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## BEHAVIOR OF FRP PEDESTRIAN BRIDGES UNDER HUMAN INDUCED EXCITATION

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BEHAVIOR OF FRP PEDESTRIAN BRIDGES UNDER  
HUMAN INDUCED EXCITATION

BY

LAURA SCHMIDT

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MASTER OF SCIENCE THESIS  
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## **ABSTRACT**

The prediction of the dynamic response of pedestrian bridges under human-induced excitation is a challenge in the design of pedestrian bridges, caused by the wide range of variables and the complex interaction effects. The use of new, lightweight materials, like FRP, and the trend to design long span and slender constructions lead to structures more sensitive to dynamical impact, which caused some vibrational problems at newly built bridges in the recent past. This brought increased attention to this topic.

The present thesis aims to analyze the dynamic properties of the new material fiber reinforced polymer (FRP), to estimate the changes they cause in the dynamic response of respective constructions and to validate the current guidelines. The first part of the research includes a literature review in terms of pedestrian loading, their interaction with the structure, the characteristics of FRP and the specification of the current guidelines. In order to analyze the dynamic properties and the effects on the dynamic response, the second part presents a parametric analysis of simplified bridge structures and their dynamic response to different loads induced by pedestrians. In order to classify the new composite material, the estimated mechanical properties and dynamic characteristics are compared to the traditional material steel.

FRPs are significantly lighter and less stiff than steel. The first property leads to a higher fundamental frequency, the later one counteracts this effect. The actual fundamental frequency of the unloaded system, which is also the main component in the dynamical evaluation specified in the AASHTO guideline, depends on the ratio stiffness to weight. In contrast to steel, FRP is more sensitive to human-induced loads.

The additional mass of the pedestrians changes the fundamental frequency of the system significantly, due to the high ratio of live load to construction weight. This circumstance is disregarded by the current guidelines, which might have led to the vibrational problems at newly built pedestrian bridges. Furthermore, the lateral-synchronization-phenomenon, which is also not mentioned in the guidelines, has a significant impact on lively footbridges. A general approach for the consideration of the additional impact is introduced.

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# CHAPTER 1

## INTRODUCTION

### 1.1 Statement of the problem

During the last two decades, the dynamic behavior of pedestrian bridges under human-induced excitation attracted considerable interest, caused by several incidents, in which the excited oscillations of newly built bridges exceeded the level of serviceability and endangered the safety of the structure. The accumulated appearance of these dynamical problems is related to the recent developments in the construction of pedestrian bridges. The trend for longer spans and greater slenderness in combination with the use of new, light materials reduces the natural frequencies of the structures and increases their sensitivity to dynamic loads in the range of walking frequencies. The reported oscillation problems from all over the world prove that the current guidelines and design codes regarding the design of pedestrian bridges are unsatisfactory in terms of human-induced excitations of light and slender constructions. The aim of this study is to analyze the dynamical behavior of pedestrian bridges built out of fiber reinforced polymer (FRP), a representative for the newly used materials in the construction of bridges, and to point out their specifications and their differences to traditional construction materials.

## 1.2 Significance of the Study

Pedestrian bridges sometimes span considerably long distances and, in combination with the advantage of comparatively low design loads and the recent aesthetic request for greater slenderness and lightness, they present an opportunity for innovative architectural and engineering design, using new systems and materials. Therefore, common features of recent footbridges are “*long span, light materials and increasing slenderness*” [27]. All three of these attributes result in a reduced natural frequency of the structure. The reduced natural frequency, in turn, causes a higher sensitivity of the structure to dynamic forces induced by pedestrians, because the natural frequency of the structure drops into the range of frequencies of human walking. “*For the design of slender footbridges, the vibration serviceability under pedestrian excitation is often the governing criterion*” [55]. Consequently, this should be given special attention during the design process and should also be acknowledged in the corresponding guidelines and design codes. Predicting the response of pedestrian bridges to human-induced excitation is an exceedingly complex process, including the variability of human walking parameters and effects of human-structure and human-human interaction.

In order to simplify the design process, current guidelines (AASHTO) are limited to general restrictions of the structural natural frequency [1, 2, 3]. By doing this, the guidelines neglect the complexity of the phenomena appearing during a human-induced excitation of bridges.

Some codes of practice (e.g. OHBDC, BS5400, Eurocode 5, Setra) [4] propose instead a deterministic moving force model, which is also insufficient to describe all

aspects of the system. The true importance and complexity of the topic have been brought to the attention of civil engineers by an increasing number of reported vibration serviceability problems in newly built pedestrian structures under pedestrian loading.

A number of pedestrian-excited laterally unstable bridges reported all over the world over the past decades, including the London Millennium Footbridge and the Clifton Suspension Bridge, focused the attention towards the actual human walking mechanisms and the unique human-structure interaction. It also started a series of researches with the aim of identifying the underlying mechanisms leading to dynamic instability [47]. Extensive research in several fields over more than a decade has improved the understanding of the problem and has enabled the development of better modeling and simulation tools. The main component in these new models, which has been neglected in earlier approaches and is responsible for the vibration problems, is the human-structure interaction (HSI). The influence of this component increases with a higher ratio of live load to construction weight, which grows automatically due to the low density of the new materials and the increased slenderness of the structures. The HSI includes changed dynamical properties of the structure due to the additional weight and damping effects of the pedestrians as well as interdependency of the structural movement and the induced pedestrian load. The latter one occurs particularly in the lateral direction. It is also known as synchronized lateral excitation and leads to unusually high loads and great lateral displacements. The effects of the dynamic components of the pedestrian load are significantly higher at slender and light constructions, like, for instance, constructions out of fiber reinforced polymer.

Composite materials, like “*Fibre-Reinforced Polymers (FRPs) are an increasingly popular option for the construction of bridges as they possess high strength to weight properties and good durability qualities*” [40]. However, the advantage of the reduced weight compared to traditional materials in combination with the also reduced stiffness is regarding the dynamic properties a detriment – the structure is more sensitive to human-induced oscillations.

### **1.3 Methodology**

Predicting the response of pedestrian bridges to human-induced excitation is an exceedingly complex process, including the variability of human walking parameters and effects of human-structure and human-human interaction. Even though approaches for the modeling of the multi-physics system of a bridge loaded by a crowd of pedestrians are available, the proposing of such a model is not the aim of this research. The present research analyses the specific dynamical properties of fiber reinforced polymer pedestrian bridges. In this context, the parameters and components of pedestrian loads, including the effects of the human-structure interaction, have been estimated based on a literature review, simplified and solitary applied to generalized, simulated bridge constructions out of fiber reinforced polymers. The aim of this stepwise proceeding is the clarification of the different response mechanisms due to the different inputs and thus, a better understanding of the general dynamic response of lively bridges. The analysis has been made with the FE-Program ABAQUS. For a reliable categorization of the material, the same simulations have been made with comparable steel constructions. The differences between the two materials are



analyzed and presented. Finally, the results are compared with the current guidelines and the corresponding requirements are validated.

#### **1.4 Structure**

The present thesis follows a simple structure. The second and third chapters include a literature review. The second chapter deals with fiber reinforced polymer and presents the production and composition, the properties and characteristics, and the applications of these materials. The third chapter gives an overview of the pedestrian walking loads. It is divided into three parts: individual level, collective level, and multi-physics level, which present the different considerable aspects of dynamic loads induced by a crowd on a flexible structure. The fourth chapter presents the requirements of the design code, which is the base for the model development. During the fifth chapter, the development of the computer model and the choice of variables are explained and general settings are described. The results are presented and evaluated in chapter six. It points out the specifications of FRPs and the differences between FRPs and steel. Furthermore, a validation of the current guidelines is performed and an improved approach is presented. A summary of the research as well as an outlook for the possible future of this field of study is part of the conclusion in chapter seven.

## CHAPTER 2

### REVIEW OF LITERATURE: FIBER REINFORCED POLYMER

#### 2.1 Composites

Composite materials have been used for 6000 years and the trend is still developing and increasing. The first application of composite materials was made by the Egyptians in form of straw-reinforced clay bricks. Since then, a great progress has been achieved, especially in the last decades. New materials with a great range of improved characteristics have been developed and are used in numerous fields.

A composite material consist of at least two constituent materials, which are mixed at a nano-, micro- or macroscopic level. The constituents are not soluble, form unambiguous phases, and have significantly different physical or chemical properties. The combined material has characteristics different from the individual initial components. [8, 15, 58] The aim of the composition is to combine the benefits of the different initial materials, to create materials, which are, for instance, stronger, lighter or less expensive in comparison to traditional materials. Normally, composite materials consists of two main components, a matrix material, strengthened by a reinforcing phase. The best-known example for a composite material in the field of civil engineering is reinforced concrete. As a combination of concrete and steel, it combines the high pressure capacity and low costs of the concrete with the high tensile stress capacity of the expensive steel, which leads to an economic, efficient and high capacity material. Even though concrete is the most used composite material in the field of civil engineering, the expression composites normally refers to advanced composite materials, like fibre-reinforced polymer (FRP). These advanced composite

materials were primarily developed for aerospace to enhance the performance of commercial and military aircraft and they still play a significant role in the field of aerospace as well as in other engineering fields. *“However, in recent years composite materials have become particularly attractive for civil engineering infrastructure applications due to their exceptional strength and stiffness-to-density ratios and superior physical properties. Considerable advances have been made in the use of composite materials in the construction and building industries, and this trend will continue. Fibre reinforced polymer (FRP) composites are now widely used in civil engineering applications”* [8]

## **2.2 Materials**

*“Polymer matrix-based composites are essentially composed of fibres embedded in polymeric matrices.”* [38]. For both of these phases, polymer matrices and fibres, different materials are available – with changing material characteristics and combinability. The properties of the final materials are controlled by the properties of the initial materials, but also by the bonding conditions between them. Therefore, the interface area can be seen as the third phase. There are numerous materials available; however, this chapter is focused on the fiber-reinforced polymer materials used in bridge engineering.

### **2.2.1 Matrix materials**

The matrix materials provide the foundation for composite materials. *“The matrix materials, [...], are responsible not only for covering the reinforcements (thereby protecting them from environmental and chemical damage) but also for the*

*elimination of fibre wearing and crushing that can be caused by deformation: they fix the fibres in position, which is crucial, as the reinforcing materials could otherwise easily slip out or become damaged through wear. The matrix materials also act as load transferring media: they transfer the load in an orthogonal direction from the fibre axis.” [8]*

Polymers used in bridge constructions can be divided into the three major types: thermosetting, thermoplastic and elastomeric. The different materials require different procedures for their manufacture, but they all require two main components, the resin and curing agent. All three materials are used for constructions, but thermosetting polymer dominate the market.

*“The thermosetting polymer consists of long chain molecules, which are cross-linked in a curing reaction. The network so formed and the length and the density of the molecular units are a function of the chemicals used in the manufacture of the polymers, and the cross-linking is a function of the degree of cure of the polymer. Both the network and the cross-linking will have an influence on the mechanical and in-service properties of the material. Furthermore, the degree of cure is a function of the temperature and the length of the polymerisation (curing) period.” [8]*

Thermosets in general are brittle at room temperature, that is why there is a need for reinforcement, but they have also a number of useful characteristics. Unlike thermoplastics, the properties of thermosets improve with increasing temperature, at least until a certain temperature threshold, at which the properties starts to degrade. However, this threshold is significant higher than the corresponding degradation point for thermoplastics. [8, 10, 14]

Common materials are polyester thermoset resins, phenolic resins, vinyl ester resins and epoxy resins. “*Polyester is one of the earliest types of thermoset and is widely used in FRP composites*” [8]. Due to their great mechanical properties and their high resistance to environmental degradation, epoxy resins are the most important polymers in structural civil engineering. Another advantage of epoxy resins is the absence of styrene, which minimises the toxic emission during the production process and the resulting possibility of an ‘open-mold’ technology. Epoxy resins are more expensive than other thermosetting polymers, but they show remarkable mechanical properties and great material characteristics, what makes the cost-performance ration equal and the material favourable. Vinylesters are unsaturated esters of epoxy resins and have similar mechanical and in-service properties to epoxy resins. Due to the differences in the chemical composition, vinylesters are more flexible and have higher fracture toughness. [8, 21, 44, 57, 58]

For polymers, to reach their full mechanical properties, it is essential that they have reached a nearly complete polymerisation. Therefore, it is important that the correct mix ratio is obtained between the resin and its curing agent. Since the curing reaction is influenced by heat, the site temperature should be given some attention. Once the reaction is finished, the resins do not melt, soften upon reheating or dissolve in solvents. [8, 57]

### 2.2.2 Reinforcement materials

The main function of the reinforcement is the strengthening of the matrix material by carrying the load along its length. “*A wide range of amorphous and crystalline*

*materials can be used to form fibres, but in bridge engineering the three fibres which are generally used are the glass fibre, the aramid fibre and the carbon fibre.” [8].* A main component for the properties of the composites is the aspect ratio (length/diameter) of the fibers used as reinforcement and their orientation and fraction. The choice for type, amount, length, orientation and other properties depend on the matrix characteristics, the intended application and the necessary load capacity. One distinction to categorize reinforcements is the size of the used fibers. It can be differed between macro-, micro- and nanoscale reinforcement. Macroscale reinforcement is the most common type of reinforcement. Glass fibers, carbon fibers and aramid fibres are widely used to reinforce polymers for the application in the field of civil engineering. Usually they are used as CF fiber bundles (tows), glass fiber bundles (rovings), continuous strand mats and nonwoven surfacing veils. Glass fibers can be produced with tailored properties to meet specific applications. CF have also attractive properties “*such as low weight, high strength and high modulus, fatigue resistance and vibration damping, corrosion resistance, good friction and wear qualities, low thermal expansion, and thermal and electrical conductivity*” [8]. All three types of fibers have slightly different characteristics and can be chosen with respect to particular application. It is also possible to use a mix of different types of fibers to combine their advantages. [8, 15, 44]

Independently of type and material of the used reinforcement, it is most important to adapt matrix and reinforcement to each other. The bonding between the two phases is decisive to ensure that the composite system as a whole gives satisfactory performance. [8, 15, 22, 24]

## **2.3 Properties**

### **2.3.1 General Characteristics**

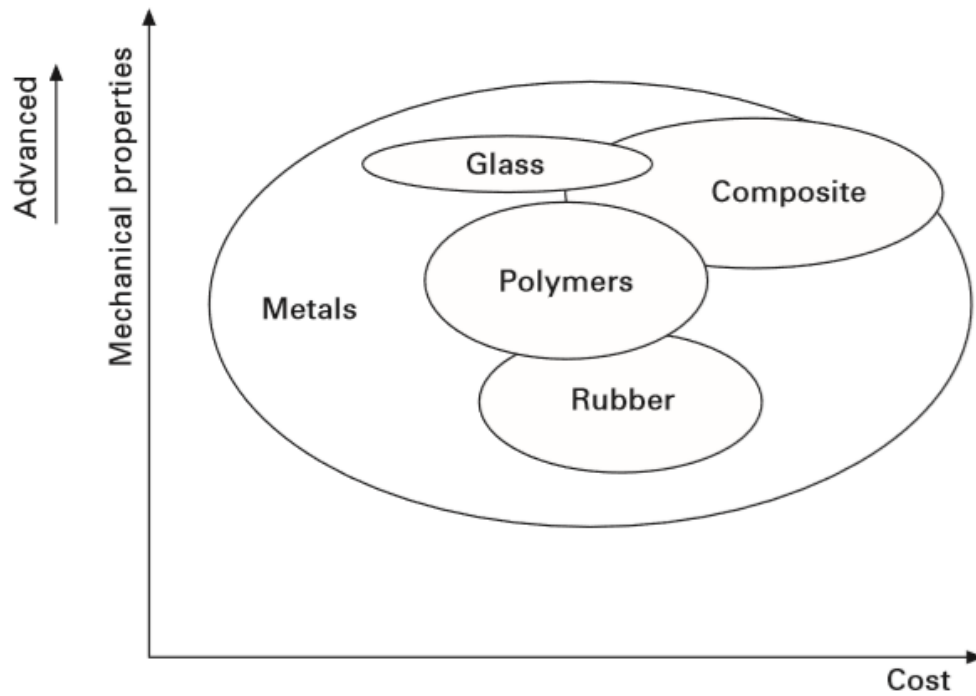
Accompanying the great importance of traffic and transportation, there is a growing concern with respect of the maintenance of the related infrastructure. The reparation, rehabilitation and replacement of old bridge structures is one of the main tasks in the field of civil engineering. The deterioration of reinforced concrete and steel bridges, especially when exposed to aggressive and hostile environments that are invariably encountered to traffic related constructions, lead to the need for durable, high strength and high stiffness materials. The result was the establishment of advanced composite materials as structural material in civil engineering. *“These materials can provide significant advantages over conventional materials for the construction of bridges”* [8]. Advanced composites have a higher resistance to oxidation than steel and a better freeze-thaw resistance than concrete, which might have been the initial reason for the introduction of FRP composite materials into the field of civil engineering. Nevertheless, FRPs provide a wide range of convenient characteristics, which led to a fast establishment of these materials. The resistance to corrosion and the freeze-thaw resistance result in low maintenance requirements and an enhanced service life. This applies also for harsh and corrosive environments, common at traffic related structures. The durability is even further improved, because also the fatigue performance is good. The high durability and resistance reduces the life cycle costs and is especially useful for areas with limited access. On the other hand, one of the main disadvantages is also related to this topic, which is the reprocessing. The high resistance of the material makes the reprocessing through

either mechanical or chemical recycling difficult. In these days this is an important aspect which has to be considered during the planning of respective constructions. FRPs provide a reduction of the dead load and a subsequent increase in live load rating. Related to this point is the advantage of a faster installation. Due to the reduced weight, the components are light and can be assembled easy and fast. Prior to shaping FRPs are liquids with low viscosity. As a result, their processing is relatively cheap and easy and they can be produced with complex shapes. It enables also the production of complete structural components under factory conditions, because they can be easily transported and installed. In comparison to concrete FRPs have also a good creep behavior that facilitates the design and planning process. [8, 21, 30, 46]

Another advantage of FRPs is the high versatility that means the possibility of adapting the material properties perfectly to the requirements of the intended application and function. Materials can be obtained with high strength, stiffness, and excellent impact strength. The actual stiffness is lower than the one of steel or concrete, but due to the reduced weight, the stiffness-weight ratio is better. Even the requirements for fire- and high-temperature-resistance of construction materials are fulfilled by the composites satisfactory, “*due to their resistance to burning and minimal smoke and toxic fumes production*” [8]. Additional tailorable characteristics are acoustic and thermal insulation as well as thermal conductivity. On the other hand, higher initial costs of materials can be considered as disadvantage, but considering the higher strength and the whole-life costs, these costs are more than compensated. However, “*composite materials are often predominantly composed of the most expensive construction materials.*” [8] The relation of mechanical properties and costs



of the most commonly used constructional materials are presented in the following figure. It can be seen, that composites are relatively expensive, but provides better mechanical properties than most of the other available materials.



