1 = 2.3 Hz (previously calculated); (22)

– lower frequency: $$f_1 = \frac{1^2\pi}{2\times 24^2}\sqrt{\frac{210\times 10^9\times 0.003322}{300}} = 2.24\ \text{Hz}\,. \quad (23)$$

The frequencies present close values because the influence of the net load considered here is small and both frequencies range in the average resonance risk domain.

As the sag and own frequency are ranged within the recommended values limits, respectively the frequency in the average resonance risk domain, the stiffness of the structure can be increased together with that of the main girders.

In order to increase structure stiffness, one can modify the height of the cores of the main girders to h_w = 700 mm.

Type 4: Composite steel concrete girder footbridge with circular hollows in the web of non-symmetrical cross section

In this case the footbridge deck with a composite structure of steel and concrete is analysed, taking into account the following known values: L = 24.0 m; net length: Bc = 3.00 m, concrete slab thickness 12 cm, quasi-identical with Type 3 structure – Fig. 8 *a*.

The deck structure is made up of two main girders with a composite steel-concrete section made with hollow laminated bars, where the welded laminated profiles differ for the two mentioned portions, Fig. 11.

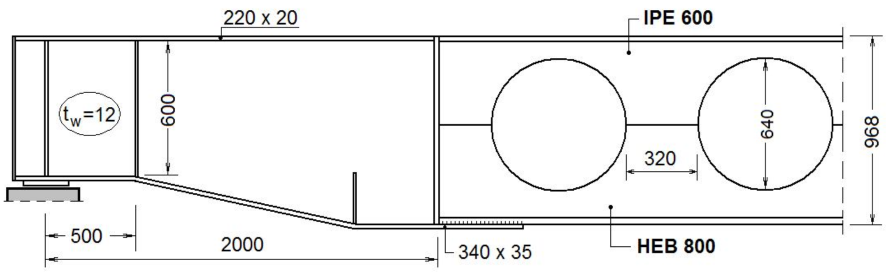


Fig. 11 – Structure of main girders with hollows in the web.

Solution:

For the type 3 footbridge 3: b_{eff} = 150 cm.

For the slab concrete, grade 30/37, E_{cm} = 33 GPa.

It yields: $n_0 = \dfrac{2.1\times10^6}{3.3\times10^5} = 6.36$ Hz.

The unique equivalence coefficient is taken as: $n = n_0 = 2 \times 6.36 =$
$= 12.72$.

The structure natural frequency

The higher natural frequency of the structure for the Vibration mode 1 is verified with the relationship:

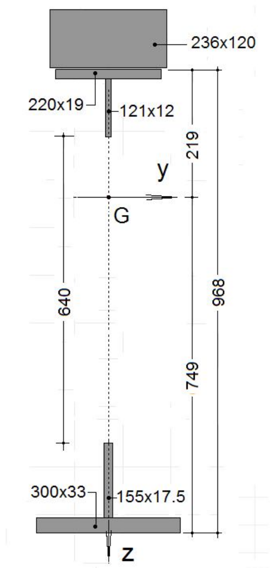
$$f_n = \frac{n^2\pi^2}{2L^2}\sqrt{\frac{EI}{\rho S}} \Rightarrow f_1 = \frac{\pi}{2L^2}\sqrt{\frac{EI}{\rho S}}, \tag{24}$$

where: $L = 24$ m; $E = 210 \times 10^9$ N/m^2.

The inertia moment is assessed for the short term loads.

One takes the average inertia moment, between the hollow area (section in Fig. 12) and the full area.

$$n_0^{\text{st}} = \frac{E_a}{E_{cm}} = \frac{2.1\times10^6}{3.3\times10^5} = 6.36 \text{ (s.t. = short term).} \tag{25}$$



Ax[cm²] =	465.64
Iy[cm⁴] =	887612.9
Iz[cm⁴] =	22263.9
Wy,pl[cm³] =	13495.6
Wz,pl[cm³] =	2659.5
Wy,el,t[cm³] =	26176.4
Wy,el,b[cm³] =	11852.0
Wz,el,t[cm³] =	1484.3
Wz,el,b[cm³] =	1484.3

Fig. 12 – Characteristics of the composite girder with hollows in the web.

