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Human induced Vibrations of Steel Structures

Design of Footbridges

Guideline

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Table of frequently used symbols

a_{limit}	Acceleration limit according to a comfort class	[m/s²]
a_{max}	maximum acceleration calculated for a defined design situation	[m/s²]
B	width	[m]
d	density of pedestrians on a surface	[P/m²]
f, f_i	natural frequency for considered mode	[Hz]
f_s	step frequency of a pedestrian	[Hz]
P	force amplitude due to a single pedestrian	[N]
$P \times \cos(2\pi ft)$	harmonic load due to a single pedestrian	[N]
L	length	[m]
m^*	modal mass	[kg]
M	mass	[kg]
n	number of the pedestrians on the loaded surface S ($n = S \times$ density)	[P]
n'	equivalent number of pedestrians on a loaded surface S	[P/m²]
$p(t)$	distributed surface load	[kN/m²]
S	area of the loaded surface	[m²]
δ	logarithmic decrement for damping	[-]
μ	mass distribution per unit length	[kg/m]
$\Phi(x)$	mode shape	[-]
ψ	reduction coefficient account for the probability of a footfall frequency in the range the natural frequency for the considered mode	[-]
ξ	structural damping ratio	[-]

1 Introduction

Vibrations are an issue of increasing importance in current footbridge design practice. More sophisticated bridges (such as cable supported or stress ribbon footbridges) with increasing spans and more effective construction materials result in lightweight structures and 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. The most common dynamic loads on footbridges, other than wind loading, are the pedestrian induced footfall forces due to walking or jogging.

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These guidelines are based on the results obtained within the research project RFS-CR-03019 "Advanced Load Models for Synchronous Pedestrian Excitation and Optimised Design Guidelines for Steel Footbridges (SYNPEX)" financially supported by the Research Fund for Coal and Steel (RFCS).

Extra detail on the information given in these guidelines can be found in the accompanying Background Document [1] which also includes further references.

2 Definitions

The definitions given here relate to the application of this guideline.

Acceleration	A quantity that specifies the rate of change of velocity with time (denoted as dv / dt or d^2x / dt^2), usually along a specified axis. Normally expressed in terms of g or gravitational units.
Amplification	The process of increasing the magnitude of a variable quantity, without altering any other property.
Damper	Device mounted in structures to reduce the amplitudes of vibration through a method of dissipation of energy.
Damping	Damping is any effect, either inherent to a system or specifically added for the purpose, that tends to reduce the amplitude of vibration of an oscillatory system. Damping is the energy dissipation of a vibrating

system with time or distance. For structures, the total damping consists of

- Material and structural damping
- Damping by furniture and finishing
- Spread of energy throughout the whole structure

Dynamic action
Action that causes significant accelerations of the structure or structural members

Modal mass = generalised mass
A multiple degree of freedom system can be reduced to a combination of several single degree of freedom (SDOF) systems with coincident natural frequencies:

$$f = \frac{1}{2\pi}\sqrt{\frac{k^*}{m^*}}$$

where f is the natural frequency, expressed in Hz
k^* is the modal stiffness
m^* is the modal mass.

Thus the modal mass can be interpreted to be the mass activated in a specific mode shape.

Mode of vibration
A characteristic pattern assumed by a vibrating system in which the motion of every particle is a simple harmonic with the same frequency. Two or more modes may coexist in a multiple degree of freedom system.

Natural frequency = eigenfrequency
The natural frequency is the frequency of free vibration of a system. For a multiple degree of freedom system, the natural frequencies are the frequencies of the modes of vibration. Each structure has as many natural frequencies and associated modes of vibration as degrees of freedom. They are commonly sorted by the amount of energy that is activated by the oscillation; the first natural frequency is that on the lowest energy level and is the most likely to be activated.

The equation for the natural frequency of a single degree of freedom (SDOF) system is:

$$f = \frac{1}{2\pi}\sqrt{\frac{K}{M}}$$

where K is the stiffness
M is the mass.

The derivation of natural frequencies is described in chapter 4.1.

The frequency f is the reciprocal of the oscillation time T ($f = 1 / T$).

Resonance	A system is at resonance when any change in the frequency of a forced vibration, however small, causes a decrease in the response of the system. When damping is small, the resonant frequency is approximately equal to the natural frequency of the system (the frequency of free vibrations).
Response spectrum	A response spectrum is a plot of the peak or steady-state response (displacement, velocity or acceleration) of a series of linear single degree of freedom oscillators of varying natural frequency that are forced into motion by the vibration. The resulting plot can then be used to pick off the response of any linear system, given its natural frequency of oscillation. The response spectrum contains precise information about the distribution of vibration energy for various frequencies.
Spectrum	Description of any time dependant signal as a series of single-frequency components, each with an amplitude and, if appropriate, phase.

3 Design procedure

An increasing number of vibration problems for footbridges encountered in the last few years show that footbridges should no longer be designed only for static loads, but also for the dynamic behaviour. The design should take into account the vibration performance of the footbridge due to walking pedestrians. It is important to note that there are currently no code regulations available.

Although from designers’ point of view this lack of regulation allows a large amount of freedom and therefore a large variety of innovative bridge structures, it is nevertheless of vital importance that the bridge will meet comfort requirements which are required by the client or owner. The question “Will the footbridge meet the comfort criteria when vibrating?” plays an important role in the design process, as dampers are not only additional bridge furniture, but may need to be included in the design.

The general principles of a proposed design methodology are given in Figure 3-1.