**Figure 1: Relationship between costs and properties of constructional materials [8]**

### 2.3.2 Mechanical Properties

It has been previously established that FRPs are highly versatile. This fact complicates the determination of general, or even average mechanical properties. Depending on the used materials, the composition and the manufacturing process a wide range of mechanical properties, as Young's modulus, density and tensile stress capacity, are possible to create. *“The mechanical properties of FRP composites are dependent upon the ratio of fibre and matrix material, the mechanical properties of*

*the constituent materials, the fibre orientation in the matrix, and ultimately the processing and methods of fabrication” [8].*

Not all of these attractive properties can be achieved at the same time; therefore, the design should be done carefully, since it can be efficiently optimized in contribution to the individual application. A main factor for the actual realisation are the costs. This factor limits the range of properties of FRPs used for constructions. The tensile strength can still vary between the strength of mild steel and values higher than the ones of prestressing steels. The specific strength, which is often used to compare materials, can be 40–60 times that of high-strength steel. [8] A collection of values of the most important mechanical properties for different types of FRPs available in the current literature is presented in the following table:

	Density [g/cm <sup>3</sup> ]	Young's modulus [GPa]	Tensile strength [MPa]	Modulus to weight ratio [10 <sup>6</sup> m]	Tensile strength to weight ratio [10 <sup>2</sup> m]	source
High strength carbon fibre-epoxy matrix (unidirectional)	1.55	137.8	1550	9.06	101.9	[8]
High modulus carbon fibre – epoxy matrix (unidirectional)	1.63	215	1240	13.44	77.5	[8]
E-glass fibre – epoxy matrix (unidirectional)	1.85	39.3	965	2.16	53.2	[8]
Kevlar 49 fibre – epoxy matrix (unidirectional)	1.38	75.8	1378	5.60	101.8	[8]
Carbon fibre – epoxy matrix (quasi-isotropic)	1.55	45.5	579	2.99	38	[8]
Sheet moulding compound (SMC) composite (isotropic)	1.87	15.8	164	0.96	8.9	[8]
Glass fibre – vinylester composite (randomly orientated fibres, fibre/matrix=67%)	1.84	19.3	269	1.07	14.9	[8]
Glass fibre – vinylester composite (randomly orientated fibres, fibre/matrix=67%)	1.80	15.8	166	0.89	9.40	[8]

**Table 1: Mechanical properties of FRP materials**

One additional aspect has to be considered during the design of fiber reinforced polymer constructions. Depending on the orientations and type of the used fibers, the mechanical properties of reinforced composites vary accordingly to the three different dimensions of space. It is a well-known fact in the field of reinforced concrete. The application to the advanced composite materials gets more complicated, since the reinforcement is much more variable and several combinations and applications are available. The anisotropic behavior of composite material has to be taken into account during the design and the dimensioning of construction parts. [8]

## **2.4 Applications**

### **2.4.1 General Aspects**

Since the middle of the last century FRP composite materials have been used foremost in the aerospace and automotive industries, due to their advantageous material characteristics. Although FRP materials have been used in the manufacturing of a wide range of fields, like cars, boats, tanks and missiles, their use in civil engineering applications is still relative recent. The fact that FRP composites, unlike steel, do not corrode in concrete led in the end to the first application in civil engineering: glass fiber reinforced polymer rods were used as reinforcements in concrete. [52] *“Since the late 1980s, FRP rebars have been used more extensively in concrete structures, especially in highway bridge decks, because of their resistance to corrosion. For the same reason, FRP composites have been used more and more widely when repairing and retrofitting deteriorated bridge superstructures, to reinforce bridge decks, girders and piles, and when replacing structural members (e.g. decks).”* [8]

#### 2.4.2 FRP pedestrian bridges

For the construction of bridges, FRP profiles were first used for short-span pedestrian bridges. The first FRP pedestrian bridge was 12 m long and was built by the Israelis in 1975 [3.3]. Since then FRP profiles have been increasingly used in the decks and superstructure members of bridges and hundreds of FRP bridges have been built worldwide. Nevertheless, a problem in the design of a pure FRP decks is the low modulus of FRP composite materials and therefore the relatively low stiffness of the material, which leads to high deflections. Other materials, like concrete, can be added to increase the stiffness of the deck, keeping the total weight low. The new material with improved characteristics made also the development of new construction types and styles possible. On the one hand are combinations in form from hybrid bridges with other traditional or newly developed construction materials and on the other the bridge designs based on the particular characteristics of the material. [52]

#### 2.4.3 FRP vehicular bridges

Seven years after the installation of the first pedestrian FRP bridge and based on the gained knowledge the first FRP vehicular bridge was built in China in 1982. It spanned 20.7 m and was 9.2 m wide. [58] In the early 1990s followed further FRP deck systems, mainly used for short-span bridges with light traffic. The development of FRP bridges in the US was accelerated by a program of the US Federal Highway Administration (FHWA) for the development of cost-effective innovative material applications in highway bridges [46]. FRP deck configurations can be cellular, sandwich, honeycomb or hybrid. For each of these configurations different versions

and products are available. As already mentioned in the case of pedestrian bridges the low stiffness can be a problem and the addition of further materials might be necessary. [52]

#### 2.4.4 Hybrid constructions

*“Hybrid constructions are built to enable two or more structural materials to take full advantage of their superior properties.”* [8]

##### 2.4.4.1 FRP reinforced concrete bridge decks

The first application FRP composite materials was the reinforcement of concrete structures. Due to the corrosion of steel reinforcement, worsened by the traffic and deicing salts, FRP composite materials are an attractive alternative as reinforcement material. Due to the lower production costs, GFRP reinforcement is the most common material, but carbon FRP and aramid FRP are also used. The actual durability of FRP reinforced bridge decks is still a concern, because there are no in-situ test results available and the laboratory tests are not sufficient descriptive. [52]

##### 2.4.4.2 FRP stay-in-place formworks for bridge construction

Another form of FRP-concrete combination is the FRP-SIP formworks, which *“serve as formworks for fresh concrete during construction and as reinforcement during service.”* [52]. The advantages of this system are, that the formwork does not have to be removed, is lightweight and provides a crack control for the concrete. There is no need for an additional reinforcement; nevertheless, the respective concrete slab is normally made of fiber reinforced concrete (FRC) for crack control. Both, open

profiles for bridge decks and closed forms for bridge columns, are in use. A major point in the design of such structures is to assure the bond between FRP-SIP formwork and the concrete. [44, 52]

#### 2.4.4.3 FRP Bridge Decks

An attractive solution to combine the advantages of different materials is a FRP bridge deck. The substructure as well as girders and trusses are built in reinforced concrete or steel, to assure the stiffness and reduce the deflection of the bridge. To build the bridge deck in FRPs is attractive, because it reduces the weight remarkable and creates a higher live load capacity. [8, 30]

#### 2.4.4.4 FRP cable-stayed bridges

FRP cable-stayed bridges can be produced in both ways, as hybrid constructions and as all FRP constructions. The limitations of the span of traditional steel and concrete materials, due to the self-weight, and the problems of sag effect and corrosion problems at steel cables, can be avoided by the use of FRP composite materials. The use of advanced composite materials might enable engineers to build super-long-span cable-stayed bridges. [52]

#### 2.4.5 Rehabilitation and Reparation/ Retrofitting

As previously mentioned the rehabilitation and retrofitting of deteriorated, damaged or substandard bridges is nowadays one of the most important issues for the civil engineer with even growing importance. Besides the trend of building new

bridges with composite materials for an improved durability, reinforced polymer composites have been established as a viable and competitive alternative for the repair, rehabilitation and retrofitting of existing constructions. For this application FRP is a very attractive solution, because of the exceptional strength-to-weight ratios and the quick and easy installation. The constructions can be retrofitted effectively, without long traffic constrictions. Flexural strengthening is one of the commonly used procedure, where FRP strips, bars, fiber fabrics or sheets are adhesively bonded to the soffit of the bridge girders or decks. Alternatively, *“FRPs can also be wrapped around concrete columns or piers to increase their capacity and ductility”* [52] and to build a barrier to prevent further steel corrosion. Another way to strengthening an existing bridge for increased traffic is the replacement of deteriorated reinforced concrete decks with FRP decks. Due to the reduced construction weight, the live load capacity can be considerable improved. The change of weight, stiffness and natural frequency has to be considered. [8, 22, 24]

## CHAPTER 3

### REVIEW OF LITERATURE: PEDESTRIAN WALKING LOADS

#### 3.1 Introduction

Determining live loads correctly is a main challenge in the design of each human build structure. The dimensions should be in an economic range and the safety of the users still has to be assured. Unlike dead loads, the distribution of live loads is not defined and the distributions with the maximum impact for different load combinations have to be estimated. In addition, depending on the individual characteristics of the live loads, aspects like dynamic response, fatigue and cumulative failure have to be considered during the design process. The correct specification of all possible loads is essential for a successful design. The design codes provide specified values for the different types of loads as design loads, based on experiences and the actual state of knowledge. These values are conservative estimations to give the designer reliable numbers for the design of safe constructions.

However, the peculiarity of live loads – moving, varying, and temporary – makes a generalization tedious. A compromise has to be found to assure a safe structure, without making it an uneconomical design, due to an overestimation of the load and the following dimensions. It gets even harder when, additional to the amount, the duration, impact and dynamic effects have to be considered. Traffic loads are especially hard to describe sufficient, due to the fact that they consist of an accumulation of many individual elements.



Because of the great importance and the major size of their shares at the live loads on bridges and other traffic areas, there is a great database for the impact of motorized traffic on structures. Numerous tests and studies regarding this topic have been conducted and led to a well-established model to compute traffic loads.

For non-motorized traffic, like pedestrians and bicyclists, such a detailed and convincing load model does not exist. Caused by small loads, compared to a car or a train, and less occurrence the pedestrian load has not got any special attention and thus is why there is a lag in the database and knowledge regarding the dynamical components of pedestrian loads. Pedestrians on footbridges and on each other kind of bridges or official walkways are, like humans on structural elements in general, often assumed as moderate live loads of a static nature. Therefore, structures constructed for such loads are often build with slender dimensions, overlooking or underestimating the possibility of a dynamic load and that the corresponding serviceability might determine the design dimensions. Additionally, the recent aesthetic trend requests for light and slender structures, which results in new constructions with reduced mass, stiffness and damping. Combined with the development and use of new materials, for example FRP, which are characterized by reduced weight and stiffness, this led to the construction of footbridges that are extremely sensitive to vibrations. The large oscillations of the so-called 'lively footbridges' have attracted the attention of the structural engineers and researchers, especially since these incidents have compromised the serviceability and structural safety of the constructions.

An unexpected phenomenon occurring at these bridges is the synchronous lateral excitation (SLE). Until then the focus was mainly directed towards the effects of

vertical excitation. However, incidents like the closure of the London Millennium Footbridge in 2000 have raised the awareness for the lateral excitation and the related synchronization. These incidents have also shown that the pedestrian load is much more complex than we have assumed before and that the approaches for the design of such bridges have to be adjusted. For a targeted adaption of the guidelines, the complex system of a footbridge loaded with a crowd of pedestrians and the resulting phenomena have to be analyzed and understood.

*“The interaction between the structure and the crowd walking on it, and among the pedestrians within the crowd, gives rise to a multi-scale multi-physic complex dynamic system. The latter is characterised by collective phenomena that are not only due to the features of the single system components but also to their interactions. Specifically, the crowd behaviour, in particular the pedestrian force exerted on the structure, affects the structural dynamic properties and response, and the latter modifies the behaviour of the pedestrian walking on the moving structural surface.”*

[49]

First, the understanding and modelling of such complex phenomena require the consultation of several research fields of a multidisciplinary frame. Even though the resulting problem (lively footbridges) is a topic in the field of civil engineering, neither are the description of the walking process, nor the crowd behavior are located in this field of study. Besides the civil engineering aspects, contributions from biomechanics, transportation, physics, applied mathematics and even biology and psychology have to be taken in consideration. For a clear arrangement of the combination of these different fields and for an easier understanding of the multi-scale

multi-physic complex dynamic system, the system has to be split into different levels, corresponding to an increasing degree of complexity. The first level is the individual level, which presents the behavior of a single pedestrian. This level provides the information regarding walking parameters and the time-load function of a single pedestrian. The second level is the collective level. Based on the assumption that the structure is loaded by a crowd and not a single pedestrian, the crowd behavior and the occurring phenomena are analyzed, the changes in the behavior of a single pedestrian are elaborated and the conditions for the accumulation of the single pedestrian forces to a crowd force are estimated.

The last and most complex level is the multi-physic level and it deals with the crowd-structure interaction. In detail, the last level contains the analysis of the change of dynamical properties of the structure due to the pedestrians and the influences of a moving structure to the walking behavior of the pedestrians. [49]

All three levels are presented in the following paragraphs.

## **3.2 Individual Level: Single Undisturbed Pedestrian**

### **3.2.1 General Characterization**

There are various types of dynamic loads produced by human activities. Both, transient and periodic loads are possible. The former might be a single impulse, caused, for instance, by landing on the floor after jumping from an elevated position or bumping against a wall. The latter is usually the result of the more common forms of human motion: walking, running, skipping and dancing. This is a wide categorization and for the analysis of particular structures, the categories have to be specified. The different types of motion include foot stamping and body rocking at a concert for

stadiums and rhythmic skipping during fitness classes for a gymnasium. For the purpose of a load model for pedestrian bridges the considerable motion categories can be reduced to walking and running. In the time of big sport events like city marathons and the increasing popularity of jogging as sportive leisure activity both, walking and running have to be taken into consideration. There are some significant differences between the two, regarding the load-time function, pacing rate, speed, motion sequence and dynamic impact factor. [7]

### 3.2.2 Walking Parameters

The basic component of human induced dynamic loads is the walking force of a single, undisturbed pedestrian. Since the presence of other pedestrians and the motion of the walking surface generally affect the walking behavior of pedestrians, an unimpeded behavior has to be generated for the measurement of the neutral parameters. The walking behavior of a pedestrian can be considered as unimpeded, when a single pedestrian is walking on a fixed ground. The velocity of a pedestrian walking undisturbed like this is generally referred to as free speed. In the past there have been made numerous researches concerning the walking parameters, mainly in the fields of biomechanics and transportation, but more recently also in the field of structural engineering. The tests have been performed in order to collect reliable data concerning the following parameters.

The measurement of the walking parameters is an elaborate process, because the walking parameters vary strongly, caused by a wide range of factors. The walking parameters are influenced by both, physiological and psychological factors. The

physiological factors contain the biometric characteristics of the walker, like body weight and height, age, gender and fitness. In addition, the walking parameters' variability is also contributed by cultural and racial differences, travel purpose and type of walking facility and other psychological factors. Furthermore, the test results depend on the method of measurement.

Because of the wide range of influence factors, the aim of the test conduction should be the collection of data to estimate the walking parameters of a person with average biometric characteristics and under average conditions. The actual variability of the parameters can be taken into account by the consideration of the probability function. Generally, the Gaussian probability density function is assumed the best fit for the measured data [59]. The median in combination with the standard deviation is sufficient as the base for a detailed and realistic simulation of the load.

Nevertheless, it is worth stressing that the geographical location of the structure should be taken into closer consideration, due to the fact, that even the average values vary strongly between different countries and cultural regions. [25, 36] For instance,

*“Japanese people are expected to walk with a higher frequency than European, as a consequence of both their different lifestyle and smaller average body dimension.”*

[15]. In the following the parameters necessary to describe the walking process sufficient are described and the average values for these parameters, collected from data out of a series of publications, are presented.

### 3.2.2.1 Frequency

Since pedestrian walking is a periodic load, the main parameter to describe walking is the frequency  $f_p$ . Sometimes it is also referred to as pacing rate  $f_s$  and is given as footfalls per second [FF/s] or, regarding its nature as loading frequency more adequately expressed, in Hz. In each case, it has to be differentiated between the vertical frequency  $f_{pv}$  and the lateral frequency  $f_{pl}$ . The former is intended as the walking frequency and describes the number of times a foot touches the ground in a time unit. Therefore, it can be measured in steps per time unit and it is relevant for the vertical excitation. The lateral frequency on the other hand describes the number of times the same foot touches the ground and its value is therefore half the vertical frequency. In the analysis of the lateral excitation of the structure this parameter is determining and hence, caused by the increasing number of lateral excitation incidents at lively footbridges, of growing interest. The two frequencies are strictly related by the factor of 2, thus the lateral frequency can be derived from the existing data for the walking frequency.

Source	$\mu_{f_{pv}}$ [Hz]	$\sigma_{f_{pv}}$ [Hz]	$\mu_{f_{pl}}$ [Hz]
Butz et al. [11]	1.84	0.126	0.92
Kerr and Bishop [23]	1.90	n.a.	0.95
Matsumoto et al. [31]	2.00	0.173	1.00
Pachi and Ji [35]	1.83 – 2.00	0.11 – 0.135	0.92 – 1.00
Ricciardelli [39]	1.84	0.172	0.92
Sanhaci and Kasperski [43]	1.82	0.12	0.91
Zivanovic et al. [A26]	1.87	0.186	0.94
Schulze [20]	2.00	0.13	1.00
Kramer [20]	2.20	0.3	1.10

**Table 2: Walking Frequencies: vertical (v) and lateral (l)**

As shown in Table 2, the walking frequency ranges between 1.5 and 2.5 Hz. Consequently, the mean ( $\mu$ ) of the Gaussian normal distributions is often assumed as 2.0 Hz with varying standard deviations ( $\sigma$ ). For jogging and running the walking frequencies rise with the higher pace and they also vary stronger, due to the individual biometric characteristics of the pedestrian. For normal jogging the pace rate ranges between 2.4 and 2.7 Hz and for sprinting it may be as high as 5.0 Hz. Considering public places like bridges and walkways, the expected frequencies can be limited to 3.5 Hz. [7]

#### 3.2.2.2 Velocity

Another characteristic of walking is the movement forward. This movement can be easily described by the walking velocity  $v_s$ . As already mentioned the considerable property in this section is the free speed that describes the velocity of an undisturbed pedestrian on a fixed ground. As mentioned above the values vary, caused by psychological and physiological influences, as well as the geographic areas. The waling velocity is also the topic of a wide range of publications and the results of numerous tests are available. [49]

Source	$\mu_{vs}$ [m/s]	$\sigma_{vs}$ [m/s]
Fruin [16]	1.40	0.15
Hankin and Wright [19]	1.60	n.a.
Koushki [26]	1.08	n.a.
Lam et al. [28]	1.19	0.26
Older [34]	1.30	0.30
Pauls [36]	1.25	n.a.
Ricciardelli et al. [39]	1.41	0.224
Sanhaci and Kasperski [43]	1.37	0.15
Sarkar and Janardhan [39]	1.46	0.63
Tanariboon et al. [11]	1.23	n.a.
Virkler and Elayadath [51]	1.22	n.a.
Weidmann [53]	1.34	n.a.

**Table 3: Walking Velocity ( $v_s$ )**

### 3.2.2.3 Stride Length

Another way to describe the pedestrian propagation is the step or stride length  $l_s$ . It is normally given in m. Like the other parameters it varies due to a number of influence factors, but above all it is depending on the biometric characteristics, like body height and weight, the length of the legs and the walker's fitness. The stride length is the walking parameter to whose statistical description the smallest number of works are devoted.

Source	$\mu_{l_s}$ [m]	$\sigma_{l_s}$ [m]
Sanhaci and Kasperski [43]	0.75	0.07
Wheeler [54]	0.75	n.a.
Ricciardelli et al. [39]	0.768	0.098

**Table 4: Stride Length ( $l_s$ )**



This might be because the three parameters, frequency, velocity and step length, are coupled by the fundamental law  $v_s = f_{pv} \cdot l_s$  [49] and therefore just two of them are necessary to describe the walking process sufficient. [7, 49]

### 3.2.2.4 Correlation

The law  $v_s = f_{pv} \cdot l_s$  describes how two of the parameters determine the third one, but caused by the complexity of the walking behavior it is difficult to find relationships between the three walking parameters. Nevertheless, relations between frequency and velocity have been proposed

by Butz et al.:  $f_{pv} = 0.7886 + 0.7868 \cdot v$  [11]

by Ricciardelli et al.:  $f_{pv} = 0.024 + 0.754 \cdot v$  [39]

and by Bertram and Ruina:  $f_{pv} = 2.93 \cdot v - 1.59 \cdot v^2 + 0.35 \cdot v^3$  [48]

Ojeda et al. gave the relation between velocity and stride length as

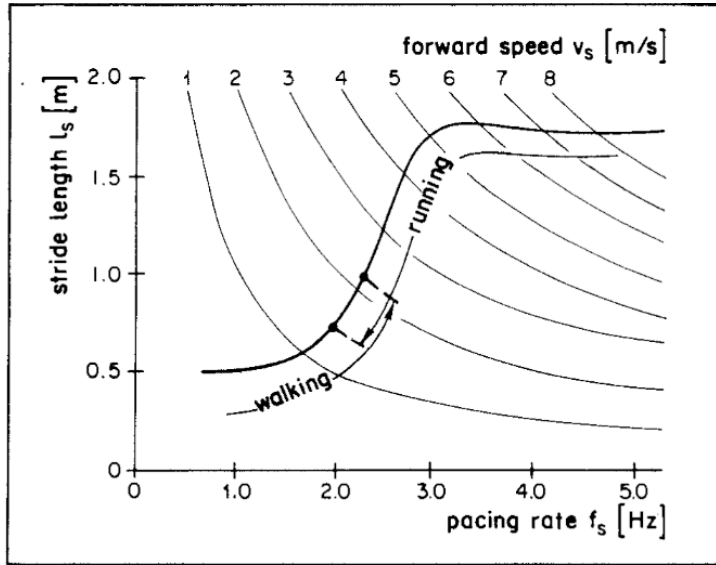
$$l_s = a \cdot v^b \quad [33]$$

where a and b are subject-specific constants.

	$f_s$ [Hz]	$v_s$ [m/s]	$l_s$ [m]
slow walk	~ 1.7	1.1	0.60
normal walk	~ 2.0	1.5	0.75
fast walk	~ 2.3	2.2	1.00
slow running (jog)	~ 2.5	3.3	1.30
fast running (sprint)	> 3.2	5.5	1.75

**Table 5: Correlation of pacing rate, forward speed and stride length [7]**

Even though these relations are not well established, the walking velocity is generally assumed as related to the walking frequency, as well as walking velocity and step length, while step length and walking frequency can be considered as uncorrelated. [49] Based on test results and average correlations can be estimated, as presented in the following figure.