It yields:
In the hollow area:

$$I_1 = 2I_{\text{y.el.gol}}^{\text{compus}} \approx 2\times 887{,}613\ \text{cm}^4 = 1{,}775\times 10^{-2}\ \text{m}^4.$$

In the full area:

$$I_2 = 2I_{\text{y.el.plin}}^{\text{compus}} \approx 2\times 952{,}090\ \text{cm}^4 = 1.9\times 10^{-2}\ \text{m}^4 = 1.07 I_{\text{y.el.gol}}^{\text{compus}}.$$

Average inertia moment:

$$I = \left(I_1 + I_2\right)/2 = 1.84\times 10^6\ \text{cm}^4 = 1.84\times 10^{-2}\ \text{m}^4.$$

$$f_1 = \frac{\pi}{2L^2}\sqrt{\frac{EI}{\rho S}} = \frac{\pi}{2\times 24^2}\sqrt{\frac{210\times 10^9\times 0.0184}{1{,}440}} = 4.46\ \text{Hz}. \qquad (26)$$

Linear density for the partially loaded footbridge: ρS = 1,440 + 370 = = 1,810 kg/m.

It yields: f_1 = 3.98 Hz – low resonance risk.

The same result is found with the relationship from EC1-2:

$$n_0\ [\text{Hz}] = \frac{17.75}{\sqrt{\rho_0\ [\text{mm}]}}$$ (notations complying with EC1-2).

Sag: $$\rho_0 = \frac{5qL^4}{384 E I_{\text{y.el.}}^{\text{compus}}} = \frac{5\times 18.1\times 2{,}400^4}{384\times 2.1\times 10^6\times 1.84\times 10^6} = 2.02\ \text{cm} = 20\ \text{m}..$$

Natural frequency: $n_0 = \dfrac{17.75}{\sqrt{20}} = 3.97\ \text{Hz}$ – low resonance risk.

The steel-concrete equivalence coefficient corresponding to short term loads is taken into account to calculate the sag and frequency in the software for mixed steel-concrete structures for bridges– the *ACOBRI software.*

The footbridge sag from the net load is:

$$\rho_{0,p} = \frac{5p_t L^4}{384 E I_{\text{y.el.}}^{\text{compus}}} = \frac{5\times 15\times 2{,}400^4}{384\times 2.1\times 10^6\times 1.84\times 10^6} = 1.68\ \text{cm}. \qquad (27)$$

The sag represents:

$$\rho_{0,p} = \frac{L}{1{,}428} \ll \rho_{\lim} = \frac{L}{500}. \qquad (28)$$

4. Conclusions

It should be mentioned that structures of Type 1,...,Type 4 have the same span – 24.0 m and the same width between the handrails – 3.00 m.

In the case of footbridges, besides the verifications corresponding to ultimate limit states and serviceability limit states, it is necessary to verify the pedestrians' traffic comfort directly correlated with the structure vibration frequency (resonance risk) and with its acceleration.

If the mentioned characteristics – frequency and acceleration – are found in the critical range, measures to alter the values of the parameters should be taken, so that they are included within the limits recommended by norms or other recognised technical documents.

Table 4 presents in a synthetic form, the own frequencies found for Vibration mode 1 in the vertical plane, of the investigated footbridges, as well as the domains of the resonance risks and several comments, where necessary.

Mention should be made that Type 1... to Type 4 structures have the same span 24.0 m and the same width between the handrails – 3.00 m.

Paper (Moga, 2016) presents in detail aspects regarding the strength analysis of the footbridges and the dynamic behaviour of the structures synthetically shown in the examples in this paper. Other footbridge structures not included in the present paper are also described there.

Table 4
Synthesis of Analysis

Footbridge type	Main girders type	Natural frequency f_1 [Hz] lower / higher	Domain of resonance risk
Type 1	Rolled steel girders. IPE 750. L = 24 m	3.39/3.08	Negligible risk of resonance
Type 2	Rolled steel girders. IPE 500. L = 24 m	2.24/1.95	Medium risk/maximum risk Inacceptable level of acceleration
Type 3	Composite steel-concrete structure. Welded steel girders. L = 24 m	2.30/2.29	Medium risk of resonance
Type 4	Composite steel-concrete structure. Steel girders with circular holes in web. L = 24 m	3.98/3.97	Negligible risk of resonance

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ANALIZĂ COMPARATIVĂ PRIVIND RĂSPUNSUL DINAMIC AL UNOR PASARELE PIETONALE PE GRINZI PRINCIPALE METALICE

(Rezumat)

Sunt prezentate unele aspecte privind comportarea dinamică a structurii pasarelelor pietonale sub acțiunea încărcărilor generate din deplasarea pietonilor, în corelare cu confortul de circulație a pietonilor care traversează structura.

Criteriul de confort la traversarea pasarelei presupune încadrarea frecvențelor de vibrație şi ale accelerațiilor structurii între anumite limite, astfel încât să fie evitat fenomenul de amplificare a vibrațiilor sau de rezonanță.

În lucrare sunt analizate patru structuri de pasarele pietonale pe grinzi metalice, din punct de vedere al parametrilor dinamici şi al influenței acestora asupra riscului de rezonanță a structurii, precum şi din punct de vedere al încadrării pasarelelor într-un anumit domeniu referitor la confortul de circulație al pietonilor.