JRC Scientific and Technical Reports

Design of Lightweight Footbridges for Human Induced Vibrations

Christoph Heinemeyer, Christiane Butz, Andreas Keil, Mike Schlaich, Arndt Goldack, Stefan Trometer, Mladen Lukić, Bruno Chabrolin, Arnaud Lemaire, Pierre-Olivier Martin, Álvaro Cunha, Elsa Caetano

Background document in support to the implementation, harmonization and further development of the Eurocodes

Joint Report

Prepared under the JRC – ECCS cooperation agreement for the evolution of Eurocode 3 (programme of CEN / TC 250)

Editors: G. Sedlacek, Chr. Heinemeyer, Chr. Butz together with M. Géradin

JRC First Edition, May 2009

EUR 23984 EN

2009

JRC Scientific and Technical Reports

Design of Lightweight Footbridges for Human Induced Vibrations

Christoph Heinemeyer, Christiane Butz, Andreas Keil, Mike Schlaich, Arndt Goldack, Stefan Trometer, Mladen Lukić, Bruno Chabrolin, Arnaud Lemaire, Pierre-Olivier Martin, Álvaro Cunha, Elsa Caetano

Background document in support to the implementation, harmonization and further development of the Eurocodes

Joint Report

Prepared under the JRC – ECCS cooperation agreement for the evolution of Eurocode 3 (programme of CEN / TC 250)

Editors: G. Sedlacek, Chr. Heinemeyer, Chr. Butz together with M. Géradin, JRC First Edition, May 2009

EUROPEAN COMMISSION

ECCS CECM EKS

The mission of the JRC-IPSC is to provide research results and to support EU policy-makers in their effort towards global security and towards protection of European citizens from accidents, deliberate attacks, fraud and illegal actions against EU policies.

European Commission
Joint Research Centre
Institute for the Protection and Security of the Citizen

The European Convention for Constructional Steelworks (ECCS) is the federation of the National Associations of Steelworks industries and covers a worldwide network of Industrial Companies, Universities and Research Institutes.

http://www.steelconstruct.com/

Contact information

Address: Mies-van-der-Rohe-Straße 1, D-52074 Aachen

E-mail: sed@stb.rwth-aachen.de

Tel.: +49 241 80 25177

Fax: +49 241 80 22140

http://ipsc.jrc.ec.europa.eu/
http://www.jrc.ec.europa.eu/

Legal Notice

JRC 53442

EUR 23984 EN

ISBN 978-92-79-13387-9

ISSN 1018-5593

DOI 10.2788/33846

Luxembourg: Office for Official Publications of the European Communities

Printed in Italy

Acknowledgements

This report is based on the results of two European research projects funded by the Research Fund for Coal and Steel (RFCS), namely

- Advanced Load Models for Synchronous Pedestrian Excitation and Optimised Design Guidelines for Steel Footbridges – **SYNPEX** prepared by RWTH-Aachen University, Centre Technique Industriel de la Construction Métallique, TICM, Faculdade de Engenharia da Universidade do Porto and Schlaich Bergermann und Partner

and

- Human induced vibrations of steel structures – **HIVOSS** prepared by Arcelor Mittal, TNO, SCI, RWTH-Aachen University, CTICM, FEUP and Schlaich Bergermann und Partner

The project partners gratefully acknowledge the financial contributions of RFCS as well as their agreement to publish the results in a "JRC-Scientific and Technical Report" to support the maintenance, further harmonisation, further development and promotion of the Eurocodes.

Foreword

(1) The EN Eurocodes are a series of European Standards which provide a common series of methods for calculating the mechanical strength of elements playing a structural role in construction works, i.e. the structural construction products.

They make it possible to design construction works to check their stability and to give the necessary dimensions of the structural construction products.

(2) They are the result of a long procedure of bringing together and harmonizing the different design traditions in the Member States. In the same time, the Member States keep exclusive competence and responsibility for the levels of safety of works.

(3) According to the Commission Recommendation of 11 December 2003 on the implementation and use of Eurocodes for construction works and structural construction products, Member States should undertake research to facilitate the integration into the Eurocodes of the latest developments in scientific and technological knowledge. Member States should pool the national funding available for such research so that it can be used at Community level to contribute to the existing technical and scientific resources for research within the European Commission, in cooperation with the Joint Research Centre, thus ensuring an ongoing increased level of protection of buildings and civil works.

(4) This report is a part of activities for the maintenance, further harmonisation, further development and promotion of the Eurocodes. It has been prepared in a field where so far no unified design rules applicable across different materials and ways of construction exists in the Eurocodes, namely in the field of limiting human induced vibrations of foot bridges.

It is a contribution to the evolution of EN´s 1990 and 1991 and may also be used together with some specific rules in EN 1995-2 that apply to timber bridges as a source of support to

- further harmonize design rules across different materials, and
- further develop the Eurocodes.

(5) The rules for the "Design of lightweight footbridges for human induced vibrations" given in this report are the result of projects funded by the Research Fund for Coal and Steel (RFCS), initiated and carried out by a group of experts from RWTH Aachen, Germany, Arcelor Mittal,

Luxembourg, TNO, The Netherlands, SCI, United Kingdom, CTICM, France, FEUP Porto, Portugal and Schlaich, Bergermann und Partner, Stuttgart, Germany.

(6) The agreement of RFCS and of the project partners to publish this report in the series of the "JRC-Scientific and Technical Reports" in support of the further development of the Eurocodes is highly appreciated.

(7) The examples given in this guideline are mainly from light-weight steel structures, where the consideration of human induced vibrations is part of the optimization strategy of the structure. Therefore the publication has been carried out in the context of the JRC-ECCS-cooperation agreement in order to support the further harmonisation of National procedures and the further evolution of the Eurocodes.

Aachen, Delft, Paris and Ispra, May 2009

Gerhard Sedlacek

ECCS-Director of Research

Frans Bijlaard

Chairman of CEN/TC 250/SC3

Jean Armand Calgaro

Chairman of CEN/TC 250

Michel Géradin, Artur Pinto, Silvia Dimova and Humberto Varum

European Laboratory for Structural Assessment, IPSC, JRC

Table of Contents

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 a considered mode	[Hz]
f_s	step frequency of a pedestrian	[Hz]
P	force amplitude due to a single pedestrian	[N]
$P \times \cos(2\pi f t)$	harmonic load due to a single pedestrian	[N]
L	length	[m]
k_i*	modal stiffness	[N/m]
m_i*	modal mass	[kg]
M	mass	[kg]
N	number of the pedestrians on the loaded surface S ($N = S \times d$)	[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 in 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.

The fundamentals of these rules and of the accompanying background information (presented in grey background) were prepared by:

- Christiane Butz and Christoph Heinemeyer from RWTH Aachen University, Germany,
- Andreas Keil, Mike Schlaich, Arndt Goldack and Stefan Trometer from Schlaich Bergermann und Partner, Germany,
- Mladen Lukić, Bruno Chabrolin, Arnaud Lemaire and Pierre-Olivier Martin, from Centre Technique Industriel de la Construction Métallique, France,
- Álvaro Cunha and Elsa Caetano from Faculdade de Engenharia da Universidade do Porto, Portugal.

The editing of this report was made by Christoph Heinemeyer, Christiane Butz and Gerhard Sedlacek together with Michel Géradin, JRC.

Background information

In recent years, there has been a growing trend towards the construction of lightweight footbridges. Due to the increased flexibility of such structures, the dynamic forces can cause larger amplitudes of the vibration. The more slender the structures become, the more attention must be paid to vibration phenomena.

The increase of vibration problems in modern footbridges shows that footbridges should no longer be designed for static loads only. But fulfilling the natural frequency requirements that are given in many codes (**[1]**, **[2]**, **[3]**, **[4]**) restricts footbridge design: very slender, lightweight structures, such as stress ribbon bridges and suspension bridges may not satisfy these requirements. Moreover not only natural frequencies but also damping properties, bridge mass and pedestrian loading altogether determine the dynamic response. The design tools should consider all of these factors. Provided that the vibration behaviour due to expected pedestrian traffic is checked with dynamic calculations and satisfies the required comfort, any type of footbridge can be designed and constructed. If the vibration behaviour does not satisfy some comfort criteria, changes in the design or damping devices could be considered.

These lightweight footbridges have lower natural frequencies, resulting in a greater risk of resonance. Resonance occurs if one natural frequency of the bridge coincides with the frequency of the excitation, e.g. the step frequency of pedestrians. Pedestrian induced excitation is an important source of vibration of footbridges. Pedestrian loading is by nature unsteady, transient and waddling in a small range of excitation frequency. It is therefore obvious that dynamic responses play a fundamental role in the design of vibration susceptible structures. Vibrations of footbridges may lead to serviceability problems, as effects on the comfort and emotional reactions of pedestrians might occur. Collapse or even damage due to human induced dynamic forces have occurred very rarely.

Vibrations of footbridges may occur in vertical and horizontal directions, even torsion of the bridge deck is possible. Dynamic actions of cyclists are negligible compared to the actions caused by walking and running individuals.

In recent years some footbridges were excited laterally by dense pedestrian streams in which pedestrian motion and bridge vibration were strongly coupled, in which case a self-excited response of large amplitude may take place and cause discomfort. Footbridges should be designed in such a way that this pedestrian-bridge-interaction phenomenon, also called 'lock-in', does not arise.

Another dynamic loading on footbridges is intentional excitation by people that are jumping on the spot, bouncing, swaying body horizontally, shaking stay cables etc. at resonance frequency to produce large vibrations. In that case, the comfort is certainly not fulfilled but the structure must not collapse.

Hence, in modern footbridge design, the assessment of human-induced vibrations needs to be considered by the designer to ensure that

- Vibrations due to pedestrian traffic is acceptable for the users,
- The lock-in phenomenon does not arise,
- The footbridge does not collapse when subjected to intentional excitation.

In order to help the bridge designer, dynamic response of various footbridges under pedestrian loading was investigated by means of measurements and numerical simulations, providing these design guidelines which include

- Design requirements,
- Comfort range in terms of acceleration,
- Load models for pedestrian streams,
- Criterion to avoid lock-in phenomenon.

If a footbridge is susceptible to vibrations that might affect the comfort, additional information is given concerning

- Measurement procedures and methods for evaluation of dynamic properties,
- Design modification and introduction of damping devices.

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 $\mathrm{d}v / \mathrm{d}t$ or $\mathrm{d}^2x / \mathrm{d}t^2$), 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 energy dissipation.
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. The total damping in a structure consists of • Material and structural damping, • Damping associated to furniture and finishing, • Energy dissipation through special devices.
Dynamic action	Action that causes significant time-dependent force input to the structure or structural members
Modal mass = generalised mass	A multiple degree of freedom system can be reduced to an uncoupled set (*i=1,N*) of single degree of freedom (SDOF) systems with same natural frequencies as the original system: $f_i^* = \frac{1}{2\pi}\sqrt{\frac{k_i^*}{m_i^*}}$ where f_i^* is the i^{th} natural frequency, expressed in Hz k_i^* is the modal stiffness of mode *i* m_i^* is the modal mass of mode *i*. Thus, the modal mass can be interpreted to be the mass activated in a specific mode of vibration.
Mode of vibration	A characteristic pattern assumed by a vibrating system in which every particle is in synchronous harmonic motion with the same frequency. Two or more modes may coexist in a multiple degree of freedom system.
Natural frequency = eigenfrequency	A natural frequency is a 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 period 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 dependent 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 their 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.

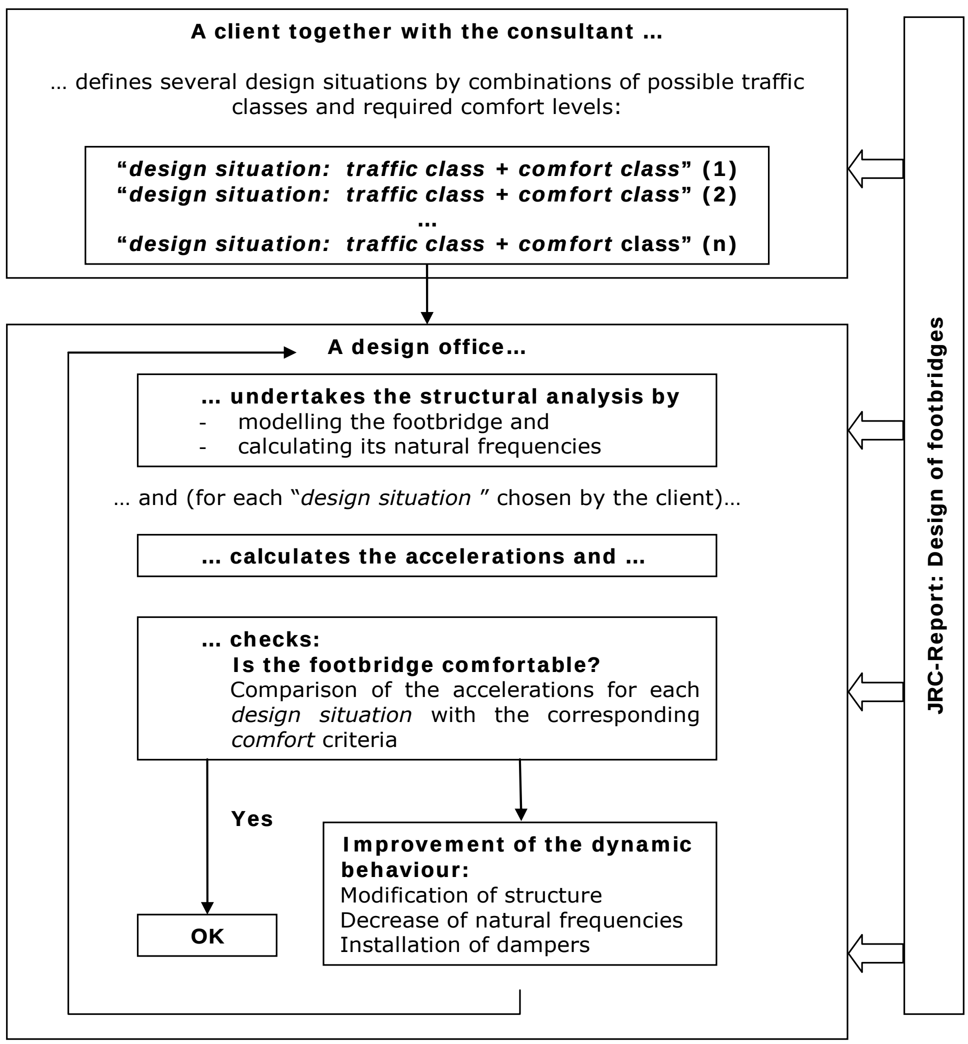


Figure 3-1: Methodology for the design

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.

Background information:

It is recommended to consider dynamic actions and the vibration behaviour of the structure in an early design stage, even when damping and some foundation properties are unknown and have to be estimated. Hence, the calculated vibration behaviour gives only an indication of the real behaviour. If the response is in the critical range, provisions for change structural properties or introduction of damping devices should be made in the early design stage. Damping and accelerations caused by several dynamic loads should then be measured after finishing the construction. Based on the real dynamic properties it should be decided whether the damping devices are necessary or not.

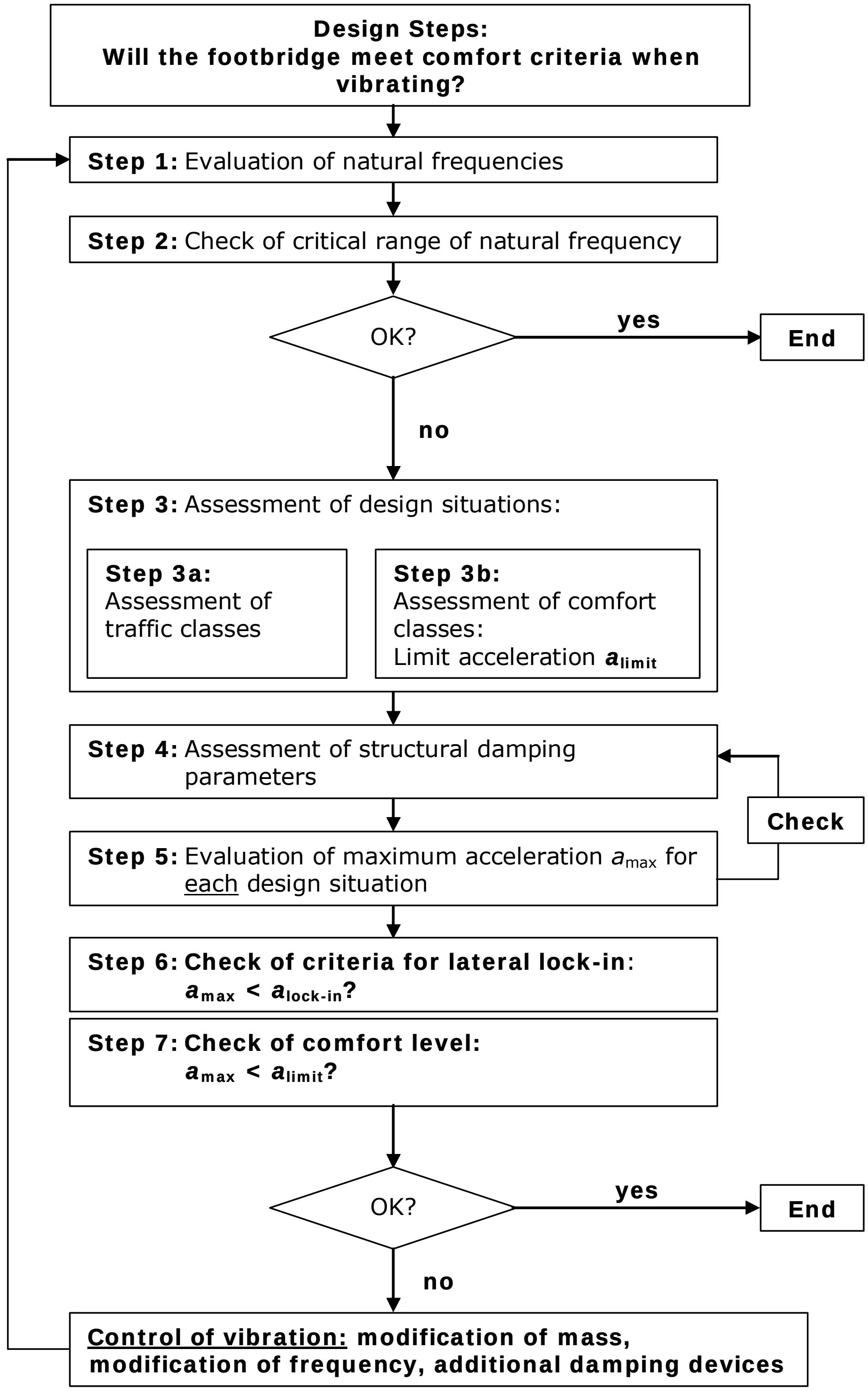


Figure 3-2: Flowchart for the use of this guideline

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.:

- By the finite element (FE) method
- Using hand formulas derived e.g. from closed-form solutions 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.

