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# DYNAMIC ANALYSIS OF A FOOTBRIDGE STRUCTURE ON A CENTRAL ARCH

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***Rezumat***

În lucrare sunt prezentate unele aspect privind comportarea dinamică a structurii pasarelelor pietonale sub acţiunea încărcărilor generate din deplasarea pietonilor, în corelare cu confortul de circulaţie a pietonilor care traversează structura. Criteriul de confort la traversarea pasarelei presupune încadrarea frecvenţelor de vibraţie şi ale acceleraţiilor structurii între anumite limite, astfel încât să fie evitat fenomenul de amplificare a vibraţiilor sau de rezonanţă. În lucrare se analizează răspunsul dinamic al structurii unei pasarele pietonale pe arc central, şi soluţia analizată pentru ca parametrii dinamici să se încadreze în limitele necesare satisfacerii condiţiilor de siguranţă legate de riscul de rezonanţă şi de asigurare a confortului de circulaţie a pietonilor.

**Cuvinte cheie: pasarele pietonale, confort de circulaţie, analiză dinamică, frecvenţe şi acceleraţii critice, analiza practică a parametrilor dinamici**

***Abstract***

In this paper some aspects regarding the dynamic behavior of footbridge structures under traffic actions correlated with the people's comfort are presented. The comfort criterion during footbridge passing depends of the frequencies and accelerations of the structure which must be situated between certain limits. If the frequencies and accelerations of the structure are in the critical domains, some measures to modify them must be taken.

**Keywords: footbridges, traffic comfort, dynamic analysis, critical frequency and acceleration, dynamic parameters modification.**

# 1. INTRODUCTION

This paper presents aspects related to the dynamic behavior of footbridge structures under pedestrian traffic, without considering the dynamic behavior given by the wind action.

The maximal comfort criterion for pedestrians requires zero vibrations, which would lead to a heavy structure or would require vibration damping systems that implies higher construction costs and a complex maintenance.

A moderate comfort criterion allows for limited structural vibrations that lead to slender and esthetical pleasing structures, some equipped with damping systems.

Depending on the nature of the deformations produced in the structural elements, the vibrations are classified as:

- transverse, when bending or shearing deformations occur;
- longitudinal, when axial compression and stretching deformations occur;
- torsion, when alternating deformations are torsional.

The paper briefly presents the methodology for assessing the pedestrian traffic comfort on footbridges, correlated with the structure frequency and its acceleration, evaluated from the permanent loads or from some dynamic load models.

The dynamic response of the structure of a footbridge on arches is analyzed; the solution requires that the dynamic parameters fall within the limits necessary to meet the conditions regarding the resonance risk and to ensure the pedestrian traffic comfort.

# 2. THE DYNAMICS OF FOOTBRIDGES STRUCTURE AND THE TRAFFIC COMFORT CRITERION

The load given by the pedestrian walking or running is equivalent to a time-concentrated force.

Experimental measurements have shown that the load has a periodic character and is characterized by frequency and the number of steps per second. The estimated frequency values are given in Table 1 [1].

Conventionally, for walking, the frequency can be described by a Gaussian curve with an average value of 2 Hz and a standard deviation of 0.20 Hz.

**Table 1.Frequencies induced by pedestrians**

| | **Walking characteristics** | **Frequency** [Hz] |
|---|---|---|
| Walking | continuous contact with the surface | 1.6...2.4 |
| Running | discontinuous contact | 2.0...3.5 |

The conventional model for the dynamic response at a point of the structure can be obtained by amplifying the effect of a single pedestrian with the factor that represents the number of pedestrians synchronized on the walkway [1], [2]:

- $\lambda$ - pedestrian entrance rate (person/sec);
- T - the time a pedestrian requires to cross the walkway (T=L/v);
- $\lambda$T - the number of pedestrians crossing the bridge at a point in time

Experimental tests and computer simulations helped establish the following relationships for the number of synchronized people:

- moderately dense crowd: $N_{eq} = 10.8\sqrt{N\xi}$;
- very dense crowd: $N_{eq} = 1.85\sqrt{N}$.

where N is the number of pedestrians on the footbridge (density*area) and $\xi$ is the critical damping ratio (factor).