**Figure 2: Correlation Forward Speed and Stride Length [7]**

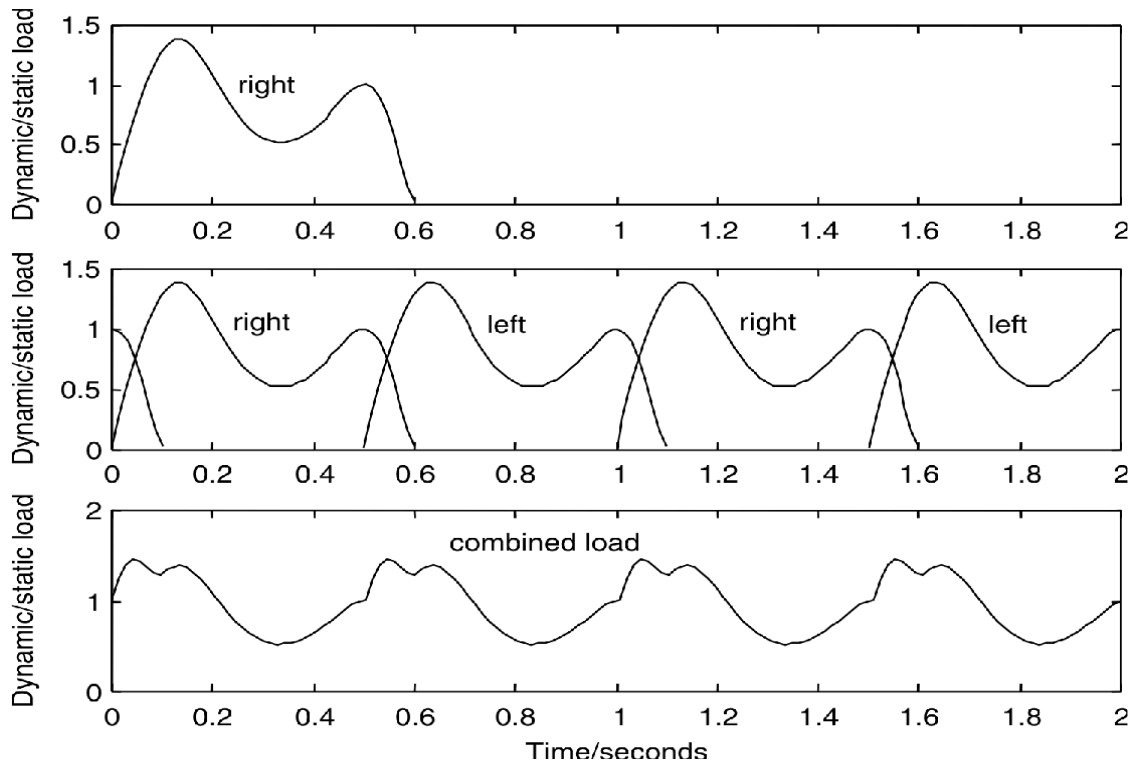
### 3.2.3 Walking Loads

During walking, the pedestrian exerts dynamic forces on the ground. These forces include components of all three directions: vertical, horizontal-lateral and horizontal-longitudinal. They are produced by the combination of a normal and a shear stress field applied on the ground. As expected the vertical component has the highest magnitude of the three, but the recent experiences show that the horizontal components can not be neglected. The actual load-time function is affected by the individual walking parameters, the weight of the pedestrian as well as his footwear, the surface conditions and measuring techniques. Many studies have focused on the

walking force, thus numerous and detailed data are available, measured with force plates or treadmills. The load-time function is affected by a lot of factors, but the pacing rate is the parameter of the greatest importance. The walking frequency is dispositive for the general shape of the load-time function and also influences the dynamic impact factor. There are three significant characteristics of the load-time function, which are changing with an increasing pacing rate. [49]

### 3.2.3.1 The Load-Time Function of the Vertical Component

During walking with a medium frequency the load-time function of a single step has normally the shape of a saddle, characterized by two observable load maxima. Caused is this feature by the general walking sequence, where the foot steps with the heel, what causes the first maximum, and pushes off with the ball of the foot, resulting in the second maximum. With an increasing walking frequency width the trough between the two maxima is shrinking so that this particular “*feature disappears with increasing pacing rate and degenerates to a single maximum of sharp rise and descent when the person is running.*” [7]. At a really slow pacing rate the function has the shape of a block, caused by the decreasing impact factor. The slow loading of the foot eliminates the amplification of the load caused by the step and push off movement, that is why the load-time function stays even during the foot is loaded.



**Figure 3: Load-Time Function: Vertical Component [29]**

The maximum load:

With an increasing walking frequency, the load maximum increases. This is caused by the load impact factor. A rapidly applied load causes generally larger stresses than those that would be produced if the same load would have been applied gradually. This dynamic effect of the load is referred to as impact and the ratio between the weight of the element and its dynamically increased load is referred to as impact factor. With a rising pacing rate the speed of the foot stepping on the ground is also increasing, therefore the impact factor rises and maximum load ascends. *“While for strolling with a frequency below 1 Hz the maximum load hardly exceeds the weight of the person, it increases by a quarter or a third for 2 Hz and by a half around 2.5 Hz; at about 3.5 Hz the maximum reaches about double the weight of the test person. [...] For fast running the maximum load can increase to three times the weight.”* [7] In

the vertical case, the fundamental frequency is controlling and harmonics amplitude of the harmonics are less than about 30 per cent. [7]

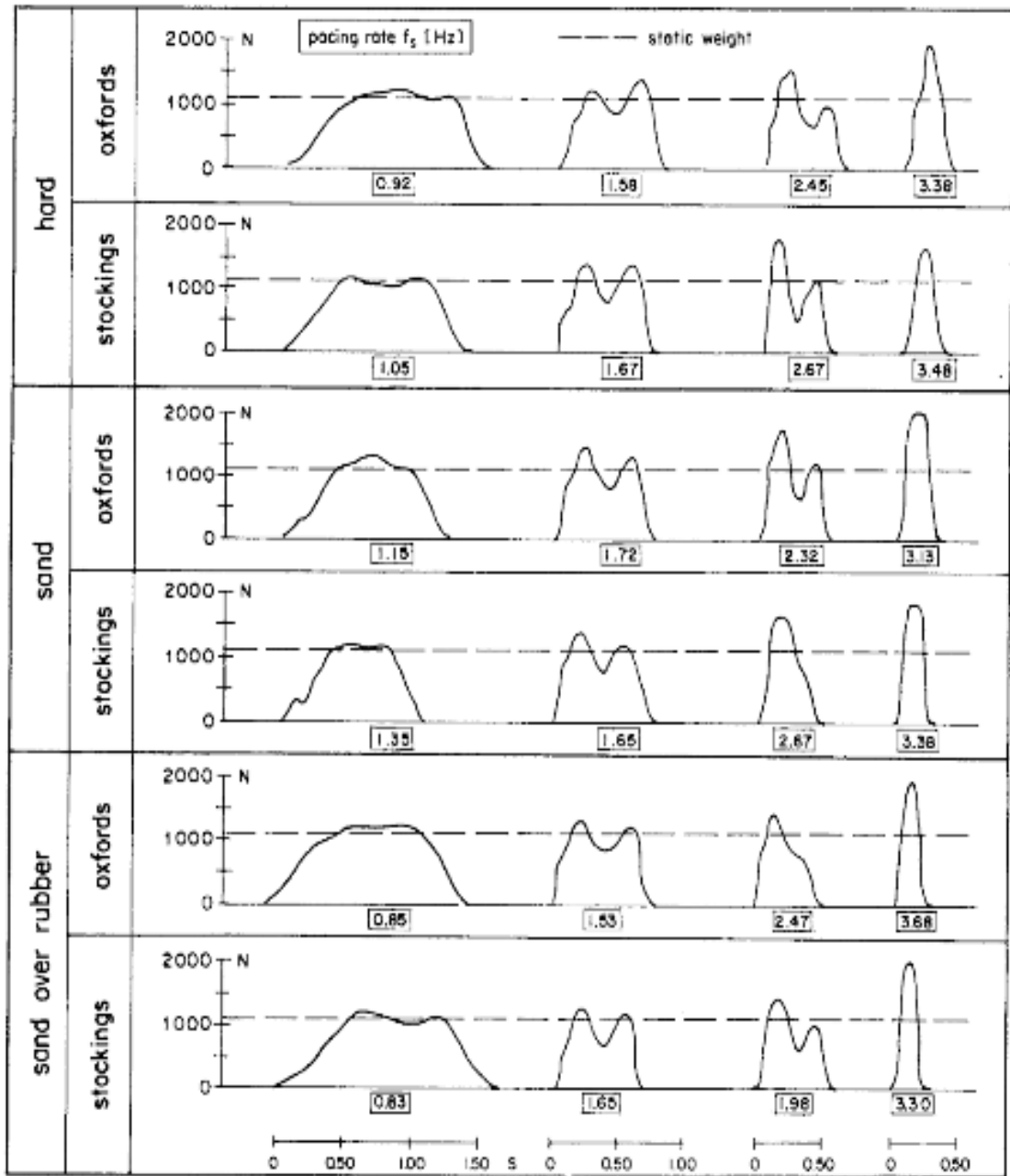


Figure 4: Load-Time Function: various pacing rates and pavements [7]

Duration of foot contact:

The last considerable characteristic is the contact duration of the single foot and the ground contact considering both feet. During walking, at each time one of the feet is touching the ground. In case of the load-time function this can be seen by the overlapping of the contact duration and load. One foot is being unloaded while the other one is being loaded, before the now unloaded foot is moved for the next step. This leads to a time variation of the total dynamic load during walking, which has components in the 2<sup>nd</sup> and 3<sup>rd</sup> harmonic – maxima of the function appears also at the double and triple of the pacing rate. In contrast to the walking behavior, the behavior during running is characterized by an interrupted ground contact. The contact times of the two feet are separated by periods with no contact to the ground. [29]

For one approach of the mathematical idealized formulation of the dynamic load, they differ between ‘continuous ground contact’ and ‘discontinuous ground contact’. The load, excited by walking, which exhibits an overlap of the individual contact times of either foot and produces therefore a continuous ground contact, can be idealized by the following expression: [11]

$$F_p(t) = G + \Delta G_1 \sin(2 \cdot \pi \cdot f_s \cdot t) + \Delta G_2 \sin(4 \cdot \pi \cdot f_s \cdot t - \varphi_2) + \Delta G_3 \sin(6 \cdot \pi \cdot f_s \cdot t - \varphi_3)$$

where:

- $G$  = weight of the person (generally assumed to  $G = 800$  N)
- $\Delta G_1$  = load component (amplitude) of 1st harmonic
- $\Delta G_2$  = load component (amplitude) of 2nd harmonic
- $\Delta G_3$  = load component (amplitude) of 3rd harmonic
- $f_s$  = pacing rate
- $\varphi_2$  = phase angle of the 2nd harmonic relative to the 1st harmonic

$\varphi_3$  = phase angle of the 3rd harmonic relative to the 1st harmonic

In most cases the forced vibration induced to simulate a walking person is governed by just one harmonic and the phase angles become immaterial. The force component of the 1<sup>st</sup> harmonic is given in the literature as:

$$\Delta G_1 = 0.4 \cdot G \text{ for } f_s = 2.0 \text{ Hz}$$

$$\Delta G_1 = 0.5 \cdot G \text{ for } f_s = 2.4 \text{ Hz.}$$

If the 2<sup>nd</sup> and 3<sup>rd</sup> harmonics are considered, both force components are often assumed as

$$\Delta G_2 \cong \Delta G_3 \cong 0.1 \cdot G$$

with the approximated phase angle of

$$\varphi_2 \cong \varphi_3 \cong \pi/2. [11]$$

It is also possible to describe the function in a more general form as a Fourier series:

$$F_p(t) = G + \sum_{i=1}^{\infty} G \cdot \alpha_i \cdot \sin(2 \cdot \pi \cdot i \cdot f_s \cdot t - \varphi_i) \quad [11]$$

where:

- $G$  = weight of the person (generally assumed to  $G = 800 \text{ N}$ )
- $\alpha_i$  = Dynamic Load Factors of the  $i^{\text{th}}$  harmonic (i.e. ratio of the force amplitude to  $G$ )
- $i$  = the order number of the harmonic
- $f_s$  = pacing rate
- $\varphi_i$  = phase angle of the  $i^{\text{th}}$  harmonic relative to the 1<sup>st</sup> harmonic

In the case of the discontinuous ground contact, the description of the load-time function within one period has to be differentiated between the duration with contact to the ground and without contact to the ground. The former one, “generally characterized by a single load maximum, can be expressed by a sequence of semi-sinusoidal pulses” [49]. The function within one period is given by:

$$F_p(t) = \begin{cases} k_p \cdot G \cdot \sin(\pi \cdot t/t_p) & \text{for } t \leq t_p \\ 0 & \text{for } t_p \leq t \leq T_p \end{cases} \quad [49]$$

with:  $k_p$  =  $F_{p,max}/G$  = dynamic impact factor  
 $F_{p,max}$  = peak dynamic load  
 $G$  = weight of the jogger (generally assumed to  $G = 800$  N)  
 $t_p$  = contact duration  
 $T_p$  =  $1/fs$  = pace period.

But each of these deterministic descriptions is not sufficient in the detailed description of the single pedestrian load, affected by the intra-subject variability. In addition to the already mentioned psychological, physiological and environmental variables, which influence the walking parameters of an individual pedestrian, the loading varies at each step. In fact, pedestrians are not able to reproduce the loading of one step exactly. To a certain extent the loading follows a randomness, which can be acknowledged by the statistical characterization of the walking variables as means of the probability density function within a periodic force model, or by describing the force in terms of its Power Spectral Density (PSD). [49, 7]

### 3.2.3.2 The Load-Time Function of the Horizontal Component

The horizontal components of the loading from human walking or running are much smaller as the vertical component. But so are the design values for these components too and in the case of lively footbridges they may become a problem. It is worth pointing out that especially the so-called ‘Synchronous Lateral Excitation’ [49] effect has become a problem in a number of cases, because it leads to pedestrians walking in step, which in turn leads to the in phase accumulation of the dynamic



forces of the individual persons. The lateral component of the walking force is produced by the sway of the person's center of gravity. This sway occurs due to the small distances between the feet and the centerline of the body and the altering movement of the center of gravity in correlation with the load shift from one foot to the other. The lateral loading can be described as:

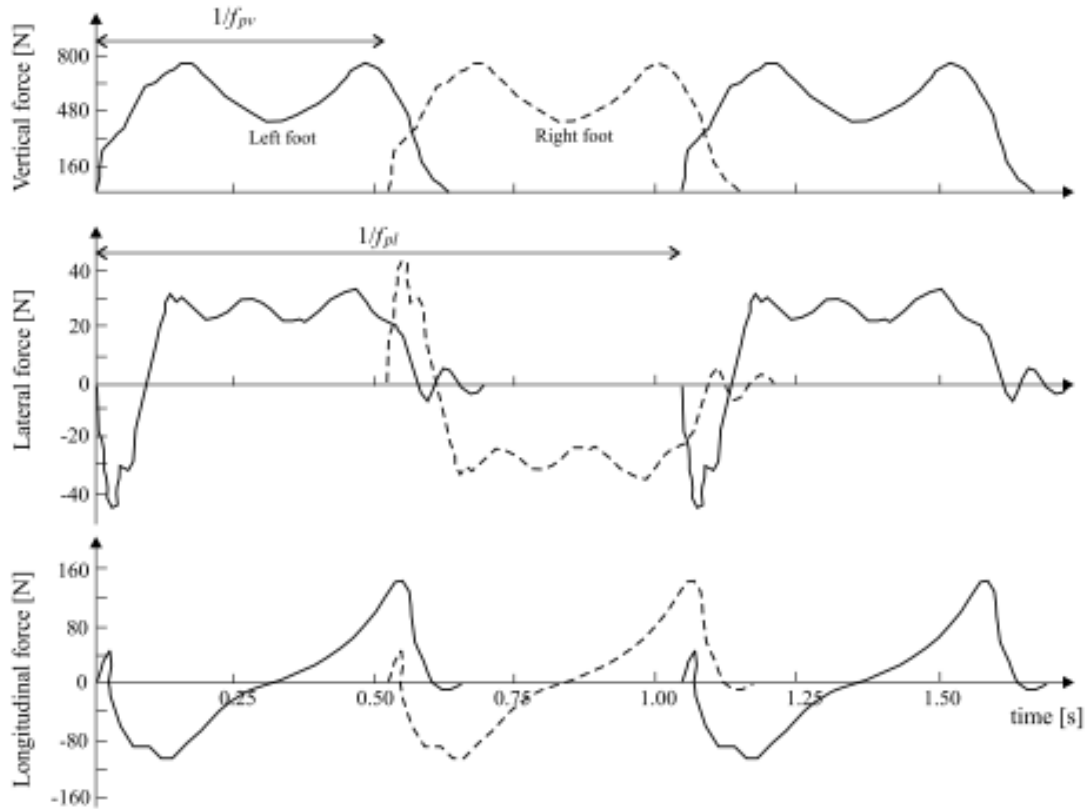
$$F_l(t) = \sum_{i=1}^{\infty} G \cdot \alpha_i \cdot \sin\left(2 \cdot \pi \cdot i \cdot \frac{f_s}{2} \cdot t - \varphi_i\right) \quad [49]$$

where:

- $G$  = weight of the person (generally assumed to  $G = 800$  N)
- $\alpha_i$  = Dynamic Load Factors of the  $i^{\text{th}}$  harmonic
- $i$  = the order number of the harmonic
- $f_s$  = pacing rate
- $\varphi_i$  = phase angle of the  $i^{\text{th}}$  harmonic relative to the 1<sup>st</sup> harmonic.

Only a few researches report results concerning the dynamic load factor of the lateral component, but a dominance of the first and third harmonics has been found. [7, 49] Bachmann and Ammann have also quoted the value of “4 per cent of the static weight” [32, 5] as the lateral dynamic loading during normal walking.

The longitudinal component, even though it is larger than the lateral component, is in the structural analysis often neglected, due to the great stiffness of line-like structures in the longitudinal direction. The load-time function of all three components, vertical, lateral and longitudinal are presented in the following figure:



**Figure 5: Load-Time Function: Vertical, Lateral and Longitudinal Component [49]**

### **3.3 Collective Level: Human-Human Interaction**

#### **3.3.1 General Characterization**

Theoretically, the loading induced by a number of pedestrians can be described by the accumulation of the load-time functions of the walking loads produced by the individual participants. However, for the description of the actual loading this approach is not sufficient, due to the human-human interaction. Even this component of the complex phenomenon ‘crowd induced walking load’ has a comparatively small impact, it is worth pointing out some aspects, which influence the walking load slightly and might even change the outcome of the dynamic response analysis.

### 3.3.2 Analysis Level

#### 3.3.2.1 General Aspects

“*Crowd modelling can be distinguished into microscopic-level and macroscopic-level, based on the level at which crowd analysis is being performed.*” [42] The former describes how each individual within a crowd reacts to its surrounding and therefore to the surrounding individuals and their behavior. In addition, described in latter level, group dynamics can be observed, which leads to “*a complex and coordinated collective behavior*”[25] and indicates an interaction that exceeds the reaction of an individual to its surrounding. This phenomenon is known as emergent behavior. Among others, it is responsible for the formation of walking lanes and the prevention of collisions. Besides the description of the behavior of an average crowd, coincidentally compounded of independent individuals, it is worth to consider public events like demonstrations or city marathons. Because of the shared goal or walking purpose, the cohesiveness within the crowd and the interaction between the individuals gets stronger and influences also the resulting walking load. [10, 9, 2] All of the three cases are briefly described in the following paragraphs.

#### 3.3.2.2 Microscopic-Level

The individuals in a crowd interact with their environment; “*they unconsciously alter their behavior in line with the response of neighbouring entities*” [25]. This interaction, even though it is unconscious, follows a set of simple rules to assure the realisation of the personal goals without a discrepancy with the social norms. An average walking pedestrian moves towards a goal destination and aims to stay as close

as possible to the shortest route between his origin and the aspired destination. Inside a train station, individuals tend to move towards an entry or exit, walking on a bridge, they head to the other side. Even though each individual has his own goal destination and motion tendency, on their way they keep a distance to persons and obstacles in order to prevent collisions and, out of comfort, they avoid sudden changes in direction and velocity. The pedestrians have to adjust their routes according to these rules and the walking parameters are affected by them.

In contrary to the free speed, the walking parameters, especially the velocity, of a person moving in a crowded area are not determined by the person's abilities and wishes, but additionally by the crowd density (number of pedestrians per unit of area) and the general velocity of the crowd. With an increasing crowd density, each pedestrian has less space to its disposal, the strived distance to each other is not realisable anymore and the walking velocity has to be reduced to avoid collisions.