Background information:

Although hand formulas and simplified methods can be used in a preliminary evaluation of natural frequencies, whenever these are close to a critical range from the point of view of pedestrian excitation, a more precise numerical model should be used. In modern bridge design the use of finite element software is widely spread in all stages of design, even during the conceptual one. Consequently, it is suggested to use a FE-Model of the bridge not only to calculate the stress distribution and deformation of the footbridge but also to determine its natural frequencies. Hence, preliminary dynamic calculations can easily be performed without additional means.

A first approach is to keep the model as simple as possible and to model the bridge with beam elements, cable elements, spring or truss elements in a three dimensional FE model. The latter should always allow for vertical, horizontal, and torsional mode shapes. A rough overview over the natural frequencies and the corresponding mode shapes is obtained and problems regarding the dynamic behaviour can be identified. The more complex the static system and the higher the mode shape order, the more finite elements are required. A more refined model may take advantage of various types of finite elements such as plate, shell, beam, cable or truss elements. To get reliable results for natural frequencies, it is absolutely necessary that bearing conditions, foundation stiffness, stiffness and mass distribution are modelled 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. The superimposed dead load of the bridge caused by furniture, barriers, pavement and railings is considered as additional mass distribution as exactly as possible. A lumped mass approach, in which rotational masses are neglected, is in many cases sufficient. For the modelling of abutments and foundations, dynamic soil stiffness should be used. Otherwise the obtained results will be very conservative or very inaccurate.

In any case it is recommended to determine first and foremost the natural frequencies of a built footbridge by experimental investigation in addition to computer calculations before the final configuration of the damping units are determined.

The modal mass for each mode shape should be available, when verification of comfort is done with the SDOF-method (cf. section 4.5.1.2).

The investigation of dynamic characteristics for selected footbridges shows clearly that, especially for lightweight structures, the additional mass due to pedestrians has a great influence on the natural frequencies of the system. For individuals and group loading this effect is usually negligible, but if pedestrian streams have to be taken into account, this influence may cause a significant decrease in natural frequency. This depends on the ratio between mass distribution of the deck and pedestrian mass distribution. The decrease in frequencies is higher for footbridges having less dead load.

The natural frequencies might fall to a more or to a less critical frequency range (cf. section 4.2) for pedestrian induced dynamic excitation. With additional dead load or live load, the natural frequencies of the footbridge could decrease and shift into the critical frequency range or leave it. Furthermore, it has to be noted that the given limit values of critical frequency ranges should not be taken as sharp values but rather as soft values.

In some cases the obtained increase of modal mass can be even greater than 50 % of the modal mass of the bridge.

The influence of the static pedestrian mass can be estimated easily: the modal mass m^{**} including the additional static pedestrian mass is calculated according to eq. 4-1.

$$m^{**} = \rho \int_{L_D} \mu_D \left(\Phi(x)\right)^2 dx = \rho m^*$$

Eq. 4-1

where

μ_D [kg/m] is the bridge deck mass per unit length

$\rho = \dfrac{\mu_D + \mu_P}{\mu_D}$ is the influence factor for additional pedestrian mass

μ_P [kg/m] is the pedestrian mass per unit span length

$\phi(x)$ is the mode shape

An answer to the question of the threshold of taking the additional pedestrian mass into account can be given by eq. 4-2, which shows that the influence of a 5 % higher modal mass results in a decrease of the natural frequency by 2,5 %.

$$f'(\rho = 1,05) = \sqrt{\frac{k^*}{\rho \cdot m^*}} = \sqrt{\frac{k^*}{1,05 m^*}} = 0,976 f \qquad \text{Eq. 4-2}$$

This is within the accuracy of the whole model compared to the natural frequencies that will be measured in reality. Therefore, it is recommended to neglect the influence of an increased modal mass lower than 5 % on the natural frequency.

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 in the range

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

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 affected 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 significant vibration of footbridges due to the second harmonic of pedestrians have occurred.

Background information:

Pedestrian effects are generally characterised on the basis of harmonic load models which coefficients are systematised in Section 9. The dominant contribution of the first harmonic leads to the following critical range for natural frequencies f_i:

- for vertical and longitudinal vibrations:

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

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

There are situations in which natural frequencies lie in an interval susceptible of excitation by the second harmonic of pedestrian excitation. Under these circumstances, if it is considered relevant to investigate the effects associated with the second harmonic of pedestrian loads, the critical range expands to:

- for vertical and longitudinal vibrations:

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

Footbridges which have natural frequencies f_i in the critical range should be subject to a dynamic assessment to pedestrian excitation.

Lateral vibrations are not affected 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 significant vibration of footbridges due to the second harmonic of pedestrians have occurred.

The critical range of natural frequencies is based on empirical investigation of the step frequencies f_s of pedestrians. In order to be coherent with the Eurocodes principles, the characteristic values $f_{s,5\%,\mathrm{slow}}$ and $f_{s,95\%,\mathrm{fast}}$ used are based on the 5th and 95th percentile values.

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 4.3.1) 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 situations 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 the different probable design situations are specified. An example of this is given in Table 4-2.

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

Design Situation	Description	Traffic Class (cf. 4.3.1)	Expected occurrence	Comfort Class (cf. 4.3.2)
1	inauguration of the bridge	TC4	once in the lifetime	CL3
2	commuter traffic	TC2	daily	CL1
3	rambler at the weekend	TC1	weekly	CL2
⋮	⋮	⋮	⋮	⋮

Background information:

It is strongly recommended to discuss comfort requirements and expected pedestrian traffic – in relation to the obtained dynamic response – with the owner to develop realistic limits and boundary conditions for the design of the particular structure. A constructive dialogue about the vibration susceptibility between the designer and the owner may help clarifying issues such as comfort requirements and the potential need for damping measures (cf. section 6).

Eurocode principles for reliability **[5]** state some design situations out of which the ones listed below could be relevant for footbridges subjected to pedestrian loading. They can be associated with the frequency of exceeding a certain limit state like a comfort criterion in question:

- Persistent design situations, which refer to the conditions of permanent use
- Transient design situations, which refer to temporary conditions
- Accidental design situations, which refer to exceptional conditions.

There are design situations which might occur once in the lifetime of a footbridge like the inauguration of the bridge. But on the other hand there might be a design situation where few commuters will pass daily.

Realistic assumptions of the different design situations should be taken into account by using defined traffic classes (cf. section 4.3.1) for the verification of pedestrian comfort. As aforesaid, an event such as the inauguration of the footbridge could by itself govern entirely although it happens only once in the lifetime of the bridge. It must therefore be decided which comfort criteria are to be taken into account in the footbridge design (cf. section 4.3.2) for an extreme and rare situation such as the inauguration and for the everyday density of pedestrians on the structure.

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
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.

Background information:

The expected type of pedestrian traffic and traffic density governs the dynamic loading and influences the design of footbridges. Structures in more remote locations with sparse pedestrian traffic are not subjected to the same dynamic loading as those in city centres with dense commuter traffic.

Pedestrian formations, processions or marching soldiers are not taken into account in the general traffic classification, but need additional considerations. The difference between pedestrian formations and the aforesaid pedestrian traffic is that all pedestrians of the formation march in time. The pace is thus highly synchronized and may be enforced by music.

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.

Background information:

Criteria for pedestrian comfort are most commonly represented as limit acceleration for the footbridge. National and international standards as well as literature propose limit values which differ among themselves for many reasons. Nevertheless, most of these values coincide within a certain bandwidth.

Generally, the perception and assessment of motion and vibration are subjective and therefore different for each pedestrian. Users of pedestrian bridges that are located near hospitals and nursing homes may be more sensitive to vibrations than hikers crossing a pedestrian bridge along a hiking trail.

Even the visual appearance and the location of the bridge may influence the assessment by each pedestrian. Figure 4-1 shows the bandwidth of personal subjective perception regarding bridge vibration. Although the two analysed bridges have very similar dynamic properties the vibration assessment of the questioned persons differs greatly. The percentage of individuals feeling disturbed while crossing the sturdier-looking Wachtelsteg Footbridge, Pforzheim, Germany, on the right, is 4 times higher than for the lighter-looking Kochenhofsteg Footbridge, Stuttgart, Germany, on the left. Similarly, the likeliness that a person is excited or amused by the vibrations is nearly 3 times higher in the second case.

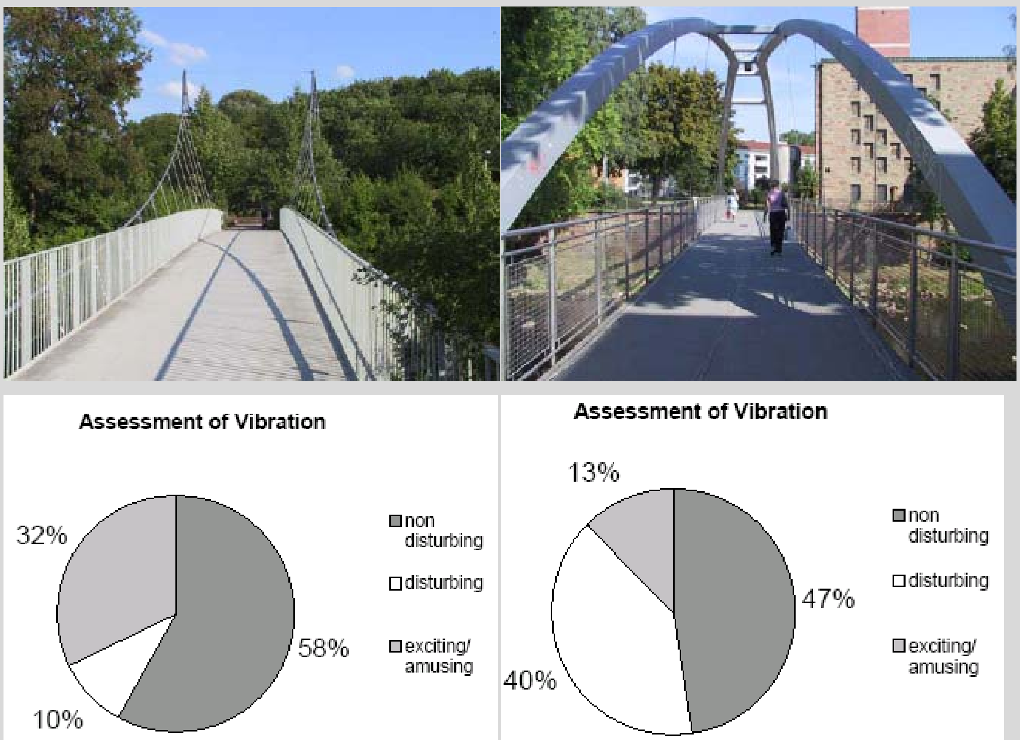


Figure 4-1: Comparison of vibration assessment of two footbridges

Hence, the assessment of 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
- Expectancy of vibration due to bridge appearance.

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 vibration level, 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.

Background information:

Considering that civil engineering structures are normally lightly damped and develop low levels of stress under service loads, the hypothesis of linear behaviour is normally accepted. The combination of this hypothesis with the assumption of a damping distribution along the structure characterised by a damping matrix C proportional to the mass *M* and stiffness *K* matrices (Rayleigh damping assumption)

$$C = \alpha \cdot M + \beta \cdot K \qquad \text{Eq. 4-3}$$

allows a decoupling of the dynamic equilibrium equations and the use of the modal superposition analysis in the evaluation of dynamic effects induced by pedestrians. Idealising the N-degrees-of-freedom system as N single-degree-of-freedom (SDOF) systems (cf. section 4.5.1.2), a set of N damping ratios ξ_n are defined, which represent the fraction of the damping of a mode of order n to the critical damping, defined as a function of the modal mass m_n* and of the circular frequency ω_n

$$\xi_n = C_n / 2m_D^* \cdot \omega_n \qquad \text{Eq. 4-4}$$

These damping ratios relate to the constants α and β in eq. 4-3 by

$$\xi_n = \frac{1}{2}\left(\frac{\alpha}{\omega_n} + \beta\omega_n\right) \qquad \text{Eq. 4-5}$$

Therefore, by fixing two values of ξ_n associated with two different modes, a damping matrix can be obtained. These values are normally based on past experience in the construction of structures of the same type and material.

4.4.2 Damping ratios for service loads

For the design of footbridges for adequate 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	0,8%	1,3%
Prestressed concrete	0,5%	1,0%
Composite steel-concrete	0,3%	0,6%
Steel	0,2%	0,4%
Timber	1,0%	1,5%
Stress-ribbon	0,7%	1,0%

Background information:

Values comparable to these from Table 4-5 are proposed by the SETRA/AFGC guidelines [9], by Bachmann and Amman [10], by EN 1991 [11] and by EN 1995 [12].

Figure 4-2 and Figure 4-3 summarise the variation with frequency and span, respectively, of measured damping ratios on various footbridges within the

SYNPEX Project [13]. These figures include also additional data published in the literature. Despite the large scatter, it is shown that numerous steel bridges exhibit damping ratios lower than 0,5% for natural frequencies that are critical from the point of view of pedestrian excitation.

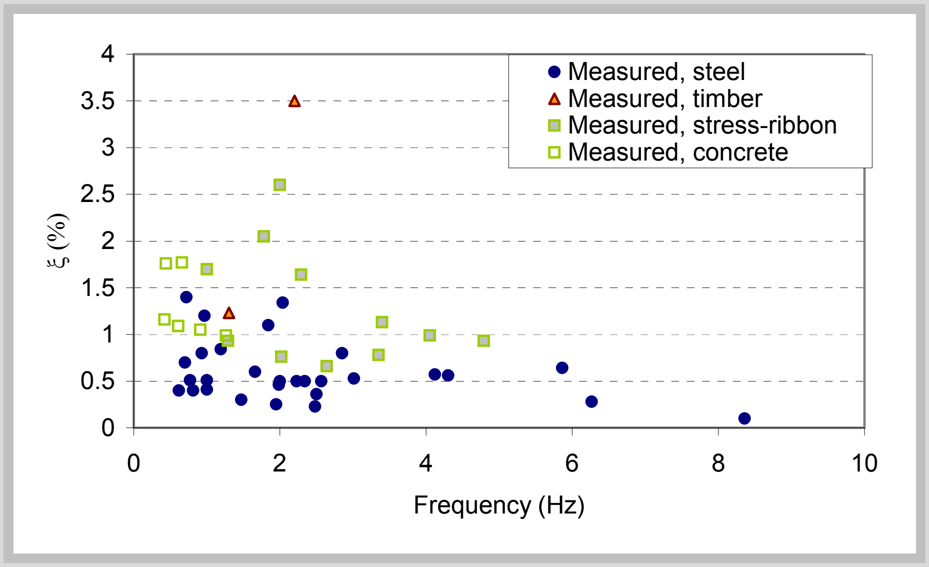


Figure 4-2: Measured damping ratios under service loads: variation with natural frequency

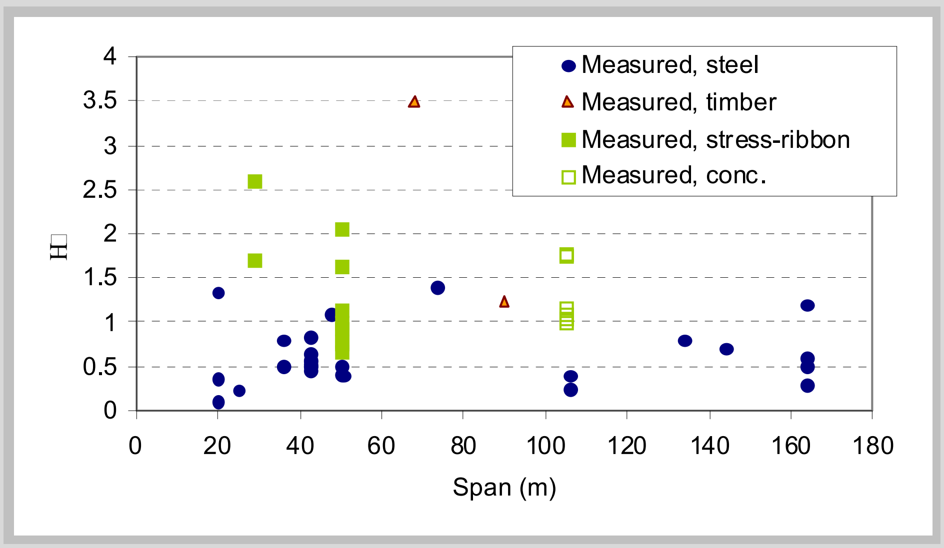


Figure 4-3: Measured damping ratios under service loads: variation with span

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 ratio 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%

Background information:

EN 1998 [14] gives the range of structural damping ratios for dynamic studies under earthquake loads. These values can be used as a reference for large amplitudes.

Table 4-7: Damping ratios according to EN 1998[14] for dynamic responses under earthquake loads

Construction type	Interval of variation of damping ratio ξ
Concrete	2,0 ÷ 7,0%
Steel	1,0 ÷ 4,0%

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 bridges. This design guideline recommends using one of the methods shown in Figure 4-4, which will be discussed in the following chapters.

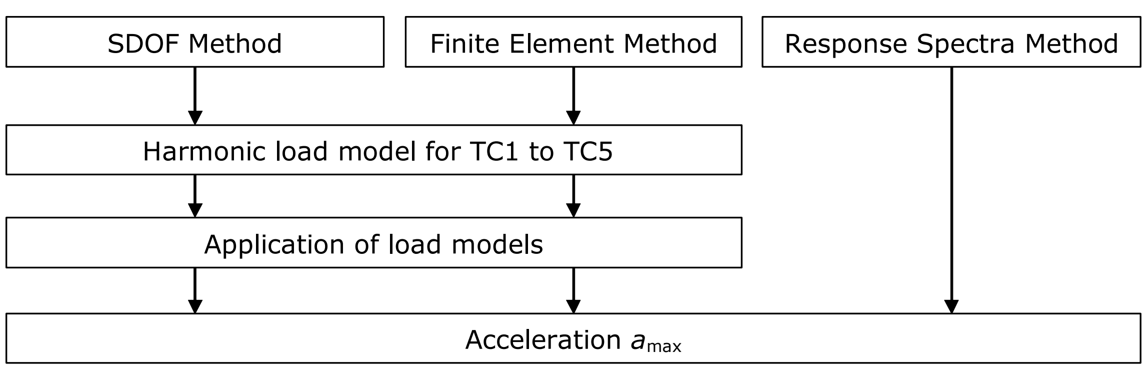


Figure 4-4: Methods for calculating the maximum acceleration

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.

Background information:

Footbridges are in reality most often subjected to simultaneous action of several pedestrians whereas this action is not simply the sum of individual actions of single pedestrians. Hence, pedestrian loads on bridges are stochastic loads. Depending on the density of pedestrians on a bridge, pedestrians walk more or less synchronously and possibly interact with a vibrating footbridge.

The loading depends on the density of pedestrian streams, the individual pace frequency, the track people are walking, the synchronisation of people walking, persons' weight, etc. The system answer depends on the loading and on structural properties as (modal) mass of the bridge, natural frequencies and damping. As it is not possible to determine structural properties as e.g. frequencies and damping without uncertainties, there is also uncertainty on the calculated system response.

