EUROPEAN DESIGN GUIDE FOR FOOTBRIDGE VIBRATION

Christoph HEINEMEYER
Doctor Engineer
RWTH Aachen University
Aachen, Germany

Markus FELDMANN
Professor
RWTH Aachen University
Aachen, Germany

Summary

Increasing vibration problems encountered in the last few years show that footbridges should no longer be designed for static loads only but also for the dynamic actions and vibration behaviour of the footbridge due to pedestrian loading.

For this reason European research has been performed to come up with a design concept for footbridges that takes into account different traffic situations and individual demands on vibration comfort. So the elaborated guideline considers different types of pedestrian traffic and the traffic density which can greatly influence comfort requirements of the bridge and the dynamic behaviour. It is important to predict the effect of pedestrian traffic on footbridges at the design stage and in the later verification of serviceability in order to guarantee a comfort level for the user. This guide gives recommendations for designer and client to find relevant design situations as well as methods how to prove if vibration requirements are fulfilled.

This paper presents the design guide methodology and gives some background information.

Keywords: Footbridge; dynamics; structural concepts; planning; vibration; design guide; damping; comfort.

1. Introduction

Vibrations of footbridges are an issue of increasing importance in current design practice. More sophisticated bridge types like cable supported footbridges or stress ribbon bridges, increasing spans and more effective construction materials result in lightweight structures, which have a high ratio of live load to dead load. As a result of this trend, many footbridges have become more susceptible to vibrations when subjected to dynamic loads. Besides wind loading the most common dynamic loads on footbridges are the pedestrian induced footfall forces due to walking or jogging.

As a consequence of these lightweight structures, the decrease in stiffness leads to lower natural frequencies with a greater risk of resonance, while the decrease in mass reduces the mass inertia. Resonance occurs if the frequency of the bridge coincides with the frequency of the excitation, e.g. the step frequency of pedestrians. Due to the small mass of slender lightweight structures the mass inertia is much lower and hence the dynamic forces can cause larger amplitudes of response. The more slender constructions become, the more attention must be paid to vibration phenomena.

Pedestrian induced excitation is an important source of dynamic excitation on footbridges. The caused vibrations may occur in vertical and horizontal direction, even torsion of the deck is possible.

Vibrations of footbridges may lead to serviceability problems, as effects on the comfort and emotional reactions of pedestrians might occur, although collapse or even damage due to human induced dynamic forces has occurred very rarely.

In recent years some footbridges were excited laterally by dense pedestrian streams in which pedestrians interacted with the bridge vibration. A self-excited large response was produced and caused discomfort. Footbridges should be designed in such a way that this pedestrian-bridge-interaction phenomenon, also called ‘lock-in’, does not arise.

In Building codes ([2],[3] ,[4] ,[5]) this dynamic problem is considered by giving limits for the natural frequency. This rough assumption restricts pedestrian bridge design. E.g. Slender, lightweight bridges, such as stress ribbon bridges and suspension bridges may not satisfy these requirements.

The presented design according to the guideline elaborated within the SYNPEX project [1] takes into account that pedestrian bridges have different traffic situations that may be more or less relevant for design. One exceptional situation is e.g. the inauguration of a bridge with a very dense traffic that occurs often once in the life of a bridge only.

Thus this guideline gives help to find together with the client relevant traffic situations and to define a related comfort that should be fulfilled under that traffic situation. Traffic class and related comfort criteria are the goal of the bridge design.

The guideline also gives methods how to determine the relevant dynamic bridge characteristics and the bridge acceleration under pedestrian traffic.

This paper concentrates on the design procedure. Further information may be found in the SYNPEX final report [1].

2. Design Procedure

The design method aims on the proof of comfort for vertical and horizontal vibration. It does not aim in design for structural integrity or fatigue.

By tests and surveys of pedestrians who have just passed a bridge - performed within the SYNPEX project [1] - it has been found that a general definition of comfort is not reasonable but individual definition of comfort criteria should be applied. Thus the specification of design scenarios is the first design step in the flow chart illustrated in Fig. 1. The flow chart also contains the links to the relevant chapters of this paper which include further descriptions.