In addition, assuming a crowded scenario, passing of the foregoing pedestrian is not possible; the pedestrian with the lowest walking velocity determines the speed of the whole crowd or at least the people behind him. In conclusion, the average walking velocity in a crowded scenario is smaller than the average free speed. The other walking parameters, correlated to the velocity, might change too, but not necessarily on the same scale. [25]

### 3.3.2.3 Macroscopic-Level

Even though each individual is self-organized and its walking behavior inside a crowd seems to follow just the described rules to avoid collision, forming a crowd,

they appear to share common motion dynamics and together “*they portray a complex and coordinated collective behavior*” [25]. An example for these crowd dynamics is the forming of uniform walking lanes in crowds with groups of opposite moving directions, even without communication or a leadership. This emergent behavior is the subject of several researches with the purpose to investigate the underlying mechanism that allow this unity in the crowd. Collective crowd behavior arises in swarms or crowds with certain class of entities (e.g. insects, human, animals, etc.) and follows some physical laws. That is why there are models in both fields, biology and physics, available.

From the biology point of view, individuals in a crowd resemble the entities in a swarm. They observed that each entity acts independent and conform to a set of rules, while as a whole the swarm acts in a sophisticated way and forms something like a collective ‘group-mind’, which helps individuals to reach their goals. One aspect of this model is the “*natural reflect that is deeply rooted in each entity (specifically human) to conform to social norm*” [25].

On the contrary, associating crowd behavior with the laws of physics, the crowd is assumed as a homogenous mass of bodies. “*The idea of relating the motion of crowd with fluid, liquid or electrons in aerodynamics, hydrodynamics or continuum mechanics respectively, has generated many research in crowd analyses since the past years. Accordingly, physics-inspired studies assume that the individual in a crowd tends to follow the dominant flow of the crowd and thus, the motion of highly dense crowd resembles fluid. Hence, theories and methods in fluid mechanics are adopted to comprehend the flow of human crowd. In another physics-inspired example, the*

*kinetic theory of gases is applied to model the sparse and random interaction forces amongst individuals in a crowd.*”[25]. In the field of physics the individuals are characterized as non-thinking particles whose motions are dictated by external forces.

Both approaches gain convincing results and share similar understanding and perspectives. Nevertheless, existing models are insufficient in understanding the interaction between individuals and their environment in total. Additionally, they do not take the possibility of subgroups and their influence on the crowd behavior into consideration. [25] Social interactions such as walking in pairs or in groups and the resulting harmonization of the walking parameters to each other leads in average to a smaller velocity.

#### 3.3.2.4 Special events

The accumulation of the walking loads of a group of pedestrians is coincidental. The maxima and minima of the individual load-time functions are randomly shifted to each other and the result is a nearly constant load function, because the peak values compensate each other. A higher risk for line-like structures poses crowds marching in step, because in that case, the accumulation is not coincidental anymore and the maximum of the resulting load function is the sum of the single load function maxima. Therefore, the synchronization of the pedestrians within a crowd is a hazard, which has to be considered during the dynamic analysis.

There have been some accidents in history, where soldiers marched in step over slender bridges and combination of the summed loads and a marching frequency close to the natural frequency of the structure led to a resonance response and eventually to

the collapse of the structure. Nowadays the risk of people marching in step is still existing, even though it is likely that they are not soldiers but demonstrators or participants of an big event, supported with music or drums and marching in step with the music. Even a bridge that has been designed to carry motorized traffic and therefore great loads can be affected by such an event. [49]

Another kind of public event that should be considered is sport events, especially marathons. The risk of these is not the synchronization but the changed walking mode. Jogging and running produce higher amplitudes in the load function than walking and an accumulation of these loads might result in considerable high loads. However, the crowd density in this scenario is noticeable lower, because the increased speed demands greater distances to surrounding peoples. As a result, the summed load would not be significant higher. In an approach for a simplified dynamic analysis of slender bridges, the scenarios of special and public events and the resulting loads should be included. [7]

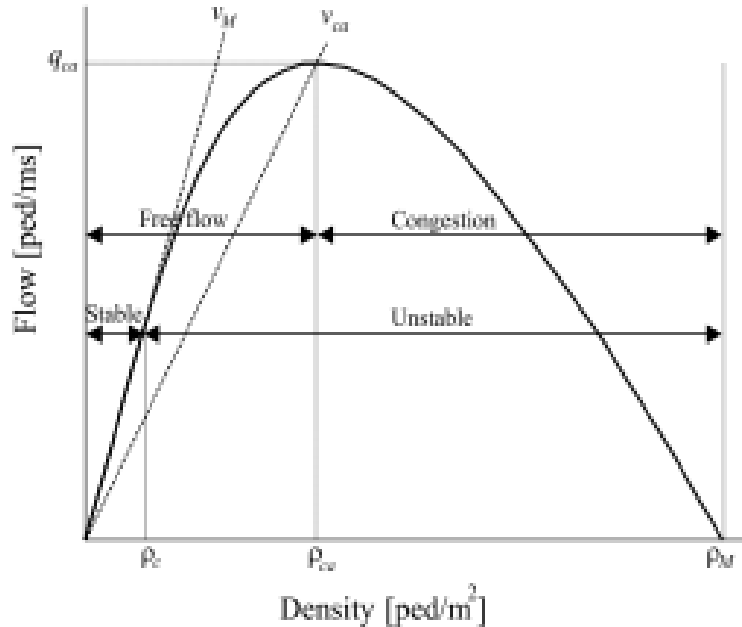
*“In order to complete the introductory overview of the topic of interest, an additional issue to be considered arises by the onset of panic conditions, which substantially modify the crowd dynamics.”* [49]. In addition to aspects of a safe evacuation panic conditions can lead to an increased crowd density and hence to higher loads. However, since synchronization does not occur in panic conditions and a high crowd density restricts the movement of individuals it is less a dynamic but more a general load problem and hence this issue is not of particular interest in this text. [7, 49]

### 3.3.3 Simplifications

In the field of structural engineering, especially for a structural analysis, the detailed analysis of a microscopic level is not expedient. The used measurements for walking parameters found in literature usually refer to averaged quantities and within a crowd occurs a assimilation. Additionally, the focus is normally at the general crowd behavior and if it has influence on the resulting load. *“One of the main feature of crowd behaviour is that the walking velocity is affected by the crowd density, namely the higher the crowd density, the lower the walking velocity. Many studies have been directed to the determination of a law that links the walking velocity to the crowd density.”* [49].

For a computational simulation of a pedestrian crowd on a structure, the adjusted walking velocity is worth to consider. A crowd can be described by three main variables.  $q$  [ped/ms] is the flow, namely the number of pedestrians passing a cross-section of an area in a unit of time.  $v$  [m/s] is the average walking velocity and  $\rho$  [ped/m<sup>2</sup>] is the crowd density. The three parameters are related by the fundamental relation  $q = \rho \cdot v$  [32] and are graphical represented by fundamental diagrams.





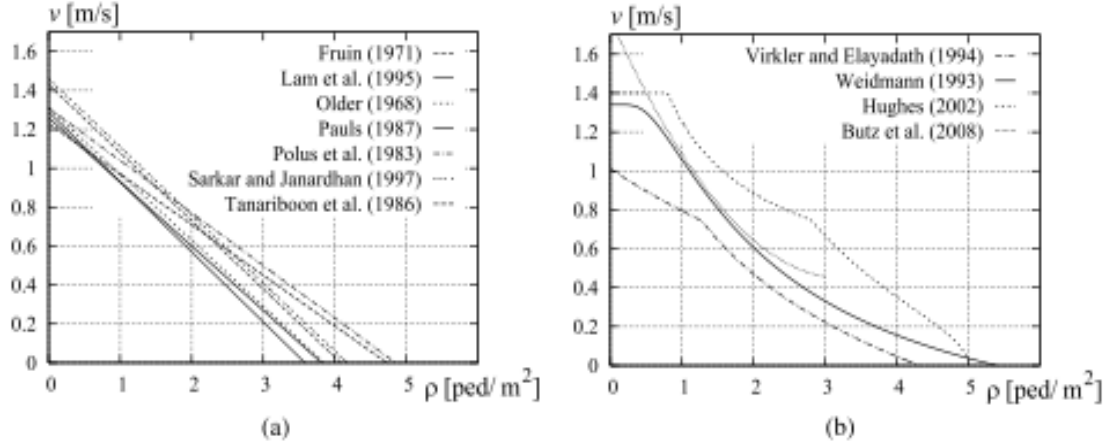
**Figure 6: Flow-Density Fundamental Diagram [10]**

Looking at the diagram some relevant quantities regarding crowd behavior can be identified:

- Until the critical density  $\rho_c$  is reached the pedestrians are unimpeded and walk with constant free speed  $v_M$ .
- For higher densities ( $\rho > \rho_c$ ) the walking speed decreases with increasing density.
- The highest flow occurs at the combination of a capacity speed  $v_{ca}$  and a capacity density  $\rho_{ca}$ .
- $\rho_M$  is the maximum admissible density corresponding to null speed and flow.

*“The values of the aforementioned variables are not expected to be universal, since walking behaviour is influenced by a great number of microscopic factors, such as age, culture, gender, travel purpose, type of walking facility and single or multiple walking direction, as observed for the walking parameters at the individual level.”*

[49] Nevertheless, based on experimental measurements some authors have proposed approaches to describe the correlation between density and velocity.



**Figure 7: Speed-Density Relations [10]**

Another approach introduces factor to account for the influence on the walking velocity of both psychology and physiological level.

$$v = v_M \left\{ 1 - \exp \left[ -\gamma \left( \frac{1}{\rho} - \frac{1}{\rho_M} \right) \right] \right\} \quad [50]$$

where:  $\rho_M = \frac{1}{\beta_G S_m}$  (jam density)

$S_m = 0.13 \text{ m}^2$  (surface occupied by motionless ped)

$v_M = \bar{v}_M \alpha_G \alpha_T$

$\bar{v}_M = 1.34 \text{ m/s}$  (average free speed)

Travel purpose $\gamma$			Geographic area $\alpha_G$			Travel purpose $\alpha_T$		
Rush hour/ Business	Commuters/ Events	Leisure/ Shopping	Europe	USA	Asia	Rush hour/ Business	Commuters/ Events	Leisure/ Shopping
$0.273\rho_M$	$0.214\rho_M$	$0.245\rho_M$	1.05	1.01	0.92	1.20	1.11	0.84

**Table 6: Coefficients of Geographic Area and Travel Purposes [50]**

All the relations refer to a one-directional flow. Adding a contrariwise flow leads to a reduction of the flow capacity, due to passing pedestrians. [49]

### **3.4 Multi-Physic Level: Human-Structure Interaction**

#### **3.4.1 General Characterization**

The interaction between human and structure is a sophisticated process in which various components and phenomena are included. Bridges are complex constructions and several mechanisms contribute to their dynamic response. The dynamic properties of a structure are hard to predict and depend on materials, construction dimensions, weight and general assembly of the structure, as well as the realization of the details.

Due to occupants and the specific characteristics of human bodies, these dynamic properties are additively changed. On the other hand, the loading function of a crowd in motion, which is also complex and related to many variables, can be decisively influenced by the interaction with the respective construction. This was brought to particular attention in the field of structural engineering by the opening event of the London Millennium Footbridge in June 2000. Here caused this interaction the great oscillation amplitudes in lateral direction. Since then a great number of researches have dealt with this topic and some interesting correlations and connections have been detected. Overall, two main parts of the interaction have been identified: in the context of dynamic response analysis, they can be described as positive and negative damping effects, as described in the following.

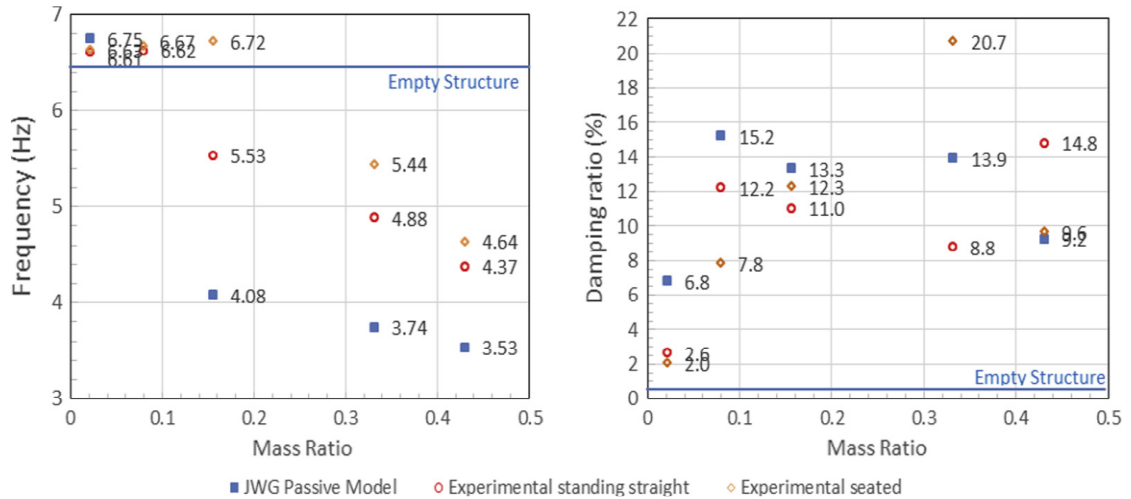
### 3.4.2 The Positive Damping Effect

In the field of dynamical analysis, damping describes the dissipation of energy due to several effects like material dissipation and connections. Besides the dynamic load impact and the positive feedback effect, which is presented in the following paragraph, pedestrians have also a positive that means stabilizing effect on the structure. Contrary to other dynamic loads, like wind, earthquakes, etc., traffic loads add additionally to the load input also a mass component to the hypothetical single degree of freedom system and, in case of pedestrians, it changes also the damping coefficient. One of the main components in a dynamical analysis is the mass. In the case of a single degree of freedom system, it is directly related to the natural frequency. To model a bridge construction as a SDOF system the equivalent mass has to include the added mass of the occupants. The relative ratio of the mass of the occupants to the structure is significant; in some cases, it can reach values higher than 1.0, which means that the weight of the occupants exceeds the weight of the empty structure. The changes in the dynamical analysis, especially the natural frequency, are relevant. [41]

Additionally, caused by the unique body characteristics of humans, the pedestrians influence the damping coefficient of the considered system. “... *the occupant acts as a dynamic spring-mass-damper system attached to the empty structure thereby affecting the dynamic properties of the combined system.*” [41] The case of stationary people is well known. Here it can be differed between people in different postures, like sitting or standing, with straight or bent knees. The different results for these postures give the explanation for the effect.

The human body is not rigid and has spring-like characteristics, due to the joints, like knees and ankles. Even though it is unconscious, the people counteract the appearing vibrations due to the softness and flexibility of their bodies. The effect increases in the case of bent knees in comparison to the case of the straight knees. That validate the theory of people as spring-mass-damper. In the case of walking people, the effect is less well known and established. In addition to the spring-mass-damper approach, which applies also on walking people, another approach to explain the effect has been published. It claims, that the *“humans’ inability to synchronise their pace with vertically moving surfaces causes the vibration to diminish.”* [49]. The actual effect might be a combination of both theories. [11, 20, 49]

In each case it is important to account for the effect within a dynamical analysis, because neglecting to do so *“may result in an overestimation of the dynamic response of a structure, and as a result, a more costly structural design.”*[41] It is of great importance, because experiments demonstrate that the described effect, meaning occupants at a bridge, can change the damping factor of the system by a factor of 10. The difficulties are in the estimation of the applicable values for specific cases, since they depend on several parameters, among others on the relative ratio of the average walking frequency of the occupants to the natural frequency of the empty structure and the relative ratio of the mass of the occupants to the structure. It is worth stressing out that the effect of additional damping applies in this form just on the vertical direction. In the lateral direction contrary effects can occur. [41]



**Figure 8: Frequencies and Damping Ratios due to the Positive Damping Effect [41]**

### 3.4.3 The Negative Damping Effect – Synchronization

Incidents like the closure of the London Millennium Footbridge after great vibrations at the opening event or the excitation of serious vibrations at the Auckland harbour Bridge in New Zealand, an eight-lane motorway bridge, due to a crossing political march, brought the negative damping effect caused by pedestrians to attention of structural engineers. A similar self-excitation mechanism is known in the field of wind-engineering, where vortexes result alternating forces, which causes initially small oscillations to build up. The effect is called negative damping, because it amplifies the dynamical response of the structure, instead of reducing it. It is also called positive feedback. *“The phenomenon of ‘synchronization’ by which people respond naturally to an oscillating bridge when this has a frequency close to their natural walking or running frequency is a feature of this phenomenon.”* [32]

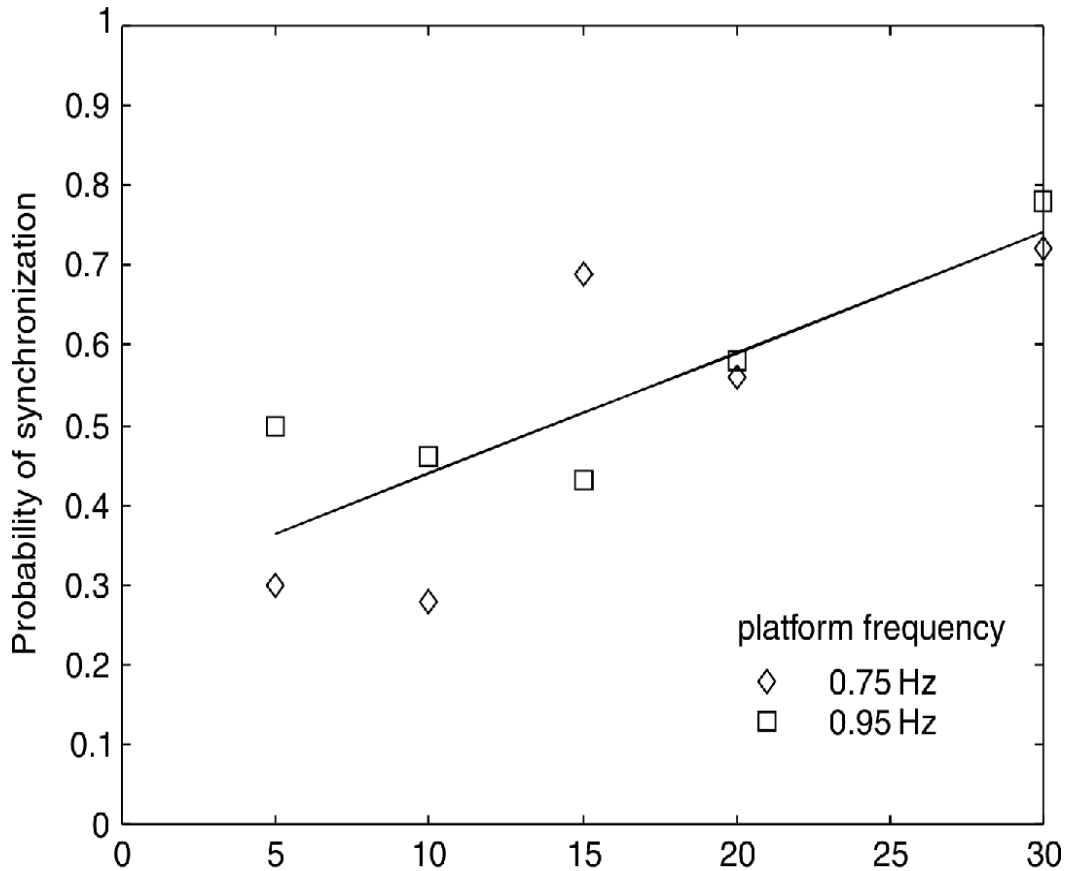
When a bridge is loaded with a crowd of pedestrians small lateral motion might occur, caused by the random lateral walking forces. The human body is sensitive to lateral motions and automatically he attempts to re-establish the balance by moving

his body in the opposite direction. This reaction leads to changes in the walking behavior of the pedestrian. Firstly, the lateral width between the feet increases, because the pedestrian has to counteract the lateral acceleration of the pavement, and that leads to higher lateral forces. Secondly, the pedestrian synchronise his walking to the swaying frequency, that is the natural frequency, of the structure.

The enlarged load, adjusted to the resonance frequency, causes in turn an increased motion of the structure. The threshold at which a pedestrian starts to synchronize with the oscillation varies from person to person, which is why the number of synchronized people growth gradually. Consequently, the motion of the structure increases respectively. *“Of course, because of adaptive nature of human being, the girder amplitude will not go to infinity and will reach a steady state.”* [17] Mainly, the induced force is restricted by the physiological limitation of the step width and characteristic of humans to stop walking when the motion is high enough to scare them.