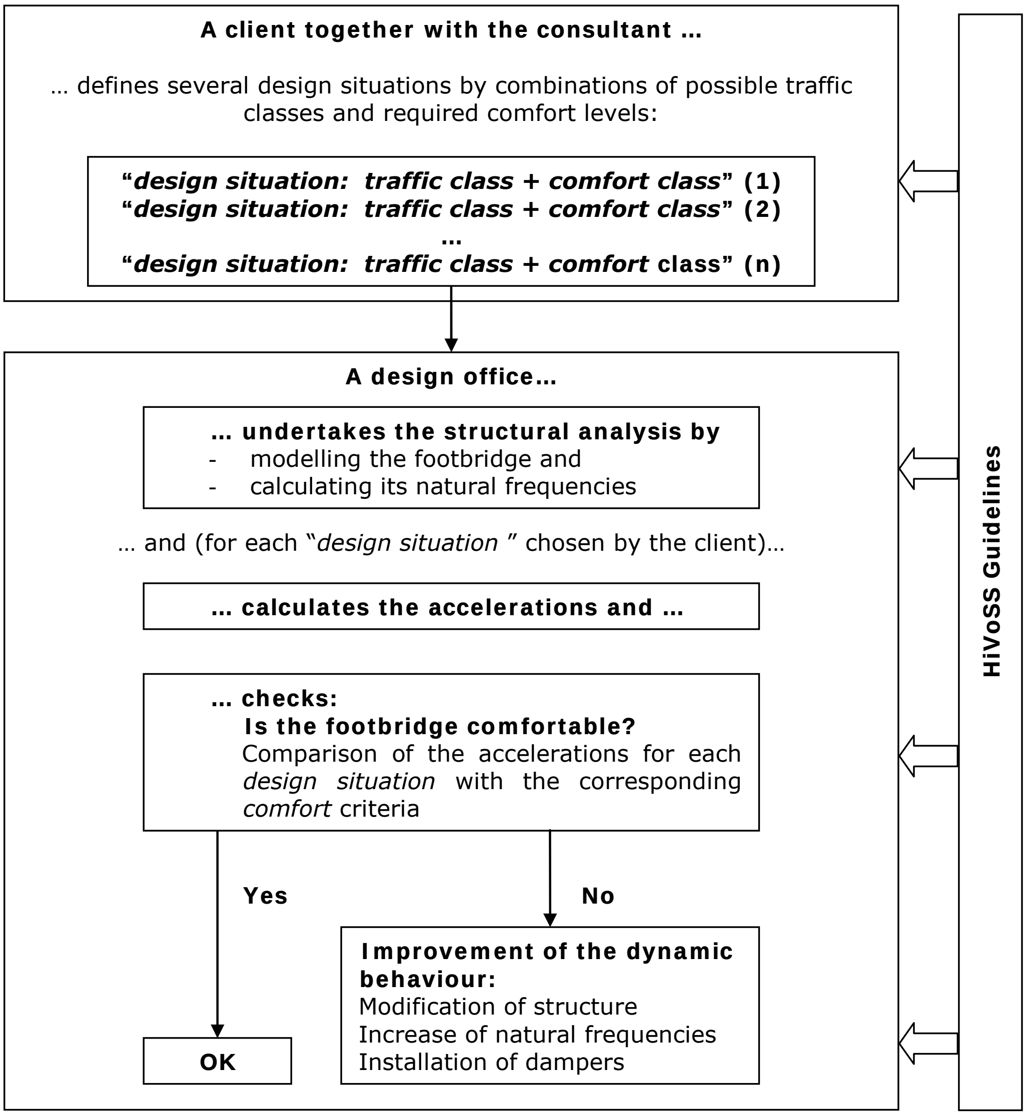


Figure 3-1: Methodology for the design

The flowchart in Figure 3-2 shows how to check the dynamic behaviour of the footbridge in the design phase and how this guideline can be employed. The various steps mentioned in the flowchart will be discussed in section 4.

Safety problems due to overstressing or fatigue may also occur and should also be considered in the design of footbridges - this guideline only treats reversible serviceability, as defined by the Eurocodes. Design rules for overstressing and fatigue are given elsewhere. It should be noted that all the usual verifications in Serviceability Limit State (SLS) and Ultimate Limit State (ULS) must be carried out according to the standards in use.

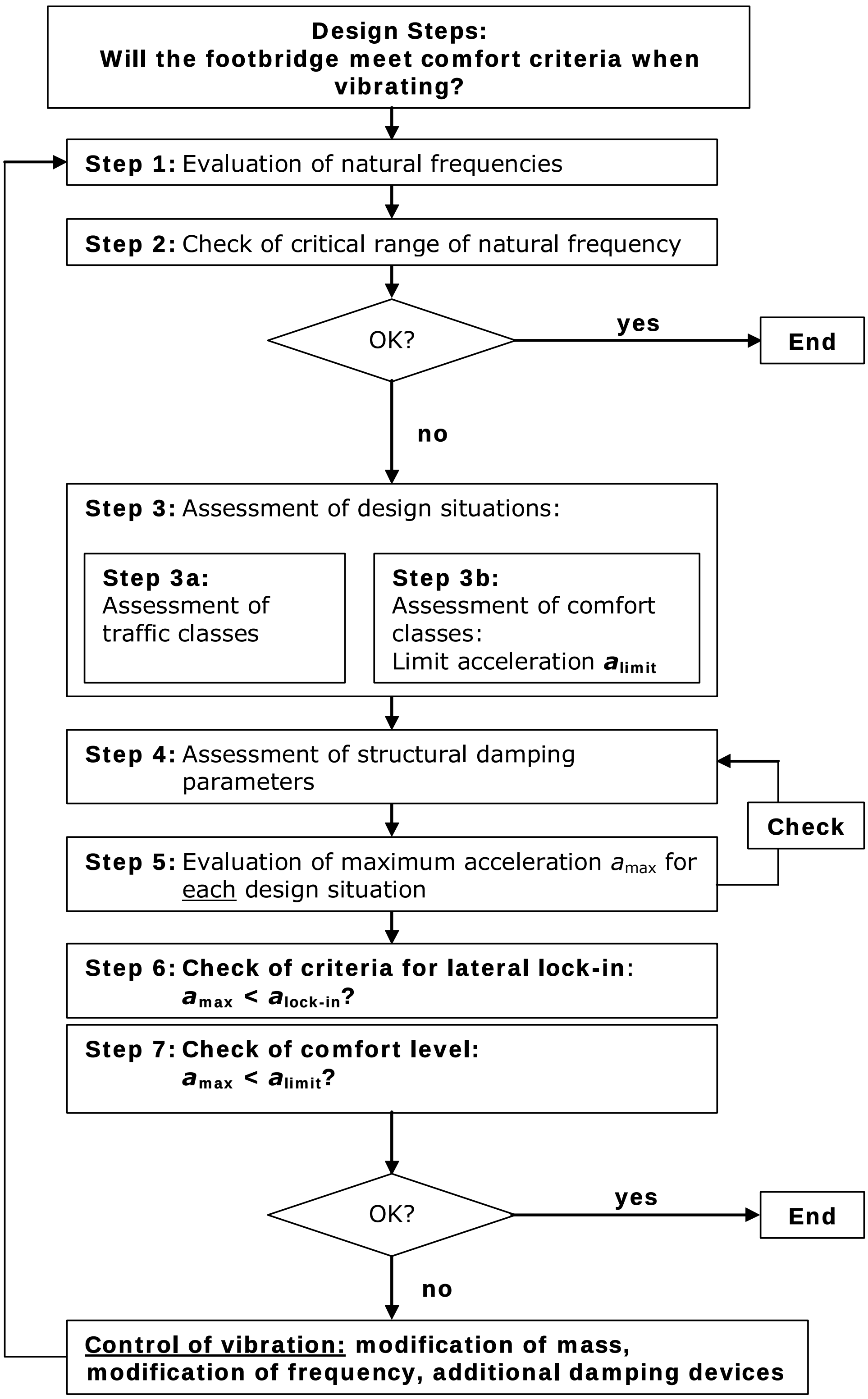


Figure 3-2: Flowchart for the use of this guideline

4 Design steps

4.1 Step 1: Evaluation of natural frequencies

There are several ways to calculate the natural frequency of a footbridge during design, especially for the preliminary check of the bridge vibration, e.g.:

- Finite element (FE) method
- Hand formulas e.g. for beams, cables and plates.

It must be kept in mind that 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 calculations and the measured data of the real structure.

It is recommended that the mass of pedestrians should be considered when calculating the natural frequencies only when the modal mass of the pedestrians is more than 5 % of the modal deck mass.