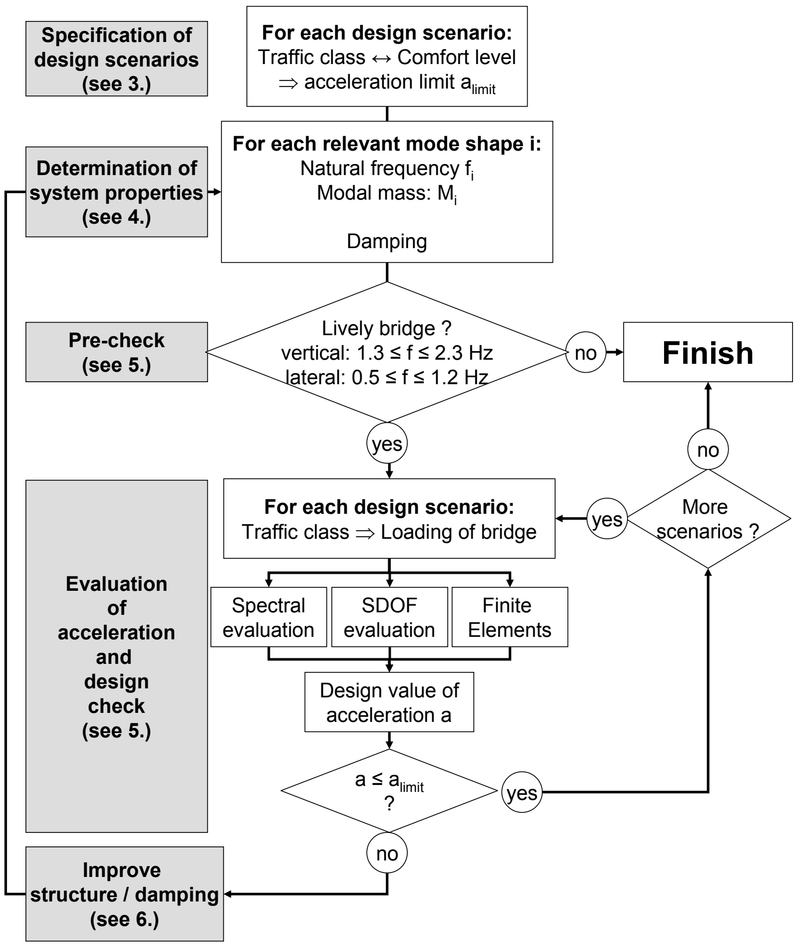


Fig. 1 Procedure for vibrations design

Another key point in the guideline is the evaluation of expected acceleration of the bridge. Three alternative methods – spectral method, Finite Element Analysis and SDOF method (single degree of freedom) – are presented in chapter 5 and some general information about the determination of system properties are give in chapter 3.

3. Specification of Design Scenarios

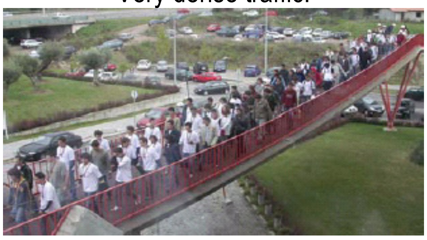
3.1 Traffic Classes

The expected pedestrian traffic on footbridges depends on various boundary conditions as e.g. location of the bridge:

- in parks or on the country site one expects few people to promenade,
- in a city centre one expects a more persistent weak or dense stream of people and
- close to a exhibition hall or a stadium people may cross in intervals and dense or very dense streams.

Table 1 gives traffic classes (TC) with appropriate densities of persons , illustrations and descriptions.

Table 1. Traffic classes

Traffic Class	Density d (P = Person)	Description	Characteristics
TC 1	group of 15 P; d=15P/bl	Very weak traffic	15 single persons (b=width of deck; l=length of deck)
TC 2	d= 0.2 P/m²	Weak traffic: 	Comfortable and free walking, Overtaking is possible, Single pedestrians can freely choose pace
TC 3	d= 0.5 P/m²	Dense traffic: 	Significantly dense traffic, Unrestricted walking, Overtaking can intermittently inhibit.
TC 4	d= 1.0 P/m²	Very dense traffic: 	Freedom of movement is restricted. Uncomfortable situation, obstructed walking, Overtaking is no longer possible.
TC 5	d= 1.5 P/m²	Exceptional dense traffic	Very dense traffic and unpleasant walking. Crowding begins, one can no longer freely choose pace.

3.2 Comfort levels

The assessment of the horizontal and vertical footbridge vibration includes many ‘soft’ aspects such as:

- Number of people walking on the bridge,
- Frequency of use,
- Height above ground,
- Position of human body (Sitting, standing, walking),
- Harmonic or transient excitation characteristics (vibration frequency),

- Exposure time,
- Transparency of the deck pavement and the railing and
- Expectancy of vibration due to bridge appearance.

The comfort levels for different acceleration ranges of the bridge recommended by the guideline are presented in Table 2. In general there are four comfort levels: maximum comfort, medium comfort, minimum comfort and unacceptable discomfort.

Table 2. Defined comfort classes with limit acceleration ranges

Comfort level	Degree of comfort	Acceleration level vertical	Acceleration level horizontal a_{limit}
CL 1	Maximum	< 0.50 m/s²	< 0.10 m/s²
CL 2	Medium	0.50 – 1.00 m/s²	0.10 – 0.30 m/s²
CL 3	Minimum	1.00 – 2.50 m/s²	0.30 – 0.80 m/s²
CL 4	Unacceptable discomfort	> 2.50 m/s²	> 0.80 m/s²

It should be clear that passing the bridge is possible for the three acceptable comfort levels C1 to C3.