In case of transverse vibrations, the lock-in phenomenon may occur, which refers to the fact that a crowd composed of units with different frequencies tend to gradually receive a common frequency, that of the structure, and enter in phase with the movement of the bridge.

The easiest way to avoid the resonance phenomenon is not to include the natural frequencies (one or more) of the structure to be included in the pedestrian frequency range.

Table 2 gives the risk frequencies domains for vertical vibrations, specified in various technical papers, norms and regulations.

**Table 2. Risk frequencies**

| **Norm, regulations** | **Frequency** [Hz} |
|---|---|
| Eurocode 2 | 1.6 – 2.4 |
| Eurocode 5 | 0 - 5 |
| Eurocode 0 | < 5 |
| BS 5400 | < 5 |
| Japan regulation | 1.5 -2.3 |

According to EN 1990-EC 0 – Annex A2, [3], the maximum recommended accelerations are as follows:

- 0.7 $m/s^2$ – vertical vibrations;
- 0.2 $m/s^2$ – horizontal vibrations;
- 0.4 $m/s^2$ – exceptional situations (agglomerations).

Verification of the comfort criterion should be performed if the fundamental frequency of the structure is less than the value of 5 Hz for vertical vibrations and 2.5 Hz for horizontal (lateral) vibrations and torsional vibrations.

If the limit acceleration criterion is not met, measures to improve dynamic behavior should be considered.

## 3. METHODOLOGY FOR THE DYNAMIC ANALYSIS OF FOOTBRIDGES

The methodology presented in the papers [1], [2] and [4] aims at avoiding the resonance phenomenon that can occur in the case of very light bridge structures.

The first step is to establish *the footbridge class* by the beneficiary according to the presumed level of traffic, i.e. the establishment of the necessary comfort level to be satisfied.

After evaluating the natural frequencies of the structure, one or more dynamic load cases are selected, depending on the frequency range, and with these loads the values of the structure's accelerations can be determined. Depending on the values obtained for the accelerations, the level of comfort can be determined.

### 3.1. Footbridge class and comfort level

There are four Traffic Classes for walkways, depending on the size of the estimated traffic, Table 3 [1], [2].

In general, the circulation speed is reduced as the traffic density increases, the pedestrian having to adjust its velocity to the movement of the mass. The first restriction occurs at a density of 0.6 $pers/m^2$, when crossing the footbridge becomes difficult. Over a density of 1.0 $pers/m^2$, freedom in circulation is severely low; the pedestrian in motion must adjust its frequency and speed according to the other pedestrians. At a density of more than 2.0 $pers/m^2$, it is only possible to move with small steps, resulting in a "very crowded stream", and the pedestrians cannot move independently.

**Table 3. Traffic classes**

| Class | Traffic characteristics |
|---|---|
| I | urban footbridges with high pedestrian density |
| II | urban footbridges, occasionally loaded on the entire surface |
| III | footbridges for normal use; occasionally crossed by large groups of pedestrians |
| IV | footbridges occasionally crossed |

The footbridge beneficiary can establish one of the threelevels of comfort for the bridge users shown in Table 4.

If the resonance risk is negligible, after the assessment of the natural frequency, the level of comfort is sufficiently implicit.

**Table 4. Levels of comfort**

| Comfort levels | Characteristics |
|---|---|
| Maximum comfort | structural acceleration is virtually imperceptible |
| Medium comfort | acceleration is slightly perceptible |
| Minimum comfort | acceleration is perceived by the users, but at a tolerable level |

## 3.2. Range of acceleration values associated with comfort level

Pedestrian comfort level correlates with the structure acceleration level determined through the calculation for different dynamic load cases.

Four conventional areas for vertical and horizontal accelerations are defined (Figure 1) in ascending order, corresponding to maximum, medium and minimum comfort levels, where field 4 corresponds to inadmissible acceleration values, [1].

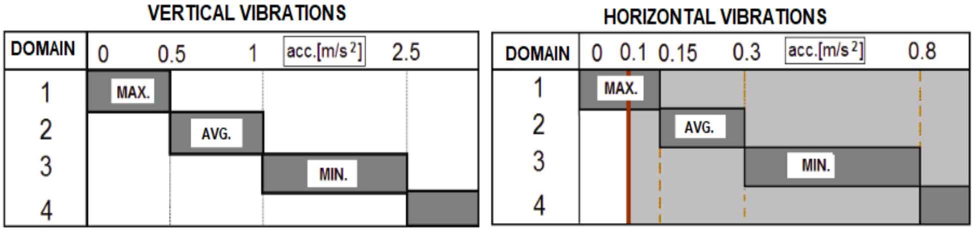


**Figure 1.**Acceleration ranges for vibrations

Horizontal acceleration is limited to 0.10m/s$^2$ to avoid the "lock-in" phenomenon.