4.2 Step 2: Check of critical range of natural frequencies

The critical ranges for natural frequencies f_i of footbridges with pedestrian excitation are:

- for vertical and longitudinal vibrations:

$$1{,}25\ \text{Hz} \leq f_i \leq 2{,}3\ \text{Hz}$$

- for lateral vibrations: $0{,}5\ \text{Hz} \leq f_i \leq 1{,}2\ \text{Hz}$

Footbridges with frequencies for vertical or longitudinal vibrations of

$$2{,}5\ \text{Hz} \leq f_i \leq 4{,}6\ \text{Hz}$$

might be excited to resonance by the 2nd harmonic of pedestrian loads [1]. In that case, the critical frequency range for vertical and longitudinal vibrations expands to:

$$1{,}25\text{Hz} \leq f_i \leq 4{,}6\text{Hz}$$

Lateral vibrations are not effected by the 2nd harmonic of pedestrian loads.

Note: A vertical vibration excitation by the second harmonic of pedestrian forces might take place. Until now there is no hint in the literature that onerous vibration of footbridges due to the second harmonic of pedestrians have occurred.

4.3 Step 3: Assessment of Design Situation

The design of a footbridge starts with specifying several significant design situations - sets of physical conditions representing the real conditions occurring during a certain time interval. Each design situation is defined by an expected traffic class (cf. section 0) and a chosen comfort level (cf. section 4.3.2).

There are design situations which might occur once in the lifetime of a footbridge, like the inauguration of the bridge, and others that will occur daily, such as commuter traffic. Table 4-1 gives an overview of some typical traffic situation which may occur on footbridges. The expected type of pedestrian traffic and the traffic density, together with the comfort requirements, has a significant effect on the required dynamic behaviour of the bridge.

Table 4-1: Typical traffic situations

	Individual pedestrians and small groups Number of pedestrians: 11 Group size: 1-2 P Density: 0,02 P/m^2 Note: P = pedestrian
	Very weak traffic Number of pedestrians: 25 Group size: 1-6 P Density: 0,1 P/m^2
	Weak traffic Here: event traffic Number of pedestrians: 60 Group size: 2-4 P Density: 0,2 P/m^2
	Exceptionally dense traffic Here: opening ceremony traffic Density: > 1,5 P/m^2

To get an insight into the dynamic bridge response, it is recommended that several different probable design situations are specified. An example of this is given in Table 4-2.

Table 4-2: Example of a specification of significant design situations

Design Situation	Description	Traffic Class (cf. 0)	Expected occurrence	Comfort Class (cf. 4.3.2)
1	inauguration of bridge	TC4	once in the lifetime	CL3
2	commuter traffic	TC2	daily	CL1
3	rambler at the weekend	TC1	weekly	CL2
⋮	⋮	⋮	⋮	⋮

4.3.1 Step 3a: Assessment of traffic classes

Pedestrian traffic classes and corresponding pedestrian stream densities are given in Table 4-3.

Table 4-3: Pedestrian traffic classes and densities

Traffic Class	Density d (P = pedestrian)	Description	Characteristics
TC 1*)	group of 15 P; d=15 P / (B L)	Very weak traffic	(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 	Still unrestricted walking Overtaking can intermittently be inhibited

Traffic Class	Density d (P = pedestrian)	Description	Characteristics
TC 4	d = 1,0 P/m²	Very dense traffic 	Freedom of movement is restricted Obstructed walking Overtaking is no longer possible
TC 5	d = 1,5 P/m²	Exceptionally dense traffic	Unpleasant walking Crowding begins One can no longer freely choose pace

*) An equivalent pedestrian stream for traffic class TC1 is calculated by dividing the number of pedestrians by the length L and width B of the bridge deck.

Pedestrian formations, processions or marching soldiers are not taken into account in the general traffic classification, but need additional consideration.

4.3.2 Step 3b: Assessment of comfort classes

Criteria for pedestrian comfort are most commonly represented as a limiting acceleration for the footbridge. Four comfort classes are recommended by this guideline and are presented in Table 4-4.

Table 4-4: Defined comfort classes with common acceleration ranges

Comfort class	Degree of comfort	Vertical a_{limit}	Lateral 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

Note that the given acceleration ranges are just comfort criteria; lock-in criteria for horizontal vibrations are given in section 4.6.

4.4 Step 4: Assessment of structural damping

The amount of damping present is very significant in the evaluation of the amplitude of oscillations induced by pedestrians. The attenuation of vibrations, i.e., the energy dissipation within the structure, depends both on the intrinsic damping of construction materials, which is of distributed nature, and on the local effect of bearings or other control devices. Additional damping is also provided by non-structural elements, like handrails and surfacing.

In general, the amount of damping depends on the level of vibrations, as higher amplitudes of vibration cause more friction between structural and non-structural elements and bearings.

The co-existence of various mechanisms of dissipation within the structure makes damping a complex phenomenon whose accurate characterisation can only rely on measurements taken once the footbridge has been constructed, including installation of handrails, surfacing and any type of furniture.

Flexible and light footbridges are further affected by wind, which generates aerodynamic damping, and an increase of wind velocity can lead to increased damping. This added damping can be taken into consideration for the purpose of wind studies, but not for the evaluation of pedestrian induced effects.

4.4.1 Damping model

For the purpose of design and numerical modelling, it is necessary to specify a model and define the corresponding parameters. The common approach uses linear viscous dampers (sometimes referred to as dashpot dampers), which implies that the generation of damping forces is proportional to the rate of change of the displacements with time (velocity). This model has the advantage of leading to linear dynamic equilibrium equations, whose analytical solution can be easily obtained. However, it only approximates the real damping of a structure for low levels of oscillation.

The inclusion of control systems (cf. section 6.4.3) may lead to structures for which the damping matrix is no longer proportional and consequently conventional modal analysis is no longer applicable. The tuning of the damper system and the calculation of the damped structure response then requires more powerful algorithms, namely iterative calculations based on direct integration methods, or else on a state space formulation.

4.4.2 Damping ratios for service loads

For the design of footbridges for comfort level, which is in terms of Eurocode reliability consideration a serviceability condition, Table 4-5 recommends minimum and average damping ratios.

Table 4-5: Damping ratios according to construction material for serviceability conditions

Construction type	Minimum ξ	Average ξ
Reinforced concrete	0,80%	1,3%

Construction type	Minimum ξ	Average ξ
Prestressed concrete	0,50%	1,0%
Composite steel-concrete	0,30%	0,60%
Steel	0,20%	0,40%
Timber	1,0%	1,5%
Stress-ribbon	0,70%	1,0%

4.4.3 Damping ratios for large vibrations

Intentional loads can produce large levels of oscillation in light footbridges, which lead to higher damping ratios, as listed in Table 4-6.

Table 4-6: Damping ratios according to construction material for large vibrations

Construction type	Damping ratio ξ
Reinforced concrete	5,0%
Prestressed concrete	2,0%
Steel, welded joints	2,0%
Steel, bolted joints	4,0%
Reinforced elastomers	7,0%

4.5 Step 5: Determination of maximum acceleration

When one or several design situations (cf. section 4.3) are defined and the values for damping are determined (cf. section 4.4), the next step is to calculate the maximum acceleration a_{max} for each design situation.

There are various methods for calculating the acceleration of the bridge; this design guideline recommends using one of the methods shown in Figure 4-1, which will be discussed in the following chapters.