3.3 Specification Matrix

As stated above comfort is a soft aspect. For assigning comfort levels to traffic classes the following should be considered:

- Very slender bridges may not be feasible when specifications are too severe – Maximum comfort may not be reached, but depending on further boundary conditions as type of users and location a less restrictive traffic class may be acceptable in design.
- The occurrence of traffic: For unusual traffic situations a minimum comfort may be sufficient; For frequent traffic situations a higher comfort class may be adequate.
- The location of the bridge: Close to hospitals and nursing homes where people pass who are made feeling insecure by vibration a high comfort level might be applicable; close to a stadium or exhibition hall medium comfort might may be demanded; in a forest where only few hiker pass the bridge a minimum comfort should be enough.

The assignment of desired comfort levels to traffic classes should be performed together with the client. It is proposed to use a specification matrix as shown in Table 3. Table 3 is exemplarily filled in for the most frequent situation of footbridges where exceptional dense traffic is not expected and critical persons are exceptional users of the bridges.

Table 3: Specification Matrix

Comfort Level	Traffic class				
	TC 1 Very weak	TC 2 Weak	TC 3 Dense	TC 4 Very dense	TC 5 Exceptional dense
CL 1: Maximum	0	0	0	0	-
CL 2: Medium	X	X	X	0	-
CL 3: Minimum				X	-

Legend:
- : Not expected
0 : Not demanded
X : Demand

3.4 Lateral Lock-in

Unlike vertical vibrations which are absorbed by legs and joints so that pedestrian streams synchronising with vertical vibrations have not been observed on footbridges, people are much more sensible to lateral vibrations. As for walking the centre of gravity is not only varied vertically but also laterally from one foot to the other, the frequency of the movement of the human centre of gravity is half the walking frequency. If a person walks on a laterally vibrating bridge, he tries to compensate the additional movement of his centre of gravity by swaying with the bridge displacement for lateral stability. This behaviour is intuitive and even small and not perceptible vibrations are assumed to cause an adjustment of the movement of the centre of gravity. This change of the movement of the centre of gravity is accompanied by an adaptation of the walking frequency and a widening of the gait. The person tends to walk with twice the vibration frequency to move his centre of gravity in time with the vibration [9].

The swaying of the body in time with the lateral vibration causes that the lateral ground reaction forces are applied in resonance and the widening of the gait causes an increase in the lateral ground reaction forces. The forces are applied in such a way that they introduce positive energy into the structural system of the bridge (Fig. 2). Hence, if a footbridge vibrates slightly in lateral direction and it happens that the pedestrian adjust their walking pattern, then due to this synchronisation effect a low-damped bridge can be excited to large vibrations.

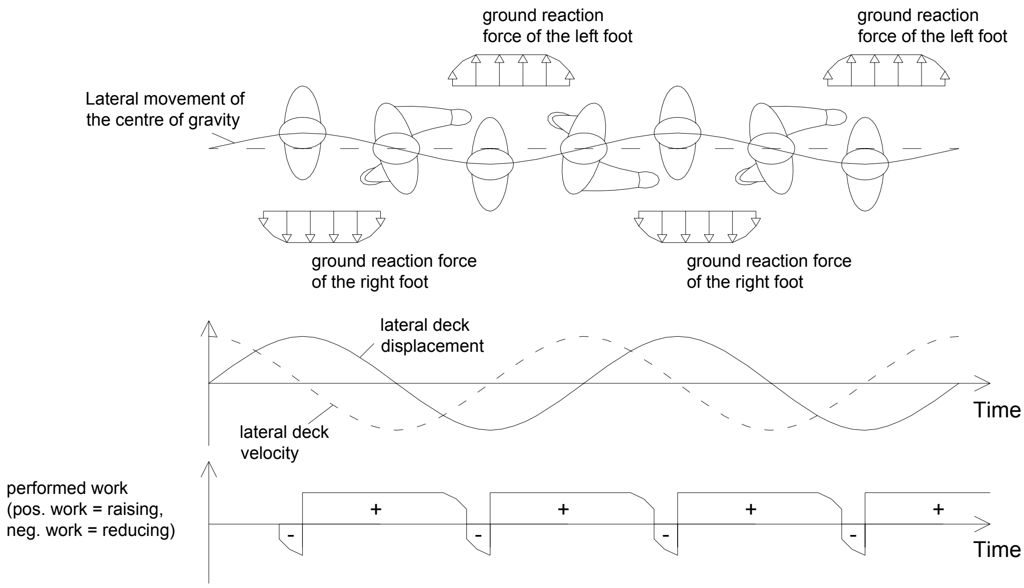


Fig. 2 Schematic description of synchronous walking

Tests in France [10] on a test rig and on the Passerelle Solferino indicate that a trigger amplitude of 0,1 to 0,15 m/s² exist when the lock-in phenomenon begins.

$$a_{lock-in} = 0.1 \text{ to } 0.15 \ m/s^2 \qquad (1)$$

Experiments within the project SYNPEX [1] on a test rig indicate that single persons walking with a step frequency f_i +/- 0,2 Hz tend to synchronise with deck vibration. Fast walking persons are nearly not affected by the vibration of the platform, as the contact time of the feet is short and the walking speed very high. They seem to be less instable than when walking with slow and normal speed.