## 3.3. Frequencies that require dynamic calculation

For footbridges that fall into classes I, II and III, it is necessary to calculate the natural vibration frequency of the structure. These frequencies are evaluated for the three directions: vertical, horizontal transverse and horizontal longitudinal.

Frequencies are determined for two mass assumptions of the system:

- unloaded footbridge;

- the bridge loaded on the walkway with 700N/m$^2$.

Depending on the area in which these frequencies are located, the risk of resonance caused by pedestrian traffic can be assessed, and the load cases for dynamic calculation can be determined and the comfort criterion can be checked.

Vertical and horizontal frequencies may fall into four areas of the resonance risk (Figure 2), [1], where:

Domain 1: maximum resonance risk;

Domain 2: average resonance risk;

Domain 3: low resonance risk;

Domain 4: negligible resonance risk.

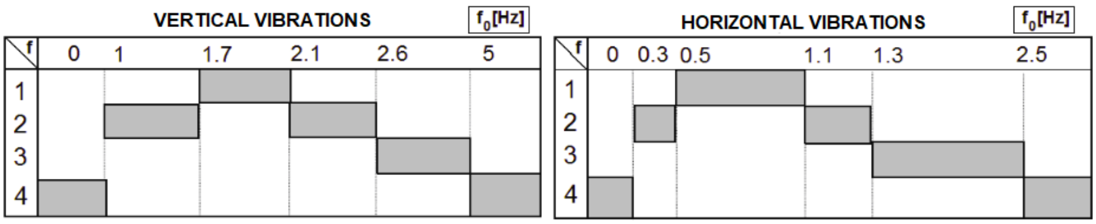


**Figure2.** Frequencyranges for vibrations

## 3.4. Dynamic loading cases

Depending on the bridge class and the range of the natural frequencies, the dynamic calculation for three load cases is required:

Case 1: moderate (scattered) and dense agglomeration;

Case 2: very dense agglomeration;

Case3: very dense agglomeration; the effect of the secondary harmonics is considered.

*Case 1* is considered for footbridges that fall within categories III and II, and density d of pedestrian crowding is considered as follows:Class III: d = 0.5 pedestrians/$m^2$; Class II: d = 0.8 pedestrians/$m^2$.

The load to be considered is altered by the factor ψ that considers that the resonance risk becomes less likely outside the range 1.7Hz ... 2.1Hz - for vertical vibrations and 0.5Hz ... 1.1Hz - for horizontal vibrations (Figure 3) [1].

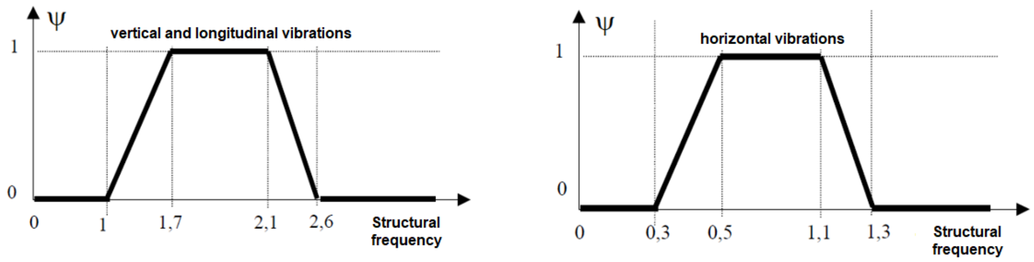


**Figure 3.**The reduction Factor ψ

Table 5 evaluates the unit surface loads ($m^2$) that are applied to each vibration direction.