There are various methods for calculating the acceleration of the bridge. The ones recommended within the current document will be discussed in the following sections.

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 4-5). 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.

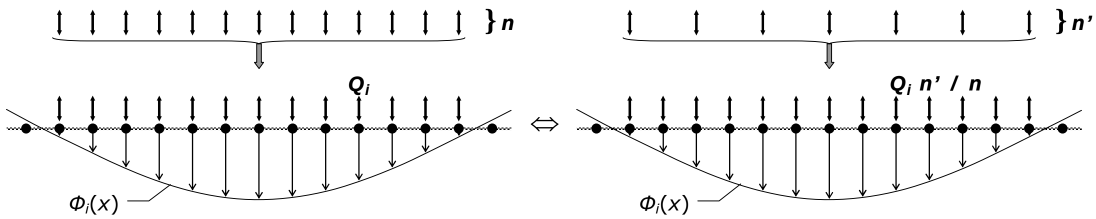


Figure 4-5: Equivalence of streams

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.

Background information:

Introduction

If a harmonic load ($F_0 \cdot \sin(2\pi f_0 \cdot t)$) is applied to a damped SDOF system, the response of the system can then be given in the form which will be used throughout the procedure for the assessment of an equivalent number n' of pedestrians using modal analysis:

$$x(t) = \frac{F_0 / 4\pi^2 M}{\sqrt{\left(f^2 - f_0^2\right)^2 + 4\xi^2 f^2 f_0^2}} \sin(2\pi f_0 t - \varphi) \qquad \text{Eq. 4-6}$$

with: F_0 amplitude of the harmonic load,

M system mass,

f system natural frequency,

f_0 harmonic load frequency,

ξ structural damping ratio,

$\varphi = \arctan\left(\frac{2\xi f f_0}{f^2 - f_0^2}\right)$ phase shift.

Modal analysis

Let a beam be modelled as a system with N degrees of freedom (cf. Figure 4-6) and let a loading be represented as point loads on each of the (loaded) nodes. When a solution to describe the dynamic behaviour of a system is sought by modal analysis, displacements of the nodes are found in the form of superposition of displacements belonging to different vibration modes:

$$y(t) = \sum_{i=1}^{r} x_i(t)\Phi_i \quad , \quad r \le N \qquad \text{Eq. 4-7}$$

where:

$y(t)$ is the vector of the displacements of the concentrated masses,

ϕ_i are the vectors of modal displacements taken into consideration,

$x_i(t)$ are the responses of the system for each mode i taken into consideration.

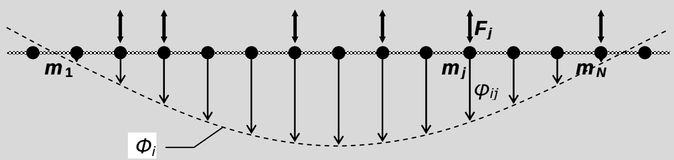


Figure 4-6: $n \leq N$ harmonic loads

If all the loads share the *same* frequency, $f_0 \neq f_i$, the response of the system for one mode only (e.g. mode i, with modal displacements φ_{ij}, cf. Figure 4-6) is:

$$x_i(t) = \frac{\Phi_i^T F_0 / 4\pi^2 m^*_i}{\sqrt{\left(f_i^2 - f_0^2\right)^2 + 4\xi_i^2 f_i^2 f_0^2}} \sin\left(2\pi f_0 t - \varphi_i\right) \qquad \text{Eq. 4-8}$$

with: $\phi_i^T = \{\varphi_{i1}, \varphi_{i2}, \ldots \varphi_{ij}, \ldots \varphi_{iN}\}$ vector of modal displacements,

F_0 vector of load amplitudes ($F_0^T = \{F_1, F_2, \ldots F_j, \ldots F_N\}$),

$m^*_i = \sum_{j=1}^{N} m_j \varphi_{ij}^2$ modal mass,

f_i frequency for mode i,

f_0 loading frequency,

ξ_i damping ratio for mode i,

φ_i phase shift for mode i.

Response to a distributed harmonic load – Deterministic approach

In the **most general case**, the distributed harmonic load is represented as $n = N$ point loads ($Q_j \sin(2\pi f_{0j} t - \psi_j)$), regularly distributed on half-waves of the mode ϕ_i (cf. Figure 4-7), where:

- The amplitudes of the loads are Q_j, j = 1 to n;
- Each point load has a frequency f_{0j}, j = 1 to n;
- Each point load has a phase shift ψ_j, j = 1 to n.

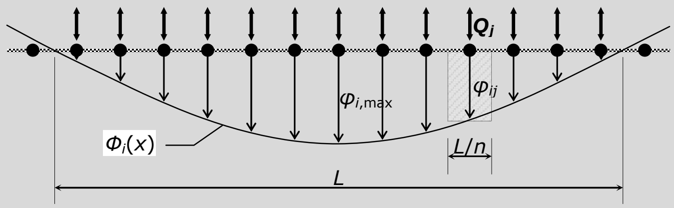


Figure 4-7: $n = N$ harmonic loads

If the loaded length is L, the position of each point load is found within the interval $\left[\frac{j-1}{n}L, \frac{j}{n}L\right]$ (cf. Figure 4-7). In order to take into account the mode rank and the distributed character of the loads:

$$\Phi_i^T F_0 = \sum_{j=1}^{N} \frac{\alpha_{nij}}{L} Q_j \ ,$$

where $\alpha_{nij} = n \int_{(j-1)L/n}^{jL/n} \Phi_i(x)dx$.

The response is found as a superposition of responses to particular loads as:

$$y_{i,\max}(t) = \sum_{j=1}^{n} \frac{\left(\frac{\alpha_{nij}}{L} Q_j \sin\left(2\pi f_{0j} t - \varphi_{ij}\right) / 4\pi^2 m^*_i\right)\varphi_{i,\max}}{\sqrt{\left(f_i^2 - f_{0j}^2\right)^2 + 4\xi_i^2 f_i^2 f_{0j}^2}} \ ,$$

where the phase shift for mode i and a point load at node j is:

$$\varphi_{ij} = \arctan\left(\frac{2\xi_i f_i f_{0j} \cos\psi_j + \left(f_i^2 - f_{0j}^2\right)\sin\psi_j}{\left(f_i^2 - f_{0j}^2\right)\cos\psi_j - 2\xi_i f_i f_{0j} \sin\psi_j}\right).$$

If the assumption that all the loads share the same amplitude but are not necessarily in phase ($Q_j = Q\sin\psi_j$) is adopted, the response becomes:

$$y_{i,\max}(t) = Q \sum_{j=1}^{n} \frac{\left(\alpha_{nij}\, \varphi_{i,\max} / 4\pi^2 m^*_i L\right)\sin\left(2\pi f_{0j} t - \varphi_{ij}\right)}{\sqrt{\left(f_i^2 - f_{0j}^2\right)^2 + 4\xi_i^2 f_i^2 f_{0j}^2}} \qquad \text{Eq. 4-9}$$

Response to a distributed harmonic load – Probabilistic approach

The effect of a pedestrian stream consisting of $n = N$ “random” pedestrians is now to be analysed. The differences comparing to the case given above are:

- Each point load has a random frequency f_{sj} which follows a normal distribution $N[f_{s1}, \sigma]$;
- Each point load has a random phase shift ψ_i which follows a uniform distribution $U[0, 2\pi]$;

- The response/displacement (eq. 4-9) is here a random variable, too – because of f_{sj} and ψ_j – and hence its mean value and its standard deviation could be assessed.

If the following notation is adopted:

- $\lambda_i = f_i / f_{s1}$ ratio between the natural frequency for mode i and the mean of the loading frequencies,
- $\mu = \sigma / f_{s1}$: coefficient of variation of the loading frequencies,
- $f_{sj} = f_{s1}\,(1 + \mu\, u_j)$: random frequency of a point load placed at a node j,

where u_j is a standardised normal random variable, and if – instead of displacements – accelerations are considered, each component of the sum in eq. 4-9 should be multiplied by:

$$\left(2\pi f_{sj}\right)^2 = \left(2\pi\right)^2 f_{s1}^2 \left(1 + \mu u_j\right)^2 .$$

The absolute maximum acceleration is then:

$$\ddot{Z}_i = \max_t \left[\ddot{y}_{i,\max}(t)\right] = (2\pi)^2 f_{s1}^2 \frac{Q}{f_{s1}^2} \times \underbrace{\max_t \left[\sum_{j=1}^{n} \sqrt{\frac{\left(\alpha_{nij}\, \varphi_{i,\max} / 4\pi^2 m^*_i L\right)^2 \left(1+\mu u_j\right)^4}{\left(\lambda_i^2 - 1 - 2\mu u_j - \mu^2 u_j^2\right)^2 + 4\xi_i^2 \lambda_i^2 \left(1 + 2\mu u_j + \mu^2 u_j^2\right)}} \times \sin\left(2\pi f_{sj}\, t - \varphi_{ij}\right) \right]}_{z_i},$$

with the phase shift for mode i and a point load at node j:

$$\varphi_{ij} = \arctan\left(\frac{2\xi_i \lambda_i \left(1+\mu u_j\right)\cos\psi_j + \left(\lambda_i^2 - \left(1+\mu u_j\right)^2\right)\sin\psi_j}{\left(\lambda_i^2 - \left(1+\mu u_j\right)^2\right)\cos\psi_j - 2\xi_i \lambda_i \left(1+\mu u_j\right)\sin\psi_j} \right) = \arctan\left(\frac{2\xi_i \lambda_i \left(1+\mu u_j\right)}{\left(\lambda_i^2 - \left(1+\mu u_j\right)^2\right)} \right) + \psi_j,$$

and, finally:

$$\ddot{Z}_i = (2\pi)^2 Q z_i \qquad \text{Eq. 4-10}$$

Note: For λ_i = 1, μ = 0 and ψ_j = 0 (deterministic resonant loading case):

$$\ddot{Z}_i = (2\pi)^2 f_{s1}^2 \frac{Q}{f_{s1}^2} \times \underbrace{\max_t \left[\sum_{j=1}^{n} \frac{\alpha_{Nij}\, \varphi_{i,\max} / 4\pi^2 m^*_i L}{2\xi_i} \sin\left(2\pi f_{sj}\, t - \frac{\pi}{2}\right)\right]}_{z_i'} = (2\pi)^2 Q z_i' \qquad \text{Eq. 4-11}$$

Determination of the equivalent number of pedestrians

The equivalent number of pedestrians in an equivalent, idealised stream – i.e. the number of pedestrians, all with footfalls in the natural frequency of the mode

i and with no phase shift causing the same behaviour of the structure as the one caused by the random stream of pedestrians – can be obtained by equalising the absolute maximum accelerations from the following two cases (cf. Figure 4-5):

Random stream with $n = N$ pedestrians (eq. 4-10): $\ddot{Z}_i = (2\pi)^2 \, Q \, z_i$

Equivalent stream with $n' \leq n$ pedestrians (eq. 4-11): $\ddot{Z}_{i,eq} = (2\pi)^2 \, Q \, z_i' \dfrac{n'}{n}$

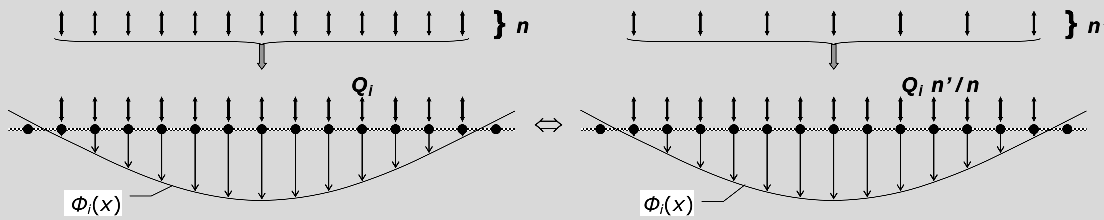


Figure 4-8: Equivalence of streams

Thus: $\ddot{Z}_i = \ddot{Z}_{i,eq} \;\Rightarrow\; z_i = z_i' \dfrac{n'}{n} \;\Rightarrow\; n' = \dfrac{z_i}{z_i'} n$

If the approach proposed in [9] is adopted,

$$n' = k_{eq} \sqrt{n \, \xi_i} \,, \qquad \text{Eq. 4-12}$$

and the coefficient k_{eq} can be obtained as follows:

$$\boxed{k_{eq} = \frac{n'}{\sqrt{n \, \xi_i}} = \frac{z_i}{z_i'} \sqrt{\frac{n}{\xi_i}}} \qquad \text{Eq. 4-13}$$

The random feature in equation 4-13 is z_i. The mean value $E(z_i)$ and the standard deviation $\sigma(z_i)$ can all be assessed by simulations for different values of intervening parameters:

$$z_i = \max_t \left(\sum_{j=1}^{n} \sqrt{ \frac{ \left(\alpha_{Nij} \, \varphi_{i,\max} \big/ 4\pi^2 \, m^*_i L \right)^2 \left(1 + \mu \, u_j \right)^4 }{ \left(\lambda_i^2 - 1 - 2\,\mu \, u_j - \mu^2 \, u_j^2 \right)^2 + 4 \, \xi_i^2 \, \lambda_i^2 \left(1 + 2\,\mu \, u_j + \mu^2 \, u_j^2 \right) } } \times \atop \times \sin\left(2\pi \, f_{sj} \, t - \varphi_{ij} \right) \right) \qquad \text{Eq. 4-14}$$

Results

Sensitivity analyses were done on the basis of Monte-Carlo simulations carried out on a half-sine mode shape Φ_i (cf. Figure 4-5) in order to represent the random nature of pedestrian loading. In those analyses, the following parameters have been varied:

- Damping ratio, ξ_i
- Ratio of frequencies, λ_i
- Coefficient of variation, μ
- Number of pedestrians, n.

Histograms of maxima of z_i (eq. 4-14) are firstly obtained on the basis of 2500 simulations for each set of parameters, every simulation consisting of taking n random values of both the standardized normal variable u_i and the phase shift ψ_i. A maximum of z_i is taken on a 2-period range (simulations carried out have shown that an 8-period range gives the same results). Coefficient k_{eq} is then calculated (eq. 4-13) on the basis of values of z_i obtained as explained above. Figure 4-9 gives an example of histogram of k_{eq}. Finally, 95th percentile of k_{eq} is determined.

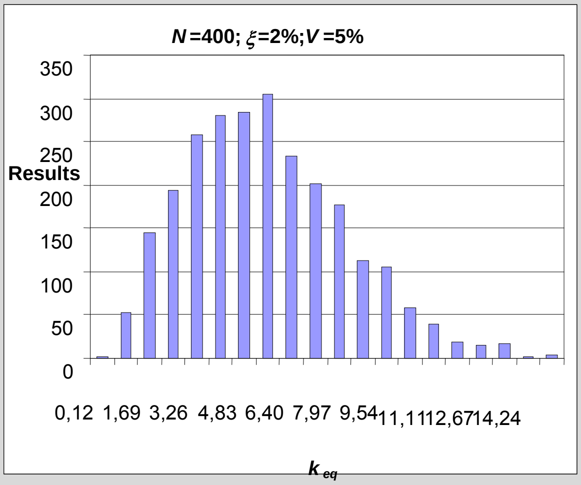


Figure 4-9: An example of resulting histogram

With such a value of k_{eq}, the equivalent number of pedestrians, n' can be obtained. Expressions for this equivalent number have been derived by regression as a function of the damping ratio and the total number of pedestrians on the footbridge.

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 ≥ 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:

$$p(t) = P \times \cos(2\pi f_s t) \times n' \times \psi \qquad \text{Eq. 4-15}$$

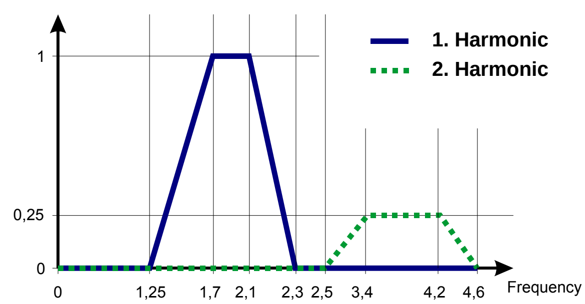
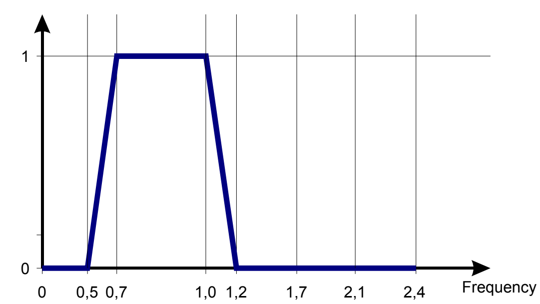
where $P \times \cos(2\pi f_s t)$ is the harmonic load due to a single pedestrian,

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 4-8, considering the excitation in the first harmonic or second harmonic of the pedestrian load (see Section 4.2).

Table 4-8: 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="2">Vertical and longitudinal

</td><td>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>TC1 to TC3</td><td>(density d < 1,0 P/m²):</td><td>$n' = \frac{10{,}8\sqrt{\xi \times n}}{S}$ [m^{-2}]</td></tr>
<tr><td>TC4 and TC5</td><td>(density $d \geq$ 1,0 P/m²):</td><td>$n' = \frac{1{,}85\sqrt{n}}{S}$ [m^{-2}]</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 4-10 a harmonic load $p(t)$ is applied to the structure for a particular mode shape.

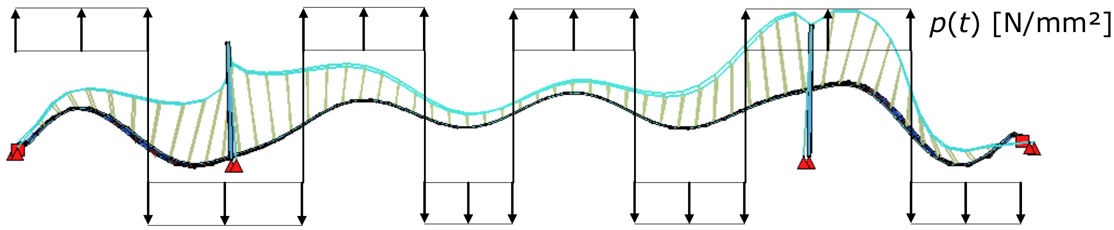


Figure 4-10: Application of a harmonic load according to mode shape $\phi(x)$

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 4-11) has one natural frequency and one mass that is equal to each natural frequency of the structure and the accompanying modal mass.