The lock-in trigger amplitude is expressed in terms of acceleration. Further frequency dependence could exist but has not been detected in measurements.

The definition of comfort classes for horizontal vibration and their determination consider the effects described above.

4. Determination of System Properties

The determination of the properties of a footbridge depends on the design stage and on the type of structural system.

In early design stages (e.g. pre-design) it may be adequate to apply hand formulas - as given in Table 9 for a simply supported beam - if the structure is not too complex.

In later design stages and for complex structures Finite Element Analysis (FEA) is usually applied. The FEA can be performed using beam and/or shell elements. Applying FEA it should be considered if small or large deformations are expected as this has an influence on the structural model. If small deformations are expected hinged connections (as assumed in Ultimate Limit State ULS) may act more like rigid connections. Thus the structural system for dynamic investigations may differ from that for the ULS design.

Independently from the calculation method damping properties of the structure need to be defined. For the design of footbridges for comfort level Table 4 recommends minimum and average damping ratios. Comparable values are proposed by the SETRA/AFGC guideline [11] and by Bachmann and Amman [12].

Table 4: Damping ratios ξ according to construction material [11],[12]

Construction type	Minimum ξ	Average ξ
Reinforced concrete	0.80%	1.3%
Prestressed concrete	0.5%	1.0%
Composite steel-concrete	0.30%	0.60%
Steel	0.20%	0.40%
Timber	1.50%	3.0%

Minimum values may be applied for short span bridges (e.g. < 20m) in other cases the average value may be appropriate.

5. Evaluation of acceleration

The guideline gives three alternative methods for the evaluation of acceleration:

- Spectral approach: This method has been developed for the analysis of beam bridges. It gives results with minimum calculation effort and is thus very suitable for preliminary design studies.
- SDOF-Method: The SDOF-Method reduces the structural system in regarded mode shape to a single degree of freedom system that can be examined easily.
- FEA-Method: With finite elements the most detailed investigation is possible. If the evaluation method presented here can not be applied with FE-program at hand one of the two other methods should be used.

Before starting the evaluation of acceleration it should be checked if it is likely that the footbridge vibrates due to pedestrian traffic. The bridge is lively and acceleration check should be performed if the natural frequency f_i of the regarded mode shape is in the following range:

- For vertical vibration: $1.3 \leq f_i \leq 2.3$ Hz
- For horizontal vibration: $0.5 \leq f_i \leq 1.2$ Hz

The acceleration needs to be determined and checked against the acceleration limit of the appropriate comfort level (Table 2) for each design scenario.

5.1 Load Models

5.1.1 General

For the spectral evaluation of acceleration the loading of the bridge is sufficiently defined by the crowd density in P/m² (Persons per square metre). The effects mentioned below have been considered in development of the spectral method.

In the two other evaluation methods the pedestrian induced action is represented as oscillating distributed load *p(t)*. This load is applied in accordance with the mode shape as shown in Fig. 3.

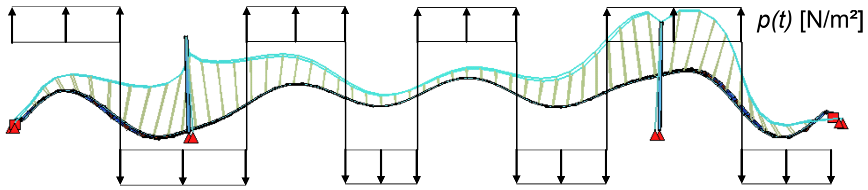


Fig. 3 Application of harmonic load according to a mode shape configuration

The harmonic oscillating load is defined by:

$$p(t) = G \times \cos(2\pi\, ft) \times n' \times \psi \qquad (2)$$

where:

- $G \times \cos(2\pi ft)$ is the harmonic load due to a single pedestrian,
  - G is the considered weight of a person,
  - f is the natural frequency under consideration,
- n' is the equivalent number of persons (pedestrians or joggers) on the loaded surface *S*,
- S is the loaded surface (according to some approaches [13] it depends on the shape of the normal mode under consideration, according to others [14] the whole 'loadable' surface should be considered),
- ψ is the reduction coefficient to take into account the probability that the footfall frequency approaches the natural frequency under consideration. This coefficient is different for each of the load models given below.