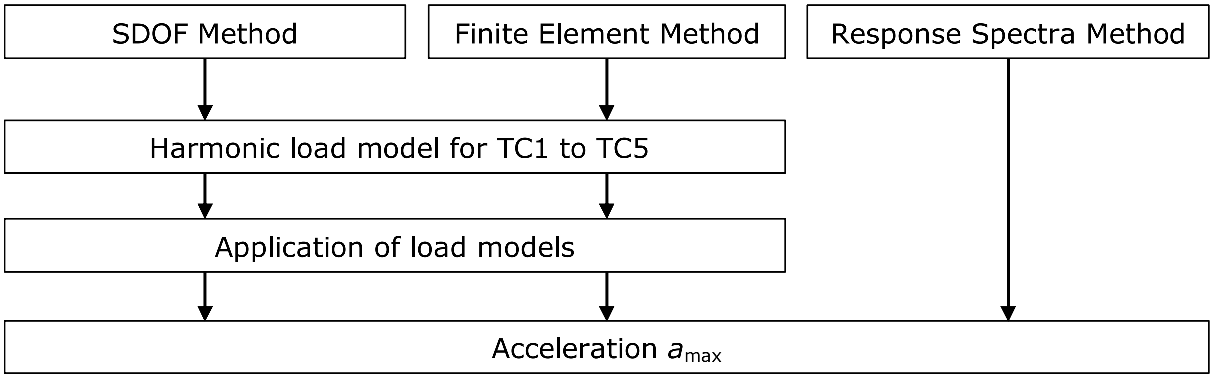


Figure 4-1: Various methods for calculating the acceleration

Note: It is important to check whether the acceleration calculated with the assumed damping parameters for large or small vibrations (cf. section 4.4) corresponds to the acceleration on the built structure (cf. section 5). Experience has shown that it is very difficult to predict the structural damping of the finished footbridge. Therefore, damping always has a broad scatter and consequently acceleration also has a broad scatter.

4.5.1 Harmonic load models

4.5.1.1 Equivalent number of pedestrians for streams

Once a numerical model of the footbridge has been developed, the design situations and corresponding load models chosen and the damping ratios specified, the footbridge response can be calculated. Harmonic load models are required to calculate the acceleration when using either Finite Element methods or Single Degree of Freedom (SDOF) methods (cf. section 4.5.1.3). For the modelling of a pedestrian stream consisting of n "random" pedestrians, the idealised stream consisting of n' perfectly synchronised pedestrians should be determined (cf. Figure 4-2). The latter would be synchronised only among themselves (without taking into account the influence of the vibrating structure on their footfall frequency). The two streams are supposed to cause the same effect on a structure, but the equivalent one can be modelled as a deterministic load.

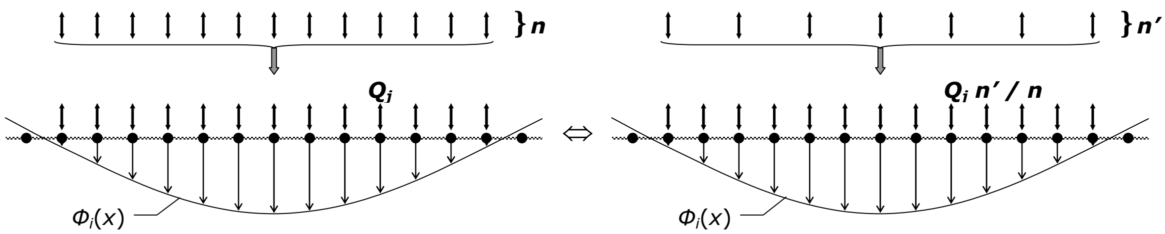


Figure 4-2: Equivalence of streams

For the evaluation of the response with respect to group or pedestrian stream loading, the application of a distributed harmonic load along the bridge deck (simulating an equivalent number of pedestrians at fixed locations) meets almost all requirements for practical design of footbridges.

Care is needed in the choice of the range of frequencies for which this kind of calculation makes sense. The problem of the influence of the structure on the behaviour of the pedestrians is not taken into account and this can aggravate the response.

4.5.1.2 Application of Load models

In the recommended design procedure, harmonic load models are provided for each traffic class TC1 to TC5 (cf. Table 4-3). There are two different load models to calculate the response of the footbridge due to pedestrian streams depending on their density:

- Load model for TC1 to TC3 (density d < 1,0 P/m²)
- Load model for TC4 and TC5 (density $d \geq$ 1,0 P/m²)

Both load models share a uniformly distributed harmonic load $p(t)$ [N/m²] that represents the equivalent pedestrian stream for further calculations:

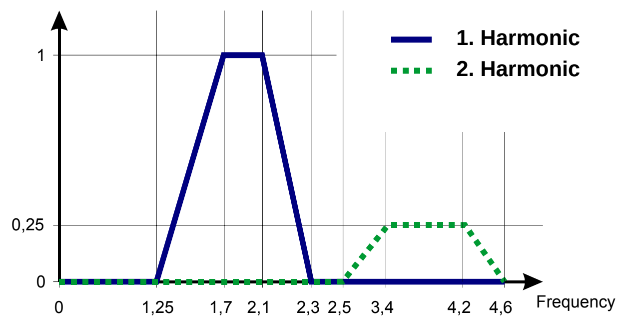
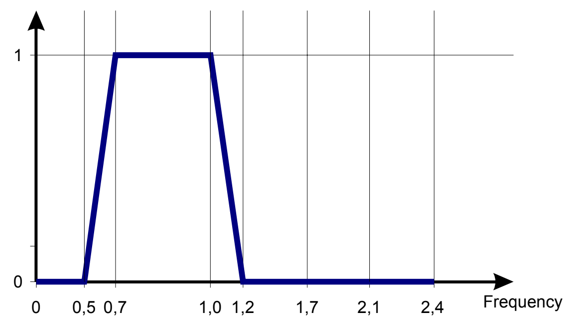
$$p(t) = P \times \cos(2\pi f_s t) \times n' \times \psi \qquad \text{Eq. 4-1}$$

where $P \times \cos(2\pi f_s t)$ is the harmonic load due to a single pedestrian,

- P is the component of the force due to a single pedestrian with a walking step frequency f_s,
- f_s is the step frequency, which is assumed equal to the footbridge natural frequency under consideration,
- n' is the equivalent number of pedestrians on the loaded surface S,
- S is the area of the loaded surface,
- ψ is the reduction coefficient taking into account the probability that the footfall frequency approaches the critical range of natural frequencies under consideration.

The amplitude of the single pedestrian load P, equivalent number of pedestrians n' (95th percentile) and reduction coefficient ψ are defined in Table 4-7, considering the excitation in the first harmonic or second harmonic of the pedestrian load (see Section 4.2).