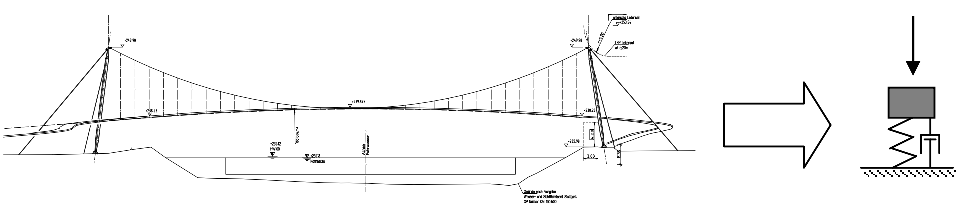


Figure 4-11: Equivalent SDOF oscillator for one natural frequency / vibration mode of the structure

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:

$$a_{\max} = \frac{p^*}{m^*}\frac{\pi}{\delta} = \frac{p^*}{m^*}\frac{1}{2\xi}$$ Eq. 4-16

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.

Background information:

As an example of application of the SDOF method, a simple supported beam is considered. This beam has a distributed mass μ [kg/m], which corresponds to the cross section area times the specific weight, a stiffness k and a length L. The uniform load $p(x)\sin(\omega t)$ is distributed over the total length.

The mode shapes $\phi(x)$ of the bending modes are assumed to be represented by a half sine function $\phi(x) = \sin(m \times x/L \times \pi)$ whereas m is the number of half waves.

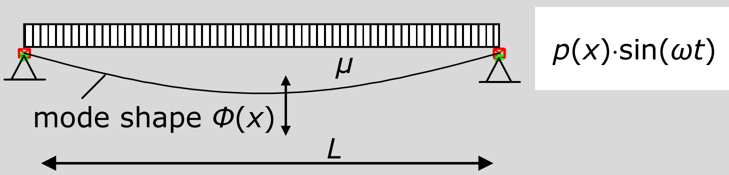


Figure 4-12: Simple beam with harmonic mode shape $\phi(x)$, m=1

The generalised mass m^* and the generalised load $p^*\sin(\omega t)$ are calculated as follows:

$$m^* = \int_{L_D} \mu \cdot (\Phi(x))^2 dx$$ Eq. 4-17

$$p^*\sin(\omega t) = \int_{L_D} p(x)\Phi(x)dx \cdot \sin(\omega t)$$ Eq. 4-18

Expressions for the generalised mass m^* and the generalised load $p^*\sin(\omega t)$ are systematised in Table 4-9 for a simple supported beam. The generalised load for a single load $P_{\mathrm{mov}}\sin(\omega t)$, moving across the simple beam is also given in this table. 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 4-9: Generalised (modal) mass and generalised load

Mode shape	generalised mass	generalised load p^* for distributed load $p(x)$	generalised p^* load for moving load P_{mov}	tuning time
	m^*	p^*	p^*	$t_{\max}$
m=1: $\varphi(x)=\sin\left(\frac{x}{L}\pi\right)$	$\frac{1}{2}\mu L$	$\frac{2}{\pi}p(x)L$	$\frac{2}{\pi}P_{mov}$	L/v
m=2: $\varphi(x)=\sin\left(\frac{2x}{L}\pi\right)$	$\frac{1}{2}\mu L$	$\frac{1}{\pi}p(x)L$	$\frac{2}{\pi}P_{mov}$	$L/(2v)$
m=3: $\varphi(x)=sin\left(\frac{3\,x}{L}\pi\right)$	$\frac{1}{2}\mu L$	$\frac{2}{3\pi}p(x)L$	$\frac{2}{\pi}P_{mov}$	$L/(3v)$

with:

P_{mov} [kN]:	moving load	L [m]:	length
$p(x)$ [kN/m]:	distributed load	m [-]:	number of half waves
μ [kg/m]:	mass distribution per length	v [m/s]:	velocity of moving load

The 2nd mode shape of a single span beam has two half waves (m = 2). When loading the entire length and when half of the uniformly distributed load is acting against the displacements of one belly and the other half is acting within the sense of displacements, then the generalized load will result in a value of $p^* = 0$. The generalised load in the given table is based on the assumption that each belly of the mode shape is loaded, which results in larger oscillations. In doing so, the load is always acting in the sense of displacements of the bellies and the generalised load p^* for all mode shapes is the same as for the first bending mode (m = 1). It must be noted that this approach may differ from other recommendations. According to some approaches [32], the loaded surface depends on the shape of the normal mode under consideration, according to others [9] the whole 'loadable' surface should be considered.

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:

$$a_{\max,d} = k_{a,d}\ \sigma_a \qquad \text{Eq. 4-19}$$

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:

$$\sigma_a^2 = k_1 \xi^{k_2} \frac{C \times \sigma_F^2}{m_i^{*2}} \qquad \text{Eq. 4-20}$$

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 [kN^2]	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 4-10 for vertical accelerations and in Table 4-10 for lateral accelerations.

Table 4-10: Constants for vertical accelerations

d [P/m^2]	k_F	C	a_1	a_2	a_3	b_1	b_2	b_3	$k_{a,95\%}$
≤ 0,5	1,20×10^{-2}	2,95	-0,07	0,60	0,075	0,003	-0,040	-1,000	3,92
1,0	7,00×10^{-3}	3,70	-0,07	0,56	0,084	0,004	-0,045	-1,000	3,80
1,5	3,34×10^{-3}	5,10	-0,08	0,50	0,085	0,005	-0,060	-1,005	3,74

Table 4-11: Constants for lateral accelerations

<table>
<tr><th>d [P/m^2]</th><th>k_F</th><th>C</th><th>a_1</th><th>a_2</th><th>a_3</th><th>b_1</th><th>b_2</th><th>b_3</th><th>$k_{a,95\%}$</th></tr>
<tr><td>≤ 0,5</td><td rowspan="3">2,85×10^{-4}</td><td>6,8</td><td>-0,08</td><td>0,50</td><td>0,085</td><td>0,005</td><td>-0,06</td><td>-1,005</td><td>3,77</td></tr>
<tr><td>1,0</td><td>7,9</td><td>-0,08</td><td>0,44</td><td>0,096</td><td>0,007</td><td>-0,071</td><td>-1,000</td><td>3,73</td></tr>
<tr><td>1,5</td><td>12,6</td><td>-0,07</td><td>0,31</td><td>0,120</td><td>0,009</td><td>-0,094</td><td>-1,020</td><td>3,63</td></tr>
</table>

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 $f_{s,m} = f_i$:

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

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 4-12 for vertical bending and torsion modes and Table 4-10 for lateral bending modes)

Table 4-12: Constants for the vertical bending and tension modal mass

<table>
<tr><th>d [P/m^2]</th><th>k_1</th><th>k_2</th><th>k_3</th><th>k_4</th></tr>
<tr><td>≤ 0,5</td><td>0,7603</td><td rowspan="3">0,468</td><td>0,050</td><td rowspan="3">0,675</td></tr>
<tr><td>1,0</td><td>0,5700</td><td>0,040</td></tr>
<tr><td>1,5</td><td>0,4000</td><td>0,035</td></tr>
</table>

Table 4-13: Constants for the 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.

Background information:

The general design procedure is adopted from wind engineering where it is used to verify the effect of gusts on sway systems. Pedestrian loads on bridges are stochastic loads. As it is not possible to determine structural properties as e.g. frequencies without uncertainties, these properties are also stochastic.

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 class to be proofed.

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

$$a_{\max,d} = k_{a,d}\ \sigma_a$$

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

The standard deviation of acceleration is obtained as a result of application of stochastic loads to a determined system. These loads have been defined considering bridges with spans in the range of 20 m to 200 m and a varying width of 3 m and 5 m, loaded with four different stream densities (0,2 P/m², 0,5 P/m², 1,0 P/m² and 1,5 P/m²). For each bridge type and stream density 5 000 different pedestrian streams have been simulated in time step calculations where each pedestrian has the following properties, 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 footfall forces (mean = 0,0378, standard deviation = 0,0144),
- Start position (randomly) and
- Moment of first step (randomly).

The peak factor $k_{a,d}$ is used to determine the characteristic response of the system. In serviceability limit states the characteristic value is the 95th percentile $k_{a,95\%}$. This factor is also a result of Monte Carlo simulations.

Another result of the simulations where the first 4 vertical and the first two horizontal and torsional modes have been considered is the risk of lateral lock-in.

To identify this risk a trigger amplitude of horizontal acceleration of 0,1 m/s² has been defined. The following frequency range is relevant for horizontal lock-in:

$$0{,}8 \leq \frac{f_i}{f_{s,m}/2} \leq 1{,}2\ Hz\ ,$$

where: f_i is the horizontal lateral natural frequency and

$f_{s,m}$ is the mean value of step frequency.

Natural frequencies to be considered should coincide with mean step frequencies of pedestrian streams.

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:

$$N_L = \frac{8\pi \times \xi \times m^* \times f}{k} \qquad \text{Eq. 4-21}$$

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:

$a_{\text{lock-in}}$ = 0,1 to 0,15 m/s^2 Eq. 4-22

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.

Background information:

As for walking the centre of gravity is not only varying vertically but also laterally from one foot to the other, the frequency of the movement of the human centre of gravity being half of the walking frequency.

Pedestrian streams synchronising with vertical vibrations have not been observed on footbridges. Vertical vibrations are absorbed by legs and joints which provide a certain amount of damping so that the centre of gravity is not affected by vertical vibrations. People are able to react on vibrations by adjusting their walking pattern. Although generally not considered, from experimental investigations it is known that single pedestrians can synchronise with harmonic vertical vibrations of 1,5 m/s^2 [7].

On the contrary, they react much more sensibly to lateral vibrations compared to vertical ones. If a pedestrian walks on a laterally vibrating bridge, he tries to compensate this additional movement of his centre of gravity by swaying with the bridge displacement. This behaviour is intuitive and even small not perceptible vibrations are assumed to cause an adjustment of the movement of the centre of gravity. Such a change of 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 [2]. The swaying of the body in time with the lateral vibration causes that the lateral ground reaction forces are applied in resonance. 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 (Figure 4-13). Hence, if a footbridge vibrates slightly in lateral direction and it happens that the pedestrians adjust their walking pattern, then due to this synchronisation effect a low-damped bridge can be excited to large vibrations.

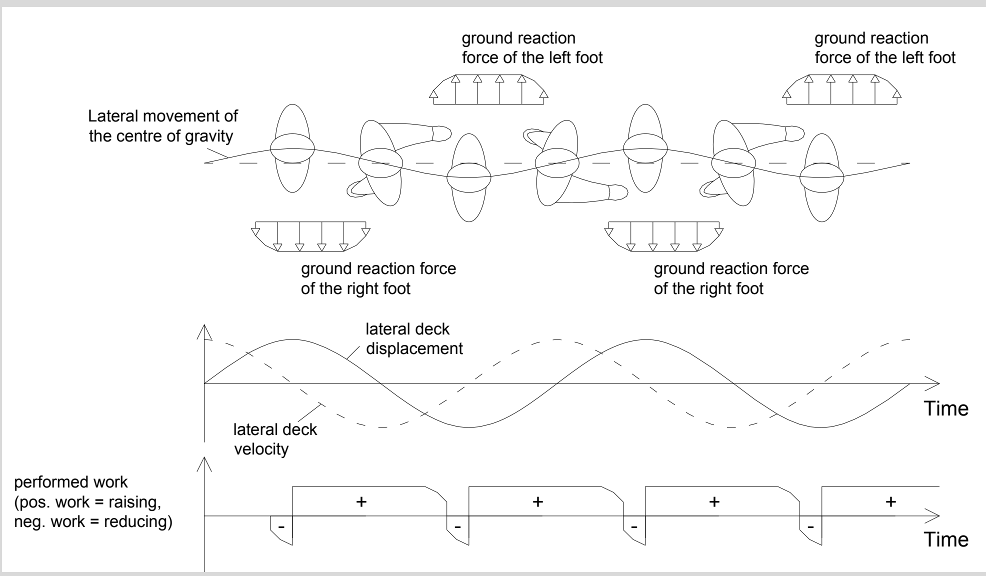


Figure 4-13: Schematic description of synchronous walking

Experiments on a test rig within the project SYNPEX [13] indicate that a single person walking with a step frequency f_i ± 0,2 Hz tends to synchronise with deck vibration. Faster walking persons are nearly not affected by the vibration of the deck, as the contact time of the feet is short and the walking speed high. They seem to be less instable than those 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. Tests in France [6] on a test rig and on the Passerelle Solferino indicate that a trigger amplitude of 0,1 to 0,15 m/s^2 exist when the lock-in phenomenon begins:

$a_{lock\text{-}in}$ = 0,1 to 0,15 m/s^2 Eq. 4-23

On a different perspective, the research centred in the Millennium footbridge [16] has led to an interpretation of lock-in as a phenomenon associated with the generation of a negative damping dependent on the number of pedestrians on the bridge. The triggering number of pedestrians for 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, has been defined as a function of the structural damping ratio ξ, of the modal mass m^*, of the natural frequency f, and of a constant k as

$$N_L = \frac{8\pi \cdot \xi \cdot m^* \cdot f}{k}$$ Eq. 4-24

On the basis of the Millennium footbridge tests, Dallard et al. [16] derived the constant k to be approximately equal to 300 Ns/m over the range 0,5-1,0 Hz.

Recent experiments on two footbridges in Coimbra and Guarda, Portugal [17] have shown the adequacy of the Millennium formula to describe the triggering for lock-in. Amplitudes of acceleration of the order of 0,15-0,2 m/s^2 have been observed in correspondence, suggesting that the two approaches may be related.

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.

Background information:

Although a comprehensive knowledge of materials and loads and a significant modelling capacity provide a high degree of understanding of the structural behaviour at the current state-of-art, numerous uncertainties remain present at the design stage of civil engineering structures. As a consequence, the corresponding dynamic properties and behaviour can only be fully assessed after construction. This fact has special importance in the context of pedestrian bridges, considering the narrow band of frequency excitation that frequently includes important bridge frequencies, and the typical low damping ratios of modern footbridges.

Standard tests, here designated as **Level 2** tests, should be developed at the end of construction of any potentially lively footbridge and should consider the identification of critical natural frequencies, damping ratios and the response measurement to a single, a small group or a stream of pedestrians.

Whenever the use of control devices is expected, **Level 1** tests are required; they additionally involve the identification of vibration modes.

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.

Background information:

In the simplest situation one single sensor, an accelerometer normally, is used for response measurement. The following procedure can be employed: for each measurement section, the sensor is mounted and the ambient response is collected, on the basis of two test series.

One of the series is collected, if possible, with the bridge closed to pedestrians, subjected to ambient loads, in order to eliminate the frequency content associated with pedestrian excitation, provided the transducers sensitivity is sufficiently high to capture ambient vibration response (typical acceleration peak amplitudes of the order of 2-5 mg). That procedure allows for an identification of the critical natural frequencies for vertical and/or lateral vibrations.

The second series should be collected under the current pedestrian excitation which provides a better characterization of bridge frequencies, as well as a measure of the intensity of vibrations under current use.

The choice of sampling rate and processing parameters should respect the following points:

- Assuming the frequencies of interest lie in the range 0,1-20 Hz, a sampling frequency of 50 Hz to 100 Hz should be selected. The acquisition equipment should include analogue filters in order to avoid aliasing errors, otherwise higher sampling rates may be required;

- Designating by f_{low} the expected lowest natural frequency of the bridge, the collected time series should have a minimum duration given by the formula

  $$(A / f_{low}) [n - (n\text{-}1)\ overl]\ [s] \qquad \text{Eq. 5-1}$$

  where A is a constant, with a value of 30 to 40, n is the number of records that will be employed in the obtainment of an average power spectral density (PSD) estimate of the response, and $overl$ represents the rate of overlap used for that estimate. Current values of n are 8-10, and a common rate of overlap is 50 %. Considering as an example a structure with a lowest natural frequency of 0,5 Hz, the averaging over a number n of records of 10, and an overlap rate of 50 %, the minimum duration of the collected time series should be 330-440 s. So a total number of 33 000 to 44 000 samples should be collected at a sampling frequency of 100 Hz, leading to average power spectra with frequency resolution of 0,017 Hz to 0,0125 Hz;
- The collected time series should be processed in order to obtain an average Power Spectrum Density (PSD) estimate. One procedure to form this PSD is as follows: divide the collected series into n records, considering the defined overlapping rate; remove trend for each record; apply time window (Hanning window, for example) in correspondence; evaluate normalised PSD of each record; average the set of raw PSDs;
- The analysis of PSD estimates collected at one or various sections allows for a former identification of the prototype natural frequencies;
- The peak response of the series collected under current pedestrian walking should be retained for comparison with acceptability limits.

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.

Background information:

The application of a single degree of freedom identification algorithm to the free decay response (eventually band-pass filtered, whenever close modes or noise are present) allows for a raw estimation of damping coefficient by segments of the time series. A plot of damping coefficient versus amplitude of oscillation can be made, where the amplitude of oscillation is taken as the average peak amplitude of oscillation within the analyzed series segment.

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.

Background information:

The response of the bridge to the action induced by one pedestrian crossing the bridge at the relevant step frequency is measured at the most critical section(s). Given the random characteristics of excitation, a number of realisations should be performed for each combination frequency / motion. A reference number is 5.

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.

Background information:

Looking in the literature, it can be noticed that the number of pedestrians used in group tests varies in the range 10-20 pedestrians.

The response should be measured based on the considerations for the crossing of one pedestrian, i.e., for each motion type / frequency combination, 5 realisations of one crossing of the bridge in declining sense (for non-symmetric slope) should be collected, at a sampling frequency of 50 Hz-100 Hz. The weight of the group members should be retained, and the group response should be the highest of the peak responses recorded.

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.

Background information:

The identification of modal parameters, i.e., natural frequencies, vibration modes and damping coefficients can be based on forced, free or on ambient vibration tests.

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.

Background information:

5.3.1.1 Hammer excitation

Even for the softest tips, hammer excitation produces a short duration pulse (typically 10 ms, on a concrete surface), whose frequency content is defined in a wide range, such as DC-200 Hz. Although analogue filters may be incorporated in the conditioning or acquisition equipment, the spectral content of the input can only be accurately defined if the time description is accurate. Assuming this pulse is represented by a half sinusoid, three points should be used to describe accurately this curve, with a minimum spacing of 5 ms. Hence, a minimum sampling frequency of 200 Hz should be employed, even though the frequency content of interest lies in the range 0,1 Hz-20 Hz.

Another aspect to retain is that, given that the input force is applied manually, some differences in the quality of the signal applied may occur. In particular, it is important for the operator to avoid double hits on each recorded time series, which significantly affect the quality of frequency response estimates.

Referring to the length of each recorded time series, it should be defined, if possible, in such a way that the structural response to hammer impulse vanishes within the collected series. In that case, time windowing is not necessary, therefore increasing the quality of damping estimates. A reference maximum duration of the series is 20,48 s, corresponding to a number of 4096 points sampled at 200 Hz. This corresponds to obtaining spectral estimates with a frequency resolution of 0,04 Hz, which is manifestly insufficient to characterise mode shapes at very low frequencies. Hammer excitation should not in effect be used for the characterisation of those modes. It should be noted that, even though longer records can be collected, the last part of the signal may contain only ambient vibration response and therefore does not provide an input correlated signal.