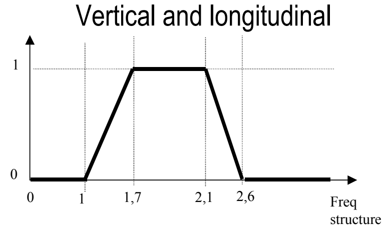
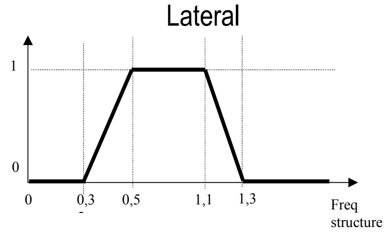
The load value depends on the crowd density and on the walking behaviour. When the density is less than 1 P/m² people are not interacting. If the density increases free walking is no longer possible an the interaction of walking people should be considered. Two groups of load models in the guideline are presented here additionally the guideline provides load models for single pedestrians and for joggers.

5.1.2 Load Model for TC1 to TC3 (density d<1.0 P/m²)

A uniformly distributed harmonic load *p(t)* represents the equivalent pedestrian stream for further calculations. The load model for pedestrian groups takes into account a free movement of the pedestrians. And so the synchronization among the group members is equal to a low density stream

Static force of a single pedestrian *G*, equivalent number of pedestrians *n'* (95 % percentile) and reduction coefficient ψ are given in Table 5 [14].

Table 5. Parameters for Load models of TC1 to TC3

G [N]			***n'* [1/m²]**
Vertical	**Longitudinal**	**Lateral**	
280	140	35	$\frac{10.8\sqrt{\xi \times n}}{S}$
Reduction coefficient ψ			
Vertical and longitudinal		Lateral	
			

where:

- ξ is the structural damping and
- n is the number of the pedestrians on the loaded surface *S* ($n = S \times$ density).

5.1.3 Load Model for TC 4 and TC 5 (density d≥1.0 P/m²)

In the case of a heavy congestion, walking is obstructed: the moving forward is slow and the synchronisation increases. Beyond the upper limit values for the dense stream (up to 1.2 or 1.5 or even 2 [15] Persons/m²), walking becomes impossible therefore significantly reducing dynamic effects. When a stream becomes dense, the correlation between pedestrians increases, but the dynamic load tends to decrease.

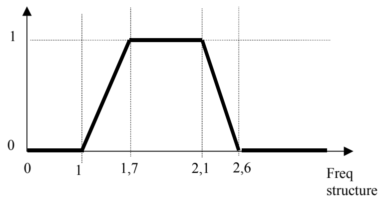
The amplitude of the dynamic force of a single pedestrian *G*, equivalent number of pedestrians *n'* (95 % percentile) and reduction coefficient ψ are given in Table 6 [14].

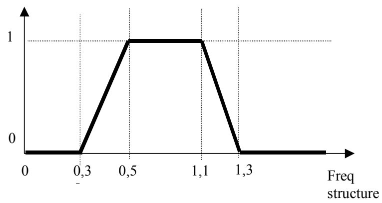
Table 6. Parameters for Load model of TC4 and TC5

G **[N]**			***n'*** **[1/m²]**
Vertical	**Longitudinal**	**Lateral**	
280	140	35	$1.0 \times 1.85\sqrt{n}$

Reduction coefficient ψ

Vertical and longitudinal	Lateral





where:

n is the number of the pedestrians on the loaded surface *S* ($n = S \times$ density).

5.2 Evaluation Methods

5.2.1 Spectral Evaluation

The spectral evaluation method [16] is the result of an extensive study of the system response to various pedestrian streams. The aim of the development of a spectral design model was to find in a simple way a descritpion of the stochastic loading and system response that gives design values with a specific confidence level. The general design procedure is adopted from wind engineering where it is used to verify the effect of gusts on sway systems.

As design value the system response "maximum peak acceleration" was chosen. In the design check this acceleration is compared with the tolerable acceleration according to the comfort level to be verified.

This maximum acceleration is defined by the product of a peak factor k_a and a derivation of acceleration σ_a .

Both factors have been derived from Monte Carlo simulations which are based on numerical time step simulations of various pedestrian streams on various bridges geometries.

The bases of the variance of acceleration are the stochastic loads. To determine these loads bridges with spans in the range of 20 m to 200 m and a varying width of 3 m and 5 m with four different stream densities (0.2 Pers/m², 0.5 Pers/m², 1.0 Pers/m² and 1.5 Pers/m²) have been investigated. For each bridge type and stream density 5000 different pedestrian streams have been simulated in time step calculations where each pedestrian had the following properties which are taken randomly from the specific statistical distribution:

- Persons weight (mean = 74,4 kg; standard deviation = 13 kg),
- Step frequency (mean value and standard deviation depend on stream density),
- Factor for lateral foot fall forces (mean = 0,0378, standard deviation = 0,0144),
- Start position (randomly) and
- Moment of first step (randomly).