The requirements for a synchronous lateral excitation are a natural frequency of the structure close to the average walking velocity and initial motions higher than the thresholds. In case of the vertical direction and therefore for a frequency around 2 Hz the threshold is 8-12 mm. For lateral vibrations with a frequency of about 1 Hz the threshold seems to be 4-6 mm [6]. This consonance to the research of Arup [32] following the Millennium Bridge incident. The low lateral threshold confirms the human sensitivity to lateral vibrations and underlines the importance of this effect, because even massive concrete bridges can be affected. The graph confirms also the

trend that people synchronize with each other, even when there is no pavement motion.



**Figure 9: Probability of Synchronization [49]**

*“They also found that the lateral forces of the feet-apart gait are phase synchronized to the structure and approach 300N amplitude per person, which these researchers pointed out is four times the Eurocode DLM1 value of 70N for normal walking.” [32]. The general conclusion to this topic is to avoid natural structural frequencies in the range of the walking frequency and its third harmonic. This rule might not be adequate, because it leads to unnecessary heavy and costly constructions,*



and eliminates innovative designs and the use of new materials. A model to consider this effect has to be found. [5, 6, 17, 32, 49]

## CHAPTER 4

### REVIEW OF LITERATUR: DESIGN CODE REQUIREMENTS

#### 4.1 General Aspects

Design codes and guidelines formulate requirements for buildings and constructions to ensure their structural safety and durability as well as serviceability. Their aim is to establish standards for the design of structures and a general level of safety, especially for public constructions. Design codes and guidelines differ within different countries and are separated in different fields, materials and construction types. The most relevant guidelines for pedestrian bridges and FRP pedestrian bridges available in the United States are AASHTO Guide Specifications for Design of Pedestrian Bridges [2], AASHTO LRFD Guide Specifications for the Design of Pedestrian bridges [3] and AASHTO Guide Specifications for Design of FRP Pedestrian Bridges [1]. The following paragraphs summarizes the requirements and restrictions formulated by these guidelines.

#### 4.2 Definition and Application

The AASHTO (American Association of State Highway and Transportation Officials) claims that pedestrian bridges “shall be designed for specified limit states to achieve the objectives of safety, serviceability, including comfort of the pedestrian user (vibration), and constructability with due regard to issues of inspectability, economy, and aesthetics” [3] and their formulated requirements are meant to reach this goal.

The Guide Specifications apply for pedestrian bridges, which is defined as a bridge “intended to carry, primarily pedestrians, bicyclists, equestrian riders and light maintenance vehicles, but not designed and intended to carry typical highway traffic” [3]. Consequently, the bridges has to be designed considering both a live load representing a dense pedestrian crowd and a maintenance vehicle. The configuration of the latter one can be determined by the Operating Agency; alternatively, there are design values available in the guidelines. The vehicle load has to be applied, even without a vehicle allowance, but it can be neglected, provided vehicular access is physically prevented. [1] Bicyclists are not expected to induce design-controlling loads that is why they are not further considered within the guidelines. The equestrian load is also not expected to control the design of the total structure, but can produce a significant patch load due to a high hoof pressure during a canter of the horse, which may control only the deck design. [3] Thus is why this load case can be also neglected within this research.

### **4.3 Design Loads**

#### **4.3.1 Pedestrian Live Load**

The guidelines demand the application of a uniform pedestrian loading to the walkway area. “*This loading shall be patterned to produce the maximum load effects.*” [3]. The actual values vary within the different specifications and guidelines, but are generally based on the maximum credible pedestrian load. Due to physical limits, the maximal load induced by pedestrians is restricted. It depends on the compounding of the crowd and if individual movement is still possible. Are standing crowd can have a

high pedestrian density that cannot be reached within pedestrian traffic. 85 psf (4.07 kN/m<sup>2</sup>), which is proposed in [1], is considered “*a reasonably conservative service live load that is difficult to exceed with pedestrian traffic*” [1]. Other guideline specifications provide higher values, but allow reductions based on loaded length or area, considering the lower probability that a big area is crowded on a maximum level. In cases of special events or locations, for instance close to stadiums with big sport events, this reduction might not be appropriate and includes an unnecessary risk. The following table presents the pedestrian live load design values provided by different guidelines.

Guideline	[kN/m <sup>2</sup> ] ([psf])	comment
[1]	4.07 (85)	Exceeds 400 ft <sup>2</sup> : $w=85 \cdot (0.25 + (15/VA))$
[2]	(90)	Consideration of dynamic load allowance is not required with this loading
[3]	4.07 (85)	average person occupying 2 ft <sup>2</sup> (.186 m <sup>2</sup> ) of bridge deck area

**Table 7: Pedestrian Live Load**

#### 4.3.2 Vehicle Live Load

As mentioned above pedestrian bridges has to be designed for an occasional single maintenance vehicle. This applies regardless a vehicle allowance. Just in the case where the vehicle access is physical prevented the corresponding load can be neglected during the design. If the Operating Agency determines a specified vehicle configuration, this can be used for the design. In all other cases, the AASHTO

Standard H-Truck shall be used. Depending on the width of the walking area the following values has to be applied [1]:

Clear deck width from 6 ft to 10 ft: H-5 Truck = 10,000 lb (44.48 kN)

Clear deck width over 10 ft: H-10 Truck = 20,000 lb (88.96 kN)

The combination of pedestrian and vehicle load can be neglected. The considered Truck has to be placed to produce the maximum load effects.

#### 4.3.3 Wind Load

Regarding the considered guidelines, the wind loads are the only live loads, which has to be applied in the horizontal direction. The wind load has to be applied in a 90° angle to the longitudinal direction of the structure and “*shall be applied to the projected vertical area of all superstructure elements, including exposed truss members on the leeward truss.*” [1]. The following intensity should be used for the design [1]:

For Trusses and Arches: 75 psf (3.59 kPa)

For Girders and Beams: 50 psf (2.39 kPa)

#### **4.4 Design Details**

Besides the recommendations for design loads the guidelines provides also requirements for design details, like deflection limitations and instructions regarding vibrations, to assure the structural safety and serviceability.

#### 4.4.1 Deflection

The present guideline formulates limitation for deflections in relation to the corresponding span to assure users and observer a secure feeling and restrict the stresses in secondary construction members due to the movement. “*Members shall be designed so that the deflection due to the service pedestrian load does not exceed 1/500 of the length of the span.*” [1]. The same value applies for cantilever arms due to the pedestrian live load and for the horizontal deflection due to lateral wind load. These values are more liberal than the AASHTO highway bridge values (1/1000), recognizing the differences between vehicle and pedestrian loads. While the maximum load, which is applied for the calculation of the maximum deflection, is expected to appear frequently, the maximum loading due to pedestrians and the resulting deflection is expected to be exceptional. [1]

The limitation of maximal deflections correlates also with the vibration sensitivity of the structure. The structural stiffness, which is required to reach minimal deflections, ensures at the same time the fulfilment of the demanded vibration limitations. The reduction of the vertical deflection criterion for bridges out of traditional materials such as steel, concrete, wood, and aluminum, would cause a drop of the structural natural frequency, potentially below the threshold of 3 Hz, which represents the “*comfort level of pedestrians and runners*” [1]. Due to the reduced weight of FRP in comparison to traditional materials, one can satisfy the minimum vertical natural frequency criterion even with a more liberal deflection criterion. Nevertheless, due to the serviceability in terms of observable high deflections, the limitation of the maximal deflection applies unmodified to FRP pedestrian bridges.

#### 4.4.2 Vibrations

The requirements in terms of vibration restrictions are divided in vertical and horizontal directions. To avoid any issues regarding vibrations the AASHTO guidelines restrict the fundamental frequency of pedestrian bridges. *“To avoid any issues associated with the first and second harmonics”* [1] the fundamental frequency of pedestrian bridges should be higher than 5 Hz in the vertical direction. *“The range of the first through the third harmonic of people walking/running across pedestrian bridges is 2 Hz to 8 Hz, with the fundamental frequency being from 1.6 Hz to 2.4 Hz.”* [1]. Thus the fundamental frequency of traditional pedestrian bridges is restricted to values higher than 3 Hz.

In all other cases, like *“pedestrian bridges with low stiffness, damping, and mass, such as bridges with shallow depth, lightweight (such as FRP), etc., and in areas where running and jumping are expected to occur on the bridges, the design should be tuned to have a minimum fundamental frequency of 5 Hz (in the vertical direction) to avoid the second harmonic.”* [1]. In the horizontal direction the fundamental frequency of the pedestrian bridges should be higher than 3 Hz to avoid issues due lateral motion involving the first and second harmonics. Additionally, the aspect ratio (length/width), which also influences the lateral dynamic response of the construction, higher than 20 should be avoided. Finally, the fundamental frequencies in horizontal and vertical direction should be different *“to avoid potential adverse effects associated with the combined effects from the first and second harmonics in these directions”* [1].

If the aimed fundamental frequency cannot be reached by changes at structural level, for instance by changing stiffness, construction weight etc., additional effective

measures to reduce the vibrations are “*stiffening handrails, vibration absorbers, or dampers*” [1].



## CHAPTER 5

### MODEL DEVELOPMENT

#### 5.1 General Aspects

The purpose of this research is the estimation of the dynamical response behavior of FRP pedestrian bridges. As shown in the previous chapters the live load induced by a crowd of people, especially the dynamic part of it, is a complex phenomenon and hard to predict. The establishment and increasing use of advanced composite materials, namely fiber reinforced polymers, for civil engineering constructions and the related changes of dynamic properties and dynamic response behavior of these constructions, makes this topic to an important issue for the assurance of the safety and serviceability of newly build structures. In the corresponding design codes this topic is solved by giving general limitations for the natural frequency of structures.

This research is meant to analyze the dynamical response behavior of FRP pedestrian bridges. One major aspect of this analysis is the comparison to a traditional construction material. Due to its similarity regarding mechanical properties and general application, the used material is steel. The comparison is firstly a possibility to evaluate the results and estimate the specific material characteristics by determining the differences. Secondly, it verifies the model, since steel is a well-known material and the results might provide information about the quality of the used model. The final goal of the analysis is to check the accuracy of the recommendations given by the design code and, if necessary, to formulate an improved approach.

The current chapter presents the development of the test series and the model itself. It explains the general setting, presents the chosen parameters and their

respective calculations and points out limitations and restrictions regarding the test and the results.

## **5.2 Simplifications**

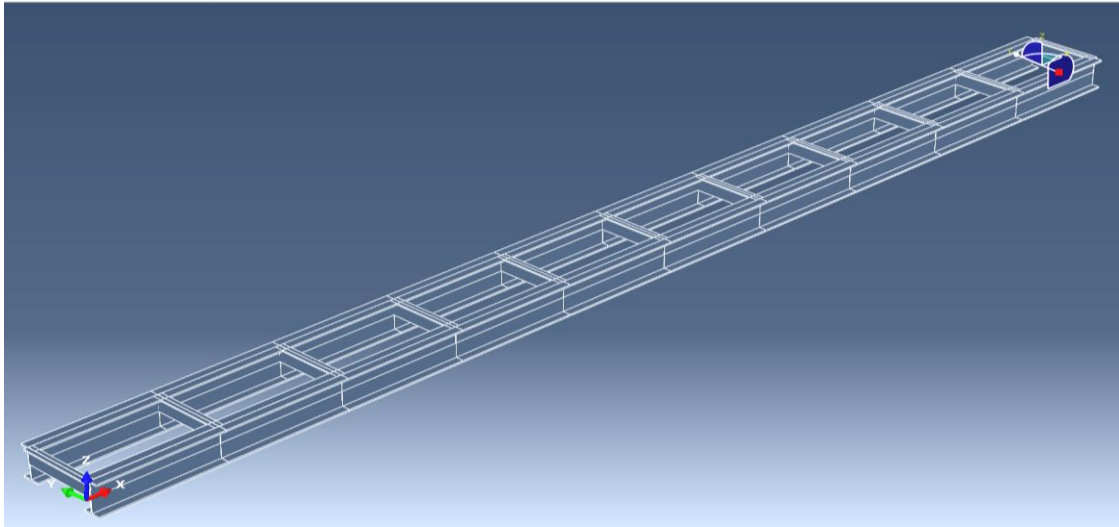
### **5.2.1 General System**

One part in the process of the model development was the simplification and generalization of the bridge structure. The aim was to find a model, which represents the characteristics of an average pedestrian bridge. Several parameters have to be chosen during the design of bridges, including the construction type, the number, profile and dimensions of the main girders, the cross-sectional beams and the deck and pavement design. Especially the first two points made a generalization of bridge properties complicated. Consequently, the final analysis has been made with an simplified girder system.

The common structural system of pedestrian bridges is a single span, traversed by two main girders, connected by secondary crossbeams, carrying the deck construction including pavement and handrails. Since the deck construction has a minor influence on the structural properties considered in this research and has, on the other hand, a wide range of variables in design and composition independent from the girder material, the analyzed system has been reduced to a girder system, consisting out of two main girders and the connecting crossbeams. In order to identify the dynamical properties of FRP pedestrian bridges this reduction to the main constructional members has been necessary to determine the particular specifications of this material without the influences of deck constructions and materials and other design components, which might have changed the results.

### 5.2.2 Load Application

As presented in chapter 3 the load induced by pedestrian crowds on a flexible structure is a complex phenomenon, which includes several interactions and countless variables. To model this phenomenon sufficiently a multi-physical and extensive model is necessary. The development of such a model falls outside the scope of this paper. The aim of the research is to identify the different components of the dynamic response and the evaluation of the guideline requirements. In order to do this, the load is applied separately for the different components and, in order to estimate the maximum values, a uniform distributed and an harmonic load is assumed. The latter one does not represent an average pedestrian loading, but it represents a maxima crowd load, with pedestrians walking in step, which represents the conditions of the ‘worst case’ in terms of a dynamic response.



**Figure 10: Construction Type (ABAQUS Model)**

### 5.3 Model parameters

#### 5.3.1 Construction Type

The modelled bridges are one span pedestrian bridges, with no camber, spanning in the range of 10.00 m to 25.00 m and are 2.00 m wide. The range of analyzed lengths corresponds to the normal length of average one span, beam bridges. The width is within the average width of pedestrian bridges, but comparatively small, in order to acknowledge the fact that small width to length ratios cause vibrational problems in lateral direction. The regarded constructions consist of two or three main I-shaped girders, connected with crossbeams in T-shape and half the height of the main girders.

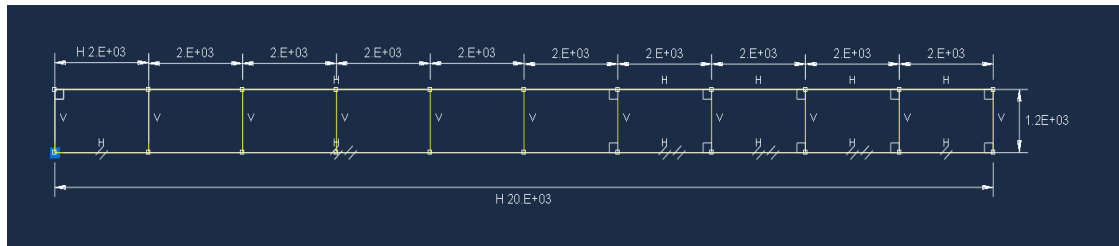


Figure 11: Model Construction - FRP 2/Steel (ABAQUS Model)

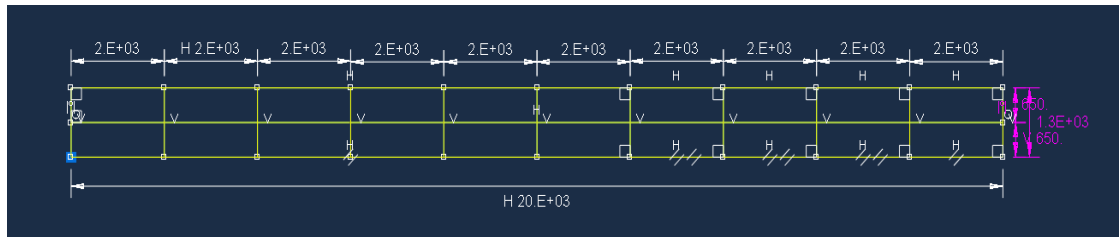


Figure 12: Model construction - FRP 1 (ABAQUS Model)

### 5.3.2 Materials

The bridge models are designed for the use of three different materials. As stated earlier, the traditional material, which is used, is steel, with average material properties. The selection of the FRP mechanical properties is challenging, due to the wide range of available properties. In order to investigate the effects of the single properties, two hypothetical fiber reinforce polymer materials with different stiffness, meaning different Young's moduli, are used in the simulation. The properties of the three materials are presented in the following table:

Material	Density [kN/m <sup>3</sup> ]	Young's modulus [MPa]	$\nu$
FRP 1	1.75	50000	0.2
FRP2	1.75	175000	0.2
Steel	7.85	200000	0.3

**Table 8: Material Properties**

### 5.3.3 Girder

#### 5.3.3.1 Main Girder

The dimensions of the main girders are based on the deflection limitations of the guidelines. Assuming the maximal pedestrian live load of 4.07 kN/m<sup>2</sup> and the maximal deflection of L/500 and based on the formula

$$w_{max} = \frac{q L^4}{384 EI} \quad [18]$$

to calculate the deflection, the necessary moment of inertia can be estimated as

$$I_{min} = \frac{q L^4}{384 E w_{max}} = \frac{q L^4}{384 E (L/500)} = \frac{500 q L^3}{384 E}.$$

To maintain the comparability between the materials as well as between the different spans the ratio of the moments of inertia in Y- and Z-direction is nearly

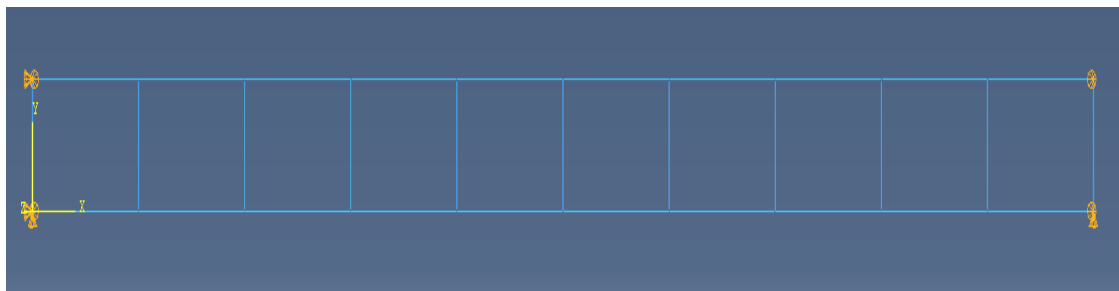
constant and the thickness of flanges and webs remains constant with increasing span within one material. Due to the low stiffness of the FRP 1 the proceeding leads to uneconomic big girders, what is why in case of this material three, instead of two, girders are used. The dimensions as well as the general properties of the used girders are given in the figures and tables A1, A2, A3 and A4 in the appendix.

### 5.3.3.2 Crossbeams

The crossbeams are chosen as T-Profiles of the half height of the main girders and are placed with a spacing of around 2.00 m. Their main task is the prevention of the torsion of the main girders and the distribution of unsymmetrical loads. In case of an actual construction, the dimensioning might be unsatisfactory, but it is chosen in terms of comparability. The dimensions of the used crossbeams are given in table A5 and A6.

### 5.3.3.3 Boundary Conditions

The abutments are placed beneath the ends of each main girder. The aim is to prevent movement in all of the three directions without producing any constraints. Vertical displacement is restricted at each abutment, two abutments provide support in lateral direction and two of the abutments prevent the longitudinal displacement. The following graphic presents the disposition of the boundary conditions.



**Figure 13: Boundary Conditions (ABAQUS Model)**

## **5.4 Analysis Steps**

### **5.4.1 General Aspects**

In order to determine the dynamic properties of the different materials and the relating bridge models the analysis includes several steps to determine the single effects and influences. The following paragraphs describe briefly the proceeding and the respective settings used in the ABAQUS simulation.

### **5.4.2 Step 1: Natural Frequency**

During the first step, the natural frequency of the modelled structure is estimated. In order to identify the influences of the material properties on the fundamental frequency of the structure no further loads or preconditions are applied. To see the development of the frequencies and modes this analysis step is made for all spans and materials and includes the first seven modes of the structures. It is expected that the development follow the general laws of structural dynamics. This step provides also a first classification in terms of the range of fundamental frequencies and therefore a base for the following steps.