Assuming the sampling frequency and duration of records are defined, one procedure for the obtainment of a set of frequency response function estimates is as follows:

(i) Selection of a section along the deck where to apply the hits. This section should be chosen considering preliminary numerically calculated mode shapes, in such a way that the minimum number of modal nodes is close. More than one section may have to be defined, depending on the configuration of mode shapes of interest;

(ii) For each input section R_i, and depending on the number of available accelerometers, install successively the accelerometer(s) on the measurement sections. For each (set of) instrumented section(s), using the sampling parameters above defined, collect the response to the impulse hammer applied at R_j, as well as the input signal at the force sensor. For each set-up, a total of 5 to 10 time sets of series are recorded;

(iii) Remove trend to all response time series. Obtain a spectral description of the input and response, through estimation of auto-power spectra $\tilde{S}_{ii}(f)$ and $\tilde{S}_{jj}(f)$. Estimate the cross-spectrum $\tilde{S}_{ij}(f)$ relating the response at each measurement section R_i, with the input applied at section R_j. Average the set of auto and cross power spectra, for the set of 5 to 10 series collected at each location

$$S_{jj}(f) = E\left[\tilde{S}_{jj}(f)\right]$$

$$S_{ij}(f) = E\left[\tilde{S}_{ij}(f)\right]$$

Estimate frequency response functions $H_{ij}(f)$, based on estimator H_2

$$H_{ij}(f) = \frac{S_{ij}(f)}{S_{ii}(f)} \qquad \text{Eq. 5-2}$$

and coherence $\gamma^2(f)$, defined as

$$\gamma^2(f) = \frac{\left|S_{ij}(f)\right|^2}{S_{ii}(f) \;\; f_{jj}(f)} \qquad \text{Eq. 5-3}$$

The functions $H_{ij}(f)$ are intrinsic of the system and form the basis for application of a System Identification algorithm (in the frequency domain) to extract natural frequencies f_k, vibration modes $\underline{\varphi}_k$ and associated damping coefficients ξ_k, while $\gamma^2(f)$ provides a measure of the correlation between the measured input and response signals.

Considering a viscous damping model and response measurements expressed in terms of accelerations, the frequency response functions $H_{ij}(f)$ relate to the modal components of mode k, $(\varphi_i)_k$ and $(\varphi_j)_k$, at sections R_i and R_j, respectively, through

$$H_{ij}(f) = \frac{-f^2(\varphi_i)_k(\varphi_j)_k}{(f_k^2 - f^2) + i(2\xi_k \, f_k \, f)} \qquad \text{Eq. 5-4}$$

5.3.1.2 Vibrator excitation, wide-band

Wide-band excitation induced by hydraulic or electrodynamic vibrators can be of transient or continuous type. Transient signals, like burst random, are treated in a way similar to those produced by hammer excitation. Continuous signals require time windowing applied to each time segment of the series, in order to reduce leakage effects. Moreover, since time windowing reduces the contribution of the edge samples, it is frequent to overlap time segments. A common procedure consists in the application of Hanning windows to the input and response time segments, combined with an overlapping rate of 50 %. This allows a considerable reduction of the duration of the time series collected at each pair of input-output sections. Common wide-band generated signals are random or chirp-sine.

5.3.1.3 Vibrator excitation, sinusoidal tests

The performance of sinusoidal tests provides the best results, as long as the vibrator has sufficient power to induce the vibration modes of interest. This point is critical for very low natural frequencies, even though pedestrian bridges are very flexible.

The procedure for construction of frequency response functions and identification of vibration modes and damping coefficients comprehends a preliminary collection of ambient response, which provides an approximation of natural frequencies. Once the vicinity of each natural frequency of interest has been identified, a sinusoidal test is developed that consists in the construction of parts of the frequency response function, point by point, each point corresponding to the pair frequency of excitation, frequency content of the measured response at each measurement section. The following points should be considered:

(i) Although it is desirable to measure the applied force, that is not always possible, particularly if an eccentric mass shaker is employed. The force applied by such type of shakers can however be estimated with a certain precision;

(ii) The precise identification of the natural frequency of the structure is made by application of a sinusoidal excitation and recording of the response at one particular location where the estimated mode shape has a significant component. For each excitation frequency a time series of the response at a particular location can be extracted, with a short duration, corresponding for example to 512 samples. Assuming the induced signal is a perfect sinusoid, the amplitude and phase of the response can be extracted by single degree-of-freedom time domain data fit. The frequency response function dot is obtained by the ratio to the input excitation amplitude measured or estimated;

(iii) Although very short time series are required, it is necessary that the shaker operates for each frequency for a period of at least one minute, in order to guaranty that stabilisation of the response has been achieved;

(iv) Once the natural frequency has been identified, the vibrator is tuned to that frequency and one accelerometer, or a set of accelerometers are successively mounted at each measurement location to collect a small time series of response. When a force sensor is not employed, it is necessary to install an accelerometer close by the vibrator, which remains fixed. Simultaneous records of response at two locations are then collected, for an evaluation of the relative phase and amplitude to the reference section. The set of amplitudes and phase ratios to the reference point constitute mode shape components;

(v) The best quality of damping estimates is obtained with sinusoidal tests. Damping estimates are obtained from the analysis of the measured free vibration response obtained by sudden interruption of sinusoidal excitation at resonance. Provided that no close modes are present, a single degree-of-freedom algorithm is sufficient to identify the damping ratio. Given that this ratio depends on the amplitude of response, the free vibration response should be analysed by segments of the response record in the form described at Section 5.2.2.

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.

Background information:

The basic hypothesis for ambient vibration tests is that the input, i.e., the ambient excitation, can be idealized as a white-noise defined in a bandwidth corresponding to the frequency range of interest. This means that, within a certain frequency range, all mode shapes are excited at a constant amplitude and phase. The recorded response is therefore an operational response, and the technique of constructing so-called frequency response functions, relating the responses at two measurement sections, leads to identification of operational deflection shapes, instead of modal shapes. Assuming that the frequencies of the system are well separated, and that the damping coefficients are low, a good approximation exists between operational deflection shapes and modal shapes. However, if frequencies are close, the operational deflection modes comprehend a non-negligible superposition of adjacent modes, therefore providing erroneous results. Although some possibilities exist for providing a separation of mode shapes, like separating bending and tensional response on a bridge by constructing two signals, the half-sum and half-difference of the edge deck measured response, some other possibilities are offered in terms of signal processing, that allow identification of modal components and damping coefficients. That is the case of the stochastic subspace identification method, which is an output-only parametric modal identification technique that can be applied directly to acceleration time series or to the corresponding response covariance matrices [33]. This method has been implemented as a toolbox for Matlab (Macec) [37]. Also commercially available is a software based on the stochastic subspace identification and frequency domain decomposition methods (Artemis) [38], as well as another one based on Polymax method, which are also powerful tools for modal shape identification.

Although damping estimates are provided by the more powerful algorithms, the precision in the estimates is limited and so results should be used with care. In effect, not only the precision of sensors is currently so high, that the structural response can be measured for very small levels of vibration, but also powerful data processing techniques are available ([33], [34], [35]) that can be employed to identify modal parameters.

The conventional technique for identification of operational deflection shapes requires the building of frequency response functions between outputs. This is done exactly as described in section 5.3.1.2 for forced vibration tests with wide-band excitation.

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.

Background information:

Considering that the sudden release of a tensioned cable is equivalent to the application of an impulse, the identification of modal parameters from a free vibration test can follow the procedure described in section 5.3.1.2, in which the frequency spectrum of the input is assumed constant for the range of analysis. Alternatively, the output-only identification algorithms of the type described in section 5.3.2 can be applied. In any case, it is expected that higher quality modal estimates are obtained than those resulting from ambient vibration tests.

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

Background information:

Given that acceptability limits for pedestrian comfort are generally defined in terms of acceleration, the usual measured response quantity is acceleration.

Three main categories can be employed in civil engineering measurements:

1. piezoelectric;
2. piezoresistive and capacitive;
3. force-balanced.

Compared with the other two types, piezoelectric accelerometers have several advantages, such as: not requiring an external power source; being rugged and stable in the long term, and relatively insensitive to the temperature; being linear over a wide frequency and dynamic range. A serious inconvenience exists in applications involving very flexible structures, which is the limitation for measurement in the low frequency range. Many piezoelectric accelerometers only provide linear response for frequencies higher than 1 Hz, although some manufacturers produce accelerometers that operate for very low frequencies.

Both piezoresistive and capacitive and force-balanced accelerometers are passive transducers, which require external power supply, normally an external 5 VDC-15 VDC excitation. These accelerometers operate however in the low frequency range, i.e., from DC to approximately 50 – 200 Hz, therefore being adequate for almost all types of measurements in civil engineering structures.

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).

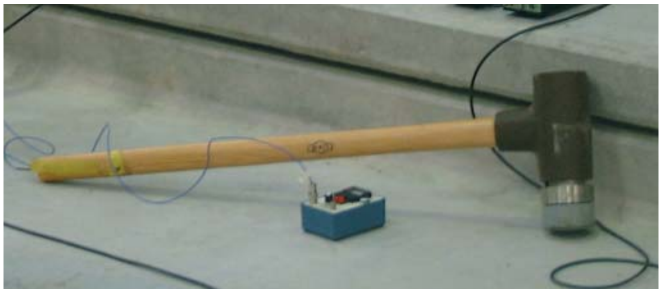


Figure 5-1: Impact hammer for civil engineering applications

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

Background information:

Impact hammer excitation is the most well-known and simple form of providing a controlled input to a Mechanical Engineering structure or component. For Civil Engineering applications, the same technique can be employed, provided that the impact hammer has adequate characteristics. For these particular structures, one solution available in the market is the hammer represented in Figure 5-1, weighting about 55 N, whose tip is instrumented with a piezoelectric force sensor, having a sensitivity of 1 V/230 N and a dynamic range of 22,0 kN. The hammer operates in the range 0 - 500 Hz. Given that pedestrian bridges are normally flexible and relatively small, the impact hammer meets for this type of structures one of the most interesting applications. It is noticed however that the energy input in the very low frequencies is very small, meaning that mode shapes of very low natural frequency are not possibly mobilised into a measurable level.

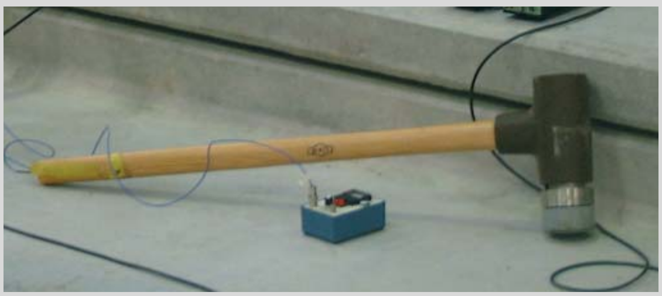


Figure 5-3: Impact hammer for civil engineering applications

Vibrators employed in Civil Engineering applications can be of three different types, electromagnetic, hydraulic and mechanical. The shaker represented in Figure 5-2 is one of the solutions available in the market, and weights around 800 N, operating in the range 0-200 Hz, and delivering a maximum force of 445 N for frequencies greater than 0,1 Hz. This device is configurable for excitation both in horizontal or vertical directions and is driven by means of a signal generator, which feeds the shaker amplifier. Typical generated signals for tests are sinusoidal or random. The measurement of applied force is possible through load cells installed between the shaker and the structure. Given the limitations in the amplitude of generated load, electro-dynamic shakers can only be used for excitation of small and medium sized structures. On the contrary, both hydraulic and mechanical shakers can be employed for excitation of large structures. Mechanical shakers based on the rotation of eccentric masses apply a sinusoidal excitation in a varying frequency range. These devices are seldom used at the current state of art, given the significant requirements for the setup and operation.

Figure 5-4: Electromagnetic shaker for civil engineering applications. Vertical mounting

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.

Background information:

Former work developed by Fujino et al. [36] has shown that the trajectory of pedestrians can be measured through measurement of the motion of pedestrians head and shoulders 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.

Background information:

The control of the vibration response in a footbridge implies the introduction of modifications, which can comprehend variation of the mass, frequency or structural damping. For an already constructed structure, 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 finishing, like the hand-rail and surfacing.

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.

Background information:

Possible strategies for modification of structural frequency 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 stabilising 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 tensional 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.

6.4 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 footbridges 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)
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%

Background information:

External damping devices comprehend 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.

Table 6-2 systematizes some examples of footbridges where damping systems have been implemented, referring characteristics of implemented measures and the overall effect in the dynamic behaviour.

Table 6-2: Footbridges where damping systems have been implemented

Bridge	Controlled frequencies (Hz)	Number of spans/length (m)	Type	Predominant vibration direction	Type of damping system implemented	Effect of the damping system on the overall behaviour	Ref.
T-Bridge, Japan	0,93	2 spans 45+134	Cable-stayed, continuous steel box girder	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.	[15]
Millennium Bridge, London	0,8 (main) 0,5 1,0	3 spans 108+144+80	Suspension tension-ribbon	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	[16]

Bellagio to Bally's footbridge, Las Vegas	Simply supported footbridge	Bridge	Footbridge on large atrium	Mjomnesundet bridge, Norway	Schwedter Straβe bridge, Berlin	Britzer Damm footbridge, Berlin
1,7 to 2,2	1,84Hz	Controlled frequencies (Hz)	4,3	0,8	1,9	5,6
1 span	1 span 47,4	Number of spans/length (m)	1 span 28	3 spans	1 span 209	1 span 33,83
Steel beam girder	Steel box girder	Type	Steel beams	Steel box girder	Cable stayed steel deck, by means of a steel arch	2 hinged steel arch, with orthotropic plate cross section
Vertical	Vertical	Predominant vibration direction	Vertical	Vertical	Vertical	Vertical
6 vertical tuned mass dampers	1 vertical tuned mass damper; mass ratio of 1,0% of structural modal mass	Type of damping system implemented	tuned mass dampers, each weighting ≈1000kg; mass ratios of ≈5% of structural	1 vertical tuned mass damper, weighting 6000kg was fitted on the bridge	4 vertical tuned mass dampers, each weighting 900kg were fitted on the bridge	2 vertical tuned mass dampers, each weighting 520kg were fitted on the bridge
Increase of structural damping by ≈16 times	Increase of structural damping by 12,7 times	Effect of the damping system on the overall behaviour				
[20]	[19]	Ref.	[18]	[17]	[17]	[17]

Pedro e Inês footbridge, Coimbra	Solférino footbridge, Paris	Stade de France footbridge, Paris	Forchheim footbridge, Germany
0,85 1,74; 1,80;2,34; 2,74; 3,07; 3,17	0,81 1,94 2,22	1,95	1,0 to 3,0
110m central span	106 central span	-	117,5
Shallow arch / girder	Arch	girder	Cable-stayed
Lateral Vertical	Lateral Vertical Vertical	Vertical	Vertical
1 lateral TMD with 14800kg and 6 vertical TMDs	1 lateral TMD with mass 15000kg and 2 vertical TMDs with masses 10000kg and 7600kg	Tuned mass dampers with mass of 2400kg per span	1 semi-active TMD, fitted with MR damper
Increased lateral damping from 0,5% to 4% and vertical damping from 0,3%-2,2% to 3%-6%	Increased structural damping from 0,4% to 3,5% (lateral), and from 0,5% to 3% and 2% (vertical)	Increased structural damping from 0,2-0,3% to 4,3-5,3%	
[17]	[8]	[8]	[21]

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: Example of viscous dampers

Background information:

The output of a viscous damper is generally defined by

$$F_{damper} = C \times v^{\alpha} \qquad \text{Eq. 6-1}$$

where: C = damping constant (N.sec/m)

v = velocity (m/sec)

α = velocity exponent ($0{,}3 \leq \alpha \leq 1{,}0$)

The inclusion of such a device into a structure results therefore necessarily in a non-proportional damping matrix that can be obtained from the original proportional damping matrix added by the appropriate damping coefficients in correspondence with the degrees of freedom associated with the damper locations. One particular advantage of viscous dampers is the possibility of simultaneous control of various vibration modes. In curved bridges, where modes have typically more than one type of significant displacement component, the use of a concentrated damper at the abutment, for instance, can effectively damp several modes that involve displacements in such direction. However, in several cases, viscous dampers may not be the best solution when compared to other alternatives. This is because viscous dampers work from the relative displacements of their two extremities. If available retrofitting locations only allow small relative displacements velocities, then these dampers are not of interest and TMDs or TLDs should be considered instead. Figure 6-12 shows an example of installation of viscous dampers interposed between the deck and the pylons.

Figure 6-2: Viscous dampers installed at Minden footbridge (Germany)

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 motion 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 6-3 shows two examples of TMDs, one vertical and one horizontal, installed in footbridges.

Figure 6-3: Examples of TMDs

Background information:

Tuned Mass Dampers (TMDs) are normally tuned so that the two peaks of the damped system frequency response curve have the same dynamic amplification, when expressed in terms of displacements. Design curves have been derived from the dynamic equations of motion and are available in the literature [18], [23].

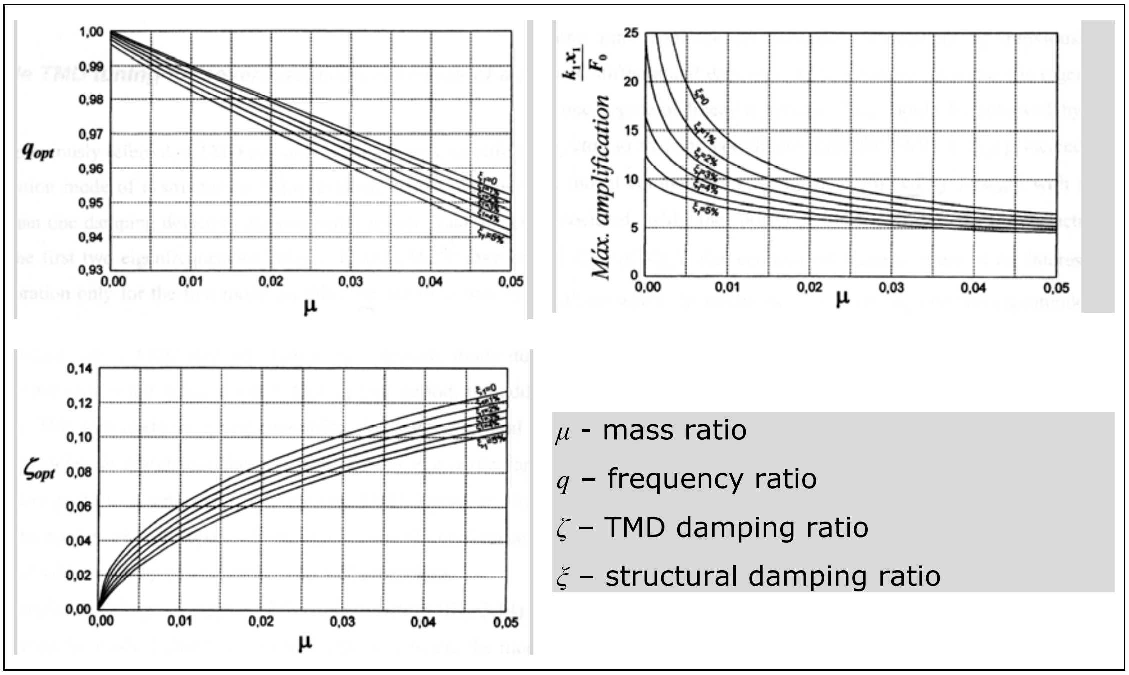


Figure 6-4: Design curves of TMDs

The design procedure may be as follows:

1. Choice of TMD mass m_d, based on the ratio μ to the structural modal mass m_s ($\mu = m_d / m_s$). Typical values of the mass ratio can range from 0,01 to 0,05.
2. Calculation of optimum TMD frequency ratio, expressed by the ratio δ between the TMD's, f_d, and the system's frequency f_s ($\delta = f_d / f_s$) [18].

$$\delta_{opt} = \frac{1}{1+\mu} \qquad \text{Eq. 6-2}$$

3. Calculation of optimum TMD damping ratio ξ_{opt} [18]

$$\xi_{opt} = \sqrt{\frac{3\mu}{8(1+\mu)^3}} \qquad \text{Eq. 6-3}$$

4. Calculation of the TMD constants:

Spring constant: $k_d = (2\pi f_d)^2 m_d$ Eq. 6-4

Damping constant: $c_d = 2 m_d (2\pi f_d) \xi_{opt}$ Eq. 6-5

The performance of a TMD is extremely sensitive to frequency de-tuning, which can occur as consequence of slight frequency changes associated with pedestrian loads or with modifications within the structure during its lifetime. Therefore it is of interest to evaluate the TMD efficiency for an estimated range of frequencies.