The result is the characteristic acceleration, which is the 95 % fractile of the maximum acceleration. For different pedestrian densities it can be determined according to the following formulas and tables while it is assumed that

- the mean step frequency of the pedestrian stream coincides with the considered natural frequency of the bridge,
- the mass of the bridge is uniformly distributed,
- the mode shapes are sinusoidal,
- no modal coupling exists,
- the structural behaviour is linear-elastic.
- for vertical vibrations $f_{s,5\%,slow} = 1{,}25\ \text{Hz} \leq f_i \leq 2{,}3\ \text{Hz} = f_{s,95\%,fast}$
- for lateral vibrations $0{,}5\ \text{Hz} \leq f_i \leq 1{,}2\ \text{Hz} = f_{s,95\%,fast/2}$

Note: 0,5 Hz is chosen because that natural frequency was excited during the inauguration of the Millennium Bridge.

The following empirical expression for the determination of the variance of the response is recommended:

$$a_{\max} = a_{\max,95\%} = k_{a,95\%} \frac{d \cdot l \cdot b}{M_i} \sqrt{C \cdot k_f^2 k_1 \xi^{k2}} \qquad (3)$$

Where:

k1	:	constant $k_1 = a_1 f_i^2 + a_2 f_i + a_3$
k2	:	constant $k_2 = b_1 f_i^2 + b_2 f_i + b_3$

a_1, a_2, a_3 : constants, see Table 7 for vertical and Table 8 for lateral vibrations

b_1, b_2, b_3 : constants, see Table 7 for vertical and Table 8 for lateral vibrations

$n = d \cdot l \cdot b$: number of persons on the bridge, d: pedestrian density, l: bridge length, b: bridge width

k_F : constant

σ_F^2 : variance of the loading (pedestrian induced forces)

f_i : considered natural frequency that coincides with the mean step frequency of the pedestrian stream

M_i : modal mass of the considered mode i

ξ : damping ratio

C : constant describing the maximum of the load spectrum

σ_a^2 : variance of the acceleration response

$k_{a,94\%}$: peak factor to transform the standard deviation of the response σ_a to the characteristic value $a_{max,95\%}$

The constants a_1 to a_3, b_1 to b_3, C, k_F and $k_{a,95\%}$ can be found in Table 7 for vertical accelerations and in Table 8 for lateral accelerations

Table 7. Constants for vertical accelerations

d [P/m²]	k_F	C	a_1	a_2	a_3	b_1	b_2	b_3	$k_{a,95\%}$
≤ 0.5	1.2 · 10-2	2.95	-0.07	0.6	0.075	0.003	-0.04	-1	3.92
1.0	7 · 10-3	3.7	-0.07	0.56	0.084	0.004	-0.045	-1	3.80
1.5	3.335 · 10-3	5.1	-0.08	0.5	0.085	0.005	-0.06	-1.005	3.74

Table 8. Constants for lateral accelerations

d [P/m²]	k_F	C	a_1	a_2	a_3	b_1	b_2	b_3	$k_{a,95\%}$
≤ 0.5		6.8	-0.08	0.5	0.085	0.005	-0.06	-1.005	3.77
1.0	2.85 · 10-4	7.9	-0.08	0.44	0.096	0.007	-0.071	-1	3.73
1.5		12.6	-0.07	0.31	0.12	0.009	-0.094	-1.02	3.63

5.3 SDOF Evaluation

The dynamic behaviour of a structure may be evaluated by modal analysis. Thereby, an arbitrary oscillation of the structure is described by a combination of n different harmonic oscillations with different frequencies for each mode shape *i*. In doing so, the structure is divided into n different spring mass oscillators, each with a single degree of freedom (SDOF). Each SDOF oscillator has a modal mass M_i, a modal stiffness K_i and a modal load P_i. The equivalent spring mass system is found with the method of generalization, Fig. 4.

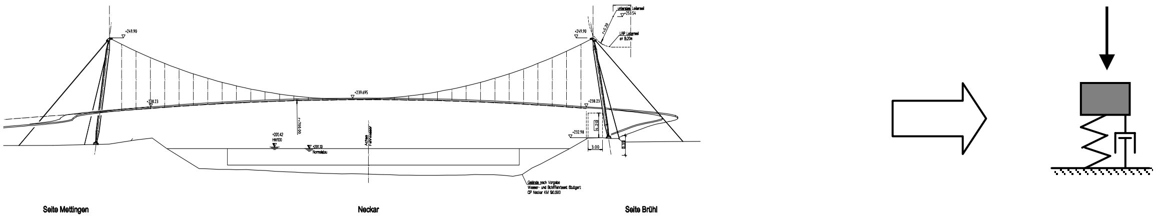


Fig. 4 One equivalent SDOF oscillator

The basic idea is to use one single equivalent SDOF system for each natural frequency *i* of the footbridge in the critical range and to calculate the associated maximum acceleration for a dynamic loading. The maximum acceleration a_{max} in the resonant for the SDOF is calculated by:

$$a_{\max} = \frac{P_i \cdot \pi}{M_i \cdot \delta} = \frac{P_i}{M_i \cdot 2\xi} \qquad (4)$$

As a simple example, a single span beam, Fig. 5, is considered. This beam has a distributed mass μ [kg/m] a stiffness k and a length l. The uniform load $p(x) \cdot sin(\omega t)$ is distributed over the total length. The mode shapes $\phi(x)$ of the bending modes are assumed to be a half sin distribution $\phi(x) = sin(m*x/l*\pi)$ whereas m is the number of half waves which is here equal to the number of mode shape *i*. The load oscillates with sin (ωt).

