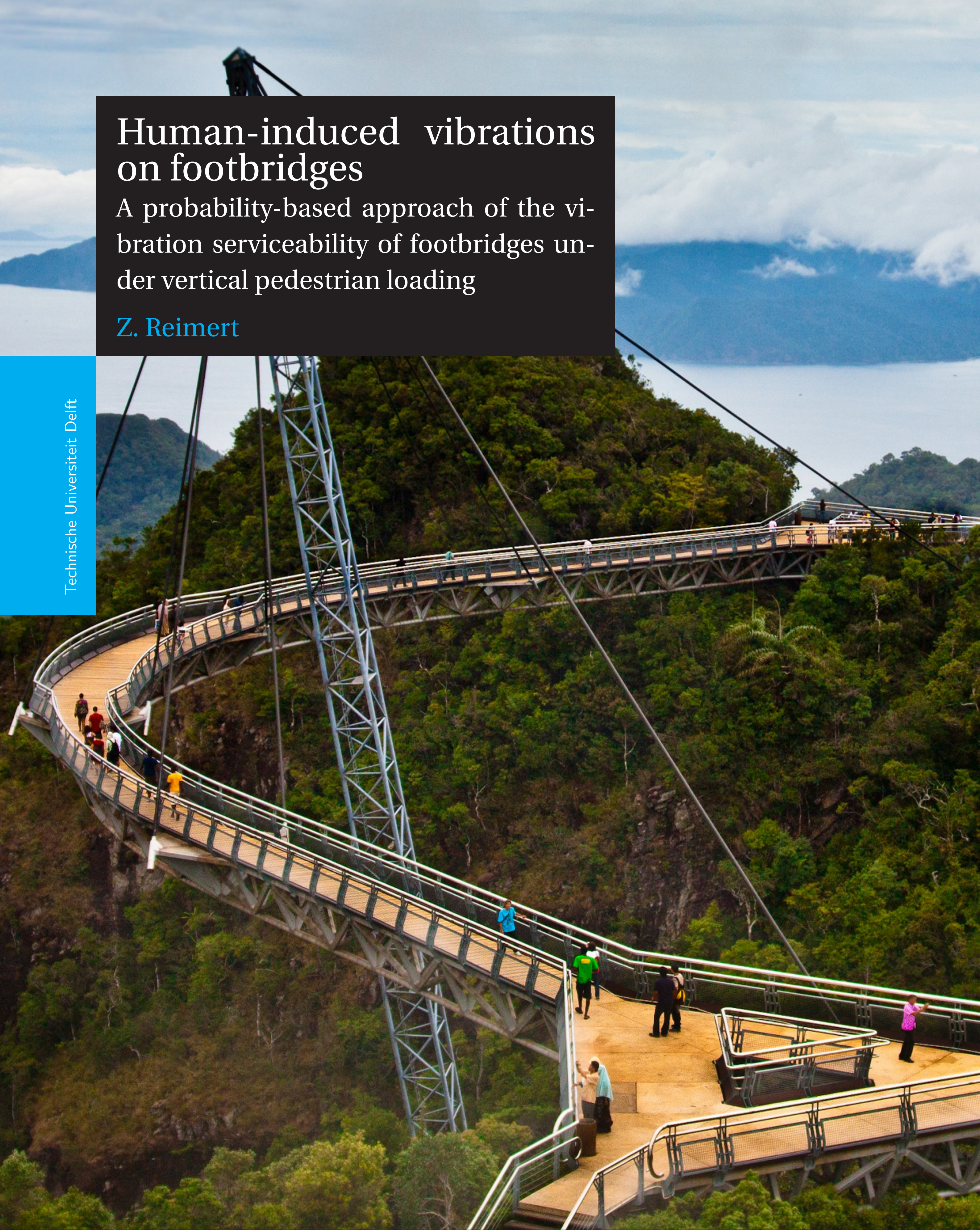

Human-induced vibrations on footbridges

A probability-based approach of the vibration serviceability of footbridges under vertical pedestrian loading

Z. Reimert

Technische Universiteit Delft

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Cover image
Langkawi Sky Bridge: a 125 m curved cable-stayed footbridge, crossing over the peak of Gunung Mat Chinchang, a mountain on the island of Pulau Langkawi, Malaysia. (photograph by Boud, J. 2009)

HUMAN-INDUCED VIBRATIONS ON FOOTBRIDGES

A PROBABILITY-BASED APPROACH OF THE VIBRATION SERVICEABILITY OF FOOTBRIDGES UNDER VERTICAL PEDESTRIAN LOADING

by

Z. Reimert

in partial fulfillment of the requirements for the degree of

Master of Science
in Civil Engineering
specialisation Structural Mechanics

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An electronic version of this thesis is available at `http://repository.tudelft.nl/`.

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I really enjoyed working on this master thesis and I hope you will enjoy reading it!

Z. Reimert
Delft, December 2014

ABSTRACT

In structural design there is a tendency towards more slender and challenging architectural structures. Footbridges become more slender with an increasing high ratio of live load to dead weight. A consequence of the increased slenderness of footbridges is an increased susceptibility to human-induced vibrations. The evaluation of the serviceability of footbridges therefore becomes more important.

Many studies have been carried out to cover the human-induced bridge dynamics in a series of regulations in the Eurocode, guidelines and standards, but there is still limited clarity for the users. The first aim of this study was therefore to provide a good basis and overview of the critical aspects in the evaluation of human-induced vibrations in footbridges. This study has been performed by means of a literature study, discussions with the engineering companies ARUP, RoyalHaskoningDHV and GemeentewerkenRotterdam and an impact study on three simply supported and two cantilever structures. The focus has been on vibrations in the vertical direction.
From the literature study, the discussions and the impact study it became clear that there are four critical aspects particularly relevant for further study at the moment. First of all the structural safety of short footbridges and especially the safety of the more sensitive footbridges designed with new lightweight materials, should be examined for vandal loading. Secondly, the impact of structural damping has been shown to be a significant large percentage of the bridge response, which makes it an interesting and important topic for further study. Furthermore, the possibility of pedestrians to add mass and damping to the structure is of importance in the design of footbridges. The final bottleneck that has been indicated as an important issue for further research is the probability of occurrence of accelerations in footbridges in relation to the probability of discomfort of individual pedestrians.

As a result of the investigation of the critical aspects, the second part of this thesis was focused on the question whether a probability-based approach can demonstrate that the Eurocode is conservative in the evaluation of human-induced vibrations in footbridges. Hereto, a probability-based analysis has been performed for the vibration serviceability of footbridges and implemented in a case study with three simply supported footbridges. Instead of looking at the maximum acceleration that is expected for the bridge deck, the accelerations which individual pedestrians experience when crossing the bridge have been investigated. Four scenarios have been compared to the approach prescribed by the Eurocode. The pedestrian loading has been modelled based on two assumptions for the scenarios: scenarios based on densities of the pedestrian flow and scenarios based on group formation.
The conducted research based on assumptions for the pedestrian traffic and a fixed criterion for human comfort, has given valuable insight in the use of a more realistic evaluation of human-induced vibrations in footbridges. This insight has been obtained for the incoming pedestrian traffic and the exposure to vibrations for individual pedestrians. The adopted probability-based approach contributes to demonstrate potential conservatism in the Eurocode regarding the evaluation of human-induced vibrations in footbridges.
It was not possible to give a final answer to the question if the adopted probabilistic-approach can demonstrate whether the Eurocode is conservative or not in the evaluation of human-induced vibrations in footbridges. This is due to a lack of data about the actual incoming pedestrian traffic on footbridges and due to outstanding questions regarding the type of accelerations to be considered for measuring human comfort on footbridges. However, regarding some key aspects in the evaluation of human-induced vibrations in footbridges, this research has shown four important aspects.

The first important aspect this study showed is a distinction between two main types of pedestrian traffic: normal traffic comprising the commuter traffic, shopping area and park; and special locations with large groups of pedestrians walking in a high density, such as the train station. Whereas the Eurocode prescribes pedestrian traffic with a density of 0.5 P/m^2, this study suggests a density for normal traffic of about 0.3 P/m^2 based on a 95% non-exceedance level. For special locations on the other hand this density is expected to exceed 1.0 P/m^2 considerably.

Secondly, this study infers a dependency of the length of the footbridge for the expected pedestrian traffic, in contradiction to the fixed value prescribed by the Eurocode. With an increasing length of the footbridge the expected traffic density decreased significantly. For bridges with a span of 12 to 50 m the expected traffic density reduced in the order of 50%; a similar result was seen from 50 to 100 m.
Further, a significant reduction of 40% in the expected maximum acceleration is found, when considering the maximum acceleration an individual pedestrian experiences when crossing the bridge instead of the maximum acceleration of the footbridge in the steady state. This reduction is expected to be for 30% the result of considering individual pedestrians and for 10% a result of the evaluation method of the Eurocode. This suggests that the use of the maximum expected acceleration of the bridge in the steady state as a measure for human comfort is conservative, compared to the accelerations that an individual pedestrian is expected to perceive.
Finally, this study showed that the two current available criteria for the measure of human comfort on vibrating footbridges, based on the maximum acceleration and the root mean square per second of the weighted acceleration, do not correspond with each other, despite the high correlation between the measure methods. This indicates that more research to the criteria and measure of comfort is necessary.

ABBREVIATIONS AND NOTATIONS

Abbreviations

CC	Comfort class
CDF	Cumulative density function
DAF	Dynamic amplification factor
DL	Dead load
DLF	Dynamic load factor
FE	Finite element
FRP	Fibre-reinforced plastics
HHI	Human-human interaction
HSI	Human-structure interaction
LL	Live load
MSDS	Mass-spring-damper-system
PDF	Probability density function
P/m^2	Persons per square meter
PMF	Probability mass function
r.m.s.	Root mean square
RS	Response spectra
SDOF	Single degree of freedom
SLS	Serviceability limit state
TC	Traffic class
TMD	Tuned mass damper
ULS	Ultimate limit state
VDV	Vibration dose value

Notations

Critical range	The critical range of the walking frequency of pedestrians: in case the natural frequency of the bridge is within this critical range, the bridge has to be checked dynamically to the pedestrian load.
EN 1990	Eurocode: Basis of structural design
EN 1991	Eurocode 1: Actions on structures
Eurocode	European standard for checking the structural safety of all possible constructions
EUR 23984	Guideline to the Eurocode for the "Design of lightweight footbridges for human induced vibrations" provided by the Joint Research Centre
HiVoSS	Human induced Vibrations of Steel Structures: Guideline for the design of pedestrian bridges
ISO 10137	International standard: Bases for design of structures - Serviceability of buildings and walkways against vibrations
ISO 2394	International standard: General principles on reliability for structures
ISO 2631-1	International standard: Mechanical vibration and shock - Evaluation of human exposure to whole-body vibration-Part 1: General requirements
ISO 2631-2	International standard: Mechanical vibration and shock - Evaluation of human exposure to whole-body vibration-Part 2: Vibration in buildings (1 Hz to 80 Hz)
Sétra	Service d'Etudes techniques des routes et autoroutes: Technical guide Footbridges, assessment of vibrational behaviour of footbridges under pedestrian loading

CONTENTS

1

INTRODUCTION

1.1. Problem definition

In structural design there is a recent tendency towards more slender and challenging architectural structures. This is the result of current developments in engineering. The improved knowledge of structural behaviour in combination with the implementation of new construction materials allows more efficient designs. In addition, the fast progress in computational modelling makes it possible to solve complex design problems. The construction of more slender structures enables longer spans and due to more efficient designs to reduce the amount of material, the costs can be lowered as well.

Footbridges are slender structures and become, due to these developments, even more slender with an increasing high ratio live load to dead weight. A consequence of structures becoming more slender is the increased susceptibility to human-induced vibrations. The low stiffness, damping and mass reduce the natural frequency close to the walking frequency of pedestrians with resonance as a result. High amplitude vibrations in footbridges can cause interference with activities or even a feeling of fear and panic among pedestrians. The evaluation of the serviceability for footbridges becomes therefore more important.

Numerous problems with vibration serviceability have been reported in the past including the T-Bridge in Japan (1989), the Pont du Solferino in Paris (1999) and the London Millennium Bridge in London (2000) [1]. As a result, in a short period of time different research studies have been carried out to cover this problem in a series of regulations in codes, guidelines, standards and publications for the design of footbridges. In the Netherlands the Eurocode (in case of footbridges parts of EN-1990 [2] and EN-1991 [3]), the corresponding National Annexes (EN-1990 NB [4] and EN-1991 NB [5]) and the EUR 23984 guideline "Design of Lightweight Footbridges for Human Induced Vibrations" [6] partly cover regulations within the field of vibrations in footbridges. In these documents, various topics that have to be taken into account in the design of footbridges are mentioned, but there is limited clarity for the users on a number of aspects and the provided information is not always consistent.

In the last few years no significant issues in the use of footbridges are reported, making it wonder if the current regulations result in conservative designs. On the other hand, future lively footbridges are expected to be more susceptible to serviceability problems that can result in discomfort for the pedestrians to varying degrees. To ensure safe and comfortable designs of footbridges in the future, more research within this field of study is desirable.

1.2. Research objectives

Many studies have been carried out on the topic of human-induced vibrations, even to a greater extent after the serviceability problems of the London Millennium bridge. In order to be able to perform a useful piece of research that is supplementary to the current work, first an overview of the available literature and valid codes and regulations in the Netherlands has to be outlined. Based on the bottlenecks that were encountered, an investigation and impact study have been carried out about the critical aspects in the evaluation of the serviceability of footbridges. The first aim of this study therefore is to present a good overview of those critical aspects so as to provide a firm basis for further research in general and for this master thesis in particular.

The second main goal follows from the first one. Freedom in design is in general favourable for engineering companies. At the moment the approach of the Eurocode is expected to be rather conservative as regards the evaluation of human-induced vibrations in footbridges. Although a choice for a different design scenario is allowed, it is hard to take advantage of this freedom. This is because the guidelines are not unequivocal for the application of human-induced loading and for the criteria regarding the human perception of bridge vibrations. This has resulted in the following second main goal: to give insight in the choice between different design scenarios by investigating whether the expectation that the Eurocode approach is conservative is well founded. Hence this study provides a more realistic approach of the vibration serviceability of footbridges due to human-induced vibrations.

1.3. Main research questions

The following main questions are formulated to deal with the research objectives:

1. *What are the critical aspects in the evaluation of human-induced vibrations in footbridges? - (Part I)*
2. *Can a probability-based approach demonstrate that the Eurocode is conservative in the evaluation of human-induced vibrations in footbridges? - (Part II)*

1.4. Outline report

The report is divided into three parts. The first part deals with the available literature, the valid codes and regulations in the Netherlands and the critical aspects regarding the design of footbridges (chapter 2 - 8). The second part focuses on a probabilistic approach of the vibration serviceability of footbridges under vertical pedestrian loading (chapter 9 - 12). The conclusions and recommendations are elaborated in respectively chapter 13 and 14. The third part includes the appendices that provide the background information including a summary of the interviews with the companies (appendix A).

I

Important and critical aspects human-induced bridge dynamics

2
Introduction Part I

In the past many studies have been performed on the topic of human-induced vibrations in footbridges. The aim of this first part is to get an overview of the available literature and valid codes and regulations in the Netherlands. In addition the goal of this part is to roughly investigate the topics that are not fully covered yet and to indicate the aspects that are of importance in prediction of vibrations in footbridges. The main research question is "*What are the critical aspects in the evaluation of human-induced vibrations in footbridges?*".

2.1. Methodology

The regulations, available literature and critical aspects regarding human-induced vibrations in footbridges have been evaluated with use of the following methods:

- *Literature study* - The codes and regulations that are valid in the Netherlands in the field of footbridges (the Eurocode, the Dutch National Annex and the EUR 23984) have been studied to investigate the bottlenecks in the evaluation of the human-induced vibrations. In addition, relevant papers and previous graduation projects ([7], [8] and [9]) have been used to investigate the critical aspects. (Chapter 3 - 5)
- *Visiting engineering companies* - In conversation with the engineering companies Arup, RoyalHaskoningDHV and GemeentewerkenRotterdam it has been endeavoured to detect the kind of problems companies face in practice, concerning the use of the codes and regulations in the evaluation of the human-induced vibrations in footbridges. Furthermore, the meetings have been used to figure out what the experiences are in the use of the footbridges after construction. The full result of these meetings is presented in appendix A. The conclusions are incorporated in chapter 5.
- *Impact study* - A sensitivity study has been carried out to investigate the impact of different aspects regarding human-induced vibrations on footbridges. Therefore rough calculations have been performed, based on the calculation methods for human-induced vibrations on footbridges according to the Eurocode, Dutch National Annex and EUR 23984. The impact study is carried out in chapter 6.

2.2. Scope

In the evaluation of human-induced vibrations in footbridges it is a challenge to take all the parameters of influence into account. In this work the assumption has been made that no other than pedestrian traffic is present on the footbridges.

Lateral vibrations are already studied extensively after the serviceability problems at the London Millennium Bridge as a result of a lateral lock-in effect. Furthermore the dynamic component of the human-induced loading is the largest in the vertical direction, resulting in vibrations with high amplitudes in this direction. Therefore the focus of this work is on vibrations in the vertical direction. If desirable, the same type of study can be performed and implemented for the lateral and longitudinal direction.

The impact study has been carried out on three simply supported bridges with a length of 12, 50 and 100 m. The worked example in the EUR 23984 for a bridge with $L = 50$ m has been used as a starting point. The bridges with a length of 12 and 100 m are based on this example, assuming steel as the main structural material. In addition, two cantilever footbridges have been evaluated for a length of 2.5 and 4 m.

2.3. Reading guide

First an overview of the available literature and valid codes and regulations in the Netherlands is provided. The important aspects in human-induced bridge dynamics are presented in chapter 3 and the Dutch regulations and assessment methods are discussed in chapter 4. This is followed by the identification and investigation of the critical aspects in chapter 5 and an impact study of the critical aspects in chapter 6. The results are presented and discussed in chapter 7. Finally, chapter 8 contains the conclusions of this first part. These conclusions have been a starting point for part II.

3

IMPORTANT ASPECTS HUMAN INDUCED BRIDGE DYNAMICS

3.1. Human-induced dynamic loading

The first important aspect in the analysis of human-induced vibrations in footbridges is the pedestrian loading. Different types of human-induced forces can be important for footbridges, as discussed below. The relevance of these types of loading is investigated further in chapters 5 and 6.

3.1.1. Single pedestrian

A stationary pedestrian can be represented by a static loading equal to the weight of the human body. When a persons starts walking, the centre of gravity of the human body moves, resulting in a dynamic loading on the structure. A single pedestrian produces a dynamic time varying force with components in three directions: vertical, longitudinal and lateral. The typical shapes of the walking force for the three directions is presented in figure 3.1.

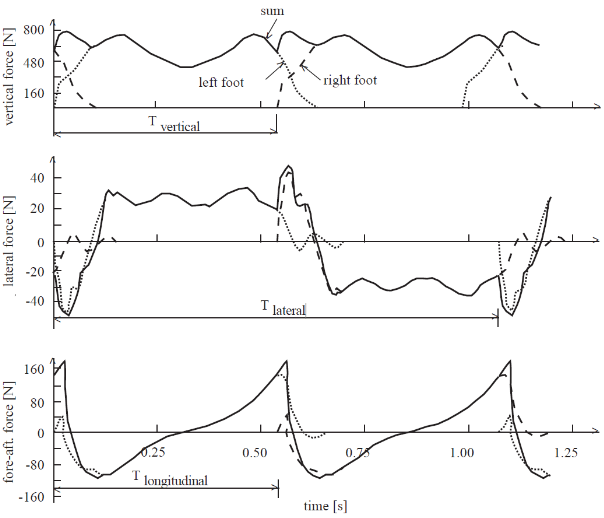


Figure 3.1: Typical shapes of walking force in vertical, lateral and longitudinal directions [10]

The walking force can be represented by a Fourier series based on the assumption the force is periodic and both feet produce the exactly the same force. For a single person the force in the vertical direction is a combination of the static and dynamic loading, as represented in equation 3.1.[10]

$$F_p(t) = G + \sum_{i=1}^{n} G\alpha_i \sin\left(2\pi i f_p t - \phi_i\right) \tag{3.1}$$

where the weight of the person is represented by G in N, f_p is the walking frequency in Hz, α_i is the Fourier's coefficient of the ith harmonic representing the dynamic load factor (DLF), ϕ_i is the phase shift of the ith harmonic, i is the order of the harmonic and n is the total number of contributing harmonics. It is pointed out that walking force can be represented very well including the first two Fourier's coefficients. For the evaluation of the human-induced dynamic behaviour of the footbridge, only the dynamic part of the pedestrian loading has to be taken into account.[11] [12]

Every individual has its own properties, therefore the human-induced loading differs from person to person. A significant number of studies on the parameters for single walking persons are performed, represented in the extensive literature review on the vibration serviceability of footbridges under human-induced excitation by Zivanovic, Pavic and Reynolds [10]. Figure 3.2 shows the result of an experiment by Kerr for the DLFs of the first four harmonics in relation with the waking frequency. The scatter in the results represents the diversity of the properties per individual person.

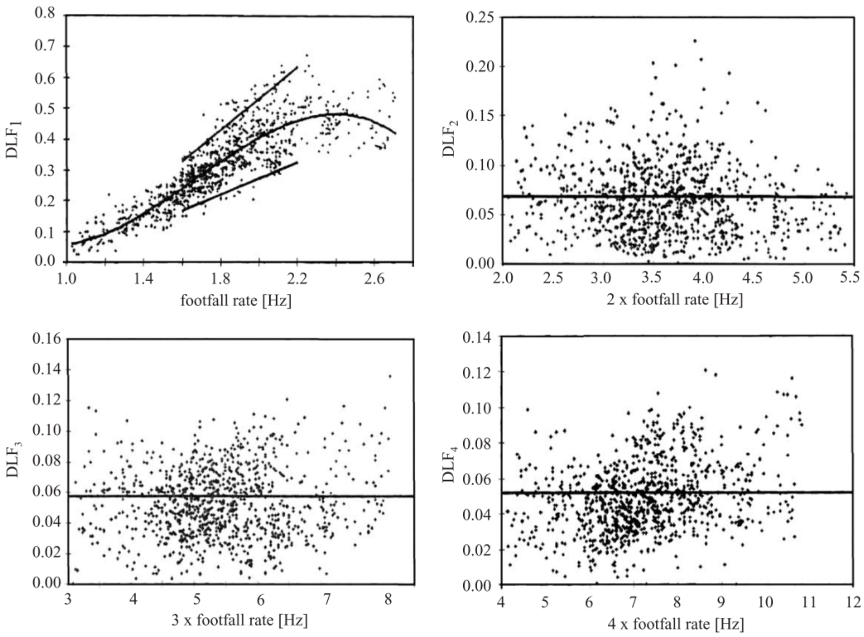


Figure 3.2: DLFs of the walking force for the first four harmonics after Kerr [13]

The walking frequency of pedestrians for the vertical and longitudinal direction is centered around 1.8 Hz for normal walking and changes with the walking velocity as can be seen both in figure 3.2 and 3.3 from 1 to 2.6 Hz.

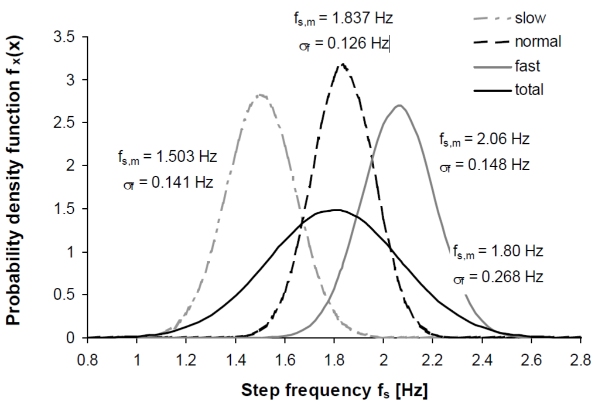


Figure 3.3: Probability density functions of the step frequencies regarding the walking intention by Butz [14]

3.1.2. Jogger loading

Instead of walking, people can jog or run across the bridge. In this case the velocity and frequency both increase and only the vertical component of the loading will remain like shown in figure 3.4 for slow jog and running. Whereas for normal walking the pedestrians have always contact with the structure, joggers detach themselves from the structure resulting in a time interval with no force present on the structure at all. In addition the first Fourier coefficient will increase, resulting in a higher dynamic component that for normal walking pedestrians.

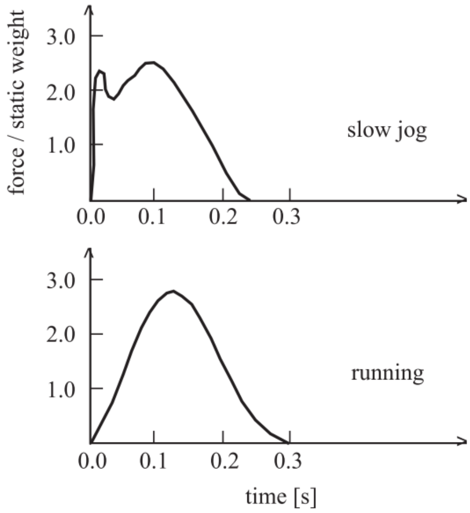


Figure 3.4: Typical vertical force patterns in relation to the static weight for slow jogging and running [10]

3.1.3. Vandal loading

A bridge can be affected to vandal loading. Vandalism is defined as "a specific kind of footbridge loading characterized by the intentional and well coordinated action of one or several persons, moving their own body with the sole aim to increase the structural vibration level to a maximum value".[15] Vandal loading is though not precisely defined in terms of type of human activity: deliberate jumping, horizontal body swaying and bouncing could be considered.[10]

3.1.4. Crowd loading

Instead of single pedestrians crowd loading is taken into account in the evaluation of human-induced vibrations in footbridges. A crowd is a group of random walking pedestrians with an individual frequency, velocity, phase and weight. The vibrations in footbridges that are the source of an uncomfortable feeling are mainly caused by crowd loading. The net result of this crowd loading is the force acting at the bridge deck. This force is not equal to the load of a single pedestrian times the number of pedestrians. A crowd is modelled as a flow of an equivalent number of pedestrians standing at a fixed position at the bridge with the same frequency, phase and weight.

People in large crowds can adjust their walking behaviour to the movement of the other pedestrians by changing their speed and walking frequency. This is behaviour is more likely if the crowd is dense.[10] In addition to synchronization within the crowd, pedestrians can synchronise their movement with the bridge vibrations as well. This behaviour is called lock-in and occurred at the London Millennium Bridge in 2000 for the lateral direction.

3.1.5. Special events

In addition to normal crowd loading, crowd loading at special events is an other typical pedestrian loading that can be expected on a footbridge. A special event can be for example the inauguration of a new bridge or a marathon. The crowd is expected to be more dense and walking more synchronously than normal crowd loading because of external effects like music. At those events it can be possible that panic occurs due to strong vibrations of the bridge, resulting in exceptional load cases. Typical for the crowd loading at special events is that it is expected to happen just once a year or once in the lifetime of a bridge.

3.2. Bridge structures and dynamic response

The pedestrian loading has to be supported by the structure of the footbridge. In the evaluation of the response of the structure to human-induced loading the properties of the footbridge structure are of significant importance.

3.2.1. Types of footbridges

Different types of footbridges can be distinguished. An example of the diversity of the bridge structures is presented in figure 3.5.

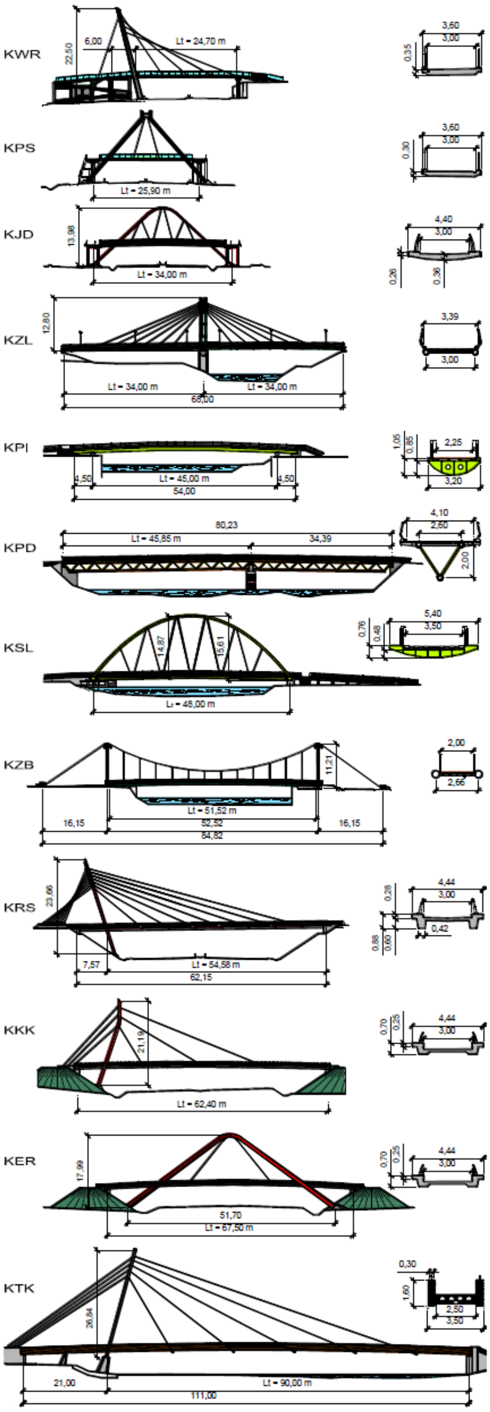


Figure 3.5: Examples of bridges structures, tested by Hawryszkow [16]

The distinction between the types of footbridges can be made based on different aspects, for example:

- *Length* - The total length of a footbridge can differ from a small span of about 3 m to for example the largest footbridge in the Netherlands with a length of 750 m and a midspan of 170 m (the Nesciobridge)[17].
- *Type of structure* - The structures can be divided in several ways, two examples of systematization are presented by Hawryszkow [16]: tension and beam structures; or cable-stayed, arched, beam, suspension and frame structures. A footbridge can be attached to another bridge as a cantilever beam as well.
- *Purpose* - The purpose of the bridge indicates the type of traffic that is expected.
- *Life time* - Roughly a distinction can be made between two lifetimes of the bridge: fixed or temporary. Both for the fixed as for the temporary bridge the lifetime can differ, but the choice between one of this two lifetimes will indicate the type of structure and probability of failure in relation with the comfort criteria that should be met.
- *Structural material* - The material of the bridge is of influence for the response of the bridge to vibration. The main structure can be built up from for example steel, concrete, timber, FRP or a combination of materials.

3.2.2. Bridge parameters

The parameters of the footbridge indicate the response of the structure to the pedestrian loading. The main topics influencing the (dynamic) response of the structure are presented in the following sub-paragraphs.

Structural damping

Damping is the energy dissipation in a vibrating structure.[10] It reduces the amplitude of vibration of an oscillatory system. The overall damping of a structure (often called the effective damping) consists of: material and structural damping; damping associated to furniture and finishing; and energy dissipation through special devices.[6] This overall damping is the modal damping that is measured in practice and it is hard to predict for many engineering structures. In general a viscous damping model is used assuming linear damping, modelled as a function of the velocity. Additional damping to the structure can be achieved by using a tuned mass damper (TMD).

Mass

The mass of the footbridge is mainly expressed in kg/m and can vary over the length of the structure. The mass is dependent on the geometry of the structure (the cross section A) and the material properties (the volumetric mass density ρ).

Stiffness

The stiffness of the footbridge is dependent on the geometry of the structure (the moment of inertia I) and the material properties (the Young's modulus E). Similar to the mass, the stiffness can vary over the length of the bridge.

Support conditions

The behaviour the structure under pedestrian loading is dependent the support conditions of the footbridge. The three basic support conditions are clamped, simply supported and free. The rows of the table in figure 3.6 provide examples of different combinations of these support conditions.

Mode shapes

The mode shapes of the bridge are related to the support conditions. Figure 3.6 shows different mode shapes, related to this support conditions. Each mode shape has its own corresponding natural frequency. For the mode shapes in figure 3.6 the proportion of the natural frequencies is indicated by the value of C, that has to be entered in equation 3.2.

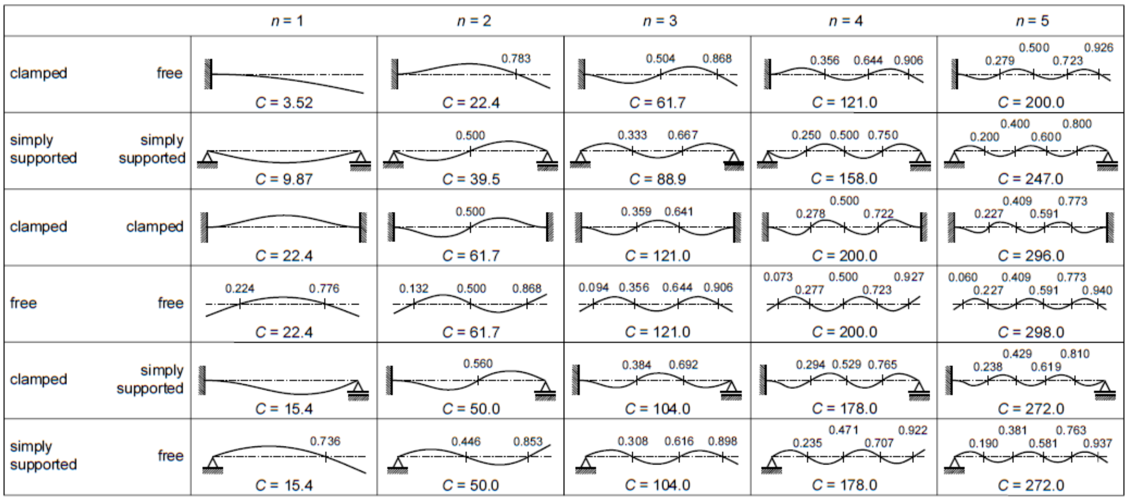


Figure 3.6: Natural frequencies and eigenmodes of beams with constant $EI\rho A$

Natural frequency

The natural frequencies of the footbridges indicate whether or not the footbridge will be susceptible for human-induced vibrations. In case the natural frequency of the bridge is in the critical range of the walking frequency of pedestrians, the bridge can be put in resonance by the pedestrian loading. The natural frequencies of the structures in figure 3.6, footbridges with a constant mass and stiffness over the length, can be calculated with equation 3.2:[18]

$$f_n = \frac{1}{2\pi}\sqrt{\frac{K_n^*}{M_n^*}} = \frac{C}{2\pi}\sqrt{\frac{EI}{\rho A L^4}} \tag{3.2}$$

Where the natural frequencies of the bridges are depending on the modal mass M_n^*, the modal stiffness K_n^* and the support conditions, represented by the factor C. When the damping ratio of the structure is considered in the determination of the natural frequency, equation 3.3 should be used. For a small damping ratio, the influence of the damping ratio is little. Therefore this term is usually not considered.

$$f_d = f_n\sqrt{1-\xi^2} \tag{3.3}$$

3.2.3. Response of the structure to pedestrian loading

Resonance

In case the walking frequency of pedestrians is close to the natural frequency of the footbridge, resonance can occur. This resonance corresponds to vibration of the structure. The response of the structure to the dynamic loading of the pedestrians is determined by means of a dynamic amplification factor (DAF). This DAF increases when the ratio of the walking frequency of the pedestrians and the natural frequency of the structure is closer to 1. The higher the damping ratio, the lower the response of the structure to the dynamic pedestrian loading (see figure 3.7). Damping in civil engineering structures in general is small, so that the resonant rise close to the natural frequency is expected to be large.[18]

Static versus dynamic loading

A bridge has to be designed to resist both the static (in the ultimate limit state) and the dynamic (in the serviceability limit state) loading. The question is which one is governing in the design. The static deflection and stresses are calculated with a static load between 2.5 and 5.0 kN/m^2 depending on the length of the bridge. Assuming a person has an average weight of 700 N, this results in 3.6 till 7.1 P/m^2 (persons per square meter). Comparing this with the dynamic loading with a standard of 0.5 P/m^2 it can be seen that this dynamic load, apart from the fact that a dynamic load can cause high vibrations due to resonance, is a small fraction of the static load that needs to be used in the design. The dynamic loading will not be governing for the structural safety of the structure in case of normal pedestrian traffic, as the stresses in the structure are significantly smaller than under static loading. However, special types of loading like vandal loading could result in high stresses.

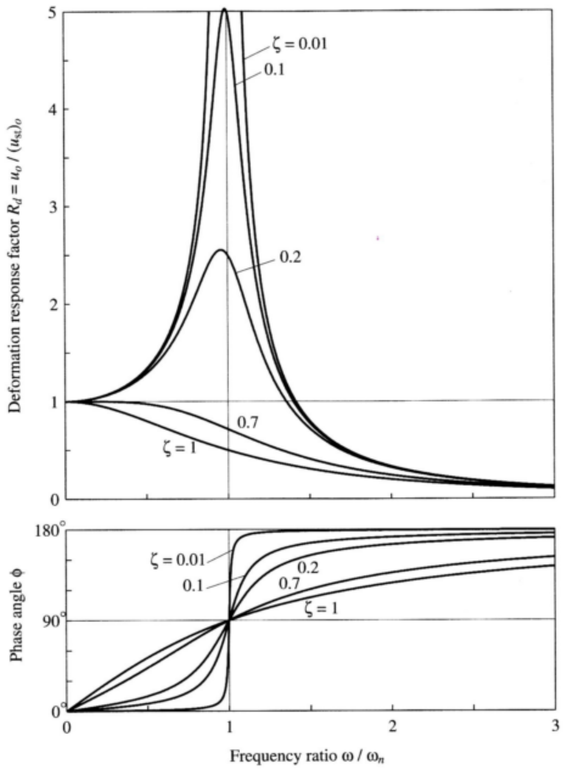


Figure 3.7: Deformation response factor for a damped system excited by a harmonic force [19]

3.3. Assessment criteria and reliability

3.3.1. Reliability

The reliability of a structure is expressed in the failure probability or risk of failure (see figure 3.8). Failure occurs when the load (curve b) exceeds the resistance (curve a). The acceptable risk depends on the limit state that is considered. Two limit states are of interest in the design of structures: the serviceability limit state (subsection 3.3.2) and the ultimate limit state (subsection 3.3.3).

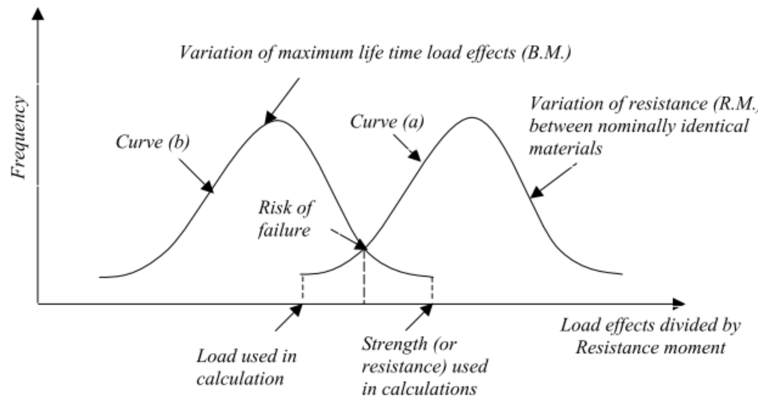


Figure 3.8: Risk of failure of a structure [20]

3.3.2. Serviceability limit state

The serviceability limit state (SLS) deals with comfort of the users and the functioning of the structure. The allowed SLS failure probability for irreversible damage for the same structure is $6.681 \cdot 10^{-2}$. A SLS failure probability for reversible damage of 10^{-2} (1%) is prescribed for buildings based on the reference period. This value may also be used for bridges.[2] The ISO 2394 on "General principles on reliability for structures" indicates the following topics as part of the evaluation for the serviceability limit state:

- *Local damage* - reduction of the working life of the structure or affection of the efficiency or appearance of structural or non-structural elements;
- *Unacceptable deformations* - the efficient use or appearance of structural or non-structural elements or the functioning of equipment can be affected;
- *Excessive vibrations* - discomfort to people or affect non-structural elements or the functioning of equipment can be the result.[21]

3.3.3. Ultimate limit state

The ultimate limit state (ULS) corresponds to a state associated with structural failure, dealing with the safety of the people or the structure. The ULS failure probability of a structure built for 50 years is prescribed to be $7{\cdot}10^{-5}$ according to the Eurocode. The ISO 2394 indicates the following topics as part of the evaluation for the ultimate limit state:

- Loss of equilibrium of the structure or of a part of the structure;
- Attainment of the maximum resistance capacity of sections, members or connections by rupture or excessive deformations;
- Transformation of the structure or part of it into a mechanism;
- Instability of the structure or part of it;
- Sudden change of the assumed structural system to a new system.[21]

3.3.4. Comfort

In the evaluation of the response of the footbridge under dynamic pedestrian loading, the level of human comfort to the vibrations of the structure has to be considered. The evaluation of (dis)comfort can be related to the serviceability limit state. Vibrations can be measured and expressed in several ways. Usually, the vibrations are measured expressed in accelerations of the structure. This is because it was established as the best parameter for describing people's reaction to vibrations and the level of comfort. Furthermore, it is easy to measure by using widely available accelerometers. Zivanovic concludes that at the moment it has been generally accepted that acceleration is the vibration parameter which should be used to describe the vibrations in footbridges.[10]

4

DUTCH REGULATIONS AND ASSESSMENT METHODS

Introduction

In 2012 the Eurocodes have been included in the Dutch building regulations. The Eurocodes are a series of ten European Standards providing technical rules for the structural design of constructions works in the European Union (EU). All the members of the EU may add a so called National Annex (NA) including national parameters, with differences between the countries as a result. The design of footbridges is verified by means of this Eurocode and National Annex, providing the requirements for the design. The National Annex recommends to make use of the EUR 23984 guideline: "Design of Lightweight Footbridges for Human Induced Vibrations"[6]. In this chapter the regulations regarding the design of footbridges by the Eurocode and the National Annex (section 4.1) and by the EUR 23984 guideline (section 4.2) are presented.

4.1. Eurocode and National Annex

The parts of the Eurocode and National Annex that cover the design of footbridges are EN 1990 (NA) and EN 1991 (NA). The content is presented in this section, in which the differences between the Eurocode and the National Annex are highlighted per section. It must be noted that fatigue calculations are not considered in the regulations for the design of footbridges.

4.1.1. Design situations (EN 1990)

Eurocode

The following items are set for the design situations that have to be taken into account in the design of footbridges:

1. The design situations have to be selected depending on the allowed amount of pedestrian traffic at the footbridge during the design lifetime of the structure.

2. A group of 8 to 15 people, normally walking, has to be taken into account in permanent design situations.

3. Different traffic categories have to be specified related to the design situations, for example:
   (i) The presence of pedestrian streams (considerably more than 15 people)
   (ii) Special festival or choreographic directed events

Two remarks are made:
i. It can be necessary to specify the traffic categories and corresponding design situations per project.
ii. The determination of the design situation dealing with special festival or choreographic directed events depends on the expected degree of surveillance by the responsible client or authorities. Some information can be found in professional literature.

National Annex
In the National Annex no further information is specified.

4.1.2. Comfort criteria (EN 1990)
Eurocode
1. The maximum allowed acceleration of a random part of the deck caused by wind or pedestrian loading are defined in terms of maximum acceleration. The recommended values are:
   (i) 0.7 m/s^2 in case of vertical vibrations for normal use;
   (ii) 0.2 m/s^2 in case of horizontal vibrations;
   (iii) 0.4 m/s^2 in case of exceptional circumstances of a crowd;
   Remark: In the project specification, other maximum permissible accelerations in relation to comfort criteria can be defined.

2. Verification of the requirements of the comfort criteria should be made in case the natural frequency of the deck is smaller than:
   (i) 5 Hz in case of vertical vibrations;
   (ii) 2.5 Hz in case of horizontal and torsional vibrations.

National Annex
In the National Annex point 2 and item (iii) from point 1 are omitted and the following remark is made:
The first two items correspond to the medium comfort class (CC2) in combination with traffic class 3 according to the EUR 23984. In general it is advised to use the values of the EUR 23984.

4.1.3. Static models for vertical loads (EN 1991)
Eurocode
The following static loads models for the vertical direction have to be taken into account:

1. Uniform distributed load $q_{fk} = 5$ kN/m^2;

2. Concentrated force $Q_{Fvd} = 10$ kN at a area of 0.10 m x 0.10 m;

3. Service vehicle Q_{serv}, if no information is available, the load model of an extraordinary vehicle may be used (see item 4):

4. Extraordinary vehicle: a biaxial load settlement of 80 kN and 40 kN with wheelbase 3 m.

The load combinations presented in table 4.1 have to be checked.

Table 4.1: Definition load combinations (characteristic values) - EN 1991

Type of load	Vertical forces		Horizontal forces
Load system	Uniformly distributed load	Service vehicle	
Group 1	q_{fk}	0	Q_{flk}
Group 2	0	Q_{serv}	Q_{flk}

National Annex
The following static loads models for the vertical direction have to be taken into account:

1. Uniform distributed load $q_{fk} = 5$ kN/m^2;

2. Concentrated force $Q_{Fvd} = 7$ kN at an area of 0.10 m x 0.10 m;

3. Service vehicle $Q_{serv} = 25$ kN as the characteristic value of the axle load, taking into account two axles with wheelbase 3 m;

4. In case no permanent obstacle prevents a vehicle is driven on the bridge, an extraordinary vehicle has to be taken into account: a biaxial load settlement of 80 kN and 40 kN with wheelbase 3 m.

Table 4.2: Definition load combinations - EN 1991 NA

Type of load	**Vertical forces**		**Horizontal forces**
Load system	Uniformly distributed load	Service vehicle	
Group 1	*characteristic value*	0	*characteristic value*
Group 2	0.8 * *characteristic value*	*characteristic value*	*characteristic value*

4.1.4. Dynamic models of pedestrian loads (EN 1991)

Eurocode

Three items are pointed out for the dynamic models of pedestrian loading:

1. The natural frequencies of the main structure have to be determined;
2. Load exerted by pedestrians with a frequency equal to the natural frequencies of the bridge can cause resonance: this should be taken into account in the verification of the limit states with respect to vibrations;
   (i) in vertical direction 1 - 3 Hz
   (ii) in horizontal direction 0.5 - 1.5 Hz
   (iii) 3 Hz in case of joggers
3. Appropriate dynamic models and comfort criteria for pedestrian loading have to be determined.

National Annex

In the National Annex the following adjustments and additions to the Eurocode are made:

1. *no adjustments*
2. *no adjustments*
3. Replacement Eurocode: load models in appendix A of the National Annex have to be taken into account;
4. In case of normal use traffic class 3 and loading by joggers has to be taken into account: for L ≤ 20 m 5 joggers, for L > 20 m 10 joggers at a fixed normative position;
5. In the project specifications different traffic classes with corresponding comfort criteria can be defined;
6. It must be shown that the bridge can not be put in vibrations due to vandalism such that the ultimate limit state is exceeded: traffic class 5 in combination with 50% of the nominal structural damping may be assumed.

4.1.5. Crowd loading (EN 1991)

Eurocode

The Eurocode doesn't provide models for the determination of the dynamic loads.

National Annex

In appendix A of the national annex of EN 1991 models for dynamic loads at footbridges are presented. Use is made of the following harmonic load model:

$$p(t) = P\cos(2\pi f_s t)n'\psi \tag{4.1}$$

In which $P\cos(2\pi f_s t)$ is the harmonic load excited by a single pedestrian. P is the dynamic force excited by a single pedestrian with step frequency f_s, subdivided into a vertical, longitudinal and lateral direction. The step frequency is assumed to be equal to the considered natural frequency of the bridge. The equivalent number of pedestrians at the loaded surface S is n'. ψ is the reduction coefficient related to the chance that the step frequency is indeed close to the critical range of natural frequencies.

The force P excited by a single pedestrian is divided into three components for the three different directions as shown in table 4.3. For the determination of the reduction coefficient ψ, the two graphs presented in figure

Table 4.3: Component of the force P excited by a single pedestrian

Vertical	Longitudinal	Lateral
280 N	140 N	35 N

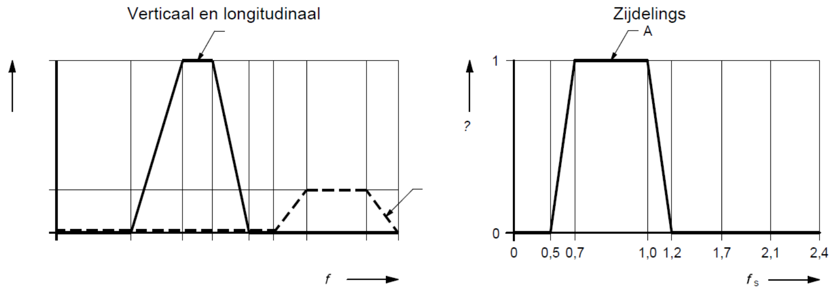


Figure 4.1: Reduction coefficient ψ according to the Eurocode [5]

4.1 have to be used. Both graphs are however missing values and/or symbols at the axis and are therefore not useful. To determine n' two different formulas are presented, depending on the traffic class (density of the crowd expressed in persons per square meter) as shown in table 4.4.

Table 4.4: Equivalent number of pedestrians n' per traffic class

Traffic class	Crowd density d (1/m^2)	Equivalent number of pedestrians n' (1/m^2)
TC1 TC2 TC3	$d < 1.0$	$n' = \frac{10.8\sqrt{\xi n}}{S}$
TC4 TC5	$d \geq 1.0$	$n' = \frac{1.85\sqrt{n}}{S}$

In which n is the number of pedestrians at the loaded surface S ($n = S \cdot d$), where d represents the number of pedestrians per m^2 and S the loaded surface in m^2. ξ is the logarithmic decrement of the damping.

4.1.6. Joggers (EN 1991)

Eurocode

The Eurocode does not provide models for the determination of the loading that is induced by joggers.

National Annex

The load model for joggers is a single load $P(t, v)$ moving across the bridge with a velocity v. Joggers run with a velocity higher than 3 m/s across the bridge. It is assumed that $v = 0$ for which the load $P(t, v = 0)$ is set at the maximum displacement amplitude of the mode shape.

$$P(t, v) = P\cos(2\pi f t)n'\psi \tag{4.2}$$

The maximum force P is presented in table 4.5: only the vertical component of the force is considered. The reduction coefficient can be taken from figure 4.2 and the equivalent number of joggers n' is equal to the number of joggers at the loaded surface: $n' = n$.

Table 4.5: Component of the force P excited by a jogger

Vertical	Longitudinal	Lateral
1250 N	-	-

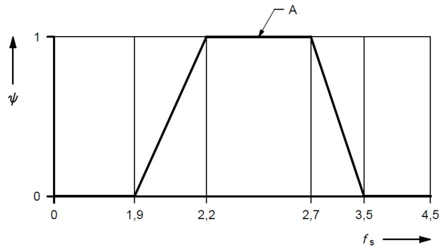


Figure 4.2: Reduction coefficient ψ for a single jogger [5]

4.2. Design of lightweight footbridges for human induced vibrations - EUR 23984

The Eurocode and corresponding National Annex recommend to make use of the guideline "Design of lightweight footbridges for human induced vibrations"[6], also known as the EUR 23984. In the following section the main design principles of a footbridge according to this document are presented. The EUR 23984 line is based on two other main guidelines in the field of the design of footbridges: "Advanced load models for synchronous pedestrian excitation and optimised design guidelines for steel footbridges (SYNPEX)" [14] and "Human induced vibrations of steel structures (HiVoSS)". In addition these documents refer frequently to "Footbridges: Assessment of dynamic behaviour under the action of pedestrians" (Sétra) [12], this is therefore an important guideline to mention as well.