6.4.3.3 Pendulum dampers

Pendulum dampers (cf. Figure 6-5) 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.

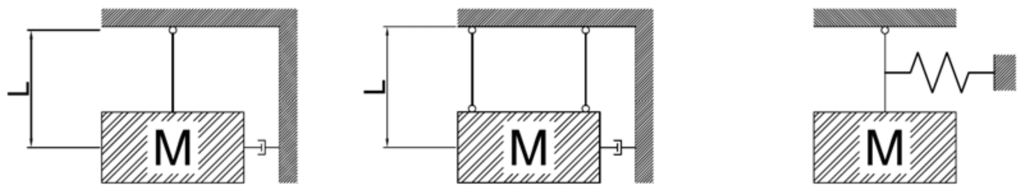


Figure 6-5: Examples of pendulum systems

Background information:

Neglecting the rotational inertia of the pendulum mass, the pendulum frequency can be calculated as follows:

1. Choice of mass ratio $\mu = m_d / m_s$
2. Calculation of the parameter $r_d = \frac{I_d}{m_d L}$, where I_d is the mass moment of inertia about the pivot, m_d the damper mass and L is the distance from the pivot to the centre of mass. If the mass is to be considered as a point mass, r_d=1.
3. Calculation of optimum frequency ratio, considering a white noise excitation force [24]

$$\kappa_{opt} = \frac{\sqrt{1+\mu\left(1-\frac{1}{2r_d}\right)}}{1+\mu} \qquad \text{Eq. 6-6}$$

4. Calculation of optimum damping ratio [24]

$$\xi_{opt} = \sqrt{\frac{\mu + \mu^2\left(1-\frac{1}{4\cdot r_d}\right)}{4r_d + 2\mu(4r_d - 1) + 2\mu^2(2r_d - 1)}} \qquad \text{Eq. 6-7}$$

5. Calculation of pendulum length $L = \frac{g}{(2\pi f_d)^2}$, where g is the acceleration of gravity and $f_d = f_s \times \kappa_{opt}$

6.4.3.4 Tuned liquid column dampers

A Tuned Liquid Column Damper (TLCD) consists of U-shaped tube (cf. Figure 6-6), 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.

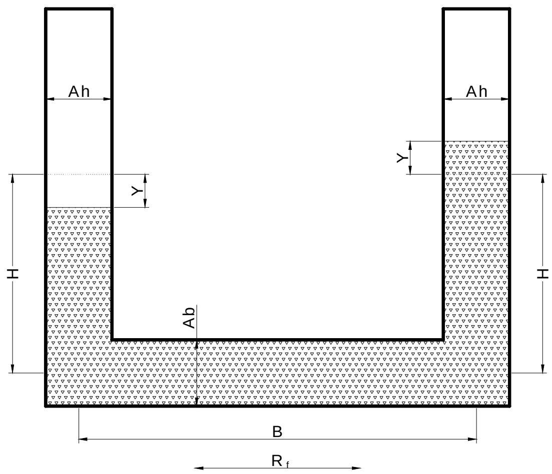


Figure 6-6: TLCD scheme

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.

Background information:

The tuning procedure of TLCDs is based on an analogy to the parameters of an equivalent TMD. Based on that principle, Reiterer and Ziegler [25] derived optimal design parameters for TLCDs.

The water mass ratio of modal building mass should be chosen around the same magnitude as in a TMD, i.e. from 0,01 to 0,05 [25].

The design procedure is illustrated for a TLCD with vertical columns ($\beta = \pi/2$) and constant cross-section ($A_h = A_b$):

1. Calculate the TMD-equivalent liquid mass ratio:

$$\mu^* = \frac{\mu}{\kappa^2 + \mu(\kappa^2 - 1)} \qquad \text{Eq. 6-8}$$

where μ is the previously chosen TMD mass ratio and κ is the geometry coefficient, defined by

$$\kappa = \frac{B + 2H\cos\beta}{L_{eff}} \qquad \text{Eq. 6-9}$$

with

$$L_{eff} = 2H + \frac{A_H}{A_B}B \qquad \text{Eq. 6-10}$$

The value of κ must be fixed. It is recommended to set it as high as possible, but below 0,8 [26] in order to prevent nonlinear effects in liquid motion.

2. Calculate the optimum TLCD frequency ratio:

$$\delta_{opt}^{*} = \frac{\delta_{opt}}{\sqrt{1 + \mu^{*}\left(1 - \kappa^{2}\right)}} \qquad \text{Eq. 6-11}$$

where δ_{opt} is the previously calculated TMD frequency ratio.

3. The values of H and B are calculated from the following set of equations:

$$\begin{cases} B = \dfrac{2g\sin(\beta)}{\left(\delta_{opt}^{*}\omega_{structure}\right)^{2}}\kappa - 2H\cos(\beta) \\ H = \dfrac{B + 2H\cos(\beta)}{2\kappa} - \dfrac{A_H}{2A_B}B \end{cases} \qquad \text{Eq. 6-12}$$

Note that since $\beta = \pi / 2$, B is obtained directly from the first equation. Also, since A_h / A_b = 1 and $cos(\beta)$ = 0, H can be extracted from the second equation.

4. Calculate the cross section areas A_h and A_b from the mass constraint:

$$\left(A_b B + A_h 2H\right)\gamma_{liquid} = M_{struct}\mu^{*} \qquad \text{Eq. 6-13}$$

$$A_h = A_b = \frac{M_{struct}\mu^{*}}{\left(B + 2H\right)\gamma_{liquid}} \qquad \text{Eq. 6-14}$$

The optimum damping of the TLCD should be the same as the analogue TMD. The TLCD has intrinsic damping due to fluid turbulence. In addition, 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 6-7) 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.

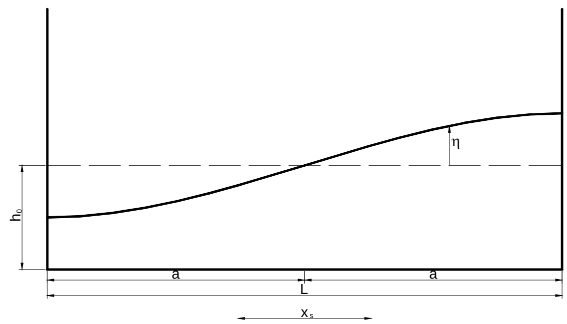


Figure 6-7: TLD scheme

Background information:

Advantages like low cost, almost zero trigger level, easy adjustment of natural frequency and easy installation on existing structures [27] have promoted an increasing interest in these devices.

The frequency of a TLD can be given by Lamb's linear theory, according to the formula [26]

$$\omega_{d,lin} = \sqrt{\frac{\pi g}{L} \tanh\left(\frac{\pi h_0}{L}\right)} \qquad \text{Eq. 6-15}$$

Sun et al. [28] have proposed the design of a TLD based on an analogy with a conventional TMD by experimental results from tests on prototype-scaled tanks. Also, experiments by Yu et al. [29] resulted in a non-linear formulation of an equivalent TMD taking into account the behaviour of the TLD under a variety of loading conditions. In this formulation, the stiffness hardening property of TLDs under large excitation was included.

In the non-linear stiffness and damping (NSD) model it is assumed that 100% of the liquid mass is effective in the damper, independently of the excitation amplitude.

TLD tuning may be accomplished by using the following procedure, developed from empirical curve-fits of experimental results, contemplating the non-linearity of the device:

1. Take the mean or frequent value of the amplitude of the deck displacement response X_s (estimated, after inclusion of TLD).
2. Calculate the non-dimensional excitation parameter $\Lambda = X_s/L$, where L is the length of the tank in the direction of motion.
3. Calculate the damping coefficient $\xi = 0{,}5\Lambda^{0{,}35}$
4. Calculate de frequency ratio χ between the non-linear and the linear TLD frequency defined by Lamb's formula:

$\chi = 1{,}038\Lambda^{0{,}0034}$ for Λ < 0,03 (weak wave breaking)

$\chi = 1{,}59\Lambda^{0{,}125}$ for Λ > 0,03 (strong wave breaking)

5. Calculate the water depth, which includes the stiffness hardening parameter χ, assuming that best tuning is accomplished by setting the TLD frequency equal to the structure's (f_s):

$$h_0 = \frac{L}{\pi}\tanh^{-1}\left(\frac{4\pi L f_s^2}{g\chi^2}\right) \quad \text{Eq. 6-16}$$

g – acceleration of gravity (9,81 m/s^2)

6. Choose tank width or number of tanks according to the necessary mass ratio for structural damping. The water mass ratio should be chosen around the same magnitude as in a TMD, i.e. from 0,01 to 0,05.

For numerical analysis, an equivalent TMD can be used. For very small deck displacement amplitudes (below 1cm) the active mass, m_d, may be as low as around 80% of the total liquid mass [28]. The stiffness k_d is obtained from $k_d = (\chi\omega_{d,lin})^2 m_d$. The damping coefficient is ξ_d, the same as the TLD.

In sum, TLD tuning may be accomplished by taking a mean or frequent value of the amplitude of ground displacement expected for the structure when in use, and the remaining parameters (tank length and/or water depth) can be derived from there.

7 Worked examples

7.1 Simply supported beam

The verification for reversible serviceability is shown for a pedestrian bridge having a span length of 50 m.

The bridge has the following properties:

Width of the deck B = 3 m
Length of the span L = 50 m
Mass m = 2,5×10^3 kg/m
Stiffness EI_{vert} = 2,05×10^7 kNm^2
EI_{lat} = 2,53×10^5 kNm^2
Damping ratio ξ = 1,5 %

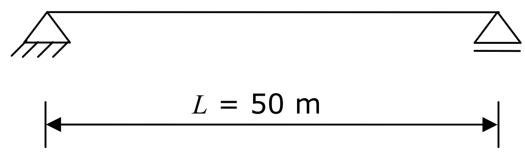


Figure 7-1: Structural system

The owner demands that medium comfort should be guaranteed for weak pedestrian traffic (d = 0,2 P/m^2) and that for very dense traffic (d = 1,0 P/m^2), which is expected for the inauguration of the bridge, minimal comfort in vertical direction should be guaranteed and a pedestrian-bridge interaction with lateral vibration should be avoided.

Loading scenarios	Required comfort
d = 0,2 P/m^2 n = 50×3×0,2 = 30	$a_{\text{limit,vert}} \leq$ 1,0 m/s^2 $a_{\text{limit,hor}} \leq$ 0,1 m/s^2
d = 1,0 P/m^2 n = 50×3×1,0 = 150	$a_{\text{limit,vert}} \leq$ 2,5 m/s^2 $a_{\text{limit,hor}} \leq$ 0,1 m/s^2

1. Determination of natural frequencies and modal masses

$$f_{1,vert} = \frac{1}{2\pi}\frac{9{,}869}{L^2}\sqrt{\frac{EI_{vert}}{m}} = 1{,}8 \text{ Hz} \qquad f_{2,vert} = \frac{1}{2\pi}\frac{39{,}478}{L^2}\sqrt{\frac{EI_{vert}}{m}} = 7{,}2 \text{ Hz}$$

$$f_{1,lat} = \frac{1}{2\pi}\frac{9{,}869}{L^2}\sqrt{\frac{EI_{lat}}{m}} = 0{,}2 \text{ Hz} \qquad f_{2,lat} = \frac{1}{2\pi}\frac{39{,}478}{L^2}\sqrt{\frac{EI_{lat}}{m}} = 0{,}8 \text{ Hz}$$

$M = ½\, m\, L$ = 62,5×10^3 kg

2. Determination of the characteristic maximum acceleration

a. for d = 0,2 P/m^2

$$a_{\max,vert} = k_{a,95\%}\sqrt{\frac{C\,\sigma_F^{\,2}}{M_i^{\,2}}\,k_1\xi^{k_2}} = 0{,}58 \text{ m/s}^2,$$

$$a_{d,vert} = \psi_1 \times a_{\max,vert} = 0{,}4 \times 0{,}58 = 0{,}23 \qquad < 1{,}0 \text{ m/s}^2 \ ✓$$

with C = 2,95 σ_F^{2} = 1,2×10^{-2}×30 = 0,36 kN^2 $k_{a,95\%}$ = 3,92

k_1 = -0,07×1,8^2 + 0,6×1,8 + 0,075 = 0,9282

k_2 = 0,003×1,8^2 – 0,04×1,8 – 1 = -1,06228

$$a_{\max,lat} = k_{a,95\%}\sqrt{\frac{C\,\sigma_F^{\,2}}{M_i^{\,2}}k_1\xi^{k_2}} = 0{,}087\ \text{m/s}^2 \qquad < 0{,}1\ \text{m/s}^2\ ✓$$

with C = 6,8 σ_F^2 = 2,85×10^{-4}×30 = 8,55×10^{-3} kN^2 $k_{a,95\%}$ = 3,77

k_1 = -0,08×0,8^2 + 0,5×0,8 + 0,085 = 0,5362

k_2 = 0,005×0,8^2 – 0,06×0,8 – 1,005 = -1,0498

b. for d = 1,0 P/m^2

$$a_{\max,vert} = k_{a,95\%}\sqrt{\frac{C\,\sigma_F^{\,2}}{M_i^{\,2}}k_1\xi^{k_2}} = 1{,}05\ \text{m/s}^2$$

$$a_{d,vert} = \psi_1 \times a_{\max,vert} = 0{,}4 \times 1{,}05 = 0{,}42 \qquad < 2{,}5\ \text{m/s}^2\ ✓$$

with C = 3,7 σ_F^2 = 7,0×10^{-3}×150 = 1,05 kN^2 $k_{a,95\%}$ = 3,80

k_1 = - 0,07×1,8^2 + 0,56×1,8 + 0,084 = 0,8652

k_2 = 0,004×1,8^2 – 0,045×1,8 – 1 = -1,06804

$$a_{\max,lat} = k_{a,95\%}\sqrt{\frac{C\,\sigma_F^{\,2}}{M_i^{\,2}}k_1\xi^{k_2}} = 0{,}20\ \text{m/s}^2 > 0{,}1\ \text{m/s}^2$$

Risk for pedestrian-structure interaction!

with C = 7,9 σ_F^2 = 2,85×10^{-4}×150 = 4,275×10^{-2} kN^2 $k_{a,95\%}$ = 3,73

k_1 = -0,08×0,8^2 + 0,44×0,8 + 0,096 = 0,4992

k_2 = 0,007×0,8^2 – 0,071×0,8 – 1 = -1,05232

7.2 Weser River Footbridge in Minden

The Pedestrian Bridge over the Weser River in Minden, Germany connects the Minden town center with a park. The structure is a suspension bridge, curved in plan, with a total length of 180 m, hung from two slanted tubular pylons. The bridge deck is 3,5 m wide (walkway 3,0 m) reinforced concrete slab and has a main span of 103 m.

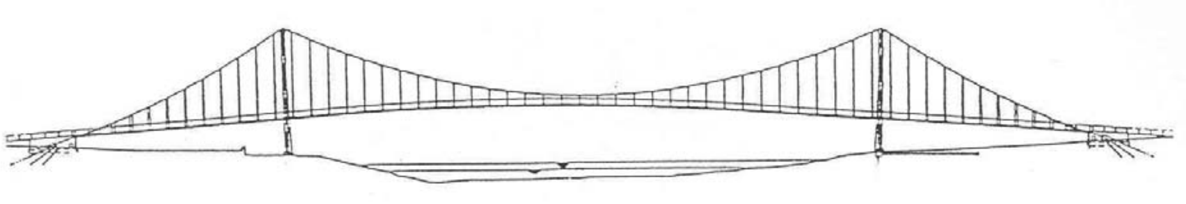


Figure 7-2: Elevation

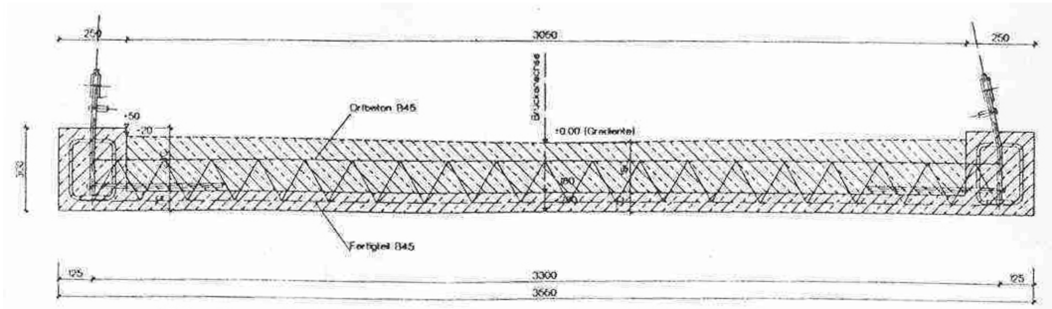


Figure 7-3: Cross-section

The following table shows the natural frequencies with the accompanying number of half waves up to a frequency of 3,00 Hz and their description.