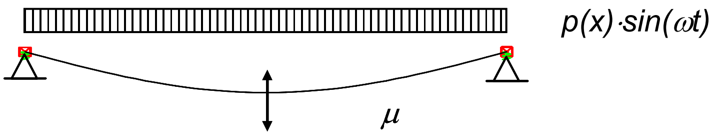


Fig. 5 Simple beam with harmonic mode shape $\phi(x)$, i=1

The generalized mass M_i and generalized load P_i sin(ωt) of the SDOF system are calculated for a single span beam with a harmonic uniform load p·sin (ωt) according to Table 9. The generalized load for a single load P_{mov} ·sin (ωt), moving across the simple beam is also given in Table 9. This excitation is limited by the tuning time which is defined as the time for the moving load to cross one belly of the mode shape.

Table 9. Generalized mass and generalized load

Mode shape	Generalized mass M_i	Generalized load P_i for uniformly load p(x) P_i	Generalized P_i load for moving load P_{mov} P_i	Tuning time t_{max}
i=1: $\Phi(x) = \sin\left(\frac{x}{l}\pi\right)$	$\frac{1}{2}\mu l$	$\frac{2}{\pi}p(x)\,l$	$\frac{2}{\pi}P_{mov}$	l/v
i=2: $\Phi(x) = \sin\left(\frac{2x}{l}\pi\right)$	$\frac{1}{2}\mu l$	$\frac{1}{\pi}p(x)\,l$	$\frac{2}{\pi}P_{mov}$	l/(2v)
i=3: $\Phi(x) = \sin\left(\frac{3x}{l}\pi\right)$	$\frac{1}{2}\mu l$	$\frac{2}{3\pi}p(x)\,l$	$\frac{2}{\pi}P_{mov}$	l/(3v)

Where:
- P_{mov} is the moving load in kN
- p(x) is the distributed load in kN/m
- μ is the mass distribution per length
- *l* is the length of the bridge
- i is the mode shape number (half wave)
- v is the velocity of the moving load

The 2nd mode shape i = 2 of a single span beam has two half waves. Loading the entire length, the generalized load will be calculated to a value of P_i = 0, when half of the uniformly distributed load is acting against the displacements of one belly and the other half is acting within the direction of displacements. The generalized load according to Table 9 is based on the assumption that only one belly of the mode shape is loaded, which results in larger oscillations. A more conservative approach to distribute the loading on the whole beam is favored by the SETRA/AFGC guidelines. There it is recommended to take the total length of the beam into account independent from the mode shape. Then, the load is always acting in the direction of displacements of the bellies and the generalized load P_i for all mode shapes is the same as for the first bending mode (i=1).

5.4 Finite Element Evaluation

Nowadays, even conceptual design of footbridges takes advantage of using the finite element method (FEM). Hence, preliminary dynamic calculations may easily be performed without additional effort. A simple approach to perform the static and dynamic calculation is by modeling the bridge deck by beam elements and the cable with cable elements, spring or truss elements in a three dimensional FEM model. A rough overview of the natural frequencies and the appropriate mode shapes is obtained and possible problems in dynamic behavior can be identified.

The non load bearing parts such as furniture and railings are considered as additional masses as exactly as possible. A more refined model may take advantage of various types of finite elements such as plate, shell, beam, cable or truss elements. The more complex the static system and the higher the mode shape, the more finite elements are required. The model should always allow for possibly vertical, horizontal, and torsional mode shapes.

To get reliable results for natural frequencies, it is absolutely necessary that stiffness and mass distribution are modeled in a realistic way. All dead load, superimposed dead load and pre-stressing of cables have to be considered for the calculation of natural frequencies. A lumped mass approach, in which rotational masses are neglected, is in many cases sufficient. The modal mass regarding to each mode shape should be available, when verification of comfort is done with simple approaches by hand calculation.

Many parameters such as properties of materials, complexity of the structure, the type of deck surfacing and furniture, boundary conditions and railings may cause discrepancies in natural frequencies between the results of computer calculations and the measured data of the real structure.

A numerical modeling should be as realistic as possible with regard to bearing conditions and their foundation stiffness. For the modeling of abutments and foundations, dynamic soil stiffness should be used. Otherwise the obtained results will be very inaccurate.

6. Improving the Structure

If the dynamic response of the structure under a specified traffic load does not fulfill the comfort requirements as specified it is necessary to improve the structure. In general there are three means to do this:

- Modification of model mass
- Modification of natural frequency
- Installation of additional damping devices

Modal mass and natural frequency can be modified at the design stage only. For an already constructed bridge, the simplest approach is based on the increase of the structural damping, which can be achieved either by implementation of control devices, or by actuation on non-structural finishings, like the hand-rail and surfacing.