### **5.4.3 Step 2: Additional Mass**

As mentioned in the chapter 3, the additional mass of the pedestrians changes the properties of the single degree of freedom system and therefore, the fundamental frequency of the structure. This effect is analyzed by a stepwise-applied mass and the calculation of the respective fundamental frequencies. The applied mass represents a load, which ranges between 0 kN/m<sup>2</sup> and 4 kN/m<sup>2</sup>. The highest values equals the

maximal pedestrian load. The load is assumed uniformly and an even distribution over the two or three girders is considered.

#### 5.4.4 Step 3: Dynamic Response (harmonic loading)

To test for the dynamic response to the pedestrian loading the steady state of the structure is estimated, over a wide range of frequencies and with a stepwise applied dynamic load. The initial condition of this step is a maximal loaded ( $4.0 \text{ kN/m}^2$ ) structure, to acknowledge the changed fundamental frequency, and stepwise a dynamic load, considering the impact factor related to different walking frequencies, is applied. In vertical direction, the first applied dynamic load is  $0.8 \text{ kN/m}^2$ , which is equivalent to an impact factor of 1.2, which correlates to walking frequencies of 1.6 Hz to 2.0 Hz and the static load of  $4.0 \text{ kN/m}^2$ . The next load steps are  $1.6 \text{ kN/m}^2$ ,  $2.4 \text{ kN/m}^2$  and  $3.2 \text{ kN/m}^2$  that acknowledges the increasing impact factor with increasing walking frequency.

The tested frequencies embrace the range of 1 Hz to 5 Hz to include walking frequencies from slow walking up to running. Since the applied load is harmonic and uniform, it represents a crowd marching in step, which is not realistic, but produces maximal response values. In horizontal direction, which is analyzed separated from the vertical direction, the load steps are  $0.2 \text{ kN/m}^2$ ,  $0.16 \text{ kN/m}^2$ ,  $0.12 \text{ kN/m}^2$ ,  $0.08 \text{ kN/m}^2$  and  $0.04 \text{ kN/m}^2$ .



## CHAPTER 6

### FINDINGS AND RESULTS

#### 6.1 Introduction

Following the analysis steps presented in the previous chapter, the results and the respective explanations and conclusions are presented. In the first part, the results are presented systematically and are related to the basic principles of structural dynamics. The focus is on the general characteristics of the dynamical behavior of the constructions, their general dependencies and the differences between the different materials. The second part includes an evaluation of the guidelines based on the conclusions of the first part. The third part presents a new approach for the guidelines in contribution to the findings of part two.

#### 6.2 Results

##### 6.2.1 Fundamental Frequencies

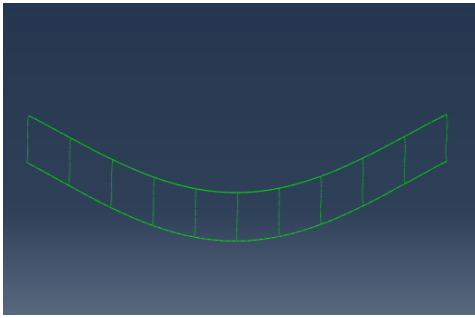
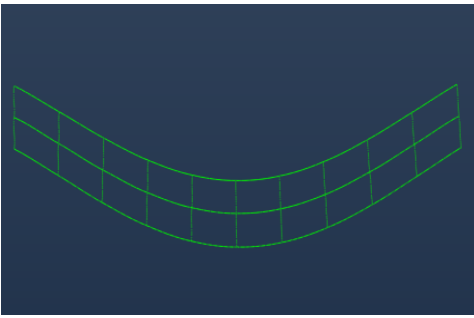
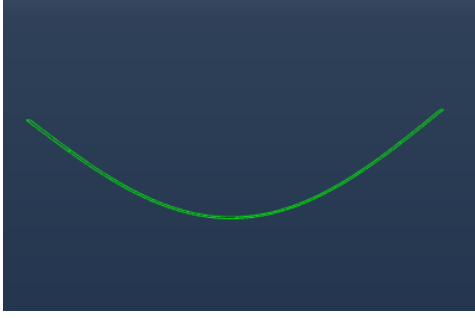
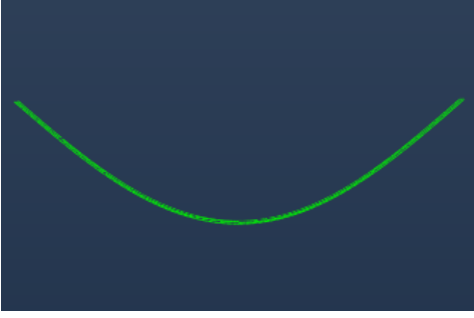
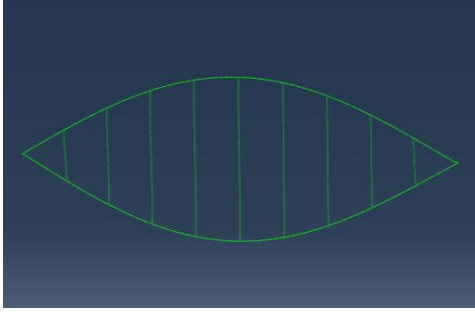
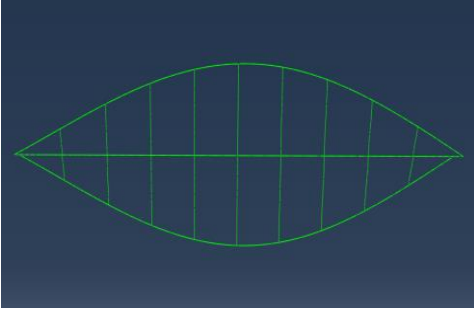
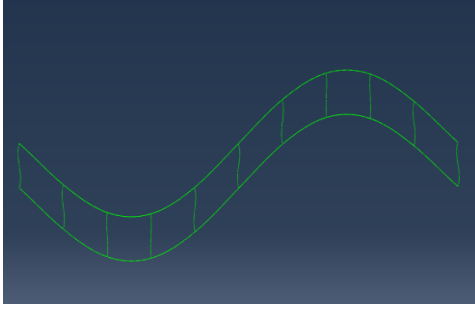
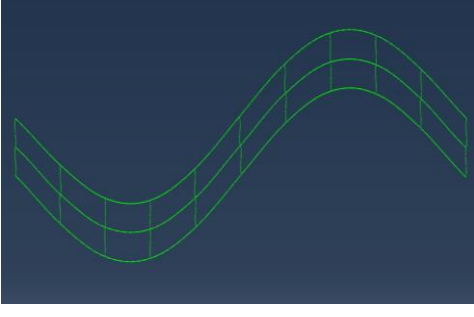
###### 6.2.1.1 General Aspects

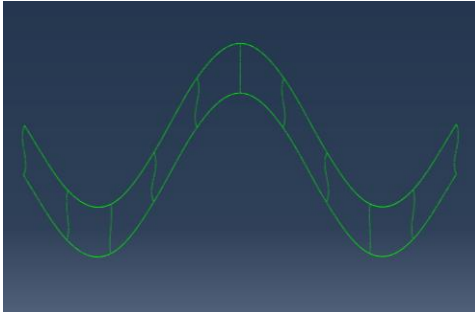
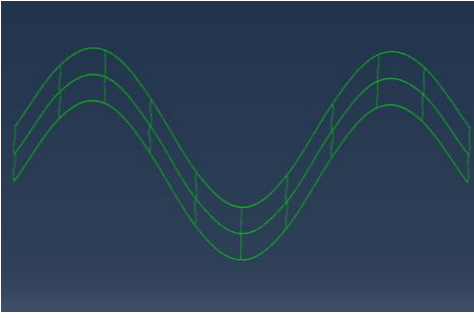
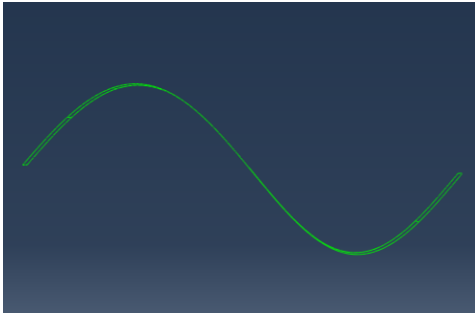
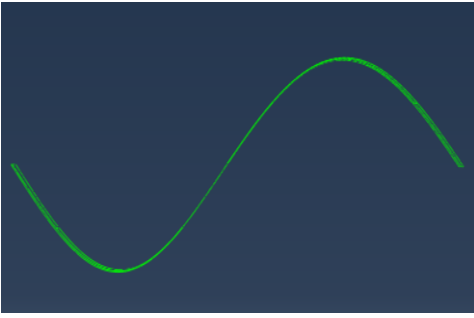
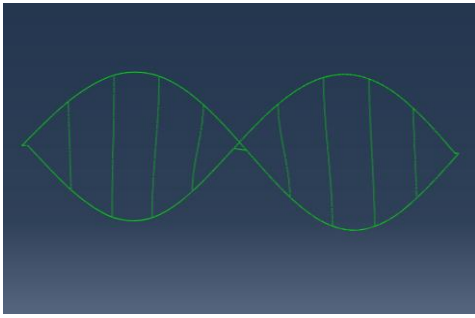
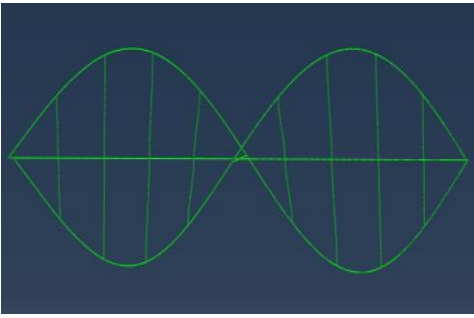
Considering a single-degree-of-freedom (SDOF) system, meaning a system with a single displacement variable, the rate at which the system chooses to oscillate in this direction is called natural or fundamental frequency. The natural frequency is governed by the mass and stiffness of the system. In terms of an undamped system the relation can be described as

$$\omega_n = \sqrt{\frac{k}{m}}$$

where  $k$  is the stiffness and  $m$  is the mass of the system. A bridge structure is much more complicated, it has several motion variables and therefore, it has to be approximated as a multi-degree-of-freedom (MDOF) system. In the case of MDOF systems, each degree of freedom is related to its own natural frequency. Each of these modes of vibration is associated with a particular deformation shape known as the mode shape. Due to the high complicity of MDOF systems the calculation of the natural frequencies is more complicated. The circular natural frequencies of an MDOF system are the square roots of the eigenvalues of  $m^{-1} \cdot k$ ; the general dependencies are equal to the SDOF equation.

During the research, the natural frequencies of the model constructions are estimated. The first seven modes are considered. In order to evaluate the dynamical response under human induced excitation the natural frequencies in vertical and lateral direction are decisive. The first seven modes of the constructions include the first three modes in lateral direction, the first two modes in vertical direction and the first two modes of a torsional motion. The order of the seven modes is equal over all span lengths and for all materials. Just the second vertical mode and the second torsional mode have similar values regarding their respective natural frequencies and represent the sixth and seventh mode of the structure in turns. Illustrative for all considered structures the mode shapes of the FRP 1 and FRP 2 bridges with 20.00 m span are presented in the following table.

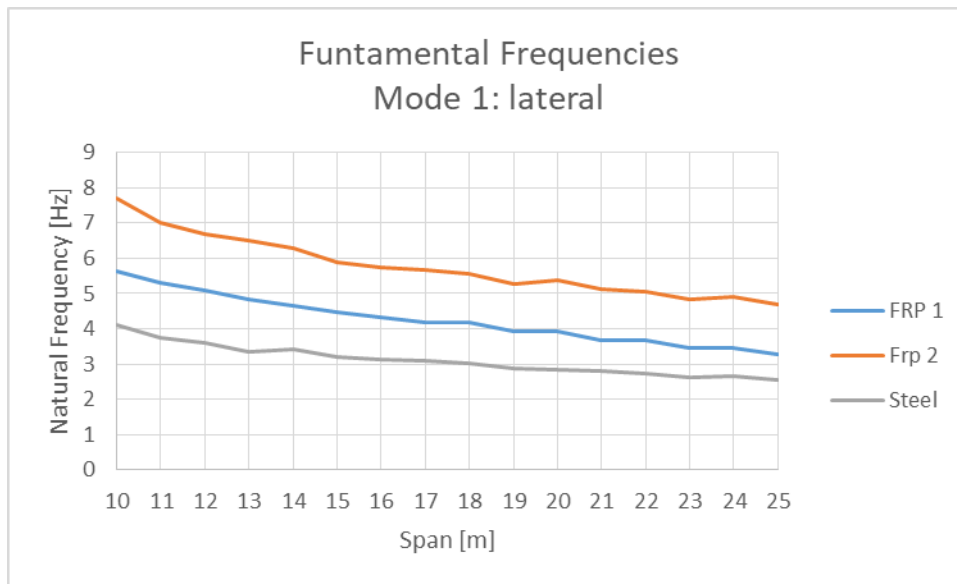
Mode	FRP 2/Steel	FRP1
1	 <p data-bbox="485 653 833 680">Figure 14: Mode 1 - FRP 2/Steel</p>	 <p data-bbox="1021 653 1304 680">Figure 15: Mode 1 - FRP 1</p>
2	 <p data-bbox="485 1056 833 1083">Figure 16: Mode 2 - FRP 2/Steel</p>	 <p data-bbox="1021 1056 1304 1083">Figure 17: Mode 2 - FRP 1</p>
3	 <p data-bbox="485 1459 833 1486">Figure 18: Mode 3 - FRP 2/Steel</p>	 <p data-bbox="1021 1459 1304 1486">Figure 19: Mode 3 - FRP 1</p>
4	 <p data-bbox="485 1862 833 1890">Figure 20: Mode 4 - FRP 2/Steel</p>	 <p data-bbox="1021 1862 1304 1890">Figure 21: Mode 4 - FRP 1</p>

Mode	FRP 2/Steel	FRP1
5	 <p data-bbox="483 615 834 646"><b>Figure 22: Mode 5 - FRP 2/Steel</b></p>	 <p data-bbox="1019 615 1305 646"><b>Figure 23: Mode 5 - FRP 1</b></p>
6	 <p data-bbox="483 1014 834 1045"><b>Figure 24: Mode 6 - FRP 2/Steel</b></p>	 <p data-bbox="1019 1014 1305 1045"><b>Figure 25: Mode 6 - FRP 1</b></p>
7	 <p data-bbox="483 1413 834 1444"><b>Figure 26: Mode 7 - FRP 2/Steel</b></p>	 <p data-bbox="1019 1413 1305 1444"><b>Figure 27: Mode 7 - FRP 1</b></p>

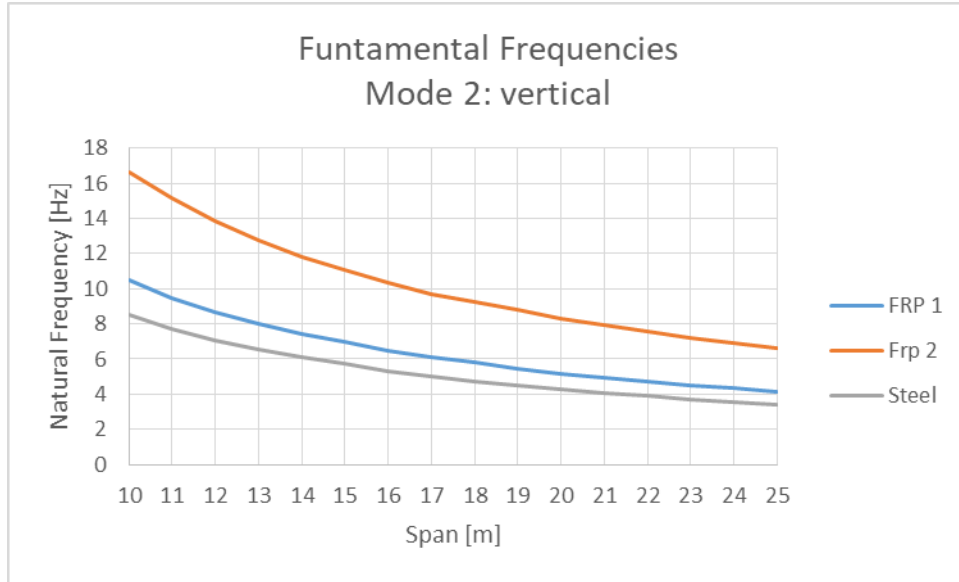
**Table 9: Mode Shapes (FRP 1, FRP 2/Steel)**

### 6.2.1.2 Unloaded system (Analysis-step 1)

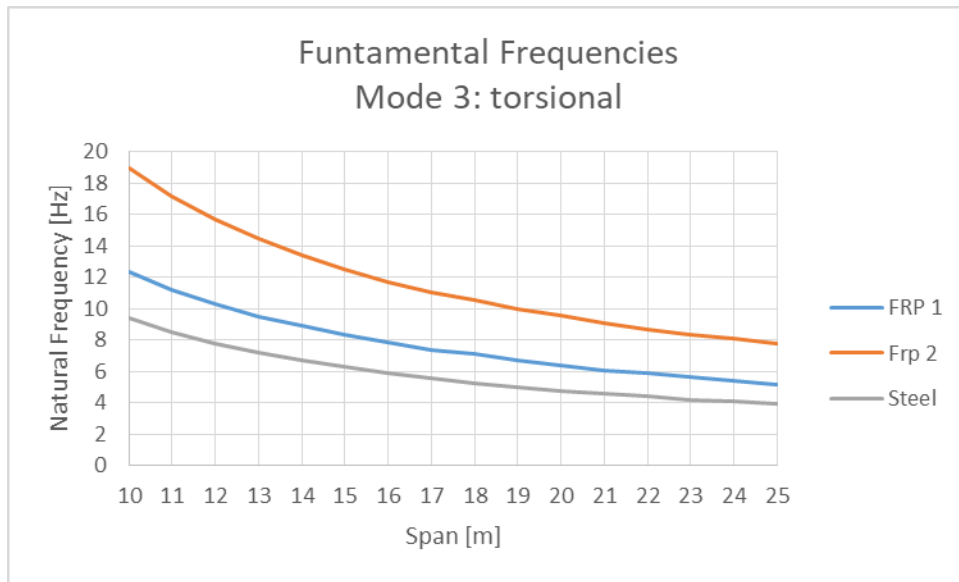
The comparison of the natural frequencies of the unloaded systems gives a first overview of the dynamical properties of the different materials. The table A5 with the detailed data is included in the appendix. The following graphs present the development, with increasing span, of the natural frequencies of the first lateral, vertical and torsional mode, respectively, which represents the first three modes of the structure. The curves for the three materials are printed in the same graph, in order to simplify the comparison.



**Figure 28: Fundamental Frequencies - Mode 1 - lateral**



**Figure 29: Fundamental Frequencies - Mode 2 - vertical**



**Figure 30: Fundamental Frequencies - Mode 3 - torsional**

The natural frequencies of the structures depends mainly on the parameters mass and stiffness. The mass consists of the construction weight and depends on the mass distribution. The stiffness of the applied system consists of the mechanical material properties, the characteristics of the profile, type and span of the structural system. The effects of three of these parameters can be seen in these graphs.

The first considerable parameter is the span. The curves of all three materials have similar shapes, because the dependency of the natural frequency to span is material independent. The dependency is in general complex, since the span influences more than one included parameter. The stiffness of a dynamical system can be calculated as

$$k = \alpha \cdot \frac{EI}{L^3} \quad [18]$$

Where  $\alpha$  is a factor depending on the statically system. Thus, the natural frequency is dropping with increasing span. Since the moment of inertia  $I$  is also estimated based on the span length  $L$  (see chapter 5.3.3.1 Main Girder), the interdependency between the stiffness and the length  $L$  is even more complicated. Additionally the span also dictates the mass, since the construction weight is closely related to the span, because of both, the span and the resulting profile dimensions. The continuity with which the curves of all materials develop shows that the proceeding to model comparable systems has been sufficient. Minor discontinuities can be explained by the stepwise increase of the moment of inertia and the changes in the cross beam spacing.

The next considerable parameter, which is in order to estimate the material depending dynamical properties of greater importance, is the stiffness of the material,

namely the Young's modulus  $E$ . The equation above shows, the stiffness of the structure is proportional to the Young's modulus. However, it has to be considered that the moment of inertia depends also on the material properties, so the actual correlation is not linear. An increase of the structure's stiffness leads to higher natural frequencies. This can be easily observed in the direct comparison of the two FRP materials, whose properties differ just in terms of stiffness. This aspect requires consideration during the design process of FRP constructions, because advanced composite materials vary strongly regarding their mechanical properties and a reduced natural frequency can cause resonance related problems.