4.2.1. Design procedure

The EUR 23984 is composed of different steps, presented in figure 4.3. The guideline only treats the reversible serviceability, as defined by the Eurocode.

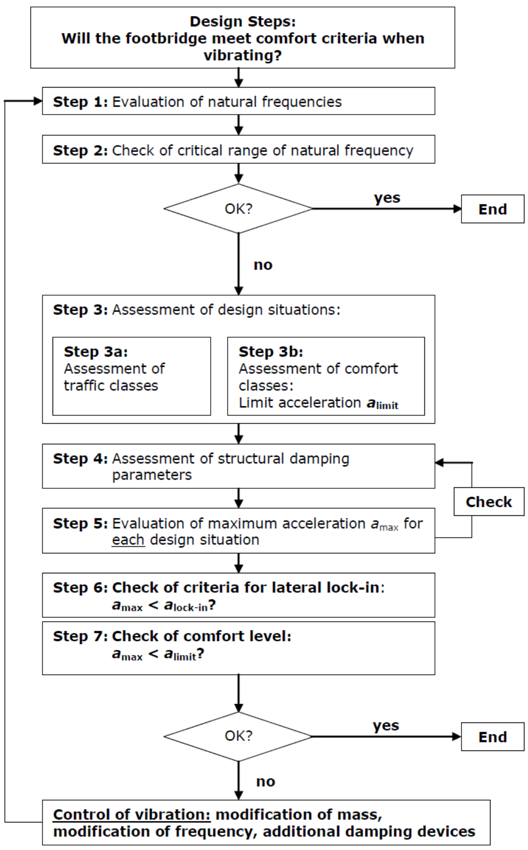


Figure 4.3: Flowchart for the use of the EUR 23984 [6]

4.2.2. Evaluations of natural frequencies

First the natural frequencies of the footbridge have to be calculated, in order to check if they are in the critical range of the walking frequencies of pedestrians. These natural frequencies can be determined in several ways, for example by use of a finite element method or with hand formulas.

In case the modal mass of the pedestrians is more than 5% of the modal mass of the deck it is recommended to consider the mass of the pedestrians in the evaluation of the natural frequencies. The modal mass of the system is now a combination of the modal mass of the structure and the modal mass of the pedestrians.

4.2.3. Check critical range of natural frequencies

After the evaluation of the natural frequencies it must be checked whether those natural frequencies are in the critical range of the walking frequencies of pedestrians. The critical range is the range of natural frequencies for which the structure can be excited to resonance by pedestrians. In case the natural frequencies of the footbridge are within this critical range, the footbridge has to be checked to excitation by pedestrians. If the natural frequencies of the footbridge are outside of the critical range, no check has to be performed.

The critical range for natural frequencies f_i is:

- For vertical and longitudinal vibrations (1st and 2nd harmonic): 1.25 Hz $\leq f_i \leq$ 4.6 Hz
- For lateral vibrations: 0.5 Hz $\leq f_i \leq$ 1.2 Hz

4.2.4. Assessment of design situation

Several significant design situations have to be specified. A design situation is defined as a "set of physical conditions representing the real conditions occurring during a certain time interval". This can be set up by the combination of an expected traffic class from figure 4.4 and a chosen comfort level as defined in figure 4.5. It should be noted that pedestrian formations, processions or marching soldiers are not taken into account in the general traffic classification, but it is mentioned that they do need additional consideration.

4.2.5. Assessment of structural damping

The model used for the specification of the structural damping makes use of linear viscous dampers: the generation of damping forces is proportional to the rate of change of the displacements with time (velocity). For the determination of the damping minimum and average damping rations are recommended, shown in table 4.6. Large levels of oscillation, for example by intentional loads, lead to higher damping ratios, shown in table 4.7.

Table 4.6: Damping ratios according to construction material for serviceability

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 %

Table 4.7: 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 %

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

Figure 4.4: Traffic classes presented in the EUR 23984 [6]

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

Figure 4.5: Comfort classes presented in the EUR 23984 [6]

4.2.6. Determination of maximum acceleration

For each design situation the maximum accelerations of the bridge have to be determined. There are several ways to calculate the maximum acceleration as shown in figure 4.6. The single degree of freedom (SDOF) method and the response spectra (RS) method are hand calculations and the finite element (FE) method makes use of a FE program. It has to be noted that experience has shown that it is very difficult to predict the structural damping of the finished footbridge. Damping therefore always has a broad scatter and consequently the calculated acceleration has a broad scatter as well.

Harmonic load model

To determine the acceleration with the SDOF or FE method use is made of a harmonic load model. For the modelling of a pedestrians flow consisting of n "random" pedestrians, the idealised flow consisting of n' perfectly synchronized pedestrians should be determined. In this model the influence of the response of structure on the behaviour of the pedestrians is not taken into account. The uniformly distributed harmonic

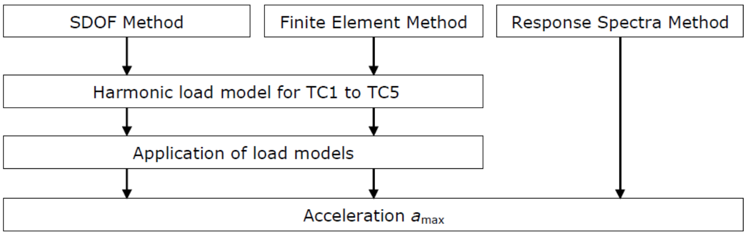


Figure 4.6: Methods for calculating the maximum acceleration [6]

load $p(t)$ represents the idealized stream of pedestrians, presented in equation 4.3.

$$p(t) = P\cos(2\pi f_s t)n'\phi \tag{4.3}$$

In which P is the component of the force due to a single pedestrian with a walking step frequency f_s in N. f_s is the step frequency, which is assumed equal to the footbridge natural frequency, expressed in Hz. The equivalent number of pedestrians on the loaded surface S is represented by n' expressed in m^{-2}. S is the area of the loaded surface in m^2. ϕ is the reduction coefficient.

The parameters P, n' and ψ are defined in figure 4.7 for the different traffic classes considering the excitation in the first of second harmonic of the pedestrian load. In case of dense pedestrian streams like TC4 and TC5 walking gets obstructed, the forward movement of the stream gets slower and the synchronization increases. Walking of pedestrians is impossible beyond the upper limit value of 1.5 $\mathrm{P/m}^2$. It is mentioned that in this case the dynamic effects significantly reduce: the correlation between pedestrians increases, but the dynamic load tends to decrease.

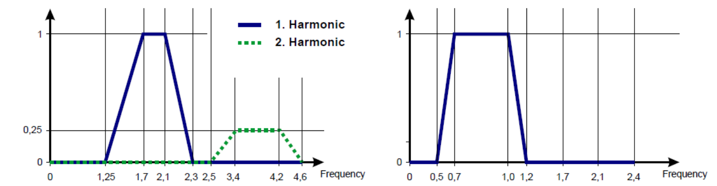
P [N]		
Vertical	Longitudinal	Lateral
280	140	35
Reduction coefficient ψ		
Vertical and longitudinal		Lateral
		
Equivalent number n' of pedestrians on the loaded surface S for load model of:		
TC1 to TC3 (density d < 1,0 P/m²):	$n' = \frac{10{,}8\sqrt{\xi \times n}}{S}$ [m⁻²]	
TC4 and TC5 (density $d \geq$ 1,0 P/m²):	$n' = \frac{1{,}85\sqrt{n}}{S}$ [m⁻²]	

Figure 4.7: Parameters for the harmonic load model [6]

The harmonic load $p(t)$ is applied on the structure such that it follows the considered mode shape as shown in figure 4.8. This is explained in more detail in appendix C.

SDOF method

The dynamic behaviour of a structure can be described by a linear combination of different harmonic oscillations in the natural frequencies of the structure. For each natural frequency of the footbridge in the critical range of natural frequencies, a single equivalent SDOF system will be used to calculate the associated maximum structure acceleration for dynamic loading (see figure 4.9). For each individual SDOF system the

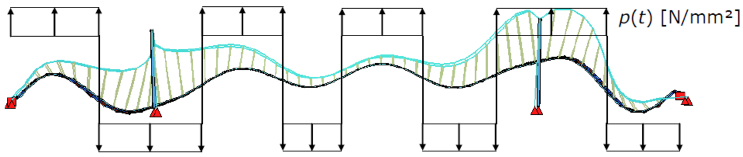


Figure 4.8: Application harmonic load [6]

maximum acceleration a_{max} at resonance caused by dynamic loading is now calculated with equation 4.4.

$$a_{max} = \frac{p^*}{m^*} \frac{1}{2\xi} \tag{4.4}$$

In which p^* and m^* are respectively the generalised load and generalised mass and ξ is the structural damping ratio, determined according to subsection 4.2.5. In appendix B is explained how to determine this generalised mass and load.

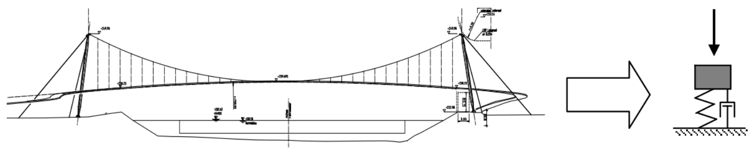


Figure 4.9: Equivalent SDOF oscillator for one natural frequency [6]

Finite element method

The same harmonic load model used for the SDOF method can be applied to a finite element model: this method is called the finite element method (FE method). After applying the load, the response of the FE model will provide the maximum accelerations that may occur.

Response spectra method

The third method is the response spectra method (RS method). It can be used to find a simple way to describe the stochastic loading and system response that provide design values with a specific confidence level. The following assumptions are made:

- The mean step frequency $f_{s,m}$ of the pedestrian flow 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 model coupling exists;
- The structural behaviour is linear-elastic.

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

$$a_{max,d} = k_{a,d}\sigma_a \tag{4.5}$$

The EUR 23984 provides formulas and tables to derive both factors, which are derived from Monte Carlo simulations based on numerical time step simulations of various pedestrian streams on various bridges geometries.

4.2.7. Check of criteria for lateral lock-in

Lateral lock-in can be triggered by a certain number of pedestrians N_L or an acceleration amplitude $a_{lock-in}$. These values are epected to lead to a vanishing of the overall damping producing a sudden response.

$$a_{lock-in} = 0.1 \text{ to } 0.15 \text{ m/s}^2 \tag{4.6}$$

$$N_L = \frac{8\pi\xi m^* f}{k}\frac{1}{2\xi} \tag{4.7}$$

In which ξ is the structural damping ratio. m^* is the modal mass. f is the natural frequency of the bridge and k is a constant (300 Ns/m approximately over the range 0.5 - 1.0 Hz).

4.2.8. Check of comfort criteria

The response acceleration calculated for the specified load models has to be compared with the specified comfort limits according to the corresponding design situation. If these do not comply, measurements to improve the dynamic behaviour of the footbridge have to be applied, including:

- Modification of the mass
- Modification of the frequency
- Modification of structural damping
- Addition of damping

4.2.9. Jogger

A single load $P(t,v)$ which is moving across with a velocity v the bridge is proposed for the joggers. The harmonic load for the evaluation of human-induced vibrations by joggers is represented by equation 4.8.

$$P(t,v) = P \cdot \cos\left(2\pi f t\right) \cdot n' \cdot \psi \tag{4.8}$$

The parameters are equal to the parameters of equation 4.3. For the determination of the reduction factor ψ and the equivalent number of pedestrians n' figure 4.10 has to be used. Only the vertical component is prescribed in the evaluation of the dynamic loading by joggers. It must be mentioned as well that the walking frequencies of joggers differ from normal walking pedestrians.

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

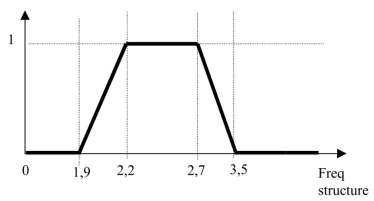


Figure 4.10: Parameters for joggers in the EUR 23984 [6]

It is considered that a group of joggers is perfectly synchronized in frequency and phase with the footbridge. A velocity of 3 m/s is mentioned, but it is indicated as well that it seems sufficient to place the load $P(t, v = 0)$ at the maximum displacement amplitude of the mode shapes.

5

IDENTIFICATION AND INVESTIGATION CRITICAL ASPECTS

The critical aspects in human-induced bridge dynamics are identified by means of literature study, the work of previous graduate students ([7], [8] and [9]) and meetings with engineering companies (ARUP, Royal-HaskoningDHV and GemeentewerkenRotterdam) and with the supervisors of this work. The critical aspects are divided into four main topics: design scenarios (section 5.1), load definition (section 5.2), structural response (section 5.3) and interaction effects (section 5.4).

5.1. Design scenarios

"A common human problem is that motion causes the pedestrian to become anxious about the safety of the structure even to the extent of refusing to use it. In such cases the actual danger of structural collapse is most unlikely, the strains involved often being 10 to 100 times less than those which might initiate damage. Nevertheless it is a serious matter for the designer and account must be taken of the human response to vibration in terms of disquiet anxiety or even fear." Bachmann [22]

5.1.1. Lack of clarity

In one of the first stages of the project the client has to define the design situations which the footbridge must meet. Each design situation is a combination of a traffic class in P/m^2 and a level of comfort in m/s^2. In the EUR 23984 a distinction is made in five traffic classes (figure 4.4) and four comfort classes (figure 4.5) as mentioned before. The Eurocode advices to use a load model corresponding to traffic class 3 ($0.5\ P/m^2$) in case of normal use of the footbridge. The comfort criteria that is recommended to be met in this case is $0.7\ m/s^2$ in case of vertical vibrations and $0.2\ m/s^2$ for vibrations in horizontal direction. However, it is mentioned as well that in the specifications of a project other traffic classes with related levels of comfort can be captured.[4] [5] The impact of the choice for different design situations on the design of the footbridge can be considerable. The EUR 23984 indicates the importance of a realistic assumption of the different design situations. Recommendations of how to make this realistic assumption are however not provided.

Figure 5.1 shows the wide deviation in criteria for comfort that are provided in literature. Some of these comfort criteria are dependent on the frequency, the others seems fixed values. The probability of the comfort of a single person to these accelerations is not provided. Therefore these criteria are used as fixed values, independent of the individual person.

Heinemeyer and Feldmann advise to consider three topics for assessing design scenarios. First it is indicated that in case the specifications are too severe, it may not be feasible to design slender structures. The occurrence of traffic should in addition be taken into consideration: minimum comfort may be sufficient for unusual traffic, whereas a higher comfort level may be more adequate for frequent traffic situations. Finally the location of the bridge is of importance: a footbridge close to a hospital might need a higher comfort level than the footbridge in a park.[23] They captured these considerations in a specification matrix presented in figure 5.2. The EUR 23984 indicates in addition that the assessment of vibrations includes soft aspects as well, such as the height above the ground, the frequency of use, the expectancy of vibration and the exposure

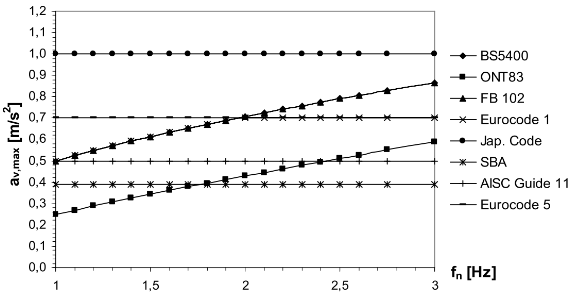


Figure 5.1: Limit values of vertical acceleration according to International Standards [14]

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

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

Figure 5.2: Specification matrix for design scenarios by Heinemeyer and Feldmann [23]

time.[6] The Setra guideline underlines that the subjectivity of comfort is depending on, among other things, the individual pedestrian.[12]

The issues the engineering companies face in the choice for design scenarios differ (see appendix A). On the one hand the choices are agreed with the client based on the traffic classes and comfort levels provided by the EUR 23984. On the other hand it is mentioned that the client is not familiar with these design scenarios and more guidance in this choice would be favourable. In addition the traffic classes are expected to be unrealistic high for large spans, resulting in designing for special events for large footbridges all the time. In case of comfort, other aspects like the circumstances and expectations turned out to be of influence in addition to vibrations. The exposure time is foreseen to have an impact as well.

A choice for design scenarios that will be taken into account in the design has to be made by the client, the consequences of the choices are however not clear. The difference between the traffic classes are investigated in the impact study in section 6.4.1.

5.1.2. Exceptional load cases

Due to the collapse of a bridge in the past it is no longer aloud for soldiers to march across a bridge. A more recent situation occurred at the opening of the London Millennium Bridge: the bridge experienced large vibrations in lateral direction due to crowd loading and interaction between the pedestrians and the structure. These incidents show that a high level of synchronisation between humans can cause serious problems. Next to the regular design situations the possibility of the occurrence of special load case should therefore be taken into account. According to the Eurocode "Determining the design situations that deal with occasional festival or choreographic registered events depends on the expected degree of surveillance by the responsible client or government."[2]. In addition the EUR 23984 mentions "Pedestrian formations, processions or marching soldiers are not taken into account in the general traffic classification, but need additional consideration."[6]

Both the Eurocode and the EUR 23984 recognize the importance of taking special load cases into account in the design. The question is how likely it will be that these exceptional load cases will occur in the lifetime of a bridge. On the other hand, if such an exceptional load case occurs, by definition a large amount of people is involved and exposed to the expected vibrations. Therefore, the choice for a corresponding comfort class is not obvious. Looking at the risk, the probability of occurrence is small, but the consequences are higher. It is important to know for sure if a special event can cause damage to the bridge or if it is a serviceability problem expressed in comfort. This is part of the impact study in section 6.4.2.

Exceptional load cases like special events are in general not taken into account in the design according to the engineering companies. In case it turns out in the conversation with the client that such kind of exceptional loadings are expected to occur, the check will be performed.

5.1.3. Probability of occurrence and consequences

Risk is defined as the probability of occurrence times the consequences. In relation to the human-induced loading for footbridges the probability of occurrence for certain traffic classes should be taken into account in combination with the risk that the client is willing to take. This defines the consequences (the vibrations) the bridge should be designed for. The probability of occurrence of certain traffic classes in combination with the consequences of the vibrations for that particular situation and for individual pedestrians are of importance.

These aspects are not taken into account in the current design methodology.

In reality the maximum accelerations that are used as a measurement for comfort at the moment occur only at specific point at the bridge and for a short time (a peak value). The probability that an individual pedestrians is exposed to this peak acceleration is not taken into account. In addition it is not expected that in reality a constant flow of pedestrians in one specific density is present all the time. The combination of these elements results in a probability that a person feels uncomfortable at a bridge. It is interesting to determine the probability of discomfort for an individual person in the use of the bridge. The acceptance of discomfort by the client can be related to the probability of discomfort for the users in order to determine if the design is correct. Other aspects like the duration of exposure are expected to be of influence as well in the analysis of the probability of discomfort.

In the EUR 23984 it is mentioned that in case of dense streams, walking gets obstructed. The correlation between pedestrians increases, but the dynamic load tends to decrease. Beyond the upper limit value of 1,5P/m^2 walking of pedestrians is impossible, so that dynamic effects significantly reduce.[6] Higher densities are of no risk and therefore do not have to be taken into account. A risk assessment for footbridges provided by Butz is presented in figure 5.3. The risk is defined as the combination of the frequency of occurrence of a defined hazard and the consequences of its occurrence.[14] The conditions in which a hazard can occur, can highly affect the frequency of occurrence of this hazard and the acceptance of its consequences. Therefore design situations for the design of footbridges are introduced. These have to be defined by the client. According to Augusti the most rational way of addressing such risks is Performance-Based Design or Performance-Based Engineering. With a sufficiently high probability, the satisfaction of relevant performance requirements are required throughout the lifetime of the facility.[24]

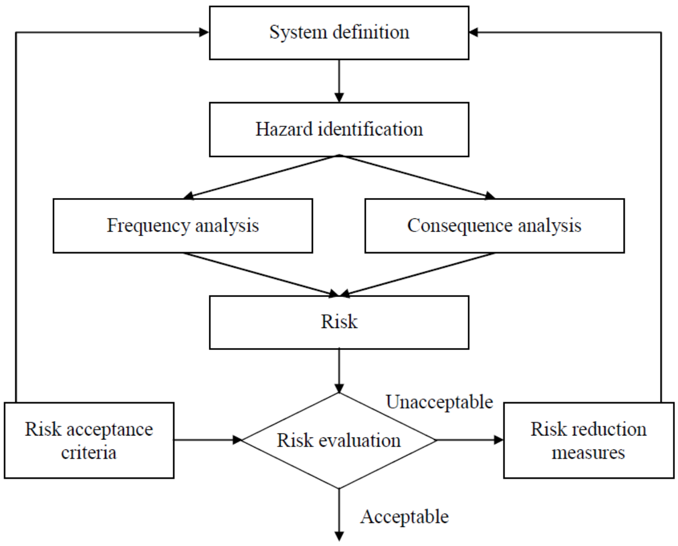


Figure 5.3: Risk assessment spreadsheet [14]

The probability of occurrence is an issues in the expected crowd densities for large bridges. Engineering companies expect that the proposed traffic classes are not likely to be present at a large bridge, as the number of people to reach the same density as for a short bridge increases. Application of this loading is therefore expected to result in conservative designs. In case of special events the probability of occurrence is taken into account in the question whether or not the design should checked for this load scenario.

5.2. Load definition

A warning notice at the suspension bridge at the Niagara falls (1860) mentions "A fine of $50 to $100 will be imposed for marching over this bridge in rank and file or to music, or by keeping regular step. Bodies of men or troops must be kept out of step when passing over this bridge. No musical band will be allowed to play while crossing except when seated in wagons or carriages." Taylor [25]

5.2.1. Difference between guidelines

Every country has its own National Annex in addition to the general valid Eurocode. Furthermore different guidelines and standards for the design of footbridges are applicable, presenting different values and models for the calculations of the human-induced vibrations. This results in confusion for the users of the guidelines, as it is not clear which calculation method or value should be used. One of the major differences is presented in figures 5.4 and 5.5: the reduction coefficient ψ which determines the critical range of natural frequencies pedestrians are able to excite, differs per guideline. For the Eurocode the values at the graph are even missing, as has been shown in figure 4.1.

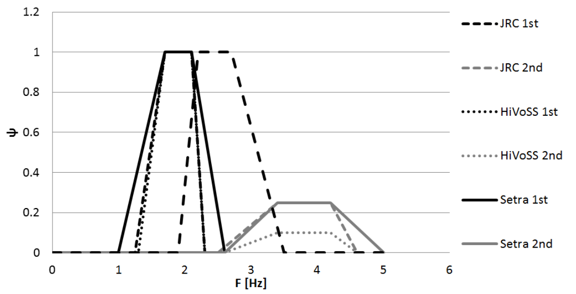


Figure 5.4: Reduction coefficient for the vertical and longitudinal direction according to several guidelines

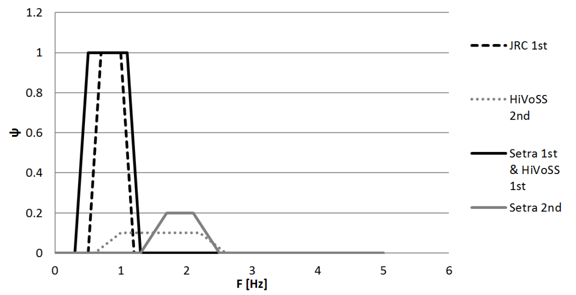


Figure 5.5: Reduction coefficient for the lateral direction according to several guidelines

The graph of the reduction coefficient according to the EUR 23984 has a gap between the first and the second harmonic. Strictly, the mode shape of the footbridge with a natural frequency in this range will according to the guideline not be susceptible for human-induced vibrations. There is however an uncertainty in the determination of the natural frequency in the design stage. In addition, the natural frequency of the footbridge can reduce due to the added mass of the pedestrians (in case this mass is a significant amount of the dead load of the footbridge). The natural frequency of a mode shape can therefore in reality shift towards the part that has to be checked for dynamic loading.

In discussion with RoyalHaskoningDHV the choice for the calculation method for the maximum acceleration turned out to be a bottleneck in the design of footbridges. The difference in the use of these methods was not clear. The other companies did not have problems with the use of the code and the guidelines, though it was mentioned that it would be favourable to improve it in general.

5.2.2. Vandal loading

Vandalism is defined as "a specific kind of footbridge loading characterized by the intentional and well coordinated action of one or several persons, moving their own body with the sole aim to increase the structural vibration level to a maximum value".[15] Vandal loading is though not precisely defined in terms of type of human activity: deliberate jumping, horizontal body swaying and bouncing could be considered.[10] The Eurocode advises to use traffic class 5 (TC5: a dense pedestrian flow of 1.5 P/m^2) for the load model for vandalism. It must be proven that the bridge cannot be put in vibration by vandalism by which it exceeds the ultimate limit state. In combination with TC5 a lower bound of the attenuation measurement for the structural damping equal to 50% of the nominal damping of the construction should be used.[5] The EUR 23984 guideline mentions a type of vandal loading 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". In reality, vandal loading is expected to be closer to people jumping or pulling at the cables, than a pedestrian flow intentionally putting the bridge in vibration. Instead of a reduction in the structural damping proposed in the Eurocode, an increase is expected according to the

EUR 23984. First it is mentioned that the structural safety of a low damped lightweight footbridge to might be affected by large amplitudes. With an increase in vibration amplitude on the other hand structures are expected to develop an increase in damping.[6] Taking both documents into account, it is therefore questioned whether the load model for vandal loading according to the Eurocode is realistic to use.

Different types of vandal loading on footbridges are discussed in literature. Research of Caetano and Cunha shows that the peak amplitude of the applied force depends on the frequency of excitation and is generally lower for bending knees excitation than for jumping. However, when considering a group excitation, it is observed that the degree of synchronization amongst pedestrians is higher for this situation than for jumping. Therefore it can happen that higher loads are applied to the bridge using this type of excitation.[26] A study by Schwartz et al. suggests that no real problems are expected by vandal loading. On the one hand, for small amplitudes of vibration (< 1.5 cm), a person is not capable of identifying the natural frequency of the structural loading. This makes it impossible for the person to properly synchronize. On the other hand, it is examined that a person is physical not capable to excite the footbridge fast enough for large frequencies (3.7 and 4.2 Hz). This result is a chaotic ineffective motion. In both cases, the force transmitted from the vandal to the structure is not optimally synchronized with the support motion. The amplitudes of vibrations remain therefore small. [15] Zivanovic recommends more caution in relation with vandalism: "The guidance on footbridge design with regard to running and vandal loading is sparse and require further research. This especially applies to possibility of synchronisation during running and jumping in small groups, to the more realistic mathematical description of the loading, and to expected duration of vandal excitation." The current design codes should therefore according to Zivanovic be used with care, and should always be accompanied with calculations according to more advanced design procedures.[27] Bachmann quantifies the properties of jumping people. He mentions that jumping occurs at a frequency range of 1.8 - 3.4 [Hz] with a first Fourier coefficient of 1.9 (2 [Hz]) or 1.8 (3 [Hz]).[22]

At the moment footbridges are not checked to vandal loading according to the engineering companies. In the future is seems to become more relevant in relation with increased slenderness and new materials. Problems with vandalism are expected to be more likely for short bridges and not an issue at all for large spans. It is questioned whether people are capable to maintain their action long enough with the same frequency and phase to reach the stress limit. Vandalism is not critical at the moment for FRP bridges according to the supplier, partly due to the high applied safety factor. When a lower safety factor is used for these bridges, it could however become a problem.

The influence of two load models is checked in the impact study. Both the load model according to the Eurocode and an assumed load model with people jumping at the normative position on the bridge are checked. The results are presented in subsection 6.4.3.

5.2.3. Special events

The term "special events" can cover all kind of events, for example a music event, a marathon or the inauguration of the bridge. In general it can be said that special events are events with a large amount of people moving more synchronous with each other than a normal crowd. Therefore it is more likely that larger accelerations will occur during these events. As the name implies special events occur in exceptional cases. According to the Eurocode “Determining the design situations that deal with occasional festival or choreographic registered events depends on the expected degree of surveillance by the responsible client or government. There is no test rule provided and it can be necessary to consider to perform special studies. Some information about an applicable design criteria can be found in the literature.”[2]. As shown in subsection 5.1.2 as well the Eurocode and EUR 23984 do mention the importance of taking into account the load cases for special events, they however do not provide guidance how to define this load.

It is expected that structural designers face a high uncertainty in the evaluation of vibrations in footbridges excited by people running with the increase in popular marathon events in urban environments. Racic presents a numerical generator of random near-periodic running force signals which can reliably simulate the experimental measurements. The expected force amplitude is divided around two times the weight of the person. Accorsing to Racic such a model is essential for future models of human-induced dynamic loading by individuals, groups and crowds running under a wide range of circumstances.[28] Silva et al. mention that there is evidence in literature that the human-induced loading is not well modelled by the application of force-only models in case of vertical vibrations in footbridges under crowd loading. Therefore a new crowd model for crowds with densities from 0.3-0.9 P/m^2 is proposed. To represent the total action of pedestrians biodynamic models are included. The results with the biodynamic models turned out to agree better with the measurements than models with forces only moving along the footbridge (see figure 5.6 and 5.7). An increase in the damping and a reduction on the dominant vibration frequency were observed when the biodynamic model is added as a component of the structural system.[29] This topic will be described in more detail in section 5.4.

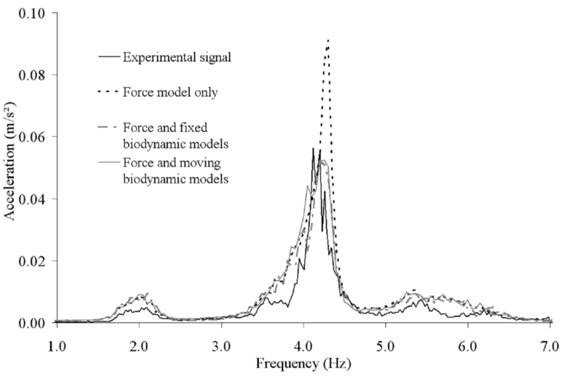


Figure 5.6: Mean spectra for density 0.3 pedestrians/m^2 [29]

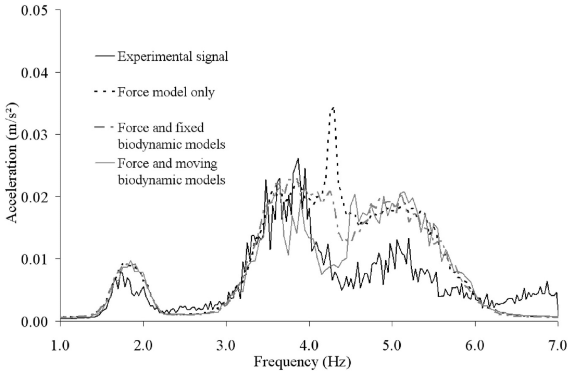


Figure 5.7: Mean spectra for density 0.9 pedestrians/m^2 [29]

Special events are not taken into account in the design at the moment. When the client indicates that an event like a marathon is expected, the engineering companies indicate this should be checked. Such an event is expected to have a large influence on the vibrations in the structure. It is not mentioned though how this check should be performed.

The characteristic of special events can described by a crowd that behaves more synchronous than a normal flow of pedestrians. Therefore in the impact study in section 6.4.2 two types of crowd loading with an increased number of equivalent pedestrians are evaluated, in order to see the impact on the expected accelerations.

5.2.4. Equivalent number pedestrians

To predict the response of a structure due to crowd loading, use can be made of a harmonic load model (see equation 4.3). The basic idea of this model is that the load model with an equivalent number of pedestrians results in the same structural response as simulations for a crowd of people. In both the Eurocode and the EUR 23984 a model is proposed with a value n', representing this equivalent number of people distributed along the structure at a fixed position (see figure 5.8). The people are modelled by a force changing in time and all the same mass, phase and frequency, equal to the natural frequency of the mode shape. The response (maximum acceleration of the bridge) from the Monte Carlo simulations and the harmonic load model should be the same, therefore the equivalent number of pedestrians can determined from this results depending on the different properties.

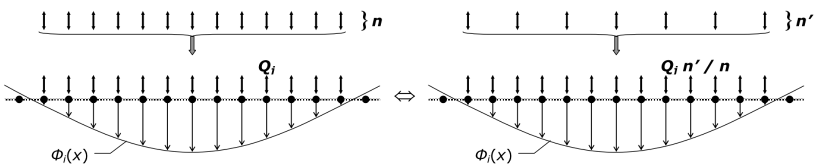
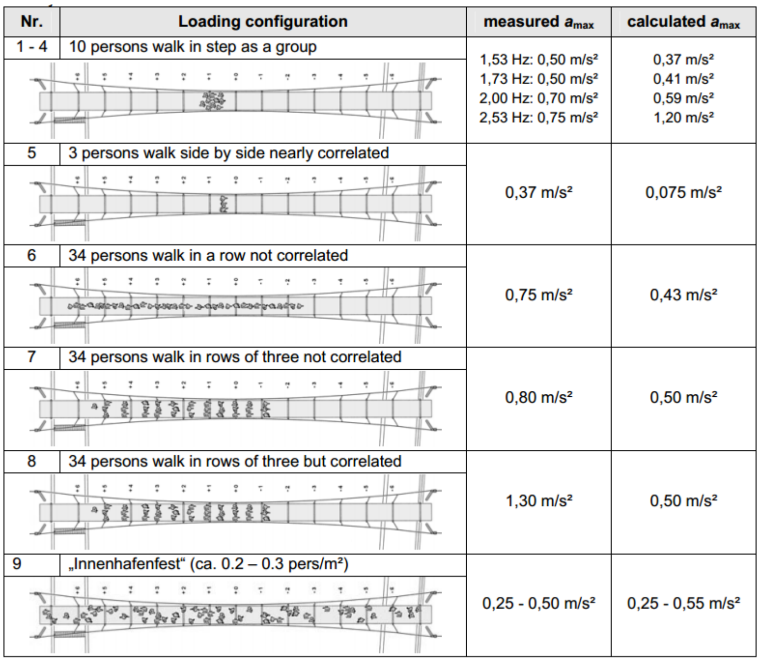


Figure 5.8: Equivalence of streams to determine n' [6]

In the simulations of the EUR 23984 a half-sine mode shape is used varying the damping ratio, frequency ratio, the number of pedestrians and the variation coefficient.[6] The pedestrians are modelled with all the same mass and fixed at a position. In the simulations of the Sétra the damping, number of pedestrians and mode shapes are varied and the pedestrians are modelled as a random mass with a velocity of 1.5 m/s.[12] For detailed description of the methods, reference is made to the guidelines themselves.

Both simulations result in the same formula for the equivalent number of pedestrians. The result of the expected maximum of the acceleration of the bridge is based on the 95% non-exceedance level of the equivalent number of pedestrians. Buur showed in his thesis that the determination of n' based on simulations seems correct.[9] The simulations that are used in the comparison are already approximations of the reality themselves. Instead of simulations it would be favourable to carry out experiments with real crowds on different bridges in order to get a more realistic model.

According to Venuti there is still al lack of reliable models for multiple pedestrians. "In the last fifteen years, excessive vibrations of footbridges and floors induced by pedestrians walking have become one of the leading research topic in structural dynamics. Despite considerable advances have been made in the experimental characterisation and numerical modelling of individual pedestrian loading there is still a lack of reliable models and adequate design guideline relevant to dynamic loading due to multiple pedestrians."[30] Butz shows that depending on the traffic configurations, a deviation in the maximum acceleration can be expected. The results of this simulations are presented in figure 5.9. These results question the current approach for the evaluation of human-induced vibrations in footbridges with a fixed density of the pedestrian flow.



Nr.	Loading configuration	measured a_{max}	calculated a_{max}
1 - 4	10 persons walk in step as a group	1,53 Hz: 0,50 m/s² 1,73 Hz: 0,50 m/s² 2,00 Hz: 0,70 m/s² 2,53 Hz: 0,75 m/s²	0,37 m/s² 0,41 m/s² 0,59 m/s² 1,20 m/s²
5	3 persons walk side by side nearly correlated	0,37 m/s²	0,075 m/s²
6	34 persons walk in a row not correlated	0,75 m/s²	0,43 m/s²
7	34 persons walk in rows of three not correlated	0,80 m/s²	0,50 m/s²
8	34 persons walk in rows of three but correlated	1,30 m/s²	0,50 m/s²
9	„Innenhafenfest“ (ca. 0.2 – 0.3 pers/m²)	0,25 - 0,50 m/s²	0,25 - 0,55 m/s²

Figure 5.9: Traffic configurations of a single pedestrian, a group pedestrians and a continuous pedestrian stream are numericaly simulated [31]

5.2.5. Jogger loading

The current load model prescribes a jogger loading of 5 (for L < 20 m) or 10 joggers (for L > 20 m) at a fixed governing position at the footbridge. The accelerations of the footbridge in the steady state have to be considered. In reality however it is expected that joggers move along the structure. Beers studied joggers in his master thesis, therefore reference is made to his work.[8] In this study, next to the prescribed method for the determination of the accelerations under jogger loading, two other methods are considered. First the response of the structure is determined for the timespan the jogger is present at the bridge, instead of for the steady state. Further the jogger loading is applied as a moving load, instead of a harmonic load at a fixed position. The results of applying these methods is presented in the impact study in chapter 6.

5.3. Structural response

"Damping represents energy dissipation in a vibrating structure. Each structure inherently possesses some capability to dissipate energy. That capability is very beneficial because it reduces structural response to a dynamic excitation near resonance. The near-resonant condition is the governing condition when considering footbridge vibration serviceability due to human-induced load. Therefore, it is very important to model damping as accurately as possible." Zivanovic [10]

5.3.1. Structural damping

Footbridges are in general relatively light structures, resulting in a change of damping after adding non structural elements like handles. This characteristic makes it difficult to generate a realistic approximation of the structural damping at the design stage. However, it is of importance to estimate the damping properly: "We have seen it: the resonance phenomenon causes displacements in constructions and stresses (very) much higher that would be obtained by a static calculation. In addition, the maximum amplification is directly linked to the damping. It is therefore essential for this parameter to be estimated properly in order to achieve correct dynamic sizing.".[12] The Eurocode does not mention how to determine the structural damping of footbridges. The EUR 23984 guideline provides a table with minimum and average damping ratios according to construction material for serviceability conditions. According to Heinemeyer for short span footbridges (e.g. < 20 m) the minimum values may be applied, in other cases the average value is appropriate.[23] The guideline is however very clear "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." [6] In addition structural damping is dependent on response amplitude: with increasing vibration amplitude the damping increases as well.[10] Therefore the natural frequencies of the footbridge are dependent on the amplitude.[32] In the EUR 23984 a different table for the damping ratios in case of vibrations with large amplitudes is presented. The size of this large amplitude is though not provided, therefore it is not known when this table may be applied.

The structural damping can not be determined exactly before erection of the structure. This can result in higher amplitudes of vibration than the vibrations expected in the design stage. A solution is the use of tuned mass dampers (TMDs) in the structure. TMDs do have to be maintained regularly to make sure they do their job when the structure is excited in the frequency the TMD is built for. So, in addition to the higher costs and the harm to the architectural design, TMDs have to be tuned for the right frequency that is expected to result in high amplitudes of vibrations. It is favourable to estimate the structural damping more precisely in the design stage, so that the application of TMDs is not needed.

The research presented in figures 5.10 and 5.11 shows the difference in damping ratio for several footbridges in different materials. Thought there is a difference visible between the materials, within one material there is a significant deviation in the damping ratio. However, no clear relation can be drawn from these graphs.

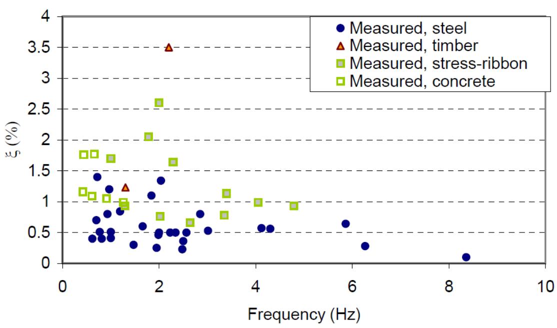


Figure 5.10: Measured damping ratios under service loads: variation with natural frequency [6]

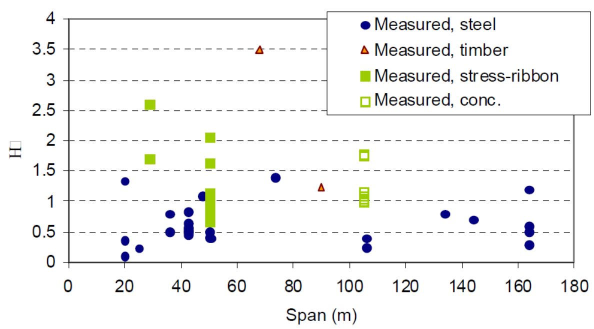


Figure 5.11: Measured damping ratios under service loads: variation with span [6]

Kasperski studied the difference in damping ratio between different amplitudes of vibrations. The critical damping for the studied structure is 0.5% for amplitudes up to 0.25 m/s^2. For larger amplitudes of vibrations of 0.78 m/s^2 an in crease of 8% in the damping ratio is measured.[33] An experiment by Brownjohn et al.

underlined the increase in damping for higher vibrations magnitudes. The result of a free decay test showed that the structural damping of the footbridge increased from 0.22% to 0.26% for higher vibration magnitudes, such as those expected to be generated by human-walking.[34]

Current designs account for the application of TMDs after erection to fulfil the comfort criteria to vibrations. The determination of the structural damping in the design stage is a difficult task according to the engineering companies. It would be beneficial if to be able to determine the structural damping more precise in advance, making the use of TMDs unnecessary. For steel, the determination of the damping is less of a problem. In practice the TMDs are not applied, because no complains about the vibrations in the use of the bridges were mentioned according to the GemeentewerkenRotterdam. The expectation of the structural damping of FRP bridges is now based on comparison with other materials. The supplier checks the natural frequencies of the footbridge, but not the damping. It is expected that due to the fact that the design of a FRP bridge is more homogeneous, the estimation of the structural damping for FRP footbridges can be more precise than for other bridges.

Uncertainties in the determination of the structural damping in the design stage can result in a deviation in the response of the structure to dynamic loading. This deviation due to differences in the structural damping is evaluated in the impact study in section 6.4.4.

5.3.2. Determination maximum acceleration

As shown in subsection 4.2.6 the EUR 23984 provides three methods to determine the maximum acceleration: SDOF, FE and RS method. The SDOF and RS method do not use a FEM program. By using these methods, the 95th percentile of the maximum acceleration of the bridge is determined. The response spectra method has the aim, different than the other two methods, to describe the stochastic loading and system response in a simple way. Therefore design values with a specific confidence level are provided.

It is questioned whether all three methods can be used to estimate the maximum accelerations of the bridge and which method provides the most realistic results. Reference on this topic is made to the master thesis of Buur [9], where he compares the results according to the SDOF method and the response spectra method. In addition to the differences between the calculation methods of the maximum acceleration of the bridge, it is expected that there is a difference between the maximum acceleration of the bridge in the steady state under a certain loading and the maximum accelerations a pedestrian experiences crossing the bridge.

In discussion with one engineering company it turned out that the difference between the calculation methods for the maximum acceleration according to the EUR 23984 was not clear. It was questioned which method should be use to give the best approximation.

5.3.3. Stresses and deflection

In the static design of a bridge the maximum stresses may not exceed the stress limit (the ultimate limit state). In addition the deflection can be a requirement of the client, but no restriction for deflection is given in the code. For the dynamic design there is only a requirement for comfort expressed in maximum acceleration of the bridge. In the case of vandalism or special events maximum stresses or deflection could become an issue in the structural safety too.

The question arises if vandals are able to cause high stresses in the structure. For short (light) bridges this seems to be likely, for large structure it is expected that the stress limit will not be reached (see subsection 5.2.2). In case of short bridges it might be necessary to perform an extra check to stresses. In case of deflections no serious problems are expected, it is questioned though if a high deflection of the bridge can cause panic. Deflection of the bridge deck could in that sense be a measurement of comfort.

When an ill intentioned person is bouncing on the footbridge at an appropriate frequency, intolerable accelerations (around 6 m/s^2) are easily reached for footbridges with cables according to Canor.[35] The more people, the more difficult it becomes to synchronize though: even if reaching the ultimate limit state is theoretically possible, it is not likely to occur in reality.[10] This is partly due to the fact that the force peak amplitude per person decreases with an increasing number of people.[36] A study of Caetano for a stress-ribbon footbridge has shown that stresses associated with vandal loads remain inferior to those associated with static loads. For very large accelerations, with peak value of 8.6 m/s^2, the measured dynamic displacements of 53 mm remain much smaller than the static displacements evaluated for the crowd load of 4 kN/m^2

(0.286 m).[26]

In the impact study both the deflection and stresses under dynamic loading are checked. The stresses for different types of loading are presented in chapter 6 in the different sections.

5.3.4. New materials

In the past years new (combinations of) materials are introduced for engineering purposes. In the design of footbridges fibre reinforced polymer (FRP) is used more frequently. The design codes and guidelines however do not provide material-specific information and design parameters for these new materials. The safety factors used in bridge engineering are unrealistic high at the moment, to ensure safe designs. In addition the expectations of the strength of the structure in for example 50 years is unclear. Composite footbridges can be constructed to obtain an unprecedented slenderness by using new materials and technologies.[37]

Fibre reinforced polymer is expected to be used increasingly in the design of footbridges. It is favourable to be able to built safe and comfortable footbridges without unnecessary high safety factors. Due to the fact that the knowledge about structures built in these materials is limited, the risks are uncertain as well.

FRP profiles are highly potential for structural application due to the corrosion resistance, high strength and low self-weight. On the other hand the general use of these materials is obstructed by the high formability, the sensitivity to instability and the shortcoming in specific design codes.[38] Gonilha et al. performed tests on a 6 m long footbridge and checks with a FE program to be able to predict the dynamic response of a footbridge by developing numerical tools.[39] Kendall presented four major advantages of FRP bridges: reduced mass, superior durability, ability to mould complex forms and desirable electrical and thermal properties. It is stated that FRP composites have the potential to revolutionise the construction of long span bridges, resulting in more efficient and cost-effective structures.[40] Guide specifications for the design of FRP footbridges are published in 2008 by the American Association of State Highway and Transportation Officials [41], but not in the Eurocode or EUR 23984. A study by Sadeghi has shown that the investigated composite structure offers human safety and comfort against vibration due to running persons.[37]

RoyalHaskoningDHV uses FRP already in the design of footbridges. It is expected to be the main construction material for footbridges in the future. Longer spans and architectural challenging structures in FRP will be the new challenge. It is favourable to experience more with the material to fully understand the properties.

Short footbridges designed with lightweight new materials are expected to become more susceptible to vandal loading in the future. In the impact study a check for vandal loading on a short footbridge with a low dead load is performed in section 6.4.3.

5.4. Interaction effects

"A walking person adapts to and synchronizes his/her motion in frequency and phase with vibrating deck. This phenomena depends from the human personal features and vibration characteristics of the deck." Bachmann[22]

5.4.1. External interaction effects

Pedestrians have the tendency to change their walking behaviour to the other pedestrians within a crowd, which can be referred to as human-human interaction. A group or crowd of pedestrians walking more synchronously than normal is expected to cause larger accelerations of the structure. External effects like music can provoke this behaviour. The influence of these external effects is not taken into account in the current design regulations for footbridges, though as special events it is mentioned (see subsection 5.2.3).

In case a high level of synchronization is reached the enhancement factor increases according to a study by Caprani et al. They argue that the enhancement factor can be expressed as a function of crowd density and proportion of synchronization. For high crowd densities and synchronization proportions the enhancement factor presented can become unrealistic large, but it is expected that this will not occur in practice. It is thought that in case of large vibrations pedestrians will tend to stop walking and therefore damp the vibrations.[36] Another study, based on full-scale measurements on a bridge both before and after an American football game, unveiled that the susceptibility of the bridge to pedestrian excitation was attributable to synchronization of pedestrians footfalls with the bridge vibration. It was shown that the development of the bridge frequency is an indication of the level of synchronization of pedestrians.[42] According to Racic not only the perceptible motion of the structure can be a incentive to affect the human-induced loads, also different auditory, visual and tactile stimuli can be of influence. "To include all these parameters in the model, the variations in the force amplitudes, energy, timing and shape for successive running cycles should be modelled as a function of bridge dynamic response, human perception to vertical vibrations and different external stimuli."[28]

The result of the loading model for external interaction effects is expected to be close to the load model of special events: including more synchronization between the pedestrians. The effect of a higher level of synchronization is therefore studied in the impact study at the part of special events (subsection 6.4.2). The amount of interaction at for example a marathon is however hard to determine.

5.4.2. Lock-in

In lateral direction a phenomenon called lock-in is observed in footbridges (like in the London Millennium Bridge and Solférino Passerelle in Paris). The lock-in effect is the probability that a pedestrian will synchronize his footfall rate to the frequency of the swaying platform as a function of the frequency and amplitude of the sway.[43] A schematic description is provided by Heinemeyer in figure 5.12. Lock-in can result in high accelerations of the structure, therefore checks for lateral lock-in are provided in the EUR 23984. No observations in reality have been reported of pedestrian streams synchronizing with vertical vibrations on footbridges. The EUR 23984 guideline states though that by adjusting their gait people are able to react on vibrations. Experimental investigations have shown that single pedestrians can synchronize with vertical harmonic vibrations of 1.5 m/s^2, although this is not generally considered.[6]

The lock-in effect is an interesting topic in research for several years. Zivanovic states that since humans are quite sensitive vibration receivers, this leads to pedestrian interaction with the structure. There is a general agreement that a crowd of people could act as negative dampers when walking across a laterally vibrating structure. The interaction of the same crowd with the vertically vibrating structure is though far less understood.[44] An early study performed tests with a simply supported beam for a range of activities. These results showed that the human and structure interact when the person is stationary on the beam, but the person is simply acting like a load when he is moving.[45] The analysis of visual material (film) by Zoltowski shows that pedestrian adapt their step to vibrations in order to reduce forces in legs. The consequence is a reduction of load action on the platform. He states the following hypothesis: the lock-in effect in vertical direction for walking pedestrians on the vibrating deck has an origin in natural adapting mechanism by reducing the forces in the human body.[46] A study by Qin with finite element analysis of dynamic interaction between the footbridge structure and pedestrians presents some significant results. The dynamic interaction is expected to increase at for larger amplitudes of vibrations and for an increased mass ratio. In addition the results suggest that the leg stiffness of a pedestrian has a considerable effect on the dynamic response of a slender structure.[47] This last topic will be discussed in more detail in the next subsection. Studies by Bach-

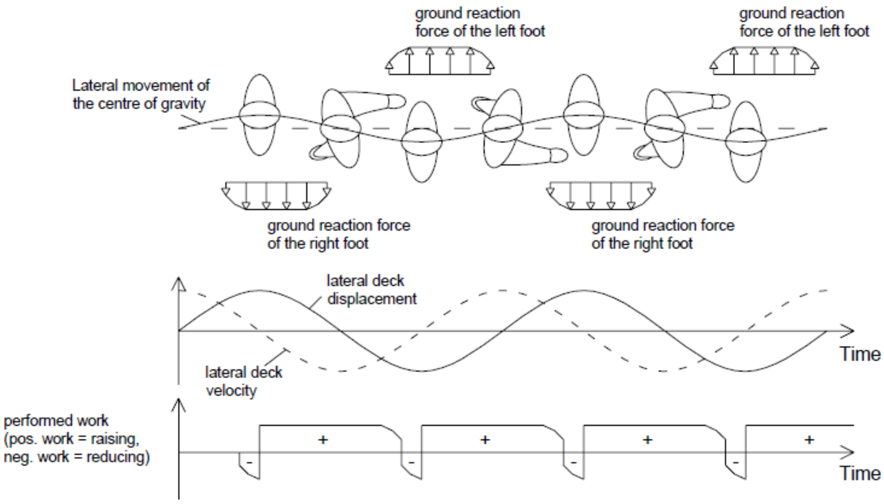


Figure 5.12: Schematic description of synchronous walking in lateral direction [23]

mann suggest that two criteria for lock-in may be applied: $d_{max,vertical} \leq 10$ mm and $a_{max,vertical} \leq 0.49$ to 0.98 m/s^2.[48] According to the Setra guideline the concept of a critical accelerations and critical number of pedestrians may be linked in the determination of a limit for the lock-in effect. The use of a critical acceleration is expected to be more relevant, because it corresponds to the perception of a pedestrian, while the number of pedestrians is depending on the position of the pedestrian on the footbridge.[12]

The lock-in effect is an interesting topic for the engineering company ARUP, as structural engineer of the London Millennium Bridge. Where in lateral direction pedestrian are expected to only add energy to the system, in vertical direction both energy and damping could be added. Aspects like the slope of the bridge could have an influence on the lock-in effect as well. ARUP expects an upper limit for the added energy to a system for the vertical direction, but vertical synchronisation is thought to be not possible to occur.