**Table 5. Unit surface loads**

| Direction | Unit surface loads ($m^2$) |
|---|---|
| Vertical (v) | $d \times (280N) \times \cos(2\pi f_v t) \times 10.8 \times \sqrt{\xi / n} \times \psi$ |
| Longitudinal (l) | $d \times (140N) \times \cos(2\pi f_l t) \times 10.8 \times \sqrt{\xi / n} \times \psi$ |
| Transversal(t) | $d \times (35N) \times \cos(2\pi f_t t) \times 10.8 \times \sqrt{\xi / n} \times \psi$ |

Dynamic load *case 2* will only be considered for Class I footbridges. The density in this case is considered d = 1 pedestrian/$m^2$, the loading being evenly distributed on the surface S.

Dynamic load *case 3* is like cases 1 and 2, but the secondary harmonics is considered with a double frequency compared to the first harmonic.

For the simply supported beam with constant characteristics, the analytical calculation for the natural vibration modes is performed using the relationships in Table 6.

In practice, because the footbridges are narrow compared to their length and with a high torsional stiffness for closed sections, the torsional and axial stresses frequencies are high; therefore the analysis is performed only for bending vibrations (vertical and horizontal).

In papers [4], [5]and[6]several footbridges on steel beams and with a composite steel-concrete superstructure are presented and analyzed from a dynamic point of view.

In the synthesis book *Lightweight steel structures* [6], the dynamic response of footbridges using different constructive solutions is evaluated.

**Table 6. Analytical relations for the natural vibration modes**

| Mode | Natural pulse | Natural frequency | Form of the vibration mode |
|---|---|---|---|
| Simple bending with *n* half-wave | $\omega_n = \frac{n^2\pi^2}{L^2}\sqrt{\frac{EI}{\rho S}}$ | $f_n = \frac{n^2\pi}{2L^2}\sqrt{\frac{EI}{\rho S}}$ | $v_n(x) = \sin\left(\frac{n\pi x}{L}\right)$ |
| Tension compression with *n* half-wave | $\omega_n = \frac{n\pi}{L}\sqrt{\frac{ES_N}{\rho S}}$ | $f_n = \frac{n}{2L}\sqrt{\frac{ES_N}{\rho S}}$ | $u_n(x) = \sin\left(\frac{n\pi x}{L}\right)$ |
| Torsion with *n* half-wave | $\omega_n = \frac{n\pi}{L}\sqrt{\frac{GI_\omega}{\rho I_r}}$ | $f_n = \frac{n}{2L}\sqrt{\frac{GI_\omega}{\rho I_r}}$ | $\theta_n(x) = \sin\left(\frac{n\pi x}{L}\right)$ |
| Maximum acceleration | $Acc_{max} = \frac{1}{2\xi_n}\frac{4F}{\pi\rho S}$ (see [3], [7]) | | |
| **Measurement units:** L [m]; E=210x10$^9$ N/mm$^2$; I [m$^4$]; $\rho S$ [kg/m]; m [kg/m]. **Parameters:** $\rho S$ - the linear density of the construction (includes permanent and variable loading); $\rho I_r$ - moment of inertia at twist; $ES_N$ - stiffness at axial stresses; EI - flexural stiffness; $GI_\omega$ - torsion stiffness. | | | |

## 4. DYNAMIC RESPONSE OF A FOOTBRIDGE ON ARCHES

The dynamic response of the superstructure of a footbridge on a central arch is presented, and the solution is analyzed so that the dynamic parameters fall within the limits necessary to meet the safety conditions related to the resonance risk and to ensure the pedestrian traffic comfort [8].

### 4.1. The footbridge structure

The footbridge superstructure [8] has 3 spans (Figure 4), a central span of 130.0 m with the solution of a double hinged central arch. The arch cross-section is a variable box steel that supports the deck through several steel cable hangers.

The arch separates in two arches near the ends, thus ensuring the horizontal stability and allowing for an easier erection of the arch using smaller sections.

The deck is a steel box, with the lower flange with holes that ensures the maintenance and reduces the steel consumption.

The deck is suspended through hangers at a distance of 7.20m in between, anchored by the side cantilevers of the steel box. The deck is also thrust for the arch, under important bending and tensile stresses.