Table 4-7: Parameters for load model of TC1 to TC5

<table>
<tr><td colspan="3">P [N]</td></tr>
<tr><td>Vertical
280</td><td>Longitudinal
140</td><td>Lateral
35</td></tr>
<tr><td colspan="3">Reduction coefficient ψ</td></tr>
<tr><td colspan="3">Vertical and longitudinal

Lateral
</td></tr>
<tr><td colspan="3">Equivalent number n' of pedestrians on the loaded surface S for load model of:</td></tr>
<tr><td colspan="3">TC1 to TC3 (density d < 1,0 P/m²): $n' = \frac{10{,}8\sqrt{\xi \times n}}{S}$ [1/m²]</td></tr>
<tr><td colspan="3">TC4 and TC5 (density $d \geq$ 1,0 P/m²): $n' = \frac{1{,}85\sqrt{n}}{S}$ [1/m²]</td></tr>
</table>

where ξ is the structural damping ratio and

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

The load model for pedestrian groups (TC1) takes into account a free movement of the pedestrians. Consequently, the synchronization among the group members is equal to a low density stream. In the case of dense streams (TC4 and TC5) walking gets obstructed, the forward movement of the stream gets slower and the synchronisation increases. Beyond the upper limit value of 1,5 P/m² walking of pedestrians is impossible, so that dynamic effects significantly reduce. When a stream becomes dense, the correlation between pedestrians increases, but the dynamic load tends to decrease.

In Figure 4-3 a harmonic load $p(t)$ is applied to the structure for a particular mode shape.

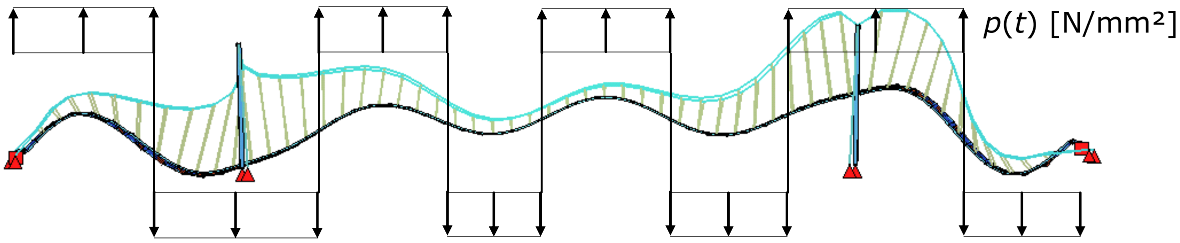


Figure 4-3: Application of a harmonic load according to mode shape $\Phi(x)$

The harmonic load models above describe the loads induced by streams of pedestrians when walking along the footbridge. Some footbridges may be further affected by the action of joggers which is further described in [1].

4.5.1.3 SDOF method

Generally, the dynamic behaviour of a structure can be evaluated by a modal analysis, where an arbitrary oscillation of the structure is described by a linear combination of several different harmonic oscillations in the natural frequencies of the structure. Therefore, the structure can be transformed into several different equivalent spring mass oscillators, each with a single degree of freedom. Each equivalent single degree of freedom (SDOF) system (cf. Figure 4-4) has one natural frequency and one mass that is equal to each natural frequency of the structure and the accompanying modal mass.

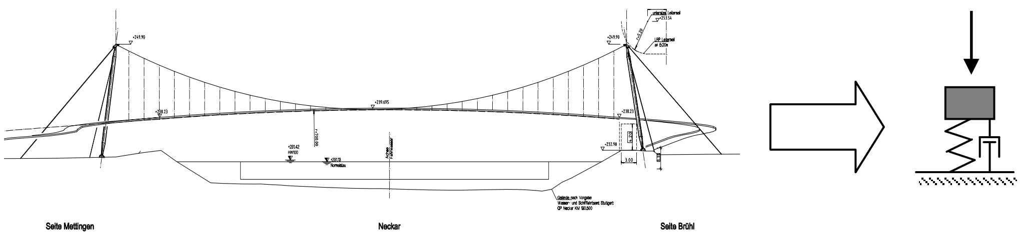


Figure 4-4: Equivalent SDOF oscillator for one natural frequency / vibration mode of the structure

The basic idea is to use a single equivalent SDOF system for each natural frequency of the footbridge in the critical range of natural frequencies and to calculate the associated maximum acceleration for a dynamic loading.

The maximum acceleration a_{max} at resonance for the SDOF system is calculated by:

$$a_{max} = \frac{p^*}{m^*}\frac{\pi}{\delta} = \frac{p^*}{m^*}\frac{1}{2\xi} \qquad \text{Eq. 4-2}$$

where p^* is the generalised load

m^* is the generalised (modal) mass

ξ is the structural damping ratio and

δ is the logarithmic decrement of damping.

4.5.2 Response Spectra Method for pedestrian streams

At the design stage it is not necessary to apply a time domain analysis in every case.

The aim of a spectral design method is to find a simple way to describe the stochastic loading and system response that provide design values with a specific confidence level.

It is assumed that:

- the mean step frequency, $f_{s,m}$, of the pedestrian stream coincides with the considered natural frequency of the bridge, f_i,
- the mass of the bridge is uniformly distributed,
- the mode shapes are sinusoidal,
- no modal coupling exists,
- the structural behaviour is linear-elastic.

The system response – "maximum peak acceleration" – was chosen as the design value. In the design check, this acceleration is compared with the tolerable acceleration according to the comfort class to be proofed.

For different pedestrian densities, the characteristic acceleration, which is the 95th percentile of the maximum acceleration, can be determined according to the formulas and tables given below.

This maximum acceleration is defined by the product of a peak factor $k_{a,d}$ and a standard deviation of acceleration σ_a:

$$a_{\max,d} = k_{a,d}\ \sigma_a \qquad \text{Eq. 4-3}$$

Note: The peak factor $k_{a,d}$ serves to transform the standard deviation of the response σ_a to the characteristic value $a_{\max,d}$. In serviceability states, the characteristic value is the 95th percentile, $k_{a,95\%}$.

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

The result is an empirical equation for the determination of the variance of the acceleration response:

$$\sigma_a^2 = k_1 \xi^{k_2} \frac{C \quad \sigma_F^2}{m*_i^2} \qquad \text{Eq. 4-4}$$

where $k_1 = a_1 f_i^{\,2} + a_2 f_i + a_3$

$k_2 = b_1 f_i^{\,2} + b_2 f_i + b_3$

a_1, a_2, a_3, b_1, b_2, b_3 are constants

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

ξ is the structural damping ratio

C is the constant describing the maximum of the load spectrum

$\sigma_F^2 = k_F\ n$	is the variance of the loading (pedestrian induced forces)
k_F	is a constant
$n = d \times L \times B$	number of pedestrians on the bridge, with d: pedestrian density, L: bridge length, B: bridge width
m^*_i	is the modal mass of the considered mode i

The constants a_1 to a_3, b_1 to b_3, C, k_F and $k_{a,95\%}$ can be found in Table 4-8 for vertical accelerations and in Table 4-9 for lateral accelerations.