Equal to the expectations for external interaction effect, the result of lock-in will be close to the load model of special events. Therefore reference is made to the part of the impact study about special events as well (subsection 6.4.2) for the effect of a lock-in.

5.4.3. Additional mass and damping by pedestrian loading

The current codes and guidelines provide design models to describe the dynamic behaviour of footbridges with a structure that is put into vibration by a harmonic force. In these models the structure and force are separate systems, where in reality they interact with each other. Recent studies do take this interaction into account, resulting in added mass, damping or energy by pedestrians. The additional mass by pedestrians is taken into account in the EUR 23984 for the evaluation of the natural frequencies, but not in the determination of the maximum acceleration. The influence of the other aspects on the response of the structure is uncertain and not implemented in the regulations at the moment.

Experiments by Brownjohn on human-structure interaction confirm that the human body acts dynamically with the structure. This results in a modification of the natural frequencies of the structure and a considerably increase of the damping capacity. [49] When including the biodynamic models as part of the structural system as discussed in subsection 5.2.3, this increase in damping is noticed as well.[29] Research of Sachse et al. showed the influence of presence of humans on the structure in the two graphs in figures 5.13 and 5.14. Both a decrease in the natural frequency of the total structure as well as in the response of the structure can be seen due to the presence of human occupants or additional mass. Caprani shows that there are two different approaches about damping in literature. The first assumes that sitting or standing people affect the damping of a structure, but walking people do not and should be represented as load only. The second approach assumes that both walking pedestrians and stationary pedestrians can increase the damping ratio of a bridge in the vertical direction.[36] Research by Zivanovic shows that the structural damping was much larger occupied by either an passive of active crowd than for the empty structure. The human presence on the structure is thus suggested to increases the damping and to mitigate the vibration response of the structure. Based on these

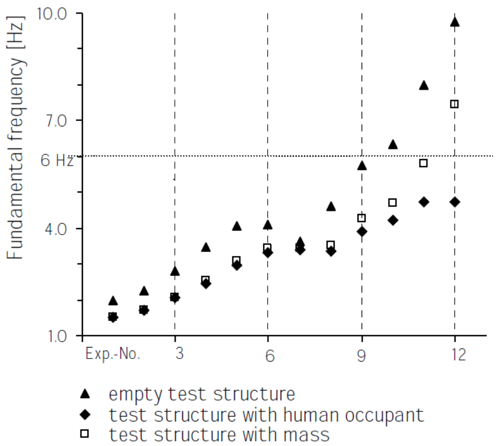


Figure 5.13: The influence of a standing human occupant and an equivalent mass on fundamental natural frequencies [50]

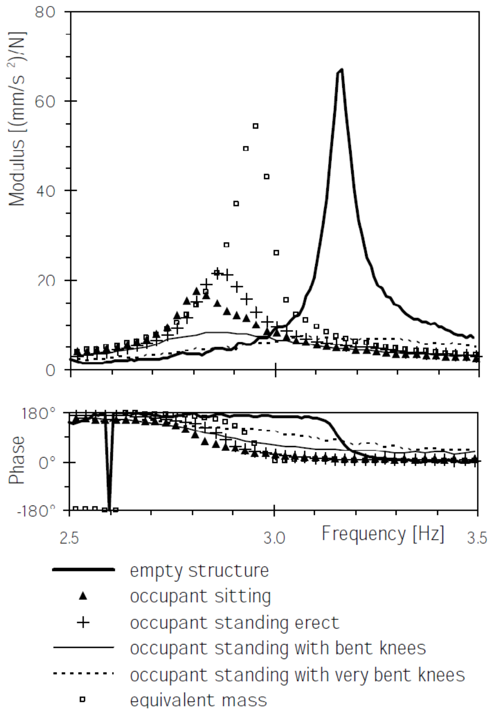


Figure 5.14: FRFs of an empty and occupied concrete plank [50]

results Zivanobic suggests that it is cost effective and useful to take the increased damping into account in the design of structures under human-induced loading.[44] An experiment by Wang et al with a single beam showed that it is possible to influence the dynamic characteristic of a footbridge by a large flow of pedestrians. The parameters of the structure can be influenced by both passive and active persons. The change in parameters is though much stronger due to the passive person than to the active person.[51] Agu mentions that standing persons can though lead to misinterpreting observed structural vibration amplitudes, because the large damping capacity of passive persons can masks the dynamic problems. For jumping audience the effective damping reduces to the low structural damping value. In this case the resonance response of the structure becomes large.[52] A simple model based on the understanding of a full-scale study as well as literature is presented by Zuo et al. It suggests that significant pedestrian-induced footbridge vibration is a type of self-limited oscillation. The largest vibration amplitude that might be critical for serviceability of footbridges could therefore be estimated.[42] As result of experiments Georgakis et al. found that the average added mass of a single pedestrian is 102% of the pedestrian's mass. In addition the results suggest that when pedestrians cross the footbridge they always add damping to the structure for the vertical direction. This added damping is found to decrease though with increasing amplitudes of vibration. A conservative fixed value of 500 kg/s is suggested to use a as an individual damping coefficient for vibrations up to an amplitude of 5 mm.[32] Based on experiments on a footbridge with a natural frequency of 1.8 Hz Kasperski shows that the induced damping of a walking person is larger than for a passive person. The amount of induced damping depends on the walking frequency. The additionally induced damping by the walking person increases from 5% for a walking frequency of 1.6 Hz to 13% for 1.9 Hz. Then, the additional damping decreases again to a value of 6% for a frequency of 2.1 Hz.[33]

The engineering companies do take into account the additional mass of the pedestrians in the design. The effect that these additional mass can cause a reduction in the natural frequency is used in evaluation for human-induced vibrations in the design of footbridges.

Based on the studies presented it is not clear if active persons increase or decrease the added damping by pedestrians compared to the passive persons. The values for the additional mass and damping presented by Georgakis et al. are used in the impact study in 6.4.5 to see the influence of taken this type of interaction into account in the evaluation of human-induced vibrations.

6

IMPACT STUDY CRITICAL ASPECTS

Introduction

In order to verify the critical aspects discussed in chapter 5 and to check their relevance, rough calculations are performed. Three simply supported footbridges and two structures modelled as a cantilever beam are investigated, with respectively the first and second natural frequency in the critical range of the walking frequency of pedestrians. The calculations are performed by use of the SDOF method according to the EUR 23984. In this chapter first the properties of the model are presented, followed by the definition of the loads and finishing with the results and considerations of the calculations presented in graphs. Additional explanation about the calculations of the impact study can be found in appendix C.

6.1. Input properties beam model

An Euler-Bernoulli beam model is used for the calculations. Two types of structures are taken into account to represent footbridges: a simply supported beam and a cantilever beam. A footbridge has to be checked to human-induced vibrations when the natural frequencies are in the range of the walking frequencies of pedestrians. Therefore first a plot is presented with the likelihood that a footbridge with a certain length and dead load (designed according to the static loading) has the first natural frequency within this range. In figure 6.1 the first natural frequencies for simply supported footbridges with different dead loads are plotted against the length of the bridge. The red bands indicate the critical range of the walking frequencies for pedestrians, in combination with the graph for the reduction factor ψ. Figure 6.2 shows the same graph for cantilever footbridges. The red bands correspond to the critical range of natural frequencies for pedestrians. The blue bands correspond to the critical range of natural frequencies for joggers (both according to the EUR 23984). Looking at the figures it is shown that it is not likely for a short bridge to be in the critical range of walking frequencies of pedestrians. Experiments by Pernica though have revealed that people jump in a range of 1.4 - 4.0 [Hz], which makes it more likely that a small bridge can be excited by people as well.[53]

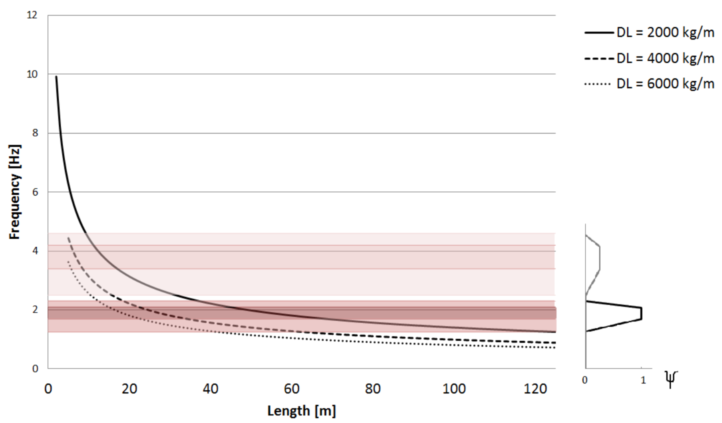


Figure 6.1: Relation between natural frequency and length of simply supported footbridges with different dead loads

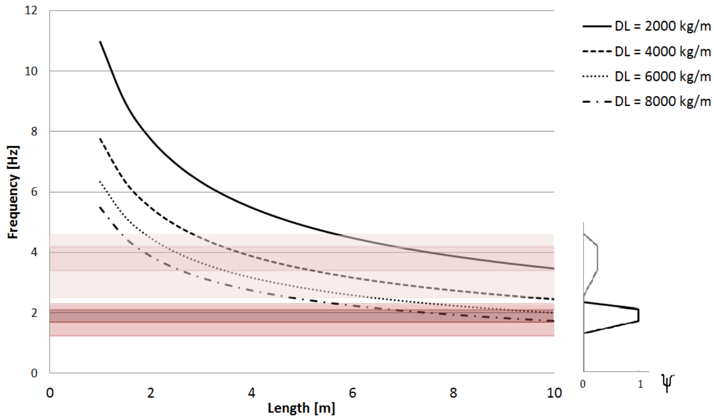


Figure 6.2: Relation between natural frequency and length of a cantilever footbridge with different dead load with critical frequency range for pedestrians

6.1.1. Simply supported beam

The models of the simply supported beam are based on the properties of a worked example for a simply supported beam used in the EUR 23984.[6] The properties for a simply supported beam with a length of 50 m are provided in this guideline. Using this as a starting point, the properties for the simply supported beam models with a length of 12 and 100 m are determined such that the first natural frequency will be 1.8 Hz. This frequency is within the critical range of the walking frequency of pedestrians. In case of the large bridge with a length of 100 m the width is taken 4 m instead of 3 m in order to get a more realistic design. This results in a higher number of pedestrians on the bridge in case of crowd loading, due to the fact that the traffic classes are expressed in persons per square meter. The first three mode shapes for simply supported bridges are presented in figure 6.3. The mode shapes can be described with following formula 6.1.[54] The natural frequencies can be determined with equation 6.2.

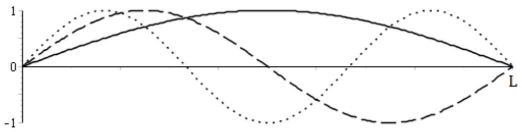


Figure 6.3: First three mode shapes for a simply supported beams

$$\phi_n(x) = \sin\left(\frac{n\pi x}{L}\right) \tag{6.1}$$

$$f_n = \frac{1}{2\pi}\sqrt{\frac{k_n^*}{m_n^*}} = \frac{C_n}{2\pi}\sqrt{\frac{EI}{\rho AL^4}} \tag{6.2}$$

Where, for the first three natural frequencies $C_1 = 9.87$, $C_2 = 39.5$ and $C_3 = 88.9$. The properties of the three simply supported bridges that are taken into account in this impact study are presented in tables 6.1, 6.2 and 6.3. The properties are constant over the length.

6.1.2. Cantilever beam

The part of the total bridge where the pedestrians walk can be represented by a cantilever beam like in figure 6.5. In addition to the simply supported beam a case with a footbridge suspended from an existing bridge is checked. The assumption is made that in this case for the check with static loading no exceptional vehicle has to be carried, because this vehicle will be able to cross the bridge at the main part of the bridge instead of the footbridge part. A service vehicle still has to be carried. Two models with both a different length and dead load are set up.

Two cantilever footbridges are taken into account. The design is based on the requirement for maximum

Table 6.1: Properties simply supported beam with length 12 m

Property	Symbol	Value	Unit
Length	L	12	m
Width	B	3	m
Modulus of elasticity	E	$2.1 \cdot 10^{11}$	N/m^2
Moment of inertia	I	$3.25 \cdot 10^{-4}$	m^4
Self weight	ρA	2500	kg/m
Damping ratio	ξ	0.015	-
Natural frequency 1st harmonic	f_1	1.8	Hz

Table 6.2: Properties simply supported beam with length 50 m

Property	Symbol	Value	Unit
Length	L	50	m
Width	B	3	m
Stiffness	EI	$2.05 \cdot 10^{10}$	Nm^2
Self weight	ρA	2500	kg/m
Damping ratio	ξ	0.015	-
Natural frequency 1st harmonic	f_1	1.8	Hz

Table 6.3: Properties simply supported beam with length 100 m

Property	Symbol	Value	Unit
Length	L	100	m
Width	B	4	m
Modulus of elasticity	E	$2.1 \cdot 10^{11}$	N/m^2
Moment of inertia	I	2.0	m^4
Self weight	ρA	3333	kg/m
Damping ratio	ξ	0.015	-
Natural frequency 1st harmonic	f_1	1.8	Hz

stress (< 300 N/mm^2) and in addition on a maximum deflection (< 2*L/250). This deflection is not a requirement from the Eurocode, but an assumption made to get a realistic design. Footbridges do not have to be designed to deflection. The natural frequency of the two cantilever footbridges is expected to be in the range of the second harmonic following from figure 6.2. This results in a reduction coefficient ψ of 0.25 for the normal crowd load models, according to figure 4.7. The mode shapes of the cantilever beam can be represented by formula 6.3. Figure 6.4 shows the first three mode shapes. The natural frequencies for these mode shapes can be determined with equation 6.4.[54]

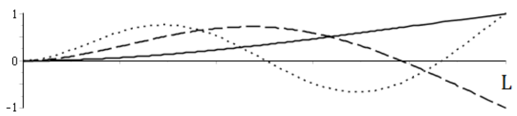


Figure 6.4: First three mode shapes for a cantilever beam

$$\phi_n(x) = \cosh\left(\frac{\lambda_n x}{L}\right) - \cos\left(\frac{\lambda_n x}{L}\right) - \sigma_n\left(\sinh\left(\frac{\lambda_n x}{L}\right) - \sin\left(\frac{\lambda_n x}{L}\right)\right) \tag{6.3}$$

$$f_n = \frac{1}{2\pi}\sqrt{\frac{k_n^*}{m_n^*}} = \frac{\lambda_n^2}{2\pi}\sqrt{\frac{EI}{\rho A L^4}} \tag{6.4}$$

In which:

λ_1	=	1.87510407	σ_1	=	0.734095514
λ_2	=	4.69409113	σ_2	=	1.018467319
λ_3	=	7.85475744	σ_3	=	0.999224497

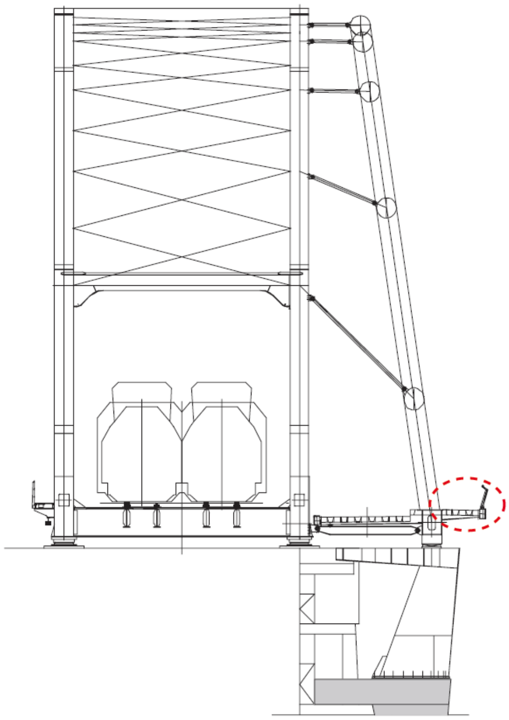


Figure 6.5: Example of a cantilever footbridge at the "Snelbinder" in Nijmegen in the red dotted circle [55]

The properties for the two cantilever beams that are taken into account are presented in tables 6.4 and 6.5.

Table 6.4: Properties cantilever with length 2.5 m

Property	Symbol	Value	Unit
Length	L	2.5	m
Width	B	4	m
Modulus of elasticity	E	$2.1 \cdot 10^{11}$	N/m^2
Moment of inertia	I	$2.33 \cdot 10^{-5}$	m^4
Section modulus	W	$6.25 \cdot 10^{-4}$	m^3
Dead load	DL	6000	N/m^2
Self weight	ρA	2446	kg/m
Damping	ξ	0.015	-
Natural frequency 1st harmonic	f_1	4.00	Hz

Table 6.5: Properties cantilever with length 4 m

Property	Symbol	Value	Unit
Length	L	4	m
Width	B	4	m
Modulus of elasticity	E	$2.1 \cdot 10^{11}$	N/m^2
Moment of inertia	I	$9.52 \cdot 10^{-5}$	m^4
Section modulus	W	$1.33 \cdot 10^{-3}$	m^3
Dead load	DL	4000	N/m^2
Self weight	ρA	1631	kg/m
Damping	ξ	0.015	-
Natural frequency 1st harmonic	f_1	3.87	Hz

6.2. Applied loading

Two types of load are used in the calculations: a crowd of pedestrians walking and single people jumping. The crowd is represented by a uniformly distributed load changing in time, determined with the equivalent number of people like the harmonic load model presented in subsection 4.2.6. The single jumping people are represented by a harmonic point load at a fixed position on the bridge deck, like the model for the jogger presented in subsection 4.1.6. Figure 6.6 shows the representation of the load on the two types of structure. In case of the jumping people, this load should be placed at the most unfavourable position at the bridge for the particular mode shape taken into account.

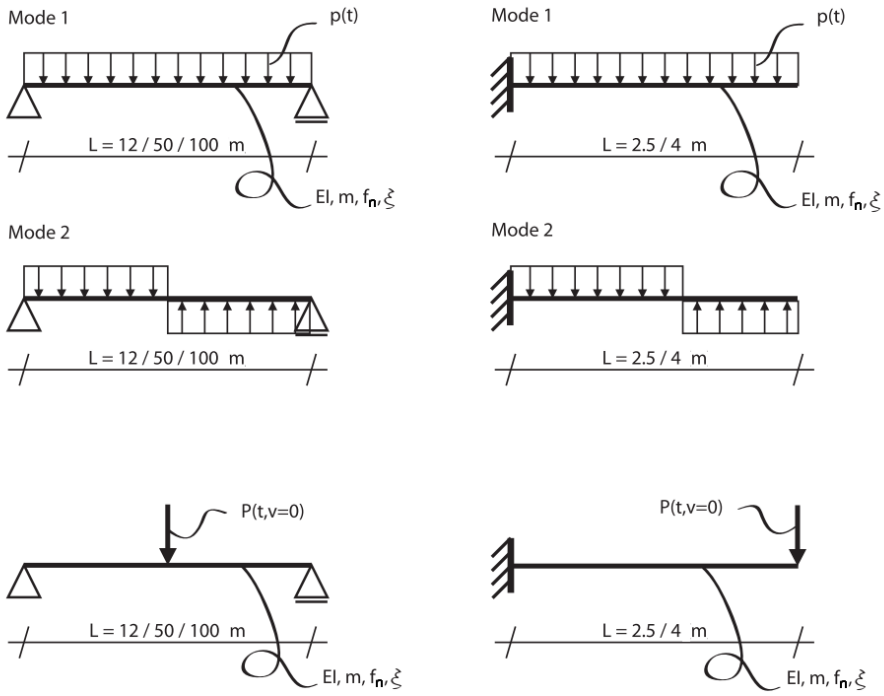


Figure 6.6: Load models for the first and second mode of the simply supported beam and for the cantilever beam

Vandal loading

As shown in the sections above vandal loading is represented by the Eurocode as a crowd crossing the bridge in combination with decreased structural damping of the bridge. A more realistic case would be people jumping and trying to excite the bridge. This load model is represented by a harmonic point load (according to the jogger-model) at the most unfavourable position at the bridge, as shown at the bottom of figure 6.6. Both the load model according to the Eurocode and the load model with jumping people are evaluated in this impact study and the results of the two approaches are compared.

Special event

Special events are represented by a crowd of people. The assumption is made that the pedestrians interact with each other, resulting in a higher equivalent number of pedestrians (n'). The choice is made to represent the increased number of equivalent pedestrians in two ways: in the first case the equivalent number of pedestrians is taken twice as big as for normal crowd loading, for the second case the assumption is made that 50% of the people present at the bridge moves synchronous, resulting in $n' = 50\%n$.

Joggers

Joggers are in general expected to have a higher walking frequency than normal walking pedestrians. According to the Dutch National Annex their walking frequency differs from 1.9 to 3.5 Hz, with the main frequencies between 2.2 and 2.7 Hz (see figure 4.2). Therefore the bridges that are taken into account will not be susceptible to jogger-induced loading, because the first natural frequency is 1.8 Hz. To see the difference in the impact on the dynamic behaviour of the footbridge between the normal pedestrian loading and the jogger loading, the jogger loading is assumed with a walking frequency of 1.8 Hz as well. It must be kept in mind that in practice, one mode shape of a bridge is not expected to be susceptible for both normal pedestrian traffic and joggers. The three types of considered jogger loading are discussed in 5.2.5. The application of the loading is explained in section C.4 of appendix C.

Interaction
Several studies have been performed about the human-structure interaction in footbridges, but the currently used models are still based on two separate systems without interaction: the pedestrians (loading) and the structure (response). It is interesting to see what the impact of the expected interaction will be. To model interaction, the proposed values for additional mass and damping according to Georgakis et al [32] (as explained in subsection 5.4.3) are applied. The cases with and without interaction are compared with each other to study the impact of this assumed interaction. The study is performed for the simply supported beam (for three different lengths), comparing the results for traffic classes 3 and 5. It must be mentioned that in this impact study only one proposed assumed interaction is considered. Other assumptions of interaction can be have a different impact.

6.3. Assumptions model

The design of the beam models used for the calculations is based on a requirement for the stresses and an assumption for maximum deflection. The deflection is not a requirement in the design according to the code, but it is though used as a check to create a realistic design.

Stresses
The maximum allowed stress used in the design is assumed to be 300 N/mm^2. Both the dead load and the live load are taken into account using the safety factors according to the Eurocode and National Annex to determine the design values of the pedestrian loading. The stresses that are expected under the dynamic pedestrian loading are determined in this impact study as well. In section C.2 in the appendix is the procedure for the calculation of both the static and the dynamic stresses explained.

Deflection
The check for the deflection of the structure is, as said before, not a requirement in the Eurocode. The deflection is though determined to check if the design for the structure used in the model is a realistic assumption. The requirement of the deflection is only based on the variable loading, whereas the influence of the dead load is assumed to be taken into account in the design and erection of the footbridge. In case of the simply supported beam the requirement for the deflection used is the deflection should be smaller than $\frac{L}{250}$. This value is the result of conversations with engineering companies and supervisors of this graduation project. For the case of a cantilever beam the deflection should, in line with the deflection for the simply supported bridge, be smaller than $2 \cdot \frac{L}{250}$. In section C.3 in the appendix the procedure to calculated both the static and the dynamic deflection is explained.

6.4. Results of the impact study

A first general remark has to be made. Traffic class one (TC1) is defined in the EUR 23984 as a fixed number of people, whereas the other four traffic classes (TC2-5) are set in a density expressed in number of people per square meter. In case of short bridges, this will result in a higher number of pedestrians at the bridge for TC1 than for TC2. Due to this difference in the definition of the traffic classes, the results for the stresses and accelerations can have a high peak value at TC1.

6.4.1. Design scenarios

In order to get an impression of the differences between the design scenarios, the maximum accelerations and stresses for the dynamic analysis are determined for the different traffic classes. Both the simply supported beam (figures 6.7 to 6.12) and the cantilever beam (figures 6.13 and 6.14) are checked for the mentioned lengths. At the left the accelerations are presented in m/s^2, at the right the stresses in N/mm^2. On the bottom of the graphs the number of pedestrians n and the equivalent number of pedestrians n' are presented as well.

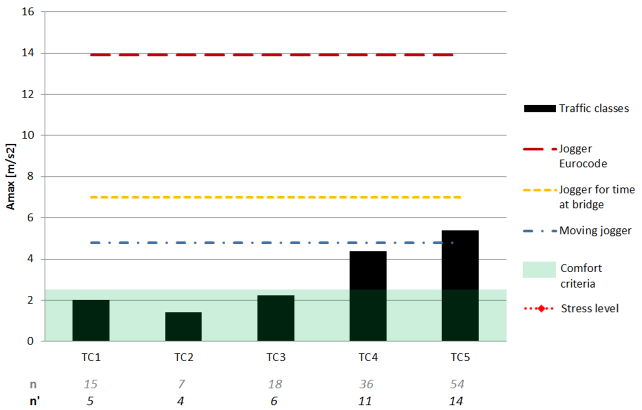


Figure 6.7: Accelerations for beam L = 12 m

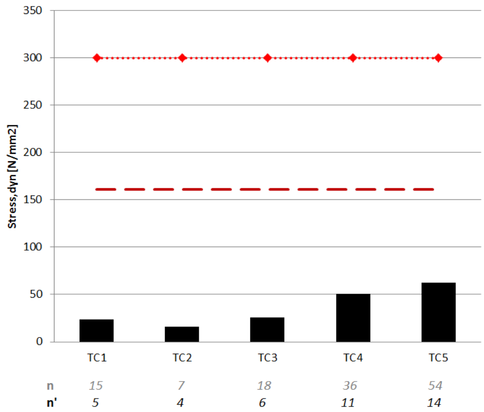


Figure 6.8: Stresses for beam L = 12 m

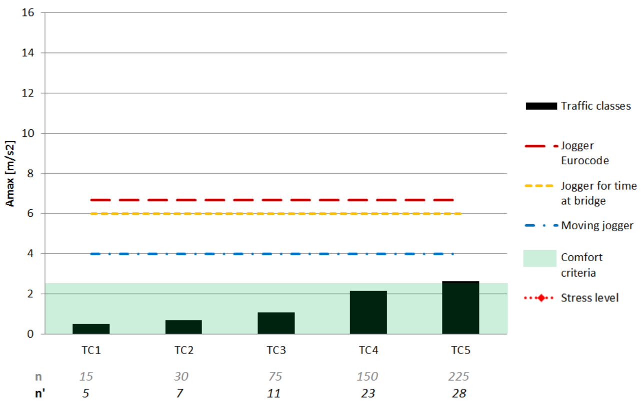


Figure 6.9: Accelerations for beam L = 50 m

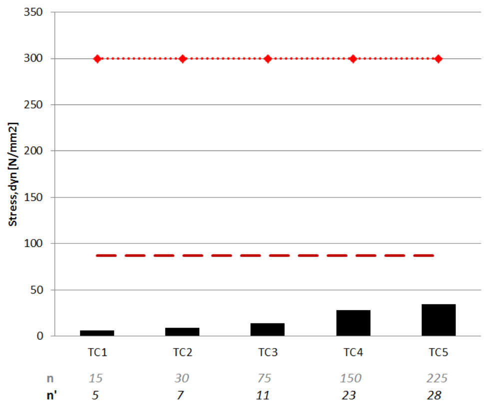


Figure 6.10: Stresses for beam L = 50 m

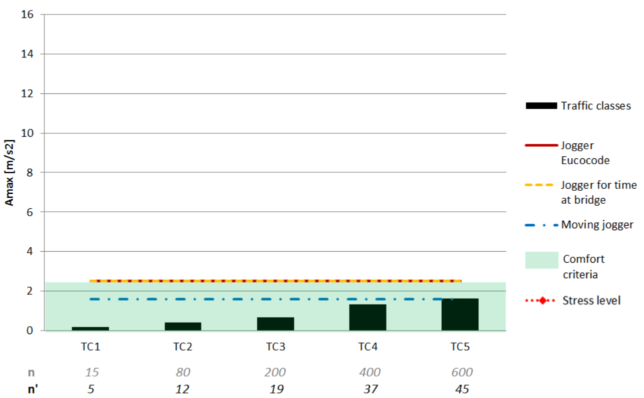


Figure 6.11: Accelerations for beam L = 100 m

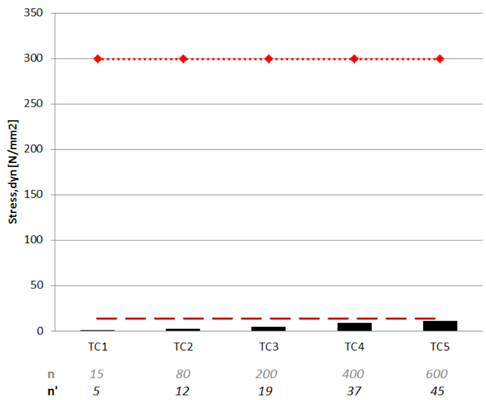


Figure 6.12: Stresses for beam L = 100 m

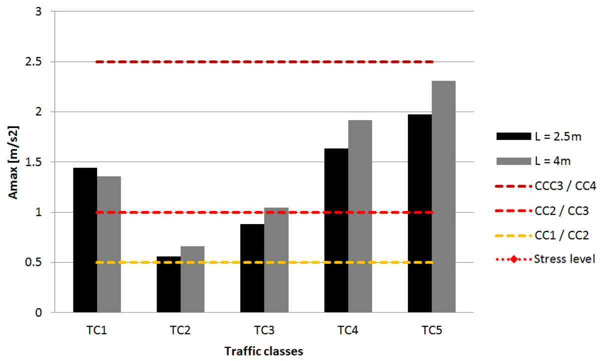


Figure 6.13: Acceleration for different traffic classes in case of a cantilever footbridge

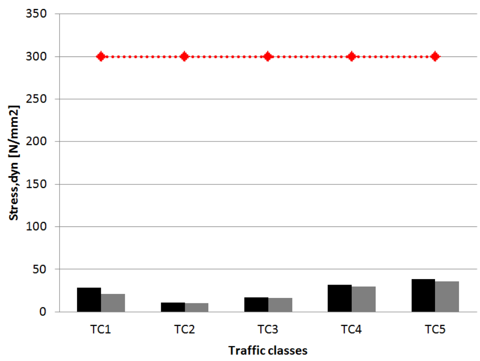


Figure 6.14: Stresses for different traffic classes in case of a cantilever footbridge

For the simply supported bridge the accelerations and stresses due to joggers are presented in the figures as well the. For determination of the jogger loading see section C.4 in the appendix. It should be mentioned that the amount of joggers is the same for the bridge with length 50 m and with length 100 m, namely 10 joggers (see subsection 4.1.4). For the bridge with a length of 12 m 5 joggers have to be considered.

It can be seen that the differences between the traffic classes are significant. This results in a large influence of the choice for a certain design scenario. In addition it can be seen as well that it can be expected that no damage to the structure can occur in case a crowd of traffic class 5 will cross the bridge: the stresses are far from the ultimate limit state. When for example the lowest traffic class in combination with the lowest comfort level is chosen as a design scenario, the accelerations of the bridge when an unexpected crowd with traffic class 5 passes can thought result in discomfort.

6.4.2. Special events

The two types of loading for special events as explained in section 6.2 are investigated for the simply supported bridges. In case of a cantilever footbridge attached to another bridge, special events are assumed to take place at the total bridge structure, resulting in a different response that is outside the scope of this impact study. The results are presented below and compared with normal crowd loading in relation to the traffic classes. It is interesting to see the impact of a more synchronized crowd of people for the level of comfort (the maximum bridge accelerations) and the stresses. The expected response of the bridge under jogger loading according to the Eurocode is plotted as well. The comparison with the joggers is made to see if it would be favourable to check the bridge for these design scenarios. The maximum accelerations are presented at the left for three different lengths in figures 6.15, 6.17 and 6.19. The stresses are presented at the right in figures 6.16, 6.18 and 6.20.

The two assumed load models are close to each other in case of short bridges, for long bridges the load model with $n' = 50\%n$ results in much higher accelerations and stresses compared to the load model with twice as much people synchronizing ($n'_{new} = 2 * n'$) as for the normal pedestrian loading. Damage (by reaching the stress limit) due to special events can be excluded for these assumed load models of special events. For short bridges the requirement of the joggers causes much higher accelerations and stresses than the load models for special events. This result suggests that for this type of footbridges no additional check for special events is necessary. In the case of long bridges, the results for the joggers are comparable to the load model with $n'_{new} = 2 * n'$. The load model with $n' = 50\%n$ gives much higher accelerations and stresses. It should be questioned which load model is more realistic. However, the impact of a more synchronized crowd is shown. For the shortest footbridge the current load model for joggers is governing, for the larger bridges a load model with a more synchronized crowd can become governing.

6.4.3. Vandalism

Two different models are set up as explained in subsection 6.2: i) the model according to the Eurocode with traffic class 5 and 50% of structural damping; ii) the jogger model: people jumping at the normative position of the footbridge perfect synchronized. The calculations are performed for the three simply supported structures and the two cantilever footbridges. Both the accelerations and the stresses are determined. For vandal

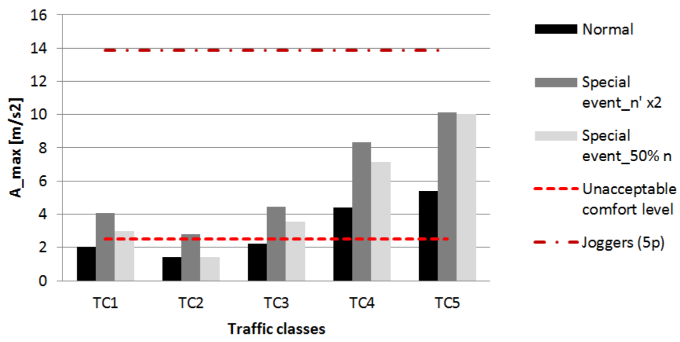


Figure 6.15: Accelerations for a bridge L = 12m

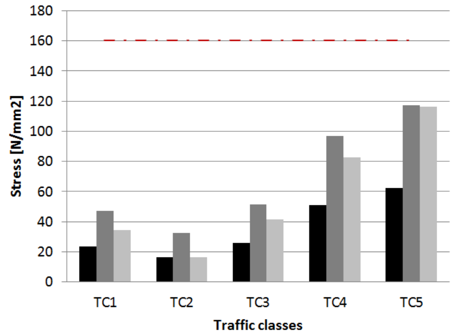


Figure 6.16: Stresses for a bridge L = 12m

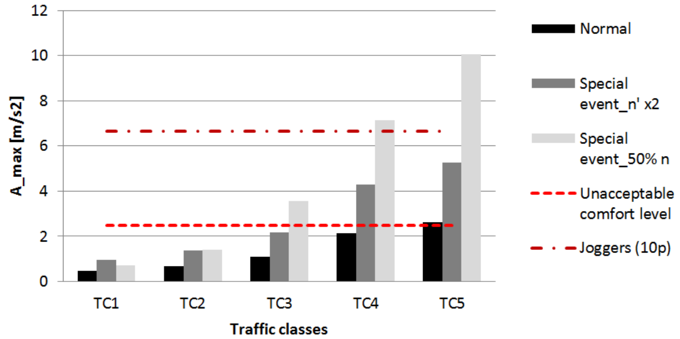


Figure 6.17: Accelerations for a bridge L = 50m

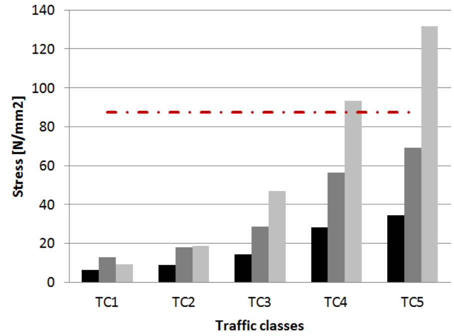


Figure 6.18: Stresses for a bridge L = 50m

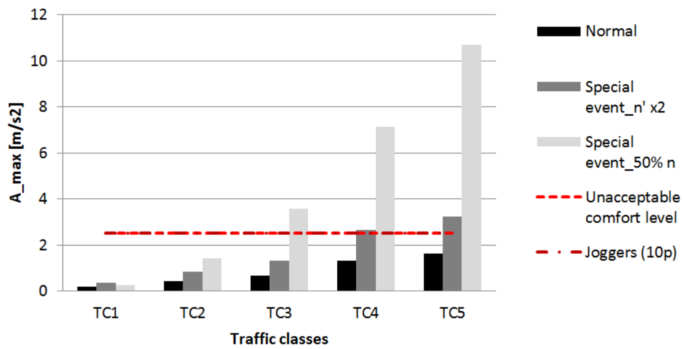


Figure 6.19: Accelerations for a bridge L = 100m

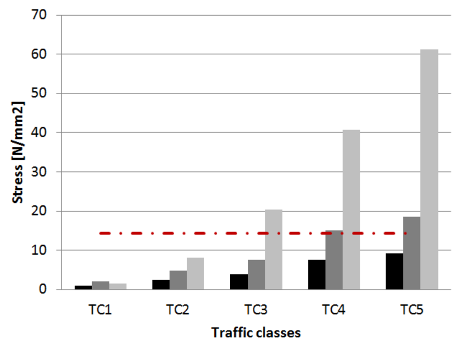


Figure 6.20: Stresses for a bridge L = 100m

loading, the maximum stresses that can be reached are the most interesting, as the intention of this loading is to damage the structure. Therefore only the graphs with the stresses are presented. It should be considered when the stress limit is reached and if in this case the load model is still realistic. In addition it is interesting to see the extent to which the two proposed load models correspond.

The first four graphs in figures 6.21 to 6.24 present the vandal loading for the simply supported beam. The two graphs in figures 6.25 and 6.26 show the stresses due to vandal loading for the two cantilever structures. The black line represents the stresses due to the vandal loading according to the Eurocode (model i), the grey line the stresses due people jumping (model ii). In all the graphs additional results are presented with a modification in the damping ratio. These results are discussed in subsection 6.4.4.

Future lightweight short footbridges are expected to be susceptible to vandal loading. This is because the short footbridge is expected to be most susceptible to vandal loading especially for new lightweight materials. For this bridge length the difference between two dead loads is presented in figure 6.22.

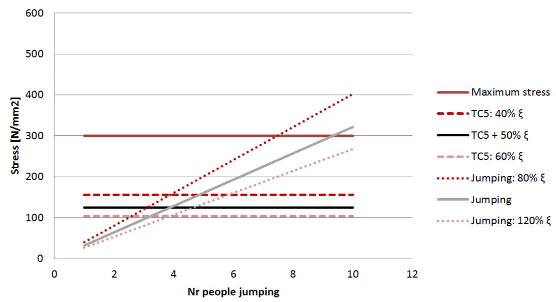


Figure 6.21: Stresses for a bridge L = 12 m

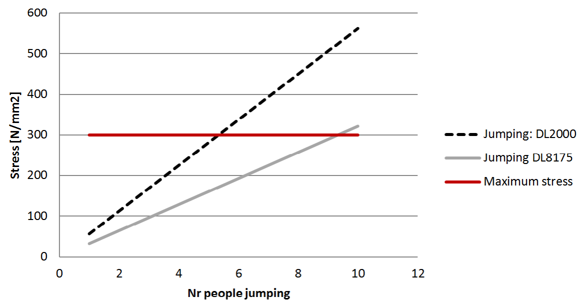


Figure 6.22: Stresses for a bridge L = 12 m with low dead load of 2000 [N/m^2]

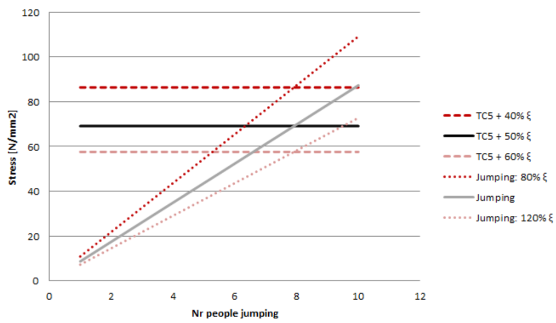


Figure 6.23: Stresses and acceleration for a bridge L = 50 m

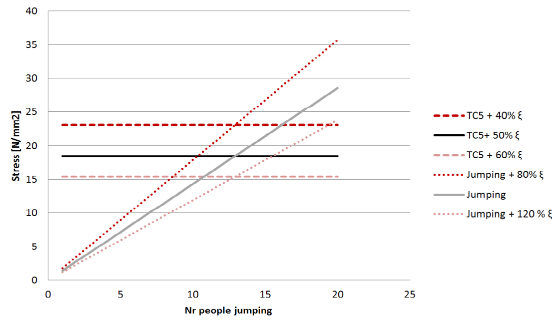


Figure 6.24: Stresses and acceleration for a bridge L = 100 m

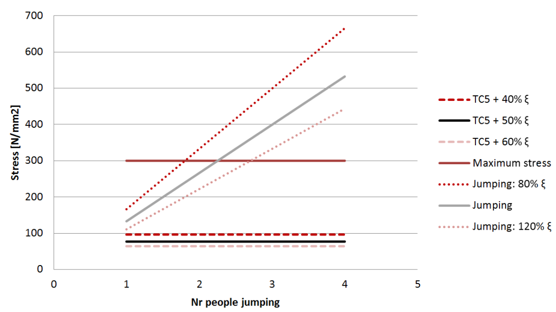


Figure 6.25: Stresses in a cantilever beam footbridge of 2.5 m

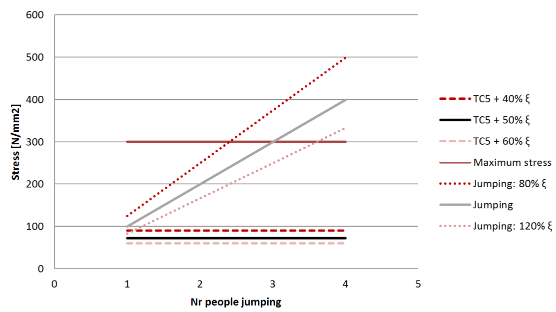


Figure 6.26: Stresses in a cantilever beam footbridge of 4 m

A first impression of the results shows that for a short simply supported bridges and cantilever bridges it is theoretically feasible to reach the stress limit with jumping people, whereas for large bridges it seems impossible. In the last case the number of people that is necessary, jumping all synchronous, has to be unrealistic large. The dynamic impact of the loading and the number of people that are able to jump synchronously is discussed in literature as well. Zivanovic concludes that larger groups have reduced synchronization between jumping people. "Rainer et al reported that individuals jumping in groups of two, four and eight people produced on average lower DLFs than when jumping alone. This holds particularly well for higher harmonics, but not for the fundamental harmonic which DLF exhibits values approximately the same as when a single person is jumping. Pernica added that the average vertical DLFs per person tend to decrease with increasing number of people (in all walking, running and jumping activities)."[10] However Bachmann stated that synchronizations of a small number of people seems possible at least when considering the first loading harmonic. Not only the number of people able to jump synchronously is of importance, the period of time they can maintain this behaviour should be taken into account as well.[11] The calculated stresses are the values of the steady state, but it can be questioned if this will be reached at all.

In addition it can be seen in figure 6.22 that the ULS in lightweight bridges is reached faster than in case of "normal" bridges. Though in general footbridges with a lower dead weight are expected to have a higher first natural frequency (and therefore expected to be not put into vibration by humans), for future lightweight designs with new materials like FRP the vandal loading could become an issue. Comparing both load models the results suggest that for large bridges the load model according to the Eurocode is at the safe side, but for short simply supported footbridges and cantilever footbridges people jumping at the bridge are expected to cause higher stresses.

6.4.4. Structural damping

Two different types of graphs are presented. The first two graphs (figures 6.27 and 6.28) show the accelerations for different structural damping ratios in case of a short and a long bridge. The original damping ratio used in the model and the damping ratio that may be applied for large vibrations (assumption of three times the original structural damping based on the ratios provided in the EUR 23984), are indicated by respectively a cross and a triangle. The result of a deviation in the structural damping ratio that can be caused by the difficulty in prediction, is visible related to the maximum expected accelerations. The others graphs in figures 6.29 to 6.36 show the difference in maximum acceleration caused by a deviation in structural damping of 20% (an assumption for the percentage that can be the expected difference between the design and reality). This result in deviation of acceleration caused by difference of structural damping, is presented in the graphs in previous subsection about vandalism as well (figures 6.21 to 6.26). For the simply supported structure the graphs for both acceleration and stresses for the three different bridge length are presented. In case of the cantilever beam the result for the accelerations for both bridge lengths is presented.

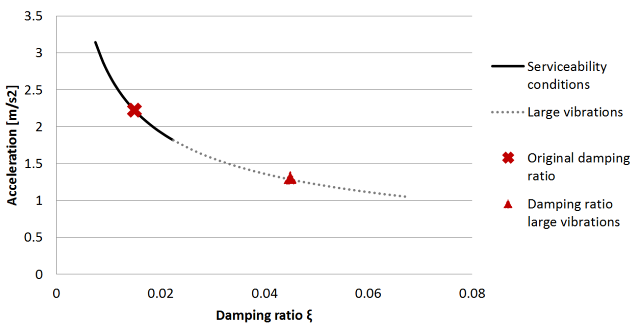


Figure 6.27: Deviation in damping ratio for a simply supported bridge L = 10 m

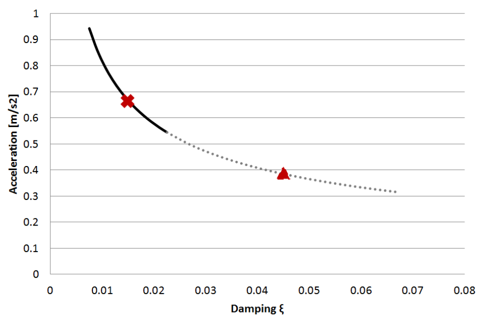


Figure 6.28: Deviation in damping ratio for a simply supported bridge L = 100 m

It can be seen that in case of vandal loading by jumping people a deviation in the damping ratio can have a large influence in the amount of people necessary to reach the ultimate limit state (see figures 6.21, 6.25 and 6.26). The figures 6.27 and 6.28 show that a small deviation in damping ratio can cause quite a change in the accelerations that are relevant for the evaluation of human comfort. Considering the stresses, the deviation is in the same order of magnitude, but the stresses still remain far from the stress limit.

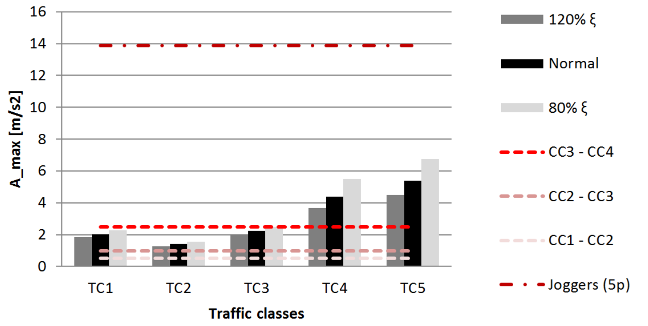


Figure 6.29: Accelerations for a bridge L = 12 m with different damping properties

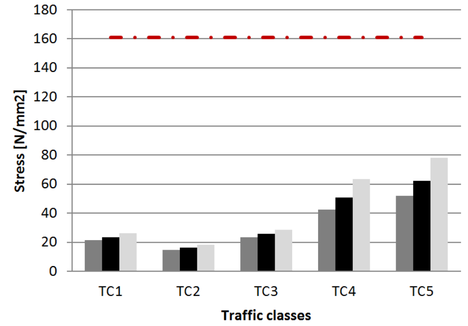


Figure 6.30: Stresses for a bridge L = 12 m with different damping properties

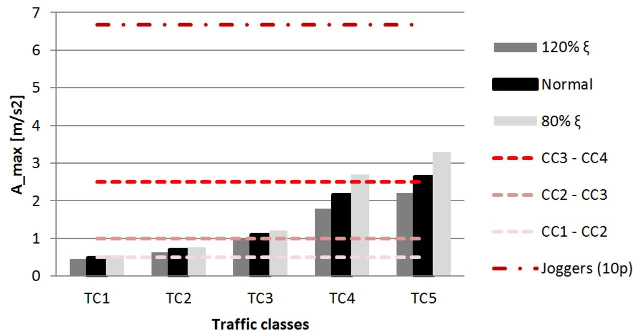


Figure 6.31: Accelerations for a bridge L = 50 m with different damping properties

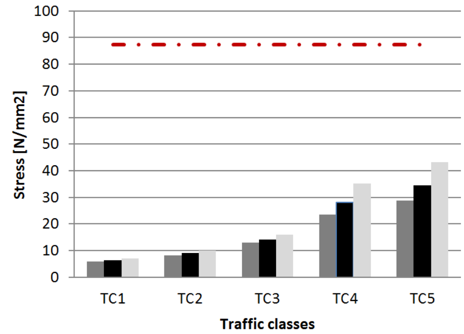


Figure 6.32: Stresses for a bridge L = 50 m with different damping properties

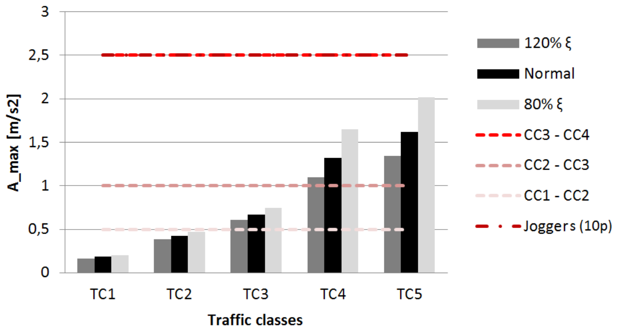


Figure 6.33: Accelerations for a bridge L = 100 m with different damping properties

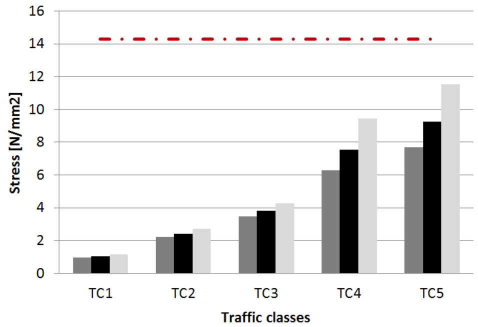


Figure 6.34: Stresses for a bridge L = 100 m with different damping properties

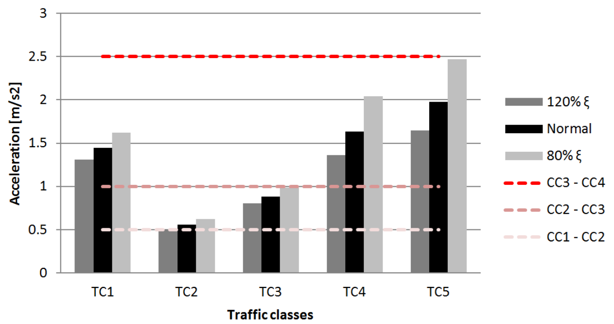


Figure 6.35: Accelerations in a cantilever beam footbridge of 2.5 m

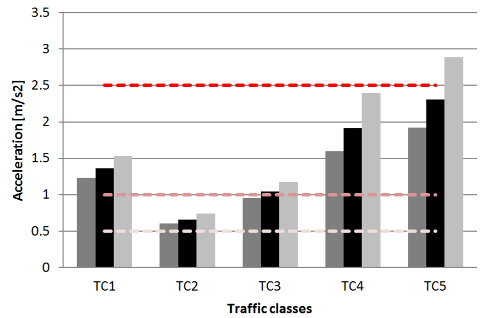


Figure 6.36: Accelerations in a cantilever beam footbridge of 4 m

6.4.5. Additional mass and damping pedestrians

Two types of human-structure interaction are presented in the model as discussed in subsection 6.2: added mass and added damping to the system due to the interaction between human and structure. The proposed values for additional mass and damping according to Georgakis et al [32] are used to get an impression of change in acceleration, frequency and damping according to this theory: an average added mass for a single pedestrian of 102% of the pedestrian's mass and an added individual damping coefficient with a fixed conservative value of 500 kg/s for vibrations up to 5 mm amplitude.