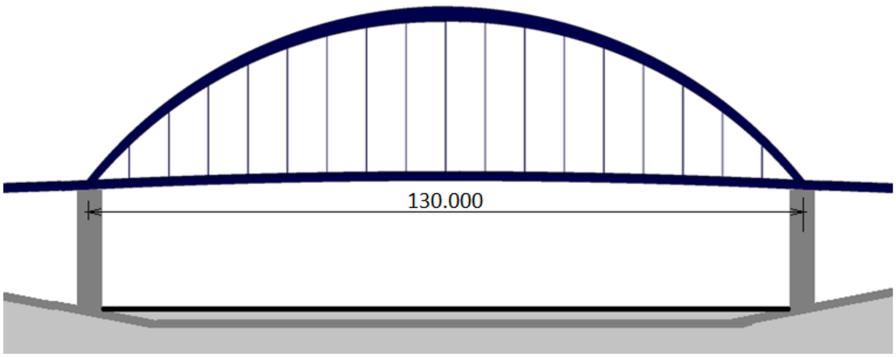


**Figure 4.**Footbridge longitudinal view

## 4.2. Vibration mode 1, in the vertical plane

The footbridge falls within Class II: urban footbridges, occasionally loaded on the entire surface. The deck cross-section analyzed in the first step is presented in Figure 5.

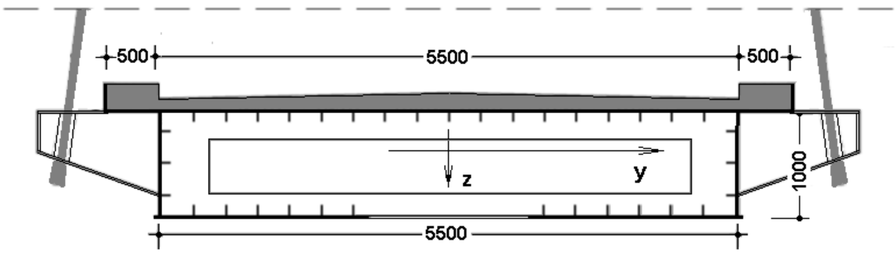


**Figure 5.**Deck cross-section

The permanent loading (box own weight, cantilevers, stiffeners, pedestrian guardrails, concrete slab) is of 4500 kg/m.

The walkway consists of a reinforced concrete slab than ensures the transverse slope and an upper layer of synthetic resins. The concrete slab ensures the steel box protection against rain or snow and allows for the pedestrian guardrails to be fixed on the sides and improves the dynamic behavior of the structure.

On the central span the concrete slab does not work together with the steel box, but some constructive connectors are provided.

*Deck characteristics*

The inertia moment for the steel box (including the longitudinal stiffeners) in relation to the horizontal axis y-y is: $I_y = 0.034m^4$;

The natural linear density, given by the box weight, the longitudinal and transverse stiffeners, the concrete slab, cantilevers and pedestrian guardrails: m = 4500 kg/m;

The elasticity modulus: $E = 210 \cdot 10^9 N/m^2$.

*Frequency for vibration Mode 1:*

The linear density of the deck, considering the pedestrian density (which for Class II is d= 0.8 P/m$^2$) is evaluated:

The number of pedestrians on the footbridge: $n = S \times d = (130 \times 5.50) \times 0.8 = 572P$.

Pedestrians total mass: 70 x 572 = 40 040 kg.

Pedestrians linear density: $m_P$ = m/L =40 040/130 = 308 kg/m.

Linear density:

- unloaded footbridge: $\rho S = 4500kg/m$;
-footbridge loaded with density d: $\rho S = 4500 + 308 = 4808kg/m$.

The frequencies for Vibration Mode 1:

-superior frequency: $f_1 = \frac{1^2\pi}{2 \times 7.2^2}\sqrt{\frac{210 \times 10^9 \times 0.034}{4500}} = 38Hz$

-inferior frequency: $f_1 = \frac{1^2\pi}{2 \times 7.2^2}\sqrt{\frac{210 \times 10^9 \times 0.034}{4808}} = 37Hz$

The frequencies for Vibration Mode 1 fall within Domain 4: negligible risk.

For this domain the dynamic calculation is not required, or the system acceleration calculation is not required ($\psi = 0$).

## 4.3. Vibration mode 1, in the horizontal plane

*The frequencies for Vibration* Mode 1:

The inertia moment in relation to the vertical axis z-z: $I_z$=0.694m$^4$.