Table 4-8: 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	$1{,}20\times10^{-2}$	2,95	-0,07	0,60	0,075	0,003	-0,040	-1,000	3,92
1,0	$7{,}00\times10^{-3}$	3,70	-0,07	0,56	0,084	0,004	-0,045	-1,000	3,80
1,5	$3{,}34\times10^{-3}$	5,10	-0,08	0,50	0,085	0,005	-0,060	-1,005	3,74

Table 4-9: 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	$2{,}85\times10^{-4}$	6,8	-0,08	0,50	0,085	0,005	-0,06	-1,005	3,77
1,0		7,9	-0,08	0,44	0,096	0,007	-0,071	-1,000	3,73
1,5		12,6	-0,07	0,31	0,120	0,009	-0,094	-1,020	3,63

Alternatively, for a simplified estimation of the required modal mass for a given pedestrian traffic to ensure a given comfort limit a_{limit}, an expression is derived that is valid for $f_{s,m} = f_i$:

$$m_i^* \geq \frac{\sqrt{n}\left(k_1\,\xi^{k_2} + 1.65\,k_3\,\xi^{k_4}\right)}{a_{\text{limit}}}$$

where	m^*_i	modal mass for the considered mode i
	n	number of pedestrians on the bridge
	ξ	structural damping coefficient
	k_1 to k_4	constants (cf. Table 4-10 for vertical bending and torsion modes and Table 4-11 for lateral bending modes)

Table 4-10: Constants for required vertical modal mass

d [P/m²]	k_1	k_2	k_3	k_4
≤ 0,5	0,7603	0,468	0,050	0,675
1,0	0,5700		0,040	
1,5	0,4000		0,035	

Table 4-11: Constants for the required lateral modal mass

d [P/m^2]	k_1	k_2	k_3	k_4
≤ 0,5	0,1205	0,45	0,012	0,6405
1,0				
1,5				

The design method was elaborated with beam bridge models. If the structural behaviour of a bridge differs significantly from that of beam bridge, limits of application of the spectral method may be reached.

4.6 Step 6: Check of criteria for lateral lock-in

The triggering number of pedestrians for lateral lock-in, that is the number of pedestrians N_L that could lead to a vanishing of the overall damping producing a sudden amplified response, can be defined as:

$$N_L = \frac{8\pi\xi m^* f}{k} \qquad \text{Eq. 4-5}$$

where ξ is the structural damping ratio

m^* is the modal mass

f is the natural frequency

k is a constant (300 Ns/m approximately over the range 0,5-1,0 Hz).

Another approach is to define the trigger acceleration amplitude when the lock-in phenomenon begins:

$$a_{lock\ \ in} = 0{,}1 \text{ to } 0{,}15 \text{ m/s}^2 \qquad \text{Eq. 4-6}$$

Recent experiments have shown the adequacy of both formulae to describe the triggering for lock-in.

Note: Pedestrian streams synchronising with vertical vibrations have not been observed on footbridges.

4.7 Step 7: Check of comfort level

According to the design verification methodology specified in Figure 3-2, the response calculated for the specified design situations and the corresponding load models has to be compared with the specified comfort limits given in Table 4.4. The non-compliance with those limits implies the need of measures that improve the dynamic behaviour of the footbridge. These measures include:

- modification of the mass
- modification of frequency
- modification of structural damping
- addition of damping

For an already constructed bridge, the simplest approach is based on the increase of structural damping, which can be achieved either by implementation of additional control devices, or by actuation on non-structural finishings, like the hand-rails and surfacing (cf. chapter 6).

5 Evaluation of dynamic properties of footbridges

5.1 Introduction

The experimental characterisation of the dynamic behaviour of a footbridge may be an important component of the project and can be performed based on two different levels of complexity:

- **Level 1-** Identification of structural parameters, with the purpose of calibrating numerical models and eventually tuning control devices. Natural frequencies, vibration modes and damping coefficients are the parameters of interest;
- **Level 2-** Measurement of the bridge dynamic response under human excitation for assessment of comfort criteria and/or correlation with the simulated response.

The adoption of one of the above mentioned strategies depends on the characteristics of the structure and on the aims of the study.

Level 2 tests can be characterised as standard tests that should be developed at the end of construction of any potentially lively footbridge, providing important information for design and verification purposes. Based on the results of these tests, the bridge owner may decide whether to implement control measures or not. It should be noted that the use of experimental tests to check the comfort class of a specific footbridge requires the performance of measurements for all vibration phenomena and design situations considered in the development of design load models and involves the obtainment of characteristic values of the response.

Level 1 tests are required when it is clear that the dynamic behaviour of the footbridge is beyond acceptability limits and control measures are necessary. The appropriate design of control devices requires an accurate knowledge of structural parameters, namely natural frequencies and vibration modes.

The current chapter presents general guidelines for testing and data analysis of footbridges.

5.2 Response measurements

The performance of **Level 2** tests should consider the following items:

1. Identification of critical natural frequencies;
2. Identification of damping ratios;
3. Measurement of response induced by one pedestrian;
4. Measurement of the response induced by a small group of pedestrians;
5. Measurement of the response induced by a continuous flow of pedestrians.

The verification of acceptability limits of vibration for a particular pedestrian bridge should be based on the results of these tests, under consideration of the particular use of the bridge.

5.2.1 Measurements of ambient response for identification of critical natural frequencies

Tests should preferably be conducted on the bridge closed to pedestrian traffic. Assuming that a preliminary dynamic analysis of the bridge has been conducted, providing an estimation of natural frequencies and vibration modes, the instrumented sections should correspond to the sections of maximum estimated modal response for the estimated critical frequencies.

5.2.2 Raw measurement of damping ratios associated with critical natural frequencies

Raw estimates of the damping ratios associated with critical natural frequencies can be obtained from a simple free vibration test in which a pedestrian jumps / bends knees / bounces on a fixed location at a particular frequency, trying to induce resonant response of the bridge for the corresponding vibration mode. After a few cycles of excitation, the pedestrian action is suddenly interrupted and the free vibration response is recorded. This process should be repeated a number of times, in order to provide average estimates of damping coefficient as a function of amplitude of oscillation.

5.2.3 Measurement of the response induced by one pedestrian

The tests described above provide an update of the expected critical natural frequencies. The response of the footbridge is now measured at the relevant sections (the maximum modal displacement section for each critical frequency), considering the motion of a single pedestrian over the bridge. Several types of motion should be explored, as a function of the frequencies of interest:

- walking, for critical natural frequencies below 2,5 Hz;
- walking or running, for critical natural frequencies between 2 Hz and 3 Hz;
- running, for natural frequencies above 3 Hz.

Given the random characteristics of excitation, a number of tests should be performed for each combination of frequency and motion., typically about 5. A metronome should be used to ensure the correct walking rate is obtained. The maximum acceleration and dynamic displacement (which can also be derived from acceleration) of the bridge should be recorded for each collected series, and the peak response induced by one pedestrian can be taken as the maximum of the peak responses from the various tests. The weight of the pedestrian should be noted. Whenever the bridge has a non-symmetric incline, the response should be recorded with the pedestrian travelling down the slope.

5.2.4 Measurement of the response induced by a group of pedestrians

The response should be measured in two conditions:

- walking / running of group under current use, and
- walking / running of the group with the goal of inducing high response (vandalism).

Whenever possible the group should be formed of 10 pedestrians if the deck width is no greater than 2.5 m and 15 pedestrians for larger widths. The response should be measured based on the specification made in section 5.2.3 for the crossing of one pedestrian. The response associated with the synchronised group should be collected, again making use of a metronome in order to achieve synchronisation at a particular frequency.

Given that it is expected that the presence of people on the deck might result in higher damping ratios and that for high amplitudes of vibration these ratios increase, it is suggested that measurements are made of the free vibration response after resonant excitation of the bridge by the group, jumping at a fixed position.