The results are presented in four graphs. Figure 6.37 shows frequency as function of the length including additional mass. Figure 6.38 shows damping as function of the length including additional damping. Figure 6.39 presents acceleration as function of length including additional mass and figure 6.40 shows acceleration as function of length including additional damping. Both TC3 and TC5 are examined with the changed properties and compared to the normal case.

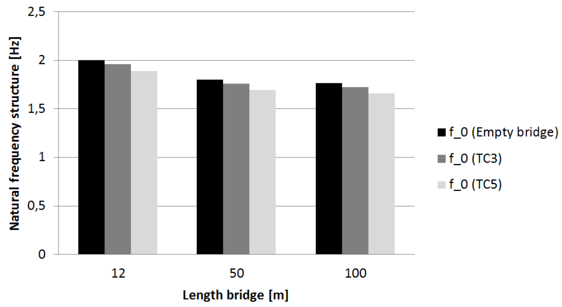


Figure 6.37: Change in frequency by taking into account additional mass due to pedestrians

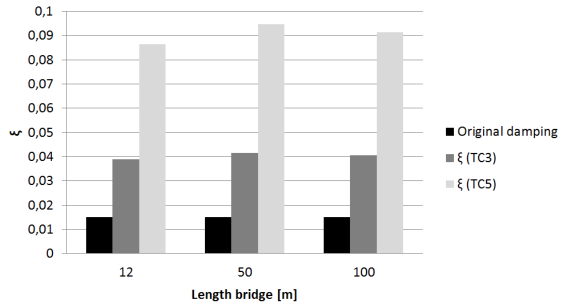


Figure 6.38: Change in damping by taking into account additional damping due to pedestrians

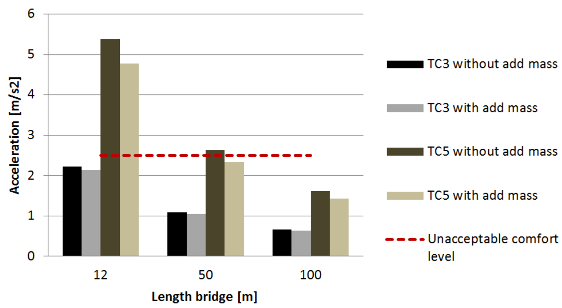


Figure 6.39: Change in acceleration by taking into account additional mass due to pedestrians

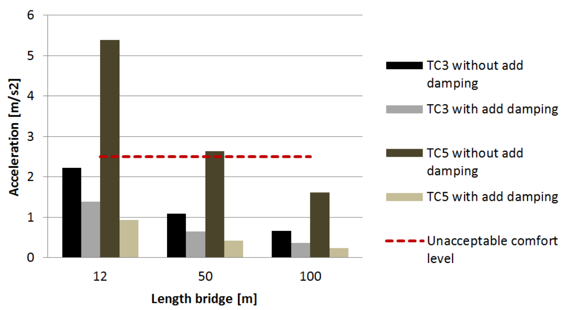


Figure 6.40: Change in acceleration by taking into account additional damping due to pedestrians

The graphs on the left take into account the assumed additional mass of the pedestrians. The deviation in natural frequency of the structure and expected accelerations due to this additional mass are relatively small. In case of acceleration the additional mass seems negligible, though in case of the natural frequency a small difference can result in changing the natural frequency of the bridge from outside the critical range into the critical range. Therefore it is important to consider this in the design of the footbridge.

The graphs on the right take into account additional damping caused by the interaction of the humans and the vibrating structure. Figure 6.38 presents the change in structural damping due to this added damping by pedestrians. The result of taking the additional damping into account in the calculations of the accelerations is presented in figure 6.40. The result is a significant difference. Especially for larger crowds (a higher traffic class) the reduction of the accelerations is major, due to the higher number of people.

7
Results part I

The combination of the results of the investigation and the impact study of the critical aspects in human-induced bridge dynamics is presented in this chapter. Divided into the four main topic, each topic is discussed separate, summarising the results of the two chapters.

7.1. Design scenarios

Lack of clarity

A design scenario is the combination of a traffic class (TC) and a comfort criteria (CC). The choice for a design scenario is a task of the client, making use of the EUR 23984. For engineering companies this guideline is a proper document, but it would be favourable for the client to give more guidance for the difference between the TCs and CCs, making the choice for a design scenario for the client easier. The difference in expected accelerations (and therefore in CCs) between the TCs has been shown to be significant, indicating the importance of a proper choice. There is a deviation in limits for human comfort between regulations internationally, making the regulations valid in the Netherlands being questioned. In literature a risk based approach in the choice for design scenarios is mentioned as a tool to make a proper choice.

Exceptional load cases

Exceptional load cases seem to take no part in the design of footbridges at the moment. It is the choice of the client to indicate if a check for an exceptional load cases is favourable. In this case it is important that a proper choice for a load case is used. This is discussed in section 7.2.

Probability of occurrence and consequences

The code prescribes one fixed pedestrian loading for all cases with one fixed level of comfort. As indicated before, a choice for a different design scenario can be made. In both the choice for the traffic class and the comfort level however, the probability of occurrence and the probability of the consequences is not taken into account. Therefore it is expected that the existing load models do not describe realistic design scenarios. This can result in misinterpretations of the serviceability of footbridges in the design. In addition this topic is not much studied in literature at the moment, therefore it is interesting to see what the result of a probabilistic approach would be.

7.2. Load definition

Differences between guidelines

The current available regulations are not consistent with each other. The most significant difference between the guidelines is the deviation in values for the reduction coefficient ϕ in the formula for the harmonic load model. This deviation turns out to be not much of a problem in the use of the guidelines. It is however a shortcoming in the Eurocode that the graph for the reduction coefficient does not provided the values at the axes.

Special events
Special events seem to be an important topic in the design of footbridges. However, in general little attention is paid to the modelling of crowd loading in literature. At the moment special events are not taken into account in the dynamic design check of footbridges. In addition the probability of occurrence in combination with acceptance is of importance as well. In the impact study two assumptions for the loading of special events have been considered, both based on a higher number of equivalent pedestrians. The results diverge, showing the importance of a proper load model.

Vandal loading
Vandal loading a well studied topic in literature. In combination with the impact study vandal loading is expected to not be an issue for large bridges. However, for short bridges vandal loading is expected to be able to reach the stress limit with jumping people. The load model according to the Eurocode seems to result in too low stresses for a short bridge, compared to people jumping on the bridge. A load model with jumping people is expected to be more realistic. This load model is governing, resulting in higher stresses than the current load model in case of the short bridges. For new short lightweight material footbridges vandal loading is therefore expected to be an important issue in the design.

Equivalent number pedestrians
The formulas for the equivalent number of pedestrians are in accordance with the numerical simulations. An experiment with crowds of pedestrians crossing a footbridge to check the reliability of the models is however not performed to verify to current load model. In addition this equivalent number of pedestrians is based on a flow of pedestrians uniformly distributed over the surface. The configuration of the traffic is therefore not taken into account in the determination of this equivalent number of pedestrians.

Jogger loading
The difference in the accelerations of the footbridge induced by the three types of jogger loading that are considered is large. The jogger loading seems to be governing in the design of footbridges. However, the critical range of the walking frequencies of joggers differs from the critical range of normal walking pedestrians. Therefore, dependent on the considered natural frequencies of the footbridge the choice for the relevant load model must be made.

7.3. Structural response

Structural damping
The results of the impact study have shown that the a deviation in the structural damping ratio will not cause damage to the structure. A change in the expected accelerations by this deviation in structural damping, can though cause more discomfort than considered in the design. The changes in response caused by the deviation in the structural damping ratio are in the same order of magnitude for all bridge types discussed. To avoid the use of TMDs after erection of the footbridge and to reduce the costs of the project it is favourable to be able to predict the amount of structural damping more precisely in the design stage. In relation with new materials it is expected that due to the more uniform design, the approximation of the structural damping can be closer to reality than for other materials if more research is performed.

Determination maximum acceleration
The determination of the maximum acceleration according to calculations methods (without FE programs) is expected to give a good estimation of the maximum acceleration of the bridge. Instead of looking at the maximum acceleration of the bridge it would be favourable to take into account the accelerations a pedestrian is exposed to crossing the bridge. It is questioned as well whether or not people are able to perceive peak values of acceleration.

Stresses and deflection
The deflection of a footbridge is only expected to be an issue in the design when it can cause panic. The stresses under dynamic loading are favourable to examine under vandal loading for short bridges, as indicated before. Besides exceptional load cases, it has been indicated that stresses are not likely to be a problem in the evaluation of human-induced dynamic loading on footbridges.

New materials

With the tendency towards more slender designs of footbridges, the use of new materials for the structure is of importance. Especially FRP is expected to be used increasingly in the design of footbridges in the future. More knowledge on the material would be favourable, to be able to reduce the high safety factors that are currently used in the design. The experimental research is still limited and therefore first more projects designed in FRP are favourable to gain more knowledge on and experience with the material.

7.4. Interaction effects

External interaction effects

External interaction effects like music are expected to cause higher synchronisation between the pedestrians. This synchronisation results in higher accelerations of the structure. The probability of occurrence of such load cases, expected to be more related to special events, in combination with the level of acceptance of vibrations should be taken into account in the design as well. At the moment it is the choice of the client to design a bridge for special events.

Lock-in

The lateral lock-in effect is studied extensively. For the vertical direction lock-in is possible, but not observed in reality. The lock-in effect results in higher accelerations, but the structural safety is not in danger. In addition a limit in the maximum accelerations is expected, as humans are expected to not only put energy but damping as well to the structure for vibrations in the vertical direction. Lock-in seems at the moment though more a topic for research, than an issue that is taken into account in the design of footbridges.

Additional mass and damping by pedestrian loading

In case of the expected accelerations of the structure, the additional mass seems negligible, though for the natural frequency a small difference in mass can result in changing the natural frequency of the bridge from outside into the critical range. Therefore it is important that this is taken into account in the design, which is the case at the moment (see subsection 4.2.2). The results of experiments about additional damping are not consistent. Additional damping by pedestrians could result in a limit value for the maximum expected accelerations due to normal crowd loading. In case of effects like lock-in, instead of additional damping added pedestrians are expected to put energy into the system.

8

CONCLUSIONS PART I

The goal of this first part was to detect the critical aspects in the evaluation of human-induced vibrations on footbridges. Resulting from the overview of the available literature and valid codes and regulations in the Netherlands four main topics with several subtopics came forward. By use of a literature study, meetings with engineering companies and an impact study with rough calculations the topics have been investigated and the most critical aspects for extensive research have been detected. It has been shown that despite the considerable amount of research that has been carried out on human-induced vibrations on footbridges, still some topics are not fully covered yet and therefore of relevance for more extensive study. The following four topics are indicated as important critical aspects in the evaluation of human-induced vibrations on footbridges.

Vandal loading

The structural safety of short footbridges and in addition of the more sensitive footbridges designed in new lightweight materials, should be checked to vandal loading. It is suggested that applying a load of jumping people instead of the dense crowd that is proposed in the Eurocode gives a more realistic approximation. The impact study underlines that this load model can become governing for short bridges.

Structural damping

The structural response of a footbridge is highly influenced by the amount of structural damping, which is a hard parameter to accurately quantify. For the use of new materials the knowledge about structural damping is even less. The impact study shows that the influence of structural damping is significant in the response of the structure, which makes it an interesting and important topic for further research.

Additional mass and damping by pedestrians

The possibility of pedestrians to add mass and damping to the structure is of importance in the design of footbridges. The results of the impact study confirm that a change of properties of the total structural system of the footbridge due to the human-induced loading is expected to have a substantial impact.

Probability of occurrence and consequences

The final bottleneck that has been detected as an important issue for further research, is the probability of occurrence of accelerations in footbridges in relation to the probability of discomfort for individual pedestrians. The current guidelines provide the same criteria for all types of footbridges, independent of the situation. It is expected that this is not a realistic assumption, resulting in too conservative designs.

II

PROBABILITY-BASED APPROACH OF THE VIBRATION SERVICEABILITY OF FOOTBRIDGES UNDER VERTICAL PEDESTRIAN LOADING

9
General aspects part II

In part I it has been demonstrated that the probability of occurrence of accelerations in relation to the probability of discomfort for individual pedestrians is a critical aspect in the design of footbridges. At the moment the codes and regulations provide the same criteria for all types of footbridges. As a result, it is expected that the Eurocode and Dutch national annex are conservative in the evaluation of vertical human-induced vibrations in footbridges. The applicability of the codes and guidelines is of importance in the design of future footbridges. Therefore this topic is studied in this second part of the work.

9.1. Problem definition

Initially the Dutch national annex of the Eurocode provides a fixed pedestrian loading for the evaluation of the dynamic loading for footbridges of 0.5 P/m^2 (corresponding to the third traffic class (TC3) of the EUR 23984). For this pedestrian loading a fixed criterion for (dis)comfort is presented in terms of a maximum allowable vertical bridge acceleration of 0.7 m/s^2. In reality the traffic density is not uniform for every situation and in addition the accelerations perceived by pedestrians will differ per situation and per individual pedestrian. This is recognized by the Eurocode as well, resulting in the following two statements that allow a certain freedom in design:

- "It might be required to link the traffic categories and the applicable design scenarios for each single project."[2]
- "In the project specification other maximum admissible accelerations in relation to the comfort criteria might be set down."[4]

These statements provide a certain freedom in the design for engineering companies, but there is no unequivocal guideline for the application of human-induced loading and for criteria of the human perception of bridge vibrations. This makes it hard to take advantage of this freedom.

The uncertainty in quantification is underlined by Bruno and Corbetta by presenting some key-features of pedestrian traffic: it is a dynamic transport and multi-scale phenomenon; the pedestrian density is not constant; pedestrians are active fragments; and a detailed description of the traffic is only possible, once more information of coarse distribution is available.[56]

Recently Racic and co-workers presented a review paper where they outlined the underdeveloped stochastic concept in the field of footbridges. "Although the concept of variability and uncertainty is well developed in structural dynamics disciplines such as wind, wave and earthquake engineering, the stochastic concept is surprisingly underdeveloped in the area of human-structure dynamic interaction where there is a considerable randomness of the key design parameters related to the human-induced dynamic loads and structural dynamic properties."[57]

It is expected that the Dutch national annex is conservative in the evaluation of human-induced vibrations in footbridges, resulting in oversized structures. This idea is based on two aspects. The first aspect concerns the probability that the prescribed pedestrian traffic (in pedestrians per square metre) will indeed occur, is not taken into account. This probability is also likely to be much lower for a large footbridge than for a short one:

to get the same density for all bridges, the number of pedestrians present on a large bridge has to be much higher and therefore the probability of occurrence can be expected to be lower. Secondly, it can be questioned if a pedestrian can really perceive a single peak acceleration. Additional to this, not every pedestrian will perceive the peak acceleration occurring at the critical point of the bridge. This is relevant because at the moment this maximum peak acceleration is used as a measure for (dis)comfort.

9.2. Research objectives

The problem definition described above resulted in the following main goal: to find out if the suspicion that the Eurocode is conservative regarding the evaluation of human-induced vibrations in footbridges is well founded. To achieve this goal, an approximation closer to reality is suggested: a probability-based approach, based on a combination of probabilities of occurrence rather than on fixed values. Therefore, in the second part of this study the vibration serviceability of footbridges under vertical pedestrian loading has been investigated using such a probability-based approach. The aim is to give a more realistic approximation of the actual accelerations pedestrians are expected to perceive when they cross a bridge in combination with the perception of discomfort in relation to the serviceability of the footbridge.

Main question

Can a probability-based approach demonstrate that the Eurocode is conservative in the evaluation of human-induced vibrations in footbridges?

Sub-questions

To answer the main research question the following sub-questions are of interest to discuss:

- Which information is required to make a reliable probability-based estimation of pedestrian comfort on footbridges?
- When is it allowed to make a free choice for the serviceability limit state design scenarios related to pedestrian induced bridge vibration?
- What is the probability of occurrence for certain pedestrian traffic?
- Which accelerations does an individual pedestrian experience walking across a bridge?
- When do people feel uncomfortable in the use of footbridges?
- What is the probability a pedestrian experiences a certain level of (dis)comfort?

9.3. Probability-based design

The basic idea of the probability-based design in engineering is that failure occurs when the load exceeds the resistance (grey area in figure 9.1). The probability of failure (P_F) is the probability that the load (S) exceeds the resistance (R). This P_F can be determined by combining the probability distribution of the load on the structure (f_S) and of the resistance of the structure (f_R) as in figure 9.1.

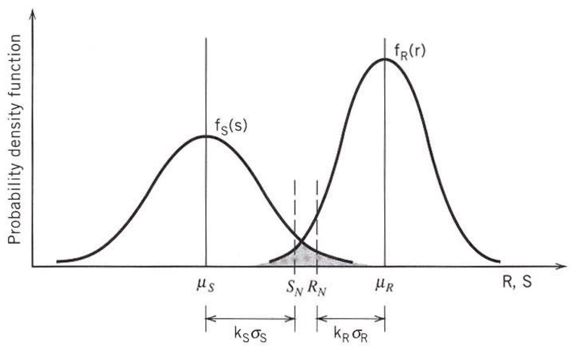


Figure 9.1: Fundamentals of risk evaluation [20]

In general the probability of failure can be described with equation 9.1 based on the probability distributions of the load and the resistance of the structure. In this formula $F_R(x)$ is the cumulative density function (CDF) of the resistance of the structure and $f_S(x)$ is the probability density function (PDF) of the load on the structure.

$$P_F = P(S > R) = \int_{x=-\infty}^{\infty} \left(\int_{y=-\infty}^{x} f_R(y)dy \right) f_S(x)dx = \int_{x=-\infty}^{\infty} F_R(x) \cdot f_S(x)dx \tag{9.1}$$

In general two types of limit states can be distinguished: the ultimate limit state (ULS) and the serviceability limit state (SLS). The ULS corresponds to a state associated with structural failure, dealing with the safety of the people or the structure. The SLS deals with comfort of the users and the functioning of the structure. The ULS failure probability of a structure built for 50 years is prescribed to be $7 \cdot 10^{-5}$ according to the Eurocode. The allowed SLS failure probability for irreversible damage for the same structure is $6.681 \cdot 10^{-2}$. A SLS failure probability for reversible damage of 0.01 is prescribed for buildings based on the reference period. This value may also be used for bridges.[2]

The safety of people in relation with structural reliability is captured in the ISO 2394 standard as well by an acceptable maximum value for the failure probability of 10^{-6}.[21] This value is based on the overall individual lethal accident rate of 10^{-4}. The maximum allowable probability of failure of the structure ($P(f|\text{year})$) depends on the probability of a person begin killed, given the failure of the structure ($P(d|f)$):

$$P(f|\text{year}) \cdot P(d|f) < 10^{-6}\text{year}^{-1} \tag{9.2}$$

If the translation is made to the serviceability of footbridges the load is similar to the accelerations a pedestrian experiences when crossing the bridge, the resistance is the (dis)comfort criterion for a pedestrian in that particular situation and moment in time, and failure is equal to the feeling of discomfort for a pedestrian.

In the case of serviceability to comfort of a footbridge the client can decide what is an acceptable percentage of the pedestrians that may experience a feeling of discomfort crossing the footbridge, other than the 0.01 prescribed by the Eurocode.

Following on from the probability-based approach a new flowchart is proposed in figure 9.2, based on the flowchart in the EUR 23984 (figure 4.3, including design steps that have to be taken into account to check if a footbridge will meet the comfort criteria under pedestrian loading). The probability-based approach of this second part is set up based on this flowchart and evaluated by use of a case study.

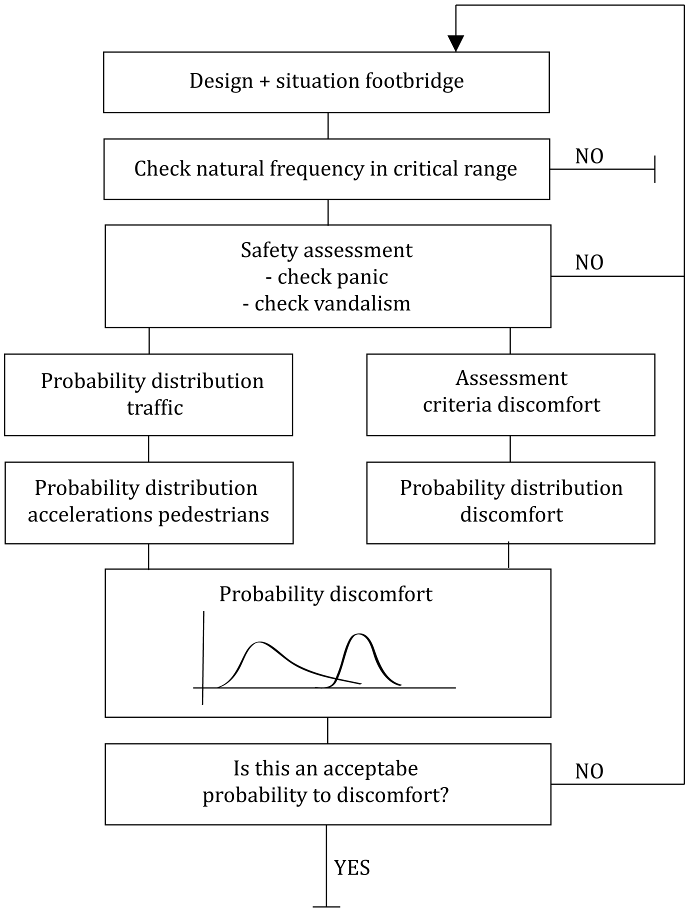


Figure 9.2: Flowchart of the probability-based approach

9.4. Scope

The same constraints as for the first part hold: only pedestrian traffic is taken into account and the vibrations are evaluated in the vertical direction. In addition only normal walking pedestrian traffic is considered (excluding joggers, vandal loading and special events) and interaction (both human-structure and human-human interaction) is left out.

The case study is performed with the same three simply supported bridges as used in the first part of this study. The first natural frequency of the three bridges is in the critical range of the walking frequencies of pedestrians and the bridge properties are constant across the length.

9.5. Reading guide

This second part focuses on a probabilistic approach of the vibration serviceability of footbridges under vertical pedestrian loading. The probability-based approach is investigated in chapter 10 and a case study of this approach is performed in chapter 11. Each sub-question is discussed in a separate section for both the probability-based approach and for the case study. First the requirements for a probability-based approach are discussed in section 10.1 and the safety assessment is evaluated in section 10.2. This is followed by the determination of the pedestrian traffic in sections 10.3 and 11.1. The accelerations of individual pedestrians are discussed in sections 10.4 and 11.2. In sections 10.5 and 11.3 is dealt with human (dis)comfort to vertical vibrations. Finally the combination of acceptance and the probability of discomfort is discussed in sections 10.6 and 11.4. The results of the probability-based approach are presented in chapter 12. The conclusions and recommendations are elaborated in respectively chapter 13 and 14.

10
Probabilistic approach

10.1. Requirements probability-based approach

In order to be able to evaluate the vibration serviceability of footbridges under vertical pedestrian loading with a probability-based approach, first it has to be considered which information is required to make a reliable probability-based estimation of pedestrian comfort on footbridges. For probability-based design, as discussed in section 9.3, two probability distributions have to be considered. First the probability distribution of the accelerations pedestrians perceive when crossing the bridge has to be considered. Secondly, this probability distribution has to be combined with the probability distribution for the comfort criterion for individual pedestrians. These two probability distributions have to be determined specifically for the situation that is considered.

To determine the probability distribution of the accelerations pedestrians perceive when crossing the bridge, the expected traffic has to be determined first. For the expected traffic the accelerations the individual pedestrians perceive can be evaluated. This results in the considered probability distribution. The combination of the two probability distributions results in the probability of (dis)comfort for individual pedestrians in a specific situation. This probability can be compared to the level of acceptance. In case these do not match, the design can be adapted.

10.2. Safety assessment

10.2.1. Principles

Above all, the safety in the use of a structure should be guaranteed. In the design of a footbridge a check with the static pedestrian loading is performed for the ultimate limit state (ULS). The ULS corresponds to a state associated with structural failure.[21] In addition, safety in the use of the structure under the dynamic pedestrian loading should be ensured as well. Once this safety is guaranteed, design choices regarding the serviceability of the structure are of concern. This leads to the first sub-question: "When is it allowed to make a choice for design scenarios?", in other words: "Which checks must be performed prior to the choice for design scenarios?" Two topics are of interest to answer this question: panic and vandalism.

10.2.2. Panic

General aspects

Back to the year 1883 when a crowd with hundreds of people walked across the just finished Brooklyn Bridge in New York City. A cry from the crowd "The bridge is collapsing!" that caused panic to get off the bridge. The result: the death of 12 people that were trampled.[58] More recent, 2010 in Phnom Penh (Cambodia), at the celebration of the Khmer Water Festival a stampede occurred at a suspension bridge connecting an island to the main land. The sway of the suspension bridge caused panic among a few people thinking the bridge would collapse, not knowing this sway was a normal behaviour without risk. This arise of panic resulted in the death of at least 350 people.[59]

These incidents indicate the importance of avoiding panic at any time for all pedestrians. Though panic itself is only unfavourable, the consequences can be disastrous. Panic is the sudden fright or fear of danger, which can have various origins. These origins can be divided into three main categories: structure, humans and external. The arise of panic due to one of the last two categories is hard to predict within the scope of this study. Therefore only the first item, panic with the behaviour of the structure as a cause, will be taken into account.

In addition it should be mentioned that there is the phenomena of escape panic: the panic that occurs after a first panic attack due to the time it takes to get off the bridge. It is also known as crowd stampede, often leading to fatalities as people are trampled or crushed.[60] Stampede can be described as a result of panic, but it is not the cause of it. Therefore it can be evaded by avoiding the arise of panic in general.

Panic due to structural behaviour
Panic due to the structure of the bridge is caused in general by the feeling that the bridge is unsafe. The reason for this feeling can for instance be vibrations of the bridge deck or in general an unsafe looking structure (due to high deflection, noises, damage to the bridge, etc.). It is hard to predict when the appearance of the structure can result in panic, this aspect is outside the scope of this master thesis. Therefore only the first topic, vibrations of the bridge, will be taken into consideration.

As stated before, vibrations of the bridge deck are expressed in accelerations. The higher the amplitude of the accelerations, the less the feeling of comfort. It can therefore be expected that there is a certain limit level above which the amplitudes of the accelerations are so high that panic will occur. In literature no such limit can be found for footbridges, although concerning to grandstands there is some literature available and experiments have been performed.

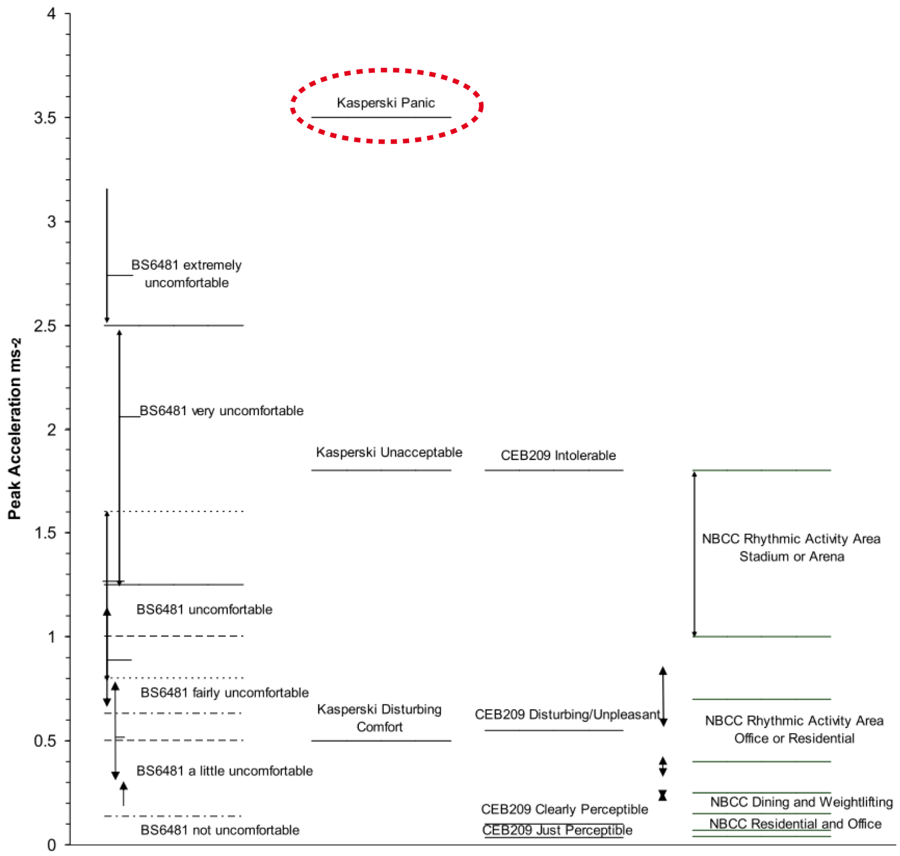


Figure 10.1: Summary of acceleration limit serviceability criteria for grandstands by Browning with the panic limit level of accelerations according to Kasperski in the red circle [61]

Browning made an overview of limit levels of peak accelerations in relation to comfort for grandstands according to different studies, presented in figure 10.1. Based on experiments that have been carried out on a permanent grandstand, Kasperski presents a limit level for panic expressed in peak accelerations as a measure of vibration exposure. The result is a limit for the peak acceleration of 3.5 m/s^2 above which panic is likely to occur (indicated in the figure by the red circle). Ellis et al underline this panic level by presenting a table with a vibration level for the peak acceleration on grandstands, in which exceeding 35% g (= 3.43 m/s^2) is presented as a probable cause of panic.[62]

The evaluation based on peak accelerations does not take into account the duration of exposure. Marques presents a graph with vibration limits for the acceleration for grandstands making use of the root mean square (r.m.s.) of the acceleration signal. The r.m.s. evaluation of accelerations does take into account the duration of measurement of vibration. The graph shows a limit level of the r.m.s. of the acceleration for panic in grandstands depending of the frequency of vibration as well. The result for the panic level according to Marques is a r.m.s. acceleration of about 3 m/s^2 at a frequency of 1.8 Hz above which panic is likely to occur (see figure 10.2).[63]

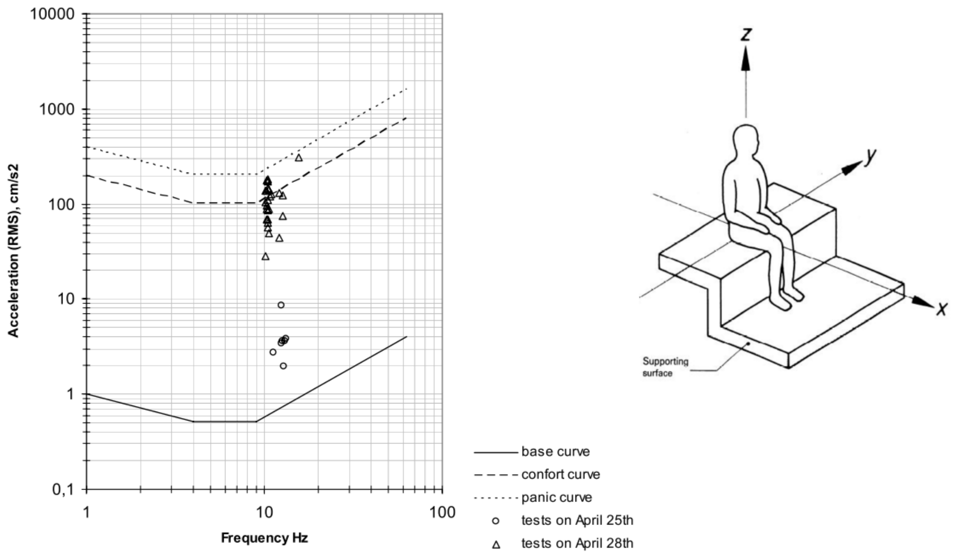


Figure 10.2: Vibration z-axis limits curve for acceleration (foot-to-head direction) and test values for grandstands by Marques [63]

Discussion
The presented limits are examples of studies concerning grandstands and therefore indicate a panic level for sitting persons. Persons who are sitting are expected to have a different perception of vibrations than persons who are standing or walking. It is expected that the limit level for panic in case of walking persons is higher than the presented values.

Beyond the upper limit value of 1.5 P/m^2 walking of pedestrians becomes impossible and the dynamic effects significantly reduce.[6] This indicates that a check for panic would be realistic for a pedestrian flow of 1.5 P/m^2, as this is the highest expected density where the bridge is expected to be put in vibration by the dynamic character of the walking pedestrians.

10.2.3. Vandalism

Vandal loading is discussed as a critical aspect in the evaluation of human-induced vibrations in subsection 5.2.2. The load model for vandalism is presented in the impact study in section 6.2 with the corresponding results in subsection 6.4.3. For more detailed information reference in made to these paragraphs. To repeat the conclusion from the first part: the structural safety of short footbridges and in addition of the more sensitive footbridges designed in new lightweight materials, should be checked to vandal loading. A load model with jumping people instead of the dense crowd that is proposed in the Eurocode is the governing load model for vandal loading in case of short bridges. It is expected as well that this load model gives a more realistic approximation.

10.3. Probability and distribution of pedestrian traffic

10.3.1. Current situation

Pedestrian traffic and crowds behave and occur randomly. Depending on the situation different types of traffic can be expected: at the end of a football game a dense stream of people leaves the stadium, where at a Sunday morning just a few people might be walking in the park. According to the Eurocode, as said before, a fixed flow of 0.5 pedestrians per square meter should be applied on the bridge structure for the evaluation of the vibrations. This value is independent of the location and size of the bridge, though there is a difference in the expected traffic. Figure 10.3 shows an example of a flow of pedestrians with an overall density of 0.5 P/m^2.

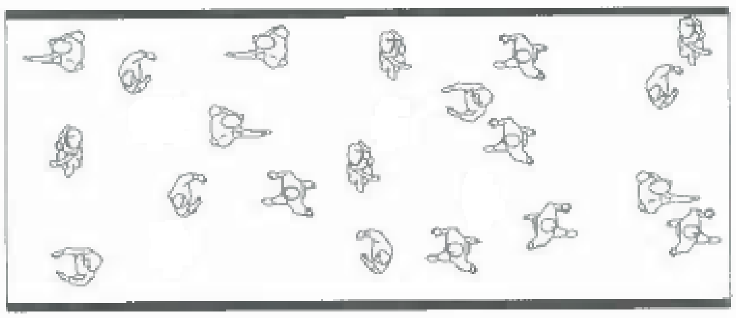


Figure 10.3: Pedestrian density of 0,5 P/m^2 based on a figure with different pedestrian densities presented by Schlaich [64]

In addition to this procedure the Eurocode refers to the EUR 23984 which provides a choice between five different traffic classes (figure 4.4). These five different traffic classes are based on a constant density. In reality, such a constant pedestrian flow is not expected to be present all the time. Depending on for example the location, the time of the day or the type of traffic, different densities of pedestrians will be present in the flow. Bruno and Corbetta mention in their paper presented at EURODYN 2014 that "most of the human-induced force models developed so far in structural engineering are deterministic, despite the intrinsic randomness of the crowd behaviour".[56]

10.3.2. Methodology approach

To answer the sub-question "What is the probability of occurrence for certain traffic classes?" in this study two approaches that are more related to the actual pedestrian traffic are proposed: the first approach is based on a combination of traffic densities and the second one on a probability distribution of expected group formation. Both methods are based on the idea that the expected pedestrian traffic is situation dependent. The Eurocode approach is based on pedestrian traffic densities, hence both methods will lead to an expected pedestrian traffic density as well so as to be able to compare the results of these methods with the prescribed density from the Eurocode.

10.3.3. Scenarios based on densities

For the first approach two main issues in the expected pedestrian traffic presented above are taken into account: (i) the pedestrian traffic will depend on the situation and location and (ii) instead of a fixed density per scenario, an expected probability of occurrence of various densities is considered. The first issue will be dealt with by using scenarios. A scenario will be described by means of a location and a situation which together include a certain expected traffic. To take the second issue into account per scenario a probability mass function is established. The probability mass function assumes a certain probability of occurrence for each density considered. This mass probability indicates the change that a person at the bridge is within a certain traffic density. The probability mass function is therefore specific for that particular scenario. This approach will be referred to as scenarios based on densities.

10.3.4. Scenarios based on group formation

The first approach is based on constant traffic densities, resulting in the same expected traffic density independent of the size of the bridge. To include the influence of the bridge dimensions, the second method describes scenarios based on group formation of pedestrians. Per scenario a probability mass function is proposed, but now expressed in the expected size of the groups within a flow. The mass probability indicates the chance that a person at the bridge is within a group with a certain dimension. In addition an expected time between the groups, depending on the scenario, is determined. This approach will be referred to as scenarios based on group formation.

10.3.5. Collection of data

In the ideal situation, a database is available with collected data about pedestrian traffic crossing bridges for different types of situations and structures, obtained during several years of data collection. Data about car and bicycle traffic is widely available, due to the convenient ways of measuring: a strip is put in the road surface to detect every vehicle that crosses this point. In the field of pedestrians traffic the collection of data is more of a challenge, resulting the availability of just a few sets of data. Bruno and Corbetta assume that the reason for the lack of statistics on the incoming pedestrian traffic is "certainly due to the technical difficulties involved in automatic pedestrian counting and pedestrian density estimation".[56]

The data that is available is expressed in number of pedestrians per hour. Therefore the average density per hour, based on an assumed average walking velocity, can be determined. This average density contains no knowledge of the group sizes within a pedestrian flow or the local densities. The few data sets with normal pedestrian traffic have been used, in order to get an approximation of the expected amount of pedestrian traffic at footbridges and to be able to add value to the two proposed methods.

10.4. Vibration analysis for individual pedestrians

10.4.1. Principles

The serviceability of footbridges is related to comfort, and in this field of study related to vibrations that are expressed in accelerations of the bridge. The current regulations for footbridges provide an acceleration analysis based on the maximum acceleration of the bridge deck under a sustained pedestrian loading for a certain traffic density (see chapter 4). The probability a single pedestrian will indeed experience this maximum bridge acceleration is not taken into account. On the other hand, the serviceability of footbridges is related to the accelerations individual pedestrians experience when crossing the bridge. Pedestrians should generally feel comfortable when crossing the footbridge. Therefore in this study a step to a more realistic approach has been made by taking into account the accelerations which individual pedestrians experience when crossing the bridge instead of the maximum structural response.

This approach is recently underlined by Sahnaci and Kasperski in a paper presented at the EURODYN 2014 where instead of using the bridge accelerations it is recommended to look at the accelerations pedestrians really experience when crossing the bridge. "In the context of comfort and safety criteria for the active user it is more reasonable to focus on the maximum experienced vibrations of the individual instead of identifying maximum structural responses." [65]

10.4.2. Equation of motion

In order to calculate the accelerations for an individual pedestrian first the equation of motion for the footbridge will be set up. The bridge is modelled as an Euler-Bernoulli beam with a distributed load (the pedestrians). The general equation of motion for this Euler-Bernoulli beam is:

$$m(x)\frac{\partial^2 u(x,t)}{\partial t^2} + \frac{\partial^2}{\partial x^2}\left(EI(x)\frac{\partial^2 u(x,t)}{\partial x^2}\right) + c(x)\frac{\partial u(x,t)}{\partial t} = p(x,t) \tag{10.1}$$

The dynamic pedestrian loading on the footbridge is represented by $p(x,t)$ and $u(x,t)$ is the deflection of the structure. In this formula $m(x)$ is the mass per length, $EI(x)$ is the bending stiffness per length and $c(x)$ is the damping ratio per length. All three parameters are in this work assumed to be constant for the length, so independent of x. This results in the following simplified equation of motion:

$$m\frac{\partial^2 u(x,t)}{\partial t^2} + EI\frac{\partial^4 u(x,t)}{\partial x^4} + c\frac{\partial u(x,t)}{\partial t} = p(x,t) \tag{10.2}$$

The deflection is assumed to be a summation of mode shapes $\phi_n(x)$ times the corresponding time dependent deflection $u_n(t)$:

$$u(x,t) = \sum_{n=1}^{\infty} u_n(t)\cdot\phi_n(x) \tag{10.3}$$

10.4.3. Mode shapes

The bridges considered in this study are simply supported bridges having sinusoidal mode shapes:

$$\phi_n(x) = \sin\left(\frac{n\pi x}{L}\right) \tag{10.4}$$

with the natural frequencies [18]:

$$f_n = \frac{1}{2\pi}\frac{(n\pi)^2}{L^2}\sqrt{\frac{EI}{\rho A}} \tag{10.5}$$

For the calculation of the accelerations only the relevant mode shapes have to be taken into account. These are the mode shapes that have a natural frequency in the critical range of the walking frequencies of pedestrians.

The assumption is made that the mode shapes can be separated. For each mode shape the deflection of the structure can be calculated and the summation of these deflections is the total deflection. This property is used to solve the equation of motion 10.2 by use of a mass-spring-damper-system (MSDS) with only one degree of freedom for each natural frequency of the structure. Modal analysis is used to transform the differential equation dependent on both x and t into a differential equation only dependent on t, by integrating over x and applying an orthogonality relation. Modal analysis is explained in more detail in appendix B. For every mode shape a modal mass, modal stiffness, modal damping and modal load are determined. The differential equation 10.6 has to be solved for each mode separately and by superposition of the deflections the total deflection can be determined (expression 10.3).

$$M_n^* \frac{\mathrm{d}^2 u_n(t)}{\mathrm{d}t^2} + C_n^* \frac{\mathrm{d}u_n(t)}{\mathrm{d}t} + K_n^* u_n(t) = P_n^*(t) \tag{10.6}$$

The real vibrations signal $u(x,t)$ is a summation of the output $u_n(t)$ for each mode shape multiplied by the influence of that particular mode shape at the position x. In figure 10.4 an overview of the simplification steps of the problem is presented.

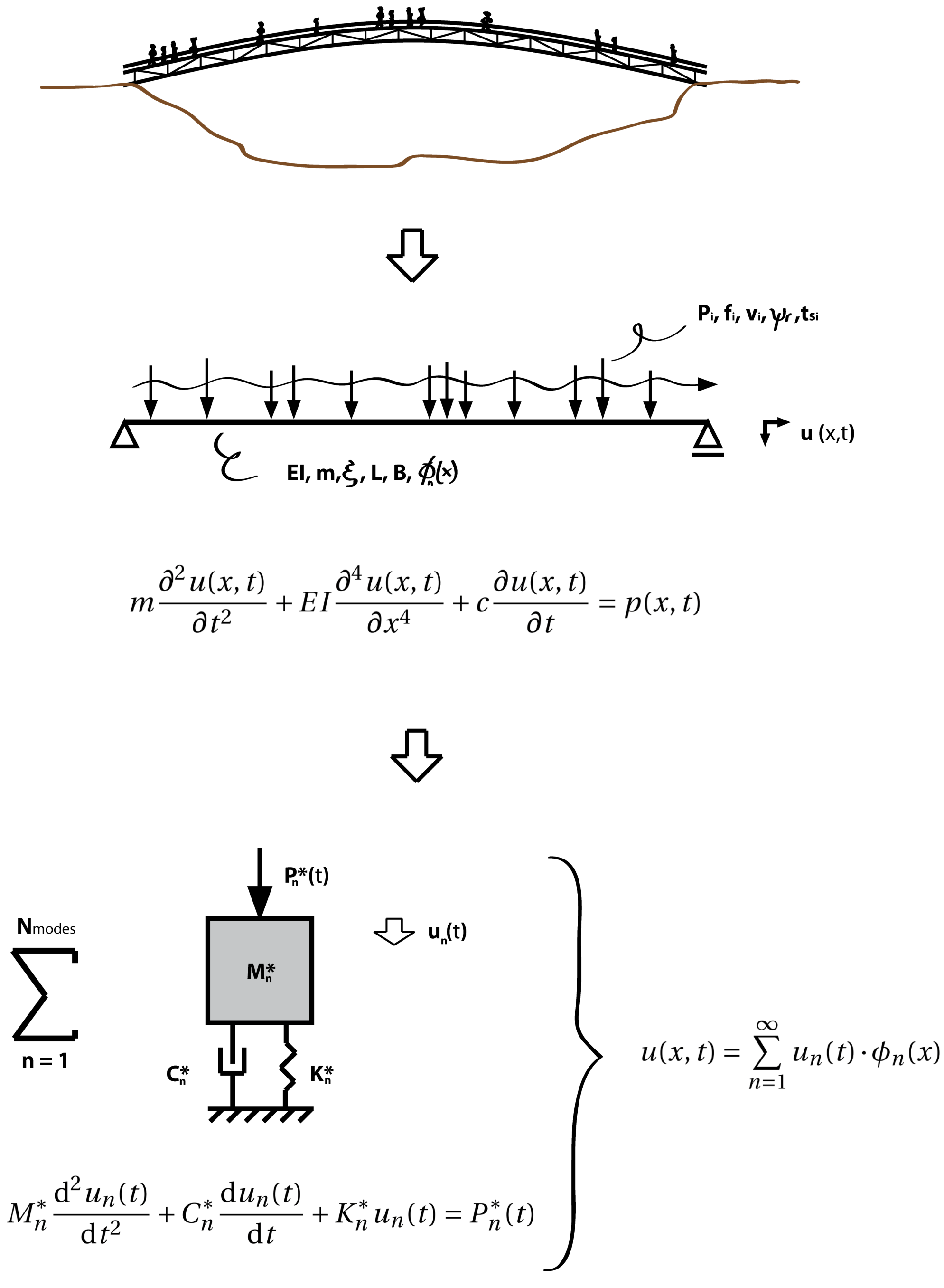


Figure 10.4: Overview of the steps taken for modelling pedestrians crossing a footbridge in Matlab

10.4.4. Modelling a footbridge

As said before (section 6.1.1) the mass, stiffness and damping are assumed to be constant over the length and are therefore independent of the position x on the bridge. For the use of equation 10.6 the modal mass M_n^*, modal stiffness K_n^* and modal damping C_n^* have to be determined to describe the footbridge. The result is shown below for the simply supported bridges with a sinusoidal mode shape. The derivation of these expressions is presented in appendix B.

$$M_n^* = m \int_0^L (\phi_n(x))^2 dx \tag{10.7a}$$

$$= m \frac{1}{2L} \tag{10.7b}$$

$$C_n^* = c \int_0^L (\phi_n(x))^2 dx \tag{10.8a}$$

$$= c \frac{1}{2L} \tag{10.8b}$$

$$K_n^* = EI \int_0^L \left(\phi_n''(x)\right)^2 dx \tag{10.9a}$$

$$= EI \left(\frac{n\pi}{L}\right)^4 \cdot \frac{L}{2} \tag{10.9b}$$

10.4.5. Modelling pedestrians

A single pedestrian can be modelled as a moving harmonic point load. The harmonic part of the loading is described by a Fourier-series, taking into account the first two harmonics only (see chapter 3). The walking frequency is represented by f_i, $t_{s,i}$ is the moment in time the pedestrian enters the bridge, ψ_i is the phase shift indicating the position of the human body and legs at the moment the pedestrian enters the bridge. α_1 & α_2 are the Fourier coefficients determining the influence of the static weight of the pedestrian to the dynamic loading. The pedestrian loading should only be taken into account for the position at the bridge where the pedestrian is at a particular moment in time and only for the time the pedestrian is actually crossing the bridge. The position of the bridge is modelled with the Dirac delta function $\delta(x)$, for which the position is described by the velocity v_i times the time the pedestrian is present at the bridge $(t - t_{s,i})$. The time the pedestrian is crossing the bridge is modelled with two Heaviside step functions $(H(t - t_{s,i}) - H(t - t_{e,i}))$, with $t_{e,i}$ the moment in time the pedestrian leaves the bridge, calculated by the velocity of that single pedestrian times the length of the bridge minus the moment in time the pedestrian entered the bridge. $\psi_{p,1}$ and $\psi_{p,2}$ are the mutual phase shifts between the harmonics. The phase shift of the pedestrian (of the total force of the pedestrian) is represented by ψ_i. The total pedestrian loading on the footbridge is the summation of the loading of all the individual pedestrians who cross the bridge, resulting in equation 10.10:

$$\begin{aligned} p(x,t) = \sum_{i=1}^{Np} P_i \cdot \big(\alpha_1 \sin\left(2\pi f_i (t - t_{s,i}) + \psi_{p,1} + \psi_i\right) + \alpha_2 \sin\left(2\pi 2 f_i (t - t_{s,i}) + \psi_{p,2} + 2\psi_i\right)\big) \\ \cdot \delta\left(x - v_i (t - t_{s,i})\right) \cdot \left(H(t - t_{s,i}) - H(t - t_{e,i})\right) \end{aligned} \tag{10.10}$$

The phase shifts $\psi_{p,1}$ and $\psi_{p,2}$ and assumed to be zero.[14] In order to be able to use equation 10.6 the modal pedestrian loading P_n^* has to be determined as well. This modal load can be determined with the following equation (for derivation see appendix B):

$$P_n^* = \int_0^L p(x,t) \cdot \phi_n(x) dx \tag{10.11}$$

This results in the following modal loading for simply supported bridges with mode shapes $\phi_n(x) = \sin\left(\frac{n\pi x}{L}\right)$:

$$P_n^*(t) = \sum_{i=1}^{Np} P_i \cdot \left(\alpha_1 \sin\left(2\pi f_i(t - t_{s,i}) + \psi_i\right) + \alpha_2 \sin\left(2\pi 2 f_i(t - t_{s,i}) + 2\psi_i\right)\right) \cdot \sin\left(\frac{n\pi v_i(t - t_{s,i})}{L}\right) \cdot \left(H(t - t_{s,i}) - H(t - t_{e,i})\right) \tag{10.12}$$

10.4.6. Calculation of the acceleration signal

Solving the equation of motion 10.2 with the proposed modal bridge properties and modal pedestrian loading, per mode shape the deflection $u_n(t)$ can be calculated for the normative position at the bridge. The accelerations per mode shape can now be determined by differentiation of the deflection to time twice:

$$a_n = \frac{\mathrm{d}^2 u_n(t)}{\mathrm{d}t^2} \tag{10.13}$$

The accelerations a pedestrian experience while crossing the footbridge (a_i) can now be calculated by a summation of the accelerations per mode shape (a_n) for the time the pedestrian crosses the bridge times the mode shape that has been taken into account.

$$a_i = \sum_{n=1}^{Nm} a_n(t) \cdot \sin\left(\frac{n\pi v_i(t - t_{s,i})}{L}\right) \cdot \left(H(t - t_{s,i}) - H(t - t_{e,i})\right) \tag{10.14}$$

The accelerations of the bridge deck $a_n(t)$ at the governing position include the vibrations caused by all the pedestrian during the time they are present at the bridge and the free vibrations as a result of the pedestrians that left the bridge. Figure 10.5 gives a representation of the calculation of the modal response for the first mode shape of a simply supported bridge.