Although the FRP materials have significant smaller Young's moduli than steel, the model structures built out of these materials have still higher natural frequencies than the respective ones out of steel. This circumstance is caused by the third considerable parameter, the mass. The assumed materials have significantly differing densities, which is decisive for the mass to be applied. The actual mass of the dynamical system is the product of span, profile area and density. The span is for all material the same, profile area and density are material depended. FRP 2 and steel have similar stiffnesses and have therefore similar profiles. In conclusion, their main difference is the density. As it can be seen clearly, this property has a major influence on the structure's natural frequency. A reduced density causes an increased natural frequency, which can be seen as a major advantage of advanced composite materials, because it counteracts the effects of the reduced stiffness. Since the factor between the materials' densities is higher than the one of the stiffness, the influence of the density



is higher, which is the explanation, why the natural frequencies of the FRP materials are higher than the one of steel.

One additional aspect can be seen in these curves regarding the construction form itself. The constructions out of FRP 1 are designed with three instead of two girders. This has not a big influence in vertical direction, because the structure's stiffness is the sum of the individual girders. In horizontal direction, the impact of this change of construction is much higher. Due to the reduced spacing and the reduced length of the crossbeams, the connection between the individual girders increases which leads to an improved stiffness in this direction. The respective position of the corresponding curve is therefore higher in the lateral direction than in the vertical.

The development and relation to each other of the different modes respectively for the different materials are presented in the following graphs. The significant similarity proves the comparability of the used models, which is important for the following analysis steps.

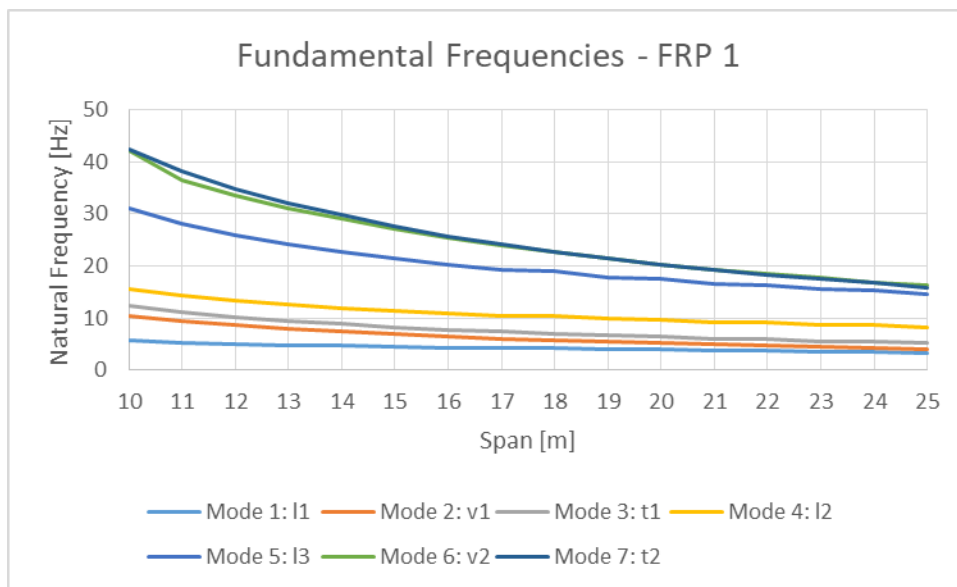


Figure 31: Fundamental Frequencies - FRP 1

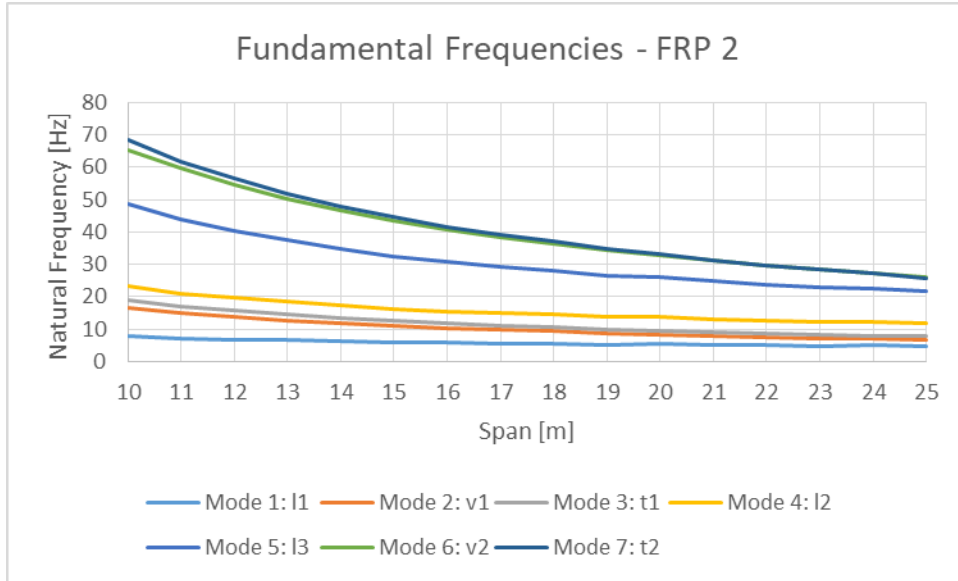


Figure 32: Fundamental Frequencies - FRP 2

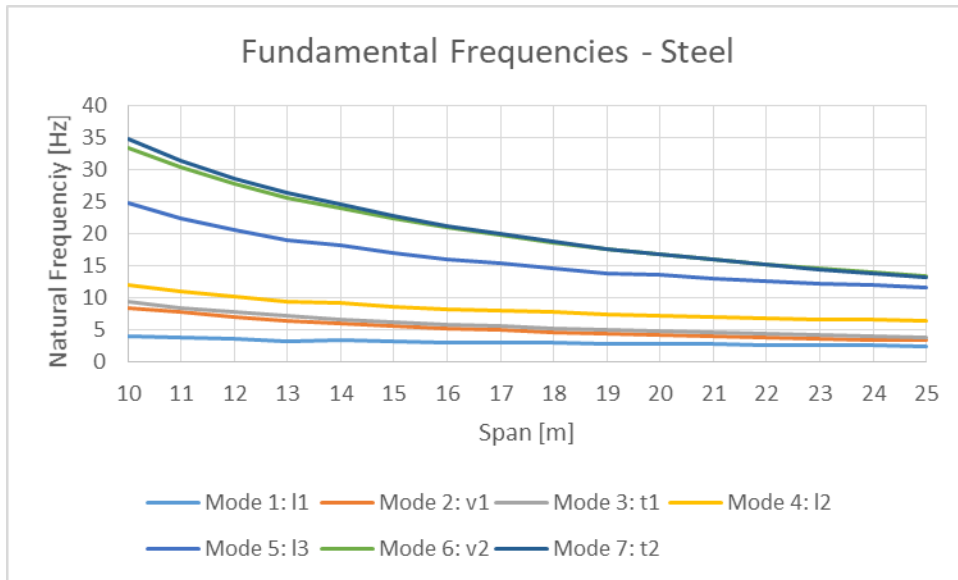
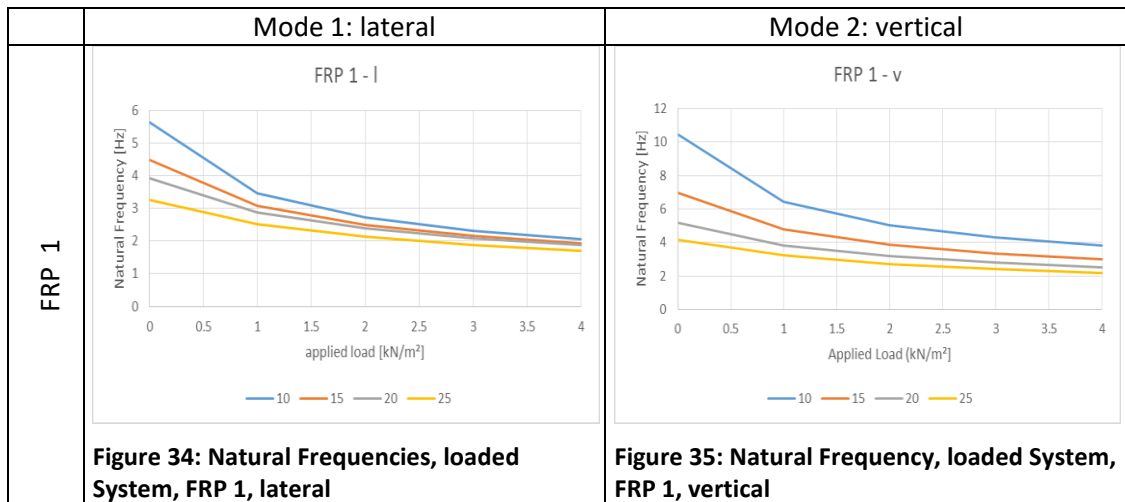


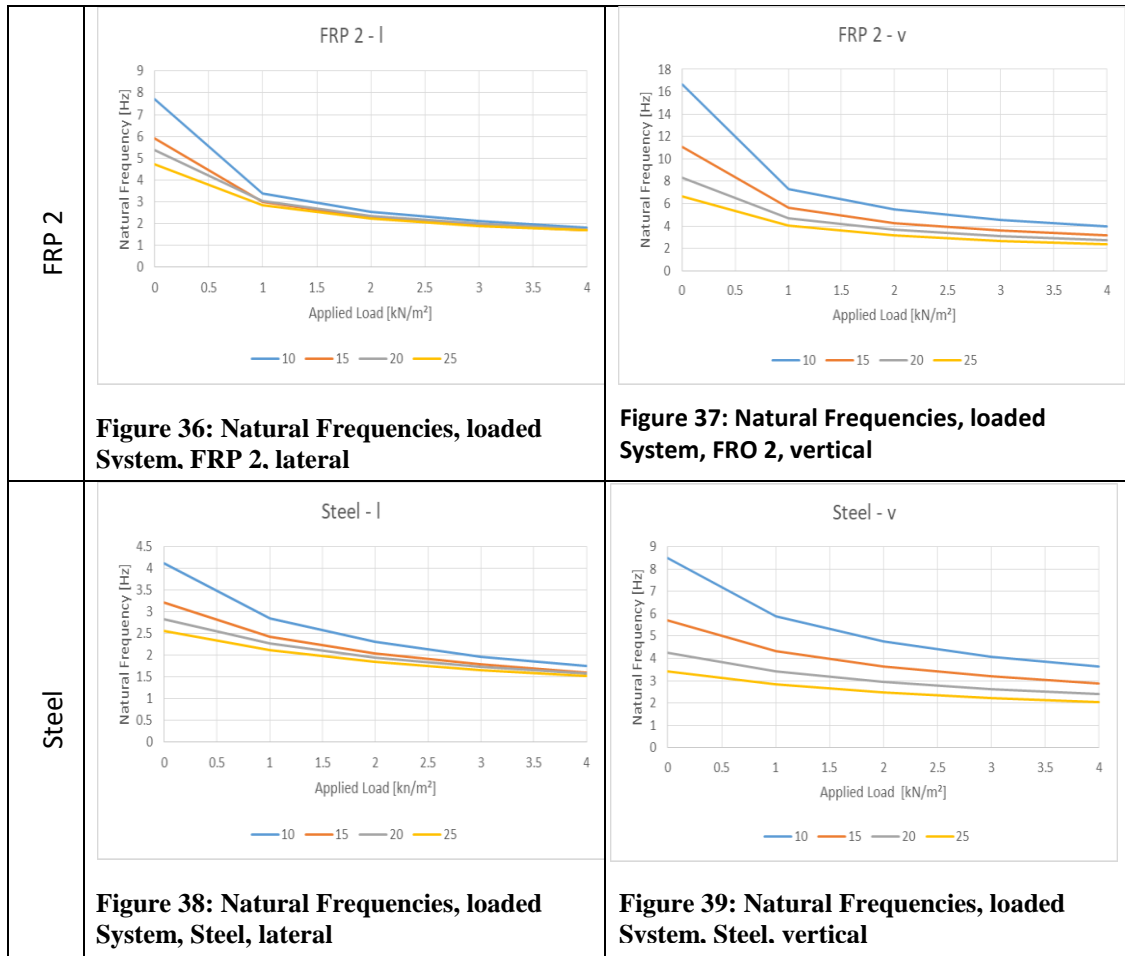
Figure 33: Fundamental Frequencies - Steel

### 6.2.1.3 Loaded Systems (Analysis-step 2)

Even though the mechanical properties of fiber reinforced polymer lead to increased natural frequencies, which is positive in terms of the resistance against human induced excitations, FRP constructions are still more sensitive to these loads, due to high live load to construction weight ratios.

Unlike wind or earthquake live loads, pedestrian live loads add an additional mass to the dynamical system. When pedestrians enter a bridge construction, their weight is added to the oscillating mass; they become a part of the system. This changes in turn the dynamical properties of this system. As it is shown above, the fundamental frequency of a dynamical system is directly correlated to its mass. A change of mass, in this case an increase, leads to a drop of the frequency. In order to analyze this effect, the modelled construction are stepwise loaded with a uniform load, up to the maximal pedestrian load of 4.00 kN/m<sup>2</sup>. The natural frequencies of the changed systems are calculated and presented in the following graphs. Shown are exemplary the graphs for the spans 10.00 m, 15.00 m, 20.00 m and 25.00 m. The complete data are available in the appendix.

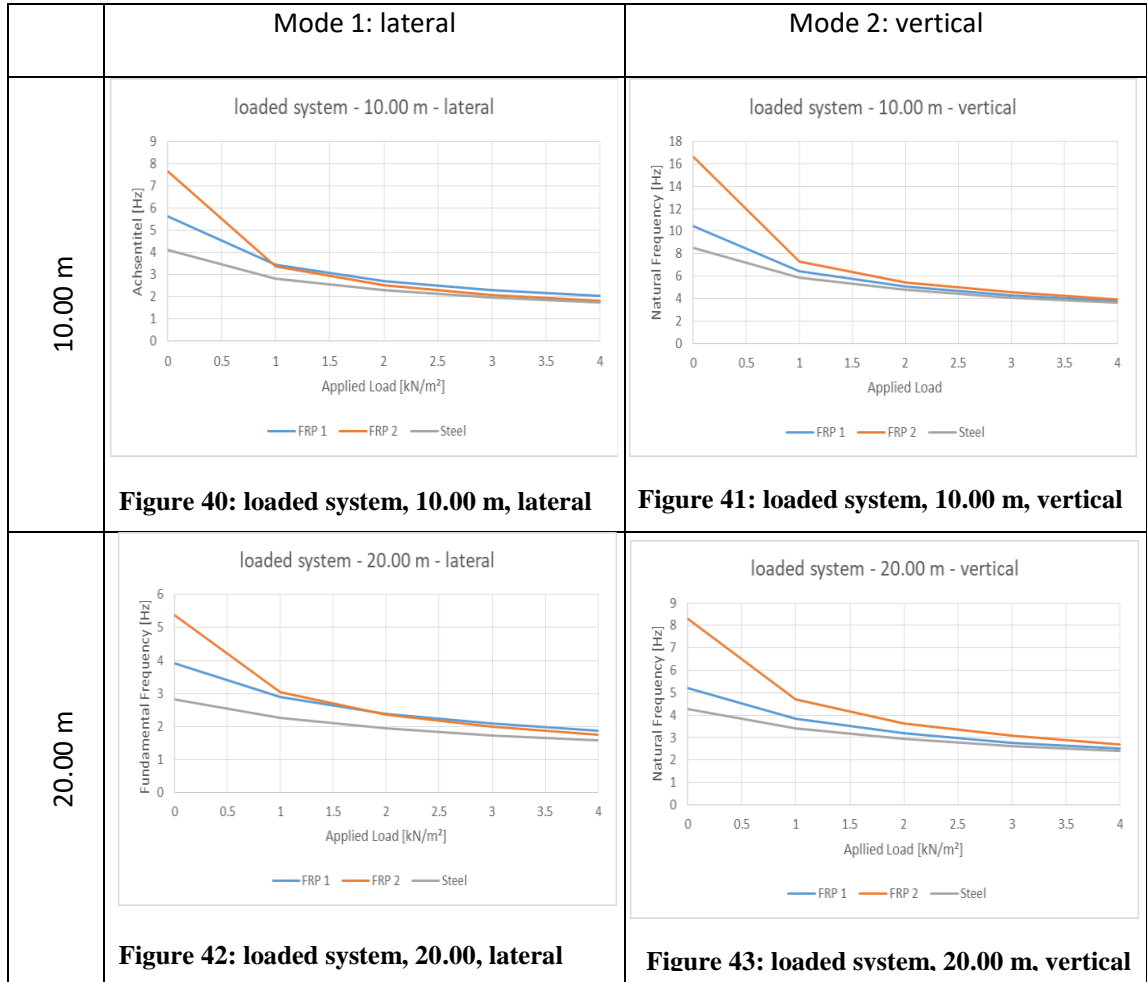




**Table 10: Fundamental Frequencies - loaded system**

As shown in the graphs, the fundamental frequency of the structure drops significantly due to the additional load. The maximum load of 4 kN/m<sup>2</sup> reduces the frequency of the structure in all materials by a factor of 2, or in the case of FRP 2 by a factor of 4. Even though the actual values are not representative for actual bridge constructions, because the ratio of construction weight to load and therefore the change of mass would be smaller due to the neglected deck and additional construction elements, but the influence of this effect is still decisive and cannot be neglected. Due to the small density of FRP, the ratio of construction weight to applied load is much higher, which causes the respectively great change of the frequencies.

The influence on the FRP 1 material is smaller than the one on FRP 2, because the additional girder used in the first case increases the construction weight and therefore decreases the ratio of construction weight to live load. The relationship between the different materials can be seen in the following graphs.



## 6.2.2 Dynamical Response

### 6.2.2.1 General Aspects

The response of a structure to dynamical loads can be divided into two parts – the transient response and the steady state response.

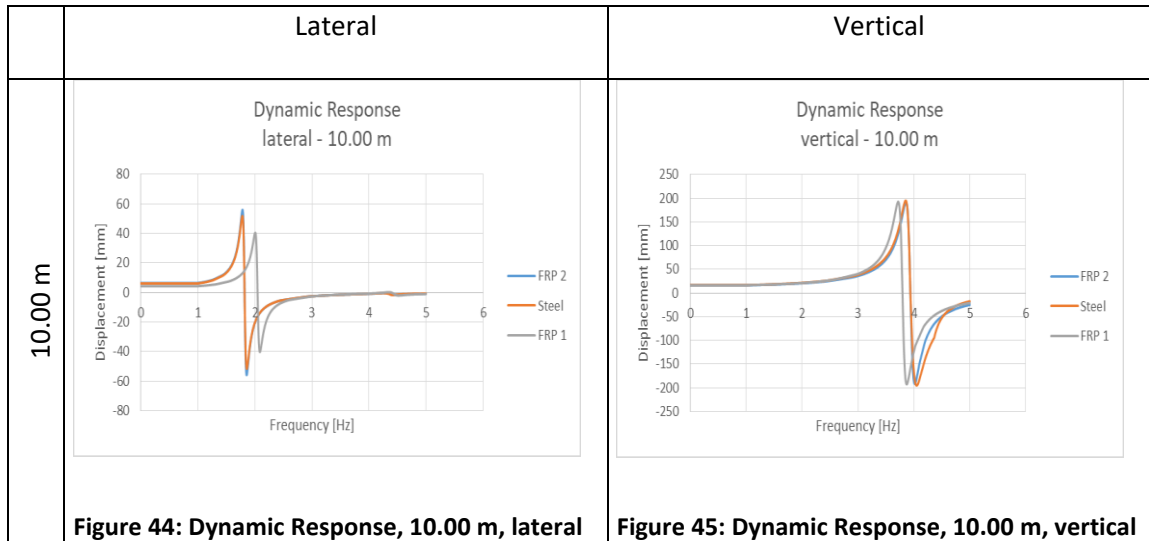
*“The transient response is a vibration at the natural frequency of the structure. It can be thought of as a free vibration initiated by the onset of the applied load, which disturbs the structure from its equilibrium position. It is called transient because the damping causes it to die away quite quickly. In relatively short-duration events, the transient response can be a significant part of the total, but it is often neglected when considering long-duration loads. The nature of the steady state response will vary with that of the applied loading, and will continue for as long as the loading.” [56]*