Table 7-1: Description of natural frequencies

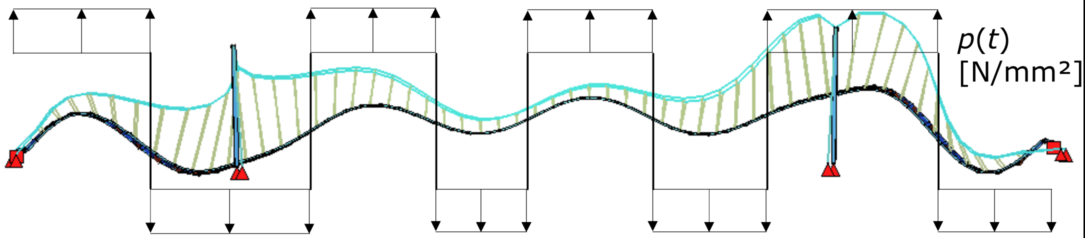
Mode No.	Natural frequency [Hz]	Number of half waves	Description of mode shape
1	0,24		Horizontal oscillation length
2	0,25	1	Horizontal oscillation
3	0,40	2	Vertical oscillation
4	0,41	3	Vertical oscillation
5	0,61	5	Vertical oscillation
6	0,61	6	Vertical oscillation
7	0,75	2	Horizontal / torsional effects
8	0,90	4	Vertical oscillation
9	0,95	7	Vertical oscillation
10	1,21	5	Vertical oscillation
11	1,42	8	Vertical oscillation
12	1,47	9	Vertical oscillation
13	1,60	3 / 1	Cable / horizontal oscillation + torsional effects
14	1,63	10	Vertical oscillation
15	1,73	-	Cable oscillation / horizontal + torsional effects

Mode No.	Natural frequency [Hz]	Number of half waves	Description of mode shape
16	1,77	-	Cable oscillation / vertical + torsional effects
17	1,82	-	Cable oscillation / vertical + torsional effects
18	1,96	11	Cable / vertical oscillation
19	2,07	11	Cable / vertical oscillation + torsional effects
20	2,13	-	Cable oscillation
21	2,27	-	Cable oscillation
22	2,36	12	Cable / vertical oscillation
23	2,57	-	Cable oscillation + vertical effects
24	2,59	-	Cable oscillation
25	2,64	13	Cable / vertical oscillation
26	2,73	-	Cable oscillation
Mode No.	Natural frequency [Hz]	Number of half waves	Description of mode shape
27	2,79	-	Cable oscillation
28	2,89	14	Vertical oscillation
29	2,91	4	Horizontal oscillation + torsional effects
30	2,96	-	Cable oscillation
31	3,15	-	Cable oscillation

As shown in the table above there are various frequencies with their accompanying mode shapes in the critical bandwidth, which means that they are prone for vertical and horizontal excitation by walking pedestrians. For a dynamic analysis all critical frequencies have to be investigated, but for this example only the 11^{th} mode shape with 8 vertical half waves will be analysed.

The Table 7-2 summarizes the given dynamic properties of the bridge and gives details about the loaded surfaces and their load directions.

Table 7-2: Summarized properties of the footbridge in Minden



(inflated shown mode shape)

Total length	L = 180 m
Deck width	B = 3,0 m
Considered mode shape	11^{th} mode shape
Description of mode shape	vertical oscillation – 8 half waves
Frequency	f = 1,42 Hz
Loaded surface	$S = L \times B$ = 540 m²
Modal mass	$m^*(f)$ = 80,5 t
Damping property (log. decrement)	δ = 0,085

According to this recommendation as well as to the recently released SETRA/AFGC Footbridge Design Guidelines [9], the loaded surface S of the whole bridge deck should be considered with load acting up and down according to the investigated mode shape directions.

The different load directions are simulating a phase shift of 180° or π for the pedestrians walking over the bridge. This can be interpreted as full synchronization between every single pedestrian and the belly of the mode shape (direction), which he is reaching or just walking over.

The design situation is defined by the combination of a traffic class and a comfort class. Generally, different design situations should be considered although this example is limited only to one design situation. As the footbridge connects the Minden town centre with a recreation area in a park traffic class TC2, weak traffic with 0,2 P/m² is chosen in combination with the comfort class CL1, maximum comfort, with amplitudes lower than a = 0,5 m/s².

Table 7-3: Description of the design scenario

Design Situation	Chosen Traffic Class	Chosen Comfort Class
1^{st} combination	TC 2: Weak traffic	CL 1: Maximum comfort

For a dynamic analysis there have to be considered more design situations, for example one with higher traffic densities and a more seldom occurrence for which lower comfort requirements in such a particular case may be acceptable.

The load model for a pedestrian stream according to these guidelines and the SETRA/AFGC Footbridge Guidelines are applied to the Minden footbridge and the dynamic response is calculated. The load model for a pedestrian stream according to the guidelines gives a distributed surface load $p(t)$, which has to be applied on the bridge structure depending on the mode shape, as shown before. The harmonic oscillating surface load $p(t)$ for the excitation is given by the following equation:

$$F(t) = P\cos(2\pi\, ft) = 280\cos(2\pi \times 1{,}42\,t) \quad [\mathrm{N}]$$ Eq. 7-1

$$n = S \times d = 108 \qquad \text{with} \quad d = 0{,}2 \ \mathrm{P}/\mathrm{m^2}$$ Eq. 7-2

$$n' = \frac{10{,}8\sqrt{\xi \times n}}{S} = 0{,}024 \ 1/m^2 \qquad \text{with} \quad \xi = \delta/2\pi$$ Eq. 7-3

$$p(t) = F(t)n'\psi \qquad \text{with} \quad \psi = 0{,}7$$ Eq. 7-4

$$p(t) = 280\cos(2\pi \times 1{,}42\,t) \times 0{,}024 \times 0{,}7$$

$$p(t) = 4{,}74\cos(8{,}92\,t) \quad [\mathrm{N/m^2}]$$

This leads to the maximum acceleration $a_{\max}$ by using the FE-Method.

$$a_{\max} = 0{,}38 \le a_{CL1} = 0{,}50 \ [\mathrm{m/s^2}]$$ Eq. 7-5

According to the chosen limit acceleration value defined by Comfort Class 1 – Maximum Comfort with a ≤ 0,50 m/s², the result of the dynamic analysis shows that the defined Comfort requirements are fulfilled and the serviceability for oscillation is confirmed for this example.

Verification according to Spectral Load Model for streams

Now, the maximum acceleration $a_{\max}$ is calculated according to the Spectral Load Model for pedestrian streams for the chosen design situation. It must be noted that the calculated acceleration, by applying the spectral load method, is a characteristic value according to Eurocode design practice.

$$a_{\max} = \psi\, k_{a,95\%}\sqrt{\frac{C\,\sigma_F^{\,2}}{m^{*\,2}_{i}}\, k_1 \xi^{k_2}} \qquad \text{with} \quad \psi = 0{,}7$$ Eq. 7-6

$$a_{\max} = 0{,}54 \approx a_{CC1} = 0{,}50 \ [\mathrm{m/s^2}]$$ Eq. 7-7

with

C = 2,95

σ_F^2 = $1{,}2 \times 10^{-2} \times 108$ = 1,30 kN²

$k_{a,95\%}$ = 3,92

k_1 = -0,07×1,42^2 + 0,6×1,42 + 0,075 = 0,7859

k_2 = 0,003×1,42^2 – 0,04×1,42 – 1 = -1,0508

ξ = 0,085 / (2×π)

M = m^* = 80 500 kg

The calculated maximum acceleration is slightly higher than the result from the FE-analysis. Both calculated accelerations meet the comfort class requirements for maximum comfort.

7.3 Guarda Footbridge in Portugal

The Guarda Footbridge (Figure 7-4) establishes a pedestrian crossing over a road that provides one of the entrances in the city of Guarda, in Portugal, connecting an urban area that includes a school to the railway station. The footbridge is formed by two central arches, hinged at the supports, with a span of 90 m and 18 m rise, suspending the steel deck by inclined cables. The deck has a total length of 123 m and is also supported by three piers near each extremity, which preclude vertical and lateral movements. It is formed by a steel grid with two longitudinal beams 2,70 m distant, connected by transverse beams every 4 m. This structure is linked to a concrete slab assembled by precast panels with 3 m width (walkway 2,0 m) (Figure 7-5).

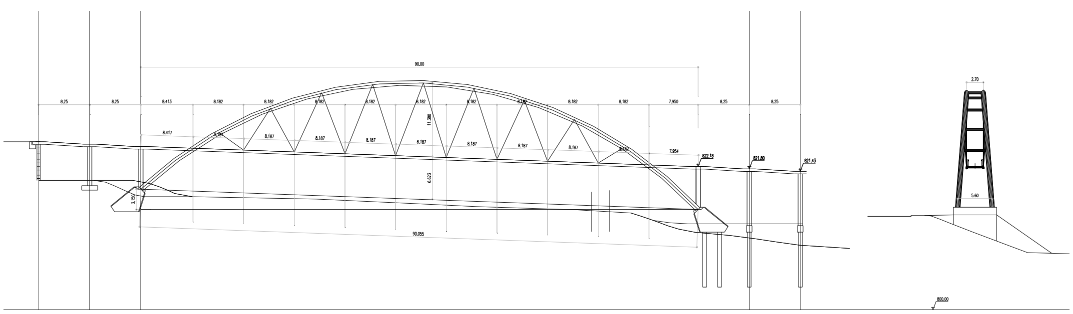


Figure 7-4: Lateral view of Guarda Footbridge

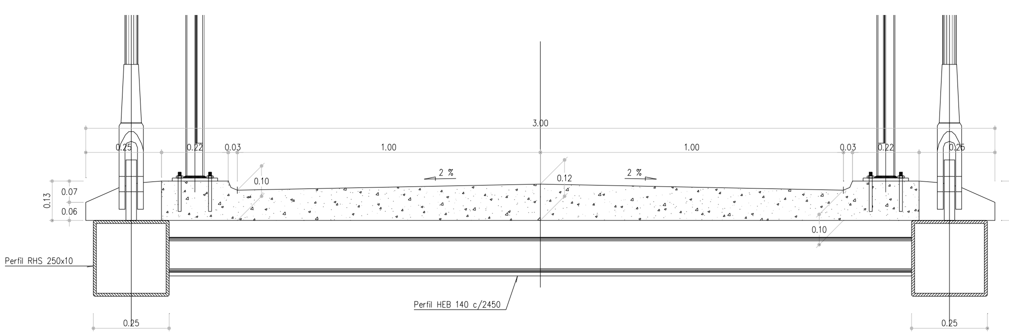


Figure 7-5: Cross section of Guarda Footbridge

Table 7-4 summarises the first five natural frequencies of the structure which were calculated after updating the numerical model based on dynamic tests conducted at the end of the bridge construction. The characteristics of the vibration modes and the values of the measured damping ratios are also indicated in this table.

Table 7-4: Natural frequencies and characteristics of vibration modes

Mode Nr.	Natural frequency [Hz]	Measured ξ [%]	Characteristics of vibration mode
1	0,63	2,2	1st lateral
2	1,24	1,7	2nd lateral
3	1,41	1,4	3rd lateral
4	2,33	0,8	1st vertical
5	3,60	0,4	2nd vertical

Based on the critical ranges of frequencies defined in the current guidelines for the lateral and vertical directions of vibration, it is concluded that the first two lateral modes of vibration are critical for horizontal excitation by pedestrians, while for the vertical direction only mode 4 is critical. Mode 5 would be of interest to investigate for possible effects associated with the 2nd harmonic of vertical pedestrian loads. For the current example, only the first lateral and first vertical modes are investigated and the corresponding characteristics used in design are summarised in Table 7-5.

Table 7-5: Characteristics of investigated vibration modes

Quantity	Mode 1	Mode 4
Natural frequency, f [Hz]	0,63	2,33
Loaded surface [m^2]	$S = L \times B$ = 123×2 = 246	
Modal mass, m^*[t]	82,5	130,7
Total mass [t]	232,2	
Damping ratio, ξ [%]	0,6	0,6

Considering the location of the footbridge close to a school, although not linking very relevant areas in town, several scenarios should be investigated. In the current example only two design situations are analysed, corresponding to: 1- the inauguration of the footbridge, with a traffic class TC4 (d = 1,0 P/m^2) and a minimum comfort class (maximum vertical accelerations of 1-2,5 m/s^2 and lateral accelerations of 0,3-0,8 m/s^2); 2- commuter traffic (TC2, d = 0,2 P/m^2) and medium comfort class (maximum vertical accelerations of 0,5-1 m/s^2 and lateral accelerations of 0,1-0,3 m/s^2). Although the measured damping ratios after construction of the footbridge (presented in Table 7-4) are higher, a value of 0,6 % was considered at design stage.

The harmonic load models for pedestrian streams are then defined in accordance with the guidelines and are systematised in Table 7-6, for the two design situations. It should be noted that for the design situation 1 the added mass associated with pedestrians represents 7,6 % of the total bridge mass. Therefore, the footbridge natural frequencies should be re-calculated with the footbridge loaded, that has not been done within the current example for simplification.

Table 7.6: Harmonic load models for pedestrian streams

	n ($S \times d$)	n'	Ψ (M 1)	Ψ (M 4)	$p_h(t)$ [N/m^2] (M 1)	$p_v(t)$ [N/m^2] (M 4)
Design situation 1	246	0,118	1	0,54	4,13cos(2π×0,63t)	17,84cos(2π×2,33t)
Design situation 2	49,2	0,0239	1	0,54	0,835cos(2π×0,63t)	3,61cos(2π×2,33t)

The signals of the loads are defined in accordance with the modal components, according to the representation of Figure 7-6.

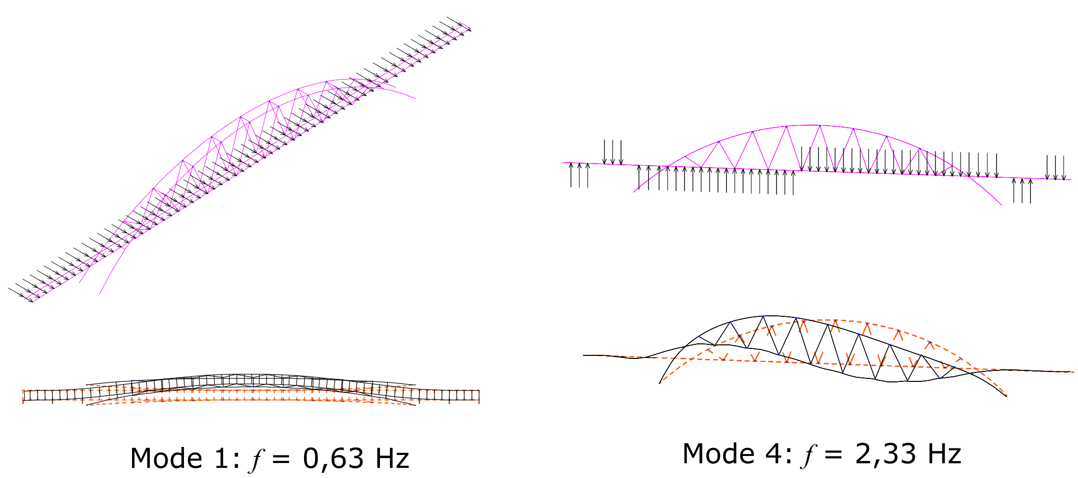


Figure 7-6: Schematic representation of harmonic loads and vibration mode

Table 7-6 summarises the maximum values of the response, expressed in accelerations, obtained on with the developed FE model, which are compared with the range of accelerations accepted for the specified level of comfort. It is observed that comfort is ensured at all circumstances. However, the lateral acceleration of 0,67 m/s^2 largely exceeds the limit of 0,15 m/s^2, that triggers lock-in according to the current guidelines. Furthermore, the application of the Millenium Bridge formula (cf. section 4.6) to determine the number of pedestrians N_L to trigger lock-in provides a value of

$$N_L = \frac{8\pi\xi m^* f}{k} = \frac{8\times\pi\times 0{,}6\times 10^{-2}\times 82{,}5\times 10^3\times 0{,}63}{300} = 26{,}1\ P \qquad \text{Eq. 7-8}$$

These 26,1 pedestrians are distributed in an equivalent length of 84 m, meaning that lock-in occurs for a density of pedestrians of 0,16 P/m^2, significantly lower than the assumed 1 P/m^2 on the inauguration day.

This fact has led to the consideration, at design stage, of a TMD for control of vibrations, adding a minimum damping of 4 %, which implied the strengthening of the deck to incorporate this device at the mid-span. In practice, a damping ratio of 2,2 % was measured after construction of the footbridge, which would increase the lock-in trigger to a pedestrian density of 0,6 P/m^2 and it was an option of the designer not to introduce a TMD to control this vibration mode.

Table 7-6: Structure response to harmonic load models

Maximum acceleration [m/s^2]	Mode 1 (lateral)	Mode 4 (vertical)	Acceptable range (lateral) [m/s^2]	Acceptable range (vertical) [m/s^2]
Design situation 1	0,67	1,11	0,30-0,80	1,0-2,5
Design situation 2	0,13	0,22	0,10-0,30	0,5-1,0

8 References

[1] BS5400, Part 2, Appendix C, *Vibration Serviceability Requirements for Foot and Cycle Track Bridges*. British Standards Institution, 1978

[2] DIN-Fachbericht 102, *Betonbrücken*. Deutsches Institut für Normung, 2003.

[3] ENV 1995-2, *Eurocode 5 - Design of timber structures – bridges*. European Committee for Standardization, 1997.

[4] *Guidelines for the design of footbridges*. fib bulletin 32, November 2005.

[5] EN 1990, *Eurocode 0 – Basis of structural design*. European Committee for Standardization, 2002.

[6] Charles, P.; Bui, V., *Transversal dynamic actions of pedestrians & Synchronisation*. Proceedings of Footbridge 2005 – 2nd International Conference, Venice 2005.

[7] Schneider, M., *Ein Beitrag zu fußgängerinduzierten Brückenschwingungen*, Dissertation. Technische Universität München, 1991

[8] Maia, N. et al., *Theoretical and Experimental Modal Analysis*. Research Studies Press, UK, 1997.

[9] SETRA/AFGC, *Passerelles piétonnes – Evaluation du comportement vibratoire sous l'action des piétons (Footbridges – Assessment of dynamic behaviour under the action of pedestrians)*, Guidelines. Sétra, March 2006.

[10] Bachmann, H. and W. Ammann, *Vibrations in Structures Induced by Man and Machines*. IABSE Structural Engineering Documents, 1987. No. 3e.

[11] EN 1991-2, *Eurocode 1– Actions on structures, Part 2: Traffic loads on bridges*. European Committee for Standardization, 2002.

[12] EN 1995-2, *Eurocode 5– Design of timber structures, Part 2: Bridges*. European Committee for Standardization, 2003.

[13] Butz, C. et al., *Advanced load models for synchronous pedestrian excitation and optimised design guidelines for steel foot bridges (SYNPEX)*, Project RFS-CR-03019, Final Report. RFCS, 2007.

[14] EN 1998-2, *Eurocode 8– Design of structures for earthquake resistance, Part 2: Bridges*. European Committee for Standardization, 2003.

[15] Nakamura, S. and Y. Fujino, *Lateral vibration on a pedestrian cable-stayed bridge.* IABSE, Structural Engineering International, 2002.

[16] Dallard, P., et al., *The London Millennium footbridge.* The Structural Engineer, 2001. 79/No 22.

[17] Caetano, E., Cunha, A. and Moutinho, C., *Implementation of passive devices for vibration control at Coimbra footbridge*. EVACES 2007, Porto, 2007.

[18] Collette, F.S., *Tuned Mass Dampers for a suspended structure of footbridges and meeting boxes.* Footbridge 2002, Paris, 2002.