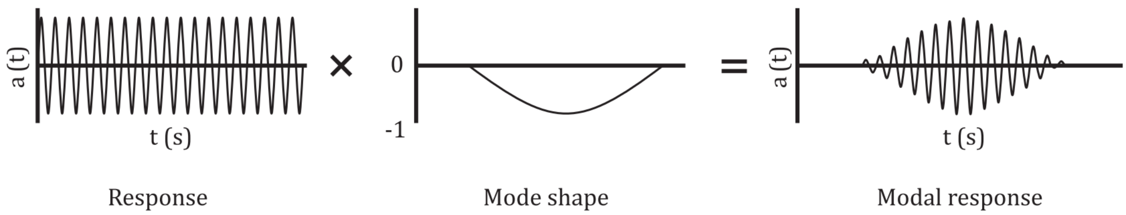


Figure 10.5: Result modal response: schematic representation of the accelerations a pedestrian experiences when crossing the bridge

10.5. Human comfort to vertical vibrations

"Human vibration is a multi-disciplinary subject involving knowledge from disciplines as diverse as engineering, ergonomics, mathematics, medicine, physics, physiology, psychology and statistics." (Griffin) [66]

Comfort or discomfort is a wide concept and can be affected by various aspects as presented by Griffin in figure 10.6. To assess the (dis)comfort of people using footbridges, different documents can be consulted. The regulations valid in the Netherlands will be discussed and relevant findings in literature related to other fields of study will be reviewed.

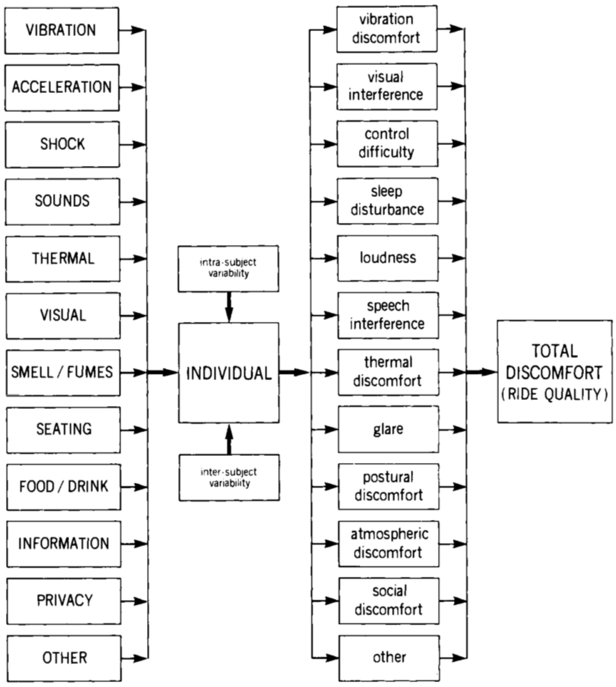


Figure 10.6: Factors contributing to discomfort according to Griffin [66]

10.5.1. Principles

The human comfort for footbridges depends on the vibrations of the structure. In this study linear vibrations in the vertical direction are considered and therefore the comfort evaluation of walking persons in the vertical direction is of interest. The frequency range of the vibrations that is relevant for the evaluation of the comfort due to human-induced vibrations is the range of walking frequencies of pedestrians. This is roughly the frequency range from 1 to 5 Hz.

Motion sickness (with symptoms like sweating, nausea or vomiting) is caused by vibrations below a frequency of 0.5 Hz.[66] This is outside the range of the vertical vibrations caused by pedestrians and is therefore not taken into account in this work.

Vibration is an oscillatory motion. Different types of oscillatory motion can be specified (figure 10.7). In general in the evaluation of the vibrations the signal is expressed in accelerations, because the instrumentation for measuring the acceleration is generally more convenient than for measuring the velocity.[66] The acceleration which a single pedestrian experiences when crossing a bridge is described by a transient oscillatory motion (figure 10.8), while for a number of pedestrians crossing the bridge the accelerations they experience resembles more a random motion (figure 10.9).

10.5.2. Measurement of vibrations

It is an open question whether humans can perceive a single peak acceleration. To be able to analyse the acceleration signal to which pedestrians are exposed when crossing a footbridge it is useful to know what type of acceleration signal people perceive and to what extent this is causing an uncomfortable feeling. The

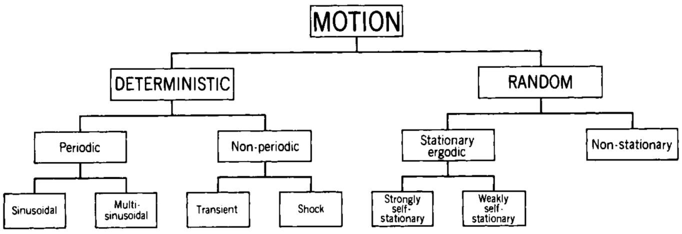


Figure 10.7: Categorization of types of oscillatory motion [66]

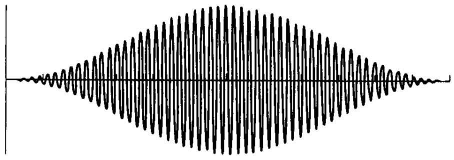


Figure 10.8: Transient oscillatory motion: general acceleration signal a single pedestrian is exposed to when crossing a bridge [66]

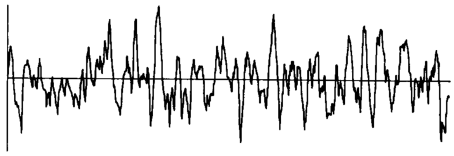


Figure 10.9: Random oscillatory motion: general acceleration signal a flow pedestrians is exposed to when crossing a bridge [66]

documents that are relevant to use in the evaluation of vibrations in the Netherlands have been evaluated, based on the measurement method for (dis)comfort.

EN 1990 & National Annex

The EN 1990 [2] and the corresponding National Annex (NA) [4] are part of the Eurocodes, European standards for the evaluation of the structural safety of all possible (building) structures. In these codes the dynamic behaviour of footbridges is covered. The degree of comfort for pedestrians has to be evaluated based on the maximum occurring acceleration of the governing part of the bridge deck. This maximum acceleration is calculated for the 95% non-exceedance level.

EUR 23984 EN

EUR 23984 [6] is the guideline "Design of Light Weight Foot Bridges for Human Induced Vibrations", published by the Joint Research Centre. This guideline purely focuses on footbridges. The degree of comfort has to be assessed based on the maximum occurring acceleration of the bridge deck, similar to the approach of EN 1990 & NA.

ISO 2631-1

ISO 2631-1 [67] is a standard for mechanical vibration and shock for the evaluation of the human exposure to whole-body vibration. The recommended method for the evaluation of vibrations that is proposed depends on the crest factor:

$$\text{Crest factor} = \frac{\text{maximum acceleration}}{\text{root mean square acceleration}} \tag{10.15}$$

For a crest factor smaller than nine the root mean square of the weighted acceleration ($a_{w,rms}$) should be used as a measure for comfort.

$$a_{w,rms} = \left[\frac{1}{T} \int_0^T a_w^2(t)dt \right]^{\frac{1}{2}} \tag{10.16}$$

In this formula $a_w(t)$ is the weighted acceleration as a function of time in m/s^2 and T is the duration of the measurement in seconds. The duration of measurement should be sufficient to ensure reasonable statistical precision and to guarantee that the vibration is typical for the exposures which are being assessed.

In case the crest factor is higher than nine two methods can be applied: (i) the running root mean square ($a_{w,rms}(t_0)$ - equation 10.17) that takes into account random shocks by using an integration time of one second and (ii) the fourth power vibration dose method (VDV - equation 10.18) which is more sensitive to peaks.

$$a_{w,rms}(t_0) = \left(\frac{1}{\tau} \int_{t_o-\tau}^{t_0} [a_w(t)]^2 \, dt \right)^{\frac{1}{2}} \tag{10.17}$$

$$VDV = \left(\int_0^T [a_w(t)]^4 \, dt \right)^{\frac{1}{4}} \tag{10.18}$$

The instantaneous frequency-weighted acceleration is represented by $a_w(t)$, τ is the integration time for running averaging, t_0 is the time of observation (instantaneous time) and T is the duration of measurement.

ISO 10137

ISO 10137 [68] is a standard giving the bases for the design of structures, specified for the serviceability of buildings and walkways against vibrations. This standard uses the crest factor as well as a criteria for the method of assessment. The weighted root mean square acceleration is used as a measure of vibration with a recommended averaging time of one second in case the crest factor is smaller than six. Otherwise, the vibration dose value should be used for evaluation.

Weighting of the acceleration

Both the ISO standards 2631-1 and 10137 use the frequency weighted acceleration for the evaluation of the vibrations. The frequency weighting curve has to be applied for the principal weightings. The curve that has to be used can be determined according to the table in the ISO 2631-1 standard [67]. The acceleration will be evaluated for comfort in the z-axis (standing). Therefore the W_k frequency weighting curve has to be used. The frequency weighted acceleration can be determined with the following formula:

$$a_w = \left[\sum_i (W_i a_i)^2 \right]^{\frac{1}{2}} \tag{10.19}$$

In this formula a_w is the frequency-weighted acceleration, W_i is the weighting factor and a_i is the r.m.s. acceleration. The graph of the frequency weighting is presented in the appendix at figure E.2, with the corresponding table for principal frequency weightings in figure E.3.

10.5.3. (Dis)comfort criteria

Human (dis)comfort is hard to define exactly. Besides vibration, other aspects like the location and the personal experience of a person have an impact as well. In the above-mentioned regulations only acceleration related criteria are considered for the evaluation of (dis)comfort. These criteria are discussed below. Additional aspects that have an impact on human comfort will be discussed in the next subsection based on literature and other fields of application.

EN 1990 and National Annex

The maximum allowed accelerations of a random part of the bridge deck for the vertical direction according to EN 1990 and the corresponding National Annex is 0.7m/s^2, as presented in subsection 4.1.2. Furthermore it is advised to make use of the EUR 23984.

EUR 23984

As presented in figure 4.5 in subsection 4.2.4 the EUR 23984 takes into account four comfort classes, both for the vertical and the lateral direction. These comfort classes are the result of three tests performed in the past[14]:

- Tests by the RWTH Aachen at the Kochenhofsteg bridge in Stuttgart;
- Tests by the RWTH Aachen at the Wachtelsteg bridge in Pforzheim;
- A study by FEUP Porto.

The results of the study by FEUP Porto are presented in figure 10.10, expressed in the maximum bridge response in g (gravity). The comfort classes are based on these values, in which a difference is made between maximum, medium, minimal and unacceptable comfort. Although the results are presented as average indicators, it is not clear if a certain probability of exceedance in the translation to the comfort classes has been taken into account.

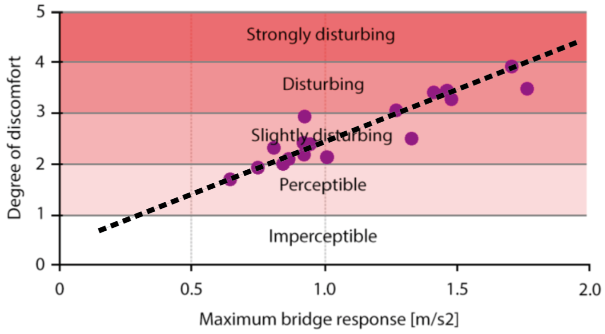


Figure 10.10: Result of the average indicators of the inquiries by FEUP Porto concerning human discomfort induced by vibrations [14]

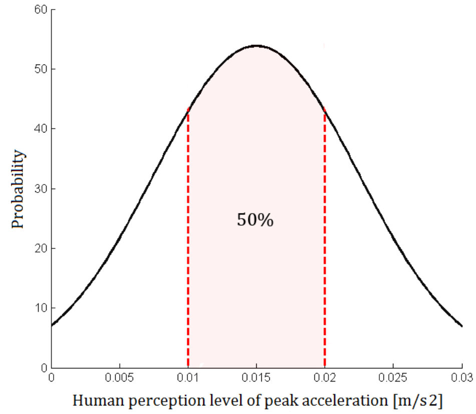


Figure 10.11: Distribution of the human perception level of peak acceleration with the interquartile range according to the ISO 2631-1

ISO 2631-1

Criteria for comfort are not presented in the ISO 2631-1. A threshold for the perception of vibrations is presented at a weighted r.m.s. acceleration of 0.015 m/s^2, for which the interquartile range due to the wide variety of perception among individuals lies between 0.01 and 0.02 m/s^2. This is visualized in figure 10.11. Furthermore a remark is made indicating there is a difference in the maximum allowed vibrations for comfort depending on the application.

ISO 10137

According to ISO 10137 walkways have to be designed in such a way that the vibration amplitudes do not alarm the potential users. To achieve this, the vibrations of the walkways may not exceed the value of the applicable base curve for the considered direction multiplied by 60. A distinction is made between walking and standing persons: in case one or more persons are standing still on the walkway a multiplying factor of 30 should be applied. The base curve for the accelerations in the vertical direction for the z-axis (standing persons) is shown in figure 10.12. At a frequency of 2 Hz the value of the base curve is 0.007 m/s^2, resulting in an acceleration criterion of 0.42 m/s^2 for the normal use of walkways and 0.21 m/s^2 in case of people standing still. This underlines that the multiplication factor of 60 is related to walking people. Unlike the other presented documents, this standard makes a distinction in the use of walkways instead of presenting a uniform criterion.

Overview applied measurements and criteria

In table 10.1 an overview of the described measurement methods and acceleration criteria for the evaluation of human comfort with respect to vibrations in footbridges is presented.

Table 10.1: Overview measurement methods and acceleration criteria for the evaluation of human comfort with respect to vibrations in footbridges

Document	Measurement	Duration	Criteria	
EN 1990 [4]	a_{max}	not specified	0.7 m/s^2	
EUR 23984 [6]	a_{max}	not specified	Comfort class 1	< 0.5 m/s^2
			Comfort class 2	0.5 - 1.0 m/s^2
			Comfort class 3	1.0 - 2.5 m/s^2
			Comfort class 4	> 2.5 m/s^2
ISO 2631-1 [67]	$a_{w,rms}$	representative	none	
ISO 10137 [68]	$a_{w,rms}$	1s	Normal use	Base curve * 60
			Standing persons	Base curve * 30

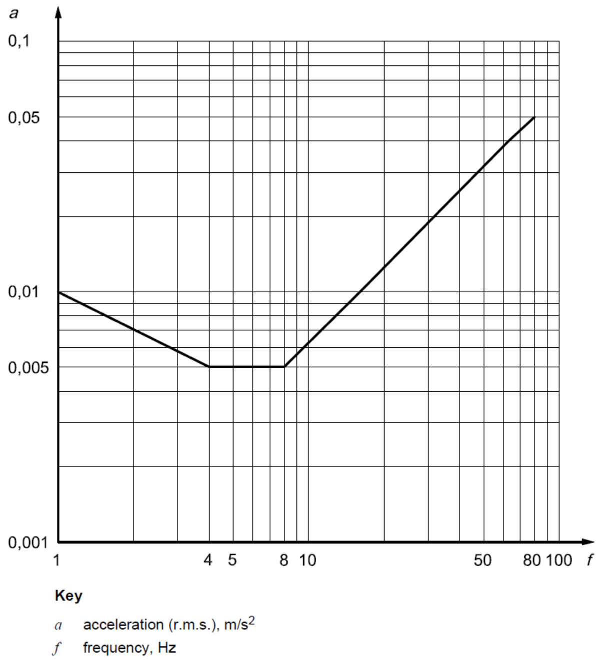


Figure 10.12: Base curve for acceleration in the vertical direction according to ISO 10137 [68]

10.5.4. Additional relevant aspects

"Whether a motion causes annoyance, discomfort, interference with activities, impaired health or motion sickness depends on many factors - including the characteristics of the motion, the characteristics of the exposed person, the activities of the exposed person and other aspects of the environment."[66]

Human comfort in relation to vibrations is, apart from external parameters, depending on the person itself. Moreover, large deviations in the response of an individual at different events can be expected. A single value for the maximum acceleration to quantify discomfort is not in accordance with the number of steps made in the improvement of the structural design and knowledge of dynamic behaviour and loading. To gain more insight in the evaluation of comfort in relation to human-induced vibrations, regulations for other fields of application and papers on the comfort of footbridges have been studied.

Type of walking

Hawryszków published a relation (figure 10.13) between the maximum accelerations and the evaluation of comfort on a scale from 0 to 7. A difference between walking, running and sprinting people is presented. In the graph it can be seen that the same acceleration causes more discomfort for someone walking then for a person who is running or sprinting. The gradual scale that is used for this evaluation is based on four different descriptions: vibrations, feelings, description of vibrations for moving persons and description of vibrations for standing or sitting persons (see figure E.1 in appendix E).[16]

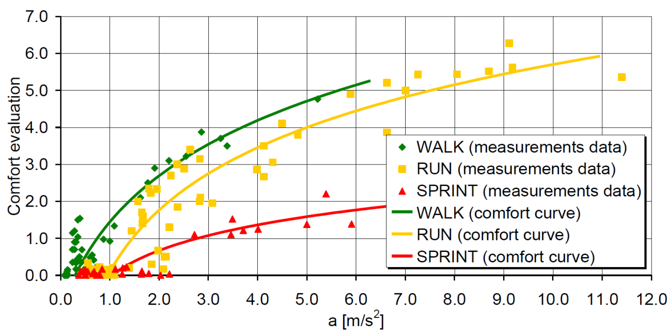


Figure 10.13: Comfort curves and areas, adjusted based on survey data by Hawryszków [16]

Duration of exposure

At the moment the duration of exposure to vibrations is taken into account in the different measurement methods, but not when it comes to setting criteria. One single high peak value of the acceleration could for example give another level of comfort than a longer exposure to this same peak value. For the application in buildings a relation between the comfort zone and the duration of exposure is presented in the ISO 2631-1 by two studies of health guidance caution zones. These values are based on the experienced level of comfort for sitting persons. As can be seen in figure 10.14 the comfort level stays constant for the first 10 minutes for the method 1, but is decreasing linearly for the method 2. It must be mentioned that in general the duration of exposure for footbridges is short compared to exposure to vibrations in buildings. Assuming a walking speed of 5 km/h, for a person to be exposed for more than 10 minutes to a vibration the bridge has to be at least 833 m.

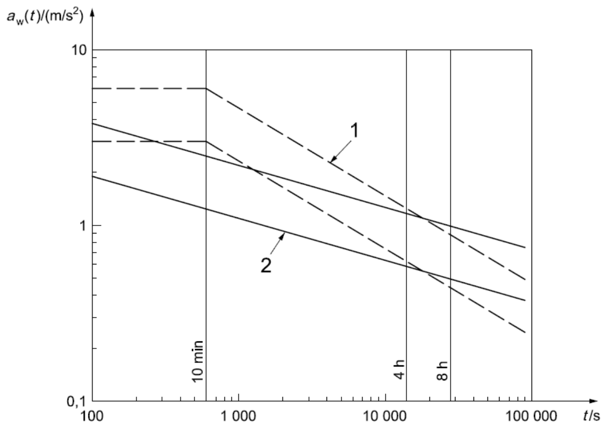


Figure 10.14: Health guidance caution zones for buildings according to ISO 2631-1 [67]

Situation dependent comfort criteria for footbridges

In addition to accelerations, aspects like the situation and characteristics of a bridge are expected to have an influence on the comfort level as well. For example: a small vibration on a bridge at a height of 100 m is likely to give a less comfortable feeling than the same amplitude of vibration on a bridge at a height of 2 m. To tackle this problem Mackenzie et al suggested the following formula [69]:

$$a_{limit} = 1.0 \cdot k_1 \cdot k_2 \cdot k_3 \cdot k_4 \tag{10.20}$$

The values for k_1 to k_3 can be taken from figure 10.15, depending on the site used, route redundancy and height of the structure. The value k_4 is the exposure design factor and must be taken 1.0 unless otherwise agreed between the designer and authority.

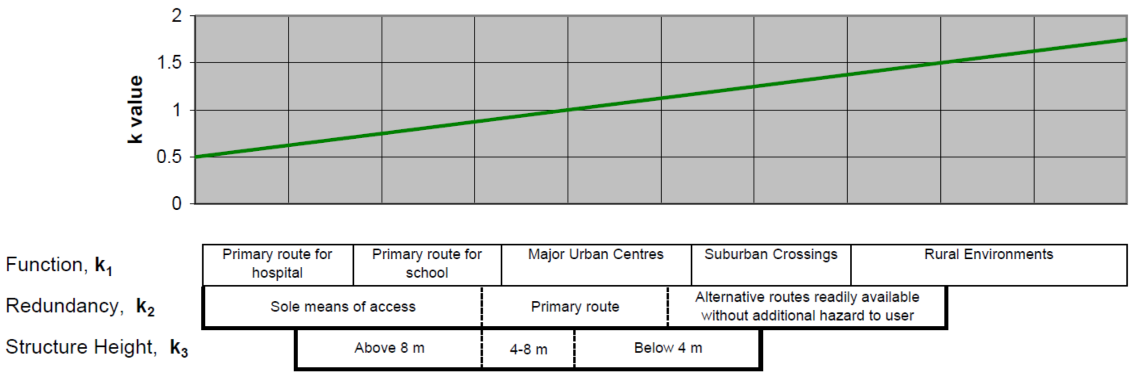


Figure 10.15: Proposed response modifiers for the comfort level of pedestrians at a footbridge for equation 10.20 [69]

Situation dependent comfort criteria for buildings

ISO 10137 provides a more extensive determination of the comfort criteria for human response to vibration in the case of buildings. A similar procedure as for walkways with the use of a base curve in combination with multiplication factors is used, but in addition the multiplication factors are specified more extensively. The table in figure 10.16 indicates a difference in the multiplication factors between day & night, continuous & impulsive vibration and depending on the location.[68] In this table the continuous vibrations have a duration of more than 30 minutes per 24 hours and the intermittent vibrations exist of more than 10 events per 24 hours. The difference between continuous and impulsive vibrations is significant and suggests that a person in sitting position is less affected by a single peak acceleration than by a sustained vibration.

Place	Time	Multiplying factors to base curve (Figures C.1, C.2 and C.3) [a]	
		Continuous vibration and intermittent vibration [b]	Impulsive vibration excitation with several occurrences per day
Critical working areas (e.g. some hospital operating-theatres, some precision laboratories, etc.)	Day	1	1
	Night	1	1 [c]
Residential (e.g. flats, homes, hospitals)	Day	2 to 4 [d]	30 to 90 [d, e, f]
	Night	1,4	1,4 to 20
Quiet office, open plan	Day	2	60 to 128 [g]
	Night	2	60 to 128
General office (e.g. schools, offices)	Day	4	60 to 128 [g]
	Night	4	60 to 128
Workshops [h]	Day	8	90 to 128 [g]
	Night	8	90 to 128

Figure 10.16: Multiplying factors used in several countries to specify satisfactory magnitudes of building vibration with respect to human response [68]

10.5.5. Discussion

Considering the r.m.s. evaluation of the accelerations for the time the pedestrian is at the bridge, leads to the fact that it is only possible to determine whether a pedestrian felt uncomfortable or not at the moment the pedestrian leaves the bridge. During his walk across the bridge, it is not possible to know if at the end the r.m.s. of the total acceleration signal will exceed the limit value. In reality, it is expected that at the moment the pedestrian is at the bridge and the accelerations are too high the pedestrian will already feel uncomfortable. Therefore, it seems more realistic to use the r.m.s. of the total acceleration signal from the moment the pedestrian steps onto the bridge till the position of the pedestrian at the bridge at the moment of evaluation.

Furthermore Zivanovic shows the relevance of using different comfort levels: "The bridge should be designed in such a way that the most frequent events cause acceptable vibrations for most people, while the vibration level could be allowed to be higher for the events that rarely occur."[27] In addition to the previous presented relevant parameters it would be recommended to select per structure different comfort levels for different expected situations. The different comfort levels of the EUR 23984 underline this as well.

From contact with Dr.-Ing. Christoph Heinemeyer about the foundation of the comfort criteria of the EUR 23984 it emerged that, beside vibrations, one factor seemed to have the greatest impact: expectation. This raises another question, whether it is a good idea or not to make people aware of the possibility of vibrations in footbridges.

10.6. Probability of (dis)comfort and acceptance

10.6.1. Principles

Discomfort will be reached when the accelerations a pedestrian is exposed to are higher than the limit value for comfort of that pedestrian. This probabilistic approach is based on the assumption that if the bridge deck will at one moment reach the limit value of the accelerations, this does not imply that all the pedestrians who crossed the bridge experienced this limit value. In addition, it can be part of the choice of the client what is acceptable for the number or percentage of people that may experience this limit level. This can for example be depending on the use of the bridge, from normal traffic to a special event. Therefore, it is not relevant whether the accelerations which a specific pedestrian is exposed to reach the limit level. It is of concern what the probability is that this limit level will be reached. This probability of an uncomfortable feeling can be determined by combining the probability distribution of the expected accelerations and the probability distribution of the criterion for comfort.

10.6.2. Probability distribution of accelerations

The accelerations each individual pedestrian within a traffic flow is exposed to, need to be evaluated according to the measurement methods for comfort discussed in section 10.5. By combining the evaluated acceleration signals a probability distribution of the expected accelerations for a certain traffic density can be established. A significant number of pedestrians should be evaluated to get a reliable distribution. The probability distribution of the expected accelerations for the scenarios based on group formation can be determined immediately by evaluating the accelerations of the pedestrians within a flow. For the scenarios based on traffic densities the probability distributions for the relevant traffic densities have to be combined by use of a mixture model.

Each scenario based on densities exists of a probability mass function to describe the probabilities of these expected traffic densities. The probability mass function can be combined with the probability distributions for the specific densities that are taken into account. This evaluation of a scenario can be realised by the use of a mixture model, with the mathematical expression as equation 10.21.

$$f(a) = \sum_{d=0.1}^{D} \lambda_d \cdot f_d(a) \tag{10.21}$$

The implementation of this equation for the scenarios is shown by an example in figure 10.17 for one density. The result of this mixture model is a probability distribution of the accelerations pedestrians are expected to experience for each scenario, based on the summation for the considered densities of the probabilities per densities times the corresponding probability distribution of the accelerations.

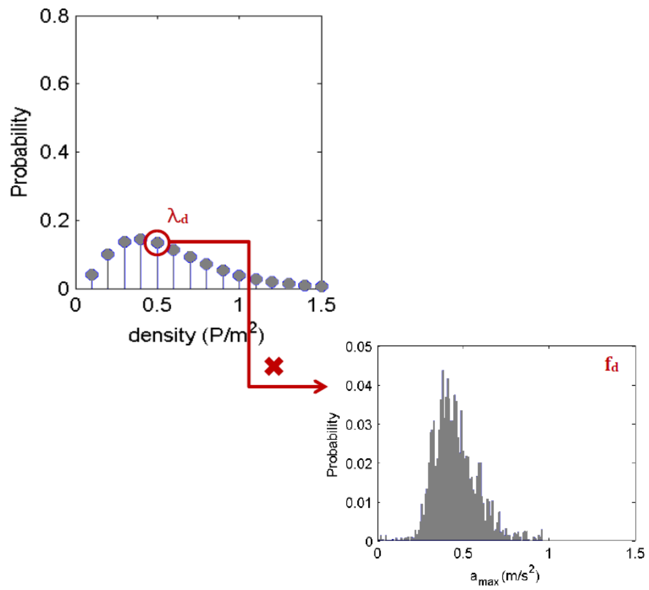


Figure 10.17: An example of the procedure of the mixture model for one density

10.6.3. Probability distribution for human (dis)comfort

The probability distribution for the expected accelerations that a pedestrian is exposed to, should be combined with the probability distribution for human (dis)comfort, in order to obtain the probability of failure (in this case (dis)comfort). Instead of the expected distribution, in the regulations fixed values for the comfort criteria are provided (see table 10.1). The probability a person feels uncomfortable at the acceleration level corresponding to the comfort criterion is not presented. Therefore it is assumed that exceeding this acceleration level will definitely result in discomfort. For a certain limit value, the probability of (dis)comfort can now be determined by considering the change that the limit value is exceeded. The probability a pedestrian experiences a maximum acceleration that exceeds the limit value for a specific bridge and scenario can be expressed as follows:

$$P(a_{max} > \text{limit value m/s}^2 \mid \text{scenario \& bridge}) \tag{10.22}$$

11

Case study

The approach described in chapter 10 is applied in this case study for three simply supported footbridges. These are the same bridges as used in the first part of this report with a length of 12, 50 and 100 m, having their first natural frequency within the critical range of the walking frequency of pedestrians. Figure 11.1 gives a schematic representation of the three bridges. The result of the case study is discussed in the chapter 12.

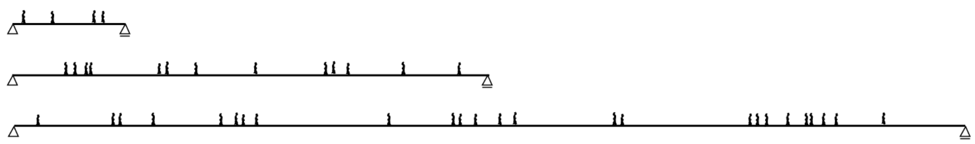


Figure 11.1: Schematic representation of the three bridges with length 12, 50 and 100 m that are used for the case study

11.1. Pedestrian traffic

Based on the proposed approach in section 10.3 a more realistic approximation of the expected normal pedestrian behaviour is made by use of scenarios for the expected pedestrian traffic.

11.1.1. Applied scenarios

For each situation different pedestrian traffic is expected. To be able to evaluate the pedestrian traffic and the accelerations for different situations, four scenarios are introduced. Each scenario describes a location in combination with the expected type of pedestrian traffic:

- *Commuter traffic* - Footbridge in the city used by people going to and coming back from work or school, especially in the morning and afternoon.
- *Shopping area* - Footbridge within a shopping area (inside or outside), mostly used at late night shopping and in the weekend by a lot of people walking around in pairs.
- *Train station* - Footbridge located at a train station, where large groups with a high density of people cross the bridge, but in the remaining time the bridge is not used.
- *Park* - Footbridge located in a park, used mostly during the weekend when people visit the park for a quiet walk, alone or in a small group.

11.1.2. Scenarios based on densities

First an estimation of the expected amount of pedestrian traffic for the four scenarios is made based on traffic densities. For each scenario a probability mass function (PMF) for the expected probabilities for certain densities is set up. The determination of these PMFs is made as good as possible with use of the limited data available ([70], [71], [72], [73], [74] and [75]) on pedestrian traffic. These datasets are expressed in a number of people per hour. Information on the distribution of the pedestrians within a flow is not available in the datasets. Therefore this data has to be transformed into densities per hour, independent of the length of the bridge.

The datasets for the four scenarios in densities per hour are transformed into probability density functions (PDFs). Use is made of the program EasyFit to find the best fit between a type PDF and the dataset for a scenario. These PDFs are transformed into probability mass functions. For the first and fourth scenario (the commuter traffic and the park situation), it is assumed the overall traffic density at the bridge will never exceed 0.3 P/m^2. The other two scenarios, the train station and the shopping area, are expected be exposed to higher densities. Therefore a division in densities is made up to a density of 1.5 P/m^2, comparable to the highest traffic density (TC5) used in the EUR 23984. For all four scenarios fifteen different traffic densities are taken into acocunt. The result of the PMFs for the four scenarios is shown in the figures 11.4, 11.6, 11.8 and 11.10.

Crowd modelling within a density

Traffic density is defined in people per square meter: P/m^2. To model the different densities that have to be taken into account, the distance between the persons has to be determined in such a way that the global required density will be met. Within a pedestrian flow a certain group formation can be expected, instead of a constant density built up from single pedestrians. Therefore the assumption is made that pedestrians walk in small groups. Group formation for humans is a study in itself which includes a social factor as well. For a detailed description and studies of the modelling of pedestrian crowds one is referred to the literature (see e.g. [76], [77],[78] and [79]).

Instead of a crowd with a constant density made up of single pedestrians, a more realistic approach is used by modelling a crowd with a certain global density formed by small groups of pedestrians. Based on data obtained during a sample counting test at the Rijnhavenbridge in Rotterdam (with a total number of pedestrians of 615) performed by the author, an estimate for a distribution of the group sizes within a large flow has been made. The result is plotted in figure 11.2.

To model these small groups of pedestrians within the large flow, the assumption is made that the density within such a group is 1.0 P/m^2. This density can be used to determine the distance between the groups that should on average be reached, so that the overall density of the flow will be met. The distance is applied in the model by use of a time interval, calculated by the required distance divided by the average velocity of the pedestrians. It is expected that the groups of pedestrians are not uniformly distributed within a flow. Therefore the time interval between the small groups in a flow is assigned as a random variable from a normal distribution, using the necessary calculated time interval as both the mean value and the standard deviation of this distribution.

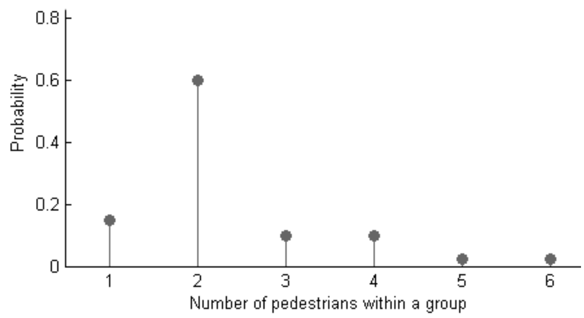


Figure 11.2: Probability mass function of the group size of the pedestrians within a flow, based on data obtained during a sample counting test at the Rijnhavenbridge in Rotterdam performed by the author

Figure 11.3: Scenario 1 - Commuter traffic

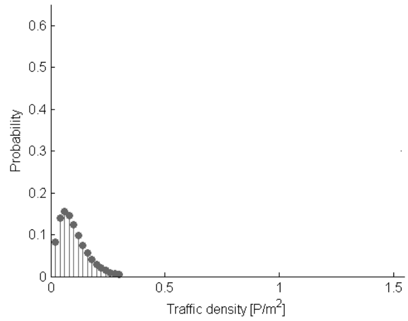


Figure 11.4: Probability mass function for scenario 1

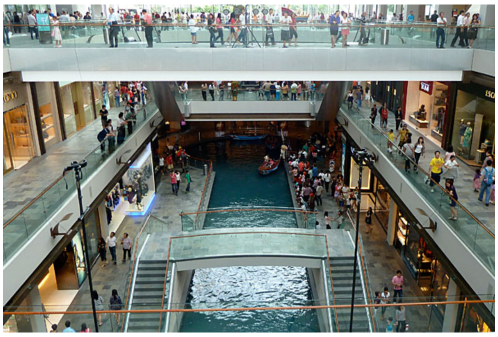


Figure 11.5: Scenario 2 - Shopping area

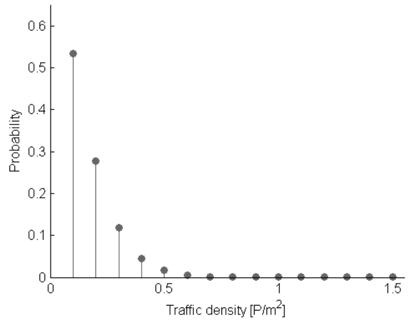


Figure 11.6: Probability mass function for scenario 2

Figure 11.7: Scenario 3 - Train station

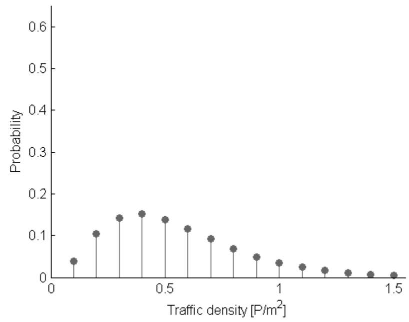


Figure 11.8: Probability mass function for scenario 3

Figure 11.9: Scenario 4 - Park

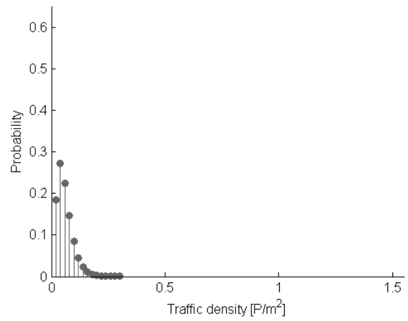


Figure 11.10: Probability mass function for scenario 4

Crowd densities

The densities that are taken into consideration in the model are dependent on the densities assumed in the proposed scenarios. Therefore two ranges of densities are considered. For the first and the fourth scenario the densities are within a small range: 0.02 to 0.3 P/m^2 with a step size of 0.02 P/m^2. For the other two scenarios a larger range from 0.1 to 1.5 P/m^2 with a step size of 0.1 P/m^2 is assumed. Therefore in total 27 densities will be evaluated. The figures 11.11 to 11.15 provide examples of the crowd densities that are used in the calculations, based on the assumptions made for the crowd modelling. The figures resemble a snapshot (top view) of five different pedestrian flows.

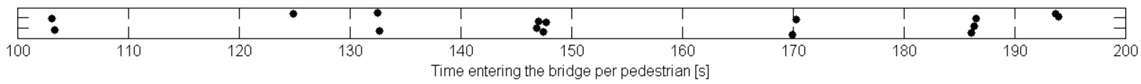


Figure 11.11: Example of the top view of a pedestrian crowd with a density of 0.04 P/m^2

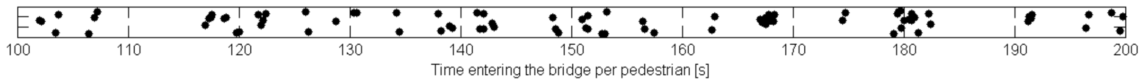


Figure 11.12: Example of the top view of a pedestrian crowd with a density of 0.2 P/m^2

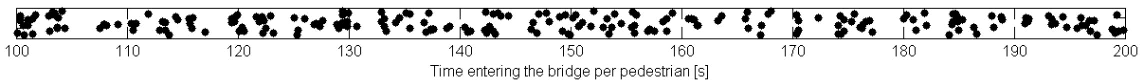


Figure 11.13: Example of the top view of a pedestrian crowd with a density of 0.5 P/m^2

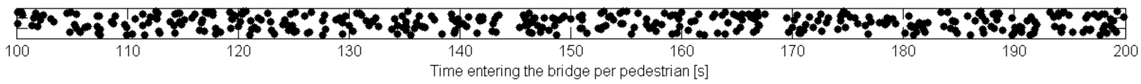


Figure 11.14: Example of the top view of a pedestrian crowd with a density of 1.0 P/m^2

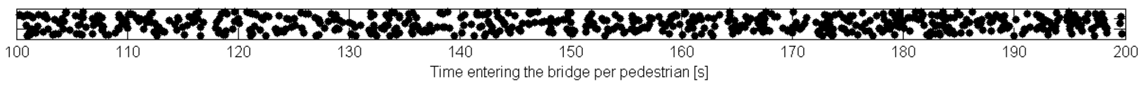


Figure 11.15: Example of the top view of a pedestrian crowd with a density of 1.5 P/m^2

11.1.3. Scenarios based on group formation

In the previous method (scenarios based on densities) pedestrian traffic is modelled according to a certain density based on a number of people per square meter, in line with the approach of the current guidelines. In reality, pedestrian traffic is expected to be more divergent built up from different types of groups and time intervals with a large diversity. Sahnaci and Kasperski underlined this effect in a paper presented at the EURODYN 2014. They assure the number of persons per second entering the footbridge is not a stationary process. Therefore they recommend to consider groups of pedestrians with different sizes for the expected pedestrian traffic.[65]

Therefore in addition to the scenarios based on densities, the same four scenarios are also set up based on group formation. To describe a pedestrian flow based on group formation, the following three parameters have to be determined:

1. *Group sizes* - What kind of group sizes are expected to be present for each scenario?
2. *Time interval* - What is the expected distance (expressed in time) between the groups to represent each scenario?
3. *Group density* - What is the expected density within a group for each scenario?

Because no extensive datasets of actual pedestrian traffic on footbridges are available, the parameters used in this case study are based on an as good as possible assumption of the actual pedestrian traffic. The assumed sizes of the groups are with the corresponding expected probabilities of occurrence are plotted for each scenario in figures 11.16, 11.17, 11.18 and 11.19 as a probability mass function. For each scenario the parameters for the assumed average time interval and density within a group are shown in table 11.1.

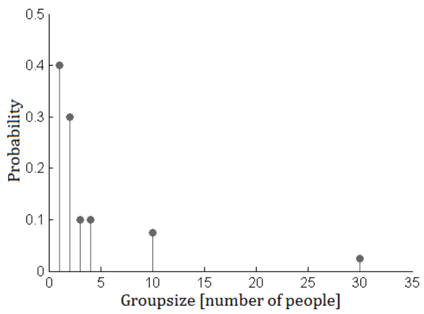


Figure 11.16: PMF scenario 1

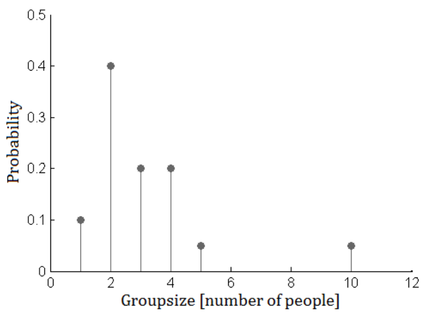


Figure 11.17: PMF scenario 2

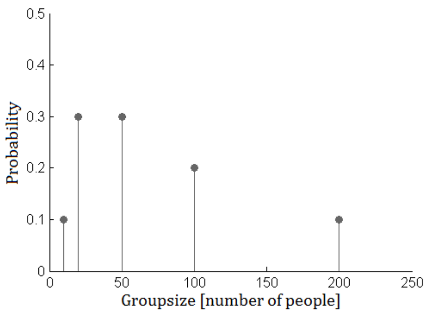


Figure 11.18: PMF scenario 3

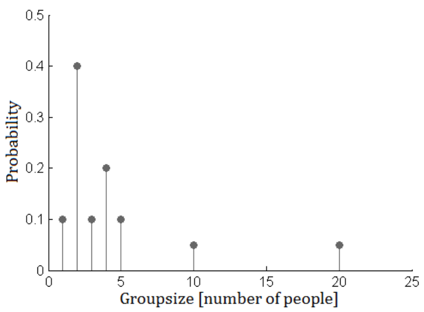


Figure 11.19: PMF scenario 4

Table 11.1: Time interval and group size for the scenarios based on group formation

Scenarios	Time interval	Group density
1	20 s	0.5 P/m^2
2	10 s	1.0 P/m^2
3	60 s	1.5 P/m^2
4	300 s	0.25 P/m^2

In line with the scenarios based on densities, the time interval between the groups for these scenarios based on group formation is applied as a random variable. This variable is determined by a normal distribution with both the mean and the standard deviation equal to the expected average time interval. Based on these three parameters for each scenario a random crowd is determined for a chosen model time by use of the program Matlab. In the figures 11.20, 11.21, 11.22 and 11.23 examples of the results of the pedestrian flows (top view) for the four different scenarios are presented.

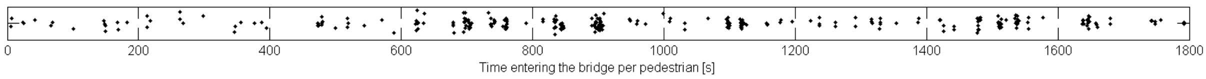


Figure 11.20: Example of a pedestrian flow for scenario 1 during 1800 seconds

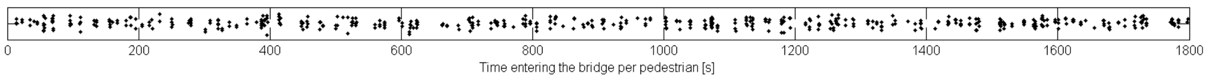


Figure 11.21: Example of a pedestrian flow for scenario 2 during 1800 seconds

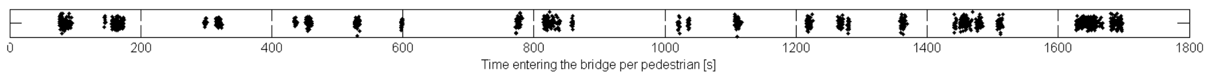


Figure 11.22: Example of a pedestrian flow for scenario 3 during 1800 seconds

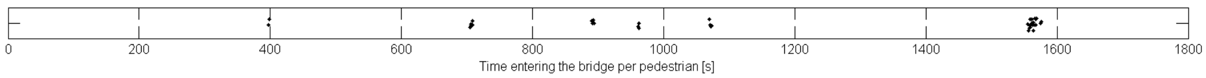


Figure 11.23: Example of a pedestrian flow for scenario 4 during 1800 seconds

11.2. Acceleration signal individual pedestrians

The acceleration signal for individual pedestrians can be calculated by solving the differential equation 10.6 presented in section 10.4 for the relevant mode shapes with use of the program Matlab.

11.2.1. Modelling bridge structure

For the properties of the bridges reference is made to part I, table 6.1, 6.2 and 6.3 for respectively a length of 12, 50 and 100 m. To summarize: the bridges are simply supported with mode shapes $\phi_n(x) = \sin\left(\frac{n\pi x}{L}\right)$ and the first natural frequency in the critical range of the walking frequency of pedestrians (1.8 Hz) with constant structural properties along the length.

Number of mode shapes

The mode shapes with a natural frequency in the critical range of the walking frequency of pedestrians have to be taken into account in the calculations. The natural frequencies of a sine mode shape are calculated with the following formula:

$$f_n = \frac{1}{2\pi}\sqrt{\frac{K_n^*}{M_n^*}} \tag{11.1a}$$

$$= \frac{1}{2\pi}\left(\frac{n\pi}{L}\right)^2\sqrt{\frac{EI}{m}} \tag{11.1b}$$

This results in the following relation between the natural frequencies of the different mode shapes:

$$f_n = n^2 \cdot f_1 \tag{11.2}$$

The natural frequency f_1 of the first mode shape for the bridges taken into account is 1.8 Hz. Therefore the second natural frequency will be 7.2 Hz, the third 16.2 Hz, etc. Except for the first natural frequency all the frequencies are outside of the critical range. The second natural frequency is close to the second harmonic of the pedestrian load. Therefore, in addition to the first natural frequency, this frequency is taken into account in the calculations, although it is expected that the contribution is negligible.

Modal bridge parameters

For the assumption that the mass, damping and stiffness of the bridge are constant. In combination with a sine mode shape, the modal bridge parameters can be determined by use of equations 10.7, 10.8 and 10.9.

11.2.2. Modelling pedestrian loading

A flow of pedestrians consists of combinations of single pedestrians crossing the footbridge, represented in the model by formula 10.10. All pedestrians have different properties, therefore each pedestrian is modelled with a different velocity v_i, weight P_i, frequency f_i, phase shift ψ_i and time entering the bridge $t_{s,i}$. The pedestrians are modelled as groups with a random size, based on the related probability density function. Within a group the pedestrians have the same velocity in order to keep the group together when they cross the bridge, while it is possible that groups pass each other on the bridge due to difference in velocity. A group of pedestrians has a general moment in time it enters the bridge, but due to the space between the pedestrian within a group, each pedestrian has an individual starting time.

The parameters of the pedestrians have to be determined and implemented in the case study. Though the properties differ from person to person, the assumption is made that they stay constant in time. The choice that is made for each parameter will be discussed below.

Walking frequency

Numerous studies on the walking frequencies of pedestrians have been performed. The normal distribution that is the foundation for the parameters used in the EUR 23984 is used in this analysis. This results in a normal distribution of the walking frequencies of the pedestrians with μ = 1.8 Hz and σ = 0.268 Hz. This is the result of a combination of slow, normal and fast walking as can be seen in figure 3.3.[14]. It is assumed that all three contribute with the same weighting within a crowd. In figure 11.24 the result of the walking frequencies of the pedestrians is plotted for a run of 500 seconds with a density of 1.5 P/m^2 resulting in a plot for about 1000 pedestrians. The first and the second harmonic of the pedestrian loading have been included, represented in the figure by the two peaks. The figure is similar to the graph used for the reduction factor in the Eurocode and EUR 23984 showing the effect of the critical frequency range of the walking frequency of pedestrians (see respectively figure 4.1 and 4.7).

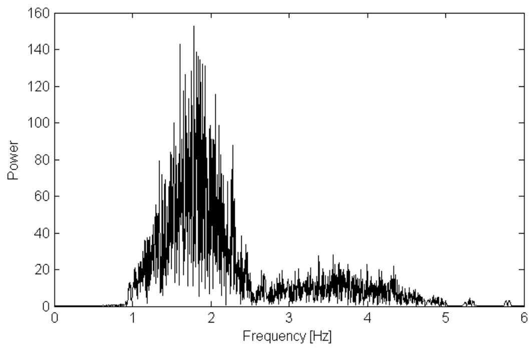


Figure 11.24: Walking frequencies of the pedestrians plotted in the frequency domain as used in the case study

Pedestrian weight
In most of the studies in literature an average weight of 700 N for a pedestrian is used. In view of the variety in weight for individual pedestrians a normal distribution for the weight with a coefficient of variation $c_v = 0.17$ is assumed according to Butz.[80] This results in a normal distribution with $\mu = 700$ N and $\sigma = 119$ N for the weight of the pedestrians.

Velocity
A linear relation between velocity and frequency is presented in literature: $v_p = 1.271 \cdot f_p - 1$.[14] In this case study instead of single pedestrians, groups of pedestrians walking together are considered. Therefore the pedestrians within a group are walking with the same velocity, but at the same time they have an individual walking frequency. In reality for example a person with short legs is likely to walk with a higher frequency in order to reach the same velocity as his friend with long legs. This formula is therefore not applicable. It does make sense that the walking velocity differs per group, resulting in groups that can pass each other when crossing the bridge. According to Tubino and Piccardo the sole direct statistical characterization of the pedestrian velocity available in literature has been carried out by Sahnaci and Kasperski representing a normal distribution with a $\mu = 1.37$ m/s and $\sigma = 0.15$ m/s.[80] This normal distribution is used in the determination of the walking velocity of groups.

As mentioned earlier, the walking velocity of pedestrians is assumed to be constant along the walkway. Therefore the time a pedestrian gets off the bridge can be calculated based on the time the pedestrian enters the bridge, the length of the bridge and the velocity of the group to which the pedestrian belongs.

Phase shift
The position of the human body and the feet when a person steps onto the bridge differ from person to person as well. This is represented in the formula by the phase shift ψ_i. The phase shift is assumed to be independent of any other parameter in this study and has therefore been modelled as a random value between 0 and 2π.

Fourier coefficients
The dynamic part of the pedestrian loading is represented by the Fourier coefficients α_1 and α_2, taking into account a percentage of the weight of the pedestrian for both the first and the second harmonic. The coefficients are assumed to be fixed values and have been taken equal to the ones used in the EUR 23984, based on extensive studies. The Fourier coefficients used are therefore $\alpha_1 = 0.4$ for the first harmonic loading and $\alpha_2 = 0.1$ for the second.

Overview parameters
In table 11.2 an overview is presented of the parameters for pedestrian loading that are used in the case study.

Distance between pedestrians
The pedestrians within a group are modelled with a certain distance between each other. This distance is expressed in a time interval, based on the overall density of the group that has to be met. In order to create a random distribution within the group, the time interval between the pedestrians is assumed to be based on a normal distribution. The mean and standard deviation are equal to the determined time interval.