6.1 Modification of Model Mass

For very light footbridges e.g. , the use of heavy concrete deck slabs can improve dynamic response to pedestrian loads, as consequence of the increased modal mass.

On the basis of the spectral design model, Butz[16] developed (see 5.2.1) also an empirical expression for the determination of a required modal mass for a given pedestrian traffic to ensure a required comfort a_{limit} that is valid for *mean pace frequency* $f_{s,m}$ *= Natural frequency i of the bridge* f_i .

$$M_i \geq \frac{\sqrt{n}\left(k_1\,\xi^{k_2} + 1.65\,k_3\,\xi^{k_4}\right)}{a_{limit}} \qquad (5)$$

where M_i modal mass for considered mode i

n number of pedestrians on the bridge

ξ damping coefficient

k_1 *to* k_4 constants (see

Table *10* and

Table *11*)

Table 10. Constants for required vertical modal mass (vertical bending and torsion modes)

d [P/m²]	k1	k2	k3	k4
≤ 0,5	0,7603		0,050	
1,0	0,570	0,468	0,040	0,675
1,5	0,400		0,035	

Table 11. Constants for the required lateral modal mass (horizontal bending modes)

d [P/m²]	k1	k2	k3	k4
≤ 0,5				
1,0	0,1205	0,45	0,012	0,6405
1,5				

6.2 Modification of natural frequency

The classical proposal of modification of structural properties in order to avoid natural frequencies in the critical range for vertical and lateral vibrations does not meet at the current state-of-art the goal of bridge designers to build light and graceful structures. In effect, given the proportionality of natural frequencies to the square root of the ratio between stiffness and mass, it is understandable that considerable modifications are required in the stiffness in order to attain a slight increase of natural frequency. However, it is of interest to consider during the design stage several simple strategies that can improve the dynamic behaviour and that are normally associated with an increase of critical natural frequencies. These comprehend, for example, the replacement of a reinforced concrete deck slab formed by non-continuous panels by a continuous slab, or the inclusion of the handrail as a structural element, participating to the overall deck stiffness.

Other more complex measures can be of interest, like the addition of a stabilizing cable system. For vertical vibrations, alternatives are the increase of depth of steel box girders, the increase of the thickness of the lower flange of composite girders, or the increase of depth of truss girders. For lateral vibrations, the most efficient measure is to increase the deck width. In cable structures, the positioning of the cables laterally to the deck increases the lateral stiffness. In cable-stayed bridges, a better torsional behaviour can be attained by anchoring of the cables at the central plane of the bridge on an A-shape pylon, rather than anchoring them at parallel independent pylons.

6.3 Installation of additional Damping Devices

The increase of structural damping is another possible measure to reduce dynamic effects of pedestrian movements on footbridges. This increase can be achieved either by actuation on particular elements within the structure, or by implementation of external control devices.

The use of external damping devices for absorbing excessive structural vibrations can be an effective solution in terms of reliability and cost. These devices can be based on active, semi-active or passive control techniques. Considering aspects like cost, maintenance requirements and practical experience, the usual option is for passive devices, which comprehend viscous dampers, tuned mass dampers (TMDs), pendulum dampers, tuned liquid column dampers (TLCDs) or tuned liquid (TLDs). The most popular of these are viscous dampers and TMDs.

7. Conclusions

This paper presents a procedure for the design of footbridges with regard to comfort criteria. A relevant and new aspect in the design procedure is the design first step in which comfort criteria for different traffic situations are fixed in communication with the client. It allows light and graceful structures by fixing realistic and adequate requirements.

Additionally different methods for the determination of the relevant accelerations are presented. These design methods cover simple evaluation methods e.g. for preliminary design studies and more complex methods for the detailed layout.

The paper finishes with the presentation of different methods of improvement of the structural properties such that comfort requirements are fulfilled.

Application examples can be found in the SYNPEX final report [1] and will be published in the scope of the HiVoSS project mentioned below.

The guideline presented here will also be published together with a guideline for vibration design for floors within the scope of the RFCS project HiVoSS in the languages English, French, Portuguese, Dutch and German. Also seminars on the vibration design of footbridges and floors will be performed in different countries. Further information can be found in the internet searching for "HiVoSS".

8. Acknowledgements

The design guidance presented here was elaborated within the research project "Advanced Load Models for Synchronous Pedestrian Excitation and Optimised Design Guidelines for Steel Footbridges (SYNPEX)" which was performed with the grant of the Research Programme of the Research Fund for Coal and Steel (RFCS) of the European Community (Project No RFS-CR-03019).

Special thanks apply for the project partners Christiane Butz, Elsa Caetano, Alvaro Cunha, Arndt Goldack, Andreas Keil and Mladen Lukic who have a large portion of the results presented in this paper.

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