The frequencies for Vibration Mode 1:

- superior frequency: $f_1 = \frac{1^2\pi}{2\times130^2}\sqrt{\frac{210\times10^9\times0.694}{4500}} = 0.52\text{Hz}$
- inferior frequency: $f_1 = \frac{1^2\pi}{2\times130^2}\sqrt{\frac{210\times10^9\times0.694}{4808}} = 0.51\text{Hz}$

The frequencies corresponding to Vibration Mode 1 fall within Domain 1: maximum resonance risk

## 4.4. Improving the dynamic behavior

If the limit acceleration criterion is not met, measures to improve dynamic behavior should be considered.

For a walkway in the design phase, the logical course of action is to change the natural vibration frequency so that it is not in the resonance risk area in relation to the excitement generated by pedestrian walking. This generally implies an increase in the damping factor of the structure.

For this structure, two ways to improve the dynamic behavior of the walkway were viable.

*The use of TMD (Tuned mass damper)*

These devices are composed of a mass connected to the construction by a viscous spring and a damping system connected in parallel; they are very effective when the excitation frequency resonates with the frequency of the structure.

The optimal frequency and the optimal damping factor of the damping device are given by the relationships [1], [2]:

$$f_{opt} = \frac{1}{1+\mu} f_s;\ \xi_{opt} = \sqrt{\frac{3}{8}\frac{\mu}{(1+\mu)^3}}\text{, where } \mu = \frac{m}{M} \qquad (2.a, b)$$

where: m – damper’s mass; M – the modal mass of the structure; $\mu = 0.03...0.05$.

The spring constant of the damping device and the damping coefficient of the device shall be determined with the relations:

$$k_d = (2\pi f_d)^2 m;\ d_d = 2m(2\pi f_d)\xi_{opt} \qquad (3.a, b)$$

TMD devices are installed after the completion of the structure and are matched exactly to the natural frequency of the structure, which can be determined by measurements.

Among the most important footbridges that used such devices are Millennium Bridge over Thames - London, North Shore Footbridge, Stockton on the Tees River [7](Figure 6).

**Figure 6.**Millennium Bridge (left), North Shore Footbridge (right)

*Increasing the deck stiffness in the horizontal plane*

Considering that the footbridge deck has lateral cantilevers that are used to suspend the deck to the arch through hangers, we verified if the dynamic parameters corresponding to the horizontal vibrations can be modified using longitudinal ties with a tubular cross-section at the ends of the cantilevers.

The ties work with the deck through a system of bracings, respectively by the development of lattice beams on the sides of the deck which contribute to the increase of the moment of inertia in relation to the z-z vertical axis.

The dynamic response of the new system is presented.

## 4.5. Vibration mode 1 in the horizontal plane – horizontally stiffened deck

*The deck characteristics*

The deck cross-section, with the new cantilever system is presented in Figure 7.

The inertia moment in relation to the horizontal axis z-z: $I_{zc}$=4.02m$^4$.

The weight of the structure (steel box weight, cantilevers, longitudinal ties, stiffeners, pedestrian guardrails, concrete deck) is of 5200 kg/m.

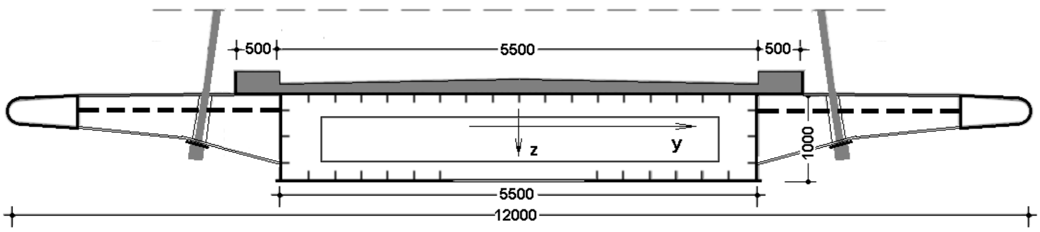


**Figure 7.** Deck cross-section, reinforced with longitudinal ties

*Vibration mode 1*

Linear density:

- unloaded footbridge: $\rho S = 5200\text{kg}/\text{m}$;
- footbridge loaded with density d: $\rho S = 5200 + 308 = 5508\text{kg}/\text{m}$.