Table 11.2: Overview pedestrian parameters considered in the case study

Parameter	**Symbol**	**Distribution**
Walking frequency	f_i	$\mu = 1.8$ Hz
		$\sigma = 0.268$ Hz
Pedestrian weight	P_i	$\mu = 700$ N
		$\sigma = 119$ N
Velocity	v_i	$\mu = 1.37$ m/s
		$\sigma = 0.15$ m/s
Phase shift	ψ_i	Random from 0 to 2π
Fourier coefficients	α_1	0.4
	α_2	0.1

Pedestrian loading

Combining all those properties and parameters the total pedestrian loading per density can be determined. Depending on the amount of pedestrian traffic, typical plots for the pedestrian loading are of interest. Figure 11.25 shows the dynamic pedestrian loading for three random individual pedestrians with different properties for the first mode shape. When groups with a certain time interval between them are considered, the pedestrian loading will look like figure 11.26. Figure 11.27 shows an example of the pedestrian loading for a crowd with a density of 0.5 P/m^2.

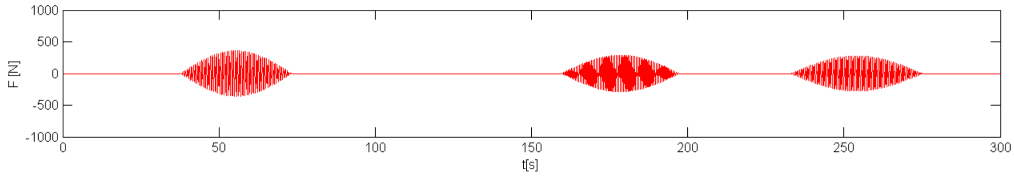


Figure 11.25: Typical pedestrian loading for three individual pedestrians walking across a bridge with L = 50 m

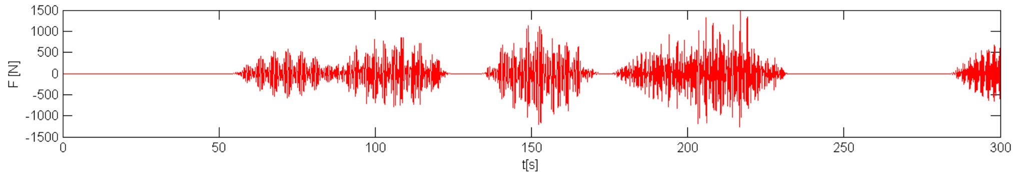


Figure 11.26: Typical pedestrian loading for several small groups walking across a bridge with L = 50 m

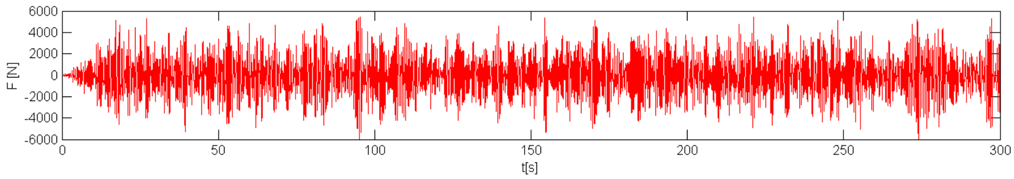


Figure 11.27: Example of the pedestrian loading for a crowd with a density 0.5 P/m^2 walking across a bridge with L = 50 m

11.2.3. Typical accelerations of individual pedestrians

The equation of motion 10.6 will be solved for each mode shape that is considered, using the numerical program Matlab. The result of this equation of motion is the deflection of the bridge for that specific mode shape in time at the normative position of the bridge. For the sinusoidal mode shapes of the simply supported bridges, this position will be at $x = \frac{L}{2 \cdot n}$. For the first mode shape this position will be at $x = \frac{L}{2}$, for the second mode shape at $x = \frac{L}{4}$, etc. (see figure 11.28).

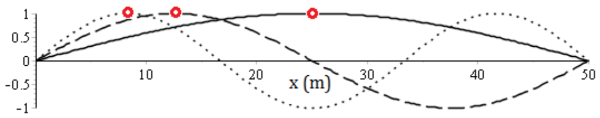


Figure 11.28: The first three mode shapes for simply supported bridges with the governing positions marked with red circles

The accelerations of the bridge can be calculated by differentiating the deflection signal twice to an acceleration signal. To check if the Matlab model is correct, an analysis for a single pedestrian and two combined pedestrians has been performed. The result of the Matlab model is compared with an analytical solution and a solution obtained with use of the program Maple. This check is presented in appendix D.

An example of the accelerations of a bridge under a pedestrian loading is presented in figure 11.29. For a bridge with a length of 50 m and a crowd pedestrian loading with density of 0.5 P/m^2 the accelerations are presented at the governing position $x = \frac{L}{2} = 25$ m.

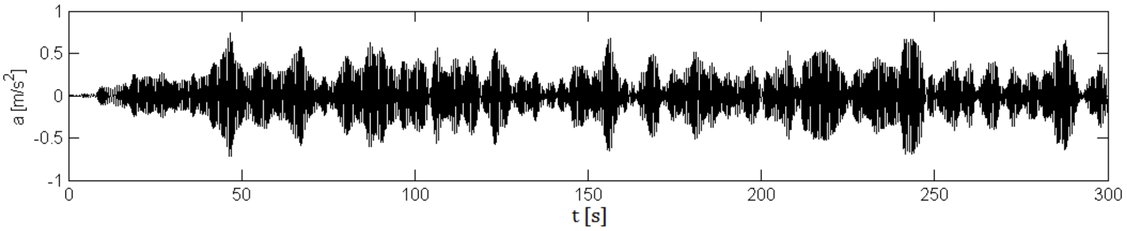


Figure 11.29: Example of the accelerations of the bridge with L = 50 m for a crowd with a density 0.5 P/m^2 for the first mode shape (at mid span: L = 25 m)

Instead of using the maximum value of this acceleration signal as prescribe by the Eurocode, the accelerations a single pedestrian perceives are considered. Therefore, for every individual pedestrian within the crowd, the acceleration signal when walking across the bridge is calculated. This is done by multiplying the acceleration of the bridge for a specific mode shape for the time-range the pedestrian is at the bridge by the shape of the same mode shape. The total acceleration signal for a single pedestrian is the summation of the acceleration signals for this pedestrian per mode shape. Figure 11.30 shows an example of the accelerations for a bridge with L = 50 m and a crowd loading based on a density of 0.5 P/m^2 in combination with the accelerations a single pedestrians experiences.

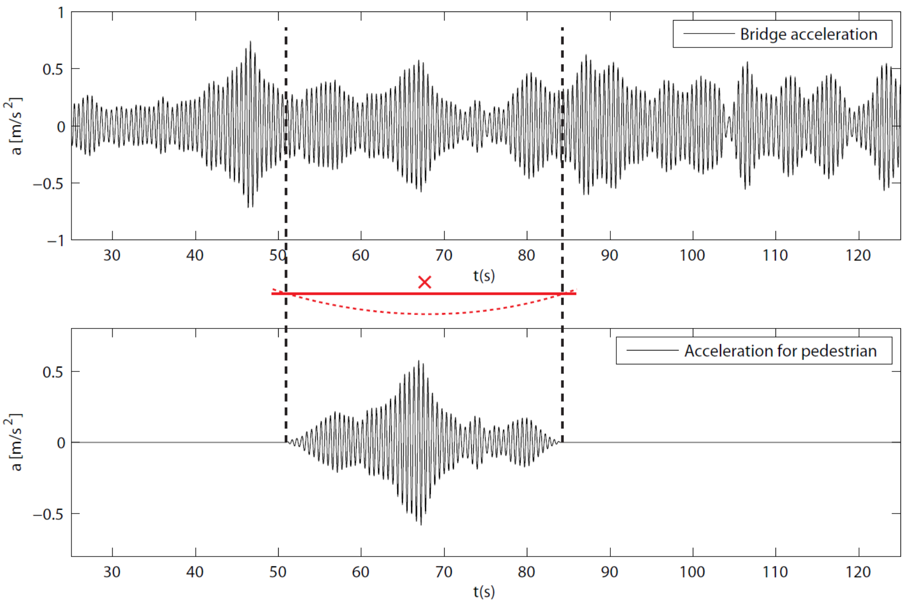


Figure 11.30: Top figure: accelerations of the bridge with L = 50 m and a crowd loading of 0.5 P/m^2; bottom figure: the accelerations a pedestrian experiences when crossing the bridge, for the first natural frequency

The first mode shape of the bridge is dominant, due to the fact that the natural frequency of this mode shape is in the range of the critical natural walking frequencies of pedestrians (and the second is not). Therefore these amplitudes of vibrations are higher (due to the resonance effect), in comparison which the amplitudes of the second mode shape are that so small (in the order of 1/1000 to 1/100 of the first) that they won't be noticed.

An example is provided for the acceleration an individual pedestrian in a crowd with a density of 0.5 P/m^2 perceives crossing the bridge with a length of 12, 50 and 100 m. The shape of the signal is the result of the mode shapes included and the characteristics of the acceleration signal of the bridge. These acceleration signals of the individual pedestrians can be evaluated by looking at the maximum acceleration these individual pedestrians perceive and the maximum value of the r.m.s. per second of the weighted acceleration, based on the study in section 10.5. This evaluation is dealt with in the next section.

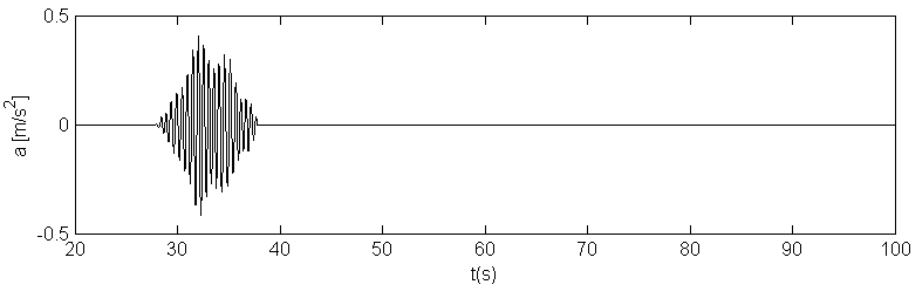


Figure 11.31: Example of a typical acceleration signal which a single individual pedestrian perceives crossing a footbridge with L = 12 m

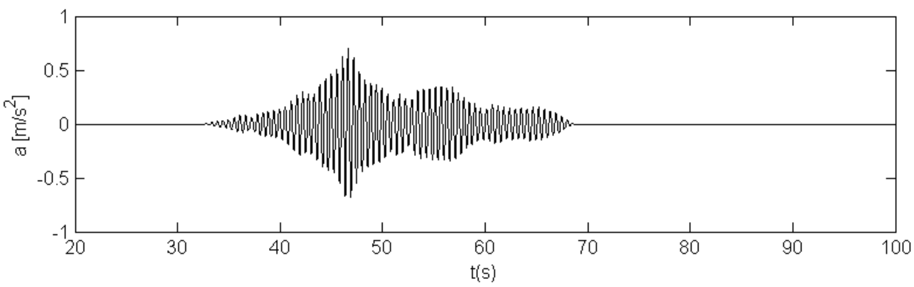


Figure 11.32: Example of a typical acceleration signal which a single individual pedestrian perceives crossing a footbridge with L = 50 m

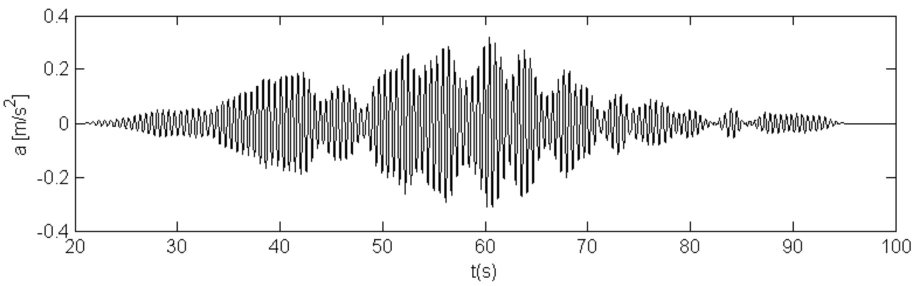


Figure 11.33: Example of a typical acceleration signal which a single individual pedestrian perceives crossing a footbridge with L = 100 m

Discussion
Relations between the weight of the pedestrian, the walking frequency and the velocity (for example adult or child) are expected to be present in reality. These kind of relations are not taken into account in this model. The parameters are random variables assigned to the pedestrian independent of the other parameters.

11.3. Human (dis)comfort

For every single pedestrian crossing the bridge in the model the acceleration signal has been calculated. In order to relate these signals to (dis)comfort they should be evaluated according to the methods discussed in section 10.5.

11.3.1. Evaluation of the accelerations

The Eurocode prescribes an evaluation of accelerations based on the maximum acceleration. In addition the choice for the method of evaluation of the accelerations according to the ISO standards is based on the the crest factor (equation 10.15). Therefore first the crest factor will be calculated for both the r.m.s. over the total time the pedestrian is at the bridge and for the r.m.s. per second.

Crest factor

In figure 11.34 the crest factor for the total the time the pedestrian is at the bridge is presented. Figure 11.35 shows the crest factor determined per second. Both crest factors remain below the ratio of six. This means that according to the ISO standards the r.m.s. of the weighted acceleration must be used for the evaluation of (dis)comfort on footbridges. In the ISO standards two evaluation methods are proposed (see table 10.1): the r.m.s. of the weighted acceleration for the time the pedestrian is at the bridge (ISO 2631-1) and the r.m.s. of the weighted acceleration per second (ISO 10137). For the r.m.s. per second of the weighted acceleration a corresponding comfort criterion is presented. In case of the r.m.s. of the weighted acceleration for the total time the pedestrian is walking across the bridge, only a perception level is provided. Therefore, in the evaluation of (dis)comfort only the r.m.s. per second of the weighted acceleration is used, next to the method of the Eurocode that considers the maximum acceleration per pedestrian.

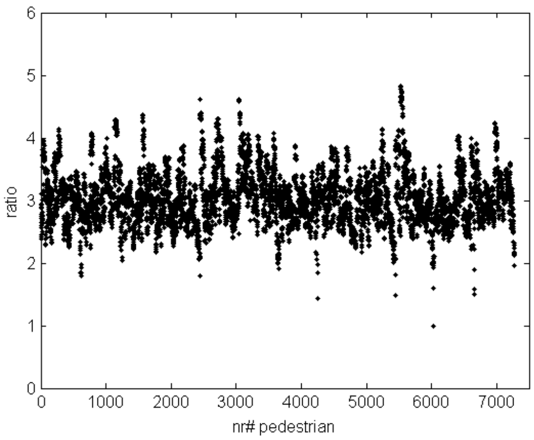


Figure 11.34: The crest factor per single pedestrian for a bridge L = 50 m for the total time

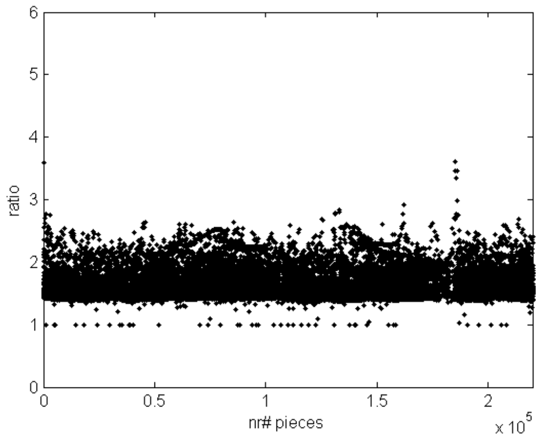


Figure 11.35: The crest factor per single pedestrian for a bridge L = 50 m per second

Frequency weighting

The r.m.s. per second is calculated based on the weighted acceleration (equation 10.19). The range of frequencies relevant for footbridges is about 1 to 5 Hz, with the main critical frequency around 1.8 Hz. Because the frequencies are relatively close to each other, in this study the approximation is made to use the same frequency weighting for all weighted acceleration signals. The size of the frequency weighting is determined with the graph of the frequency weighting, presented in appendix E at figure E.2 in combination with table E.3. To determine the weighting factor for a frequency of 1.8 Hz linear interpolation is used, resulting in a frequency weighting of $W_k = 0.513$.

Example of the evaluated acceleration signal for one pedestrian

Figure 11.36 shows the acceleration signal for one pedestrian. In this graph the maximum acceleration, the r.m.s. of the total acceleration signal and the r.s.m. per second of the acceleration are presented. For the last two measurement methods of the acceleration signal, the weighted acceleration has to be used in the evaluation of human comfort.

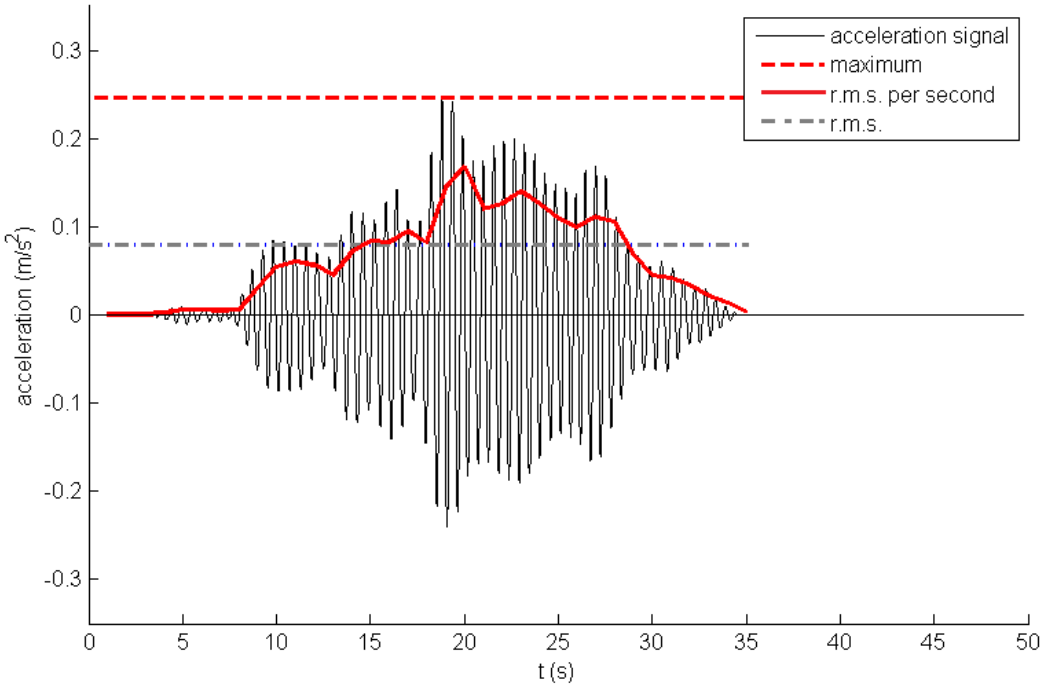


Figure 11.36: Evaluated acceleration signal for one pedestrian crossing a bridge with L = 50 m within a crowd with a density of 0.5 P/m^2

11.3.2. Application of the criteria for discomfort

Both for the maximum acceleration and for the r.m.s. per second of the weighted acceleration a comfort criterion is presented: $a_{max} < 0.7$ m/s^2 and $a_{w,rms,persec} < 0.42$ m/s^2. Information on the distribution of the comfort criteria, on the probability of exceedance and on the influence of other parameters as discussed in section 10.5 has not been presented in a usable format. Therefore the criteria are used as a fixed value in the evaluation of human comfort. This results in a probability of 100% of discomfort when these limit levels are exceeded.

11.4. Probability of (dis)comfort and acceptance

The acceleration signal for the individual pedestrians as discussed in section 11.2 and the comfort measurements & criteria presented in section 11.3 are now combined for the evaluation of the probability of (dis)comfort. The results of the calculations are presented in chapter 12.

11.4.1. Acceptance

As discussed in chapter 9 the recommended value for the probability of failure (in this case study discomfort) in case of the serviceability limit state is 1%. The procedure of the Eurocode is though based on a 5% exceedance level. To be able to combine the results of the Eurocode with the probability-based approach of this part of the work, a probability of failure of 5% is used in this evaluation as well.

11.4.2. Model time

For a probabilistic approach a significant amount of data is necessary to get statistical relevant results. The Matlab model is built up based on a certain model time. In one script the model calculates the acceleration signals for the three bridges and for all the selected densities or scenarios based on group formation. The same model time is automatically applied for all the different cases, resulting in a difference in the number of pedestrians that is included. Therefore the reliability of the results is not equal in all cases. The model time of one hour is used in order to get at least a significant number of pedestrians crossing the bridges for all the situations as well as to keep the calculation time for the model limited. The scenario for the park based on group formation is calculated separately because the time interval between the groups is on average 300 seconds, resulting in a small amount of pedestrians per hour. Therefore instead of one hour, a model time of eight hours has been applied.

11.4.3. Evaluation of discomfort for the Eurocode procedure with TC3

The Eurocode recommended pedestrian loading of 0.5 P/m^2 has been applied to the three bridges. The maximum accelerations the individual pedestrians experience when crossing the bridge have been evaluated. This result is compared to the maximum acceleration of the bridge deck for the steady state according to the evaluation method of the EUR 23984. The result for the bridge length of 50 m has been plotted in figures 11.37 and 11.38. On the left the probability distribution for the maximum accelerations of the individual pedestrians is shown, on the right the cumulative density function (CDF) of the same results is presented. The grey dotted vertical line represents the maximum acceleration of the bridge according to the Eurocode, the red dashed line is the corresponding comfort criterion of 0.7 m/s^2. The results for the bridges with a length of 12 and 100 m are presented in appendix F.

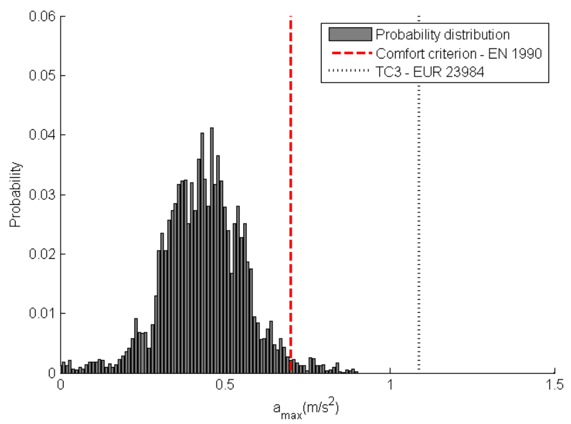


Figure 11.37: Probability distribution of the maximum accelerations per single pedestrian crossing the bridge within a crowd density of 0.5 P/m^2, compared with the maximum acceleration calculated with EUR 23984 for the bridge L = 50 m

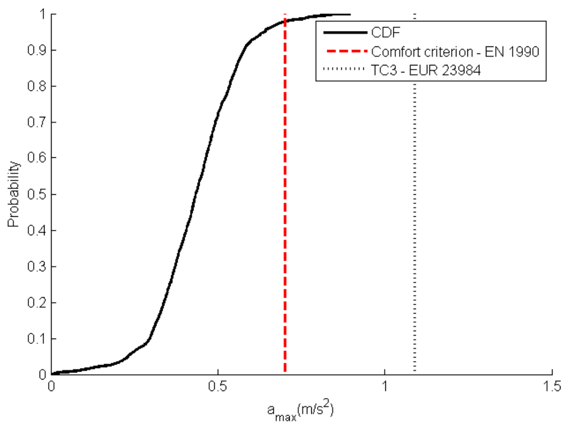


Figure 11.38: Cumulative density function of the maximum accelerations per single pedestrian crossing the bridge within a crowd density of 0.5 P/m^2, compared with the maximum acceleration calculated with EUR 23984 for the bridge L = 50 m

As can be seen in the CDF graph, there is a gap between the maximum acceleration of the bridge and the expected maximum accelerations of the individual pedestrians. This can be explained in two ways. First, the probability an individual pedestrian will indeed experience the maximum acceleration of the bridge that is expected to occur under this pedestrian loading is small for a short model time. So, when an infinitely long model time will be used, the CDF of the maximum accelerations of the pedestrians is expected to extend to the right towards the maximum acceleration of the bridge. Secondly, the maximum acceleration of the bridge is determined based on the steady state response of a group of pedestrians uniformly distributed across the bridge on a fixed position. When a group of pedestrians moves across the bridge the pedestrian loading will continuously change, whereby the steady state response will not be met.

The expected maximum acceleration which individual pedestrians experience when crossing the bridges within the recommended pedestrian loading of 0.5 P/m^2 has been calculated, based on the 5% exceedance probability (in line with the Eurocode).

11.4.4. Evaluation of discomfort for the scenarios based on densities

The probability of discomfort for scenarios based on densities has been determined according to the procedure discussed in 10.6. By using a mixture model the relevant densities and the corresponding probabilities have been combined, resulting in the expected accelerations for one scenario. Both for the maximum acceleration and for the r.m.s. per second of the weighted accelerations this evaluation has been performed for the three bridges and the four scenarios. The results for the expected maximum accelerations for the four scenarios in case the bridge has a length of 50 m are presented expressed in a CDF in figure 11.39. The red crosses represent the 5% probability of exceedance. On top of the graph the results of the maximum acceleration for the traffic classes according to the EUR 23984 are presented in relation with the levels of comfort. The comfort criterion of 0.7 m/s^2 according to the Eurocode (EN 1990) is presented as well.

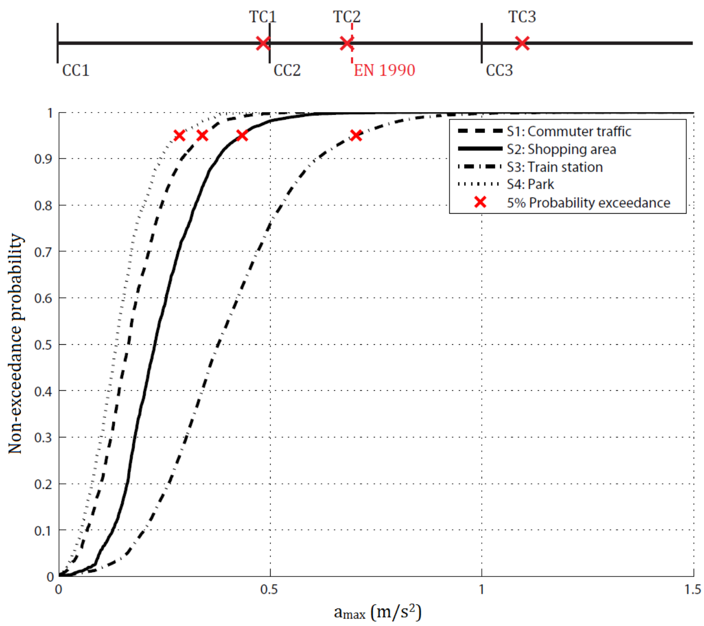


Figure 11.39: Cumulative density functions of the maximum expected accelerations per single pedestrian for the four scenarios based on densities and L = 50 m

The CDFs for the r.m.s. per second of the weighted accelerations are examined as well. The 5% exceedance level has been determined for all the CDFs for both the maximum acceleration and the r.m.s. per second of the weighted acceleration. In addition the non-exceedance probabilities for the comfort criteria have been calculated.

11.4.5. Evaluation of discomfort for the scenarios based on group formation

The probability of discomfort for the scenarios based on group formation can be determined directly from the evaluation of the accelerations of the individual pedestrians. Both for the maximum acceleration and for the r.m.s. per second of the weighted accelerations this evaluation has been performed as well as for the three bridges and the four scenarios. In figure 11.40 the CDFs for the four scenarios for the maximum accelerations in case the bridge has a length of 50 m are shown. The red crosses represent the maximum accelerations corresponding to the 5% exceedance level. On top of the graph again the traffic classes and levels of comfort are presented.

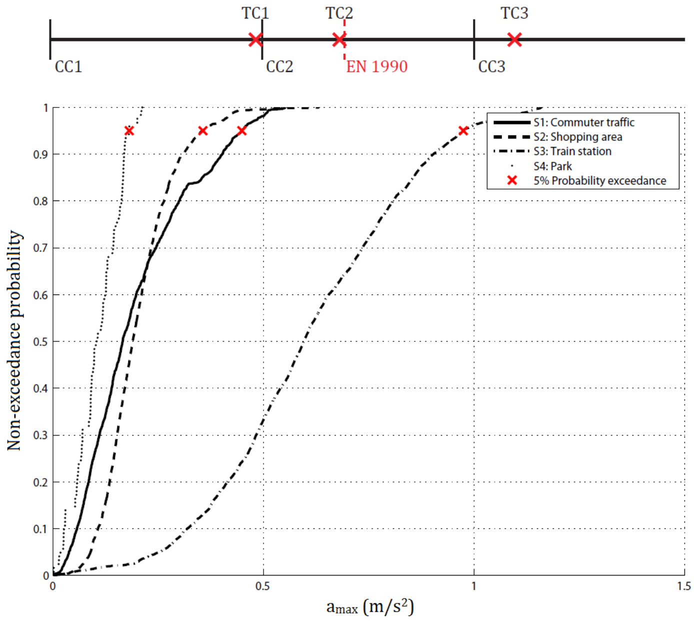


Figure 11.40: Cumulative density functions of the maximum expected accelerations per single pedestrian for the four scenarios based on group formation and L = 50 m

These CDFs are less smooth than the ones presented for the scenarios based on densities (figure 11.39). This is caused by the fact that the scenarios based on densities are built up from fifteen probability distributions of the corresponding traffic densities, whereas the CDFs for the scenarios based on group formation are only based on the single probability distribution of these scenarios. The CDFs for the r.m.s. per second of the weighted accelerations are determined as well. For all the CDFs the 5% exceedance level has been determined. In addition the non-exceedance probabilities for the comfort criteria have been calculated.

12

Results part II

The results of the case study are presented in this chapter per individual section. The tables provide an overview of the most important results. The results are visualized as well in graphs.

12.1. Pedestrian traffic

The pedestrian traffic is divided into four assumed scenarios. These scenarios are set up in two ways: scenarios based on densities and scenarios based on group formation. The scenarios based on densities consist of a probability mass function based on the combination of continuous pedestrian flows in one density, independent of the length of the bridge. The scenarios based on group formation are built up from different group sizes (based on a probability mass function per scenario) in combination with a scenario dependent time interval between the groups. The case study is performed with three simply supported bridges. For every bridge, for all the scenarios the density corresponding to a 5% exceedance level is determined. These densities are presented in table 12.1.

For the scenarios based on densities the probability distributions with the 5% exceedance level are plotted in figure 12.1. In figure 12.2 the result for the 5% exceedance level for the scenarios based on group formation is presented. On the right in figure 12.3 the same results as in figure 12.2 are presented, but without the third scenario. The red dashed line represents the prescribed value of the Eurocode. The other horizontal lines represent the 5% exceedance level for the scenarios based on densities. It can be seen that for the scenarios based on group formation, the expectation for the density is almost equal for the first, second and fourth scenario. This expected density is below the prescribed value of the Eurocode and significant different from the third scenario, that is about five times bigger. This difference is visible at the scenarios based on densities as well, though the values of the first, second and fourth scenario differ here as well.

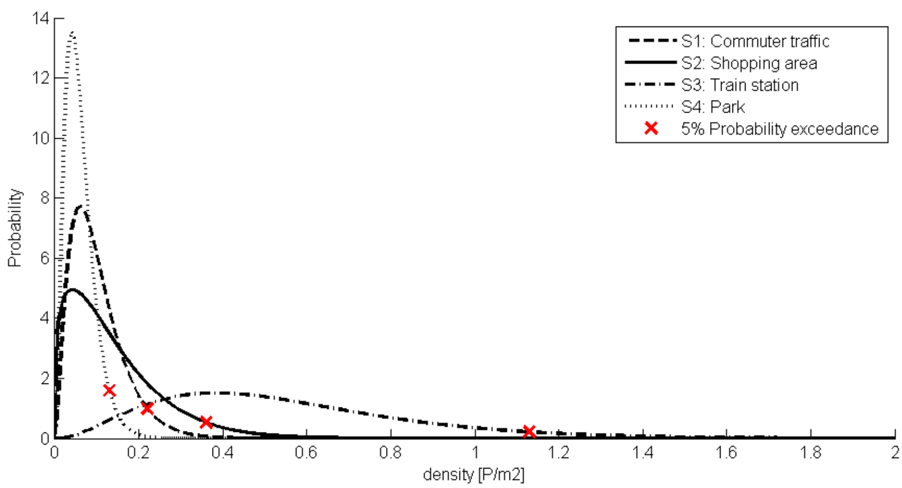


Figure 12.1: Probability density functions including the 5 percentage exceedance level per bridge for the four scenarios based on densities

Table 12.1: Result density analysis - densities for the 95% non-exceedance probability

Evaluation method	L = 12 m	L = 50 m	L = 100 m
Eurocode	0.5 P/m^2	0.5 P/m^2	0.5 P/m^2
Scenarios based on densities			
Scenario 1: d < 95%	0.22 P/m^2	0.22 P/m^2	0.22 P/m^2
Scenario 2: d < 95%	0.36 P/m^2	0.36 P/m^2	0.36 P/m^2
Scenario 3: d < 95%	1.13 P/m^2	1.13 P/m^2	1.13 P/m^2
Scenario 4: d < 95%	0.13 P/m^2	0.13 P/m^2	0.13 P/m^2
Scenarios based on group formation			
Scenario 1: d < 95%	0.31 P/m^2	0.14 P/m^2	0.09 P/m^2
Scenario 2: d < 95%	0.28 P/m^2	0.14 P/m^2	0.08 P/m^2
Scenario 3: d < 95%	2.25 P/m^2 *	1.40 P/m^2	0.75 P/m^2
Scenario 4: d < 95%	0.28 P/m^2	0.13 P/m^2	0.05 P/m^2

* *A crowd density of 2.25 P/m^2 seems unrealistic high. This value is the result of the assumption in the model that groups of pedestrians can pass each other (due to a difference in group velocity). For scenario 3 (train station) this can result in two groups of for example 50 and 100 people, with already a high density of 1.5 P/m^2, that pass each other. The total crowd density at the bridge, in particular for the short bridge of 12 m, can therefore become this high. The maximum admissible density for a walking crowd according to Bachmann is 120 kg/m^2, which is equal to 1.6-1.8 P/m^2.[11] Two more recent studies suggest a higher value. The critical density for a crowd that is still able to walk according to Still is 2-3 P/m^2, based on modelling walking pedestrians.[81] Löhner and Haug present a critical density of 5-6 P/m^2, based on purely kinematic considerations for critical densities beyond which it is impossible for pedestrians to enter and/or advance into an incoming crowd.[82] Though it seems not realistic that large groups at a train station will pass each other, it is suggested that at a density of 2.25 P/m^2 pedestrians are able to walk.*

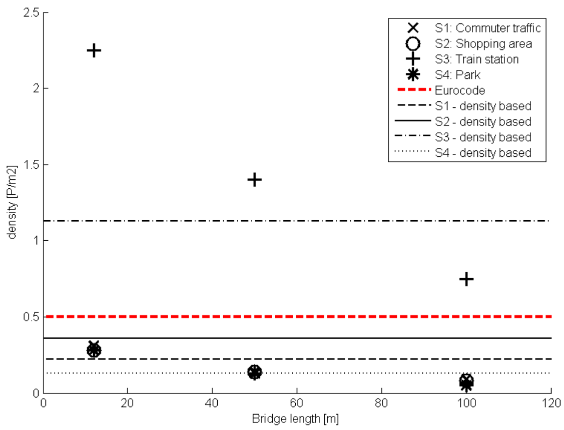


Figure 12.2: Results of the densities (based on 5 percentage exceedance level) per bridge for the four scenarios based on group formation

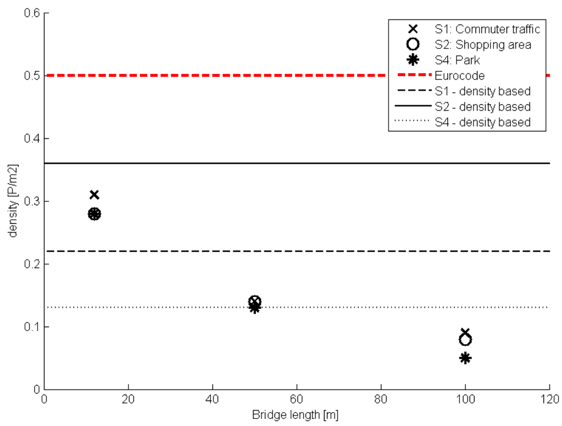


Figure 12.3: Zoom in of the left graph without scenario 3

The difference between density according to the Eurocode and the scenario-approach is presented in figure 12.4 expressed in percentage. The crowd density according to the Eurocode is represented by the red dashed line. The red bars in the graph represent the scenarios based on densities, the black/grey bars represent the scenarios based on group formation for each bridge length. It can be seen that for the third scenario the expected density is significant higher than the prescribed value of the Eurocode. For the other scenarios the expected density is lower. The scenarios based on group formation show a decrease in the expected density for an increasing bridge length as well.

In figure 12.5 the densities of the scenarios based on group formation have been compared to the scenarios based on densities expressed in percentage. The red dotted horizontal line represents the scenarios based on densities. The black/grey bars represent the scenarios based on group formation. A clear difference between the different lengths is visible. For the bridge with a length of 12 m, the expected densities based on group formation compared to the scenarios based on densities are higher for scenario 1, 3 and 4. Looking at the bridge with a length of 50 m, only scenario 3 is higher for the scenarios based on group formation. For the bridge with a length of 100 m all the scenarios based on group formation result in a lower density than the scenarios based on densities. Scenario 2 based on group formation is a pedestrian flow that is already more similar to a constant density (see figure 11.21), therefore the difference between the three bridge lengths for this comparison is less.

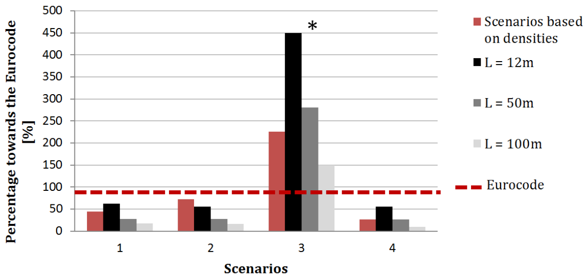


Figure 12.4: Comparison of the expected densities for the four scenarios based on densities and on group formation - expressed in percentage % of the Eurocode (0.5 P/m^2)
* See remarks at the asterisk in table 12.1

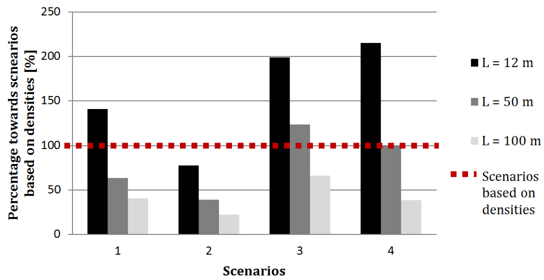


Figure 12.5: Comparison of the expected densities for the four scenarios based on group formation towards the scenarios based on densities - expressed in percentage % of the scenarios based on densities

12.2. Accelerations individual pedestrians

For each bridge and for each scenario the accelerations per individual pedestrian have been calculated with the probability-based approach. To be able to compare these accelerations with the acceleration that has been calculated with the Eurocode, the accelerations have been determined with the same probability of non-exceedance of 95%. Table 12.2 shows the results for both the scenarios based on densities and on group formation.

Table 12.2: Result acceleration analysis - accelerations for the 95% non-exceedance probability

Evaluation method	L = 12 m	L = 50 m	L = 100 m
Eurocode with d = 0.5 P/m^2			
$a_{\text{max,bridge}} < 95\%$	2.22 m/s^2	1.09 m/s^2	0.67 m/s^2
Model with d = 0.5 P/m^2			
$a_{\text{max,bridge}} < 95\%$	2.02 m/s^2	1.02 m/s^2	0.60 m/s^2
Difference with Eurocode	*9.0 %*	*7.6 %*	*10.4 %*
$a_{\text{max,pedestrian}} < 95\%$	1.16 m/s^2	0.64 m/s^2	0.41 m/s^2
Difference with Eurocode	*47.7 %*	*41.3 %*	*38.8 %*
Scenarios based on densities			
Scenario 1: $a_{\text{max,pedestrian}} < 95\%$	0.62 m/s^2	0.35 m/s^2	0.20 m/s^2
Scenario 2: $a_{\text{max,pedestrian}} < 95\%$	0.74 m/s^2	0.44 m/s^2	0.29 m/s^2
Scenario 3: $a_{\text{max,pedestrian}} < 95\%$	1.16 m/s^2	0.71 m/s^2	0.47 m/s^2
Scenario 4: $a_{\text{max,pedestrian}} < 95\%$	0.59 m/s^2	0.29 m/s^2	0.16 m/s^2
Scenarios based on group formation			
Scenario 1: $a_{\text{max,pedestrian}} < 95\%$	0.78 m/s^2	0.44 m/s^2	0.19 m/s^2
Scenario 2: $a_{\text{max,pedestrian}} < 95\%$	0.70 m/s^2	0.36 m/s^2	0.17 m/s^2
Scenario 3: $a_{\text{max,pedestrian}} < 95\%$	1.96 m/s^2 *	0.97 m/s^2	0.51 m/s^2
Scenario 4: $a_{\text{max,pedestrian}} < 95\%$	0.33 m/s^2	0.18 m/s^2	0.10 m/s^2

* *See remarks at the asterisk in section 12.1. The high traffic density results in high accelerations.*

The difference between the expected maximum acceleration of the bridge according to the Eurocode and the expected maximum acceleration per pedestrian for the prescribed traffic class of 0.5 P/m^2 is about 40%. Roughly 10% of this difference follows from the difference in calculation method: the Eurocode uses the steady state response for a fixed flow of pedestrians, whereas the probability-based approach determines the accelerations of the bridge per run of 300 seconds for a moving crowd. The difference in expected accelerations between the Eurocode (presented with a red dotted line) and the probability-based approach are presented in the graphs in figures 12.6, 12.7 and 12.8 for the scenarios based on densities and in figures 12.9, 12.10 and 12.11 for the scenarios based on group formation.

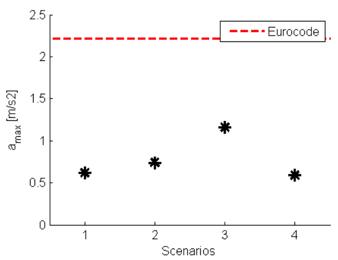


Figure 12.6: Results of the maximum accelerations for the four scenarios based on densities for L = 12m

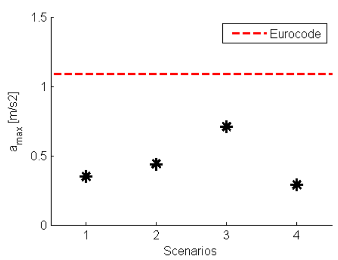


Figure 12.7: Results of the maximum accelerations for the four scenarios based on densities for L = 50m

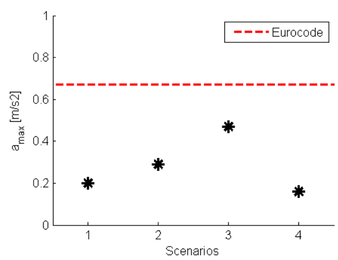


Figure 12.8: Results of the maximum accelerations for the four scenarios based on densities for L = 100m

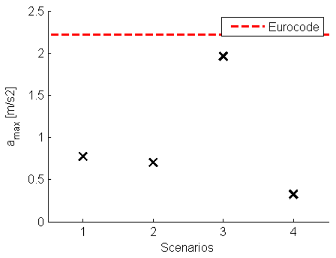


Figure 12.9: Results of the maximum accelerations for the four scenarios based on group formation for L = 12m

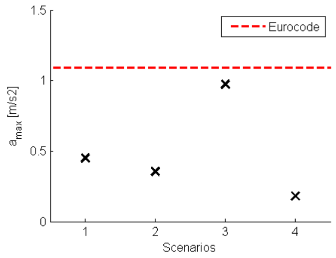


Figure 12.10: Results of the maximum accelerations for the four scenarios based on group formation for L = 50m

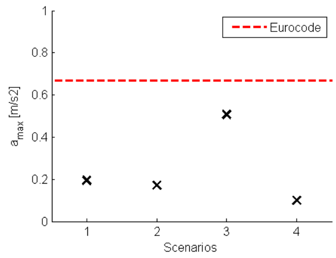


Figure 12.11: Results of the maximum accelerations for the four scenarios based on group formation for L = 100m

The maximum expected accelerations for the scenarios based on densities and for the scenarios based on group formation can now be compared. The expected accelerations at the first, second and fourth scenario are in the same order of magnitude, whereas the accelerations for the third scenario are expected to be larger for the scenario based on group formation.

The ratio of the determined maximum accelerations per pedestrian for scenarios and the maximum acceleration of the bridge according to the Eurocode is presented in figure 12.12 for the scenarios based on densities and in figure 12.13 for the scenarios based on group formation. The result of the Eurocode is represented by the red dashed line. The scenarios based on densities show an increase in the percentage for the proportion of the accelerations compared to the Eurocode for an increasing bridge length. The scenarios based on group formation show on the other hand a decrease in the proportion of the accelerations compared to the Eurocode.

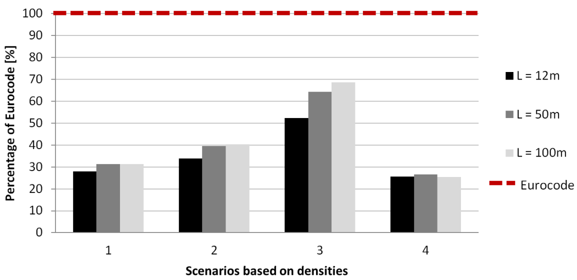


Figure 12.12: Comparison of the results of the accelerations for the scenarios based on densities with the expected bridge acceleration of the Eurocode - expressed in percentage [%] of the Eurocode acceleration

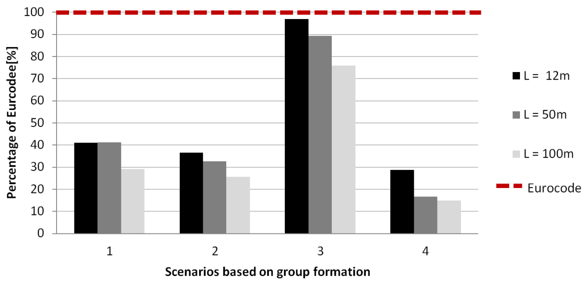


Figure 12.13: Comparison of the results of the accelerations for the scenarios based on group formation with the expected bridge acceleration of the Eurocode - expressed in percentage [%] of the Eurocode acceleration

In figure 12.14 the comparison between figure 12.12 and figure 12.13 has been made. The red dotted line represents the scenarios based on densities. The black/grey bars represent the scenarios based on group formation as a percentage of the scenarios based on densities. There is a significant difference between the different bridge lengths that have been taken into account. The scenarios based on group formation result in higher crowd densities than all scenarios based on densities for the bridge with a length of 12 m. In the case of the bridge with a length of 100 m it is the other way around: the accelerations for the scenarios based on group formation are always lower than the accelerations for the scenarios based on densities. For the bridge with a length of 50 m the scenarios based on group formation result in higher accelerations compared to the scenarios based on densities for scenarios 1 and 3. In case of scenarios 2 and 4 it is the other way around.

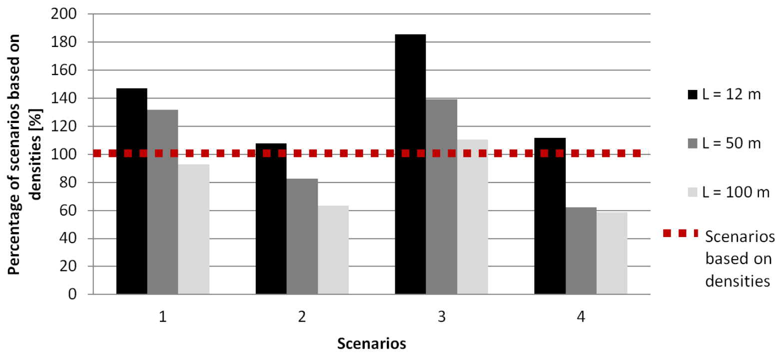


Figure 12.14: Results of the accelerations - comparison of the scenarios based on group formation towards the scenarios based on densities expressed in percentage [%]

12.3. Measurement methods for accelerations

Three different methods for measuring accelerations in relation to human comfort are presented: maximum acceleration, root mean square of the weighted acceleration and root mean square per second of the weighted acceleration. The correlation between these methods is presented in figures 12.15 and 12.16 with scatter plots. The plots show the accelerations calculated for the bridge with a length of 50 m and a traffic density of 0.5 P/m^2. The result is a high correlation between the maximum acceleration and the maximum value of the r.ms. per second of the weighted acceleration, as the scatter plot is almost a linear line (figure 12.16). The correlation between the maximum acceleration and the r.m.s. of the weighted acceleration is visible, but less strong as the first correlation, because the scatter plot has a more oval shape (figure 12.15).

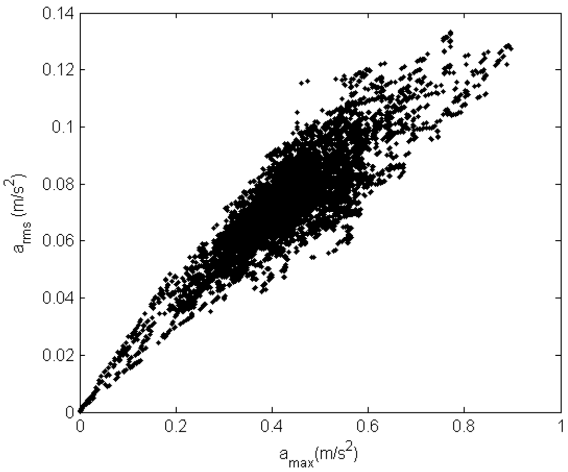


Figure 12.15: The correlation between the maximum acceleration and the r.m.s. of the acceleration per single pedestrian for a bridge L = 50 m and a traffic density of 0.5 P/m^2

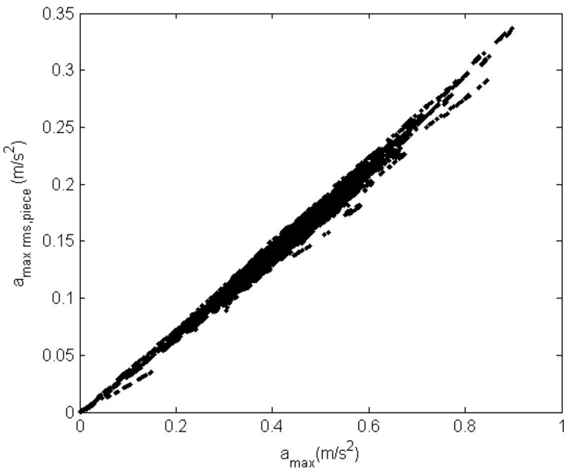


Figure 12.16: The correlation between the maximum acceleration and the maximum of the r.m.s. per second of the acceleration per single pedestrian for a bridge L = 50 m and a traffic density of 0.5 P/m^2

The high correlation between the maximum acceleration and the r.m.s. per second for the weighted acceleration indicates that the result of both evaluation methods is supposed to be the same. A difference can be expected in the use of either the maximum acceleration or the r.m.s. of the weighted acceleration in the probability-based approach, because of the less significant correlation.

12.4. Probability of (dis)comfort

The probability of a comfortable feeling is defined as the probability that the accelerations which a pedestrian is exposed to, remain below the limit level. Two limit levels for footbridges are presented: $a_{max} < 0.7$ m/s^2 (EN 1990) and $a_{w,rms,persec} < 0.42$ m/s^2 (ISO 10137). The probability a pedestrian will experience accelerations that do not exceed these limit levels, is presented in table 12.3 expressed in percentage. In other words, this percentage indicates the probability that a pedestrian is expected to feel comfortable crossing the footbridge. The results of the probability-based approach are presented for the pedestrian flow of 0.5 P/m^2, for the scenarios based on densities and for the scenarios based on group formation.