[19] Hatanaka, A. and Y. Kwon, *Retrofit of footbridge for pedestrian induced vibration using compact tuned mass damper.* Footbridge 2002, Paris, 2002.

[20] Breukleman, B., et al., *Footbridge damping systems: a case study.* Footbridge 2002, Paris, 2002.

[21] Seiler, C., O. Fischer, and P. Huber, *Semi-active MR dampers in TMD's for vibration control of footbridges, Part 2: numerical analysis and practical realisation.* Footbridge 2002, Paris, 2002.

[22] Den Hartog, J.P., *Mechanical Vibrations.* McGraw Hill, New York, 1940.

[23] Moutinho, C.M., *Controlo passivo e activo de vibrações em pontes de peões*, MSc. Thesis. 1998, Universidade do Porto: Porto.

[24] Geres, R.R. and B.J. Vicjery, *Optimum Design of Pendulum-Type Tuned Mass Dampers.* The Structural Design of Tall and Special Buildings, 2005(14): p. 353-368.

[25] Reiterer, M. and F. Ziegler, *Combined seismic activation of a SDOF-building with a passive TLCD attached*. 13th WCEE, Canada, 2004.

[26] Lamb, H., *Hydrodynamics.* The University Press, Cambridge, England, 1932.

[27] Fujino, Y. and L.M. Sun, *Vibration control by multiple tuned liquid dampers (MTLDs).* Journal of Structural Engineering, 1992. 119(12): p. 3482-3502.

[28] Sun, L.M., et al., *The properties of tuned liquid dampers using a TMD analogy.* Earthquake engineering and structural dynamics, 1995. 24: p. 967-976.

[29] Yu, J.-K., T. Wakahara, and D. Reed, *A non-linear numerical model of the tuned liquid damper.* Earthquake engineering and structural dynamics, 1999. 28: p. 671-686.

[30] Statistisches Bundesamt: http://www.destatis.de/basis/d/gesu/gesutab8.php, Mikrozensus 2004

[31] Živanović, S. et al., *Vibration serviceability of footbridges under human-induced excitation: a literature review*. Journal of Sound and Vibration 279 (2005), pp. 1-79

[32] SETRA/AFGC, *Comportement Dynamique des Passerelles Piétonnes (Dynamic behaviour of footbridges)*, Guide (Draft). December 2004.

[33] Peeters B., *System Identification and Damage Detection in Civil Engineering*, Ph.D. Thesis. Katholieke Universiteit Leuven, 2000.

[34] Brincker R., Zhang L. and Andersen P., *Modal identification from ambient responses using frequency domain decomposition*, Procceedings of IMAC-XVIII, International Modal Analysis Conference, pp.625-630, San Antonio, Texas, USA, 2000.

[35] Van Overschee P., De Moor B., *Subspace Identification for Linear Systems: Theory-Implementation-Applications*. Kluwer Academic Publishers, Dordrecht, The Netherlands, 1996.

[36] Fujino Y., Pacheco B., Nakamura S. and Warnitchai P., *Synchronization of Human Walking Observed during Lateral Vibration of a Congested Pedestrian Bridge.* Earthqauke Engineering and Structural Dynamics, Vol.22, pp.741-758, 1993.

[37] http://www.bwk.kuleuven.ac.be/bwm/macec/index.html

[38] http://www.svibs.com/

9 Appendix: Additional load models

9.1 Load model for a single pedestrian

The three-dimensional dynamic forces induced by one pedestrian are generated by the movement of the body mass and the put-down, rolling and push-off of the feet. The forces are called human ground reaction forces. When they are induced by walking, then they form an almost periodic excitation.

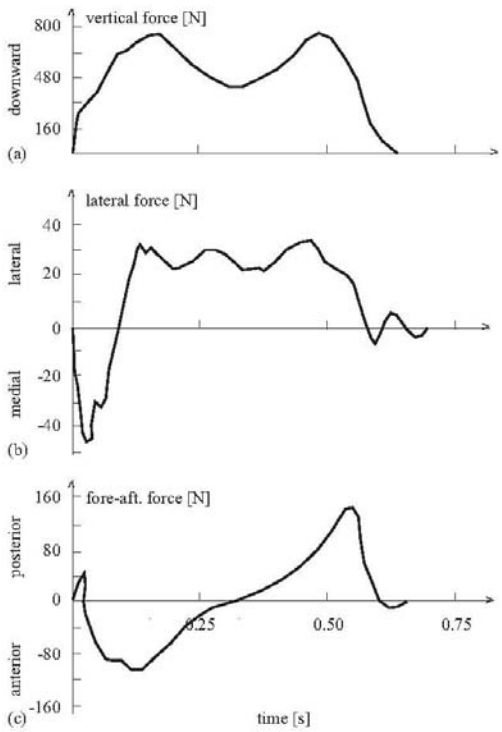
People walk with similar step frequencies due to similar physiological human constitutions. But the step frequencies are influenced by the purpose of the movement and the traffic intensity. Step frequencies between 1,25 to 2,3 Hz show the highest probability of occurrence.

As during walking one foot is always in contact with the ground, the loading does not disappear completely at any time like in the case of running. The human ground reaction forces of both feet overlap and form a periodic loading that is moving in time and space.

The magnitudes of the vertical and longitudinal forces mainly depend on the person's step frequency and body weight. Their periodicity is related to the step frequency. The lateral component is caused by the movement of the centre of gravity from one foot to the other. The oscillating motion of the centre of gravity introduces a dynamic force with half the walking frequency.

Walking induces a vertical force with a butterfly shape having two dominant force maxima. The first one is caused by the impact of the heel on the ground, while the second one is produced by the push off. The maxima rise with increasing step frequency (cf. Figure 9-1 a)). The horizontal force components in longitudinal and lateral direction are much smaller than the vertical component. The longitudinal force (x-direction) characterises the retarding and the pushing walking period (cf. Figure 9-1 c)). The lateral force (y-direction) is caused by the lateral oscillation of the body. It shows a large scatter as it is influenced by e.g. type of shoes, the toe out angle, the posture of the upper part of the body, swinging of the arms, position of the legs (i.e. knock-knees, bowlegs), way of hitting the ground. Unlike the vertical and the longitudinal force the lateral one is periodic with half the walking frequency (cf. Figure 9-1 b)).

Time domain models are the most common models for walking and running. They are based on the assumption that both human feet produce exactly the same force. Hence, the resulting force is periodic and can be represented by Fourier series (cf. Figure 9-1).



$$F_{p,vert}(t) = P\left[1 + \sum_{i=1}^{n} \alpha_{i,vert} \sin\left(2\pi\, i\, f_s\, t - \varphi_i\right)\right] \quad \text{Eq. 9-1}$$

$$F_{p,lat}(t) = P \sum_{i=1}^{n} \alpha_{i,lat} \sin\left(\pi\, i\, f_s\, t - \varphi_i\right) \quad \text{Eq. 9-2}$$

$$F_{p,long}(t) = P \sum_{i=1}^{n} \alpha_{i,long} \sin\left(2\pi\, i\, f_s\, t - \varphi_i\right) \quad \text{Eq. 9-3}$$

Figure 9-1: Typical shapes of walking force

where $F_{p,\text{vert}}$ vertical periodic force due to walking or running

$F_{p,\text{lat}}$ lateral periodic force due to walking or running

$F_{p,\text{long}}$ longitudinal periodic force due to walking or running

P [N] pedestrian's weight

$\alpha_{i,\text{vert}}$, $\alpha_{i,\text{lat}}$, $\alpha_{i,\text{long}}$ Fourier coefficient of the i^{th} harmonic for vertical, lateral and longitudinal forces, i.e. dynamic load factor (DLF)

f_s [Hz] step frequency

φ_i phase shift of the i^{th} harmonic

n total number of contributing harmonics

The periodic force is not stationary. It moves with a constant speed along the bridge. Within the SYNPEX project, the relationship between step frequency and walking speed is found by measurements for a step frequency range of 1,3 to 1,8 Hz:

$$v_s = 1{,}271 f_s - 1 \quad \text{Eq. 9-4}$$

In many Codes (e.g. EN 1995 [12]) the body weight P is given as 700 N or 800 N. The mean body mass given in the German 2004census is 74,4 kg [30].

Fourier coefficients resp. dynamic load factors have been measured by various authors [31]. As human ground reaction forces are influenced by a variety of factors (e.g. walking speed, individual physiological body properties, type of shoes), the measured load factors scatter. Table 9-1 lists Fourier coefficients and phase angles from selected authors.

Table 9-1: Fourier coefficients by different authors for walking and running

Author(s)	Fourier coefficients / Phase angles	Comment	Type of activity and load direction
Blanchard et al.	α_1 = 0,257		Walking – vertical
Bachmann & Ammann	α_1 = 0,4 – 0,5; α_2 = α_3 = 0,1	for f_p = 2,0 - 2,4 Hz	Walking – vertical
Schulze	α_1 = 0,37; α_2 = 0,10; α_3 = 0,12; α_4 = 0,04; α_5 = 0,015	for f_p = 2,0 Hz	Walking – vertical
Bachmann et al.	α_1 = 0,4/0,5; α_2 = α_3 = 0,1 α_1 = α_2 = α_3 = 0,1 $\alpha_{1/2}$ = 0,1; α_1 = 0,2; α_2 = 0,1 α_1 = 1,6; α_2 = 0,7; α_3 = 0,3 φ_2 = φ_3 = $\pi/2$	f_p = 2,0/2,4 Hz f_p = 2,0 Hz f_p = 2,0 Hz f_p = 2,0 – 3,0 Hz	Walking – vertical Walking – lateral Walking – longitudinal Running – vertical Walking – vertical & lateral
Kerr	α_1, α_2 = 0,07; α_3 = 0,2	α_1 is frequency dependant	Walking – vertical
Young	α_1 = 0,37 (f_p – 0,95) ≤ 0,5 α_2 = 0,054 + 0,0088 f_p α_3 = 0,026 + 0,015 f_p α_4 = 0,01 + 0,0204 f_p	Mean values for Fourier coefficients	Walking – vertical
Charles & Hoorpah	α_1 = 0,4 α_1 = 0,05 α_1 = 0,2		Walking – vertical Walking – lateral Walking - longitudinal
EC5, DIN1074	α_1 = 0,4; α_2 = 0,2 α_1 = α_2 = 0,1 α_1 = 1,2		Walking – vertical Walking – lateral Jogging – vertical

Author(s)	Fourier coefficients / Phase angles	Comment	Type of activity and load direction
Synpex findings	$\alpha_1 = 0{,}0115 f_s^2 + 0{,}2803\, f_s - 0{,}2902$ $\varphi_1 = 0$ $\alpha_2 = 0{,}0669 f_s^2 + 0{,}1067\, f_s - 0{,}0417$ $\varphi_2 = -99{,}76 f_s^2 + 478{,}92\, f_s - 387{,}8$ [°] $\alpha_3 = 0{,}0247\, f_s^2 + 0{,}1149\, f_s - 0{,}1518$ If $f_s < 2{,}0$ Hz $\varphi_3 = -150{,}88\, f_s^3 + 819{,}65\, f_s^2 - 1431{,}35\, f_s + 811{,}93$ [°] If $f_s >= 2{,}0$ Hz $\varphi_3 = 813{,}12\, f_s^3 - 5357{,}6\, f_s^2 + 11726\, f_s - 8505{,}9$ [°] $\alpha_4 = -0{,}0039\, f_s^2 + 0{,}0285\, f_s - 0{,}0082$ $\varphi_4 = 34{,}19\, f_s - 65{,}14$ [°]	Fourier coefficients and phase angles of step-by-step load model which represents mean human ground reaction forces	Walking – vertical

9.2 Load model for joggers

The human ground reaction forces due to running are characterised by a lift-off phase, during which no foot is in contact with the ground. The ground contact is interrupted and hence the force is zero. In comparison to walking the running-induced forces depend more on the individual way of running and the type of shoes. The vertical load curve has a single peak and is characterised by a steep increase and decrease (cf. Figure 9-2).

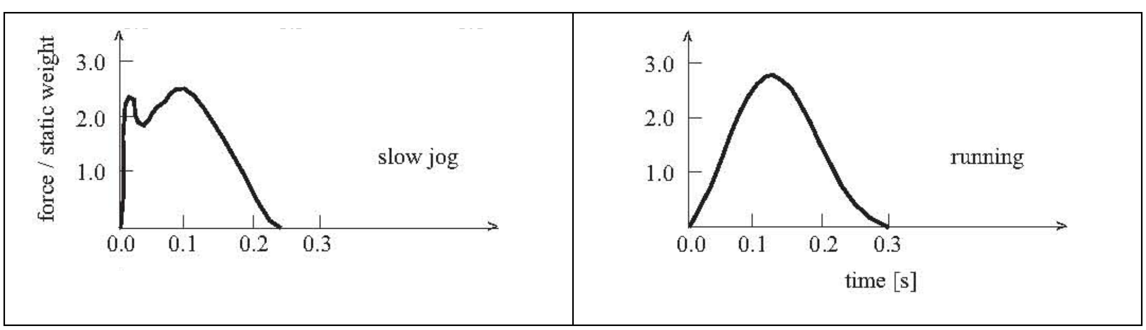


Figure 9-2: Typical vertical force patterns for slow jogging and running [1]

The proposed load model is a single load $P(t,v)$ which is moving across the bridge with a certain velocity v of the joggers. That is the reason why this load model is very difficult to apply with currently used commercial structural analysis programs and may only be modelled by specialised software (e.g. ANSYS, DYNACS).

The single load $P(t,v)$ calculates to:

$$P(t,v) = P \times \cos(2\pi\, ft) \times n' \times \psi \qquad \text{Eq. 9-5}$$

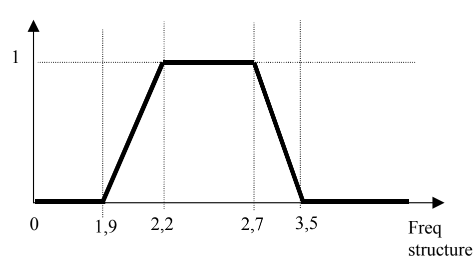
where $P \times \cos(2\pi f \cdot t)$ is the harmonic load due to a single pedestrian,

- P is the force component due to a single pedestrian walking with step frequency f,
- f is the natural frequency under consideration,
- n' is the equivalent number of pedestrians on the loaded surface S,
- S is area of the loaded surface,
- 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 maximum force P of a single pedestrian, the equivalent number of pedestrians n' and the reduction coefficient ψ are given in Table 4-10.

Table 9-2: Parameters for joggers [32]

***P* [N]**			***n' = n* [ ]**
Vertical	**Longitudinal**	**Lateral**	
1250	—	—	
Vertical reduction coefficient ψ			



According to [32], it can be considered that the group of n joggers is perfectly synchronized in frequency and phase with the footbridge natural frequency. The joggers move with a velocity of 3 m/s across the bridge. But in many cases it seems to be sufficient to place the load $P(t,v{=}0)$ at the maximum displacement amplitude of the shape modes.

It seems that there have been no measurements of the horizontal component during running, either for its longitudinal, or for its lateral component.

Nevertheless, it is reasonable to suppose that the lateral component presents relatively small amplitude comparing to the vertical one, while the longitudinal component is more important.

Note: In SETRA/AFGC guidelines [8] this load case has disappeared as non-relevant.

9.3 Intentional excitation by small groups

It might happen that people try to excite the bridge in resonance by synchronous jumping, bouncing, horizontal body swaying combined with shaking handrails and by shaking cables with their hands. A low damped lightweight footbridge can be excited to large amplitudes that might affect the structural safety.

While the impact force of a single person due to jumping is larger than the force created by bouncing, the synchronisation during jumping with the bridge vibration is much lower. During bouncing the person stays always in contact with the bridge and can synchronise its body motion with the vibration. Even if several persons try to intentionally excite the bridge by jumping, it is very hard for them to jump in phase with each other. Here bouncing is much more effective. Linking arms or introducing a beat can magnify the synchronisation and hence the excitation force considerably. Nevertheless, the result is not related linearly to the number of involved persons because there was a decreasing synchronisation with increasing number of persons observed during several tests.

It is important to note that intentional excitation is more an 'accidental ultimate limit state' than a 'fatigue problem' or than a 'comfort problem'. Structures develop an increase in damping with increase in vibration amplitude and people loose concentration and power to excite the bridge over a longer time period necessary for affecting the fatigue strength of the construction material. Intentional excitation is stopped when the amplitude does not increase for some time or when the persons have no more power for exciting the bridge.

European Commission

EUR 23984 EN – Joint Research Centre – Institute for the Protection and Security of the Citizen

Title: **Design of Lightweight Foot-bridges for Human Induced Vibrations**

Author(s): Christoph Heinemeyer, Christiane Butz, Andreas Keil, Mike Schlaich, Arndt Goldack, Stefan Trometer, Mladen Lukić, Bruno Chabrolin, Arnaud Lemaire, Pierre-Oliver Martin, Álvaro Cunha, Elsa Caetano

Luxembourg: Office for Official Publications of the European Communities

2009 – 82 pp. – 21 x 29.7 cm

EUR – Scientific and Technical Research series – ISSN 1018-5593

ISBN 978-92-79-13387-9

DOI 10.2788/33846

Abstract

Nowadays, it is well-known that for the structural safety verification of footbridges, and for the comfort guarantee of its users, it is fundamental to consider the effect of human induced vibrations, particularly: vibrations due to pedestrian traffic should be within acceptable limits for users; the lock-in phenomenon should be prevented; and, the footbridge structural safety should be guaranteed if subjected to intentional excitations. Recognizing a gap in lightweight footbridges design procedures, a guideline is presented in this document proposal for footbridges design taking into account human induced vibrations.

First, the proposed methodology is detailed. Attention is devoted also to the response measurements, identification tests and instrumentation used in the evaluation of the dynamic properties of footbridges. The potential strategies to control the vibration response of footbridges are reviewed. Finally, three worked examples of application of the proposed design methodology are exposed, namely a simply supported beam and two existing footbridges.

Keywords: Footbridges; Design procedures; Human induced vibrations; Structural safety; Human comfort; Dynamic properties; Vibration control

How to obtain EU publications

Our priced publications are available from EU Bookshop (http://bookshop.europa.eu), where you can place an order with the sales agent of your choice.

The Publications Office has a worldwide network of sales agents. You can obtain their contact details by sending a fax to (352) 29 29-42758.

The mission of the JRC is to provide customer-driven scientific and technical support for the conception, development, implementation and monitoring of EU policies. As a service of the European Commission, the JRC functions as a reference centre of science and technology for the Union. Close to the policy-making process, it serves the common interest of the Member States, while being independent of special interests, whether private or national.

EUROPEAN COMMISSION

The European Convention for Constructional Steelwork (ECCS) is the federation of the National Association of Steelwork industries and covers a worldwide network of Industrial Companies, Universities and research Institutes.

ECCS
CECM
EKS

LB-NA-23984-EN-C

ISBN 978-92-79-13387-9

Publications Office
Publications.europa.eu