5.2.5 Measurement of the response induced by a continuous flow of pedestrians

The measurement of the response induced by a continuous flow of pedestrians is also of interest for determining the footbridge response under different usage conditions. This measurement should especially be considered for footbridges that clearly exhibit a lively behaviour, namely a trend for synchronisation effects. The measurement procedures are identical to the ones adopted for single pedestrian and group tests described in sections 5.2.3 and 5.2.4.

5.3 Identification tests

The identification of modal parameters, i.e., natural frequencies, vibration modes and damping coefficients, of a structure is performed through the above designated **Level 1** tests. A conventional modal analysis technique can be applied, based on forced vibration tests or, alternatively, identification can be performed based on free or ambient vibration tests. The basic parameters of the tests are established for the two cases in the following sections.

5.3.1 Forced vibration tests

Forced vibration tests are the basis of the traditional modal analysis techniques and provide the most precise results, given that they rely on controlled inputs and outputs. This is particularly relevant for damping coefficient estimates, where the quality is highly affected by measurement uncertainties. The identification technique to apply depends on the type of excitation employed. However, there is a risk that the input energy associated with the low natural frequencies is very small and so the signal-to-noise ratio may be very low.

The devices used for these tests,

- impact hammer and
- vibrator,

are described in section 5.4.2.1.

5.3.2 Ambient vibration tests

Ambient vibration tests employ the current ambient loads on the structure as input loads, assuming that the frequency content of these is approximately constant in the frequency range of interest. Although this hypothesis is not necessarily valid, ambient vibration tests are becoming an extremely attractive alternative for identification of modal parameters in civil engineering structures, given the limited required resources, and the high precision of currently available sensors. The use of these techniques can provide significant errors in determining the damping coefficient estimates.

5.3.3 Free vibration tests

Free vibration tests consist of recording the structural response associated with the sudden release of a tensioned cable or other device that causes an initial deviation from the equilibrium position of the structure. These tests are relatively inexpensive when conducted at the end of construction of the footbridge and provide accurate estimates of damping ratios of the excited modes. They constitute an alternative to forced vibration tests, and it is expected that higher quality modal estimates are obtained than those resulting from ambient vibration tests.

For the purpose of damping identification, measurements should be performed at wind velocities lower than 2-5 m/s.

5.4 Instrumentation

5.4.1 Response devices

Acceptability limits for pedestrian comfort are generally defined in terms of acceleration, and so the usual measured response quantity is acceleration.

Accelerometers are sensors that produce electrical signals proportional to the acceleration in a particular frequency band, and can be based on different working principles. For most pedestrian bridges the frequency range of interest is 0,5 – 20 Hz. Accordingly, common specifications for accelerometers are:

- Frequency range (with 5% linearity): 0,1 – 50 Hz;
- Minimum sensitivity: 10 mV/g
- Range: ±0,5 g

5.4.2 Identification devices

5.4.2.1 Force devices

For forced vibration tests of pedestrian bridges, possible input devices are an impact hammer (cf. Figure 5-1) or a vibrator (cf. Figure 5-2).

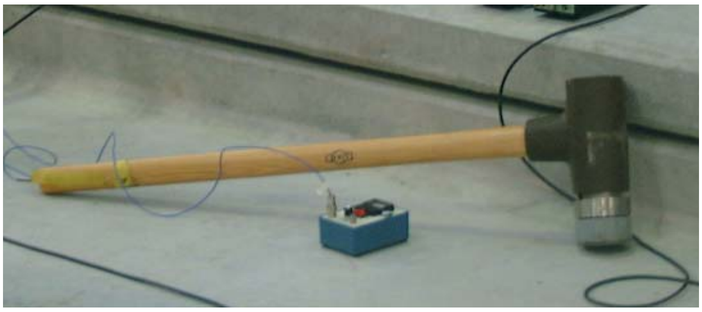


Figure 5-1: Impact hammer for civil engineering applications

Figure 5-2: Electromagnetic shaker for civil engineering applications. Vertical mounting

Note: Reference to these devices is given in the Background Document [1].

5.4.2.2 Input sensor devices

One important topic in the testing of pedestrian bridges is the measure of input loads induced by pedestrians, both when walking alone or in groups.

The direct assessment of the concentrated load applied by a pedestrian can be made through instrumentation of a walking platform with force plates. For a walking group, it is also important to measure the degree of synchronisation of pedestrians, which can be assessed by means of video recording and image processing.

6 Control of vibration response

6.1 Introduction

Following the design verification methodology specified in section 3, the response calculated for the specified load models has to be compared with the comfort limits. The non-compliance with those limits or with the lock-in criteria implies the need to develop measures that improve the dynamic behaviour of the footbridge. These measures include modification of the mass, frequency or structural damping.

6.2 Modification of mass

For very light footbridges, the use of heavy concrete deck slabs can improve dynamic response to pedestrian loads, as a consequence of the increased modal mass. This approach is particularly relevant for stress-ribbon structures.

6.3 Modification of frequency

Traditionally structural stiffness is modified to raise the frequency out of the critical range for both vertical and lateral vibration. Frequency is proportional to the square root of the ratio between stiffness and mass, and so significant structural changes are generally required to sufficiently raise the frequency. In modern bridge design, where the aim is to build light and graceful structures, these changes are usually impractical once construction has been completed, but can be considered at the design stage.

6.4 Modification of structural damping

6.4.1 Introduction

The increase of structural damping is another possible measure to reduce the dynamic effects of pedestrians on a footbridge. For an existing bridge, the simplest approach is based on the increase of the structural damping, which can be achieved either by actuation on particular elements within the structure, or by implementation of external damping 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 include viscous dampers, tuned mass dampers (TMD), pendulum dampers, tuned liquid column dampers (TLCD) or tuned liquid dampers (TLD). The most popular of these are viscous dampers and TMDs.

6.4.2 Simple measures

Hand-rails are generally considered non-structural elements whose geometry and characteristics are specified according to architectural considerations. It has been observed, however, that these elements can contribute to stiffen and dampen the footbridge, especially in the case of very slender structures. The use of wire mesh fencing, for example, has shown to contribute to a significant increase of damping of the footbridge, because of the friction generated between wires during vibrations. However, it is not possible to quantify the damping increase as it is strongly dependant on the amplitude of vibrations.

In a similar way, the use of elastomers in bearings and surfacing can contribute to an increased damping of the footbridge, but it should be remembered that the properties of elastomers degrade with time and regular maintenance will be required.

The choice of bolted instead of welded joints is another measure that can contribute to an overall higher damping, as a consequence of the friction developed in the load transfer between elements.

6.4.3 Additional damping devices

Table 6-1 lists some examples of structures where damping systems have been implemented, indicating the characteristics of those measures and the overall effect on the dynamic behaviour.