Table 12.3: Result comfort analysis - non-exceedance probabilities

Evaluation method	**L = 12 m**	**L = 50 m**	**L = 100 m**
EN 1990			
$a_{\max} < 0.7$ m/s^2	rejected	rejected	approved
Model with d = 0.5 P/m^2			
$a_{\max} < 0.7$ m/s^2	60.72%	97.72%	100%
Scenarios based on densities			
Scenario 1: $a_{\max} < 0.7$ m/s^2	98.00%	100%	100%
Scenario 2: $a_{\max} < 0.7$ m/s^2	94,80%	99,91%	100%
Scenario 3: $a_{\max} < 0.7$ m/s^2	70,15%	95,24%	99,96%
Scenario 4: $a_{\max} < 0.7$ m/s^2	97,88%	100%	100%
Scenarios based on group formation			
Scenario 1: $a_{\max} < 0.7$ m/s^2	92.03%	100%	100%
Scenario 2: $a_{\max} < 0.7$ m/s^2	95.01%	100%	100%
Scenario 3: $a_{\max} < 0.7$ m/s^2	22.45% *	65.11%	99.77%
Scenario 4: $a_{\max} < 0.7$ m/s^2	100%	100%	100%
ISO 10137			
Model with d = 0.5 P/m^2			
$a_{\text{w,rms,per sec}} < 0.42$ m/s^2	96.60%	100%	100%
Scenarios based on densities			
Scenario 1: $a_{\text{w,rms,per sec}} < 0.42$ m/s^2	99.92%	100%	100%
Scenario 2: $a_{\text{w,rms,per sec}} < 0.42$ m/s^2	99.90%	100%	100%
Scenario 3: $a_{\text{w,rms,per sec}} < 0.42$ m/s^2	96.73%	99.96%	100%
Scenario 4: $a_{\text{w,rms,per sec}} < 0.42$ m/s^2	100%	100%	100%
Scenarios based on group formation			
Scenario 1: $a_{\text{w,rms,per sec}} < 0.42$ m/s^2	100%	100%	100%
Scenario 2: $a_{\text{w,rms,per sec}} < 0.42$ m/s^2	99.75%	100%	100%
Scenario 3: $a_{\text{w,rms,per sec}} < 0.42$ m/s^2	73.75%	100%	100%
Scenario 4: $a_{\text{w,rms,per sec}} < 0.42$ m/s^2	100%	100%	100%

* *See remarks at the asterisks in sections 12.1 and 12.2. The high traffic density causes high accelerations. This results in a low non-exceedance probability for the comfort level.*

Section 12.3 has revealed that the results of the evaluation methods for human comfort are supposed to be the same. The non-exceedance probabilities for the comfort limit based on the r.m.s. per second of the weighted acceleration are however clearly higher than the non-exceedance probabilities for the comfort limit based on the maximum acceleration. This result is therefore expected to be caused by a difference between the two comfort limits.

The results have been plotted as cumulative density functions (CFDs) for the accelerations per bridge length. The CDFs for the scenarios based on densities and on group formation have been plotted both for the maximum accelerations (figures 12.17, 12.19, 12.21, 12.23, 12.25 and 12.27) as well as for the r.m.s. per second of the weighted acceleration (figures12.18, 12.20, 12.22, 12.24, 12.26 and 12.28). The red crosses in the graphs indicate the 5% exceedance level. On top of the graphs at the left the results of the evaluation of the maximum acceleration for the traffic classes according to the EUR 23984 are presented. These are combined with the comfort levels according to the EUR 23984 and the comfort limit of $a_{max} \leq 0.7$ m/s^2 from the EN 1990. On top of the graphs at the right the comfort limit of $a_{w,rms,persec} \leq 0.42$ m/s^2 according to the ISO 10137 is presented.

Scenarios based on densities

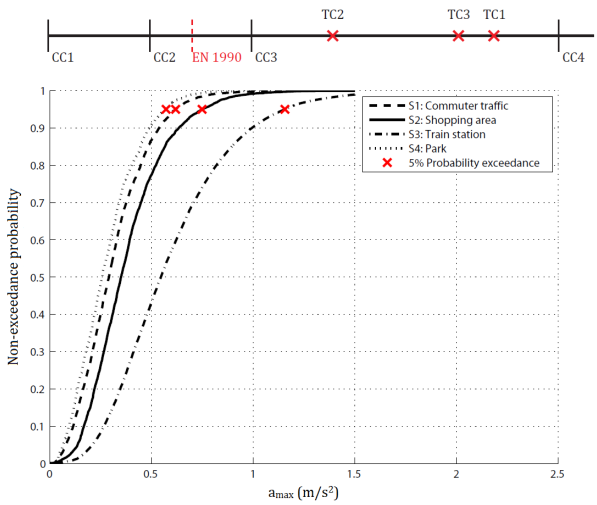


Figure 12.17: Cumulative density functions of the maximum expected accelerations per single pedestrian for the scenarios based on densities and L = 12 m

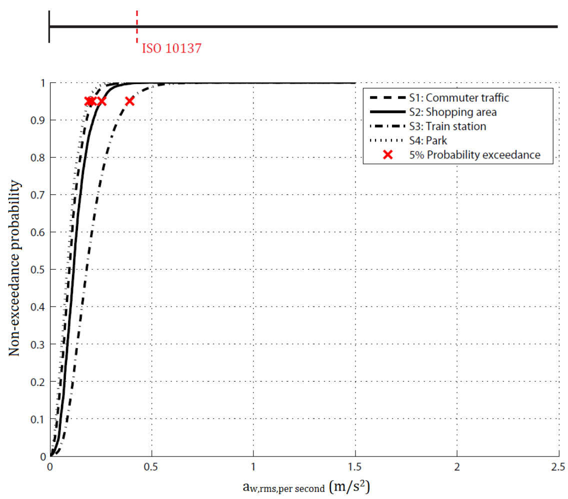


Figure 12.18: Cumulative density functions of the expected r.m.s. per second of the weighted accelerations per single pedestrian for the scenarios based on densities and L = 12 m

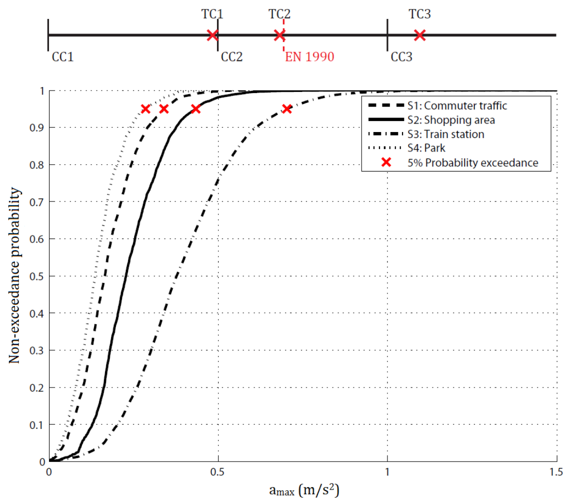


Figure 12.19: Cumulative density functions of the maximum expected accelerations per single pedestrian for the scenarios based on densities and L = 50 m

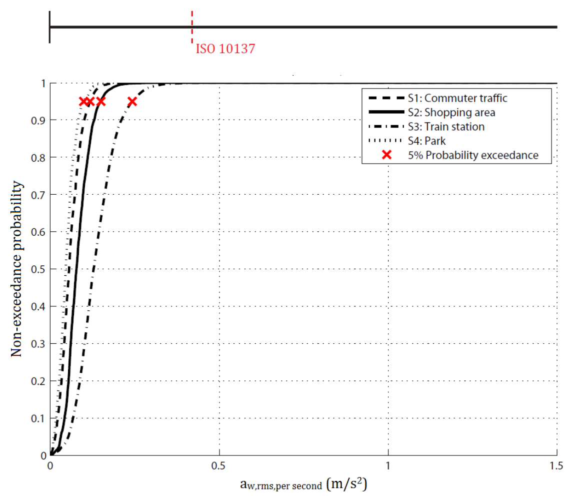


Figure 12.20: Cumulative density functions of the expected r.m.s. per second of the weighted accelerations per single pedestrian for the scenarios based on densities and L = 50 m

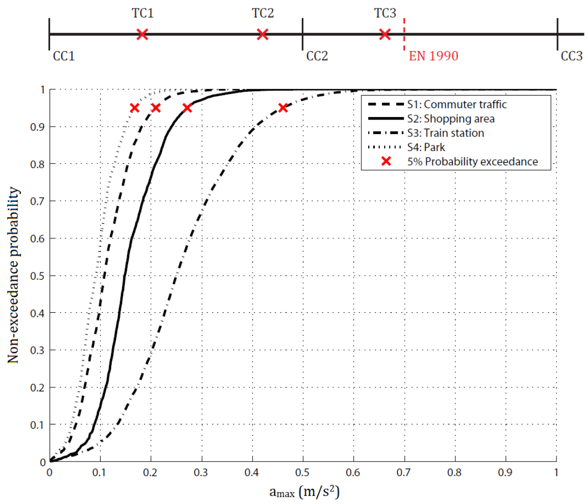


Figure 12.21: Cumulative density functions of the maximum expected accelerations per single pedestrian for the scenarios based on densities and L = 100 m

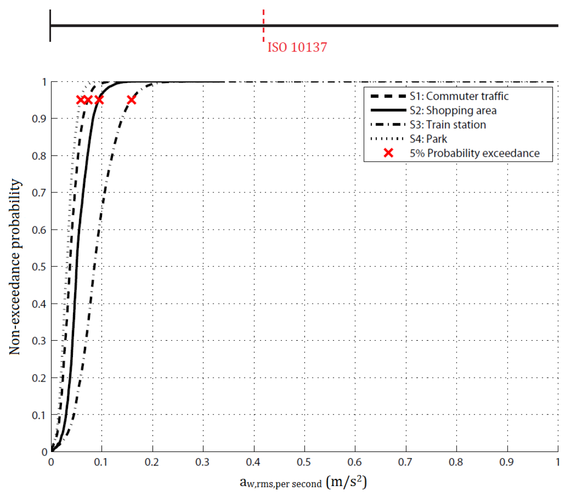


Figure 12.22: Cumulative density functions of the expected r.m.s. per second of the weighted accelerations per single pedestrian for the scenarios based on densities and L = 100 m

Scenarios based on group formation

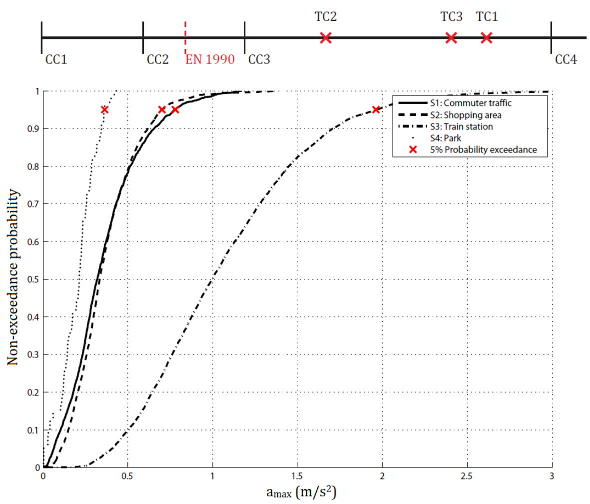


Figure 12.23: Cumulative density functions of the maximum expected accelerations per single pedestrian for the scenarios based on group formation and L = 12 m

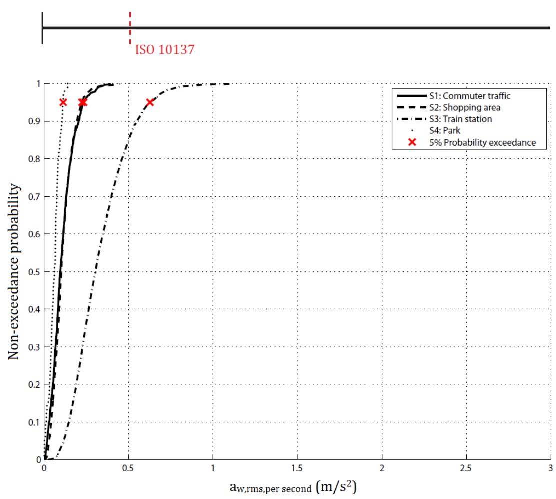


Figure 12.24: Cumulative density functions of the expected r.m.s. per second of the weighted accelerations per single pedestrian for the scenarios based on group formation and L = 12 m

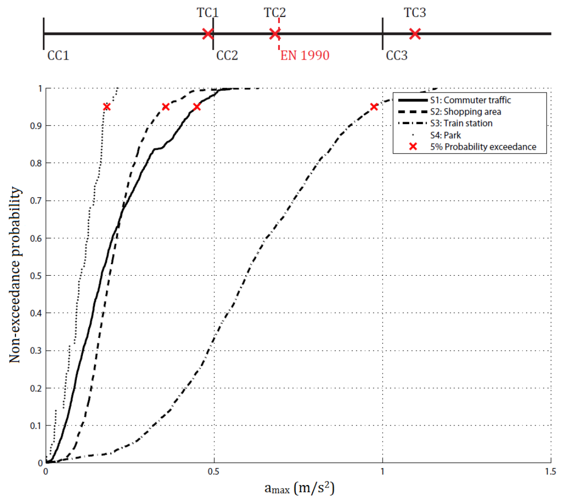


Figure 12.25: Cumulative density functions of the maximum expected accelerations per single pedestrian for the scenarios based on group formation and L = 50 m

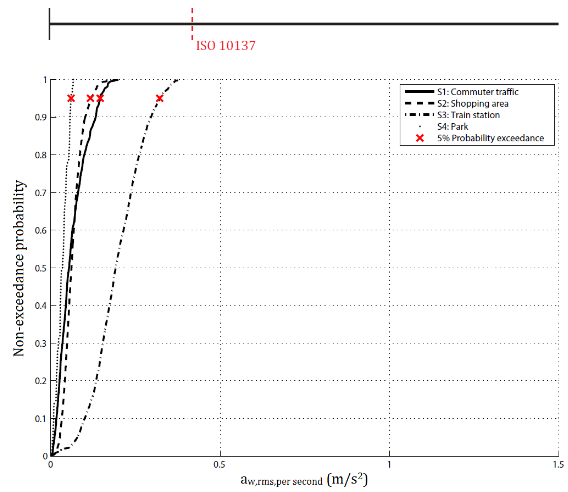


Figure 12.26: Cumulative density functions of the expected r.m.s. per second of the weighted accelerations per single pedestrian for the scenarios based on group formation and L = 50 m

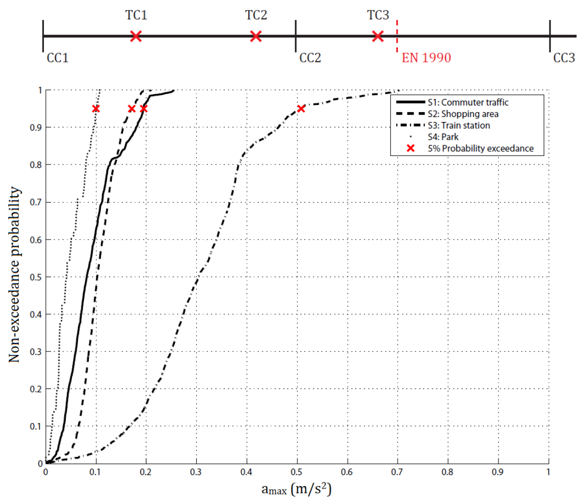


Figure 12.27: Cumulative density functions of the maximum expected accelerations per single pedestrian for the scenarios based on group formation and L = 100 m

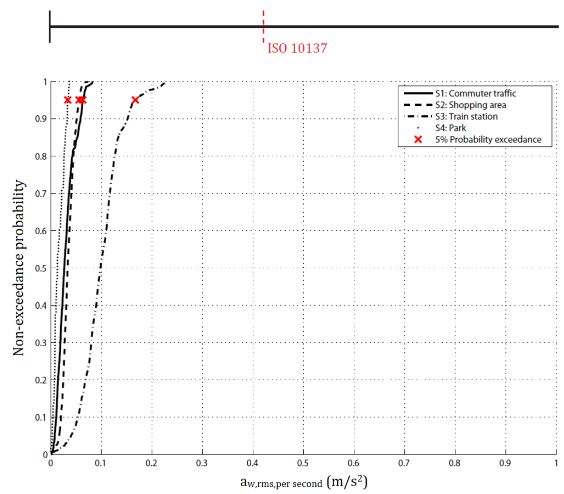


Figure 12.28: Cumulative density functions of the expected r.m.s. per second of the weighted accelerations per single pedestrian for the scenarios based on group formation and L = 100 m

12.5. Discussion of the results

The model uncertainty about the statistical nature of the incoming pedestrian crowd of the introduced probabilistic approach is expected to be high due to the assumptions that had to be made. Nevertheless, the approach shows the feasibility of the statistical character in the evaluation of the scenarios and the differences between the scenarios. It is expected that the approach can be extended to a more realistic and useful tool when additional data and information on incoming pedestrian traffic for different situations is available. Group sizes, group densities, probability of occurrence of the different group sizes and the time interval between groups should therefore be considered.

The scenarios based on densities make use of the combination of a number of continuous flows of pedestrian, of which each flow is modelled in one density. Within this density there is small group formation, but in general the crowd density is continuous. In reality pedestrians do not walk in a continuous flow. Therefore it is expected that the scenarios based on group formation give a more realistic approximation in this evaluation.

The results are based on a relatively short model time. Therefore they do give an idea of the expected normal traffic, but yearly events for example are not taken into account. In order to get a more realistic approximation of the expected traffic and corresponding accelerations the model time should be extended to one year.

The feeling of discomfort is related to the perception of humans. The current relation between the maximum expected acceleration of the bridge in the steady state and the assessment of the discomfort with a limit for individual pedestrians is therefore expected to give divergent results. Considering the bridge accelerations which a pedestrian experiences when crossing the bridge, in relation with the same assessment of discomfort, seems therefore more realistic. In addition there is a difference between the acceleration of the bridge at the position of the pedestrian and the accelerations a pedestrian really experiences, due to the damping effect of the human body itself. The current criteria for (dis)comfort are set up based on the accelerations of the bridge as well instead of on the accelerations a pedestrian experiences. Therefore in general the current evaluation method seems insufficient. However, using the combination of a criterion and a measurement method both based on the bridge accelerations seems sufficient for practice.

One of the proposed measurement methods of acceleration in relation to comfort is the r.m.s. of the weighted acceleration over the time the pedestrian is crossing the bridge (ISO 2361-1). The r.m.s. value can be determined only after the pedestrian has crossed the bridge and not during the passing. The evaluation of the feeling of discomfort takes place at the moment the pedestrian gets off the bridge, while in reality it is expected that a person can already feel uncomfortable half way the bridge. Therefore this evaluation method seems to be more realistic when the r.m.s. is measured during the crossing: from the moment the pedestrian enters the bridges till the point the pedestrian is at that moment in time. In this way, the level of comfort can be evaluated during the crossing.

In addition to the decrease of the expected densities with an increase in length, a decrease in the acceleration limit for comfort is expected as well. The same accelerations at a short and at a large bridge are expected to be more uncomfortable at the large bridge, as the exposure time to vibrations increases with the length of the bridge. Furthermore, in case a pedestrian perceives high vibrations when he is at the middle of the bridge, it takes more time to get off. So, both the crowd density and the comfort limit are expected to be lower for a larger footbridge. The result of the evaluation of human-induced vibrations in footbridges for a large footbridge could therefore end up in the same range of probability of exceedance as for the short bridge. This can only be further studied when more detailed information is available for both the expected pedestrian traffic and the comfort criteria depending on the bridge length.

The Eurocode approach for the evaluation of human comfort in footbridges is based on a 5% exceedance level. Therefore, in the determination of the probabilities this exceedance level of 5% is used in order to compare the results of the probability-based approach with the results of the Eurocode approach. However, usually a probability of exceedance of 1% is used in the SLS for reversible damage.

Expectation has an important role in the evaluation of (dis)comfort. Instead of dealing with this problem from a bridge-engineering perspective, adjustments could be made as well in the awareness and knowledge of people about vibrations of footbridges. On the other hand, this awareness could also result in a greater fear in the use of footbridges.

13 CONCLUSIONS

In the first part of this work the bottlenecks in the design of footbridges have been studied with the aim to provide an overview of the critical aspects in the evaluation of human-induced vibrations in footbridges. This overview is made using literature, discussions with engineering companies and an impact study. The second part focusses on the question whether a probability-based approach can demonstrate that the Eurocode is conservative in the evaluation of human-induced vibrations in footbridges. A probability-based analysis has been performed for the vibration serviceability of footbridges in the vertical direction and implemented in a case study with three simply supported footbridges. To provide a realistic approximation of the regular pedestrian traffic, crowds have been modelled based on four assumed scenarios: commuter traffic, shopping area, train station and park. In this study the maximum acceleration an individual pedestrian experiences when crossing the bridge has been examined, unlike previous studies, where the maximum acceleration of the footbridge in the steady state is considered.

Concerning the main research question of this study, the following can be concluded:

1. The conducted research based on assumptions for the pedestrian traffic and a fixed criterion for human comfort, gives valuable insight in the use of a more realistic evaluation of human-induced vibrations in footbridges. This insight has been obtained for the incoming pedestrian traffic and the exposure to vibrations for individual pedestrians. The adopted probability-based approach contributes to demonstrate potential conservatism in the Eurocode regarding the evaluation of human-induced vibrations in footbridges, in case damage to the structure caused by vandals and high bridge accelerations that can trigger panic can be excluded. This is achieved by considering the actual accelerations that individual pedestrians within a crowd perceive and taking better account of the actual expected pedestrian traffic.

2. It is not possible to give a final answer to the question if the adopted probabilistic-approach can demonstrate whether the Eurocode is conservative or not in the evaluation of human-induced vibrations in footbridges. This is due to a lack of data about the actual incoming pedestrian traffic on footbridges and due to outstanding questions regarding the type of accelerations to be considered for measuring human comfort on footbridges. Because no extensive datasets of actual pedestrian traffic on footbridges are available, the scenarios used in this study are based on an as good as possible assumption of the actual pedestrian traffic. For the evaluation of human comfort a fixed criterion had to be used instead of a probability distribution, as information about this distribution is not sufficient.

Regarding some key aspects in the evaluation of human-induced vibrations in footbridges, this research has shown the following:

- The first important aspect this study showed is a distinction between two main types of pedestrian traffic: normal traffic comprising the commuter traffic, shopping area and park; and special locations with large groups of pedestrians walking in a high density, such as the train station. Whereas the Eurocode prescribes pedestrian traffic with a density of 0.5 P/m^2, this study suggests a density for normal traffic of about 0.3 P/m^2 based on a 95% non-exceedance level. For special locations on the other hand this density is expected to exceed 1.0 P/m^2 considerably.

- Secondly, this study infers a dependency of the length of the footbridge for the expected pedestrian traffic, in contradiction to the fixed value prescribed by the Eurocode. With an increasing length of the footbridge the expected traffic density decreased significantly. For bridges with a span of 12 to 50 m the expected traffic density reduced in the order of 50%; a similar result was seen from 50 to 100 m.
- Further, a significant reduction of 40% in the expected maximum acceleration is found, when considering the maximum acceleration an individual pedestrian experiences when crossing the bridge instead of the maximum acceleration of the footbridge in the steady state. This suggests that the use of the maximum expected acceleration of the bridge in the steady state as a measure for human comfort is conservative, compared to the accelerations that an individual pedestrian is expected to perceive.
- Finally, this study showed that the two current available criteria for the measure of human comfort on vibrating footbridges, based on the maximum acceleration and the root mean square per second of the weighted acceleration, do not correspond with each other, despite the high correlation between the measure methods. This indicates that more research to the criteria and measure of comfort is necessary.

In order to provide a correct answer to the main question, the combination of these above mentioned aspects should be taken into consideration.

14

RECOMMENDATIONS

During this work several issues have been encountered that are not yet covered, but which are of importance to improve the knowledge and standards regarding the evaluation of human-induced vibrations in footbridges. The first three recommendations follow from the investigation and impact study of the critical aspects discussed in part I. The other recommendations emerge from the probability-based approach and case study conducted in part II.

Vandal loading

The check for vandal loading is expected to be of importance for short footbridges and especially for footbridges of new lightweight materials. The load caused by people jumping with the purpose of damaging the structure is governing in the design of short bridges compared to the current load model in the Eurocode (a dense crowd in combination with 50 % of the structural damping). It is expected to be a more realistic representation of vandal loading as well. Therefore it is recommended to adjust the load model for vandal loading in the standards by taking into account a number of people jumping in the natural frequency of the bridge. With experiments the number of people that is capable to jump synchronous and the timespan people can keep the rhythm can be determined. The result of these experiments can be transformed into a load model, applicable in the design of footbridges by engineering companies.

Structural damping

The impact of structural damping in the response is significantly large, which makes it an interesting and important topic for research. In engineering structures in general damping is difficult to capture. Due to its large impact it is favourable to predict the structural damping during the design stage. Therefore it is recommended to measure the damping of a significant number of current footbridges and in addition measure the structural damping of forthcoming footbridges during construction in different stages. The measurements on current footbridges can give an overview of the structural damping parameters for different types of structures, making it possible to predict the damping better in the future. The measurements during construction of forthcoming footbridges can give insight in the contribution of individual elements of the structure to the total damping parameter.

Additional mass and damping by pedestrians

The possibility of a change in properties of the total structural system due to the human-induced loading is expected to have a substantial impact, as a result of interaction between the structure and the pedestrian loading. It is therefore recommended to consider the additional mass and damping by pedestrians in the design of footbridges. The EUR 23984 advises already to take into account the additional mass of the pedestrians, if this increases the modal mass with more than 5%. To take into account the modification in the damping of the total structure, further research to this change in damping due to an individual pedestrian, but more important due to crowd loading should be performed. The combination of a usable load model and verifications of this model with experiments is therefore of importance, in order to get a realistic model that is applicable in the design of footbridges.

Safety assessment

The safety of a structure should be guaranteed before a probability-based approach may be applied. The importance of vandal loading for short lightweight bridges is already discussed. Furthermore, the possibility of panic is recommended to consider. Though panic can be caused by many other aspects than the vibration of the structure, a safe design to prevent extremely high vibrations that will for sure cause panic is of importance. Similar to studies on grandstands, experiments with footbridges can be carried out. By exciting the footbridge to different acceleration signals and at the same time measure the response of people on the footbridge to these vibrations, an indication of a limit level expressed in acceleration for panic can be investigated.

Pedestrian traffic

A more extensive study on incoming pedestrian traffic is of high importance, to be able to demonstrate whether the Eurocode is conservative in the evaluation of human-induced vibrations. As a result from the case study it turned out that the difference between scenarios based on densities and scenarios based on group formation is large. This indicates the importance of a more realistic model for the pedestrian loading that is not based on a continuous flow. To describe the incoming pedestrian traffic data on the group sizes, the probability distribution of groups and the (time) distance between groups is needed. It is recommended to obtain this data by the use of a smart camera system. This system should be able to distinguish pedestrians from cyclists: though cyclists are not considered in this study, in reality most footbridges allow cyclists as well. In addition to only count the pedestrians, this system should be able to store the time of arrival of the pedestrians. The information of the arrival time of pedestrians can be used to extract the group size, the (time) distance between groups and the probability distribution of the expected group sizes. In order to get a realistic overview of the expected traffic for different situations, these measurements should be performed on a significant number of bridges in different situations for a significant time interval of for example one year, to be sure "special events" are measured as well. The obtained extensive dataset about pedestrian traffic can be used to indicate whether the Eurocode is conservative in the prescribed pedestrian loading. Furthermore it can give more guidance in the choice for a project specific pedestrian loading.

In addition to the differences between the scenarios, there is a bridge length dependency in the expected traffic density. For the static loading of a footbridge, the Eurocode prescribes a length dependent loading based on a linear relation. It is recommended to see what kind of length dependent relation can be applied for the dynamic loading. This relation can be obtained by modelling the expected incoming pedestrian traffic, based on the above mentioned data, for a number of bridges with different dimensions.

The current load models for the prediction of the maximum accelerations of the bridge under pedestrian loading are based on Monte Carlo simulations. The relation between reality and the models for single pedestrians is extensively studied, but for the crowd behaviour and response of the bridge hardly any connection is made. To make the translation from model to reality for crowd loading, it is recommended to measure both the accelerations of the bridge at the normative position as well, in addition to the measurement of the pedestrian traffic. This can be used to improve the relation between the pedestrian loading of crowds and the response of the footbridge. The combination of models and experimental data makes it possible to provide a better approximation of the human-induced vibrations and therefore results in more efficient designs.

A last remark on pedestrian traffic is the recommendation to make a connection between the knowledge about the modelling of a single pedestrian and the knowledge of crowd behaviour. The loading for a single pedestrian is studied extensively and in recent years, many studies on human-structure interaction have been performed as well. In the field of crowd behaviour, studies about the physical possibility of walking in crowds and the interaction between persons within a crowd are of interest for modelling pedestrian loading on footbridges. The connection of both fields of study is expected to give a more realistic approximation of pedestrian loading on footbridges than the current models.

Human comfort

In the evaluation of human comfort in the use of footbridges there are some outstanding questions about the type of acceleration that should be considered. Therefore it is recommended to conduct further investigation to determine which measure type of acceleration and corresponding comfort criterion gives the best representation for the evaluation of human comfort in the use of footbridges. It is advised to combine knowledge about the human comfort in other fields of study and the human vestibular system, with experiments exposing walking persons to different types of acceleration signals.

To complete the evaluation of the probability of (dis)comfort regarding human-induced vibrations in footbridges, the probability distribution of human (dis)comfort should be determined as well. If a general shape of the probability distribution can be determined, this can be connected to the comfort criteria for different situations. The probability distribution of human (dis)comfort and the can be obtained by several experiments where a significant number of random pedestrians is exposed to the same acceleration signal when walking. By measuring the level of comfort for numerous different acceleration signals, a probability distribution for human discomfort can be set up. To measure the level of (dis)comfort a gradual scale like the one proposed by Hawryszków (figure E.1 in appendix E) is recommended.

Comfort is a human specific measure and should therefore be combined with the accelerations pedestrians experience when crossing a footbridge, instead of the steady state response of the bridge deck. The reduction is expected maximum acceleration is significant as well. It is therefore recommended to take into account the accelerations individual pedestrians are expected to perceive when crossing the bridge.

Human comfort is a complex phenomenon, containing diverse fields of study. At the moment the evaluation of footbridges with respect to comfort is only based on the accelerations of the bridge deck and the direction of vibration. In other fields of study additional parameters are taken into account as well, resulting in significant differences between comfort criteria for different situations. Therefore it is recommended to implement situation depending comfort criteria for footbridges in the codes and standards. To be able to make this implementation first a study on the comfort criteria for walking people for different situations should be performed. Whereas the feeling of discomfort for sitting, lying and standing people is quite well studied in the past, comfort criteria related to walking people need more attention in research.

Model time

The determination of the accelerations which pedestrians experience, is performed by modelling pedestrians crossing a bridge and evaluating all the accelerations signals of the individual pedestrians. In order to get more realistic statistical results, taking for example into account the occurrence of special events, it is advised to extend the model time to a year. By modelling a whole year all the normal traffic including yearly events is considered in the analysis, giving a realistic approximation of the expected accelerations. To reduce the computation time, it is advised to use parallel processing whereby the processes that are independent can be calculated parallel to each other. These processes include the bridges considered, different scenarios and several runs. In addition to parallel processing, it is recommended to save the data within the script immediately after calculation, to prevent that the computer runs out of memory.

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III

APPENDICES

A

Company visits

In the first part of the research project several companies have been visit to talk about the critical issues in the design of footbridges. These meetings have been of influence to make a choice for a relevant topic for the main part of the master thesis. The meetings are concluded in a table, the accompanying text provides additional explanations.

A.1. ARUP

The following people were present at the meeting on the 9th of April 2014 at the ARUP office in Amsterdam:

Beatriz Zapico Blanco	ARUP
Demis Karagiannis	ARUP
Joep Paulissen	TNO
Zinzi Reimert	TNO (*graduate*)

General

Demis performed his graduation project on parameters of a model for the lateral vibrations on footbridges. He was the successor of Joep. He is now researcher at ETA Zurich in pedestrian and concrete bridges. Beatriz studied industrial engineering and did a PhD in Spain in bridge engineering. She now works on the B500 project at Arup, about earthquakes in Groningen.

Arup, a university in Spain and the university of Warwick in England collaborate in a research project together. The university of Spain is specialised in the vertical direction and Arup in the lateral direction. Within Arup, Demis takes care of the lateral and Beatriz of the vertical direction. The statistical walking pattern from a single pedestrian will investigated first, than a model is made for the interaction of the pedestrians and the bridge structure. For the vertical direction pedestrians add damping and energy to the bridge, but the question is if there is a limitation in the vibrations due to this additional damping? Experiments for the project will be carried out in Spain and Arup will investigate the statistical part of the experiments. They are searching for more people/companies to cooperate with, in order to get a European fund. The hypotheses of the research project is "Vertical synchronisation is not possible, there will be a limit in the added energy to the system by pedestrians".

Arup uses the suggested values from the EUR 23984 in the design. They built three bridges, one at the Amsterdam Rijnkanaal (Nesciobridge) and two others. In all cases Arup was involved in the preliminary design. Vibrations in structures in general should be taken into account, for example in case of bridges inside buildings. Light designs and aesthetics are important for Arup in the design.

Level of comfort & traffic class

The client is not familiar with the choice for the levels of comfort and traffic classes, the different options have no meaning to him. It is part of the job of the engineering company to educate the client to become familiar with the different choices and to know by conversation what the client needs/wants. It is a discussion with the client, in which the preferences of the client should be raked up.

Comfort is dependent on the circumstances, like the location of the bridge, as well. It is part of the conversation between the client and the engineer to determine the right combination levels and classes which should be checked. The probability of occurrence is part of this conversation as well, the engineer should be able to make an estimation of the change of occurrence and the risks. It would be favourable to have pictures with scenarios to make this choice, for the engineer to show the client.

Special events

At the company bridges are not checked for special events. It is expected that it is not possible to have a denser traffic load model than TC5. Music or a marathon on the other hand could have a big influence on the vibrations of the structure. In case it is likely these situations occur, which is part of the conversation between the client and engineer, they should be checked. The manner in which this should be done is not prescribed. A major part of this topic is the likelihood particular events occur. Part of the education of the client could be scenario picking what the bridge should be checked for, based on the risks and probability of occurrence.

Vandalism

At the company bridges are not checked for vandalism. It would be favourable to be able to check is the ultimate limit state (ULS) of the structure can be reached. In the future it can be more relevant in relation with the more slender bridges and new materials. Slenderness is the main topic for the behaviour of the bridges and length & slenderness are related. Problems with vandalism are therefore expected to be more likely for short bridges.

Probability of occurrence in combination with the proposed load models is an important part of this topic as well. A long bridge will almost never be excited to TC 5 and an accidental load like jumping people will be closer to reality, on the other hand there are limitations to vandalism because of the people themselves: for how long can they maintain their action, will they succeed in jumping in exactly the same frequency and phase, etc. The use of influence line depending on the length of the structure would be favourable.

Synchronization & structure-pedestrian interaction

In the lateral direction pedestrians are only adding energy to the system, in the vertical direction both energy and damping can be added by pedestrians. For the interaction model of the research project it is expected that there is a upper limit in the added energy to the system from the pedestrians. If this is the case this limit can be used in further design. Crowd simulations will be performed to study this field of research.

It is possible that the design nowadays are too conservative in case of for example steel, on the other hand it is possible that it is the other way around for new materials. The interaction between bridge and pedestrian is not into account in the current regulations. According to Beatriz vertical synchronization is not possible, but the EUR 23984 [6] mentions that experiments have shown that vertical synchronization can occur at an acceleration of 1,5 m/s^2.

Structural damping

It is not possible to determine the damping in the design phase. The current method to deal with it is leaving space for the dampers during the design and after construction the accelerations will be measured and if necessary dampers will be added and tuned. Dampers are 0.3% of the total costs of the structure. The problem with damping is the non-linear behaviour and the change in properties by pedestrians walking across the bridge. A better understanding of this phenomena would be favourable. For future slender structures and the architectural design it would be beneficial if it is not necessary to put dampers in the construction.

Topic	**Problem ARUP**	**Importance ARUP**	**Problem Personal/ TNO**	**Research**	**Considerations**
Current guidelines	- -	-	-	-	• Aware of usage
Level comfort / traffic class	+/-	+	+	+	• Education client • Guidance for conversation client favourable • Influence diagrams
Special events	-	-	?	+/-	• Part of work engineer to consider it in the design
Vandalism	+/-	+	?	+/-	• Part of work engineer to consider it in the design • More likely for small bridges • No check in design at this moment
Synchronization	+ & -	++	+ & -	++	• Vertical lock-in not possible • Lateral only energy input • Vertical energy and damping input: maximum level • Effect can be positive or negative • Additional damping pedestrians
Damping	++	++	++	++	• Adding dampers after construction not favourable • Assumption damping
Future bridges	does not apply		does not apply	+	• New materials • Favourable to take into account for research

Table A.1: Overview meeting ARUP

A.2. RoyalHaskoningDHV

The following people were present at the meeting on the 29th of April 2014 at the office of RoyalHaskoningDHV (RHDHV) in Rotterdam.

Liesbeth Tromp	RoyalHaskoningDHV
Joep Paulissen	TNO
Zinzi Reimert	TNO (*graduate*)

General

Liesbeth has a masters in Aerospace engineering, worked at TNO and is now part of RoyalHaskoningDHV in the field of composite structures and in particular FRP (fibre-reinforced plastics). She is part of the committee to (re)write a guideline for composite structures.

FRP is light and strong, but less stiff compared to steel or concrete. It provides a high degree of freedom in the design, which can be used to create more stiffness. For the design of FRP bridges in the code only the comfort criteria has to be fulfilled. For the deflection L/300 is used, according to the old code NEN 6700, but this is not a requirement, unless the client marks it as a requirement.

RoyalHaskoningDHV has carried out several project of footbridges in FRP. The material is not common yet in the building industry, because of which they encounter problems in the design phase. In general the question is which assumptions can be made and how to use the guidelines. In general the SLS has to be fulfilled in combination with the comfort criteria, but questions like what if an ambulance crosses the bridge: should I use the ULS or should there be a comfort criterion as well? arise.

RoyalHaskoningDHV uses the Eurocode and the EUR 23984, HiVoSS and Sétra as guidelines for the design of footbridges. For them the difficulties in the guidelines is in the definitions: it is not clear to them what to use, for example in the case of the maximum acceleration. In general the FRP footbridges have an natural frequency in the critical range of pedestrians (in this case it is common between 2-3 Hz) and have to be checked dynamically.

FRP bridges have an aging effect which has to be taken into account as well: the stiffness of the bridge and supports can change, this can cause a different natural frequency. Due to the nature of light structures, the additional mass of pedestrians can have a major impact, therefore this is taken into account in the design.

Level of comfort & traffic class

The levels of comfort and the traffic classes as they are provided by the EUR 23984 are presented to the client. RoyalHaskoningDHV shows the options and the consequences for the design, costs and comfort to the client. In relation with the probability of occurrence of certain traffic classes, the risks involved, the location of the bridge, the type of bridge the expectations of the users, etc. RoyalHaskoningDHV will advice the client about the choice.

The engineers at RoyalHaskoningDHV have a feeling for the differences between the levels of comfort and traffic classes by comparing it to the static analysis and different guidelines they use. If they encounter a problem in the design phase, for example the bridge does not satisfy for the jogger load case, this will be discussed with the client, but not necessarily solved.

Special events

The possibility of special events is in general not taken into account. In case the client indicates the probability of occurrence of special events on the bridge, these events can be checked.

Vandalism

FRP footbridges are not checked to vandalism. The supplier assumes vandalism is not critical for FRP footbridges, but no tests are carried out to prove this. In practice to RoyalHaskoningDHV it is not clear how to deal with the load case of vandalism and what is conservative. Vandalism in the case of jumping people on the bridge could cause high stresses in the structure. Testing this case would be favourable to be able to tell the client is this will be a point of attention.

Fatigue is not an issue for FRP footbridges. The slope of the fatigue is very flat for FRP, because of which it will not occur in structures, with the exception that it is possible for the stresses over the thickness. The connections are more sensitive to fatigue and are of concern.

Synchronization

The possibility of synchronization is not part of the design process for RoyalHaskoningDHV. They do use the fact that pedestrians add mass to the structure, which can cause a lower natural frequency. This aspect is taken into account in the design.

Structural damping

The damping of FRP bridges is dependent on the layout of the bridge, but is in general higher in comparison to steel or concrete bridges. In the most unfavourable case the damping is 1%, in the most favourable case 2%. This value has been derived by comparison with other materials (like wood) and a few research studies. In reality the connections will change the structural damping of the bridge.

There is lack of experience with FRP footbridges to be able to determine the exact (in general difficult) damping. There have been no measurements to the built bridges. The supplier does check the frequencies, but not the damping. In case of the relative small footbridges which have been built up to now it is not that much of a problem, but for future longer bridges it will be favourable to know the exact properties. Due to the more homogeneous structure it will be possible to make a more precise determination of the structural damping compared to steel bridges.

Future bridges

It is expected that the bridges will become larger and more complex in the future. Not only functional, but more architectural designs will match the properties of the FRP. FRP will be used for longer footbridges, cantilevered sections, renovation projects and road bridges. The risks involved with the future bridges will be become higher, more experience with the material is favourable to be able to fully understand the material and its properties.

Topic	Problem RHDHV	Importance RHDHV	Problem Personal/ TNO	Research	Considerations
Current guidelines	++	++	+	-	• Difficulties with use
Level comfort / traffic class	-	- -	-	-	• Tables EUR are used • Advice client • External conditions taken into account • Jogger load case is issue
Special events	+/-	-	+	+	• Not checked in design • Conversation with client • Comfort instead of ULS problem
Vandalism	+	+/-	++	+	• Not checked in design • Could become an issue
Synchronization	?	?	?	?	• Lock-in phenomenon is not checked • Additional mass pedestrians is used in design
Damping	++	+	++	++	• Assumption compared to other materials • Not checked afterwards • Limited research • Becomes more important for large bridges
Future bridges	++	++	+	+	• Larger and more complex designs • Knowledge not yet available • More research is favourable

Table A.2: Overview meeting RoyalHaskoningDHV

A.3. Gemeentewerken Rotterdam

The following people were present at the meeting on the 2th of June 2014 at the faculty of Civil Engineering in Delft:

Jaco Reusink	Gemeentewerken Rotterdam
Joep Paulissen	TNO
Marcel Buur	Gemeentewerken Rotterdam (*graduate*)
Zinzi Reimert	TNO (*graduate*)

General

The Gemeentewerken Rotterdam expect long pedestrians bridges to be oversized, due to unrealistic high traffic classes that are used in the dynamic design (to fulfil the comfort criterion). A graduate student at the Gemeentewerken Rotterdam made a Matcat program which can be used to determine the extra costs for a project when the bridge fulfils the comfort criteria in addition to the strength criteria. It turned out that for short bridges no extra costs for comfort were needed. These bridges have a ratio self weight to variable loading 1 : 2. In case of long bridges, it becomes economically unfavourable to built them, due to the high additional costs. The ratio self weight to variable loading is 2 : 1.

Level of comfort & traffic class

Comfort

The duration of the accelerations is of importance in the perception of comfort. At this moment the peak values of the acceleration are measured and used to determine the comfort level (a_{max}), but these peaks don't cause discomfort. A present design from the Gemeentewerken Rotterdam showed that if you tell people before they enter the bridge that it is possible that the bridge is going to vibrate they feel already safe(r). For humans discomfort is caused by a (sea)sick feeling, due to too high accelerations. The moment a person starts feeling sick depends on the duration of the high accelerations. Questions arise: When does a person feel sick, at the first peak already? Or after 10 seconds of high accelerations?

Comfort should be specified in the EUR 23984 with for example joggers as separate load model and exposure time as an input parameter. An issue is the many input parameters that can be taken into account when determining comfort (for example length of the bridge, type of traffic and location). Probability curves with a 95% change could be used to set the parameters such that the probability of discomfort for a random passer is 10^{-4}. Comfort is a serviceability limit state and therefore the probability of occurrence is aloud to be higher than at the ultimate limit case.

In addition the probability of the number of people per square meter in reality at the bridge is not taken into account. The traffic classes presented in the EUR 23984 are realistic for short bridges, but in case of long bridges the value seem to be way too high. Every load case provided is, in case of a long bridge, already a special event. In case of the static calculations the bridge is safe due to this traffic classes, but in reality the number of people present on the bridge is much lower and therefore the assumption is that the long bridges are dimensioned unnecessary heavy.

Structural damping

It is not that hard to determine the structural damping. Damping is built up from the energy dissipation of the structure / structural elements. Cable-stayed bridges have almost no damping, the same applies for monolithic bridges. The value used for steel bridges (0.4%) is a lower bound. In case of short bridges this value can be reached, but usually the damping is higher due to for example friction in the connections.

At this moment a few bridges are designed in which a tuned mass damper had to be placed to fulfil the comfort criteria, but in reality the TMDs are left out. These bridges had no complains about vibrations, in combination with the maintenance necessary for TMD it was decided to not use them. For high amplitudes the structural damping increases, this is not yet taken into account though.

Synchronization & structure-pedestrian interaction

If a bridge has a natural frequency just higher than the critical range of frequencies it is taken into account that pedestrians add mass and can therefore reduce the natural frequency of the structure to the critical range. In this case the structure should be checked to dynamics. The possibility of added damping by pedestrians is recognized, but for the Gemeentewerken Rotterdam this topic has no priority.

Special events

The current traffic classes are already some kind of special events in case of long bridges, looking at the amount of people present on the bridge. On the other hand events like a marathon are special load cases, that can cause higher vibrations. These should be covered within the incidental load cases, but not as a normal traffic class. Depending on the use of the bridge and the probability of occurrence of such a special event in the life time of a bridge, the bridge can be designed to provide a certain comfort level in case of this special event.

Vandalism

Especially for long (steel) bridges it is impossible to reach the ultimate limit state by vandal loading. People have to stop exciting the bridge at accelerations of about 5-10 m/s^2, because of extreme discomfort, where accelerations of 50 m/s^2 are necessary to reach the maximum stress. When people stop exciting the bridge it will automatically mute off.

Another phenomenon is that at high amplitudes the structural damping increases, which makes it even harder to reach the ULS. A new type of bridges is made of fibre-reinforces polymers. These are light bridges, so in case the natural frequency of the bridge is in the critical range of the pedestrians it is likely that vandals can cause high vibrations/accelerations. FRP bridges are though designed with a factor five to ten to strength and therefore in case vandals want to reach the ULS of the bridge they have to reach an even higher level that this factor. This seems impossible, therefore it is not likely that (at this moment) these bridges are at risk in case of vandal loading. It could become a problem when these bridges are designed with a lower safety factor.

In general people are able to walk in stride for high excitations. The question is at which acceleration level pedestrians can not keep their step any more and if it is possible to reach the ULS at this acceleration level.

Future bridges

Fibre-reinforced polymer (FRP) is and will be used as the major material for footbridges the coming years. At this moment the FRP bridges are over dimensioned with a safety factor five to ten to avoid problems. FRP bridges are lively structures. If the natural frequency is in the critical range it is likely to have high accelerations.

No maintenance can take place to FRP bridges. The industry needs a norm verification to be able to design the bridges for 50 to 100 years. At this moment almost all the new pedestrian/cycling bridges are built in FRP. It would be favourable to have correct comfort regulations, not only for the FRP bridges from the company FiberCore, but also for other new materials similar like this.

Topic	**Problem GR**	**Importance GR**	**Problem Personal/ TNO**	**Research**	**Considerations**
Current guidelines	+	+/-	+	+	• EUR 23984 is not a real guideline • Favourable to improve
Level comfort / traffic class	++	++	++	++	• Traffic classes unrealistic high for long bridges • Duration of accelerations • Peak values not realistic to use
Special events	+	-	+	-	• Incidental load case • Probability has to be taken into account
Vandalism	-	-	+/-	+/-	• Now no problem • Maybe interesting future FRP bridges
Synchronization	+	-	+	+	• Added mass taken into account • Interesting, but not a problem
Damping	-	-	+	+/-	• No problem to estimate • Higher amplitude results in more damping
Future bridges	+	+	+	+	• Large safety factor • Code not yet valid

Table A.3: Overview meeting Gemeentewerken Rotterdam

B

MODAL ANALYSIS

B.1. Equation of motion

The accelerations of the bridge deck can be calculated with use of modal analysis.[19] The essence of modal analysis is the assumption of a summation of synchronised motions: the response of a structure can be expanded in mode shapes each weighted with a corresponding time function.[18] For the analysis in the appendix use is made of *Dynamics of structures* by Chopra.[19] The equation of motion for a single span Euler-Bernoulli beam structure with a constant mass m, stiffness EI and damping c is presented in equation B.1, as discussed in section 10.4:

$$m\frac{\partial^2 u(x,t)}{\partial t^2} + EI\frac{\partial^4 u(x,t)}{\partial x^4}) + c\frac{\partial u(x,t)}{\partial t} = p(x,t) \tag{B.1}$$

The dynamic pedestrian loading on the footbridge is represented by $p(x,t)$ and $u(x,t)$ is the deflection of the structure.

B.2. Orthogonality

Orthogonality indicates that the mode shapes are uncorrelated or independent, allowing to calculate the mode shapes separately. In this case the deflection of the structure is represented by the summation of all the mode shapes. First the solution for the deflection is assumed to be the multiplication of a function dependent on time and a function dependent on the position:

$$u(x,t) = \phi(x)\cdot u(t) \tag{B.2}$$

Substitution of the expression B.2 into equation B.1 and considering the free vibration results in:

$$m\cdot\phi(x)\cdot\ddot{u}(t) + EI\cdot\phi''''(x)\cdot u(t) + c\cdot\phi(x)\cdot\dot{u}(t) = 0 \tag{B.3}$$

Equation B.3 is divided by $m\phi(x)u(t)$ and becomes:

$$-\frac{\ddot{u}(t)}{u(t)} - \frac{c}{m}\frac{\dot{u}(t)}{u(t)} = \frac{EI}{m}\frac{\phi''''(x)}{\phi(x)} \tag{B.4}$$

The expression on the left is dependent on only t and the expression on the right is a function of x only. Equation B.4 should be valid for all values of t and x, therefore the two expressions must be equal to a constant, here ω^2 is applied. This results in two ordinary differential equations:

$$\frac{\ddot{u}(t)}{u(t)} + \frac{c}{m}\frac{\dot{u}(t)}{u(t)} = -\omega^2 \tag{B.5a}$$

$$\ddot{u}(t) + \frac{c}{m}\dot{u}(t) + \omega^2 u(t) = 0 \tag{B.5b}$$

$$\frac{EI}{m}\frac{\phi''''(x)}{\phi(x)} = \omega^2 \tag{B.6a}$$

$$EI\phi''''(x) - m\omega^2\phi(x) = 0 \tag{B.6b}$$

The eigenvalue problem is defined by equation B.6b and the boundary conditions for the beam considered. The vibrations modes and natural frequencies can be derived.