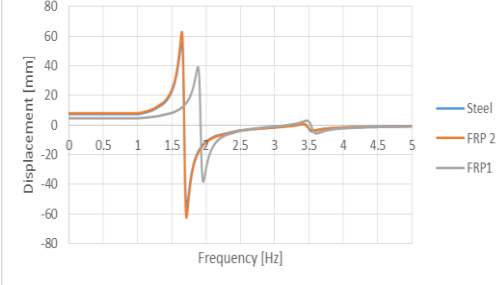
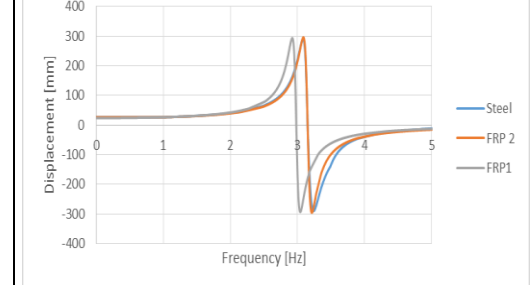
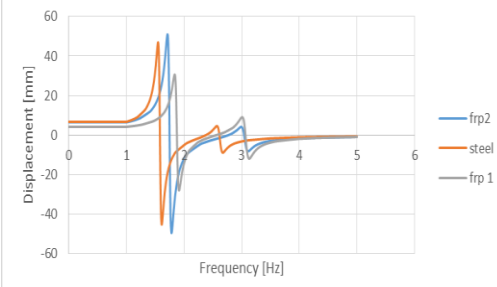
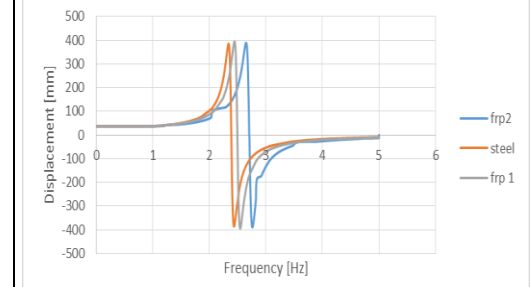
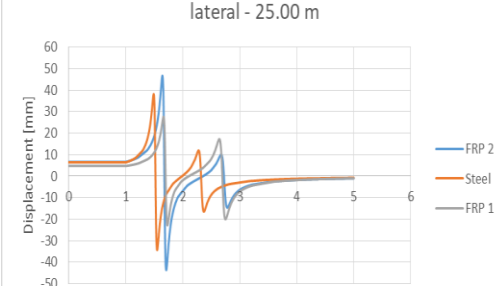
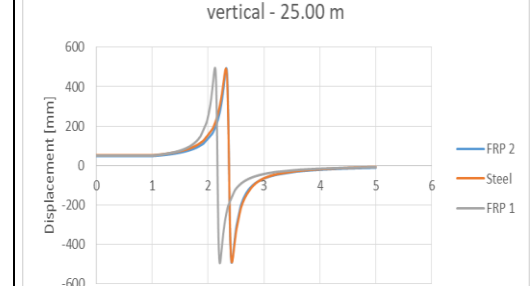
Pedestrian loading can be classified as long duration load. The transient response can be neglected, at least in terms of a general analysis. The greatest hazard due to a dynamical load is an effect called resonance. If the exciting frequency of the harmonic loading is close to natural frequency of the respective structure, the amplification factor of the equivalent static load is growing significantly. The natural frequency of a structure and its relation to the loading frequency is therefore of decisive importance in the design process. The difficulty of pedestrian loading is the high variability. The load applied by a crowd of pedestrians is normally not harmonic, due to the individual parameters of the pedestrians. In the case of randomly distributed walking parameters and phase angles, the dynamic part of the load does not have a great impact, because the minima and maxima of the individual pedestrian loads compensate each other. However, in the case of synchronously walking, intentional (marching) or

unintentional (lateral synchronisation), the load amplitudes accumulate and the resulting load is near a harmonic load. This scenario causes the greatest deformations and therefore, the analysis deals with harmonic, uniformly distributed loads, as an approximation of a uniformly distributed crowd walking in step. In this way, the “worst case” is presented.

### 6.2.2.2 Dynamic Response (Analysis-step 3)

The steady state analyses of the spans 10.00 m, 15.00 m, 20.00 m and 25.00 m are representative presented in the following figures. As the maximum range of human walking, the frequencies from 1.0 Hz to 5.0 Hz are presented. The detailed data is available in the table A10 in the appendix.



15.00 m	<p style="text-align: center;">Dynamic Response lateral - 15.00 m</p> 	<p style="text-align: center;">Dynamic Response vertical - 15.00 m</p> 
20.00 m	<p style="text-align: center;">Dynamic Response lateral - 20.00 m</p> 	<p style="text-align: center;">Dynamic Response vertical - 20.00 m</p> 
25.00 m	<p style="text-align: center;">Dynamic Response lateral - 25.00 m</p> 	<p style="text-align: center;">Dynamic Response vertical - 25.00 m</p> 

**Table 11: Dynamic Response, lateral and vertical, 10.00 m, 15.00 m, 20.00 m, 25.00 m**

As expected the maximum values appear at the frequencies equal to the fundamental frequencies of the structure. It can be seen, that the natural frequency of the loaded FRP 2 structure dropped to the same value as the one of the steel structure,



even though the natural frequency of the unloaded system is much higher. The importance of this fact can be seen in this graphs. Since the girder profiles in vertical direction are dimensioned for equal displacements, the similar maximum values are also as expected. The differences of the FRP 1 material can be justified by the changed stiffness parameters as a consequence of the additional girder.

### **6.3 Guideline Evaluation**

The results and conclusions of the previous paragraphs enables to identify some insufficiencies in the current guidelines, regarding the handling of human induced excitations on FRP pedestrian bridges. As it is shown above, the effects of the material characteristics of advanced composite materials on the dynamic properties of respective structures equalize each other. Since the reduced density leads to higher frequencies, it compensates the drop of the frequency due to the smaller stiffness of FRP materials. The final natural frequency depends on the ratio of stiffness to density. The fundamental frequencies of FRP bridge constructions, at least of the applied models, are in the same range as respective steel structures, or even higher.

The guidelines require for FRP constructions, as well as for steel structures, fundamental frequencies higher than 5 Hz and 3 Hz for the vertical and lateral direction, respectively. This limitation is meant to avoid great displacements due to a resonance response. It applies to the unloaded structure, which, regarding the results of this research, seems to be inefficient. The additional mass induced by a crowd of pedestrians can causes a drop of the natural frequency by a factor of 2 to 4. Consequently, even an apparently safe structure with a natural frequency higher than 5

Hz, could drop into the critical frequency range close to the walking frequency, due to the additional mass applied by pedestrians.

The guideline is in the case of steel structures well established, what might imply that in the threshold of 5 Hz a sufficient amount of redundancies is included. In the case of FRP structures, the impact of the additionally applied load is much bigger, due to the reduced construction weight and the resulting high live load to construction weight ratio. The neglecting of this effect might be part of the problem, which led to the vibration related serviceability problems in the recent past, and a revision of the respective guidelines should be considered.

The lateral component of the walking load is small, comparatively to the vertical component, and, due to the connection between the girders and the additional construction elements, the stiffness in lateral direction is often higher. Nevertheless, the main part of the recent vibrational problems appeared in lateral direction. The problem refers to bridges with great spans, which is why the present research does not present considerable results – the major deformations appear in the vertical direction. However, it can still be recognized, that, based on the recent past, the elision of lateral, pedestrian live loads cannot be justified.

## CHAPTER 7

### CONCLUSION

#### 7.1 Summary

In order to keep guidelines efficient and practicable, they have to be adjusted continuously to recent trends and developments. It has to be an ongoing process, in which the current requirements are evaluated and verified or improved. This applies especially to the field of civil engineering, because these constructions involve high investments and include a high hazard in the case of failure. This research points out that the current guideline regarding FRP pedestrian bridges under human induced excitation requires such an adjustment.

This circumstance has been indicated by several oscillation problems all over the world and has been confirmed by the estimated data. Depending on the density to stiffness ratio of the used material, the dynamical properties of FRP bridges are similar to the ones of steel structures – the natural frequency is in the same range. Nevertheless, FRP pedestrian bridges are more sensitive to dynamical pedestrian loading, due to the high impact of additional load on the natural frequency. The additional mass of the pedestrians changes the dynamical properties of the construction and the natural frequency drops in the range of human walking, what might lead to high deformations due to resonance.

The current guideline limits the natural frequency of the unloaded system and neglects the human-structure interaction. It also does not cover all aspects of the complex pedestrian loading, particularly the lateral component of the load, which

leads to the most vibrational issues in the recent past, is not included in the guideline's requirements. Therefore, an adjustment of the guidelines is suggested.

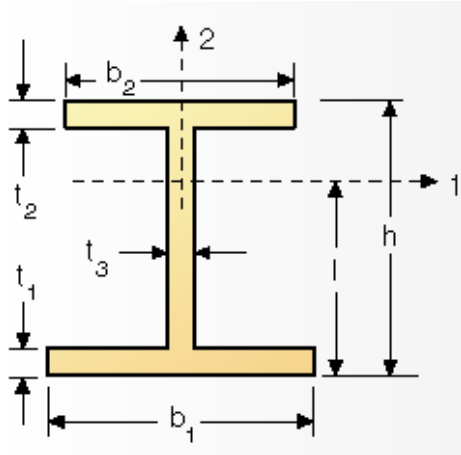
## **7.2 Future Work**

The phenomenon of a pedestrian crowd walking on a bridge structure is extremely complex, hard to simulate sufficient and still not completely understood. However, the reliable prediction of the dynamic response of bridges and pedestrian bridges in particular, is of great importance in order to realize economic and efficient structures without serviceability problems. The presented research works with simplifications and generalizations in order to understand the influence of the different parameters. A conservative estimation of the maximal deformations lead to safe but uneconomic constructions. The used load cases represent a theoretical scenario and the resulting deformations represent a maximal threshold.

Further researches regarding this topic, and especially the estimation of more realistic load cases and their probability distribution, are necessary to develop a reliable and economic guideline. Additionally, the interaction between humans and bridge structures are still not completely understood and could be the topic of several additional researches. This is of particular interest, since the dynamic response of pedestrian bridges due to human induced excitation is often the governing factor in the design of FRP bridges.

## APPENDICES

### A1: Profile Main Girders



### A2: Girder FRP 1 [mm]

L	h	b	tf	tw	A	ly	lz
10	350	175	35	35	22050	369153750	32513542
11	380	190	35	35	24150	484006250	41368542
12	410	205	35	35	26250	620593750	51719792
13	440	220	35	35	28350	780806250	63685417
14	475	238	35	35	30800	1000101667	79843294
15	505	253	35	35	32900	1217985417	95711966
16	535	268	35	35	35000	1465479167	113569076
17	565	283	35	35	37100	1744472917	133532747
18	600	300	35	35	39550	2112299583	159643750
19	630	315	35	35	41650	2466027083	184576875
20	660	330	35	35	43750	2857239583	211990625
21	690	345	35	35	45850	3287827083	242003125
22	725	363	35	35	48300	3842466250	280459831
23	755	378	35	35	50400	4364850000	316508190
24	785	390	35	35	52325	4907966510	348832240
25	815	408	35	35	54600	5547587500	397641471

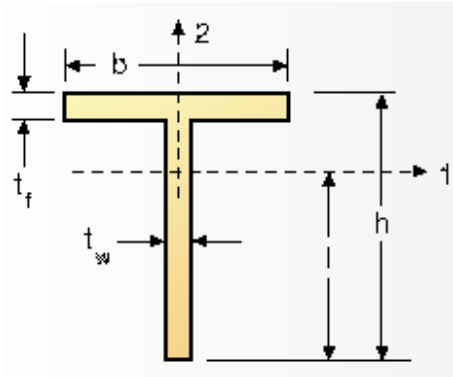
A3: Girder FRP 2 [mm]

L	h	b	tf	tw	A	I <sub>y</sub>	I <sub>z</sub>
10	280	140	25	15	10450	129367083.3	11512083.33
11	305	152.5	25	15	11450	170573854.2	14863190.1
12	330	165	25	15	12450	219733750	18810000
13	355	177.5	25	15	13450	277549895.8	23401341.15
14	385	192.5	25	15	14650	359345520.8	29830481.77
15	410	205	25	15	15650	438680416.7	36011666.67
16	435	217.5	25	15	16650	528921562.5	42993632.81
17	460	230	25	15	17650	630772083.3	50825208.33
18	485	242.5	25	15	18650	744935104.2	59555221.35
19	510	255	25	15	19650	872113750	69232500
20	535	276.5	25	15	21100	1042295833	88229831.77
21	565	282.5	25	15	21850	1201186771	94097513.02
22	590	295	25	15	22850	1374740417	107134166.7
23	615	307.5	25	15	23850	1564262813	121323164.1
24	640	320	25	15	24850	1770457083	136713333.3
25	665	332.5	25	15	25850	1994026354	153353502.6

A4: Girder Steel [mm]

L	h	b	tf	tw	A	I <sub>y</sub>	I <sub>z</sub>
10	285	143	30	20	13050	158616563	14658203
11	310	155	30	20	14300	209019167	18826042
12	335	168	30	20	15550	269140521	23720443
13	360	180	30	20	16800	339840000	29400000
14	385	193	30	20	18050	421976979	35923307
15	410	205	30	20	19300	516410833	43348958
16	435	218	30	20	20550	624000938	51735547
17	460	230	30	20	21800	745606667	61141667
18	490	245	30	20	23300	911244167	73857292
19	515	258	30	20	24550	1066709271	85712630
20	540	270	30	20	25800	1238940000	98775000
21	565	283	30	20	27050	1428795729	113102995
22	590	295	30	20	28300	1637135833	128755208
23	615	308	30	20	29550	1864819688	145790234
24	640	320	30	20	30800	2112706667	164266667
25	665	333	30	20	32050	2381656146	184243099

A5: Profile Crossbeam



A6: Crossbeams [mm]

L	FRP 1			FRP2			Steel		
	h	b	t <sub>f</sub>	h	b	t <sub>w</sub>	h	b	t <sub>f</sub>
10	175	88	25	143	71	25	140	70	25
11	185	93	25	155	78	25	153	76	25
12	208	104	25	168	84	25	165	83	25
13	223	111	25	180	90	25	178	89	25
14	235	118	25	193	96	25	193	96	25
15	253	126	25	205	103	25	205	103	25
16	265	133	25	218	109	25	218	109	25
17	280	140	25	230	115	25	230	115	25
18	295	148	25	245	123	25	243	121	25
19	310	155	25	258	129	25	255	128	25
20	330	165	25	270	135	25	277	138	25
21	350	175	25	283	141	25	283	141	25
22	363	181	25	295	148	25	295	148	25
23	375	188	25	308	154	25	308	154	25
24	390	195	25	320	160	25	320	160	25
25	408	204	25	333	166	25	333	166	25

A7: Natural Frequencies - Unloaded System [Hz]

L	mode	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
FRP 1	l	5.642	5.315	5.078	4.844	4.666	4.482	4.341	4.186	4.185	3.925	3.916	3.691	3.668	3.471	3.438	3.266
	v	10.464	9.491	8.653	7.977	7.462	6.953	6.493	6.104	5.795	5.485	5.193	4.943	4.739	4.530	4.325	4.153
	t	12.336	11.199	10.266	9.484	8.884	8.308	7.813	7.377	7.077	6.677	6.414	6.082	5.886	5.600	5.420	5.172
	l	15.555	14.343	13.410	12.605	12.010	11.426	10.960	10.515	10.423	9.811	9.736	9.214	9.159	8.705	8.629	8.243
	l	30.931	28.158	25.976	24.138	22.777	21.452	20.359	19.370	18.927	17.831	17.463	16.547	16.307	15.521	15.250	14.614
	v	41.975	36.349	33.539	31.018	29.043	27.092	25.317	23.813	22.608	21.404	20.273	19.298	19.298	18.501	17.686	16.891
FRP 2	t	42.265	38.201	34.782	31.935	29.726	27.585	25.709	24.068	22.806	21.431	20.323	19.197	18.386	17.437	16.706	15.906
	l	7.694	7.003	6.678	6.507	6.291	5.894	5.745	5.672	5.547	5.274	5.377	5.130	5.041	4.830	4.903	4.707
	v	16.659	15.164	13.833	12.776	11.818	11.045	10.322	9.721	9.254	8.778	8.306	7.921	7.538	7.220	6.897	6.630
	t	18.929	17.157	15.689	14.474	13.425	12.504	11.723	11.060	10.539	9.970	9.546	9.083	8.701	8.316	8.058	7.729
	l	23.358	21.118	19.559	18.474	17.435	16.277	15.547	15.046	14.553	13.820	13.757	13.132	12.802	12.279	12.314	11.847
	v	48.628	43.855	40.146	37.376	34.848	32.456	30.639	29.260	28.065	26.599	25.950	24.735	23.893	22.880	22.587	21.715
Steel	l	65.442	59.594	54.378	50.234	46.473	43.442	40.600	38.243	36.394	34.526	32.673	31.160	29.656	28.404	27.137	26.085
	t	68.288	61.862	56.410	51.888	47.940	44.560	41.573	38.995	36.976	34.856	33.008	31.279	29.732	28.294	27.072	25.855
	l	4.113	3.756	3.597	3.343	3.413	3.200	3.124	3.088	3.025	2.875	2.831	2.793	2.743	2.627	2.661	2.553
	v	8.500	7.751	7.058	6.534	6.117	5.715	5.334	5.021	4.724	4.481	4.262	4.082	3.881	3.718	3.548	3.411
	t	9.385	8.509	7.780	7.167	6.741	6.278	5.888	5.561	5.267	4.987	4.763	4.589	4.401	4.209	4.089	3.924
	l	12.059	10.948	10.183	9.417	9.221	8.621	8.259	8.019	7.753	7.367	7.270	7.052	6.885	6.608	6.642	6.392
Steel	l	24.750	22.412	20.578	18.978	18.168	16.943	16.031	15.353	14.676	13.921	13.717	13.074	12.651	12.129	12.005	11.549
	v	33.358	30.434	27.722	25.669	24.022	22.447	20.952	19.728	18.563	17.608	16.748	16.038	15.250	14.608	13.944	13.405
	t	34.679	31.418	28.621	26.317	24.598	22.835	21.286	19.946	18.746	17.662	16.745	15.958	15.168	14.422	13.803	13.174



A8: Fundamental Frequencies – Loaded System – lateral [Hz]

FRP 1	0	1	2	3	4
10	5.642	3.4662	2.7194	2.3122	2.0461
15	4.4818	3.0814	2.4955	2.1511	1.9191
20	3.9157	2.8887	2.3917	2.0876	1.875
25	3.2659	2.5221	2.1287	1.876	1.6961

FRP 2	0	1	2	3	4
10	7.6943	3.3831	2.5198	2.0965	1.8182
15	5.8937	3.0047	2.2825	1.9134	1.6796
20	5.3765	3.0351	2.348	1.9825	1.748
25	4.7072	2.8426	2.2296	1.8947	1.676

Steel	0	1	2	3	4
10	4.1133	2.8385	2.2994	1.9723	1.7576
15	3.2	2.4252	2.0311	1.7824	1.6072
20	2.8305	2.2726	1.952	1.7374	1.5809
25	2.5533	2.1162	1.8467	1.6593	1.5197

A9: Fundamental Frequencies – loaded System – vertical [Hz]

FRP 1	0	1	2	3	4
10	10.464	6.4304	5.0454	4.2901	3.7964
15	6.9531	4.7851	3.8764	3.3419	2.9818
20	5.1933	3.8395	3.1816	2.7783	2.496
25	4.1533	3.2164	2.7181	2.3971	2.1681

FRP 2	0	1	2	3	4
10	16.659	7.3264	5.4571	4.5403	3.39378
15	11.045	5.6355	4.2814	3.5893	3.1507
20	8.3064	4.7013	3.6387	3.0731	2.7099
25	6.6296	4.0213	3.1574	2.6843	2.3752

steel	0	1	2	3	4
10	8.4995	5.8667	4.7529	4.0769	3.6332
15	5.7148	4.3341	3.6309	3.1868	2.8738
20	4.262	3.4281	2.9472	2.6246	2.3891
25	3.411	2.8368	2.4801	2.2311	2.045

A10: Fundamental Frequencies – loaded System – 10.00 m [Hz]

vertical	0	1	2	3	4
FRP 1	10.464	6.4304	5.0454	4.2901	3.7964
FRP 2	16.659	7.3264	5.4571	4.5403	3.39378
Steel	8.4995	5.8667	4.7529	4.0769	3.6332

lateral	0	1	2	3	4
FRP 1	5.642	3.4662	2.7194	2.3122	2.0461
FRP 2	7.6943	3.3831	2.5198	2.0965	1.8182
Steel	4.1133	2.8385	2.2994	1.9723	1.7576

A11: Fundamental Frequencies – loaded System – 20.00 m [Hz]

vertical	0	1	2	3	4
FRP 1	5.1933	3.8395	3.1816	2.7783	2.496
FRP 2	8.3064	4.7013	3.6387	3.0731	2.7099
Steel	4.262	3.4281	2.9472	2.6246	2.3891

lateral	0	1	2	3	4
FRP 1	3.9157	2.8887	2.3917	2.0876	1.875
FRP 2	5.3765	3.0351	2.348	1.9825	1.748
Steel	2.8305	2.2726	1.952	1.7374	1.5809

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