Table 6-1: Footbridges where damping systems have been implemented

Bridge	Number of spans / length [m]	Type	Controlled frequencies [Hz]	Dominant vibration direction	Type of damping system implemented	Effect of the damping system on the overall behaviour
T-Bridge, Japan	2 spans, 45+134	Cable-stayed, continuous steel box girder	0,93	Lateral	Tuned liquid dampers of sloshing type, inside box girder. Total of 600 containers used, mass ratio of 0,7% of generalised mass of girder lateral vibration mode.	Lateral girder displacement reduced from around 8,3mm to 2,9mm
Millennium Bridge, London	3 spans, 108+144+ 80	Suspension tension-ribbon	0,8 (main) 0,5 1,0	Lateral	Viscous dampers and tuned mass damper used to control horizontal movements. Vertical mass dampers used to control vertical oscillation, frequencies between 1,2 to 2,0Hz	Vibrations became imperceptible for users
Forchheim Bridge, Germany	1 span, 117,5	Cable-stayed	1,0 to 3,0	Vertical	1 TMD	
Solférino Bridge, Paris	central span, 106	Arch	0,81 1,94 2,22	Lateral Vertical Vertical	1 lateral TMD with mass 15000kg and 2 vertical TMDs with masses 10000kg and 7600kg	Increased structural damping from 0,4% to 3,5% (lateral), and from 0,5% to 3% and 2% (vertical)

Bridge	Number of spans / length [m]	Type	Controlled frequencies [Hz]	Dominant vibration direction	Type of damping system implemented	Effect of the damping system on the overall behaviour
Pedro e Inês Bridge, Coimbra	central span, 110	Shallow arch / girder	0,85 1,74; 1,80;2,34; 2,74; 3,07; 3,17	Lateral Vertical	1 lateral TMD with 14800kg and 6 vertical TMDs	Increased lateral damping from 0,5% to 4% and vertical damping from 0,3%-2,2% to 3%- 6%

Note: More details about the devices described below are presented the Background Document [1].

6.4.3.1 Viscous dampers

Viscous dampers (cf. Figure 6-1) are devices used to dissipate vibrations through the deformation of a viscous fluid or solid material.

One of the most common devices consists on a piston inside a cylinder, which causes deformation and flow of a fluid and generation of heat. The output force of such devices depends on the viscosity of the fluid and is proportional to the relative velocity of both ends, therefore the efficiency of a viscous damper depends on the possibility of installation of the damper connecting points of the structure with significant relative velocity. In some configurations, the motion of the piston induces motion of the fluid through calibrated openings, in which case dissipation occurs by volume variation of fluid. This latter type of damper is less dependent on temperature than the former, which relies essentially on the viscous properties of the fluid.

Viscous elastic dampers constitute a different category of dampers which dissipate energy by shearing deformation of a solid material, normally a polymer.

Figure 6-1: An example of installed viscous dampers

6.4.3.2 Tuned mass dampers

Tuned Mass Dampers (TMDs) consist of concentrated masses that are connected to a structure via some stiffness and damping elements. These devices are designed to split the critical frequency into two new frequencies (one below and another above the initial one), and the relative movement between structure and TMD allows for energy dissipation. Since the structural mass is much higher than the TMDs', the movement of the TMD usually comprises large displacements when compared to the structure motion.

Figure 6-2 shows two examples of TMDs, one vertical and one horizontal, installed in footbridges.

Figure 6-2: Examples of installed TMDs

6.4.3.3 Pendulum dampers

Pendulum dampers (cf. Figure 6-3) are a specific type of tuned mass dampers, which are used for suppressing horizontal vibrations. The main difference with a TMD is that no springs are used, except in cases when the frequencies to suppress are higher than 1 Hz. The mass is hung by truss elements, which reduces friction forces when compared to a normal horizontal support.

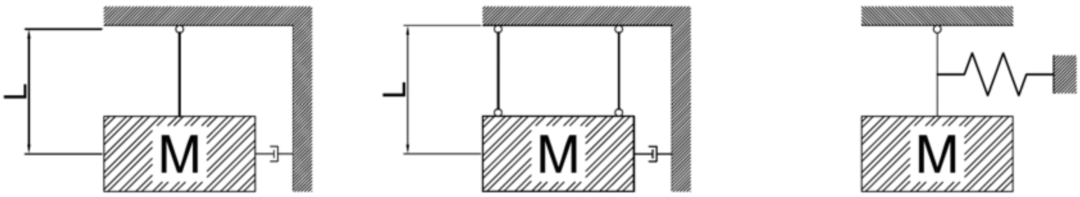


Figure 6-3: Example of pendulum systems

6.4.3.4 Tuned liquid column dampers

A Tuned Liquid Column Damper (TLCD) consists of U-shaped tube (cf. Figure 6-4), filled with a fluid (usually water), which properties are tuned in such a way that the forces at the base of the device, resultant from the movement of the liquid, counteract the horizontal movement of its support. This principle is therefore identical to that of a TMD. However, there are several advantages over other types of damping devices, such as easy tuning of frequency and damping, simple accommodation, simple construction and almost zero maintenance costs.

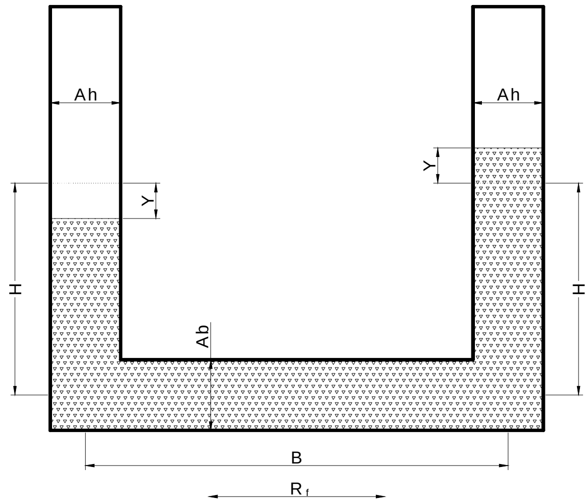


Figure 6-4: TLCD scheme

The optimum damping of the TLCD should be the same as the analogue TMD. The TLCD has intrinsic damping due to fluid turbulence and, by inserting a control valve or an orifice plate in the horizontal tube, the TLCD damping can be further enhanced. However, there is no specific literature with information concerning the quantification of TLCD damping, so it must always be obtained from tests on the TLCD prototypes.

6.4.3.5 Tuned liquid dampers

Tuned Liquid Dampers (TLDs) are passive control devices that consist of rigid tanks filled with liquid (cf. Figure 6-5) to suppress horizontal vibration of structures. Advantages like low cost, almost zero trigger level, easy adjustment of natural frequency and easy installation on existing structures have promoted an increasing interest in these devices. However, motion of the fluid can be highly nonlinear, since breaking of waves can occur for high vibration amplitudes.

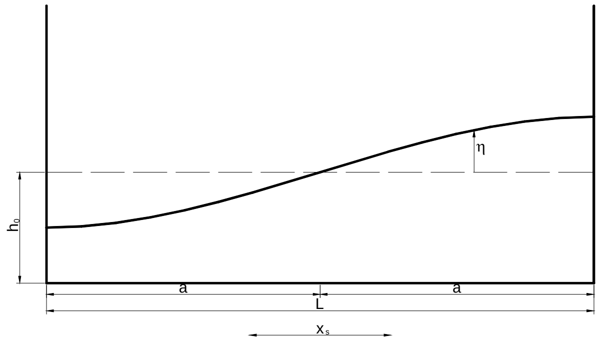


Figure 6-5: TLD scheme

7 References

[1] HiVoSS (Human induced Vibrations of Steel Structures): Design of Footbridges – Background document, September 2008.