The orthogonality properties are now derived. Equation B.6b is the starting point, rewritten for mode r, multiplied by $\phi_n(x)$ and integrated over the length:

$$\int_0^L \phi_n(x) EI\phi_r''''(x)dx = \omega_r^2 \int_0^L \phi_n(x) m\phi_r(x)dx \tag{B.7}$$

With use of integration by parts, the left side of this equation can be rewritten:

$$\begin{aligned}\int_0^L \phi_n(x)EI\phi_r''''(x)dx &= \left[EI\phi_n(x)\phi_r'''(x)\right]_0^L - \int_0^L EI\phi_n'(x)\phi_r'''(x)dx \\ &= \left[EI\phi_n(x)\phi_r'''(x)\right]_0^L - \left[EI\phi_n'(x)\phi_r''(x)\right]_0^L + \int_0^L EI\phi_n''(x)\phi_r''(x)dx\end{aligned} \tag{B.8}$$

The first two terms between the square brackets become zero in case the quantities enclosed are zero at x = 0 and x = L. This is the case for simply supported beams and as well for beams with ends that are free or clamped. Substitution in equation B.7 results in:

$$\int_0^L EI\phi_n''(x)\phi_r''(x)dx = \omega_r^2 \int_0^L \phi_n(x)m\phi_r(x)dx \tag{B.9}$$

Now equation B.6b is written for mode n, multiplied by $\phi_r(x)$ and integrated over the length. Taking the same steps as before, this results in:

$$\int_0^L EI\phi_r''(x)\phi_n''(x)dx = \omega_n^2 \int_0^L \phi_r(x)m\phi_n(x)dx \tag{B.10}$$

Equation B.9 is subtracted from equation B.10:

$$\left(\omega_n^2 - \omega_r^2\right)\int_0^L \phi_r(x)m\phi_n(x)dx = 0 \tag{B.11}$$

If $\omega_n \neq \omega_r$, this can only be true if:

$$\int_0^L \phi_r(x)m\phi_n(x)dx = 0 \tag{B.12}$$

Substitution in equation B.12 gives:

$$\int_0^L \phi_n(x)EI\phi_r''''(x)dx = 0 \tag{B.13}$$

The orthogonality relations for the vibration modes are equations B.12 and B.13. The mode shapes may therefore be calculated independently.

B.3. Modal analysis

The deflection of the beam is given by a superposition of the modes n, when the corresponding eigenvalue problem has been solved:

$$u(x,t) = \sum_{n=1}^{\infty} \phi_n(x) \cdot u_n(t) \tag{B.14}$$

Substitution of equation B.14 in equation B.1 gives the following infinite set of ordinary differential equations:

$$\sum_{n=1}^{\infty} m\phi_n(x)\ddot{u}_n(t) + \sum_{n=1}^{\infty} EI\phi_n''''(x)u_n(t) + \sum_{n=1}^{\infty} c\phi_n(x)\dot{u}_n(t) = p(x,t) \tag{B.15}$$

Multiply each term in this equation with $\phi_r(x)$, integrate over the length of the beam and rearrange the terms:

$$\sum_{n=1}^{\infty} \ddot{u}_n(t)\int_0^L m\phi_n(x)\phi_r(x)dx + \sum_{n=1}^{\infty} u_n(t)\int_0^L EI\phi_n''''(x)\phi_r(x)dx + \sum_{n=1}^{\infty} \dot{u}_n(t)\int_0^L c\phi_n(x)\phi_r(x)dx = \int_0^L p(x,t)\phi_r(x)dx \tag{B.16}$$

Only the terms for which $n = r$ will remain due to the orthogonality properties, resulting in the following equation for a mode n:

$$\ddot{u}_n(t)\int_0^L m(\phi_n(x))^2dx + u_n(t)\int_0^L EI\phi_n''''(x)\phi_n(x)dx + \dot{u}_n(t)\int_0^L c(\phi_n(x))^2dx = \int_0^L p(x,t)\phi_n(x)dx \tag{B.17}$$

Equation B.17 can be rewritten with the modal mass M_n^*, the modal stiffness K_n^*, the modal damping C_n^* and the modal load $P_n^*(t)$:

$$M_n^*\ddot{u}_n(t) + K_n^*u_n(t) + C_n^*\dot{u}_n(t) = P_n^*(t) \tag{B.18}$$

where

$$M_n^* = \int_0^L m(\phi_n(x))^2dx \tag{B.19}$$

$$C_n^* = \int_0^L c(\phi_n(x))^2dx \tag{B.20}$$

$$P_n^* = \int_0^L p(x,t)\cdot\phi_n(x)dx \tag{B.21}$$

The modal stiffness is determined with use of integration by parts:

$$\begin{aligned} K_n^* = \int_0^L EI\phi_n''''(x)\cdot\phi_n(x)dx &= [EI\phi_n'''(x)\phi_n(x)]_0^L - \int_0^L EI\phi_n'''(x)\phi_n'(x)dx \\ &= [EI\phi_n'''(x)\phi_n(x)]_0^L - [EI\phi_n''(x)\phi_n'(x)]_0^L + \int_0^L EI\phi_n''(x)\phi_n''(x)dx \end{aligned} \tag{B.22}$$

The first two terms between the square brackets become zero in case the quantities enclosed are zero at x = 0 and x = L. This is the case for simply supported beams and as well for beams with ends that are free or clamped. Therefore the modal stiffness is:

$$K_n^* = \int_0^L EI\left(\phi_n''(x)\right)^2dx \tag{B.23}$$

C
Calculations impact study

The impact study in chapter 6 is performed with use of the program Excel and the calculations are based on the procedure recommended in the EUR 23984 and the Eurocode. For a detailed description of the procedure, reference is made to the EUR 23984.[6]

C.1. Maximum acceleration

To determine the maximum acceleration of the footbridge under harmonic pedestrian loading the single-degree-of-freedom (SDOF) method is used, as explained in 4.2.6. This results in the following formula for the maximum acceleration, for the steady-state response due to harmonic loading for the considered mode n: [6]

$$a_{max,n} = \frac{p_n^*}{m_n^*}\frac{1}{2\xi} \tag{C.1}$$

The damping ratio is ξ and p_n^* & m_n^* represent respectively the modal load and modal mass. In appendix B is explained how to determine these quantities with the following result:

$$m_n^* = \int_0^L m(\phi_n(x))^2 dx \tag{C.2}$$

$$p_n^* = \int_0^L p(x,t)\phi_n(x) dx \tag{C.3}$$

Modal mass

The factor m, expressed in kg/m, is the sum of the weight of the bridge per meter length and the weight of the pedestrians per meter length. The weight of the pedestrians is only taken into account when their mass is equal to or larger than 5% of the weight of the bridge. Not the entire weight of the pedestrians has to be set in motion, due to the fact that the pedestrians are walking along the bridge instead of stand still. The assumption is made that the weight of the equivalent number of pedestrians is taken into account. The mode shape is represented by $\phi_n(x)$.

Modal load

To determine the maximum acceleration, the maximum modal load should be taken into account, corresponding to expression C.3. For a load case with a distributed flow of pedestrians (traffic classes 1 to 5) the load $p(x,t)$ is represented by the following formula:

$$p(x,t) = p(x)\sin(2\pi f_s t) \tag{C.4}$$

In which $p(x) = Pn'\psi$, a value constant over the length depending on the weight of the pedestrian for the dynamic loading P, the equivalent number of pedestrians n' and the reduction coefficient ψ. The natural frequency of the structure for the mode taken into account is f_s. This results in a maximum modal load of

$p(x,t)_{max} = Pn'\psi$. The generalised load is based on the assumption that each belly of the mode shape is loaded: the load is always acting in the sense of displacements of the bellies. An example is given in figure C.1.

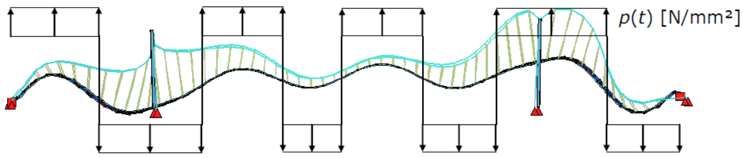

Figure C.1: Application of a harmonic load according to mode shape $\phi(x)$ [6]

In case of vandalism an additional load case with people jumping on the bridge is taken into account. The people will be represented as point loads on a fixed, governing position at the bridge as a function of time:

$$p(x,t) = P\psi\cos(2\pi f_s t)\cdot\delta\left(x-\frac{L}{2}\right) \tag{C.5}$$

The dynamic part of the loading for the considered number of vandals is P and the position of the vandals is represented by $\delta\left(x-\frac{L}{2}\right)$. As an example, the vandal loading for a simply supported beam with mode shapes of the form $\phi(x) = \sin\left(\frac{n\pi x}{L}\right)$ with $n = 1, 3, 5$, etc. is given. In this case, the vandals are assumed to jump in the middle of the bridge, where the maximum acceleration, stresses and deflection will the largest. The result for the maximum generalised load p_n^* is:

$$p_n^* = P\psi\int_0^L \phi_n(x)\cdot\delta\left(x-\frac{L}{2}\right)dx \tag{C.6a}$$

$$= P\psi\phi_n\left(\frac{L}{2}\right) \tag{C.6b}$$

$$= P\psi \tag{C.6c}$$

C.2. Maximum stress

In the design of a footbridge the maximum stress may not reach the ultimate limit state (ULS). In addition, to ensure a realistic design of the footbridges, the maximum deflection is considered as well. Provided that the bridge is designed well for the static case, the dynamic analysis can be performed.

Static

The maximum stress σ_{max} under static loading can be derived with the following formula:

$$\sigma_{max} = \frac{M_{max}}{W} \tag{C.7}$$

Where M_{max} is the maximum moment under static loading and W is the section modulus of the structure. Assuming that the load case with the full load (presented as a distributed load) is governing, the maximum moment can be determined by:

$$M_{max} = \frac{1}{8}q_d l^2 \tag{C.8}$$

In which the load is represented by q_d, based on the combination of the dead load and live load:
$q_d = 1.25\cdot DL + 1.5\cdot LL$.

Dynamic

To determine the maximum stress for the dynamic loading use is made of the SDOF system where the deflection $u(x,t)$ is described with:

$$u(x,t) = \phi(x)\cdot u(t) \tag{C.9}$$

In which $\phi(x)$ is the considered mode shape and $u(t)$ is the corresponding deflection of the beam. If a harmonic load $F_0 \sin(2\pi f_0 t)$ is applied to a damped SDOF system, the response of the system $u(t)$ will follow from the differential equation and result in:

$$u(t) = \frac{F_0/(4\pi^2 m)}{\sqrt{(f^2 - f_0^2)^2 + 4\xi^2 f^2 f_0^2}} \sin(2\pi f_0 t - \varphi) \tag{C.10}$$

Where F_0 is the amplitude of the harmonic load, m is the system mass, f is the system natural frequency, f_0 is the frequency of the harmonic load, ξ is the damping ratio and φ is the phase shift. The maximum stress can now be calculated, with the same equation as for the static case:

$$\sigma_{max} = \frac{M_{max}}{W} \tag{C.11}$$

The moment can be expressed as follows:

$$M_{max} = EI\kappa_{max} \tag{C.12}$$

Where

$$\kappa_{max} = -u''_{max}(x,t) \tag{C.13}$$

and

$$u''_{max}(x,t) = \phi''_{max}(x)u_{max}(t) \tag{C.14}$$

The translation is made to a simply supported beam with a modal force p_n^* and modal mass m_n^* per mode shape. The deflection $u_n(t)$ is at a maximum for $f = f_0$, resulting in:

$$u_{n,max}(t) = \frac{p_n^*}{8\pi^2 m_n^* \xi f^2} \tag{C.15}$$

Now $\phi''_{max}(x)$ can be determined for the nth harmonic:

$$\phi_n(x) = \sin\left(\frac{n\pi x}{L}\right) \tag{C.16a}$$

$$\phi'_n(x) = \frac{n\pi}{L}\cos\left(\frac{n\pi x}{L}\right) \tag{C.16b}$$

$$\phi''_n(x) = -\frac{n^2\pi^2}{L^2}\sin\left(\frac{n\pi x}{L}\right) \tag{C.16c}$$

$$\phi''_{n,max} = -\frac{n^2\pi^2}{L^2} \tag{C.16d}$$

Substitution in equation C.11 results in the maximum stress for dynamic loading:

$$\sigma_{n,max} = \frac{M_{n,max}}{W} = \frac{-EI\phi''_{n,max}u_{n,max}(t)}{W} = \frac{EI\left(\frac{n^2\pi^2}{L^2}\frac{p_n^*}{8\pi^2 m_n^* \xi f^2}\right)}{W} \tag{C.17}$$

C.3. Maximum deflection

The maximum deflection is determined for both the static and the dynamic loading as well. The maximum deflection is only considered for crowd loading, represented by a distributed load.

Static

The maximum deflection u_{max} for the static loading can be determined as follows, considering a distributed load:

$$u_{max} = \frac{5}{384}\frac{qL^4}{EI} \tag{C.18}$$

L is the length of the footbridge and EI is the stiffness. The distributed load is represented by q, based on the combination of the dead load and live load: $q = DL \cdot 1.0 + LL \cdot 1.0$.

Dynamic

The deflection in the dynamic case follows, like the dynamic stress, from u(t) (C.10) with the assumption that $f_0 = f$. This results in the maximum deflection for dynamic loading for the considered mode shape:

$$u_{max} = \phi_{max}(x) \cdot u_{max}(t) = \frac{p^*/(4\pi^2 m^*)}{\sqrt{4\xi^2 f^4}} \tag{C.19}$$

C.4. Jogger

In case of jogger on the bridge the system can described by the following differential equation, assuming the simplification to a single degree of freedom system with a generalised mass (m^*), stiffness (k^*), damping (c^*) and load (p^*):

$$m^* \frac{d^2u(t)}{dt^2} + c^* \frac{du(t)}{dt} + k^* u(t) = p^*(t) \tag{C.20}$$

With the following load (see chapter 4):

$$p^*(t) = P \cos\left(2\pi f t\right) n'\psi \tag{C.21}$$

The system is under critical damped ($0 < \xi < 1$). The solution for the deflection $u(t)$ can be divided into a homogeneous $u_h(t)$ and partial solution $u_p(t)$:

$$u(t) = u_h(t) + u_p(t) \tag{C.22a}$$

$$u_h(t) = -e^{-\xi\omega_0 t} \left(\frac{P_0(k^* - m^*\omega_0^2)\cos(\omega_e t)}{(k^* - m^*\omega_0^2)^2 + (c^*)^2\omega_0^2} + \frac{P_0(\xi(k^* - m^*\omega_0^2) + c^*\omega_0)\sin(\omega_e t)}{\sqrt{1-\xi^2}((k^* - m^*\omega_0^2)^2 + (c^*)^2\omega_0^2)} \right) \tag{C.22b}$$

$$u_p(t) = \frac{P_0(k^* - m^*\omega_0^2)\cos(\omega_0 t)}{(k^* - m^*\omega_0^2)^2 + (c^*)^2\omega_0^2} + \frac{P_0\omega_0 c^* \sin(\omega_0 t)}{(k^* - m^*\omega_0^2)^2 + (c^*)^2\omega_0^2} \tag{C.22c}$$

In which $\omega_0 = \sqrt{\frac{k^*}{m^*}}$, $\omega_e = \omega_0 \cdot \sqrt{1-\xi^2}$ and $P_0 = Pn'\psi$. The first term of the homogeneous solution ($-e^{-\xi\omega_0 t}$) determines the characteristic where the signal goes to the steady state. This term is therefore responsible for a reduced maximum acceleration when the tuning time is considered.

In the Eurocode (4.1.6) and EUR 23984 (4.2.9) two different approaches to determine the acceleration due to joggers are provided. According to the Eurocode the steady-state response of joggers at a fixed position on the bridge has to be determined, while following the EUR 23984 the acceleration for a certain tuning time (the time the joggers are on the bridge) is considered for moving joggers. This moving force is taken into account by assuming a different generalised load ($\frac{2}{\pi} \cdot P_0 \cos\left(2\pi f t\right)$ [6]).

The consequences of considering the tuning time in the determination of the maximum acceleration of the bridge for jogger loading is presented in figure C.2. This figure shows the response in acceleration of the bridge deck for jogger loading for the three different simply supported bridges. The vertical axes have different scales, due to the different responses of the bridges. The red solid line represents the response for the joggers jumping in the middle of the bridge, the blue dotted line indicates the maximum acceleration in case the tuning time is taken into account. The tuning time has the most influence on the expected maximum acceleration for short bridges. Considering both the moving load and the tuning time, the maximum acceleration will decrease even more.

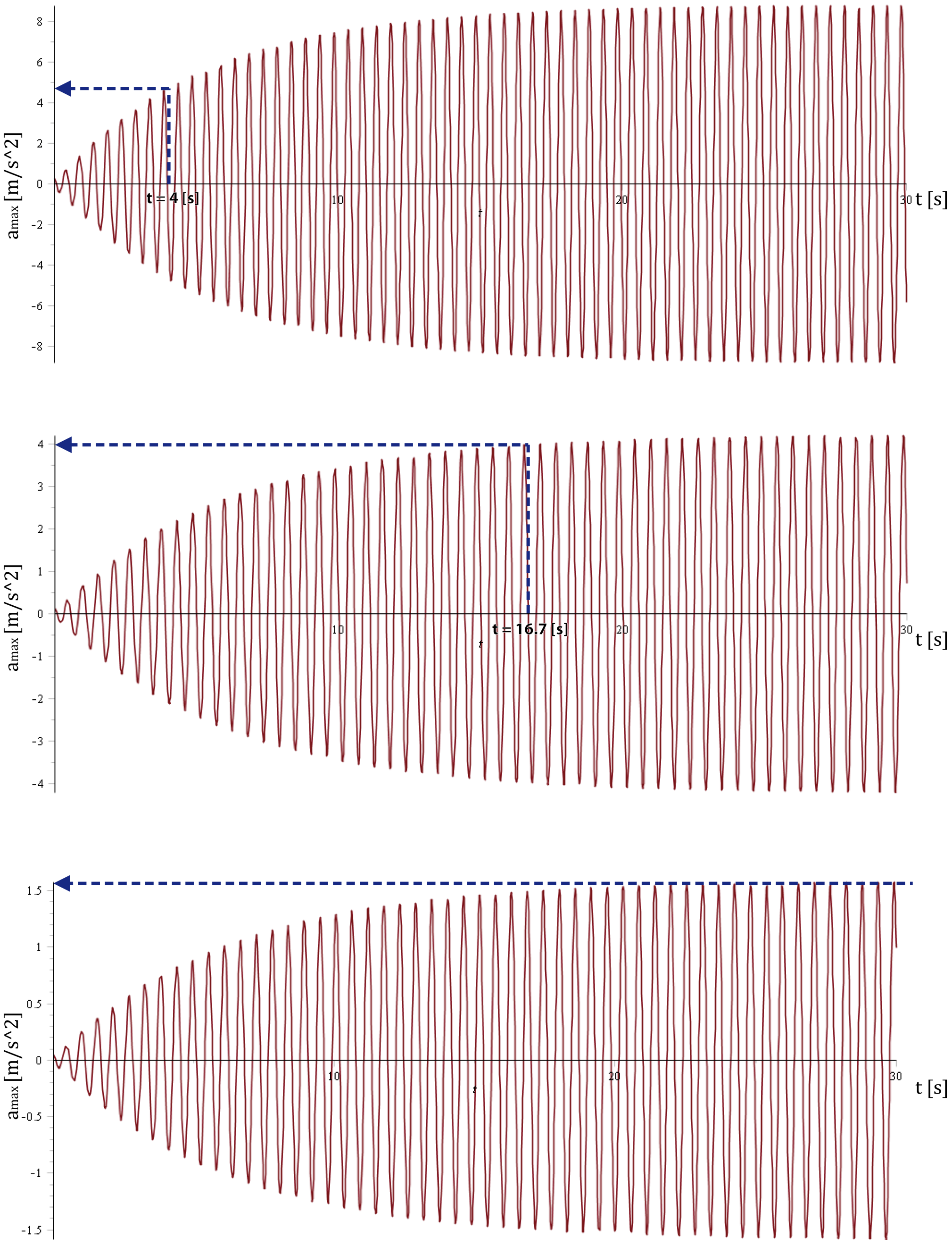

Figure C.2: Response in acceleration of the bridge deck by joggers from top to bottom L = 12, 50 and 100 m

D

CHECK MATLAB MODEL

The case study in chapter 11 is performed by making a model in Matlab to determine the accelerations a pedestrian experiences walking across a footbridge and evaluate these accelerations. In this appendix several checks of the Matlab model are performed and compared to both hand-calculations and the results with the analytical program Maple.

D.1. The case

The checks are performed for one test case: the simply supported footbridge with a length of 50 m. The following properties of the bridge and parameters for the pedestrians are used in all the checks and all the programs.

Bridge properties

n	=	2	Number of mode shapes considered
L	=	50	Length bridge in m
B	=	3	Width bridge in m
m	=	2500	Mass bridge in kg/m
EI	=	$2.05 \cdot 10^{10}$	Stiffness bridge in Nm^2
ξ	=	0.015	Structural damping ratio

Pedestrian parameters

f_p	=	1.8	Frequency pedestrian in Hz
v	=	1.2	Velocity pedestrian in m/s
P	=	700	Weight pedestrian in N
α_1	=	0.4	First Fourier coefficient
α_2	=	0.1	Second Fourier coefficient

D.2. Damping

The bridge has a certain structural damping that is considered in the model. This structural damping can be visualized by starting with an initial deflection of the system (is this case 0.1 m) without any force present. The decline in the amplitude of vibration follows from the damping. The result for the Matlab model is shown in figure D.1. It can be seen that the response of the system is damped out, like expected. To check if this result is correct, a hand-calculation is performed and the result is plotted with the use of Maple in figure D.2. To perform the hand-calculation, the following differential equation without an external force has to be solved:

$$m^* \frac{d^2 u(t)}{dt^2} + c^* \frac{du(t)}{dt} + k^* u(t) = 0 \tag{D.1}$$

With the following initial conditions:

$$u(0) = 0.1 \text{ m} \tag{D.2a}$$

$$\dot{u}(0) = 0 \text{ m} \tag{D.2b}$$

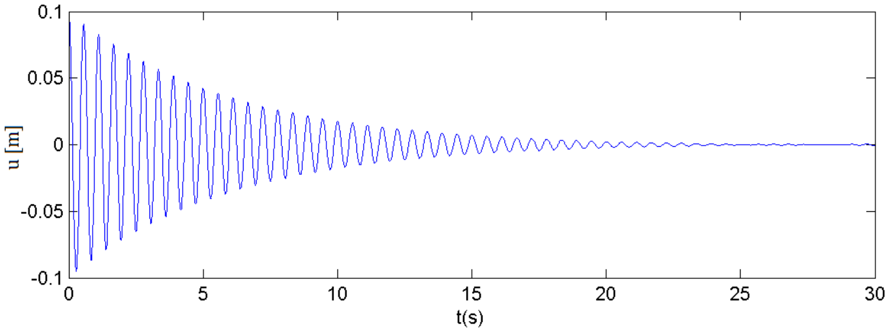

Figure D.1: Response of the structure with only an initial excitation - Matlab model

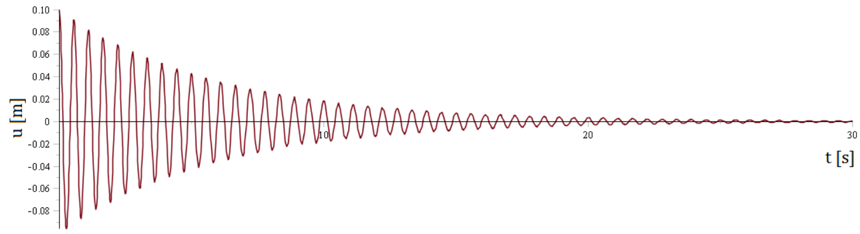

Figure D.2: Response of the structure with only an initial excitation - analytical solution plotted with Maple

The differential equation can now be solved, starting with an assumption for the solution of the deflection $u(t)$ of the structure:

$$u(t) = A_1 e^{rt} \tag{D.3a}$$

$$r^2 A_1 m^* e^{rt} + r A_1 c^* e^{rt} + k^* A_1 e^{rt} = 0 \tag{D.3b}$$

$$r^2 m^* + r c^* + k^* = 0 \tag{D.3c}$$

$$r_{1,2} = \frac{-c^* \pm \sqrt{(c^*)^2 - 4m^* k^*}}{2m^*} = -\frac{c^*}{2m^*} \pm \sqrt{\left(\frac{c^*}{2m^*}\right)^2 - \frac{k^*}{m^*}} \tag{D.3d}$$

$$u(t) = C_1 \cdot e^{-\frac{c^*}{2m^*} + \sqrt{\left(\frac{c^*}{2m^*}\right)^2 - \frac{k^*}{m^*}}} + C_2 \cdot e^{-\frac{c^*}{2m^*} - \sqrt{\left(\frac{c^*}{2m^*}\right)^2 - \frac{k^*}{m^*}}} \tag{D.3e}$$

$$u(t) = e^{-\frac{c^*}{2m^*}t} \cdot \left(C_1 \cdot e^{\sqrt{\left(\frac{c^*}{2m^*}\right)^2 - \frac{k^*}{m^*}}} + C_2 \cdot e^{-\sqrt{\left(\frac{c^*}{2m^*}\right)^2 - \frac{k^*}{m^*}}} \right) \tag{D.3f}$$

$$u(t) = e^{-\frac{c^*}{2m^*}t} \cdot \left(A \cdot \sin\left(\sqrt{\frac{k^*}{m^*} - \left(\frac{c^*}{2m^*}\right)^2}\, t \right) + B \cdot \cos\left(\sqrt{\frac{k^*}{m^*} - \left(\frac{c^*}{2m^*}\right)^2}\, t \right) \right) \tag{D.3g}$$

Applying the initial conditions, the solution for $u(t)$ can be found:

$$u(0) = B = 0.1 \tag{D.4a}$$

$$\dot{u}(0) = -\frac{c^*}{2m^*} \cdot B + \sqrt{\frac{k^*}{m^*} - \left(\frac{c^*}{2m^*}\right)^2} \cdot A = 0 \tag{D.4b}$$

$$A = \frac{\frac{c^*}{2m^*} \cdot 0.1}{\sqrt{\frac{k^*}{m^*} - \left(\frac{c^*}{2m^*}\right)^2}} \tag{D.4c}$$

The total solution for the deflection $u(t)$ is:

$$u(t) = e^{-\frac{c^*}{2m^*}t} \cdot \left(\frac{\frac{c^*}{2m^*} \cdot 0.1}{\sqrt{\frac{k^*}{m^*} - \left(\frac{c^*}{2m^*}\right)^2}} \cdot \sin\left(\sqrt{\frac{k^*}{m^*} - \left(\frac{c^*}{2m^*}\right)^2}\, t \right) + 0.1 \cdot \cos\left(\sqrt{\frac{k^*}{m^*} - \left(\frac{c^*}{2m^*}\right)^2}\, t \right) \right) \tag{D.5}$$

D.3. Single pedestrian

The case with a single walking pedestrian is now considered. The equation of motion D.6a is solved for the pedestrian loading D.6b for a single pedestrian.

$$m\frac{\partial^2 u(x,t)}{\partial t^2} + EI\frac{\partial^4 u(x,t)}{\partial x^4}) + c\frac{\partial u(x,t)}{\partial t} = P(x,t) \tag{D.6a}$$

$$P(x,t) = P\cdot\left(\alpha_1 \sin\left(2\pi f_p(t-t_s)+\phi\right) + \alpha_2 \sin\left(2\pi 2 f_p(t-t_s)+2\phi\right)\right)\cdot\delta\left(x - v\cdot(t-t_s)\right)\cdot\left(H(t-t_s)-H(t-t_e)\right) \tag{D.6b}$$

For which the assumption is made that $\phi = 0$, $t_s = 0$ and the deflection is a summation of the considered mode shapes $\phi(x)_n = \sin\left(\frac{n\pi x}{L}\right)$ times the corresponding deflection $u_n(t)$ (see appendix B):

$$P(x,t) = P\cdot\left(\alpha_1 \sin\left(2\pi f_p t\right) + \alpha_2 \sin\left(2\pi 2 f_p t\right)\right)\cdot\delta\left(x - vt\right)\cdot\left(H(t)-H(t-t_e)\right) \tag{D.7a}$$

$$u(x,t) = \sum_{n=1}^{N} \phi(x)_n \cdot u_n(t) \tag{D.7b}$$

Where $t_e = \frac{L}{v}$. The values of the other parameters are presented in section D.1. More detailed explanation about the equation is presented in chapter 10.4. The acceleration is the second derivative of the deflection with respect to time.

The same ordinary differential equation for a one mass-spring-damper-system can solved both with Matlab and with Maple. The analysis with Maple is performed to check the Matlab model. Due to the fact that Maple is an analytical solver, it takes more computation time to solve the problem. The maximum acceleration $a(t)$ at the normative position of the bridge is plotted for both the Matlab model (figure D.3) and for the solution using Maple (figure D.4). The accelerations of the bridge for the first mode shape in the case a single pedestrian is crossing the bridge is presented, therefore the normative position is x = L/2 = 25 m.

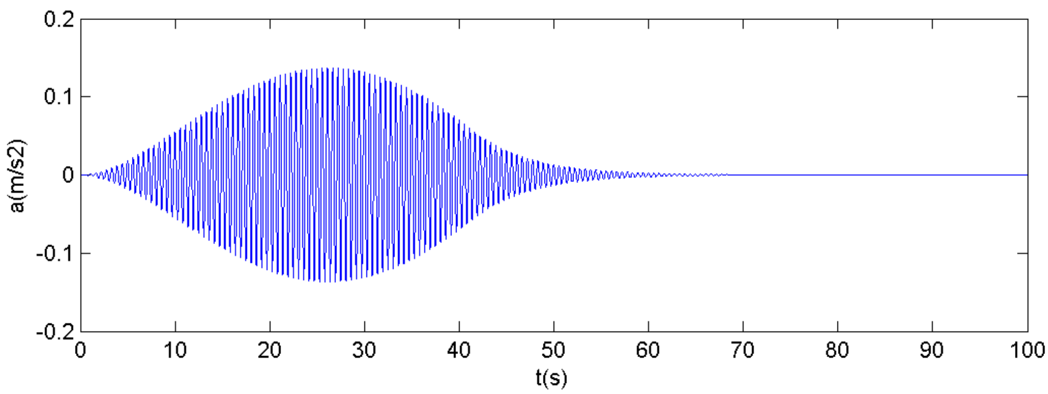

Figure D.3: Acceleration of the bridge due to a single pedestrian for the first mode at x = 25 m - Matlab

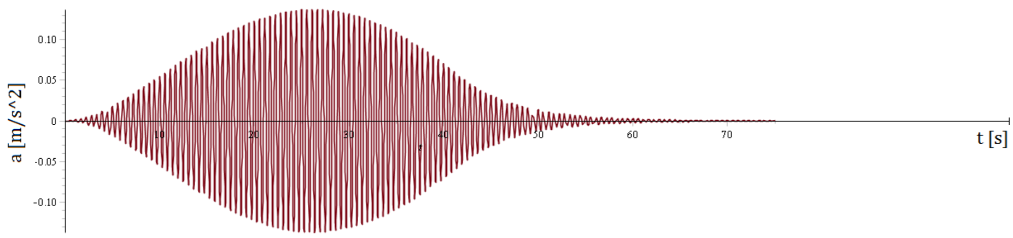

Figure D.4: Acceleration of the bridge due to a single pedestrian for the first mode at x = 25 m - Maple

D.4. Two pedestrians

Instead of one pedestrian, now two pedestrians walking across the bridge are considered. Two checks are performed: fist a check with a large gap between the time the pedestrians enter the bridge and secondly, a check with two pedestrians entering the bridge with a small time gap. Different properties are assigned to the pedestrians, as shown in the overview below.

Pedestrian parameters

f_1	=	1.8	Frequency pedestrian 1 in Hz
f_2	=	2.0	Frequency pedestrian 2 in Hz
v_1	=	1.2	Velocity pedestrian 1 in m/s
v_2	=	1.4	Velocity pedestrian 2 in m/s
P_1	=	700	Weight pedestrian 1 in N
P_2	=	800	Weight pedestrian 2 in N
ϕ_1	=	0	Phase shift 1
ϕ_2	=	1	Phase shift 2

D.4.1. Two separate pedestrians

The second pedestrian is enters the bridge 80 seconds after the first pedestrian, with the consequence that the pedestrians will not be present at the footbridge at the same time. A difference in force and acceleration is visible due a different in properties. The first pedestrian has a walking frequency almost equal to the natural frequency of the first mode shape of the bridge, the walking frequency of the second pedestrian is higher and therefore the structural response will be lower. Both the figures with forces of the pedestrians (figures D.5 and D.6) are plotted and the figures with the accelerations of the bridge (figures D.7 and D.8) for the first mode at x = 25 m.

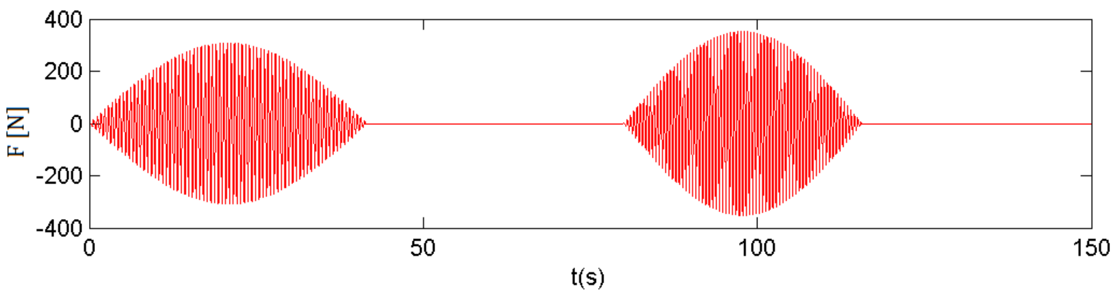

Figure D.5: Force of two separate pedestrians crossing the footbridge with L = 50 m - Matlab

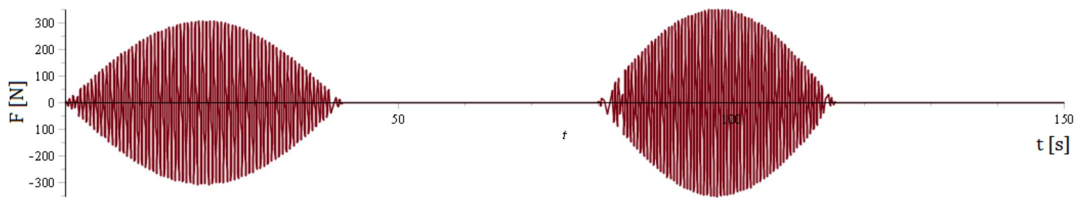

Figure D.6: Force of two separate pedestrians crossing the footbridge with L = 50 m - Maple

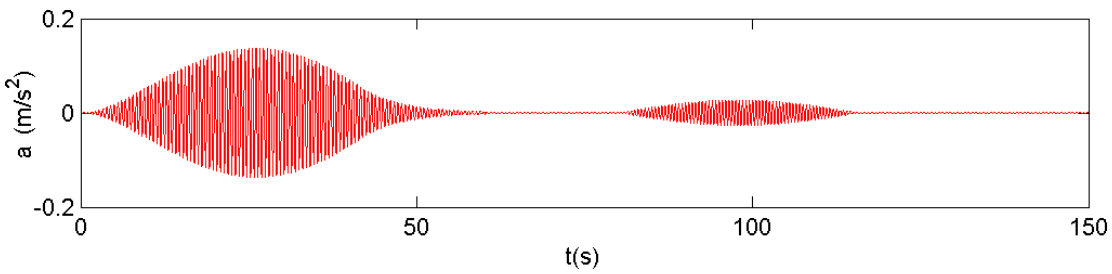

Figure D.7: Acceleration of the bridge due to two separate pedestrians for the first mode at x = 25 m - Matlab

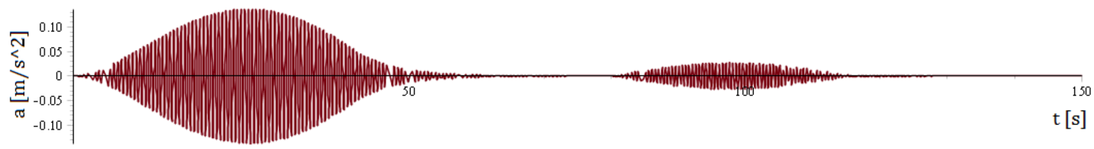

Figure D.8: Acceleration of the bridge due to two separate pedestrians for the first mode at x = 25 m - Maple

D.4.2. Two combined pedestrians

A last check is performed with two pedestrians entering the bridge with a time interval of 10 seconds. The pedestrians have the same properties as for the case above, with the two separate pedestrians. First the figures with forces of the pedestrians (figures D.9 and D.10) are plotted. Figures D.11 and D.12 show the accelerations of the bridge (red) for the first mode at x = 25 m and the accelerations the first pedestrian experiences (blue) when crossing the bridge.

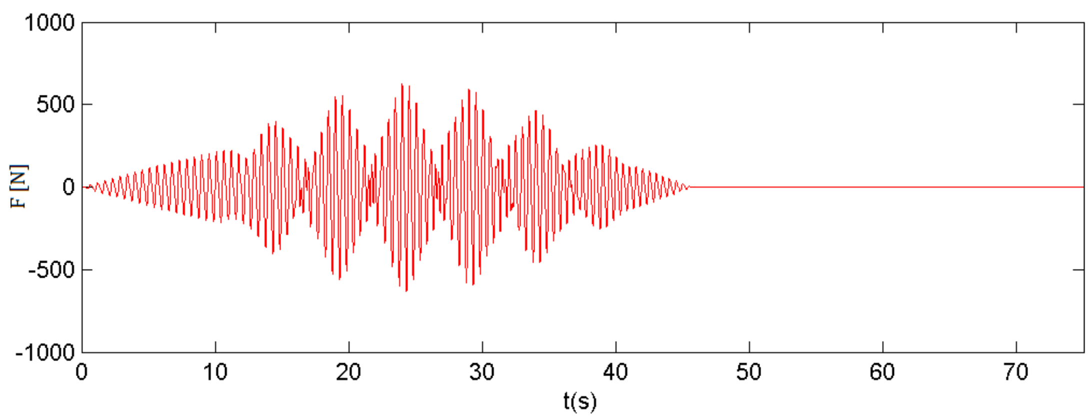

Figure D.9: Force of two combined pedestrians crossing the footbridge with L = 50 m - Matlab

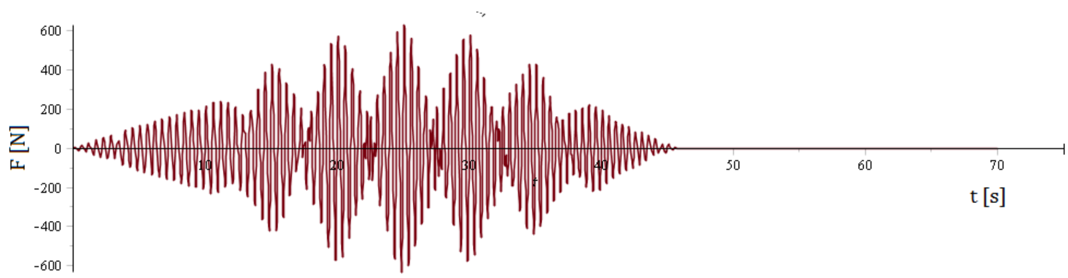

Figure D.10: Force of two combined pedestrians crossing the footbridge with L = 50 m - Maple

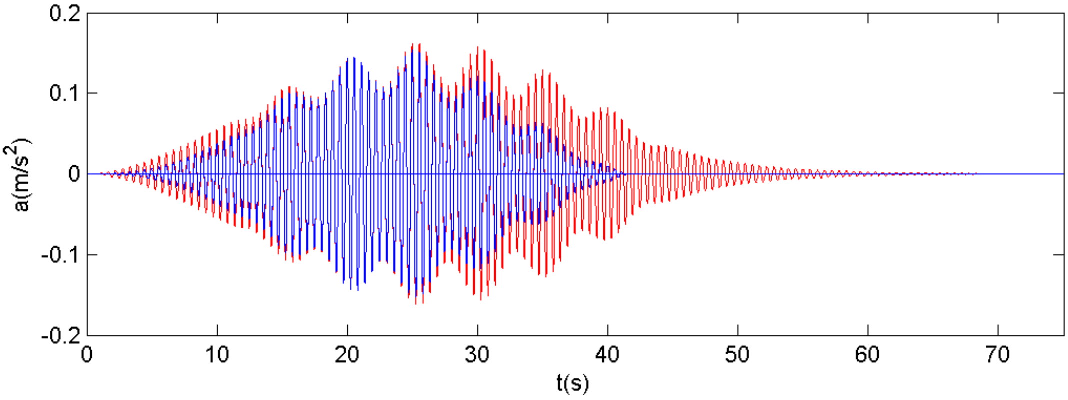

Figure D.11: Acceleration of the bridge (red) due to two separate pedestrians for the first mode at x = 25 m and the accelerations the first pedestrian experiences (blue) when crossing the bridge - Matlab

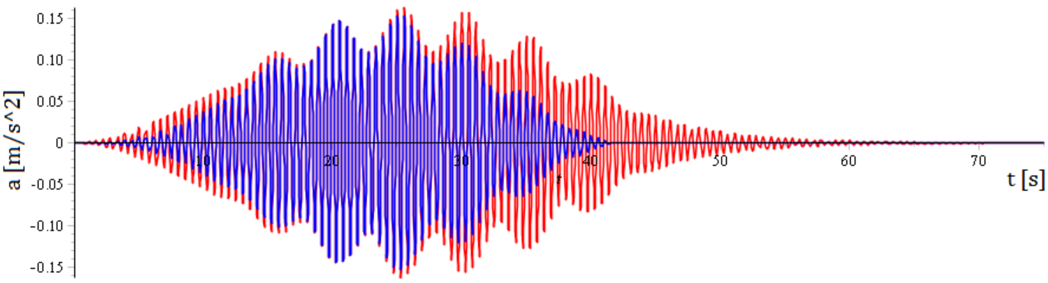

Figure D.12: Acceleration of the bridge (red) due to two separate pedestrians for the first mode at x = 25 m and the accelerations the first pedestrian experiences (blue) when crossing the bridge - Maple

E

ADDITIONAL FIGURES PART II

Grade	Vibrations	Feelings	Description of vibrations for moving persons	Description of vibrations for standing or sitting persons
0	imperceptible	totally comfortable	no vibrations	no vibrations
1	slightly perceptible	no negative feelings related to the comfort, comfort is ensured	very slightly perceptible	very slightly perceptible
2			slightly perceptible	slightly perceptible
3	clearly perceptible	a feeling of minimum or exceeded comfort	not hindering movement	noticeable, distracting from the performed activity
4			disturbing, causing some problems with smooth movement	disturbing, causing unpleasant feelings
5	strongly perceptible	a feeling of strongly exceeded comfort	strongly perceptible, causing a change in the walking pace	setting the body in motion - rocking
6			very disturbing, the possibility of smooth movement is excluded	making standing or sitting without additional support, e.g. holding the handrails, impossible, no feeling of stability
7	very strongly perceptible	a feeling of panic, the urge to run away from the vibrating footbridge	vibrations perceived as a threat to safety	vibrations perceived as a threat to safety

Figure E.1: The gradual scale of comfort evaluation according to Hawryszkow [16]

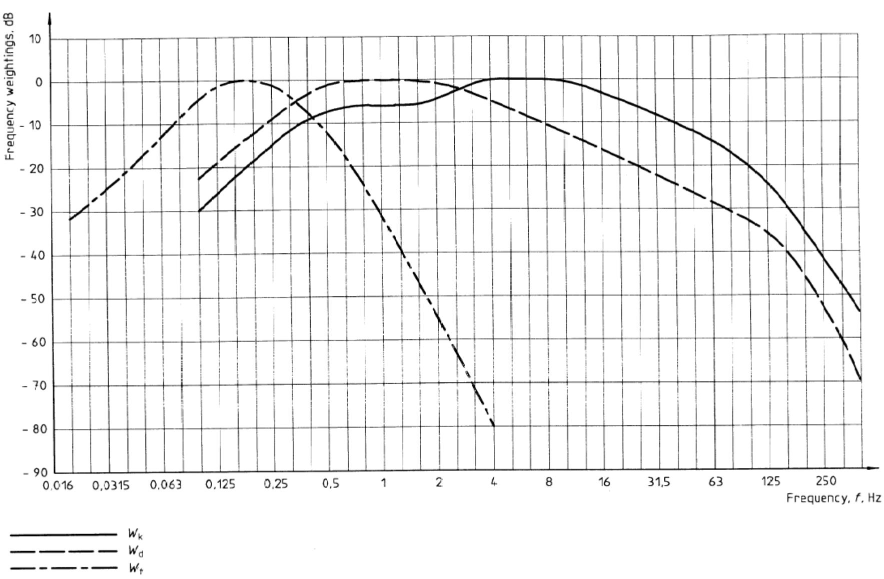

Figure E.2: Frequency weighting curves for principal weightings, with W_k as the frequency weighting curve that has to be applied in the case of comfort for standing persons [67]

Table 3 — Principal frequency weightings in one-third octaves

Frequency band number[1)] x	Frequency f Hz	W_k factor × 1 000	W_k dB	W_d factor × 1 000	W_d dB	W_f factor × 1 000	W_f dB
– 17	0,02					24,2	– 32,33
– 16	0,025					37,7	– 28,48
– 15	0,031 5					59,7	– 24,47
– 14	0,04					97,1	– 20,25
– 13	0,05					157	– 16,10
– 12	0,063					267	– 11,49
– 11	0,08					461	– 6,73
10	0,1	31,2	– 30,11	62,4	– 24,09	695	– 3,16
9	0,125	48,6	– 26,26	97,3	– 20,24	895	– 0,96
– 8	0,16	79,0	– 22,05	158	– 16,01	1 006	0,05
– 7	0,2	121	– 18,33	243	– 12,28	992	– 0,07
6	0,25	182	– 14,81	365	– 8,75	854	– 1,37
– 5	0,315	263	– 11,60	530	– 5,52	619	– 4,17
– 4	0,4	352	– 9,07	713	– 2,94	384	– 8,31
– 3	0,5	418	– 7,57	853	– 1,38	224	– 13,00
– 2	0,63	459	6,77	944	– 0,50	116	– 18,69
– 1	0,8	477	6,43	992	– 0,07	53,0	– 25,51
0	1	482	– 6,33	1 011	0,10	23,5	– 32,57
1	1,25	484	6,29	1 008	0,07	9,98	40,02
2	1,6	494	– 6,12	968	– 0,28	3,77	48,47
3	2	531	– 5,49	890	– 1,01	1,55	56,19
4	2,5	631	– 4,01	776	– 2,20	0,64	– 63,93
5	3,15	804	– 1,90	642	3,85	0,25	71,96
6	4	967	– 0,29	512	5,82	0,097	80,26
7	5	1 039	0,33	409	7,76		
8	6,3	1 054	0,46	323	– 9,81		
9	8	1 036	0,31	253	11,93		
10	10	988	0,10	212	13,91		
11	12,5	902	0,89	161	15,87		
12	16	768	– 2,28	125	– 18,03		
13	20	636	– 3,93	100	– 19,99		
14	25	513	– 5,80	80,0	– 21,94		
15	31,5	405	– 7,86	63,2	– 23,98		
16	40	314	– 10,05	49,4	– 26,13		
17	50	246	– 12,19	38,8	– 28,22		
18	63	186	– 14,61	29,5	– 30,60		
19	80	132	– 17,56	21,1	– 33,53		
20	100	88,7	– 21,04	14,1	– 36,99		
21	125	54,0	– 25,35	8,63	– 41,28		
22	160	28,5	– 30,91	4,55	– 46,84		
23	200	15,2	– 36,38	2,43	– 52,30		
24	250	7,90	– 42,04	1,26	– 57,97		
25	315	3,98	– 48,00	0,64	– 63,92		
26	400	1,95	– 54,20	0,31	–70,12		

1) Index x is the frequency band number according to IEC 1260.

NOTES

1 For tolerances of the frequency weightings, see 6.4.1.2.

2 If it has been established that the frequency range below 1 Hz is unimportant to the weighted acceleration value, a frequency range 1 Hz to 80 Hz is recommended.

3 The values have been calculated including frequency band limitation.

Figure E.3: Frequency weighting table for principal weightings, with W_k as the frequency weighting that has to be applied in the case of comfort for standing persons [67]

F

ADDITIONAL RESULTS CASE STUDY

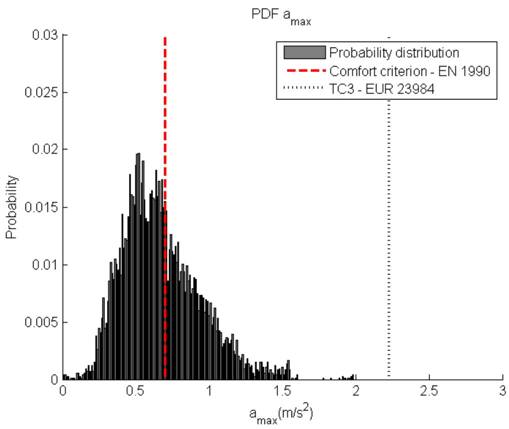

Figure F.1: Probability distribution of the maximum accelerations per single pedestrian crossing the bridge within a crowd density of 0.5 P/m^2, compared with the maximum acceleration calculated with EUR 23984 for the bridge L = 12 m

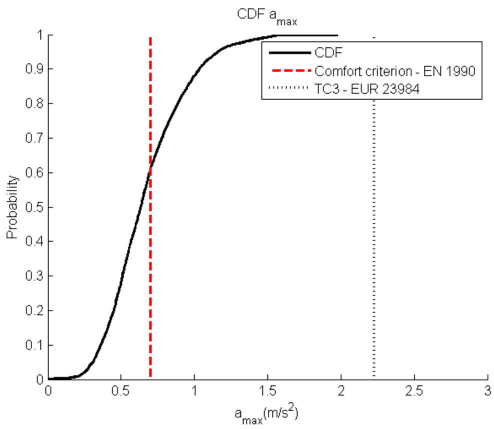

Figure F.2: Cumulative density function of the maximum accelerations per single pedestrian crossing the bridge within a crowd density of 0.5 P/m^2, compared with the maximum acceleration calculated with EUR 23984 for the bridge L = 12 m

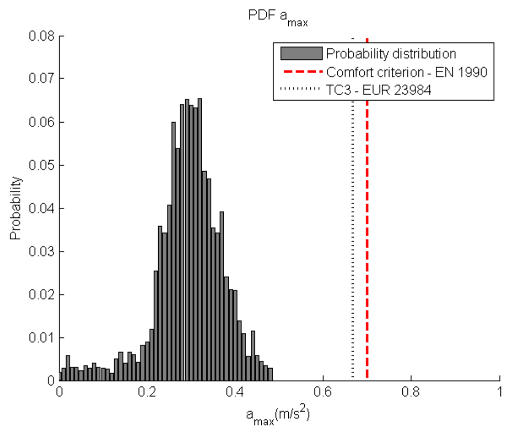

Figure F.3: Probability distribution of the maximum accelerations per single pedestrian crossing the bridge within a crowd density of 0.5 P/m^2, compared with the maximum acceleration calculated with EUR 23984 for the bridge L = 100 m

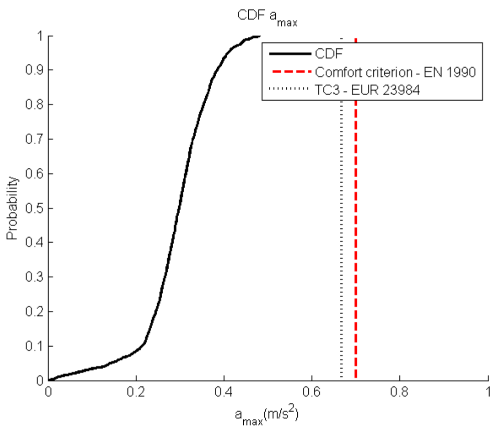

Figure F.4: Cumulative density function of the maximum accelerations per single pedestrian crossing the bridge within a crowd density of 0.5 P/m^2, compared with the maximum acceleration calculated with EUR 23984 for the bridge L = 100 m