*The frequencies for Vibration Mode 1*:

- superior frequency: $f_1 = \dfrac{1^2\pi}{2\times 130^2}\sqrt{\dfrac{210\times 10^9\times 4.02}{5200}} = 1.18\text{Hz}$
- inferior frequency: $f_1 = \dfrac{1^2\pi}{2\times 130^2}\sqrt{\dfrac{210\times 10^9\times 4.02}{5508}} = 1.15\text{Hz}$

The frequencies corresponding to Vibration Mode 1 fall within Domain 2: medium resonance risk.

*Dynamic loading*

The dynamic loading for Vibration Mode 1 is evaluated, for a damping ratio $\xi_n = 0.4\% = 0.4\times 10^{-2}$ (steel structure). The coefficient $\psi \approx 1.2$, (Figure 4).

It is obtained:

$F_s = d\times(35N)\times\cos(2\pi f_t t)\times 10.8\times\sqrt{\xi/n}\times\psi = 0.96\times\cos(7.22\pi t)\ [N/m^2]$

Load per unit length: $F = 5.5m\times F_s = 5.28\times\cos(3.9\pi t)[N/m]$.

*Dynamic response*

The system acceleration:

$$Acc_{max} = \frac{1}{2\xi_n}\frac{4F}{\pi\rho S} = \frac{1}{2\times 0.4\times 10^{-2}}\frac{4\times 5.28}{\pi\times 5508} = 0.15m/s^2$$

*The system acceleration falls within Domain 2, average comfort level.*

In paper [7], at the frequency calculation, the authors take into account the type of supports of the deck using equation 4:

$$f_1 = \frac{C^2}{2\pi L^2}\sqrt{\frac{EI}{\rho S}} \tag{4}$$

For the continuous beam we use C=3.55, therefor the frequencies are increased with the value $f_1 = \dfrac{C^2}{\pi^2} = 1.276$:

- superior frequency: $f_1 = \dfrac{3.55^2}{2\pi 130^2}\sqrt{\dfrac{210\times 10^9\times 4.02}{5200}} = 1.50\text{Hz}$
- inferior frequency: $f_1 = \dfrac{3.55^2}{2\pi 130^2}\sqrt{\dfrac{210\times 10^9\times 4.02}{5508}} = 1.47\text{Hz}$

The frequencies corresponding to Vibration Mode 1 in the horizontal plane fall within Domain 3: low resonance risk and the coefficient $\psi = 0$, therefor the acceleration evaluation is not necessary.

## 5. CONCLUSIONS

In the case of footbridges, in addition to checks for the ultimate limit state and service limit states, it is necessary to check pedestrian traffic comfort in direct correlation with the vibration frequency of the structure (resonance risk) and its acceleration. If these characteristics (frequency and acceleration) are in the critical area, measures must be taken to alter their value so that they fall within the limits of recommended norms or other recognized technical materials.

**Table 7. Superior frequencies and the resonance risk**

| The deck | Vibration mode | Frequency | Freq [Hz] | Resonance risk. Observations |
|---|---|---|---|---|
| Original unstiffened solution | Vibration mode 1 in the vertical plane | Superior frequency. Unloaded footbridge | 38 | Domain 4: Negligible resonance risk |
| | | Superior frequency. Partially loaded footbridge | 37 | |
| | Vibration mode 1 in the horizontal plane | Superior frequency. Unloaded footbridge | 0.52 | Domain 1: Maximum resonance risk |
| | | Superior frequency. Partially loaded footbridge | 0.51 | |
| Stiffened deck | Vibration mode 1 in the horizontal plane Interrupted deck at the intermediate supports | Superior frequency. Unloaded footbridge | 1.18 | Domain 2: Medium resonance risk. The acceleration corresponds to a medium comfort level |
| | | Superior frequency. Partially loaded footbridge | 1.15 | |
| | Vibration mode 1 in the horizontal plane. Continuous deck over the intermediate supports | Superior frequency. Unloaded footbridge | 1.50 | Domain 3: Low resonance risk |
| | | Superior frequency. Partially loaded footbridge | 1.47 | |

Table 7 centralizes the superior frequencies and the resonance risk for the two analyzed decks, for the case of unloaded structure and the case of partially loaded structure.

The analysis of the dynamic response of the structure of a footbridge on arches and the solution analyzed for the dynamic parameters to fall within the limits necessary for the satisfaction of the safety conditions, can be useful in the design of the structures with large spans.

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